CITY OF ASHLAND, OREGON

COMPREHENSIVE SANITARY SEWER MASTER PLAN

APRIL 2012

Adopted By Council April 17, 2012

KELLER associates

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April 2012

CITY OF
ASHLAND, OREGON

Comprehensive Sanitary Sewer Master Plan

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1.0 EXECUTIVE SUMMARY

Keller Associates, Inc. was commissioned in 2010 to complete a comprehensive master plan for the City of Ashland sanitary sewer collection system and wastewater treatment plant. This section summarizes the major findings of the master plan, including brief discussions of alternatives considered and final recommendations.

1.1 DESIGN CONDITIONS

1.1.1 Demographics

Populations in the Ashland Comprehensive Plan were utilized without alteration, per City instruction, for all study design considerations. Comprehensive Plan projections were based on an assumed steady population increase of 187 persons per year (<1.0% growth rate).

The study area was selected to match the Urban Growth Boundary (UGB) defined in the Ashland Comprehensive Plan, with its associated land use and zoning. Land use densities from the 2011 Buildable Lands Inventory (BLI) were utilized in this study for identifying growth areas and developing future flows from those areas for use in the model for analysis of collection system components.

1.1.2 Wastewater Flows

Data on daily and monthly treatment plant flows from 2004 thru 2009, and limited hourly flow data from 2008 was used to determine design flows. Design flows were calculated in accordance with Oregon DEQ guidelines, and include average and peak flows for both wet and dry weather periods as summarized in Table 1.1.

TABLE 1.1: Summary of Historical and Projected Ashland Flow Rates

	2005-2009		Existing Design		Projected	2015	2030	2060
	Avg	Max	2010		Unit Flow			
Population[1]	-	-	20,980		-	21,913	24,716	30,326
Units	MGD	MGD	MGD	GPCD	gpcd[2]	MGD	MGD	MGD
Average Day Dry-Weather[3] (ADWF)	2.06	2.15	2.1	100	100	2.19	2.47	3.04
Max Month Dry-Weather ($MMDWF_{10}$)	2.21	2.41	2.7	129	129	2.82	3.18	3.90
Annual Average Day (AADF)	2.17	2.41	2.2	105	105	2.30	2.59	3.18
Average Day Wet-Weather[4] (AWWF)	2.27	2.68	2.3	110	110	2.40	2.71	3.32
Max Month Wet-Weather ($MMWWF_5$)	2.77	3.64	3.6	172	172	3.76	4.24	5.20
Peak Week (PWkF)	3.64	5.02	5.0	238	150	5.14	5.56	6.40
Peak Day ($PDAF_5$)	5.52	8.39	7.1	338	250	7.33	8.03	9.44
Peak Instantaneous (Hour) (PIF_5)	-	10.00	10.5	500	350	10.83	11.81	13.77

[1] Populations from Comprehensive Plan
[2] gpcd = gallons per capita per day
[3] Dry-Weather = May – October
[4] Wet-Weather = November – April

Flows increase with precipitation, typically rising during the second week of December with peak flows in January before falling off in February. Winter months have more significant peak day events, and maximum monthly totals are typically 125% of average summer flows.

Analysis of hourly data revealed instantaneous flows as high as 10.0 MGD, with the largest peak events corresponding to rain events. These observations are indicative of significant infiltration and inflow within the collection system.

In addition to WWTP influent flows, flow meters were also placed at selected sites throughout the collection system to measure flows from the various sewer shed basins. These flows were utilized to calibrate the collection system model.

1.2 COLLECTION SYSTEM EVALUATION & RECOMMENDATIONS

1.2.1 Lift Station Evaluation

Keller Associates visited each of the 8 lift stations and completed a general inventory of facilities (including pump curves and data sheets where available), and conducted pump tests at select stations.

Each lift station has a unique set of deficiencies in accordance with its inventoried condition. Those requiring repair and targeted within the Capital Improvements Plan (CIP) include:

Priority 1 (2012-2020)

- Replace Grandview Lift Station (already underway)
- Equip Creek Drive Lift Station with chopper pumps and three phase power
- Abandon Nevada Lift Station – new Oak Street gravity pipeline (under design)
- Add valve vault drain at Windburn Lift Station
- Maintenance Management Software and programming upgrades
- Add SCADA to all lift stations

Priority 2 (2021-2030)

- Grandview Lift Station force main upgrade
- Convert Shamrock Lift Station to submersible pumps
- Upgrade North Mountain Lift Station to design standards

Other general recommendations not listed in the CIP include:

- Creation of Lift Station Design Standards – modeled after North Main Lift Station (ROMTEC) and including the following additional recommendations:
  - Wet well liner
  - Polyurethane sealant for all wet well joints
  - Flow meter(s)
  - Standardized controllers
  - Valve vault and drain
  - Flexible restrained couplings
  - Influent shutoff valve
- Upgrade SCADA at all stations to include:
  - Continuous level monitoring and trending
  - Continuous monitoring and trending of pump on/off status
  - Monthly reports of daily totalized flows and daily pump run times
  - Alarm when all pumps at a particular lift station are called on

1.2.2 Pipeline Condition and Capacity Evaluation

Keller Associates utilized the City's GIS record to conduct an inventory of pipe size and material for the City's 110 miles of gravity sewer. This inventory revealed approximately half of the collection system is made up of pipelines smaller than the current minimum pipe diameter standard of 8 inches. Clay and concrete pipes (generally the oldest and most susceptible to disrepair) constitute approximately 17% and 50%, respectively, of the total system. Pipes smaller than 8 inches and all clay and concrete pipes eventually should be replaced.

During an evaluation of the City's inspection process, about 16 hours of video and accompanying TV monitoring logs were reviewed by Keller Associates. The review also provided a glimpse into system conditions. Typical problems identified include cracks, roots, pipe sags, offset joints, and broken pipe, with over 400 pipeline segments currently identified for either spot repairs or pipeline replacement/rehabilitation. In addition, infiltration and inflow is encountered in many of the City's manholes; rehabilitation of these manholes is recommended. Replacement and/or rehabilitation of other manholes should be evaluated in connection with adjacent pipeline rehabilitation/replacement projects.

A GIS-based computer model (InfoSewer 7.0) of the collection system was built and exercised to evaluate capacities of the system's trunklines (generally 10-inches or larger). The modeling results were used to prioritize improvements recommended in the CIP. Generally, pipelines sufficiently sized for existing flows are also sufficient for City infill, with a few upgrades for system expansion into the UGB.

1.2.3 System Maintenance Evaluation

The City of Ashland has an active collection system maintenance program that includes schedules for jet rod cleaning, TV inspection, smoke testing, root foaming, sewer pipe repairs/replacement, and manhole repair/replacement. In the past three years, the City has exceeded their annual goals for jet rod cleaning, CCTV, and root foaming, with about 58% of the annual maintenance budget used on these three activities. Though the City exceeds industry standards, additional efficiencies may be achieved by implementing the following:

- More closely group monthly activities by geographic location
- Increase annual replacement / repair budget (target 7,800 ft/yr @ $100/ft = $780k/yr)
- Keep digital copies of CCTV inspections and photos
- Revise TV log ratings and pipeline ranking system

Adjustments to prioritization based on the judgment of an experienced operator should periodically be made, to account for limitations of any maintenance management system and considerations of overall risk.

1.2.4 Recommended Collection System Improvements

Recommended collection system capital improvements are summarized in the capital improvement plan (CIP) cost table at the end of this chapter and illustrated in Figure 8.1 in Appendix A. Notable major improvements are summarized below. The majority of Priority 1 & 2 improvements are replacements of pipe sections to correct size or slope issues identified with the model during the capacity analysis.

Priority 1A involves an extension of the Bear Creek Parallel Trunklines, including installation of an 18-inch and 24-inch pipeline parallel to the existing 12-inch pipeline, and installation of several 15-inch or 24-inch sections along the existing parallel trunks to create tiered parallel trunklines (12-inch/18-inch, 12-inch/24-inch, 15-inch/24-inch) from the I-5/Bear Creek intersection to the Ashland Creek Lift Station at the plant. This upgrade will provide sufficient capacity and redundancy for growth well into the future.

Priority 2A consists of installing a new 12-inch pipeline on West Nevada Street to intercept flows from the northwest and convey wastewater by gravity to the headworks of the WWTP, rather than being pumped by the Ashland Creek Lift Station.

Priority 3A involves construction of a new lift station and pressure main along the Rogue Valley Highway 99 to the northwest of town. The station would collect flows from new development along the highway and I-5 corridor as well as from extended service to existing development along Wrights Creek (west edge of City Limits). The existing North Main lift station could be abandoned.

1.3 EFFLUENT DISPOSAL

1.3.1 Effluent Disposal Options

Since the feasible alternatives for wastewater treatment depend on the effluent disposal method and associated effluent requirements, effluent disposal alternatives were evaluated before considering treatment options. Eight disposal alternatives were considered, including effluent recycling (maximum or partial recycling on Imperatrice property, or city-wide recycling); relocating the discharge point to Talent Irrigation District system; or continuation of the current practice of discharging to Ashland Creek.

Considerations in the development and evaluation of disposal options included:

- Land available for effluent recycling
- Phosphorus discharge limits in Ashland Creek
- Maintaining sufficient stream flow for fish in Ashland Creek
- Water rights issues
- Public and/or agency concerns
- Anticipated excess thermal load limits in Ashland and Bear Creek

Based on a review of the previous five years of temperature and flow data, there is an existing excess thermal load with the potential to exceed allowable levels during the May through October period. Therefore, the continued discharge options included various technologies to reduce temperatures and thermal loads both before and after discharge. The continued discharge alternatives evaluated included use of a cooling tower/chiller, shading/trading, blending, and a hyporheic (shallow ground water mixing) option to meet anticipated limits.

1.3.2 Effluent Disposal Recommendation

Though effluent requirements for recycling are less stringent than discharge to surface waters, a 100 percent effluent recycling program for Ashland has two major obstacles: 1) some of the water is needed to sustain flow for fish in Ashland Creek, and 2) the existing City-owned property will not be large enough for 100 percent land application in the future. Partial effluent recycling to limit discharge to periods with less restrictive discharge limits (primarily wet-weather, high-flow periods) would minimize the need for additional treatment, but would also require cost-prohibitive storage volumes. Therefore, the most feasible effluent disposal method is continued discharge, with shading/trading recommended to deal with thermal loads. Effluent recycling can be pursued as needed to address future potable water supply needs.

1.4 WASTEWATER TREATMENT

1.4.1 Existing Facilities

The Ashland WWTP consists of screening and grit removal, biological treatment in an oxidation ditch system with secondary clarification, UV disinfection, and post aeration. Alum addition and a tertiary membrane system are operated from May 1st to November 30th, to aid in meeting a seasonal phosphorus limit. Waste solids from the biological process are dewatered and hauled to the landfill for disposal. (Equipment for lime stabilization of the waste solids is currently not used.)

The Ashland WWTP currently operates and discharges to Ashland Creek under an NPDES permit. A new permit, expected to be completed in 2014, is anticipated to contain more stringent limits connected to Total Maximum Daily Loads (TMDLs) developed for the Bear Creek watershed. The 2007 TMDL addresses temperature, bacteria, and sedimentation issues, which may require a higher degree of treatment to maintain or improve effluent quality as future growth occurs. Additionally, new limits for toxins will trigger additional monitoring and may have impacts on the nature and timing of capital improvements in the future.

Reported effluent characteristics from January 2004 to December 2010 were analyzed to evaluate plant compliance with existing permit limits, and to evaluate the expected capability of the plant to continue meeting the permit limits with increased flows. Hydraulic capacity, treatment capacity based on typical operating criteria, and physical condition of the treatment plant components were also evaluated.

For CBOD, TSS, ammonia, phosphorus, and E. coli, the existing treatment plant technology should be able to meet the current limits in the future as long as treatment units are operating within the existing design criteria. As flow increases, additional components may be needed to maintain the design criteria and continue meeting the effluent limits, based on hydraulic and treatment capacity.

1.4.2 Recommended Improvements

The treatment process components that will need to be upgraded or replaced are:

Priority 1 (2012-2020)

1A – required for permit/Agency compliance

- Effluent temperature upgrade
- Fish screen for outfall (relocated outfall)
- Add UVT monitor

1B – recommended to address capacity and equipment condition issues

- Provide a 6-inch trash pump as a backup for the influent lift pumps
- Replace membranes at end of useful life
- Additional biological capacity (see treatment alternatives below)
- Option to meet DO limit (unless the limit is revised with the new permit and new outfall location).
- Replace RAS pumps with larger pumps

Priority 2 (2021-2030)

- Replace membranes at end of useful life (2023 upgrade will increase capacity, requiring piping, blower, membrane pump, and chemical treatment equipment upgrades)
- Add UV reactors to increase hydraulic capacity, and upgrade existing panels to allow flow pacing (to save energy).
- Upgrade or replace grit removal system
- Replace mechanical bar screen, clarifier mechanism in clarifier #2, and equipment in existing oxidation ditches

Priority 3 (2030-2060)

- Replace influent lift station pumps
- Replace membranes at end of useful life
- Increase biological treatment capacity
- Increase solids dewatering capacity
- Replace clarifier mechanisms in clarifiers 1 and 3

1.4.3 Treatment Alternatives

Treatment alternatives considered for continued effluent discharge included:

- No Action alternative
- Reduction of peak flows through
  - rehabilitation of collection system to minimize inflow and infiltration (I/I)
  - addition of flow equalization
- Expansion of oxidation ditch plant (third oxidation ditch or fourth clarifier)
- Parallel membrane plant to treat flows in excess of existing oxidation ditch capacity
- Enhanced biological treatment by modification of process in existing oxidation ditch
  - Staged aeration
  - Integrated Fixed Film/Activated Sludge (IFAS)
  - In-ditch membrane plant
- Adding a primary filter to reduce loading to the oxidation ditch

After consulting with the technical review committee, the following three options were evaluated in more detail with cost estimates and environmental impacts considered:

1. Expansion of oxidation ditch plant by constructing an **additional oxidation ditch** (which could be staged by initially using the shell as equalization storage);

2. Converting to enhanced biological treatment in the existing oxidation ditches, through **staged aeration** or **IFAS**

3. Adding a fine mesh sieve (**primary filter**) to reduce loading and thus increase treatment capacity.

All options are similar in that they provide capacity to 2030 and beyond. The recommended alternative is building a new ditch for initial use as an equalization basin. By 2030, the equalization basin will need to be equipped to function as a third oxidation ditch. A fourth secondary clarifier would be required by the year 2060.

Proceeding with the recommended option of constructing the outer shell of a third oxidation ditch is dependent on being able to obtain adjacent lands from the Parks Commission. If this is not feasible, the next best option is staged aeration.

1.4.4 Biosolids Handling Alternatives

The estimated amount of sludge produced is expected to increase 28% by the year 2030. The City of Ashland must have a reliable means of disposal for its sludge, since it is produced on a continuous basis and there is limited existing storage on-site.

Currently the City of Ashland disposes of their unstabilized dewatered sludge in the Dry Creek Landfill, and has adequate sludge storage and treatment facilities to manage their sludge through 2030. If this option should become unavailable or if it is desired to beneficially reuse the biosolids for fertilizer, the City would be required to stabilize their sludge before applying it to agricultural land or providing it to the public as fertilizer.

The evaluation of sludge handling alternatives involved a review of available technologies for thickening, sludge stabilization and dewatering. After consulting with the technical review committee (TRC), the following three options were evaluated in more detail with cost estimates and environmental impacts considered:

1. Dewater sludge using the **existing centrifuges, and haul to the landfill** for disposal.
2. Dewater sludge using the **existing centrifuges, and compost** to produce Class A biosolids for sale to commercial businesses and individuals.
3. Dewater sludge using the **existing centrifuges, and dry using a thermal dryer** to produce Class A biosolids for sale to commercial businesses and individuals.

Continuing to landfill is the least expensive alternative. However, it is recommended that the City consider a backup plan. Both compost and dried biosolids can be sold to generate revenue to offset the cost of sludge treatment. Thermal drying is less expensive than composting, and facilities could be located at the existing wastewater treatment plant.

1.4.5 WWTP Improvement Recommendations

Recommended capital improvements necessary to resolve existing and future deficiencies at the treatment plant are summarized in the CIP cost table and illustrated in Figure 12.1 in Appendix A.

Further, improvements were recommended and prioritized through consideration of several treatment planning objectives as outlined below.

- Eliminate NPDES Permit Violations:
  - Dissolved Oxygen - re-evaluate limit and seasons with DEQ
  - Excess Thermal Load – shading (recommended alternative) will be best accomplished by entering into an agreement with an implementation organization, and by relocating existing outfall to Bear Creek to address local plume concerns; local wetland improvements would also be beneficial

- Prevent Plant Deficiencies
  - Eliminate Bottlenecks – pipe from the oxidation ditch to clarifiers reaches capacity around 2030 flows
  - Manage Peak Flows – utilize shell of 3rd oxidation ditch as equalization basin until 2030; I/I reduction in collection system
- Stay Ahead of Growth and Maintain Equipment
  - Ashland Creek LS – provide portable backup pump on-site; replace pumps when pumped flows exceed 8.0 MGD (approx. 12.8 MGD total influent)
  - Screens – reach capacity and life expectancy in 2030
  - Grit removal system – sufficient capacity to 2030, estimated life expectancy near 2025
  - Oxidation Ditch – aerators reach useful life near 2030, new shell/equalization basin will need to be equipped as oxidation ditch #3 in 2030
  - Secondary Clarifiers #1 & #2 – mechanical life expectancy will be reached in 2030 and 2020, respectively; sufficient capacity is provided to 2050 with construction of a 3rd oxidation ditch
  - RAS pumps – replace when peak flows commonly exceed 6.5 MGD
  - UV disinfection – treatment capacity sufficient to 2030, while hydraulic capacity will be reached near 2020
  - Membrane filtration – replacement schedule provided to meet expected capacity increases and revolving life expectancies; based on that schedule, the membrane feed pumps will need to be upsized in 2023.
  - Alum feed – pump capacity will be reached near 2025
- Improve Solids Handling
  - City desires to produce Class A solids at some point
- Improve SCADA system

Relocating the outfall to Bear Creek, the fish screen, and third oxidation ditch involve construction within or near several Water Resource Protection Zones/Riparian Corridors, Locally Significant Wetlands, and Possible Wetlands. These projects will require environmental evaluations and coordination with Oregon Department of Fish & Wildlife (OWDR), Oregon State Department of Lands, and City Planning & Zoning.

1.5 CAPITAL IMPROVEMENTS PLAN & FINANCING

1.5.1 Summary of Costs

Table ES.2 presents a summary of future costs in order of priority. The basis for the need for each improvement varies, including compliance with the City's discharge permit and anticipated new regulations; achieving capacity necessary to accommodate growth; and replacing worn/old equipment.

Priority 1 improvements target existing deficiencies, and are intended to be completed within the next 5-10 years. Priority 2 improvements correct lower risk deficiencies and/or address impacts due to growth, and are expected to be required from 2020 to 2030. Priority 3 improvements are driven by growth. Flexibility in the schedule for completing many of these improvements is warranted. For example, the City should consider accelerating pipeline projects if they can be coordinated with roadway improvements. Similarly, changes in flows and efforts to reduce infiltration and inflow may allow for some improvements to be postponed.

TABLE 1.2: City of Ashland Wastewater Improvements Opinion of Probable Cost

ID#	Item	Primary Purpose	Total Estimated Cost	Growth Apportionment		City's Estimated Portion
				%	Cost	
Priority 1 Improvements (2012 - 2020)						
Wastewater Treatment						
1	Outfall Relocation / Fish Screen	Compliance	$ 856,000	15%	$ 128,400	$ 727,600
2	Shading (Capital Cost + first 6 years of O&M)	Compliance	$ 1,646,000	15%	$ 246,900	$ 1,399,100
3	UVT Monitor	Compliance	Completed	0%	$ -	$ -
4	Backup (Portable) Pump	Capacity	$ 60,000	0%	$ -	$ 60,000
5	Membrane Replacement (two trains)	Replacement	$ 1,248,000	0%	$ -	$ 1,248,000
6	Oxidation Ditch Shell	Capacity	$ 4,000,000	39%	$ 1,560,000	$ 2,440,000
7	RAS Pump Replacement	Capacity	$ 90,000	20%	$ 18,000	$ 72,000
8	Wastewater Master Plan Update	Update	$ 125,000	100%	$ 125,000	$ -
9	Wastewater Facility Plan	Financing	$ 35,000	50%	$ 17,500	$ 17,500
Wastewater Collection System						
1A	18" and 24" Parallel Trunkline Along Creek	Capacity	$ 1,248,000	70%	$ 873,600	$ 374,400
1B	15" Main Along Mountain Ave	Capacity	$ 118,000	25%	$ 29,500	$ 88,500
1C	Oak St. 24" Trunkline	Capacity	$ 40,000	15%	$ 6,000	$ 34,000
1D	A St 15" Main	Capacity	$ 522,000	10%	$ 52,200	$ 469,800
1E	12" Main Along Railroad	Capacity	$ 275,000	57%	$ 156,750	$ 118,250
1F	12" Siskiyou Blvd Main	Capacity	$ 73,000	46%	$ 33,580	$ 39,420
1G	Miscellaneous Upgrades	Various	$ 335,000	10%	$ 33,500	$ 301,500
1H	Portable Flow Meters	Operations	$ 60,000	0%	$ -	$ 60,000
1J	Storm Water Inflow Study (2012 - 2013)	Capacity	$ 60,000	0%	$ -	$ 60,000
	Total Priority 1 Improvements		**$ 10,791,000**		**$ 3,280,930**	**$ 7,510,070**
Priority 2 Improvements (by 2020 - 2030)						
Wastewater Treatment						
1	Membrane Replacement (Larger Membranes)	Capacity/ Replacement	$ 4,659,000	40%	$ 1,863,600	$ 2,795,400
2	Membrane Feed Pumps & Piping Replacement	Capacity	$ 507,000	80%	$ 405,600	$ 101,400
3	Additional UV Reactors & Upgrade Control Panels	Capacity	$ 351,000	100%	$ 351,000	$ -
4	Mechanical Bar Screen Replacement	Replacement	$ 496,000	20%	$ 99,200	$ 396,800
5	Grit Removal System Replacement	Replacement	$ 801,000	20%	$ 160,200	$ 640,800
6	Oxidation Ditch Internals	Capacity	$ 2,150,000	100%	$ 2,150,000	$ -
7	Existing Oxidation Ditch Equipment Replacement	Replacement	$ 1,551,000	0%	$ -	$ 1,551,000
8	Clarifier Mechanism Replacement	Replacement	$ 324,000	0%	$ -	$ 324,000
9	Replace Ashland Creek Lift Station Pumps with Larger Pumps	Capacity	$ 353,000	80%	$ 282,400	$ 70,600
8	Wastewater Master Plan Update	Update	$ 125,000	100%	$ 125,000	$ -
9	Biosolids Disposal (assumes thermal dryer)	Various	$ 4,100,000	20%	$ 820,000	$ 3,280,000
Wastewater Collection System						
2A	12" Pipeline on Nevada Street	Capacity	$ 217,000	38%	$ 82,460	$ 134,540
2B	8" Slope Correction on Walker Ave.	Operations	$ 168,000	28%	$ 47,040	$ 120,960
2C	12" Pipeline on Wightman St.	Capacity	$ 172,000	66%	$ 113,520	$ 58,480
2D	Miscellaneous Upgrades	Various	$ 739,000	10%	$ 73,900	$ 665,100
	Total Priority 2 Improvements		**$ 16,713,000**		**$ 6,573,920**	**$ 10,139,080**

TABLE 1.2: City of Ashland Wastewater Improvements
Opinion of Probable Cost *(Continued)*

ID#	Item	Primary Purpose	Total Estimated Cost	Growth Apportionment		City's Estimated Portion
				%	Cost	
Future Improvements (beyond 2030) or Development Related Improvements						
Wastewater Treatment						
1	Additional Centrifuge	Capacity	$ 817,000	100%	$ 817,000	$ -
2	Clarifier Mechanism Replacement (2)	Replacement	$ 646,000	0%	$ -	$ 646,000
3	Additional Clarifier	Capacity	$ 1,773,000	100%	$ 1,773,000	$ -
Wastewater Collection System						
3A	Rogue Valley Hwy 99 Collection, Lift Station, & Pressure Main (assumes City provides service)	Growth	$ 2,545,000	100%	$ 2,545,000	$ -
3B	Upsize Costs for Future Expansion	Growth	$ 18,000	100%	$ 18,000	$ -
	Total Priority 3 Improvements		**$ 5,799,000**		**$ 5,153,000**	**$ 646,000**
TOTAL WASTEWATER IMPROVEMENTS COSTS (rounded)			**$ 33,303,000**		**$ 15,007,850**	**$ 18,295,150**

1.5.2 Other Annual Costs

In addition to the capital improvement costs presented in the previous section, Keller Associates recommends the following be accounted for in setting annual budgets:

- Additional staffing needs: additional $195,000/year for 2.5 additional full time equivalent employees (collection system supervisor, treatment plant operator, and 0.5 FTE for regulatory compliance).
- Additional collection system replacement / rehabilitation needs: City should eventually budget an additional $637,000/year (either to be contracted out or completed using City crews). To minimize rate impacts, this program may not fully be funded until after 2022 when the existing wastewater loans are retired.
- Additional annual operations and maintenance costs will be required to maintain the shading improvements: anticipated to cost approximately $55,000/year for years 6-10, and closer to $39,000/year for years 11-20.
- Other additional annual operation and maintenance costs are associated with Priority 1 improvements (relocation of the outfall, larger RAS pumps, backup lift station pump, and equalization basin): the additional operations and maintenance costs for these improvements are anticipated at close to $26,000/year, most of which is associated with increased power usage of the RAS pumps.
- Short-lived assets (pumps, equipment, etc.): equates to an average of approximately $93,500/year, of which approximately $29,700/year is attributed to future facilities that will be added over the 20-year planning period.

1.5.3 Financing / Rates

A summary of the sewer financial plan can be found in Chapter 14 of this report. The financial plan considers the total annual cost of owning and operating the sewer system and recommends three new loans to pay for construction of most of the Priority 1 capital improvements. To pay for increasing costs of operation and to repay the existing and three new loans, the plan recommends increasing sewer rates 10 percent per year for the next six years. The base sewer rate paid by most single family households currently is $18.70 per month and will increase over the next 6 years to about $33.00 per month.

2.0 REGULATORY REQUIREMENTS

Regulatory requirements, existing constraints, and water quality impacts directly affect the basis of design for new improvements. These issues are discussed in this section.

2.1 COLLECTION SYSTEM REGULATIONS

2.1.1 Pump Station Design Regulatory Requirements

Pump stations are generally used to lift wastewater from a lower elevation and convey it to a high location where it is discharged. Pump stations must meet requirements of DEQ. Typical guidelines governing pump station design include:

- Redundant pumping capacity – DEQ design criteria requires that the pump stations be capable of conveying the 5-year 24-hour storm peak hourly flow with the largest pump out of service.
- Provisions for Hydrogen Sulfide removal, if required. Hydrogen Sulfide can be corrosive (especially to concrete materials) and often lead to odor problems. Where septic conditions are believed to occur, provisions for addressing hydrogen sulfide should be in place.
- Alarms – alarm system should include high level overflow, power, and pump fail conditions. DEQ design criteria require that an alarm condition results when all pumps are called on (loss of redundancy alarm) to keep up with the inflow into the pump station. This is an indicator that the pump station capacity is exceeded.
- Standby power. Since extended power outages may lead to wastewater backing up into homes and onto the streets, provisions for standby power are required for every pump station. Mobile generators or portable trash pumps may be acceptable for lift stations, depending on the risk of overflow, available storage in the wet well and pipelines, alarms and response time.
- DEQ has established a set of design guidelines for gravity collection system and pump stations (refer to http://www.deq.state.or.us/wq/rules/div052guides.htm).

2.1.2 Pipeline Regulatory Rules

cMOM Rules

cMOM refers to Capacity Management, Operation, and Maintenance of the entire wastewater conveyance system.

The vast majority of all sanitary sewer overflows originate from three sources in the collection system – infiltration and inflow (I/I), roots, and fats, oil and grease (FOG). Infiltration and inflow problems are best addressed through a program of regular flow monitoring, TV monitoring and pipeline rehabilitation and replacement. Blockages from roots or FOG are also addressed via a routine cleaning and monitoring program. A FOG control program may also involve public education, and city regulations (i.e. requirements for installation and regular maintenance of grease interceptors). All new facilities believed to contribute FOGs should be equipped with grease interceptors.

All SSOs are prohibited by EPA. The Oregon Sanitary Sewer Overflow (SSO) rules include both wet weather and dry weather design criteria. DEQ has indicated that they have enforcement discretion and that fines will not occur for overflow that result from storm events that exceed the Oregon DEQ design criteria (i.e. greater than winter 5-year storm event and a summer 10-year storm event).

In December 2009, DEQ developed a SSO Enforcement Internal Management Directive [1] that provides guidance for preventing, reporting, and responding to SSOs. This document was later updated in November 2010. Municipalities are encouraged to adopt programs that reduce the likelihood of overflow events. Reporting requirements include notice within 24 hours and written reports within 5 days. The City can expect that their new discharge permit will also include requirements for an Emergency Notification and Response Plan. This plan will replace the existing Contingency Plan for the Prevention and Handling of Sewer Spills and Unplanned Discharges. Appendix D of the directive outlines six elements to be included in the plans. These are summarized below.

1. Ensure that the permitted is aware of such events.
2. Ensure notification of appropriate personnel and ensure that they are immediately dispatched for investigation and response.
3. Ensure immediate notification to the public, health agencies, and other affected public entities.
4. Ensure that appropriate personnel are aware of and follow the plan and are appropriately trained.
5. Provide emergency operations.
6. Ensure that DEQ is notified of the public notification steps taken.

Excessive Infiltration and Inflow

EPA defines excessive infiltration and inflow (I/I) as the quantity of I/I that can be economically eliminated from a sewer system by rehabilitation. Some guidelines for determining excessive infiltration and inflow were developed in 1985 by EPA based on a survey of 270 standard metropolitan statistical area cities [2]. Non-excessive numeric criteria for infiltration was defined as average daily dry weather flows that are below 120 gallons per capita per day (gpcd). Similarly, a guideline of 275 gpcd was established as an indicator below which is considered non-excessive storm water inflow.

Keller Associates experience is that it is often difficult to determine if a particular rehabilitation project or program is cost-effective. Sometimes rehabilitation efforts in one area may increase groundwater levels and create new sources of infiltration. The proper balance of ongoing I/I reduction efforts may need to be customized for each entity.

Pipeline Surcharging

Pipeline surcharging occurs as flows exceed the capacity of a full pipe, causing wastewater to backup into manholes and services. Surcharging of gravity pipelines is generally discouraged because of 1) the increased potential for backing up into people's homes; and 2) the increased potential of exfiltration (escape of raw wastewater into the groundwater); and 3) health risks associated with Sanitary Sewer Overflows (SSOs).

Illicit Cross Connections

Any illicit cross connections from the City's storm water system should be removed.

2.2 TREATMENT PLANT REGULATIONS

2.2.1 NPDES Permit Requirements

The National Pollutant Discharge Elimination System (NPDES) permit limits are important as the plant must be capable of meeting existing permit limits, as well as anticipated future limits. The City's current permit [3] has expired, but remains in effect until a new permit is issued. Monthly permit limits are summarized in the following table (the complete permit is attached in Appendix B). Additional limits not shown in the table include E. coli (126/100 mL), pH (6.5-8.5), and $CBOD_5$ and TSS removal efficiency (minimum 85%).

Note that mass load limits (ppd) are the controlling factor; i.e. at plant design flow, the mass load limits may require a lower concentration than specified in the permit. For example, a load of 120 ppd $CBOD_5$ at a flow of 2.3 mgd represents a concentration of 6.25 mg/L (vs. the 10 mg/L limit in the permit). Similarly, mass loads of 96 and 400 ppd at 2.3 mgd represent concentrations of 5 mg/L and 21 mg/L, respectively.

TABLE 2.1: Summary of Existing NPDES Effluent Limits

Period	Avg. Monthly Limits: mg/L / ppd				DO, mg/L	Excess Thermal Load, mil kcal/day
	$CBOD_5$	TSS	NH3	P		
Jan thru April	25 / 400	30 / 400	0.80 / -	-		
May thru August	10 / 120	10 / 96	0.52 / -	- / 1.6		
Sept thru October	4 / 77	10 / 96	0.52 / -	- / 1.6		
November	10 / 120	10 / 96	0.52 / -	- / 1.6		
December	25 / 400	30 / 400	0.80 / -	-		
Oct. 15 thru May 15					≥9.0	≤78
May 16 thru Oct. 14					-	≤38

A new permit is anticipated to be completed by 2014. New permit limits may impact future plant operation and facility improvements. Since Total Maximum Daily Loads (TMDLs) have been developed for the Bear Creek watershed (see following section), limits in future permits are expected to be no less (and possibly more) stringent than the current permit. Thus, a higher degree of treatment may be necessary to maintain and even improve effluent quality as future growth occurs.

2.2.2 TMDL Requirements

In 1992, DEQ developed a TMDL for Bear Creek that established water concentration targets for total phosphorus, ammonia nitrogen, and biochemical oxygen demand. The current NPDES permit for Ashland, issued in 2004, reflects the waste load allocations of the 1992 Bear Creek TMDL.

A second TMDL for Bear Creek [4] finalized in 2007, addresses temperature, bacteria, and sedimentation issues. Thermal load discharge, which can raise the temperature of the creek (and adversely affect aquatic life by impacting spawning and/or migration) is the main concern for point sources such as the Ashland wastewater treatment plant. The 2007 Bear Creek TMDL targets as a maximum of 13°C for October 15 to May 15 (spawning season), and 18°C for May 16 to October 14 (rearing and migration). Cumulative anthropogenic impacts are allowed to exceed these criteria by at most 0.3°C (termed the Human Use Allowance, HUA), with specific sources on the creek receiving portions of that total thermal load allocation.

Temperature Loads per TMDL

The Ashland wastewater treatment plant (WWTP) is permitted a maximum HUA of 0.1°C above the biological based numerical criteria. This condition must be met during flow event greater than the seven-day rolling average that has the probability of occurring once every 10 years (7Q10). Currently, the Ashland WWTP exceeds this allocation during the months of May through October.

TMDLs are established on a watershed basis. When meeting target TMDLs, excess thermal loads can be mitigated with thermal offsets above the point of maximum impact (for Bear Creek this is four miles upstream of the confluence of Rogue River). Watershed requirements are referred to as “far field”. In addition to meeting far field impacts within the watershed, DEQ has developed guidelines for addressing local, or “near field” impacts. High temperature discharges can create migration barriers, impact spawning areas, create thermal shock conditions, and in some cases, can be lethal to fish. DEQ has evaluated the near field impacts and determined that thermal loads from the existing discharge presents concerns for spawning, thermal shock, and migration blockage [5].

Relocating the Ashland WWTP outfall has little impact on the total excess thermal loads (far field) impacts. However, there may be near field benefits to removing the discharge point from Ashland Creek. DEQ has also evaluated the near field impacts of discharging directly to Bear Creek below the confluence with Ashland Creek [5]. This analysis showed that the increased stream flows at this point would significantly reduce the near field impacts, eliminating concerns of thermal shock and spawning, and significantly reducing the potential of migration blockage.

2.2.3 Anticipated Additional Future Permit Requirements

Ammonia [6]

In December 2009, EPA announced a draft national recommended water quality criterion for ammonia for the protection of aquatic life entitled “*Draft 2009 Update Aquatic Life Ambient Water Quality Criteria for Ammonia – Freshwater*”. This is an update required by the Clean Water Act of the 1999 ammonia criteria. EPA accepted comments to the draft through April 1, 2010. EPA has not taken any further action on the water quality criteria for ammonia in freshwater discharges, but it is likely that new criteria will be developed using the draft criteria and comments received.

Existing criteria for ammonia developed in the 1999 Ammonia Criteria are (at pH 8 and 25^{o}C):

Acute	5.6 mg NH_4-N/L if salmon are present.
Chronic	1.2 mg NH_4-N/L if fish in early life stages are present.

If the 2009 ammonia criteria in the draft report are accepted as published in 2009, then the criteria (at pH 8 and 25°C) will change to:

Acute	2.9 mg NH_4-N/L if freshwater mussels are present. 5.0 mg NH_4-N/L if freshwater mussels are absent.
Chronic	0.26 mg NH_4-N/L if freshwater mussels are present. 1.8 mg NH_4-N/L if freshwater mussels are absent.

Note that the criteria will vary with pH and temperature. For example, at pH 8 with mussels present, the ammonia criterion varies from 0.186 mg/L at 30°C to 0.817 at 0°C. The ammonia criterion increases with decreasing temperature and decreases with increasing pH. If EPA adopts the new criteria, DEQ will need to determine whether fresh water mussels are present in Ashland Creek and Bear Creek in order to determine which limit they will have to meet. Based on conversations with DEQ, mussels are likely to be found. Since 2004, the effluent ammonia has ranged from 0.01 to 1.90 mg/L as NH_4-N, with a mean effluent concentration of 0.24 mg/L as NH_4-N. The City will have to monitor pH and temperature at the time the ammonia samples are collected to determine the effluent criteria.

Priority Persistent Pollutants – Senate Bill 737 [7]

The 2007 Oregon Legislature passed Senate Bill 737, which requires DEQ to consult with all interested parties to develop a list of priority persistent bioaccumulative toxics (Priority Persistent Pollutant List) that have a documented effect on human health, wildlife and aquatic life. In order to develop the Priority Persistent Pollutant List, DEQ assembled a technical workgroup, representing expertise in various scientific sectors, to provide advice and comment.

In June 2010, DEQ again provided a report to the Legislature. The report identified potential local, regional, and global sources of persistent priority pollutants (PPP) that may contribute to water pollution in Oregon. It also outlined measures that state agencies, local governments, businesses, manufacturers and individuals could implement to reduce the presence of these pollutants in Oregon waters.

Senate Bill 737 requires Oregon's 52 largest municipal wastewater treatment plants to prepare reduction plans for persistent pollutants in their wastewater that exceed drinking water Maximum Contaminant Levels. For priority persistent pollutants for which a Maximum Contaminant Level has not been established, Senate Bill 737 authorizes the Environmental Quality Commission to determine by rule which pollutants must be addressed in persistent pollutant reduction plans. In 2010 DEQ established the levels of persistent pollutants in municipal permittees' wastewater which, if exceeded, will initiate the requirement for the permittee to prepare a persistent pollutant reduction plan. These levels are called the Plan Initiation Level (PIL).

Aquatic Life and Human Health Criteria

The City has conducted one round of monitoring of the Ashland WWTP effluent for PPP. The only constituent that exceeded the PIL in the Ashland effluent is cholesterol with an effluent concentration of 189 ng/L (nanograms per liter, or parts per trillion) and the PIL is 60 ng/L. Coprostanol was measured at 36 ng/L just under the PIL of 40 ng/L. All other constituents were either nondectable or well under the PIL. In October 2011, DEQ published Human Health Water Quality Criteria for Toxic Pollutants. Based on the limited data available for Ashland, there may be some toxins of concern (e.g. copper and phthalates). In 2012, the

City will begin completing additional testing to determine which constituents may be of concern. The potential impacts on Ashland's future permit are yet to be determined. Some of these toxins currently have no known treatment technologies and others will best be addressed by treating the water supply or regulating what is disposed of in the wastewater collection system.

Temperature Criteria

The existing temperature criteria used by the Oregon DEQ is currently being challenged. If the criteria are lowered, than additional treatment measures may be required in the future to further remove excess thermal loads.

It should also be noted that the Oregon DEQ allows for site specific criteria to be developed for waterways. It is possible that with additional input from fish biologists, that the criteria could also allow for higher thermal loads in the future.

2.2.4 Plant Reliability Criteria

The plant should have sufficient redundancy to continue operating when primary equipment units are in need of repair, when maintenance is required, and under emergency conditions. A number of concerns have been identified within the existing plant operating system in meeting the above criteria. These concerns are addressed in later sections of this report.

2.2.5 Oregon's Regulations for Biosolids Management

Waste Activated Sludge (WAS) is the term used for biomass removed from wastewater during treatment. Once WAS is separated from the wastewater treatment process and stabilized, it is termed a biosolid. Biosolids can be used for beneficial purposes such as domestic and commercial fertilizers. To ensure safe use of the nutrient-rich biosolids, regulations have been developed regarding the generation, handling, and ultimate disposal of biosolids.

State Regulations

While EPA has not officially delegated enforcement of Federal biosolids regulations to the State of Oregon, the Oregon DEQ administers the biosolids management program through their Water Quality Program. The State of Oregon first adopted regulations regarding land disposal of biosolids in 1983. In 1995, the rules were revised to comply with the new Federal biosolids regulations (i.e. 40 CFR Part 503) and can be found in *Oregon Administrative Rules (OAR) Chapter 340, Division 50 – Land Application of Domestic Wastewater Treatment Facility Biosolids, Biosolids Derived Products, and Domestic Septage*. OAR Chapter 340, Division 50 includes regulations for land application criteria, monitoring and reporting, and best management practices specific to the State of Oregon.

Biosolids are regulated by the Oregon Department of Environmental Quality [8] as part of their Water Quality Program. A treatment plant's NPDES permit is used to describe specific sludge handling practices which are approved for each individual facility. Each facility must have a current sludge management plan and site authorization letters which detail how sludge is stabilized and ultimately disposed on a specific land application site. These documents also include monitoring and reporting requirements. The permit, sludge management plan, and the site authorization letters can be used in enforcement actions by the Agency.

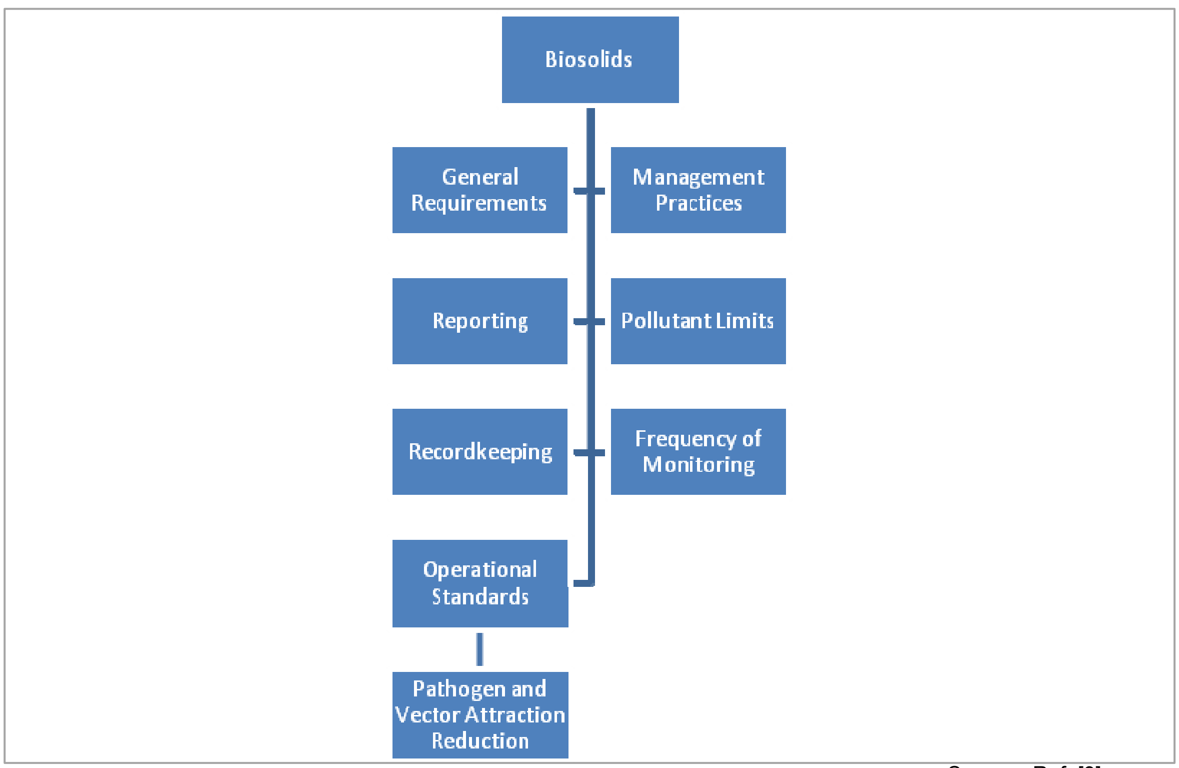
In 1998, the Ashland plant upgrade included facilities to lime stabilize waste activated sludge to meet Class B criteria and to dewater the biosolids using centrifuges prior to land application. The City of Ashland currently dewaters their waste activated sludge using centrifuges, and landfills the dewatered sludge without stabilization. The City's NPDES permit [3] states that the City is exempt from requirements to have a sludge management plan since they landfill their sludge in a State-approved facility. Landfilled sludge is regulated as a solid waste under OAR Chapter 340, Division 93.

Federal Regulations

The OARs for biosolids management are based on EPA biosolids regulations and contain detailed requirements regarding facility permits, responsibility for proper handling, limitations on the use of biosolids, agronomic rate application, land application site selection and approval, and biosolids management plans. The OARs also describe the State requirements for monitoring, recordkeeping, and reporting for land application sites [8].

In selecting the appropriate methods of solids processing, reuse, and disposal, consideration must be given to the established EPA biosolids regulations which are referenced in the OARs. In the United States, biosolid regulations are contained in *The Standards for the Use or Disposal of Sewage Sludge* (Title 40 of the Code of Federal Regulations, Part 503). This standard was published on February 19, 1993 and is commonly referred to as the Part 503 Rule. These regulations are all encompassing, and include requirements for monitoring, record-keeping, transporting, and disposing biosolids (See Chart 2.1). Biosolids management agencies apply for a permit covering biosolids use or disposal if they own or operate a treatment works treating domestic sewage [9].

CHART 2.1: Regulation Subparts Applicable to Ashland WWTP



Source: Ref. [9]

Pathogen Reduction [9]

Under the Part 503 Rule, biosolids are designated Class A or Class B in regard to the level of pathogen reduction achieved through treatment. These classifications indicate the density (numbers/unit mass) of pathogens in biosolids where applicable. Class A designations require greater reduction, but offer more disposal options than Class B or solids without pathogen reduction treatment.

Exceptional Quality (EQ) or Class A biosolids are considered to be the highest quality biosolid characterized by low pollutants, pathogens below detectable limits (including enteric viruses, pathogenic bacteria, and viable helminth ova) and reduced levels of degradable compounds that attract vectors. Once steps have been taken to generate a Class A biosolid, it is considered a product that is virtually unregulated and can be given away to the general public for use in home gardens as a compost or fertilizer.

Pollutant Concentration (PC) or Class B biosolids meet the same low pollutant concentration limits as EQ or Class A biosolids. However, they do not have similar pathogen reductions and are therefore, subject to site management practices. It should be noted that pathogens are reduced to levels that are unlikely to pose a threat to public health and the environment under specific use conditions. Class B biosolids cannot be sold or given away in bags or other containers to the general public, but may be applied to crops as fertilizer.

The Part 503 Rule lists six alternatives for treating biosolids to Class A standards (the treatment must address pathogen and vector reduction):

- Alternative 1: Thermally Treated Biosolids – Biosolids must be subjected to one of four time-temperature regimes.
- Alternative 2: Biosolids Treated in a High pH-High Temperature Process – Biosolids must meet specific pH, temperature, and air-drying requirements.
- Alternative 3: Biosolids Treated in Other Processes – The applicant must demonstrate that the process can reduce enteric viruses and viable helminth ova and then maintain operating conditions used in the demonstration after the pathogen reduction demonstration is completed.
- Alternative 4: Biosolids Treated in Unknown Processes – In lieu of demonstrating a treatment process to be maintained, biosolids are tested for several pathogens which include Salmonella sp. or fecal coliform bacteria, enteric viruses, and viable helminth ova at the time the biosolids are used or disposed, or, in certain situations, prepared for use or disposal.
- Alternative 5: Biosolids Treated in a Process to Further Reduce Pathogens (PFRP) - Biosolids must be treated using one of the listed PFRP options below:
  - Composting
  - Heat Drying
  - Heat Treatment
  - Thermophilic Aerobic Digestion
  - Beta Ray Irradiation
  - Gamma Ray Irradiation
  - Pasteurization

- Alternative 6: Biosolids Treated in a Process Equivalent to a PFRP -The regulatory agency can approve a process that is shown to be equivalent to the PFRPs listed under Alternative 5.

Chart 2.2 lists the specific pathogen requirements that must be satisfied by the selected treatment alternative in order for a biosolid to be considered Class A.

CHART 2.2: Class A Pathogen Reduction Requirements

The following requirements must be met for ***all*** six Class A pathogen alternatives.

Either:

the density of fecal coliform in the biosolids must be less than 1,000 most probable numbers (MPN) per gram total solids (dry-weight basis),

or

the density of *Salmonella* sp. bacteria in the biosolids must be less than 3 MPN per 4 grams of total solids (dry-weight basis).

Either of these requirements must be met at one of the following times:

— when the biosolids are used or disposed;

— when the biosolids are prepared for sale or give-away in a bag or other container for land application; or

— when the biosolids or derived materials are prepared to meet the requirements for EQ biosolids (see Chapter 2).

Pathogen reduction must take place before or at the same time as vector attraction reduction, except when the pH adjustment, percent solids vector attraction, injection, or incorporation options are met.

The Part 503 Rule lists three alternatives for treating biosolids to meet Class B standards:

- Alternative 1: The Monitoring of Indicator Organisms – Testing for fecal coliform density is used as an indicator for all pathogens. The geometric mean of seven samples must be less than 2 million MPN per gram per total solids or less than 2 million CFU's per gram of total solids at the time of use or disposal.
- Alternative 2: Biosolids Treated in a Process to Significantly Reduce Pathogens (PSRP) – Biosolids must be treated using one of the listed PFRP options below:
  - o Aerobic Digestion Air Drying
  - o Anaerobic Digestion
  - o Composting
  - o Lime Stabilization
- Alternative 3: Biosolids Treated in a Process Equivalent to a PSRP – Biosolids are treated using a process that has been determined to be equivalent to a listed PSRP by the regulatory agency.

Vector Attraction Reduction [9]

In addition to pathogen reduction, biosolids have different disposal options according to the level of Vector Attraction Reduction (VAR) achieved through treatment. The pathogens in biosolids pose a disease risk to humans via vector transmission. Vectors of concern include flies, mosquitoes, fleas, rodents, and birds. The Part 503 Rule contains 12 options, which are summarized in Chart 3, for demonstrating VAR.

CHART 2.3: Vector Attraction Reduction Options

Option 1: Meet 38 percent reduction in volatile solids content.

Option 2: Demonstrate vector attraction reduction with additional anaerobic digestion in a bench-scale unit.

Option 3: Demonstrate vector attraction reduction with additional aerobic digestion in a bench-scale unit.

Option 4: Meet a specific oxygen uptake rate for aerobically digested biosolids.

Option 5: Use aerobic processes at greater than 40°C for 14 days or longer.

Option 6: Alkali addition under specified conditions.

Option 7: Dry biosolids with no unstabilized solids to at least 75 percent solids.

Option 8: Dry biosolids with unstabilized solids to at least 90 percent solids.

Option 9: Inject biosolids beneath the soil surface.

Option 10: Incorporate biosolids into the soil within 6 hours of application to or placement on the land.

Option 11: Cover biosolids placed on a surface disposal site with soil or other material at the end of each operating day. (Note: Only for surface disposal.)

Option 12: Alkaline treatment of domestic septage to pH 12 or above for 30 minutes without adding more alkaline material.

Current sludge handling and disposal practices used by the City of Ashland will be evaluated based on these regulations and additional alternatives developed for consideration. Further discussion is included in Chapter 11 of this report.

2.2.6 GASB-34 Requirements

GASB-34 is short for Governmental Accounting Standards Board Statement 34: *Basic Financial Statements and Management's Discussion and Analysis for State and Local Governments*. This 1999 document requires state and local governments to switch from cash-based accounting to accrual-based accounting, which is considered to have less room for distortion.

Since 2005, the City of Ashland has implemented GASB 34 accounting practices. In fact, the City was awarded the “Certificate of Achievement for Excellence in Financial Reporting” by the Government Finance Officers Association. The City uses modified accrual,and it has set up sound criteria for capitalizing any fixed assets acquired whether for maintenance or for new acquisitions.

2.2.7 Greenhouse Gas Policies [10]

The Oregon legislature passed a bill in 2007 to curb the state’s greenhouse gas (GHG) emissions. Using 1990 emission levels as a benchmark, the bill established goals for GHG emissions of 10% below 1990 levels by the year 2020 and 75% below 1990 levels by the year 2050. In 2010 the Oregon Global Warming Commission began a "*Roadmap to 2020*" Project to offer recommendations for how to meet those goals. No policies or guidance relative to wastewater treatment plants have been developed at this point, and reporting of GHG emissions from wastewater treatment facilities has temporarily been deferred by DEQ pending adoption of a quantification protocol (GHG reporting is required for other facilities emitting 2,500 metric tons or more of carbon dioxide equivalent).

2.3 RECYCLED WATER (REUSE) REGULATIONS

Recycled water use in Oregon typically requires an NPDES or WPCF permit and a Recycled Water Use Plan (RWUP).

Reuse of wastewater effluent is governed by recycled water regulations as outlined in Oregon Administrative Rules (OAR) 340-55. The April 2008 revisions to Oregon's Recycled Water Use Rules allow the use of recycled water for beneficial purposes if the use provides a resource value and protects public health and the environment. Replacing another water source that would be used under the same circumstances or supplying nutrients to a growing crop, are considered as resource values and beneficial purposes.

OAR 340-55 defines five categories of effluent, identifies allowable uses for each category, and provides requirements for treatment, monitoring, public access, and setback distances. Irrigation of fodder, fiber, and seed crops not for human consumption is allowed for any class of effluent. Fewer restrictions are imposed for higher quality effluent, as shown in the table below.

TABLE 2.2: Requirements for Reuse of Effluent by Category [11]

	Class A	Class B	Class C	Class D	Non-disinfected
Treatment[1]	O,D,F	O,D	O,D	O,D	O
Effluent coliform, #/100 mL	2.2	2.2	23	126 ecoli	Per permit
Public access[2]		Limited	Limited	Controlled	Prevented
Setback to property line[3]		10 ft.	70 ft.	100 ft.	Per RWUP
Setback to water supply source		50 ft.	100 ft.	100 ft.	150 ft.

1. O = oxidized, D = disinfection, F = filtration
2. Limited public access: no direct contact during irrigation cycle
3. Sprinkler irrigation assumed

For recycled water use, groundwater must be protected in accordance with the requirements of OAR 340-40. For agricultural use, this typically translates to irrigating at agronomic rates to match the net irrigation requirements of the crops.

Reuse in treatment plant processes or for landscape irrigation at the plant is exempt from the rules of OAR 340-055 if the water is oxidized and disinfected, there is no off-site spray drift, and public access is restricted.

2.4 CITY POLICIES & GUIDELINES

2.4.1 Phosphate Ban

The City Council, in recognition of water quality issues in the Bear Creek sub-basin, instituted a phosphate ban in 1991 (City Ordinance 2623; Municipal Code 14.09.10 Phosphate Ban).

The ordinance prohibits the sale or distribution within the City of Ashland city limits of any cleaning agents containing more than 0.5 percent phosphorus by weight, except cleaning agents used in automatic dishwashing machines shall not exceed 8.7 percent phosphorus by weight.

2.4.2 Pretreatment Ordinance

The City of Ashland is not aware of any significant industrial users that would require development of an industrial pretreatment program. At the time this study was completed, neither the City nor DEQ had any records of a recent survey being completed to identify significant industrial users. DEQ has indicated that they will require that the City complete a industrial user survey to see if any existing facilities met current criteria. In the event that significant industrial users are identified, the City would be required to make modifications to their ordinances that would provide the City with the regulatory authority required to monitor and enforce EPA pretreatment requirements. Additionally, the City may need to enter into separate agreements or develop industry-specific permits with these users.

The City also has a significant number of food service establishments that generate fats, oils and grease (FOG) with the potential to cause sewer blockages that can lead to SSOs. Further discussion of pretreatment in this document will refer only to FOG issues.

The City conducted a FOG survey in spring 2010, with 35 food service establishments filling out questionnaires. (This represents about 35% of the food service establishments listed in the Ashland yellow pages.) Facilities in existence prior to the City's adoption of the plumbing code were not required to install grease traps, and there is currently no ordinance that would require existing facilities to install grease control devices.

Regulations for controlling FOG were drafted in 2005, but the ordinance proposing addition of the regulations to the Municipal Code has not been adopted. The draft regulations are quite extensive (40 pages), and include requirements for an industrial wastewater discharge permit from the City in addition to FOG pretreatment. The ordinance would require all *existing* Food Service Establishments to install grease control devices within three years of adoption of the regulations.

Though there is no formal FOG ordinance in place, the City has taken several steps to address the issue of FOG entering the sewer system through their draft FOG pretreatment program. A public education program has also been instituted. Flyers and brochures have been prepared for customers, and a guide (*Clean Drains for food service establishments*) has been made available to assist food service personnel in developing Best Management Practices (BMPs) that will reduce FOG discharged to the sewer system. These include BMPs for clean kitchen practices, recycling FOG, grease interceptor operation, grease trap operation, and vent hood and filters.

If the results of the educational effort do not prove sufficient to address FOG issues, the City should consider a more comprehensive enforcement-based program in addition to public education. Establishing legal authority over food service discharges can be accomplished by modifying the sewer use ordinance to specifically address oil and grease sources, writing a stand-alone sewer use ordinance, or directly permitting the sources (would require the most time and resources to implement). The FOG ordinance drafted in 2005 is a stand-alone use ordinance that also requires source permitting. A simpler ordinance could be developed that would achieve the City's goals, and should include the following components:

- Declaration of policy (objectives and authorization to adopt rules)

- Installation requirement
  - New food service facility, including addition of food service facility in existing building
  - Existing food service facility being remodeled
  - Existing food service facility that has contributed to grease problems or blockages in the sanitary sewer
  - Existing food service facility with change of ownership
- Sizing: Reference State Plumbing Code
- Maintenance requirement: Required cleaning frequency could be a constant for all sources (e.g. monthly for outside units, 1-2 weeks for inside units); specific to types of sources based on amount of grease generated and history of sewer blockages; or specific to individual sources based on capacity of grease control device, amount of grease generated by the source, BMPs implemented, and history of sewer blockages
- Recordkeeping: Facility to maintain pumping reports to document compliance with maintenance schedule
- Compliance: Based on enforcement of grease control device installation requirements and established maintenance schedules, with possible submittal of pumping reports and/or periodic inspections
- Established penalties for violations (so facilities know consequences of noncompliance beforehand), based on the severity and impact of the violation and the number of successive occurrences of the violation

2.4.3 Other Policies and Procedures

The City currently has many great collection and facility sewer policies and procedures. However, many of these policies and procedures are not currently written or codified. These include the following:

1. The City encourages training and certification of their operators, and is in the process of developing internal minimum number of hours required for operations staff to train in various categories at the treatment plant (including Headworks, Oxidation Ditch, lab, etc.).

2. Some elements that could be codified include the City's unwritten policy that the service line is the responsibility of the private owner from the mainline to their establishment /residence. Related to this would be the policy or code that would enable the City to require repairs when service lines are determined to be leaking.

3. Another practice that the City follows is regular TV and cleaning. The city has proactive procedures relating to the maintenance program that include adjusting frequency of cleaning and TVing of collection system, and frequent maintenance activities.

4. The treatment plant has a number of safety plans and procedures for separate components that should be incorporated into a coordinated safety program. Public works staff are regularly trained in safety practices which include items such as first aid, fall protection, confined space entry, etc. This training is provided by a third party entity that has been hired to provide this service.

References

1. Oregon Department of Environmental Quality: *Internal Management Directive - Sanitary Sewer Overflows*, November 2010.

2. U.S. Environmental Protection Agency: *Infiltration / Inflow, I/I Analysis and Project Certification*. Ecology Publication No. 97-03, May 1985.

3. Oregon Department of Environmental Quality, Western Region, Salem Office: *National Pollutant Discharge Elimination System Waste Discharge Permit No. 101609* (issued to City of Ashland, OR), May 27, 2004.

4. Oregon Department of Environmental Quality: *Bear Creek Watershed TMDL*, July 2007.

5. Jon Gasik: *City of Ashland Excess Thermal Load Limits*, WQ File No. 3780, ODEQ Memorandum, February 16, 2011.

6. http://water.epa.gov/scitech/swguidance/waterquality/standards/criteria/aqlife/pollutants/ammonia/upload/2009_12_23_criteria_ammonia_2009update.pdf)

7. http://www.deq.state.or.us/wq/SB737/index.htm

8. Oregon Department of Environmental Quality: *Biosolids Management Program;* http://www.deq.state.or.us/wq/biosolids, January 2011.

9. U.S. Environmental Protection Agency, Office of Wastewater Management: *A Plain English Guide to the EPA Part 503 Biosolids Rule;* EPA/832/R-93/003, Washington, DC, 1994

10. http://www.deq.state.or.us/aq/climate/reporting.htm

11. http://metolian.com/assets/Exhibit%2031%20Effluent%20DEQ%20(Comp).pdf

12. Carollo Engineers: *City of Ashland Water Conservation & Reuse Study (WCRS) & Comprehensive Water Master Plan (CWMP), Technical Memorandum No. 7 – Recycled Water Regulations*, September 2010.

13. Metcalf & Eddy, Inc.: *Wastewater Engineering: Treatment and Reuse, 4th ed.,* McGraw-Hill, New York, 2003.

14. North Carolina Dept. of Environment and Natural Resources: *Considerations in Establishing a Municipal Oil and Grease Program,* November 1999.

3.0 COLLECTION SYSTEM CONDITIONS

This chapter contains an evaluation of the existing wastewater collection system for Ashland, including lift stations and pipe condition. This chapter also includes an evaluation of existing flow data and projected design flows/.

3.1 WASTEWATER COLLECTION SYSTEM OVERVIEW

The Ashland collection system is comprised of approximately 110 miles of gravity sewer and 8 lift stations. A total of 15 diversions allow wastewater flow to be split between various sewer basins. The existing collection system is illustrated in Figure 3.1 of Appendix A.

3.2 LIFT STATION EVALUATION

Keller Associates visited each lift station site and completed a general inventory of facilities, and conducted pump tests at select stations. Appendix B summarizes information for each lift station. More detailed discussions of specific lift stations follows. A table summarizing the available data for each lift station is included in Appendix B along with pump curves, data sheets, and other data resources.

Standby power and/or backup provisions are available at all the lift stations. Standby generator facilities are on-site for the Winburn lift station. Other lift stations are equipped with plugs to quickly connect to a portable generator dedicated to the sewer system. Some of the lift stations also have provisions to allow for the City's portable sewer pump to bypass the wetwell and pump directly to the force main. In addition to the wastewater generator, the City also has two other generators that could be used for emergencies. Most of the lift stations have relatively small amounts of inflow, and therefore can go several hours between pump runs. The Creek Drive Lift Station will overflow to the gravity sewer, eliminating the need for standby facilities.

3.2.1 Creek Drive Lift Station

The Creek Drive Lift Station is a small submersible duplex pump station. The service area is relatively small, with fewer than 50 homes. Pumping records suggest that the lift station operates on average less than 2.5 hours per week. When the lift station was first inventoried in July 2010, both pumps were plugged, and the upstream gravity sewer pipelines were backed up enough that sewage would bypass the lift station and gravity flow to a nearby main line. City staff reported that the pumps had not been operational for more than a month. Clogging problems frequently plague the lift station. However, the problem has only been an issue the last few years and is believed to be a result of materials (i.e. rags, etc.) that are being flushed down by residents. Efforts to educate the residents have not eliminated the current problems.

While the overflow bypass may prevent sewer from backing up into residences, extended periods of no operation will result in septic conditions and accumulation of deposits within the collection system. If problems persist at the lift station, Keller Associates recommends that the City consider upgrading the pump station with chopper pumps. The City should also look at upgrading the lift station with three phase power.

3.2.2 Grandview Lift Station

The Grandview Lift Station is one of the older lift stations in Ashland. It also is one of the larger lift stations. The lift station has a wet well/dry well arrangement. The wet well vent had been plugged at the time of inspection because of concerns about odor. However, according to City staff the concerns were not valid and the plug could be removed.

At the time the lift station was inspected, City staff reported that the lift station would soon be upgraded with a lift station arrangement similar to the North Main Street Lift Station. Design for the new lift station has already been completed.

Pumping records suggest that the pumps run on average about 5 hours per week. The maximum weekly pump run time for the 2008-2010 period was 15.8 hours (2.25 hours/day). This would suggest that the existing lift station pump capacity is more than enough for existing peak flows.

The discharge forcemain for the lift station is reported to be a 6-inch steel pipeline. Keller Associates recommends that the condition of the force main be determined at the time (or before) the lift station is upgraded. Eventually, the force main should be replaced with a more corrosion resistant pipeline material such as PVC or HDPE. Alternatively, the City could also explore using a trenchless technology such as cured in place pipe lining.

3.2.3 Nevada Street Lift Station

The Nevada Street is another small lift station with a small service area. The lift station is the oldest lift station in the City. It utilizes a vacuum tank, and replacement parts have to be custom manufactured. City staff also report that there is some uncertainty on the force main size and material, which leaves the lift station as 4-inch galvanized pipe and is reported to be 6-inch steel at the discharge. City staff were not aware if the pipeline was cased under Ashland Creek.

Pump run time records for the 2008-2010 period suggest that the lift station runs on average just over 6 hours per week. The maximum weekly pump run time for this period was reported to be 57.6 hours (8.2 hours/day).

This lift station is located near a gravity pipeline that runs to the treatment plant. City staff report that the pipelines have been surveyed, and that it is possible to construct a gravity pipeline that would eliminate the need for this lift station. Given the current condition of the lift station and discharge pipeline, Keller Associates recommends that this lift station be abandoned within the next 5 years. According to City staff, this improvement should be budgeted for fiscal year 2012-2013.

3.2.4 North Main Lift Station

The North Main Lift Station was upgraded/replaced a few years ago and represents a "standard" lift station arrangement that will be the model for future lift stations and upgrades. The lift station is a duplex pump system, with a drop inlet, mixer pump, and ultrasonic sensor for level readings. The lift station piping and valving is such that bypass pumping of the force main or the wet well could be accommodated.

The 4-inch discharge pressure line connects to an older asbestos cement pipeline. At the time of the visit in July 2010, City staff were not aware of any problems with a line break of the pressure line. Keller Associates' experience is that AC pipelines generally have a much shorter life than PVC and HDPE pipelines. We would recommend the condition of the line be assessed periodically, and that the City budget to replace the force main within the 20-year planning period.

Pump run time records for the 2008-2010 period show that the lift station runs on average less than 8 hours per week. The maximum reported weekly flows resulted in the pumps running a total of 15.6 hours in a week. This would suggest that the pumps are more than adequate to handle peak flow periods.

3.2.5 North Mountain Lift Station

The North Mountain Lift Station is a duplex pumping system with self-priming pumps. City staff report that the lift station pumps lose prime about 3 times or more per year.

Keller Associates recommends that the City budget an upgrade of the lift station within the 20-year planning period. Upgrades would include converting the lift station to use submersible pumps, and modifying the layout to reflect the more standardized lift station arrangement employed at the North Main Lift Station.

Pump run time records for the 2008-2010 period show that the lift station runs on average approximately 8.6 hours per week. The maximum reported weekly flows resulted in the pumps running a total of 22.2 hours in a week. A pump test conducted on March 14, 2011 showed a single pump capacity of approximately 400 gpm and a dual pump capacity near 530 gpm. Pump test data and flow rate calculations are shown in Appendix B.

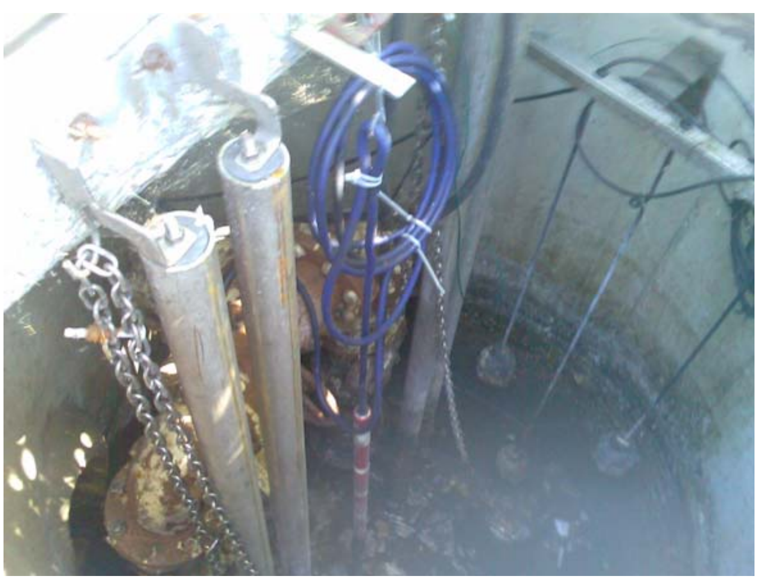
3.2.6 Shamrock Lift Station

The Shamrock Lift Station is a small lift station that services only a few connections. City staff report relatively no flow. Pump run time data is not conclusive, showing some periods with excessive pump run times. According to City staff, there is a loose coupling that occasionally comes apart which results in the high pump run times. Additionally, at least one of the services in this area (Napa Auto) could flow via gravity to the main line in Clay Street.

Many electrical panels for the lift station are located below ground in the dry well. Eventually, this lift station should be upgraded to a submersible type pump station. When the lift station is upgraded, the City should evaluate the rim elevations relative to potential flood levels from the nearby creek. As an alternative to the lift station upgrade, the City could evaluate the potential to abandon the lift station and use individual grinder pumps for the few establishments that utilize the lift station. According to DEQ, if the grinder pumps utilize a common force main, the pumps and force main will need to be owned and operated by the City.

3.2.7 Winburn Lift Station

The Winburn Lift Station is another small lift station. It is located in the parking lot adjacent to the Public Works Community Building. City staff report that the lift station is connected to the on-site generator that also services the public works facility.



The original installation had only one pump; however, City crews have since added a second pump. The old float system was recently abandoned, and a pressure transducer control system was installed.

At the time of the visit the valve vault was full of water. City staff were not certain if the floor drain was plugged or whether there was no drain. If one does not already exist, Keller Associates recommends that a floor drain be added to allow water to drain back to the wet well.

At the time of this evaluation, no pump run time data was available for review. However, City staff report that the lift station runs very little. A pump test at the lift station was not feasible, as the pumps empty the small wet well so quickly an accurate determination could not be made.

3.2.8 Ashland Creek Lift Station

The Ashland Creek Lift Station is the largest lift station in the system. It is located on the northeast corner of the treatment plant site and receives an estimated 63% of total collection system flows. The station is a triplex submersible pumping system. Previous studies reported pumping capacities of 1500 gpm per pump; however, the pump impellers were recently upgraded to return performance to the original design point. A pump test conducted on March 14, 2011 revealed each pump was individually capable of pumping 3150 gpm. This test combined with pump curve data indicate the pumping capacity of two pumps is approximately 5600 gpm, and all three pumps running simultaneously produce a flow of approximately 7400 gpm. A capacity evaluation of the Ashland Creek Lift Station is included in Chapter 9 with the evaluation of the wastewater treatment plant.

3.2.9 Lift Station Design Standards

As part of the lift station evaluation, Keller Associates reviewed the Romtec lift station design that has become the City's "standard" for new lift stations. The following recommendations were provided to improve upon this standard:

- Provide a wet well liner – Keller Associates recommends SprayWall as manufactured by SprayRoq (http://sprayroq.net/index.php/en/products/structural-spraywall).
- Wet well joints between sections – Keller Associates recommends that in addition to the rubber gaskets between the wet well sections, a polyurethane sealant be required near the inside joint and a butyl compound wrap on the outside of the joint.
- Flow meter – Typically we recommend a flow meter be installed at each lift station, with the flow meter placed in the valve vault or in a separate vault. We recommend a mag meter with the transmitter/totalizer mounted in the control panel and a continuous cable run from the meter to the totalizer.
- Standardized controllers – Keller Associates recommends the City continue with plans to standardize the controllers by requiring HydroRangers.
- Valve vault drain – As an alternative to the P-trap (which has a greater risk of clogging from rocks and debris), the City could consider a ball valve and can riser installed in the drain line between the valve vault and the wet well.
- Flexible restrained couplings – The pressure main should be equipped with flexible-restrained couplings between the wet well and valve vault.

- Influent shutoff valve – A slide gate placed on the influent pipe outlet for temporary shut off of flow to the wet well should be considered. For influent line depths less than 10', an in-line plug valve could also be considered.

It should be noted that the City could get DEQ approval of their lift station standards. This would allow City staff or a third party engineer to approve lift station plans without having to submit them to DEQ for approval.

3.2.10 Lift Station SCADA

The City of Ashland has SCADA at the lift stations and has been standardizing their controls using HydroRangers. Radio telemetry is used to transmit lift station data. Keller Associates recommends the following upgrades be made to the collection system SCADA system:

- Add continuous level monitoring and trending at each lift station.
- Add continuous monitoring and trending of pump on/off status.
- Create a monthly report that includes daily totalized flow (where flow meters are installed) and daily pump run times for each lift station.
- Add an alarm condition that is triggered when all pumps at a particular lift station are called on.

3.2.11 Summary of Lift Station Recommendations

This section summarizes the lift station recommendations by priority. Priority 1 improvements are intended to be completed within the next 10 years. Priority 2 improvements are intended to be completed within the 10-20 year period. Project costs for these improvements are included in Chapter 13, Capital Improvement Plan. A summary of Ashland Creek lift station needs and recommendations are presented with the wastewater treatment plant evaluation in Chapters 9 and 12.

Priority 1 Improvements

- Creek Drive chopper pumps and three phase power
- Replace Grandview Lift Station (already underway) and inspect force main condition. For budgeting purposes, we recommend planning on replacing the pipeline as part of the Priority 2 improvements. If the inspection of the pipeline shows significant remaining life, this improvement could potentially be delayed.
- Displacement of Nevada Street Lift Station
- Add drain from valve vault to wet well at Winburn Lift Station
- Add SCADA to lift stations

Priority 2 Improvements

- Replace Grandview Lift Station force main
- Replace North Main Lift Station force main. This upgrade should be coordinated with growth and construction of a new lift station to the northwest proposed in the Capital Improvements Plan, which would allow the existing North Main lift station and force main to be abandoned.

- Convert Shamrock Lift Station to a submersible type pump station. Flood proof lift station as required.
- Upgrade North Mountain Lift Station to reflect a more standardized lift station.

In addition to the capital recommendations above, Keller Associates recommends that the City consider shifting the responsibility of the lift station maintenance and management of the collection system staff from the wastewater treatment staff as is typically done in most communities of this size.

3.3 COLLECTION SYSTEM PIPELINE CONDITIONS

Table 3.1 summarizes the pipeline data in the City's GIS system. Approximately half of the collection system is made up of pipelines that are 6 inches in diameter or smaller. In considering future options for the replacement of these lines, the City should consider pipe bursting and open cut technologies that would allow the lines to be upsized to the current minimum pipe diameter standard of 8 inches.

TABLE 3.1: Ashland Sewer Pipe Summary

Pipe Diameter (in)	Pipe Material Lengths (ft)								*Total by Diameter (ft)*	*% of Total*
	Steel	HDPE	Ductile Iron	Clay	Concrete	PVC	Orange-burg	Unknown		
Unknown								3,082	*3,082*	*0.5%*
4"				194	184	290		1,749	*2,417*	*0.4%*
6"	142	4,053		72,661	187,565	10,581	979	17,416	*293,397*	*50.4%*
8"			358	16,003	58,402	132,128		633	*207,524*	*35.7%*
10"				7,186	16,092	982		60	*24,320*	*4.2%*
12"				2,224	14,639	8,565		1,924	*27,351*	*4.7%*
14"								1,090	*1,090*	*0.2%*
15"				429	7,624	765		33	*8,851*	*1.5%*
16"			289						*289*	*0.0%*
18"					2,993				*2,993*	*0.5%*
21"				1,517					*1,517*	*0.3%*
24"					1,718	7,075			*8,793*	*1.5%*
30"			86						*86*	*0.0%*
Total by Material (ft)	*142*	*4,053*	*733*	*100,214*	*289,217*	*160,386*	*979*	*25,988*	*581,712*	*100.0%*
% of Total	*0.02%*	*0.7%*	*0.1%*	*17.2%*	*49.7%*	*27.6%*	*0.2%*	*4.5%*	*110*	*MILES*

The pipe material information also gives some insight to the age and condition of the collection system. The oldest pipe is generally the clay pipe, which constitutes approximately 17% of the total collection system. City staff have indicated that, where the clay pipe is found to be structurally sound, the pipe is still in good condition. Concrete pipe is generally the next oldest pipe. Concrete pipe makes up approximately 50% of the City's collection system. This pipe material is susceptible to hydrogen sulfide corrosion and eventually should all be replaced. Two-thirds of the concrete pipe is also 6 inches in diameter. Steel and orangeburg pipe materials are also problematic and should be some of the first pipe sections considered for replacement. Prioritization should be based on pipeline conditions.

In addition to pipeline replacements, many of the City's manholes are in need of replacement or rehabilitation. Keller Associates recommends rehabilitation of manholes where large amounts of infiltration and inflow are encountered. Replacement and/or rehabilitation of other manholes should be evaluated in connection with adjacent pipeline rehabilitation/replacement projects.

3.4 COLLECTION SYSTEM MAINTENANCE

The City of Ashland has an active collection system maintenance program. This section discusses and evaluates City goals, TV recording and maintenance management software. A discussion of staffing is presented in Chapter 8.

3.4.1 Maintenance Goals

The City goals are summarized in the table below. The Cartegraph maintenance management software is used to measure the quantity and cost of most activities.

TABLE 3.2: Collection System Maintenance Annual Goals

Activity	Annual Goal	% of Total
Jet rod, clean lines	76 miles	69%
CCTV sewer lines	19 miles	12%
Smoke testing	1 mile	0.9%
Foaming for root control	3 miles	2.7%
Sewer pipe repairs	50	-----
MHs installed	10	-----
MHs replaced	10	0.5%
MHs repaired	10	0.5%

To meet the City's goal, the City must clean most of the pipelines annually, and CCTV the lines approximately every 5 years. A review of the previous three years' worth of maintenance records shows that the City has exceeded their annual goals for jet rod cleaning, CCTV, and root foaming. Of the approximately $380,000/year allocated for operations and maintenance of the collection system and lift stations, about $220,000/year is used for these three activities.

Keller Associates evaluated the cost per foot for each of these activities and found them to be well within industry standards:

- City Jet Roding: 90 miles per year at $0.24/ft
- City CCTVing: 25 miles per year at $0.63/ft
- City root foaming: 7 miles per year at $0.66/ft

While the City exceeds industry standards, discussions with staff suggest that there may be room for implementing additional efficiencies. Cartegraph currently provides a list of all the line segments to be cleaned in a given month. It is possible that adjacent pipeline segments could be on a similar cleaning schedule (e.g. once a year), but be scheduled months apart. One possible improvement to the system may be to look at the overall cleaning frequency of each line segment and try to more closely group monthly activities to a geographic location.

A review of the annual replacement / repair budget for manholes and pipelines shows that the City has averaged about $143,000/year for the last three years. Of this, approximately $40,000 - $50,000 is for materials and the balance of the costs are associated with labor and equipment. The annual replacement / repair budget amount is low considering the size and age of Ashland's collection system. The City currently has a backlog of several hundred identified needed system repairs. Priorities are currently given to repair projects that correspond to planned pavement projects.

Assuming a 75-year pipeline replacement schedule, the City should be looking at replacing approximately 7,750 feet per year. With a typical project cost of $100/foot for these replacements, the **City should be looking at an annual collection system replacement budget of close to $780,000/year**. Actual costs for replacement / rehabilitation will vary depending on construction techniques (i.e. open cut versus pipe bursting or lining), surface repair requirements, and project complexities.

3.4.2 CCTV Log Evaluation

The City uses the National Association of Sewer Service Companies (NASSCO) rating system for identifying problems such as cracks, roots, offset joints, and broken pipe. As of March 2012, there are two NASSCO certified employees. The majority of all pipeline inspections are completed by the same operator, making the rating system consistent over time and throughout the City. Keller Associates reviewed the CCTV logs for approximately 16 hours of video footage to compare what items were found and to make recommendations to how the City logs their system. Appendix B summarizes the conditions identified for the pipeline sections that were reviewed. Based on our review, we have the following general comments and recommendations:

- Keep digital files – The City currently does not keep a digital copy of the CCTV inspections. With the advances in digital technologies and digital storage, Keller Associates recommends that this data be stored digitally.

- Include photographs in the hard copy printouts – These photos should show the problems encountered in the field.

- Periodically review rating system – Keller Associates recommends that operations staff periodically get training refresher courses. In our review of the CCTV logs, we identified some problems (pipe sags, pipe offsets, and misaligned joints) that were not recorded in the hard copy logs. The City also identified some items that we did not initially identify. Having a second set of eyes occasionally review the ratings can also improve accuracy and may help to provide a thorough evaluation.

3.4.3 Maintenance Management System

The City's TV log ratings are entered into the City's Cartegraph maintenance management system. The SewerView module of Cartegraph is then used to develop an overall condition rating for each pipeline segment. Several years ago, the City developed weights for various conditions that are used to calculate an overall serviceability and structural rating of the pipeline segment. The City uses this ranking to guide them in prioritizing pipeline rehabilitation work. A separate spreadsheet of priority improvements is maintained. Currently, over 400 pipeline segments have been identified for either spot repairs or pipeline replacement/rehabilitation.

Keller Associates has reviewed the pipeline ranking system. Appendix B shows the condition rankings that were developed for the pipeline segments Keller Associates reviewed. These rankings were calculated using the weights and formulas developed by another community. For the most part, those receiving the worst rankings using the other communities' methods for calculating were comparable to those developed by the City. However, there were some discrepancies. Based on our review of the ranking criteria, Keller Associates offers the following recommendations for consideration:

- Overall categories – The City currently assigns all problems to two serviceability and structural categories. Cartegraph has the capability to include additional categories, such as roots and infiltration/inflow. Keller Associates recommends that the City consider using these additional categories to improve maintenance efforts. Having a root category, for example, could help prioritize the City's root foaming efforts.

- Weighting criteria – Keller Associates would recommend the following considerations in how the problems are weighted:
  - Increase weights for broken pipe, hole in sewer, and collapsed pipe. These conditions are severe enough that a more appropriate weight for severe conditions may be closer to 30.
  - The weight for cracks appears to be too high relative to more severe conditions such as broken pipes. Consider lowering these weights such that a heavy condition may be lower than 8-10.
  - Increase the weighting for pipeline sags (camera below water). Sags generally increase the risk of sediment buildup and hydrogen sulfide corrosion. Consider increasing these values by a factor of 2-3 times the current weight.
  - No weight for grease. Grease may drive operational considerations in terms of cleaning frequencies and pre-treatment programs, but generally has very little to do with condition of the pipe.
  - No weight for water level or flow. If the water level is high, then there is either a sag or the capacity of the pipe may be undersized. While it is good to indicate if there is a high water level, Keller Associates recommends that the pipeline capacities be evaluated separate from the pipeline condition.
  - Condition identifiers. The camera crews should periodically review the items being tracked in the Cartegraph system. In discussions with City staff, some of the identifiers (such as Dropped Invert) are not currently used. These should be removed to avoid confusion.
  - Increase the weight for surface wear (overall pipeline condition) such that light, medium, and heavy conditions may have weights closer to 5, 10, and 25 (or higher), respectively.

- Overall condition rating – When considering the overall condition of the pipeline, Keller Associates recommends that greater emphasis be placed on the structural rating than the serviceability rating of the pipeline segment.

- Pipeline length considerations – The current system does not account for pipeline length in determining the overall condition of the pipeline. Thus a 500 foot segment with five cracks (a crack every 100 feet) would receive the same ranking as a 50 foot segment with five cracks (a crack every 10 feet). For some types of problems, such as a broken pipe, the length of pipe may not influence whether or not a repair is required. However, for conditions such as cracks, previous repairs, and roots, the frequency of these conditions provides a better indicator of the overall pipeline condition. Additionally, failure to account for pipeline length for some problems may result in a pipeline with one collapsed pipe section getting a ranking lower than a similar pipeline with a dozen smaller, less urgent problems. It should be noted that modifying the automated ranking system may require additional programming of the Cartegraph system, and that this should be further investigated with the City's IT staff and Caretegraph.

While there may be some improvements that could be made to the City's system, it should also be pointed out that the judgment of an experienced operator should not be underestimated and adjustments to prioritization should be periodically made to account for limitations of any maintenance management system. Additionally, overall risk should be a consideration in prioritizing improvements. For example, roots in a commercial area susceptible to grease should receive a higher prioritization than roots in a residential area. Similarly, correcting structural problems in a pipeline servicing hundreds of users should be of more importance than a similar problem on a pipeline with a few services.

3.5 DIVERSIONS

Ashland has several diversion structures that allow for flow to be channeled through different trunklines and sewer basins. Table 3.3 summarizes the diversion information. Figure 3.1 also illustrates the location of these diversion structures. Through the process of calibration and evaluation of alternatives, Keller Associates analyzed different flow split arrangements and visually inspected the majority of the diversion facilities.

TABLE 3.3: Diversion Structures

Map ID#	City Manhole ID	Location	Primary Inlet(s)	Diversion Type	Primary Outlet	Div. Outlet(s)
1	4CC-007	Laurel & Hershey St.	S.W., 12-inch	Elevated Relief	N.E., 10-inch	S.E. 6-inch, elevated 7"
2	4CB-028	Laurel & Ohio St.	S.W., 10-inch	Elevated Relief	N.E., 10-inch	S.E. 6-inch, elevated 3"
3	9AA-019	N. Mountain	South, 10-inch	Elevated Relief	North, 10-inch	West, 8-inch, elevated 5"
4	9AC-041	7th & "B" St.	S.W., 10-inch	Elevated Relief	N.E., 12-inch	S.E. 10-inch, elevated 2"
5	10DB-009	Walker & Railroad	South, 8-inch	Elevated Relief	North, 8-inch	N.W. 8-inch, elevated 3"
6	14CB-008	Siskiyou & Clay	S.E., 10-inch	Elevated Relief	North, 8-inch	N.W. 8-inch, elevated 13"
7	3CC-005	Bear Creek Trunk near Fordyce	East, 15-inch (15" + 24" @ 3CC-003)	Split Flow	N.W., 24-inch	West, 15-inch
8	4DB-003	Bear Creek Trunk near N. Mountain	South, 24-inch S.E., 15-inch	Elevated Bypass	North, 24-inch	N.W., 15-inch, elevated 13"
old 9	5AD-003	Nevada near Cambridge				PLUGGED
10	15AB-037	Siskiyou & Walker	S.E., 8-inch	Elevated Relief	N.W., 12-inch	North, 6-inch, elevated 18"
11a	10BD-006	Wightman & Railroad	S.E., 8-inch	Elevated Relief	N.W., 12-inch	North, 12-inch, elevated 10"
11b	10BD-021	Wightman & Railroad	S.E., 12-inch South, 12-inch	Split Flow	N.E., 12-inch	West, 12-inch, elevated 3" N.W., 8-inch, elevated 0"
12a	10BA-004	Bear Creek Trunk at N. Wightman	East, 12-inch	Elevated Bypass	North, 24-inch	West, 12-inch, elevated 4"
12b	10BA-021	Bear Creek Trunk at N. Wightman	East, 12-inch South, 12-inch	Elevated Bypass	North, 12-inch (to west, 24-inch)	West, 12-inch
13	4DB-013	Bear Creek Trunk near N. Mountain	South, 24-inch	Elevated Bypass	N.W., 24-inch	North, 12-inch, elevated 2.5" (to N.W., 15-inch)

3.6 RECOMMENDATIONS

Recommended improvements for the Ashland wastewater collection system are in Chapters 8 and 9.

4.0 WASTEWATER DESIGN CONDITIONS

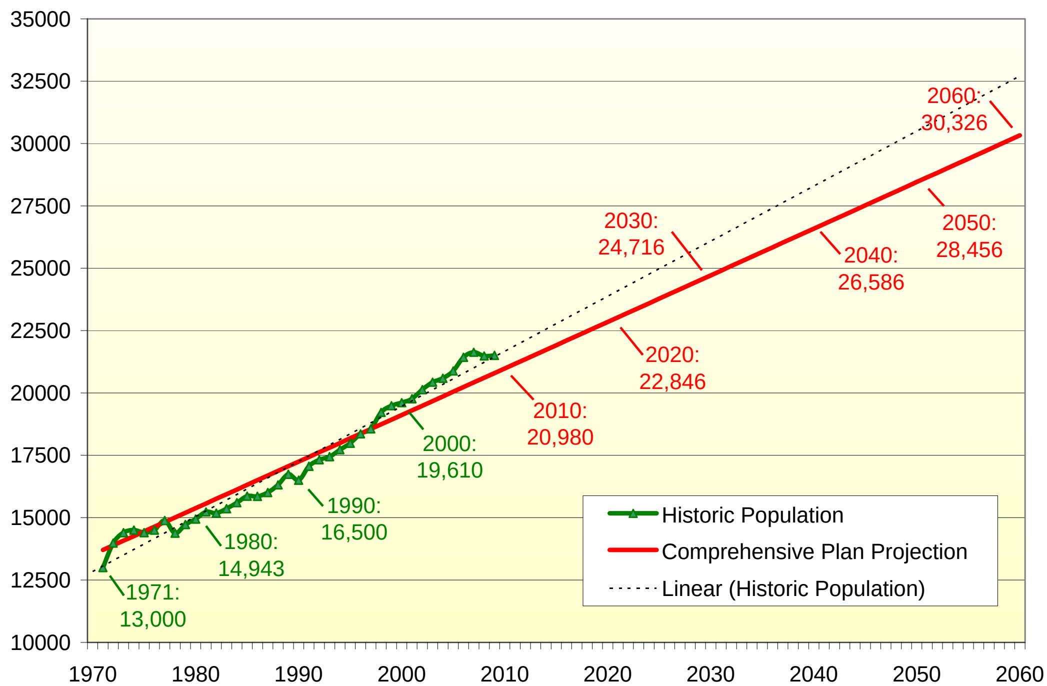
4.1 AREA DEMOGRAPHICS

For design considerations involving population, this study utilized values reported and developed in the 2005 Ashland Comprehensive Plan. A brief summary of historical and projected populations according to the plan is presented in Table 4.1 and Chart 4.1. Comprehensive plan projections were based on an assumed steady population increase of 187 persons per year.

TABLE 4.1: Historical & Projected Populations (1971-2060)
(abridged from 2005 Comprehensive Plan)

Year	Historic Population	Comprehensive Plan Projection	Population Change per year	Annual growth rate
1971	13,000		-	-
1980	14,943		216	1.56%
1990	16,500		156	1.00%
2000	19,610		311	1.74%
2009	21,505 (est.)	*20,793*	211	1.03%
2010		20,980	187	0.90%
2020		22,846	187	0.86%
2030		24,716	187	0.79%
2040		26,586	187	0.73%
2050		28,456	187	0.68%
2060		30,326	187	0.64%

CHART 4.1: Historic Population Trends and Projections (1971-2060)



4.2 STUDY AREA & LAND USE

For the purposes of this study, the study area was selected to match the Urban Growth Boundary (UGB) defined in the Ashland Comprehensive Plan. Due to the slow growth rate projected in the Plan, the UGB/Study Area boundary (illustrated in Figure 6.1) closely follows the existing City Limits with slight expansion to the northwest and southeast. Land Use and Zoning within the respective UGB and City Limits boundaries can be found in the Comprehensive Plan. Of greater importance to this study is the separate 2011 Buildable Lands Inventory (BLI). This report outlined land use densities for current and projected growth, and infill areas where projected growth could occur. A summary of the densities reported in the BLI and utilized in this study for developing future flows from growth areas is included in Table 4.2.

TABLE 4.2: Residential Density Assumptions (2011 BLI - Table 1)

Zone	Assumed Density	Type
R-1-3.5	7.2 units per acre	Suburban Residential (SR), Townhouses, Manufactured Home
R-1-5 & R-1-5-P	4.5 units per acre	Single-Family Residential (SFR)
R-1-7.5 & R-1-7.5-P	3.6 units per acre	Single-Family Residential (SFR)
R-1-10 & R-1-10-P	2.4 units per acre	Single-Family Residential (SFR)
R-2	13.5 units per acre	Multi-Family Residential (MFR)
R-3	20 units per acre	High Density Residential (HDR)
RR-.5 & RR-.5-P	1.2 units per acre	Rural Residential, Low-Density (LDR)
HC	13.5 (same as R2)	Health Care / Senior Housing
WR	Slope contingent	Woodland Reserve, Environmental Constraints
RR-1	0.6 units per acre	Rural Residential, Low-Density (LDR)

4.3 WASTEWATER TREATMENT PLANT FLOW DATA

Wastewater is treated in a wastewater treatment plant (WWTP) owned and operated by the City of Ashland. Daily and monthly flow rates into the treatment plant were provided by City personnel for years 2004 through 2009. Limited hourly flow data from 2008 was also reviewed as part of this study.

4.3.1 Historical Trends

City of Ashland Wastewater Treatment Plant (WWTP) influent flow data from 2004 to 2009 is included in Appendix C. This data was used to determine the average day, peak day, and peak monthly flows summarized in the table below.

TABLE 4.3: Historical Sewer Flows at WWTP, MGD (2005-2009)

MGD	2005	2006	2007	2008	2009	2005-9 Avg	Existing Design 2010	
Population[1]	20,880	20,900	20,920	20,940	20,960	20,920	20,980	
	MGD	MGD	MGD	MGD	MGD	MGD	**MGD**	**GPCD**
Average Day Dry-Weather[2] (ADWF)	2.14	2.15	2.08	1.95	1.96	2.06	**2.1**	**100**
Max Month Dry-Weather[2] ($MMDWF_{10}$)	2.41 May 4 - Jun 2	2.23 May 16 - Jun 14	2.15 Sep 30 - Oct 29	2.09 May 27 - Jun 25	2.15 May 2 - May 31	2.21	**2.7**	**129**
Annual Average Day[4] (AADF)	2.12	2.41	2.27	2.08	1.95	2.17	**2.2**	**105**
Average Day Wet- Weather[3] (AWWF)	2.09	2.68	2.45	2.21	1.94	2.27	**2.3**	**110**
Max Month Wet- Weather[3] ($MMWWF_5$)	2.41 Dec 6 - Jan 4	3.64 Dec 28 - Jan 26	2.96 Dec 13 - Jan 11	2.70 Jan 4 - Feb 2	2.13 Dec 20 - Jan 18	2.77	**3.6**	**172**
Peak Week (PWkF)	3.27 Dec 6-12, 2004	5.02 Dec 28-Jan3	3.98 Feb 21-27	3.51 Jan 4-10	2.41 Jan 1-7	3.64	**5.0**	**238**
Peak Day ($PDAF_5$)	5.48 Dec 4, 2004	8.39 Dec 30, 2005	4.86 Feb 24	5.88 Jan 4	3.01 May 4	5.52	**7.1**	**338**
Peak Instantaneous (Hour) (PIF_5)	-	-	-	10.00 Jan 4	6.00 May 4	NA	**10.5**	**500**

[1] Populations projected linearly between 2005 & 2010 known populations
[2] Dry-Weather Period= May – October
[3] Wet-Weather Period = November (previous year) – April
[4] Yearly Summaries Period = Nov 1 – Oct 31

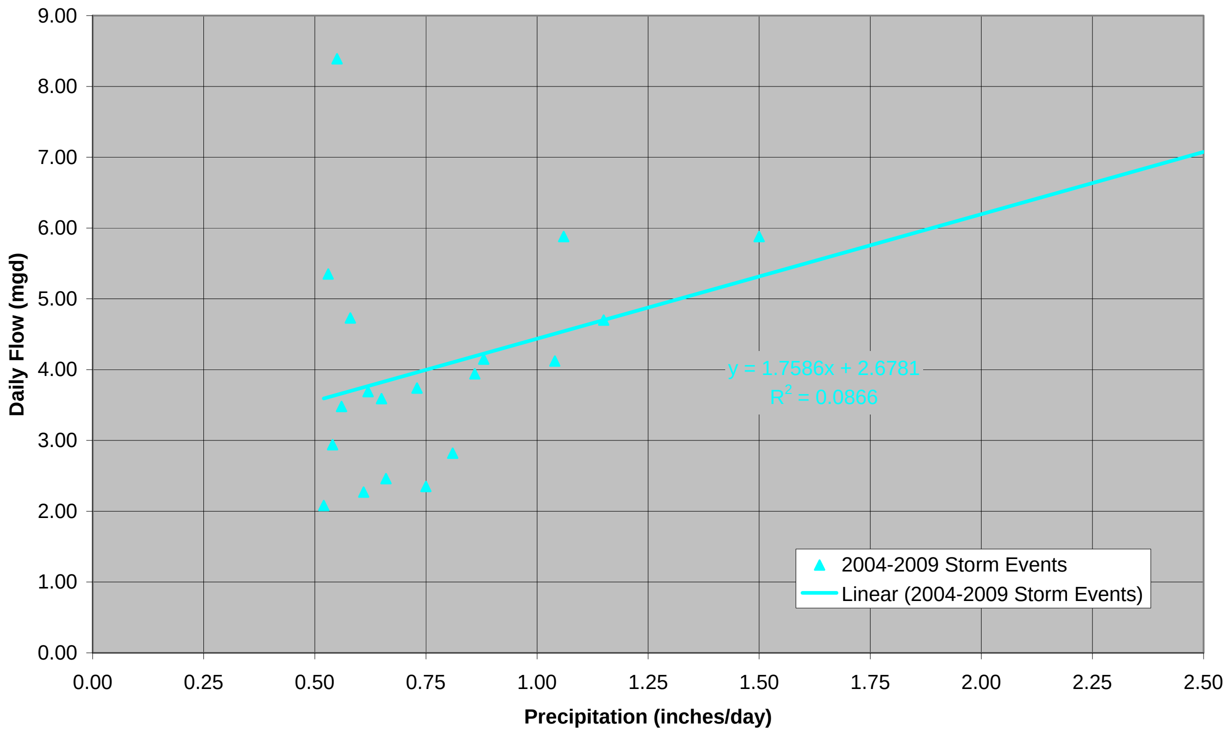
The annual average day flow represents the average flow during the entire year. The peak daily flow represents the highest average day flow during that year. The max monthly flow represents the highest average flow across an entire month for that period. **Dry-Weather** periods are **May-October** and **Wet-Weather** periods are **November-April**. Design 2010 flows are provided for reference and are discussed in Section 4.5 of this chapter.

4.3.2 Average Day and Peak Day Flow Rates

The data shows a decreasing trend in the flow rates, represented by an 8% decrease in the average day flow rates from 2005 to 2009. This downward trend is reflected in every parameter (up to 14% for MMWWF). The decreased flows are most likely the result of drier weather as seen from graphs of daily precipitation across each year (see Table 4.6).

As discussed in Appendix C the calculated Design Peak Day Flow is based on the correlation between peak precipitation events and Daily Flows. Chart 3.1 shows those points from 2005-2009 used in our analysis.

CHART 4.2: Daily WWTP Flow and Precipitation, Dec-May (2005-2009)



4.3.3 Seasonal Variations in Flow Rates

During wet-weather periods, flows are noticeably higher than during dry-weather periods. Flows increase with precipitation, typically rising during the second week of December with peak flows in January before falling off in February. Winter months have more significant peak day events and maximum monthly totals are typically 125% of average summer flows. This increase is a result of infiltration and inflow.

4.3.4 Peak Hourly Flow Rates

Hourly flow data was evaluated for several wet weather and dry weather days in 2008 and 2009 (SCADA data was only available after Sept 2007). This data was used to evaluate flows observed throughout the day during wet weather and dry weather periods. Table 4.4 lists the days and flow rates for the observed dates.

In recent years, instantaneous flows as high as 10.0 MGD have been recorded at the WWTP. The largest peak hour events correspond to rain events, believed to result primarily from inflow and shallow groundwater infiltration into the collection system.

TABLE 4.4: Wastewater Treatment Plant Peak Flow Events

Date	Avg. Day Flow (GPM)	Peak Hour Flow GPM (MGD)	Peak Hr Multiplier	Rainfall (in)
Wet Weather Period 2008				
January 4, 2008	5.88	10.00	1.70	1.06
January 5, 2008	3.74	5.20	1.39	0.73
January 6, 2008	3.20	4.60	1.44	0.09
January 14, 2008	3.13	3.60	1.15	0.01
January 31, 2008	3.20	6.00	1.88	0.05
February 2, 2008	3.28	4.50	1.37	0.05
February 3, 2008	3.20	4.60	1.44	0.33
Dry Weather Period 2008				
May 27, 2008	2.64	3.80	1.44	0.87
May 28, 2008	2.99	5.00	1.67	0.64
August 19, 2008	2.10	3.60	1.71	0.67
Wet Weather Period 2008-2009				
December 19, 2008	1.76	3.20	1.82	0.53
December 21, 2008	2.30	3.60	1.57	0.11
December 24, 2008	2.20	3.20	1.45	0.20
December 25, 2008	2.10	3.30	1.57	0.57
December 28, 2008	2.40	3.80	1.58	0.18
January 2, 2009	2.94	3.30	1.12	0.54
January 25, 2009	2.27	3.60	1.59	0.61
March 15, 2009	2.02	3.40	1.68	0.44
March 16, 2009	2.61	4.10	1.57	0.46
March 17, 2009	2.27	3.50	1.54	0.00
Dry Weather Period 2009				
May 3, 2009	2.69	5.80	2.16	0.40
May 4, 2009	3.01	6.00	1.99	0.37
May 5, 2009	2.46	4.00	1.63	0.48

The peak hour multipliers were calculated by dividing the observed peak hour by the observed average flow for each day. During drier weather periods, peak hour multipliers range from 1.44 to 2.16. During wet weather periods, the peak hour multipliers range from 1.12 to 1.88. The high peaking factors observed in both dry and wet weather periods suggest that there is a large amount of storm water inflow during storm events.

4.3.5 Per Capita Flow Data

A summary of historical flows is listed in Table 4.5. The flow per capita is based on population data or estimates for the respective years. Flows per capita were calculated by dividing the total flow (see Table 4.3) by the population and thus include commercial, industrial, and public use. A discussion of residential & commercial portions of the flow is included in Section 4.5.2.

TABLE 4.5: Historical Per Capita Sewer Flows, GPCD (2005-2009)

GPCD	2005	2006	2007	2008	2009	2005-2009 Avg	Design 2010
Population[1]	20,880	20,900	20,920	20,940	20,960	20,920	20,980
Average Day Dry-Weather[2] (ADWF)	102.7	102.8	99.6	93.3	93.7	98.4	**100**
Max Month Dry-Weather[2] ($MMDWF_{10}$)	115.1 May	104.7 May	102.4 Oct	99.5 May	102.0 May	104.7	**129**
Annual Average Day[4] (AADF)	101.4	115.5	108.3	99.4	93.0	103.5	**105**
Average Day Wet-Weather[3] (AWWF)	100.1	128.2	116.9	105.6	92.4	108.7	**110**
Max Month Wet-Weather[3] ($MMWWF_5$)	113.5 Dec 2004	160.5 Jan	128.6 Feb	125.4 Jan	97.5 Jan	125.1	**172**
Peak Week (PWkF)	156.7 Dec 6-12, 2004	240.1 Dec28-Jan3	190.0 Feb 21-27	167.4 Jan 4-10	114.8 Jan 1-7	173.8	**238**
Peak Day ($PDAF_5$)	262.5 Dec 4, 2004	401.4 Dec 30, 2005	232.3 Feb 24	280.8 Jan 4	143.6 May 4	264.1	**338**
Peak Instantaneous (Hour) (PIF_5)	-	-	-	477.6 Jan 4	286.3 May 4	381.9	**500**

[1] Populations projected linearly between 2005 & 2010 known populations
[2] Dry-Weather Period= May – October
[3] Wet-Weather Period = November (previous year) – April
[4] Yearly Summaries Period = Nov 1 – Oct 31

4.4 INFILTRATION & INFLOW

Infiltration and inflow represent extraneous groundwater and storm runoff that enters the sewer system. Infiltration refers to groundwater that enters the wastewater collection system indirectly through leaky pipes and manholes. Inflow refers to storm water that enters the collection system directly through any number of sources, including the holes in manhole lids plus roof drains, foundation/basement drains, and storm catch basins connected to the sewer system.

Chart 4.3 shows monthly historical precipitation plotted with historical WWTP inflow. A noticeable trend between average monthly precipitation and average influent flow rates reflects the influence of infiltration and inflow at the WWTP. Charts 4.4 and 4.5 illustrate the daily flows and precipitation data for 2005-2007. The rapid response between precipitation events and increased flows at the WWTP suggest that a signification component of peak plant flow is from storm water inflow.

CHART 4.3: Monthly WWTP Influent and Precipitation for 2005-2009

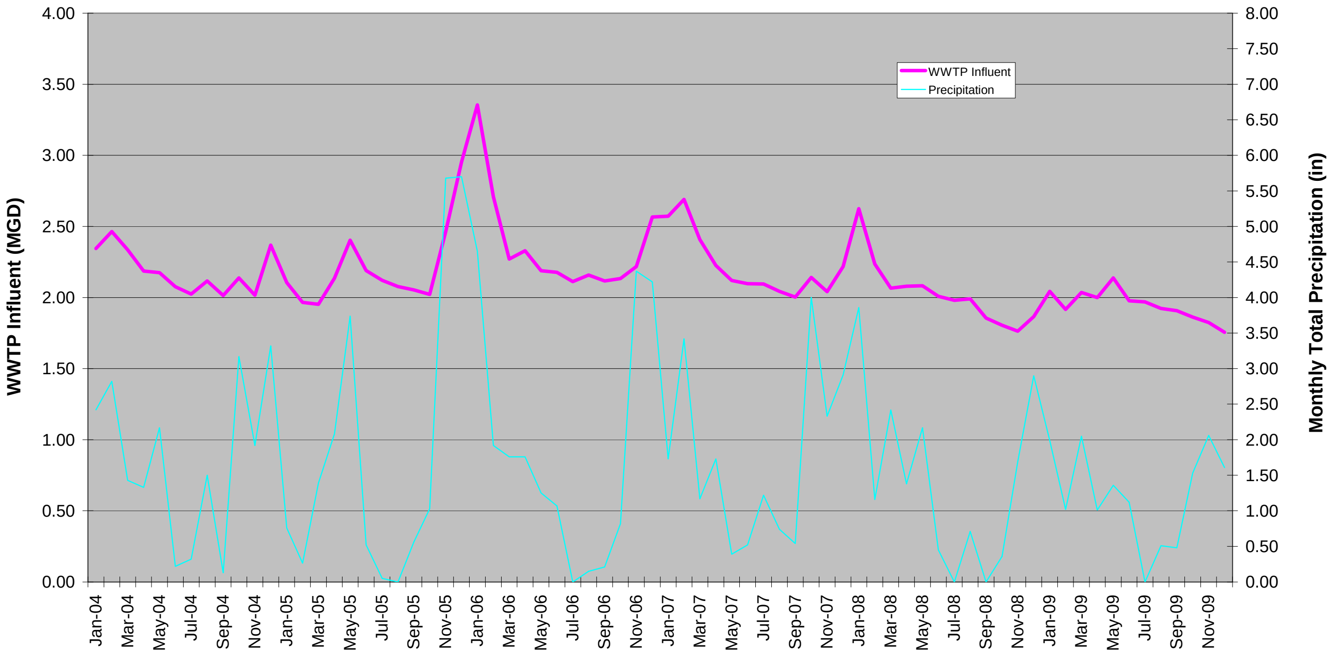


CHART 4.4: Daily WWTP Influent and Precipitation, Oct 2005 – Apr 2006

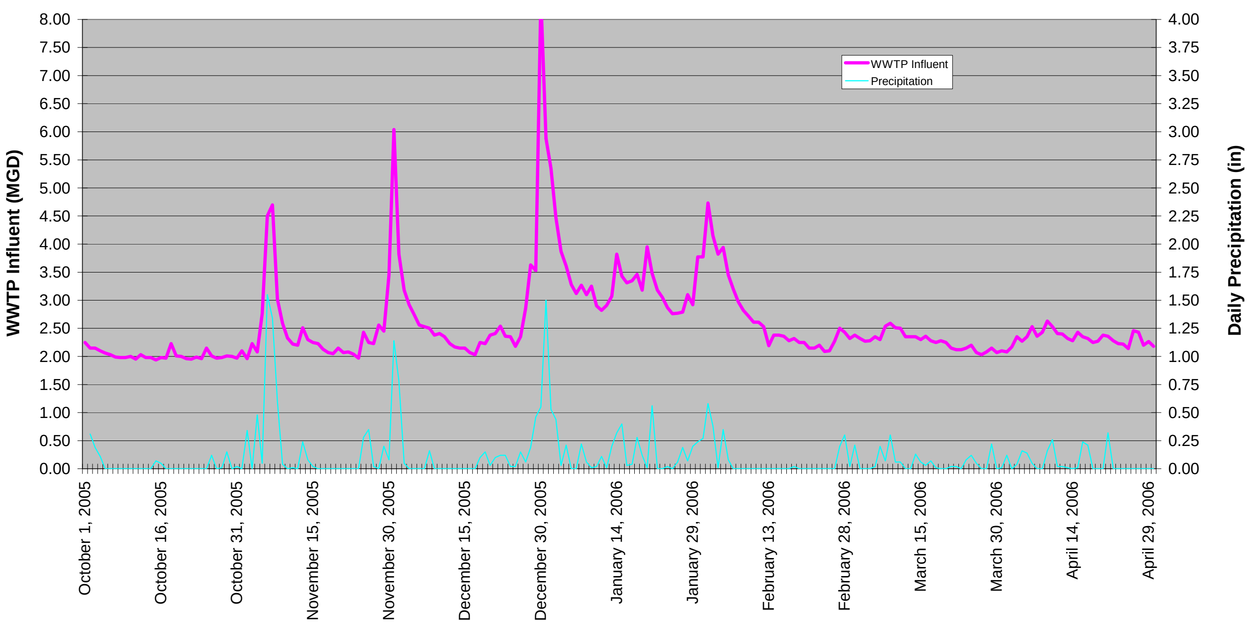


CHART 4.5: Daily WWTP Influent and Precipitation, May 2006 – Apr 2007

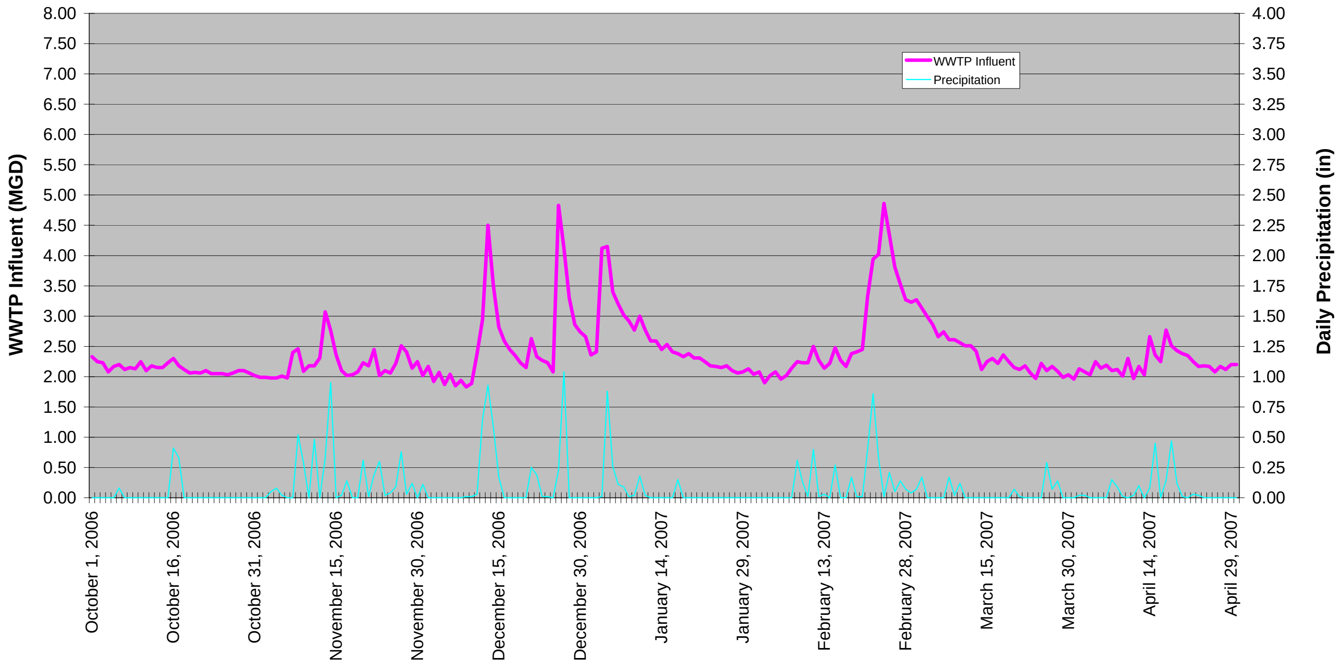


TABLE 4.6: Historical Precipitation, Total Inches (2005-2009)

Total Precipitation inches	2005	2006	2007	2008	2009	2005-2009 Avg
Average Month Dry-Weather	0.98	0.58	1.24	0.62	0.83	0.85
Max Month Dry-Weather	3.74 May	1.25 May	4.00 Oct	2.17 May	1.53 Oct	2.54
Annual Average Month	1.81	1.85	1.73	1.43	1.23	1.61
Average Month Wet-Weather	2.65	3.11	2.22	2.24	1.62	2.37
Max Month Wet-Weather	5.70 Dec	4.65 Jan	3.42 Feb	3.86 Jan	2.06 Nov	3.94
Peak Week	4.36 Nov 2-8	3.73 Dec 27-Jan2	2.45 Oct 16-21	2.43 Jan 4-10	1.40 Apr 29-May 5	2.87
Peak Day	1.55 Nov 6	1.04 Dec 27	1.47 Oct 19	1.06 Jan 4	0.61 Jan25	1.15

EPA defines excessive infiltration and inflow (I/I) as the quantity of I/I that can be economically eliminated from a sewer system by rehabilitation. Oregon DEQ has indicated that an infiltration and inflow study would be required before state revolving loan funds could be used for plant expansion projects, and that this study should include smoke testing of the City's older pipelines and additional statistical analysis of flow data. Keller Associates concurs that Ashland's existing flows are high (but not atypical for western Oregon cities) and that an I/I study could help prioritize collection system rehab work, reduce flows to the treatment plant, and potentially delay some capital improvements.

4.5 DESIGN FLOWS

Existing 2010 design flows were calculated according to the method outlined by Oregon DEQ. Future flows were calculated by adding projected growth to the existing flows. These two components are described below.

4.5.1 Existing Flows

Historical flows presented were utilized to calculate 2010 Design Flows according to the ORDEQ design memo "Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon" (refer to Appendix C for calculations).

The design peak day and peak hour flows, with peaking factors of 3.1 and 5.3 respectively, are not uncommon for cities in Western Oregon. It is also interesting to note that the 5-year, 24-hour flood event of 2.5 in/day was not approached during the 5 years of data, even during the 8.39 MGD event of December 2005.

Although the city has plans to reduce the I/I occurring in the existing system, the integrity of the system naturally degrades over time. Therefore, Keller Associates recommends that existing flows be used for existing developments in all future analysis as a conservative estimate.

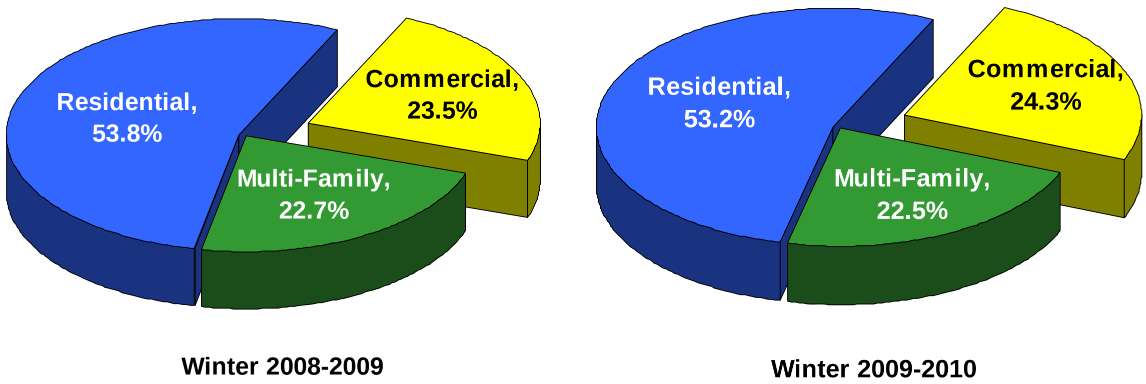
4.5.2 Projected Flow from Future Growth

Future flows produced by residential growth and commercial/industrial/public growth were calculated by adding the additional wastewater flow of new developments to the existing flows.

Nonresidential Growth

As shown in Chart 4.6, the calculated nonresidential portion of Ashland flows is approximately 24 percent. These numbers were developed from analysis of water consumption records for winter months December-February. For the purposes of projecting future flows, Keller Associates assumed that nonresidential growth will continue to make up approximately 24 percent of the total flow.

CHART 4.6: Categorized Sewer Flows (2008-2010)



In looking at potential flows for sewer basins, Keller Associates also assumed that projected flows will follow the design average day flow per acre per day shown in the table below.

TABLE 4.7: Sewer Flows Assumed for Nonresidential Growth (GPAD)

Land Use	Typical Average Day Flow (gpad) *	Design Average Day Flow (gpad)
Commercial (acres)	800-1500	1500
Commercial Retail (acres)	800-1500	1500
Industrial Ag (acres)	1500-3000	2500
Industrial Commercial (acres)	1000-1500	1500
Light Industrial (acres)	1500-3000	2000
Public (acres)	-	500

* Linsley, "Water-Resources Engineering", 4^{th} Edition.

4.6 FUTURE FLOW RATES

Existing per capita flow rate was evaluated and determined to have a significant amount of I/I. The City has a program to remove excess I/I where economically feasible. In addition, future new construction should not have the same I/I problems due to newer, more water-tight components. Taking the above into account, flows were projected based on populations outlined in the comprehensive plan and assumed flows in gallons per capita per day. The projected flow rates are presented in the table below and include residential, commercial, and I/I combined flows.

TABLE 4.8: Projected Future Ashland Flow Rates

MGD	Design 2010	Projected Unit Flow	2015	2020	2030	2040	2050	2060
Population[1]	20,980	-	21,913	22,846	24,716	26,586	28,456	30,326
		gpcd[4]						
Average Day Dry-Weather[2] (ADWF)	**2.1**	100	2.19	2.29	2.47	2.66	2.85	3.04
Max Month Dry-Weather ($MMDWF_{10}$)	**2.7**	129	2.82	2.94	3.18	3.42	3.66	3.90
Annual Average Day (AADF)	**2.2**	105	2.30	2.40	2.59	2.79	2.98	3.18
Average Day Wet-Weather[3] (AWWF)	**2.3**	110	2.40	2.50	2.71	2.91	3.12	3.32
Max Month Wet-Weather ($MMWWF_5$)	**3.6**	172	3.76	3.92	4.24	4.56	4.88	5.20
Peak Week (PWkF)	**5.0**	150	5.14	5.28	5.56	5.84	6.12	6.40
Peak Day ($PDAF_5$)	**7.1**	250	7.33	7.57	8.03	8.50	8.97	9.44
Peak Instantaneous (Hour) (PIF_5)	**10.5**	350	10.83	11.15	11.81	12.46	13.12	13.77

[1] Populations copied from Comprehensive Plan
[2] Dry-Weather = May – October
[3] Wet-Weather = November – April
[4] gpcd = gallons per capita per day

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5.0 COLLECTION SYSTEM MODEL DEVELOPMENT & EXISTING SYSTEM EVALUATION

This chapter summarizes the wastewater collection system model development process and existing collection system analysis. It outlines the model construction and model calibration process, and also documents existing deficiencies. Recommended improvements to address these deficiencies are presented in Chapter 8.

5.1 MODEL SELECTION

Innovyze (previously MWH) InfoSewer 7.0 was selected as the modeling software for this project. The software was selected for its compatibility with the City's GIS, allowing data to be efficiently updated into and exported from the model. The model was also selected for continuity with the water modeling software, InfoWater.

InfoSewer software is formatted to function through an ArcMAP application, enabling the user to operate the model while also viewing multiple GIS layers and databases from which the modeling parameters may be based. The software has capabilities for creating various scenarios within a single modeling file, which may contain unique data sets of pipes, manholes, and analysis results. This function allows a customized evaluation of multiple "what if" scenarios, without having to create a new modeling file for each option. These capabilities provide the City with a powerful management, planning, and analysis tool that can be updated and grow along with the City's system.

5.2 MODEL UPDATE

Information from a previous Hydra computer model and City-maintained GIS database were used to populate pipe diameter and invert elevation data in the model. In places where the previous model and GIS contained conflicting data, field investigations were performed by City and/or Keller staff to resolve discrepancies.

Once all manholes and pipes were created and data populated in the model, several queries were conducted to reveal anomalies in the data. These included reverse slope pipes, changes in pipe size, and anomalies in the pipe connectivity. These anomalies were then discussed with City personnel, additional field work was completed, and appropriate changes were made to the model.

Following the initial model evaluation, additional field work was completed to check pipe sizes of identified bottlenecks.

5.3 MODEL CALIBRATION

Model loads refer to the wastewater flows that enter the sewer collection system. These loads are comprised of wastewater collected from individual services (base flows), plus groundwater infiltration and storm water inflows (I/I). Loads for the model were developed and calibrated in several stages as described below.

5.3.1 Flow Monitoring

The first step in calibrating the model was collecting flow data at various manholes throughout the system. Eight (8) monitoring sites were selected to correspond to those of the previous study to allow comparison with previous modeling results (see Appendix C for map showing locations of meters). The collected data was then analyzed along with continuous precipitation data to establish average flows and typical 24-hour patterns at each site. A typical day was selected for each site which was utilized in the model for loading and calibration efforts. These typical patterns were assigned to all existing flows within each basin corresponding to the monitoring site. Appendix C contains a summary of the data and analysis used for modeling purposes.

For base flows, winter water consumption data was utilized (specifically December 2009 meter readings). Individual water meter records for every customer in Ashland were linked to the sewer model using GIS to provide a highly accurate distribution of dry weather flows. A winter month was used because it is most likely to exclude additional usage for irrigation that would not return to the sewer collection system. It was generally assumed that 90% of water consumed is returned to the sewer collection system. As shown in Table 5.1, the 2009 average winter daily discharge value (=90% of Winter Consumption) was then compared to the 2009 average dry weather plant influent flow. This value represents not only the total collected from sewer services but additional flow from other sources such as infiltration and inflow (I/I) into the collection system. The comparison revealed **additional flow due to I/I equals approximately 30% of base flows**. This additional flow was initially assigned in the model uniformly across all basins. Once the base and I/I flows were assigned, the model was run and compared to the target flow monitoring data, and then adjusted as necessary to simulate observed conditions.

TABLE 5.1: Summary of Estimated Total System Inflow and Infiltration

December 2009 Average Daily Water Consumption (MGD)
1.6
Return Flow: 90% of December 2009 Average Daily Consumption (MGD)
1.5
August 2009 Average Daily WWTP Influent (MGD)
1.9
August 2009 WWTP Inflow as Percentage of Return Flow (%)
130%

5.3.2 Dry Weather Calibration

The general procedure for achieving an accurate dry weather calibration of the model was to work downstream basin by basin and adjust the infiltration and inflow (I/I) flow up or down as necessary to make the modeled flows and observed flow data match. Several iterations were necessary due to the multiple diversion manholes located throughout the city. The percentage of flow diverted was also manipulated, with guidance from field observations, to help the calibration efforts. In addition to calibrating the model at various locations within the collection system, total modeled influent flows at the Wastewater Treatment Plant (WWTP) were also compared to the targeted design average daily flow. Example calibrations are shown in Chart 5.1 and Chart 5.2.

CHART 5.1: Sample Dry Calibration Site 8 Modeled vs. Observed Flows (MH 09AC-040)

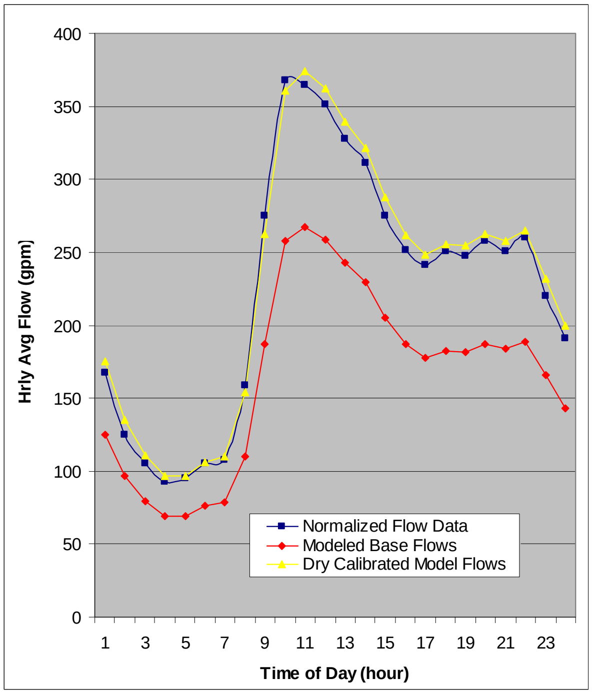
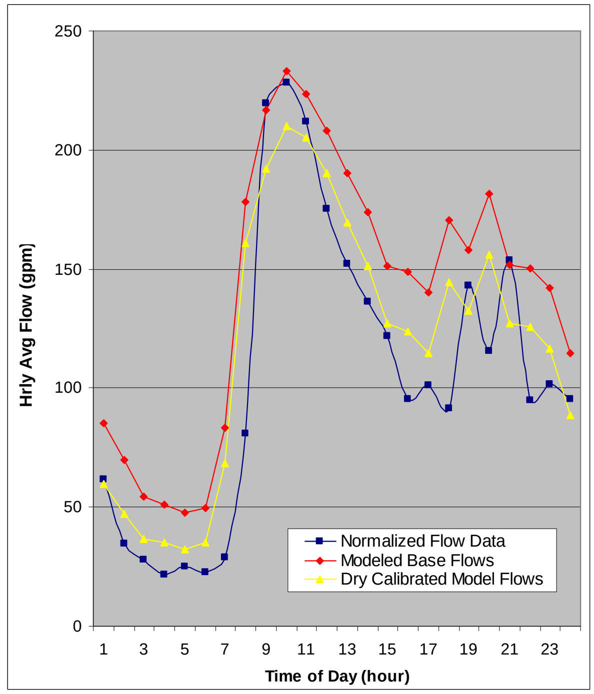


CHART 5.2: System Dry Calibration Site 7 Modeled vs. Observed Flows (MH 10BC-039)



5.3.3 Wet Weather Calibration

As part of this study, flow monitoring was completed during the wet weather period from December 2010 through January 2011. Unfortunately, flows during this period were relatively low. In fact, many days reported flows near observed dry weather flows. Because the anticipated seasonal increase was not observed, Keller Associates used an alternative approach. For the initial wet weather calibration, a peak day factor was applied globally to all base and I/I loads to reach a 3.5 MGD event, which corresponded to flows observed during the previous wastewater planning effort. Modeled flows at each monitoring site were then compared to observed flows from the previous wastewater planning study, with excellent correlation.

A second global peak day factor was applied to reach the design Peak Day Average Flow (PDAF) of 7.1 MGD. Modeling results in the form of pipeline flows (as percents of full capacity) and surcharging locations were noted and reported to City staff for validation. Further field investigations were conducted to prove or invalidate model parameters resulting in the localized high flows. Several new pipe sizes and elevations were noted and adjusted in the model. Total influent flows at the WWTP and the locations of potential surcharging were in agreement with City observations for historical flow events of a similar magnitude.

5.4 EXISTING SYSTEM DEFICIENCIES

The calibrated model was exercised to determine the effects of a 2011 peak day flow event on the system. Figure 5.1 in Appendix A illustrates the available capacity of the existing system. The figure is color-coded to show a gradation of pipes based on utilized capacity (e.g., red = flowing at >100% capacity, orange = flowing at 90-100% of capacity, yellow = flowing at 75-90% capacity, etc.). Those sections shown in red experience pipeline surcharging and represent the greatest risk for backing up services and possible flooding.

The majority of pipes nearing or at capacity are located at bottlenecks in the system created by changes in pipe size or slope.

It should be noted that some of the pipelines showing >100% capacity resulted in sanitary sewer overflows, or surcharging above manhole rim elevations. Those locations have been noted on Figure 5.1. Although present in the model, overflows at these locations have not been observed by City staff, potentially due to the extra storage available in lateral lines which were not modeled. Surcharging in these locations is still highly probable and Keller Associates recommends continued monitoring and investigations, especially during high flow events, to determine the actual extents of any surcharging that occurs.

5.5 PIPELINE CONDITIONS

In-field pipeline material conditions are discussed in Chapter 3 of this report. However, it is important to note that one of the basic assumptions of the hydraulic model is that all of the lines are free from physical obstructions such as roots and accumulated debris. Such maintenance issues, which certainly exist, must be discovered and addressed through maintenance efforts. The modeled capacities discussed in this chapter represent the capacity assuming the sewer lines are in good working order.

6.0 EVALUATE FUTURE COLLECTION SYSTEM PERFORMANCE

This chapter summarizes future flow projections and the model evaluation of future system expansion, and documents anticipated future deficiencies. Recommended improvements to address these deficiencies are presented in Chapter 8.

6.1 FUTURE FLOW RATE PROJECTIONS & MODEL SCENARIOS

Future residential and commercial/industrial loads were distributed assuming that flows per acre for new development would be similar to existing flows per developed acre. Figure 6.1 illustrates the future growth areas within the City and within the Urban Growth Area boundary that were used to apply the future loads to the system. Table 6.1 summarizes the commercial design flows utilized in the model for new growth. Commercial flows per acre were calculated utilizing known acreage of developed commercial areas (including industrial and employment zones) and an estimated percentage of total plant influent. The portion of total plant influent contributed by commercial discharges was estimated utilizing winter water usage data.

TABLE 6.1: Future Commercial Peak Day Design Flows

Area Type	City Zones	Design Flow per Area (gpad)
Commercial	C-1, C-1-D, E-1, M-1	2,600

gpad = gallons per acre per day

Residential flows were calculated utilizing standard zoning and housing densities published in the draft 2011 Buildable Lands Inventory (BLI) report. The City supplied GIS data for the 2011 BLI which already included calculations of the number of units per parcel allowed by approved zoning densities and an allocation of the projected number of units per parcel the City felt would be actualized. These assigned numbers were then multiplied by the household density (2.03 persons/home) and per capita flow rate (see Table 4.6) to calculate the total flow contributed by each residential growth parcel.

Future flows were also assigned patterns developed from flow observation data (see Appendix C). One pattern was selected for future residential flows (based on observed flows from a residential area) and one was selected for future commercial and industrial flows (based on observed flows from a commercial area), as shown in Charts 6.1 and 6.2.

CHART 6.1: Residential Unit Curve (Site 3 – Manhole 4BB-016)

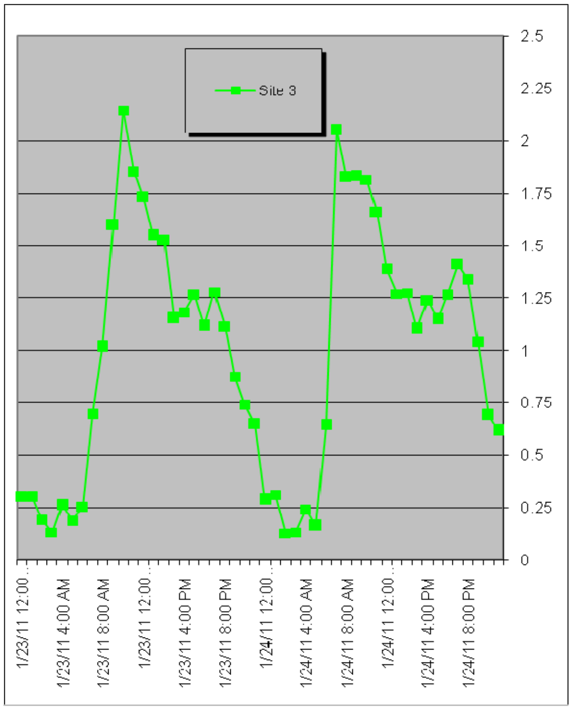
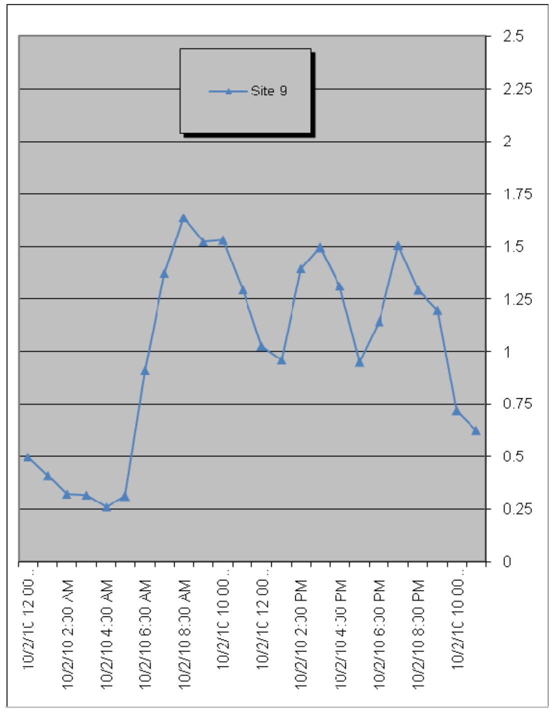


CHART 6.2: Commercial Unit Curve (Site 9 – Manhole 4CC-030)



Various model simulations were run to analyze the effects of future growth at complete infill of the City Limits (11-year horizon) and build-out of the Urban Growth Boundary (UGB, 21-year horizon). City-supplied GIS land use layers identifying areas of current service (existing zones) and anticipated growth (impact boundary) were used to calculate future flows and identify where pipeline extensions would be required.

6.2 FUTURE DEFICIENCIES

Modeling results show that the majority of pipelines with insufficient capacity for future growth flows were the same as those already identified as insufficient for current flow rates. Distributing the growth first to city limit infill areas did not result in any significant additional deficiencies. However, build-out of the urban growth boundary does result in several additional deficiencies. The additional future flows were considered in sizing of improvements required to address existing deficiencies. Appendix D contains printouts of capacity results for these scenarios. Specific discussion of each deficiency is included with the improvement descriptions in Chapter 8.

Remaining Capacity Summary

Table 6.2 summarizes the remaining capacities of key pipeline segments. The City should use this for general planning purposes to determine when future improvements will be required. This table should be updated from time to time to reflect additional information the City gathers through future flow monitoring efforts, possible future reductions in infiltration and inflow resulting from rehabilitation efforts, and as additional data is made available.

The basis for capacity analysis is the Equivalent Residential Unit (ERU). An ERU is estimated as the amount of flow expected to come from a typical single residence with an assumed number of persons per household. For this study, the assumed number of people per home was 2.03. This number was used along with the peak hour per capita value in Table 4.6 (350 gpcd) to calculate a design capacity for new growth of 0.49 gpm per ERU.

TABLE 6.2: Development Levels Triggering Improvements

Priority	Improvement/Location	Additional Upstream ERUs
1a	18-inch and 24-inch Parallel Trunkline Along Bear Creek	0
1b	Mountain Avenue Interceptor	0
1c	Oak Street Bottleneck	90
1d	A Street Interceptor	0
1e	Railroad Relief Interceptor	0
1f	Siskiyou Boulevard Bottleneck	0
---	Ashland Creek Lift Station Portable Pump	----[1]
2a	West Nevada Street Relief Interceptor	370
2b	Walker Avenue Relief Interceptor	510
2c	North Wightman Street Relief Interceptor	120
---	Upgrade Ashland Creek Lift Station Pumps	-----[1]

ERU = Equivalent Residential Unit (at Peak Hour flows 1gpm = 2.03 ERUs)

[1] Refer to Lift Station Capacity Table in Chapter 9.

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7.0 COLLECTION SYSTEM IMPROVEMENT ALTERNATIVES

This chapter discusses the various alternatives that were considered to address the existing and future deficiencies mentioned in Chapters 5 and 6. The recommended alternatives are discussed in Chapter 8.

7.1 IMPROVEMENT ALTERNATIVES

The alternative discussions below use the same labels as those listed in the Capital Improvement Plan. Improvements not discussed in this section were considered to have only a single solution, and are discussed in Chapter 8.

7.1.1 Priority 1 Alternatives – Address Existing Deficiencies

Bear Creek Trunklines – Priority 1a

The City has implemented several projects creating segments of parallel trunklines along the south bank of Bear Creek, extending from the Ashland Creek Lift Station to N. Wightman Street. However, there are several sections where the parallel 15-inch and 24-inch lines neck down to a single 15-inch or 24-inch pipeline. The 15-inch sections are surcharged at current flows, and the 24-inch sections may become surcharged under certain future flow events. In keeping with the City's precedent, it is recommended that parallel lines be installed in these sections to provide a continuous backbone of parallel trunklines.

Two alternatives exist to address the existing surcharging projected to occur to the east of Wightman Street during peak flow conditions:

<u>Alternative 1 – Parallel Lines</u>
Between N. Wightman Street and N. Walker Avenue a 24-inch pipeline could be installed parallel to the existing 12-inch line, to relieve current surcharging. To the east of N. Walker Avenue, an 18-inch pipeline could be installed parallel to the existing 12-inch line, to relieve current surcharging in this section. Parallel lines provide a significant increase in total capacity at a reduced cost compared to a single larger pipeline. Installing a parallel line also allows continued service during construction. The final configuration of the Bear Creek Parallel Trunklines would follow a 24-15, 24-12, 18-12 sizing scheme from the Ashland Creek Lift Station to I-5. Parallel trunklines also allow increased flexibility for future improvements. When dual capacity is exceeded or pipe conditions dictate replacement, the smaller or older pipeline can be replaced with a larger pipe.

<u>Alternative 2 – Single Upsized Pipeline</u>
Between N. Wightman Street and N. Walker Avenue, the existing 12-inch pipeline could be replaced with a single 30-inch pipe to sufficiently convey projected future flows from build-out of the UGB area. Demolition, removal, and bypass pumping costs would all be involved for this alternative. Keller Associates would only recommend this alternative if the condition of the existing 12-inch line was such that it would need to be replaced or rehabilitated within the 20-year planning period. The condition of the line should be assessed as part of the pre-design.

Diversion 3/ Mountain Avenue Improvements – Priority 1b

Current flows are surcharging the existing 10-inch pipeline along N. Mountain Avenue immediately upstream of the Bear Creek Trunklines. A diversion manhole is located directly upstream of this section, which provides several alternatives for improvements in this area of the system.

Alternative 1 – 15-inch Replacement
The first alternative is replacement of this line with a larger 15-inch pipeline at an increased slope. The slope can be adjusted 10 inches, which would eliminate surcharging from future buildout flows.

Alternative 2 – Diversion to East
Due to the shallow slope of the surcharged pipe, 75% flow diversion to the east at Diversion 3 is required to prevent surcharging under projected future peak flows. The diverted flows subsequently surcharge multiple existing 6, 8, and 10-inch sections between N. Mountain Avenue and Oak Street. These sections would need to be replaced with 10-inch and 12-inch pipelines.

Diversion 4/ A Street Improvements – Priority 1d

Current flows are surcharging the existing 12-inch pipeline along A Street. A diversion manhole located directly upstream of this section on 7th Street provides several alternatives for improvements in this area of the system.

Alternative 1 – 15-inch A Street Interceptor
The first alternative is replacement of this line with a larger 15-inch pipeline. The larger diameter pipe will accommodate projected future flows even with the upstream diversion sending 100% of flows north to this pipeline. During pre-design of this alternative, pipe bursting should be evaluated as a trenchless construction technique that could minimize traffic disruption and potentially lower construction costs.

Alternative 2 – Divert 50% of Flows
Forcing diversion of a portion of the flows west toward N. Mountain Drive was investigated. Even at a diversion of 50%, the line remains surcharged at peak flows. This alternative could be considered as a short-term solution for current flows, but is inadequate at projected future peak flows. The downstream pipes begin to reach capacity after the 50% diversion is implemented. Several sections of 6-inch pipeline along Williamson Way would need to be upsized to 10-inch to accommodate the flow from the buildout of the UGB.

Alternative 3 – 100% Diversion
Forcing diversion of 100% of flow entering Diversion #4 effectively relieves all current and future surcharging in the A Street pipeline. However, there are significant impacts to downstream pipelines along N. Mountain Drive. The entire downstream 10-inch pipeline along Mountain Avenue would need to be replaced with a 15-inch pipeline, which would significantly increase the cost of this option.

7.1.2 Priority 2 Alternatives – Address Future Deficiencies

West Nevada Street Relief Interceptor – Priority 2a

Improvements in this area of the system target three objectives: reducing surcharging of pipes, providing sufficient pumping capacities at the Ashland Creek Lift Station, and promoting gravity flow over pumping of wastewater (reduced power costs).

Alternative 1 – Interceptor/New Diversion
Invert elevations near the west end of Nevada Street are sufficient to allow intercepting flow at manhole BRS-08 and redirecting it to manhole 5AD-010. Investigations in this area by Keller Associates and City staff indicate an old connection in this area was abandoned and has since been built over by a new subdivision and new pipelines. The proposed 12-inch

interceptor would follow Nevada Street and intercept all flow in this subdivision. Downstream improvements are also needed to upsize an existing 8-inch pipeline along Nevada Street. This alternative would effectively address surcharging occurring in the 12-inch pipeline west of the Ashland Creek Lift Station and reduce the flow entering the lift station, thereby extending the capacity life of the pumps.

<u>Alternative 2 – Increase Pumping Capacities & Upsize Trunkline</u>
Future projected flows result in surcharging in several sections of the 12-inch pipeline west of the Ashland Creek Lift Station. These sections would need to be replaced with a 15-inch pipeline. The pumping capacity of the lift station would also need to be increased to accommodate the increased flows. A significantly greater length of pipe would need to be installed for this alternative to relieve surcharging in the existing line. This alternative would cost more than alternative 1 because more pipe would need to be installed. This alternative also has a greater potential for environmental issues than Alternative 1. Since this alternative will convey more flow in the pipelines along the creek, if a break in a pipe were to occur, the risk of the creek and surrounding wetlands becoming contaminated is relatively high.

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8.0 COLLECTION SYSTEM IMPROVEMENT PLAN

This chapter discusses the Capital Improvement Plan (CIP) based on the recommended improvements to the conveyance system. It outlines the recommended improvements, the capital improvement schedule, planning level costs, and other recommendations for implementation. (It should be noted that these improvements are limited to the City's trunklines, and that a more comprehensive list of improvements could be developed if every pipeline in the system was modeled and evaluated.)

8.1 RECOMMENDED CAPITAL IMPROVEMENTS

The following sections outline the recommended improvements necessary to resolve existing and future deficiencies identified with the model. Figure 8.1 in Appendix A illustrates the locations and phasing of each improvement as discussed below.

Concurrent with pursuing Priority 1 improvements, Keller Associates recommends that the City actively seek to reduce infiltration and inflow within the collection system. The Department of Environmental Quality has indicated they may require smoke testing of the City's system before committing state revolving loan funds to treatment plant improvements, in order to determine the cost-effectiveness of eliminating known sources of inflow vs. plant improvements.

8.1.1 Priority 1 – Address Existing Deficiencies

18-inch and 24-inch Parallel Trunkline Along Bear Creek – Priority 1a

Keller Associates recommends that the City install 18-inch and 24-inch trunklines to parallel the existing 12-inch and 15-inch pipeline sections along Bear Creek. Completion of this line is a high priority, as the current 12-inch and 15-inch pipeline is surcharged along the majority of the length during peak hour conditions. The 18-inch pipeline will extend approximately 4000 feet from manhole 11BC-006 to manhole 10AB-004. The 24-inch parallel line will need to be constructed at two separate locations along Bear Creek. The first section is approximately 1700 feet from manhole 10AB-004 to manhole 10BA-004, and the second section is approximately 800 feet from manhole 4DD-027 to manhole 4DD-008. These improvements will be capable of conveying the entire upstream projected build-out wastewater flows. The proposed grade of these trunklines is slightly greater than the minimum slopes of the existing 12-inch and 15-inch pipeline within this reach.

Mountain Avenue Interceptor – Priority 1b

The current 10-inch pipeline is surcharging and should be replaced by a 15-inch pipeline with a steeper slope. The existing topography allows for only a slight slope adjustment (approximately 10 inches on the upstream side of the pipe), but the 15-inch pipe at the adjusted slope will be able to convey projected build-out flows.

Oak Street Bottleneck – Priority 1c

This improvement involves the installation of a 24-inch pipe along Oak Street. Currently a 15-inch diameter section pipe is installed between two 24-inch diameter pipes which is creating a bottleneck in the flow. This section of pipe should be investigated because it appears the pipes were constructed at the same time, and the 15-inch pipe could have been mislabeled. The new pipe will be able to convey all future build-out flows.

A Street Interceptor – Priority 1d

This improvement consists of replacing the existing 12-inch pipeline with a 15-inch pipeline along A Street from manhole 9AB-015 to manhole 9BA-011. The pipeline can be installed at the same slope as the current pipeline. Future flow can be diverted through this line at diversion 4 (manhole 9AC-041) to relieve surcharging in other downstream lines to the north on Mountain Avenue. An alternative to excavating and installing the new pipeline would be pipe bursting, since the current pipeline is at the correct slope to convey future flows.

Railroad Relief Interceptor – Priority 1e

The existing 8-inch line is not large enough to convey current peak flows. To accommodate current and future flows, the pipeline needs to be upsized to a 12-inch pipeline. The existing line is at an adequate slope; thus, pipe bursting should be considered as an alternative to open trench installation.

Siskiyou Boulevard Bottleneck – Priority 1f

A section of pipeline at the intersection of Siskiyou Boulevard and Wightman Street is undersized and not at an adequate slope. The existing pipeline is creating a bottleneck in the line. To correct this, a 12-inch pipeline at minimum slope will accommodate projected future flows.

Purchase Portable Trash Pump (Ashland Creek Lift Station) – Priority 1g

As discussed in Chapters 3 and 9, it is recommended the City purchase a portable trash pump with enough capacity for use as a backup during a peak event. In addition to emergency redundancy, use of a trash pump will effectively provide the City a buffer period during which increasing peak flows can be monitored and the appropriate size of new pumps can be determined based on actual conditions.

Miscellaneous Lift Station Upgrades

Keller Associates recommends that the City complete the lift station upgrades outlined in Chapter 3 of this report.

8.1.2 Priority 2 – Address Future Deficiencies

West Nevada Street Relief Interceptor – Priority 2a

This improvement consists of installing a new 12-inch pipeline on West Nevada Street. As the City begins to build out to the northwest along Highway 99, the flow can be rerouted directly to the wastewater treatment plant instead of flowing to the Ashland Creek pump station to be pumped to the plant.

Walker Avenue Relief Interceptor – Priority 2b

This improvement includes adjusting the slope of a section of pipe near the intersection of Walker Avenue and Main Street. The section of pipe is essentially flat, and surcharging will occur as flows increase. The topography at the location is suitable to make the necessary slope change to allow for flows to be effectively conveyed.

North Wightman Street Relief Interceptor – Priority 2c

This improvement includes the installation of a 12-inch pipeline that would replace the existing 8-inch pipeline. The new pipeline can be constructed at the same slope as the existing line, and would be sufficient to convey build-out flows.

Ashland Creek Lift Station Upgrade – Priority 2d

This improvement consists of upgrading the pumping capacity at the lift station. At peak day build-out flows the wastewater in the wet well reaches an elevation that surcharges the lines coming into the lift station. As the pumps are replaced in the future they should be sized accordingly to eliminate the surcharging issues. The 18-inch pipeline is adequately sized to convey the increased flow. Sizing of the pumps should take into consideration the reduction in flows due to implementation of Priority 2a. Monitoring of actual flows after Priority 2a construction and prior to lift station redesign is recommended.

Miscellaneous Lift Station Upgrades

Keller Associates recommends that the City complete the lift station upgrades outlined in Chapter 3 of this report.

8.1.3 Future Pipelines and Lift Stations

As the city builds out to the northwest along the I-5 corridor, a new 12-inch trunkline may be required to convey the flow. Also a future lift station is proposed to pump the flow back to West Nevada Street where it can gravity flow to the wastewater treatment plant. Discussions with Rogue Valley Sewer (RVS) revealed a portion of this area is already serviced by RVS collection lines and two lift stations. Expansion of the City's system into this area of the UGB should consider location and sizing of existing components and must be coordinated with RVS.

Another area south of Main Street has been designated as developable land for Southern Oregon University (SOU). To convey the expected future flows into the system, a 12-inch pipeline will need to be installed. The last area of expected growth is to the southeast of the City along Highway 66. The existing 10-inch pipeline can be extended along the highway as the City builds out the UGB. Figure 8.1 shows the proposed future pipelines and lift stations needed to service the UGB.

8.2 OPERATIONAL & MAINTENANCE IMPROVEMENTS

Many of the operational and maintenance improvements were identified in Chapter 3 of this report (refer to Section 3.4 of this report for recommendations pertaining to lift station design standards, lift station SCADA upgrades, CCTV monitoring, maintenance management, and pipeline replacement/rehabilitation). This section focuses on infiltration and inflow reduction efforts and collection system staffing.

8.2.1 Infiltration and Inflow Reduction Program

Keller Associates recommends that, within the next 1-2 years, the City complete smoke testing of the City's collection system. Observed storm water cross connections should be identified and removed.

In addition to inflow reduction, Keller Associates recommends that the City establish an active infiltration reduction program. Implementing an active program may result in flow reductions to the treatment plant and will be important to prevent future increases in flow resulting from infiltration and inflow. This will build upon current TV monitoring efforts and should include activities such as night-time monitoring, post-storm flow monitoring, and continuous monitoring within the collection system. The City should budget approximately **$60,000 to acquire portable flow meters** that can be used in these efforts.

8.2.2 Collection System Staffing

Proper maintenance of a wastewater collection system will maintain the capacity, reliability and functionality of the system for conveying wastewater to the treatment plant. Adequate staff must be available for activities such as cleaning and inspecting sewers, finding problem areas, repairing and replacing failing components, maintaining pump station mechanical and electrical equipment, monitoring pump run time and flows, and responding to customer complaints. Due to the variability of collection systems, universal standards for collection system O&M are not feasible. Population served and number of connections, service area size, length of sewer, pipe age and condition, number and size of pump stations, criticality of the station, and reliance on SCADA are all factors that influence the number of personnel required to effectively run the collection system.

Collection system maintenance includes pipelines, manholes and lift stations. Maintenance may be corrective (reacting to a failure), preventive (programmed systematic approach), or predictive (scheduling maintenance activities based on observed changes in performance). Maintenance of equipment such as pumps needs to be carried out on a regular basis in accordance with manufacturer's recommended schedules (typically based on operating hours and/or months in service). The frequency of pump station inspections should be based on the criticality of the pump station. Benchmark data from 13 agencies [1] shows pump station inspection frequencies ranging from daily to monthly, with the majority inspecting their stations at least once a week.

Benchmark data from those 13 agencies indicates the percentage of their system cleaned annually varies widely (from 7 to 82%), with an average of 35%. These same agencies inspect 0 to 24% (average about 6%) of their system by CCTV annually. The Public Works Director's goals for the City of Ashland are to annually rod and clean 400,000 feet (about 69% of the system), CCTV 100,000 feet (about 17%), smoke test 5,000 feet, and apply chemical root control to 15,000 feet [2]. Also included in the projected annual work are 50 sewer pipe repairs, 10 new manholes, 10 manhole replacements, and 10 manhole repairs.

Based on typical crews and production rates for various maintenance activities, meeting Ashland's maintenance goals would require an equivalent full time staff of 6 people dedicated to the collection system. The City of Ashland should consider appointing or hiring a collection system lead or superintendent to manage this staff and to oversee the collection system operations, maintenance, and replacement activities. With the hiring of a collection system supervisor (or lead foreman), the City should also consider shifting operation and maintenance responsibilities for the lift stations from the treatment plant staff to collection system staff. For the City of Ashland, the organization structure could look as follows:

8.3 ENVIRONMENTAL CONSIDERATIONS

Most of the improvements consist of replacement of existing pipelines in their current location, which should have minimal environmental impacts since the ground has previously been disturbed. The most potentially environmentally sensitive priority project would be construction of the Priority 1a project consisting of 18- and 24-inch trunklines paralleling the existing 12-inch and 15-inch pipeline sections along Bear Creek. Based on the City's wetlands inventory [3], the proposed 24-inch section closest to N. Mountain Ave. would be near a Locally Significant Wetland (W7). The pipeline would be routed to avoid encroaching on wetlands, and use of BMPs would be required to prevent adverse impacts to the creek water quality. Some temporary impacts would be possible during construction, but no long-term impacts are anticipated.

References

1. California State University - Sacramento, College of Engineering and Computer Science, Office of Water Programs: *Collection Systems: Methods for Evaluating and Improving Performance*, *2nd ed,* 2010.

2. Nov. 10, 2010 email from Scott Fleury, City of Ashland to James Bledsoe, Keller Associates.

3. SWCA Environmental Consultants, *City of Ashland Local Wetlands Inventory and Assessment & Riparian Corridor Inventory,* rev. February 2007.

9.0 WASTEWATER TREATMENT PLANT - EXISTING CONDITIONS & CAPACITY EVALUATIONS

This chapter provides background technical information on the City of Ashland's Wastewater Treatment Plant (WWTP) including structures, controls, unit processes, and miscellaneous facilities. Plant operations and maintenance recommendations are also included. Growth impacts are applied to determine future treatment plant deficiencies, and both existing and future deficiencies are summarized.

9.1 PROCESS DESCRIPTION

Influent wastewater flows through the following treatment processes before being discharged into Ashland Creek:

- Preliminary treatment (screening and grit removal)
- Biological treatment process
  - Oxidation ditches
  - Secondary clarification
  - Return activated sludge (RAS) system
- Disinfection (UV)
- Post aeration
- Alum addition and a tertiary membrane system are operated from May 1st to November 30th, to aid in meeting a seasonal phosphorus limit

Solids handling processes include the following treatment and disposal operations:

- Waste activated sludge (WAS) system
- Lime stabilization/holding tank
- Biosolids handling and disposal at landfill

The flow schematic of the existing WWTP processes is shown in Figure 9.1 (Appendix A). The flow schematic represents the current operation configuration of the plant.

The projected flows as shown in Table 9.1 were used to evaluate hydraulic capacity of the treatment system. The hydraulic profile for the existing plant is presented in Figure 9.2 (Appendix A).

TABLE 9.1: Design Flows

	Year 2010	Year 2015	Year 2030	Year 2060
Avg Dry Weather, mgd	2.1	2.19	2.47	3.04
Max Month Dry Weather, mgd	2.7	2.82	3.18	3.90
Avg Wet Weather, mgd	2.3	2.40	2.71	3.32
Max Month Wet Weather, mgd	3.6	3.76	4.24	5.20
Peak Week, mgd	5.0	5.14	5.56	6.40
Peak Day, mgd	7.1	7.33	8.03	9.44
Peak Hour, mgd	10.5	10.83	11.81	13.77

9.2 NPDES PERMIT COMPLIANCE

Current and anticipated permit limits and TMDL requirements are discussed in Chapter 2. A summary of the plant compliance with existing permit limits from January 2004 to December 2010 is listed by constituent in the following sections. The capability of the plant to continue meeting the permit limits with increased flows is also analyzed.

9.2.1 $CBOD_5$

The lowest $CBOD_5$ effluent limits apply from September through October: 4 mg/L and 77 ppd monthly average, and 5 mg/L weekly average. Analysis of 2004-2010 data for the September/October period is shown in the following table (daily data was not analyzed since there are no daily limits during this period).

September 1 to October 31 Statistics (2004 – 2010)

	Weekly		Monthly	
	mg/L	ppd	mg/L	ppd
Maximum	1.53	27.0	1.2	24.0
Average	1.04	17.66	1.03	17.59
Median	1.00	17.0	1.02	17.28
95% conf., mg/L	0.78 – 1.3		0.85 – 1.22	
95% conf., ppd	12.09 – 23.24		13.26 – 21.92	

To meet the 77 ppd monthly average limit at the 2015, 2030, and 2060 maximum month flows of 3.76 mgd, 4.24 mgd, and 5.20 mgd, monthly effluent CBOD would have to average 2.45 mg/L, 2.17 mg/L, and 1.77 mg/L, respectively. These required effluent values are within the 95% confidence interval for effluent CBOD shown in the following table. Thus, as long as the operating parameters for the oxidation ditches (loading, MLSS, RAS return rate, and MCRT) remain in the range observed over the past seven years and the membrane tertiary filters are utilized from May 1 to November 30, the existing plant technology should be able to meet the *existing* effluent limits for CBOD through 2060 by adding capacity as necessary.

9.2.2 TSS

The lowest TSS effluent limits apply from May through November, when the limits are 10 mg/L as a monthly average, 15 mg/L as a weekly average, 180 ppd as a weekly average, 96 ppd as a monthly average, and 480 ppd as a daily maximum. Analysis of the 2004-2010 plant data for the May - November period shows the following:

May 1 to November 30 Statistics (2004 – 2010)

	Weekly		Monthly	
	mg/L	ppd	mg/L	ppd
Maximum	8.00	108.0	6.40	63.60
Average	<1.88	31.26	<1.90	31.12
Median	<2.00	33.33	<2.00	33.66
95% conf., mg/L	0.00 – <3.87		0.01 – <3.80	
95% conf., ppd	1.06 – 61.46		5.15 – 57.09	

Most of the results for TSS in the May - November period (when the membrane filters are in operation) are non-detectable, or less than (<) 2 mg/L. To meet the 96 ppd monthly average limit at 2015, 2030, and 2060 maximum month flows of 3.76 mgd, 4.24 mgd, and 5.20 mgd, monthly effluent TSS would have to average 3.06 mg/L, 2.71 mg/L, and 2.21 mg/L, respectively. Thus, as long as the operating parameters for the oxidation ditches (loading, MLSS, RAS return rate, and MCTR) remain in the range observed over the past seven years and the membrane tertiary filters are utilized from May 1 to November 30, the existing plant technology should be able to meet the *existing* effluent limits for TSS through 2060 by adding capacity as necessary.

9.2.3 Ammonia

Six violations of the effluent ammonia limits occurred during 2004-2010 as follows:

Monthly Average Effluent Ammonia Concentration

Date	Effluent	Limit
June 2004	0.83 mg/L	0.52 mg/L
February 2004	0.90 mg/L	0.80 mg/L
March 2004	0.90 mg/L	0.80 mg/L

Daily Maximum Effluent Ammonia Concentration

Date	Effluent	Limit
June 16, 2004	1.90 mg/L	1.2 mg/L
June 20, 2004	1.74 mg/L	1.2 mg/L
June 22, 2004	1.79 mg/L	1.2 mg/L

All of these violations occurred in early 2004, and the City has since modified operations to maintain the plant in compliance with the ammonia limits.

Analysis of the 2004-2010 plant data shows the following (weekly data is not provided since there is not a weekly limit):

Annual Statistics 2004 through 2010

	Daily mg/L	Monthly mg/L
Maximum	1.9	0.9
Average	0.2	0.2
Median	0.1	0.2
95% conf., mg/L	0.0 – 0.8	0.0 – 0.6

The lowest ammonia effluent limits occur from May through November, when the limits are 0.52 mg/L monthly average and 1.2 mg/L daily average. The ammonia effluent limits from December through April are 0.80 mg/L monthly average and 1.8 mg/L daily average. Analysis of the 2004-2010 plant data for the May - November and December - April periods shows the following:

May 1 to November 30 Statistics (2004 – 2010)

	Daily mg/L	Monthly mg/L
Maximum	1.90	0.83
Average	0.21	0.21
Median	0.13	0.17
95% conf., mg/L	0.0 – 0.69	0.0 – 0.50

December 1 to April 30 Statistics (2004 – 2010)

	Daily mg/L	Monthly mg/L
Maximum	1.4	0.94
Average	0.31	0.29
Median	0.18	0.20
95% conf., mg/L	0.0 – 0.91	0.00 – 0.72

The required effluent values are above the 95% confidence interval for effluent ammonia. Thus as long as the operating parameters for the oxidation ditches (loading, MLSS, RAS return rate, and MCRT) remain in the range observed over the past seven years, the existing plant technology should be able to meet the *existing* effluent limits for ammonia through 2060 by adding capacity as necessary.

If the ammonia limits discussed in Chapter 2 are adopted in the future and freshwater mussels are found in Ashland and/or Bear Creek, then a chronic ammonia limit would be added to the permit. The limit would be variable, decreasing with increasing temperature and pH. Thus the worst case limit would be during the summer. For the summer months the 95% confidence range for temperature is 14.16^{o}C to 25.74^{o}C, and for pH is 7.07 to 8.16. Based on the ammonia guidance document referenced in Chapter 2, the chronic ammonia limit with mussels present at these conditions would range from 0.218 to 1.22 mg/L NH_4-N/L. The plant may start having problems meeting this limit consistently, particularly when the temperature rises above 18^{o}C when the pH is above 7.5. The City could cool the effluent, lower the pH, or both, to meet the possible future ammonia limits referenced in Chapter 2. However, it should also be noted that new ammonia standards may take 10+ years before being implemented on a state level and it is unlikely that they would be made a part of Ashland's NPDES permit for many years to come.

9.2.4 Phosphorus

Three violations of the effluent monthly average phosphorus load limit and one violation of the effluent maximum day phosphorus load limit occurred during 2004-2010.

Monthly Average Effluent Phosphorus Loading

Date	Effluent	Limit
October 2005	1.7 ppd	1.6 ppd
August 2006	2.2 ppd	1.6 ppd
September 2007	2.0 ppd	1.6 ppd

Daily Maximum Effluent Phosphorus Loading

Date	Effluent	Limit
September 9, 2007	9.9 ppd	5.1 ppd

Analysis of the 2004-2010 plant data for May through November shows the following:

May 1 to November 30 Statistics (2004 – 2010)

	Daily ppd	Daily mg/L	Monthly ppd
Maximum	9.86	0.60	2.18
Average	1.07	<0.06	1.07
Median	0.89	<0.05	0.99
95% conf., ppd	0 – 2.33		0.47 – 1.67
95% conf., mg/L		0 – 0.13	

The allowed effluent loading values are above the 95% confidence interval for daily effluent phosphorus and near the upper range for monthly effluent phosphorus. To meet the 1.6 ppd monthly limit at the 2015, 2030, and 2060 maximum month flows of 3.76 mgd, 4.24 mgd, and 5.20 mgd, monthly effluent phosphorus concentrations would have to average 0.051 mg/L, 0.045 mg/L, and 0.037 mg/L, respectively. The effluent phosphorus is typically <0.05 mg/L (detection level) at current flows. Currently the alum does is fed at a constant rate. As the flow increases, the chemical dosage will have to be increased to maintain removal. Plant staff previously attempted flow pacing to automatically maintain the alum dosage with varying wastewater flows. The metering pumps shut down when low flows occurred. The City will also need to find a laboratory that can determine the effluent concentration to detection levels below 0.05 mg/L. If flow pacing were attempted in the future, these issues would need to be addressed.

As long as the alum dose is maintained at a sufficient level and the membranes are maintained in good operating condition, the existing plant technology can meet an effluent limit of 0.05 mg/L. By 2030, the concentration to meet the loading limit is less than 0.05 mg/L; reducing phosphorus to this level is stretching the limits of technology. Strategies for dealing with this challenge may include using reuse as a component to help meet the phosphorus limit, trading, or requesting a technology based limit for future permit cycles.

It should also be mentioned that Oregon DEQ has been issuing new permits that measure compliance in terms of monthly medium rather than monthly average values. This has the effect of lessening the impact of an unusual daily spike and is a permit modification the City should seek in its next permit.

9.2.5 E. coli

The E. coli effluent limits year-round are 126 MPN/100 mL as a monthly geometric mean and 406 MPN/100 mL as a single sample maximum. According to City staff, there were no violations of the limits in the 2004-1010 period. However, all data in the spreadsheets provided by the City (including two single sample results above 406) were included in the analysis below.

Analysis of the 2004-2010 plant data shows the following:

Annual Statistics 2004 through 2010

	Daily ppd	Monthly ppd
Maximum	800	61.2
Average	14.3	8.5
Median	2.0	2.1
95% conf., mg/L	0 – 96.1	0.0 – 33.9

The required effluent values fall within the 95% confidence interval for effluent E. coli. Assuming the operating parameters for the UV system remain in the range observed over the past seven years, the existing plant technology can meet the existing effluent limits for E. coli through 2060.

9.2.6 Excess Thermal Load

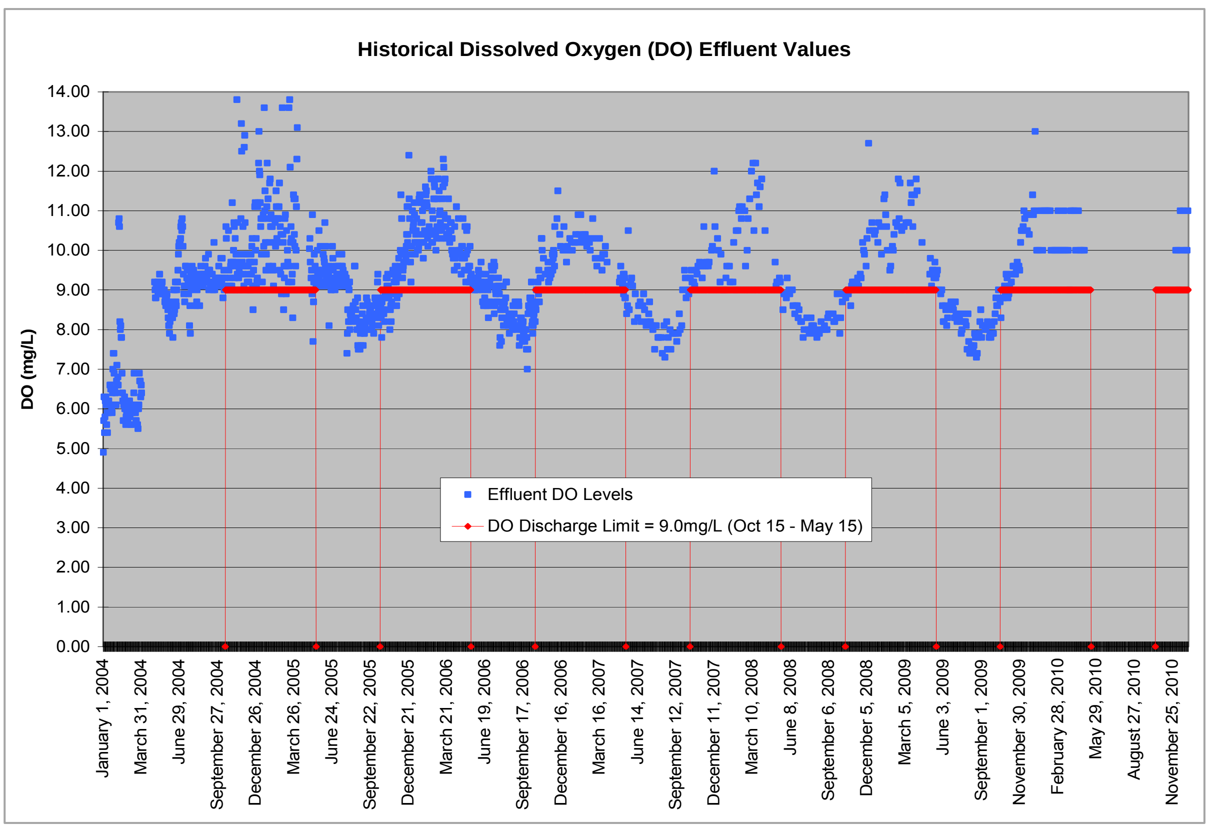
In the 2004-2010 period, there was 1 violation of the October 15 to March 15 excess thermal load limit (78 million kilocalories per day), and 35 violations of the March 16 to October 14 excess thermal load limit (38 million kilocalories per day).
The new NPDES permit is expected to have effluent temperature requirements which are more fully discussed in Chapter 10 of this report.

9.2.7 Dissolved Oxygen

From 2004-2010, there were 53 violations of the October 15 to May 15 minimum dissolved oxygen (DO) effluent limit of 9.0 mg/L. The minimum value of dissolved oxygen of the 53 violations was 7.7 mg/L. The violations tend to occur in late October and early November when the limit has just gone into effect, as shown in Chart 9.1. The saturation point of water is 9 mg/L at approximately 17.2 degrees C. Since the effluent temperature is typically higher than 17.2 in October (as high as 22.1 degrees C in October 2004), super saturation would be required in order for the Ashland WWTP to meet the 9.0 mg/L DO limit in October and perhaps in May.

It is recommended that consideration be given to revisiting (with DEQ) the effluent limits for DO during these shoulder seasons, particularly since the outfall conditions may change with the discharge to Bear Creek. If the current limit is to remain, supersaturation can be achieved by adding pure oxygen or through a pump air injection system.

CHART 9.1: Historical Dissolved Oxygen (DO) Values



9.2.8 Summary of NPDES Permit Limits Compliance

For CBOD, TSS, ammonia, phosphorus, and E. coli, the existing treatment plant technology should be able to meet the current limits in the future as long as treatment units are operating within the existing design criteria. As flow increases, additional components (e.g. oxidation ditch, secondary clarifier, UV unit, membranes, or alum pump) may be needed to maintain the design criteria and continue meeting the effluent limits. The capacity of the existing components and when modifications might be needed for increased flows is discussed in the rest of this chapter.

Though a major change in the type of treatment plant is not required for the parameters listed above, the existing treatment plant cannot meet the new expected temperature limits in the upcoming NPDES permit. Thus, new components will need to be added to the existing plant or alternative disposal methods used in order to meet the expected limits. Alternatives to address the future temperature limits are discussed in Chapter 11.

9.3 PLANT CAPACITY

An evaluation of each process within the plant was conducted. The following sections provide a summary of that evaluation and include a description of the process, the hydraulic capacity, treatment capacity, and condition of the structure or equipment.

9.3.1 Influent Pumping – Ashland Creek Lift Station

Raw sewage enters the treatment plant via two gravity sewers and a force main from the Ashland Creek Pump Station, located on the northeast corner of the plant site. Based on records provided by the City, it is estimated that 63% of the total flow from the collection system is pumped into the plant. DEQ design standards [1] require that the pump station be capable of conveying the 5-year 24-hour storm peak hourly flow with the largest pump out of service.

Table 9.2 compares the pump capacity to pumped flows (based on 63% of total plant flow) for current and year 2030 conditions. Based on the referenced DEQ requirements, the existing pumps appear to supply sufficient capacity for current and future conditions. However, due to the disproportionate amount of total system flows handled by this lift station, extra precautions for redundancy and emergency operation are recommended. An option the City has used in the past to meet redundancy requirements is ensuring that a portable pump is available during periods when one pump is out of service.

TABLE 9.2: Influent Pump Station – Peak Hour Flows

	No. Pumps Operating	Pumping Capacity	Year 2010 Flow Total Inf. / Pumped	Year 2030 Flow Total Inf. / Pumped
Peak Hour	2	8.06mgd	10.5 / 6.6 mgd	11.8 / 7.4 mgd

9.3.2 Headworks

The existing headworks facility includes a 14' square detritus tank grit basin and a reciprocating rake type screen. The grit and screen system each have an odor control system consisting of a blower and a carbon canister.

Capacity

Based on an overflow rate of 46,300 gpd/square foot (providing removal of all particles larger than 0.21 mm), the grit basin has a peak capacity of 9.1 mgd. Though this capacity will handle peak *daily* flows through 2030, the higher velocities experienced at peak hourly flows will sweep larger particles out of the tank with the result that less grit will be removed. At 12 mgd, the minimum size of particles removed would increase to about 0.3 mm. Given the infrequency of flows over 9.1 mgd, Keller Associates recommends upgrading or replacing the grit system at the end of its useful life (20-30 years). (The existing mechanism was installed in 1998. The condition should be evaluated after 20 years to determine when to replace the grit equipment.)

The mechanical screen is a reciprocating rake type, has 0.5" openings, and a capacity of 13.5 mgd. The redundant screen is a manual screen with 1.5" openings. There is sufficient capacity for projected peak hourly flows until past 2050. The City may want to consider adding a second mechanical screen for redundancy.

Condition

The grit, screen, and odor control systems have been well maintained and are in good condition. Both systems were installed in 1998 and are thus 13 years old. Both systems will thus require continued good maintenance and parts replacement to remain in good operating order. The carbon in the carbon canister should be changed out periodically to prevent breakthrough of odors.

Controls

The grit system, screen and odor system blowers are controlled by the SCADA system. The grit system and odor blowers operate continuously. The screen operates based on level in the channel. When the water level increases to a set point the reciprocating rake operates to clean the screen. The washer compactor operates whenever the rake is operating. A magnetic flow meter located prior to the oxidation ditches measures the screened flow and provides the reading to the SCADA system.

9.3.3 Biological Process

The biological process at the WWTP consists of the oxidation ditches, anoxic cells, aeration equipment, secondary clarifiers, and return activated sludge (RAS) pumping system. These components are discussed in the following paragraphs.

Oxidation Ditches

The oxidation ditch process is a variation of the activated sludge process which uses aeration of microorganisms in the form of mixed liquor suspended solids (MLSS) to remove BOD, TSS, and nutrients (ammonia and nitrate) from the influent wastewater. The WWTP's oxidation ditches are Carrousel units, which utilize vertical shaft surface aerators to aerate the mixed liquor and mix the wastewater in the basins. The oxidation ditch configuration at Ashland also includes an anoxic cell which provides denitrification to remove nitrate and reduce the nitrogen content of the effluent.

The influent BOD_5 and TSS loadings to the oxidation ditches effectively establish the required process capacity. The size of the ditches must be adequate to provide the required detention time for BOD oxidation and assimilation as well as for oxidation of ammonia to nitrate. The biological treatment process requires a sufficient mass of organisms (MLSS) together with adequate detention time and oxygen supply to accomplish the desired level of treatment.

The design and operating criteria that were used to evaluate the effective capacity of the oxidation ditches include mean cell residence time, mixed liquor suspended solids, denitrification, and aeration.

Mean Cell Residence Time (MCRT)
The operating MCRT in the oxidation ditches is a critical parameter in terms of treatment. MCRT is defined as the mass of MLSS in the oxidation ditches divided by the mass of sludge wasted daily from the system. The MCRT (MLSS inventory) must be adequate to produce good sludge quality in terms of settling and effluent TSS, as well as to achieve essentially complete nitrification.

The oxidation ditches at Ashland are designed to oxidize ammonia to nitrate (nitrification) to meet the NPDES permit limits. Because the nitrifying organisms (autotrophs) grow more slowly than the organisms that remove BOD (heterotrophs), the solids retention time (MCRT) in the system must be adequate to allow the nitrifying organisms to grow and not be washed out. The critical parameter in assessing the operation of the oxidation ditches is maintaining an adequate MCRT to achieve nitrification.

The required MCRT for nitrification varies with temperature. Longer MCRTs are needed with colder temperatures. For Ashland, the limiting condition was determined to be a wintertime temperature of 12.5 deg C. At this temperature, it was estimated that an aerobic MCRT of 14 days was required (aeration zone only). Under "average" conditions with a temperature of 18.5 deg C, it was assumed that a MCRT of 12 days would be used to maintain sludge quality (although it would be possible to decrease the aerobic MCRT to a little as 7 days and still maintain full nitrification).

For Ashland, it appears that operation during the winter months is the most critical period. In looking at the historical BOD data, maximum loadings appear to have occurred any time of the year. Therefore, in this analysis, it was assumed that maximum month BOD_5 and TSS could occur during minimum temperature conditions. On this basis, the oxidation ditch capacity is determined by the ability to maintain the desired aerobic MCRT during winter maximum month loading conditions.

Mixed Liquor Suspended Solids (MLSS)
In evaluating the biological process capacity, the MLSS concentration in the oxidation ditches was adjusted to maintain the desired MCRT under the various flow and loading scenarios. As will be discussed in more detail later, limiting the MLSS inventory to that required for nitrification during the winter effectively maximizes the capacity of the secondary clarifiers from a solids loading standpoint.

Denitrification
As part of the capacity evaluation, the ability of the biological process to remove nitrate (convert nitrate to nitrogen gas) was evaluated. While nitrate removal is not required to meet the current NPDES Permit requirements, the use of an anoxic zone for denitrification promotes better sludge quality, reduces oxygen demand, and returns alkalinity. At Ashland, the anoxic zone is about 20 percent of the total oxidation ditch volume. Based on the analysis conducted, it does not appear that the denitrification capacity is limiting, and the system should be able to produce an effluent containing 5 mg/l nitrate at all times.

Aeration

There are two surface aerators in each oxidation ditch, each equipped with a two-speed 100 HP motor. This type of aerator normally is capable of delivering 3.2 – 3.5 lb O_2/HP-hr in clean water under standard conditions (0 mg/l DO, 20 deg C). In wastewater, with 2.0 mg/l DO, the effective oxygen transfer of these aerators is reduced to about 1.6 lb O_2/HP-hr. This results in a total oxygen transfer capacity of about 15,400 lb/day with all four units in operation, and 11,500 lb/day with one unit out of service.

The effective oxygen transfer efficiency used in this evaluation is substantially less than that assumed in the 2005 Carollo capacity evaluation. The efficiency used in this evaluation is based on the review of the operation of several Carrousel oxidation ditch systems. These evaluations resulted in the determination that the field transfer efficiency of the surface aerators was less than originally assumed. Therefore, a lower transfer efficiency was used in this capacity evaluation to ensure adequate aeration capacity is available.

Secondary Clarifiers

There are three secondary clarifiers: one 65' diameter with 10' side water depth (SWD), and two 80' diameter (one with 12' SWD and one with 15' SWD). In any biological treatment system using the activated sludge process, the operation of the secondary clarifiers is normally the most important aspect of process operation. While one must first consider the operating conditions in the oxidation ditch required to remove BOD and ammonia, the ability of the secondary clarifiers to effectively remove the MLSS in the flow from the oxidation ditches is critical. If the secondary clarifiers are unable to collect and remove the MLSS in the flow from the oxidation ditches, the process will fail due to high TSS in the secondary clarifier effluent. The clarifier capacity must be adequate to handle the peak solids loading during the winter (cold weather and peak flow).

The projected peak flow is about 5 times the ADWF. The NPDES discharge permit lists a maximum winter solids limit of 1,500 lb/day, which requires an effluent TSS of 25 mg/l or less at a PDAF of 7.1 mgd. This could be hard to meet unless the clarifiers are conservatively designed. Therefore, the peak flow to the plant becomes the most important factor in determining how much clarifier capacity is needed.

The design and operating criteria that were used to evaluate the effective capacity of the secondary clarifiers are discussed below:

- Surface Overflow Rate (SOR) – Surface overflow rate must be less than the settling velocity of the sludge, or solids will be carried out of the clarifier with the effluent. While SOR is an important parameter, it is generally not the controlling factor in systems with oxidation ditches since oxidation ditch systems are normally operated with relatively high MLSS concentrations. Thus, solids loading is generally the most important factor in evaluating secondary clarifiers used with oxidation ditches.

- Solids Loading Rate (SLR) – The solids loading rate on the secondary clarifiers is determined based on the total mixed liquor flow to the clarifiers (influent plus RAS), MLSS concentration, and clarifier area. In this evaluation, a maximum month solids loading of 24 lb TSS/sqft-day and a peak solids loading of 36 lb TSS/sqft-day were used to assess the effective capacity of the secondary clarifiers under maximum month and peak flow conditions.

- Sludge Volume Index (SVI) – The SVI is an indicator of sludge settleability. The settleability of the mixed liquor has a major influence on the ability of the secondary clarifiers to remove, concentrate, and return the MLSS to the oxidation ditches. A stirred SVI of 125 ml/g was used in this evaluation to evaluate clarifier capacity. While many treatment plants operate with higher SVIs than 125 ml/g, the WWTP operating data generally indicates good sludge settleability. Therefore, the SVI selected as the basis for this evaluation appears to be reasonable.

- RAS Pumping – The RAS pumping capacity must be adequate to remove all of the MLSS entering the clarifiers, or solids will build up and potentially be carried over the weirs with the effluent flow. The MLSS concentration entering the secondary clarifiers and the SVI were used to calculate the settled volume of sludge. The settled volume of sludge represents the RAS capacity required to prevent a buildup of solids in the clarifiers.

  (The piping from each clarifier flows through a meter and a valve and comes together in a common wet well on the suction side of the RAS pumps. Currently the RAS is set at a percentage of the plant flow, with 25%, 40%, and 35% being recycled from clarifier 1, 2, and 3 respectively. Operators attempt to balance the existing clarifier configuration to prevent the following:

  - Lower RAS concentration, requiring higher RAS flows,
  - Uneven sludge blanket depths,
  - Lower WAS concentration (since WAS is taken from the same line), resulting in increased volume of sludge to be dewatered).

  When an upgrade is required for Clarifier #2, it is recommended that the draft tube mechanism be replaced with spiral scrapers that scrape to a center sump similar to clarifiers 1 and 3.

  The City has budgeted to replace the existing WAS pump with a smaller pump that can operate on a more consistent basis. This has the potential to save energy and improve operations. Also, the existing diaphragm pump has a history of high maintenance requirements.

Biological Process Capacity Evaluation

Capacity Evaluation Methodology

A spreadsheet model was developed to perform process calculations necessary to assess the capacity of the various process elements. The model uses influent flows and loadings under various scenarios as the input to the biological process, and performs kinetic calculations to determine nitrification and denitrification rates at the design temperatures. The kinetic rates are used to estimate the MCRT required for nitrification. The model also performs a solids balance on the biological process and assesses the operating conditions in both the oxidation ditches and secondary clarifiers. Because the WWTP adds alum during the summer months to remove phosphorus, this mode of operation was also modeled.

Plant Data

The daily influent CBOD and TSS load data from 2004 through 2009 (Chart 9.2 and Chart 9.3) were reviewed for use in the process capacity evaluation. An attempt was made to calibrate the spreadsheet model using the City of Ashland operating data. The results of the calibration effort indicate that there appear to be some inconsistencies and anomalies in the WWTP operating data. There is considerable variability (greater than 5) in the daily CBOD and TSS data. While there is always variability in wastewater influent data due to the difficulty of obtaining representative samples and analyzing these samples, the Ashland WWTP data shows more variability than other plants.

In addition to the variability in the daily loadings, there appear to be inconsistencies in the individual samples' measured CBOD (or BOD_5) and TSS. Domestic wastewater normally has a fairly consistent TSS:BOD_5 ratio (often around 1:1). While the average CBOD and TSS data is consistent with that seen at other facilities, the results of the individual samples are not. An analysis of the individual daily sample results shows the CBOD:TSS ratio varying from 0.54 to 5.4. This variability is much higher than normal, and may indicate problems with sample preparation and/or analysis in the laboratory. Since the daily CBOD and TSS analyses are performed on a single composite sample, the CBOD:TSS ratio should normally track together with a fairly uniform ratio between the two values.

Based on the reviewed WWTP operating data, it appears that the activated sludge process is operated to maintain an estimated MCRT of 13-17 days based on the WAS data and the MLSS inventory in the oxidation ditches. The WWTP operations staff appears to be controlling the MCRT very effectively in trying to meet the target value. However, at the time of this study, RAS concentration was measured using a Royce TSS Analyzer and the results using this instrument may not be accurate. It is understood that the treatment plant staff is now using lab results to improve the TSS accuracy.

CHART 9.2: Influent CBOD Load

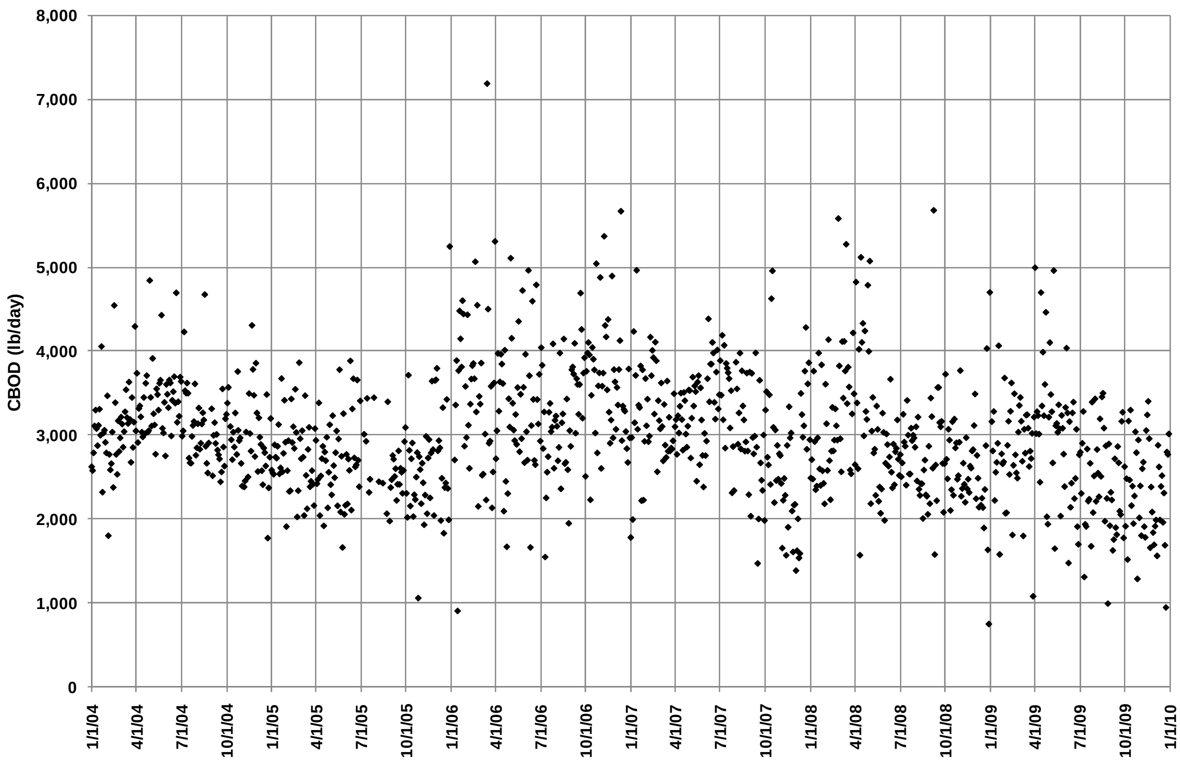
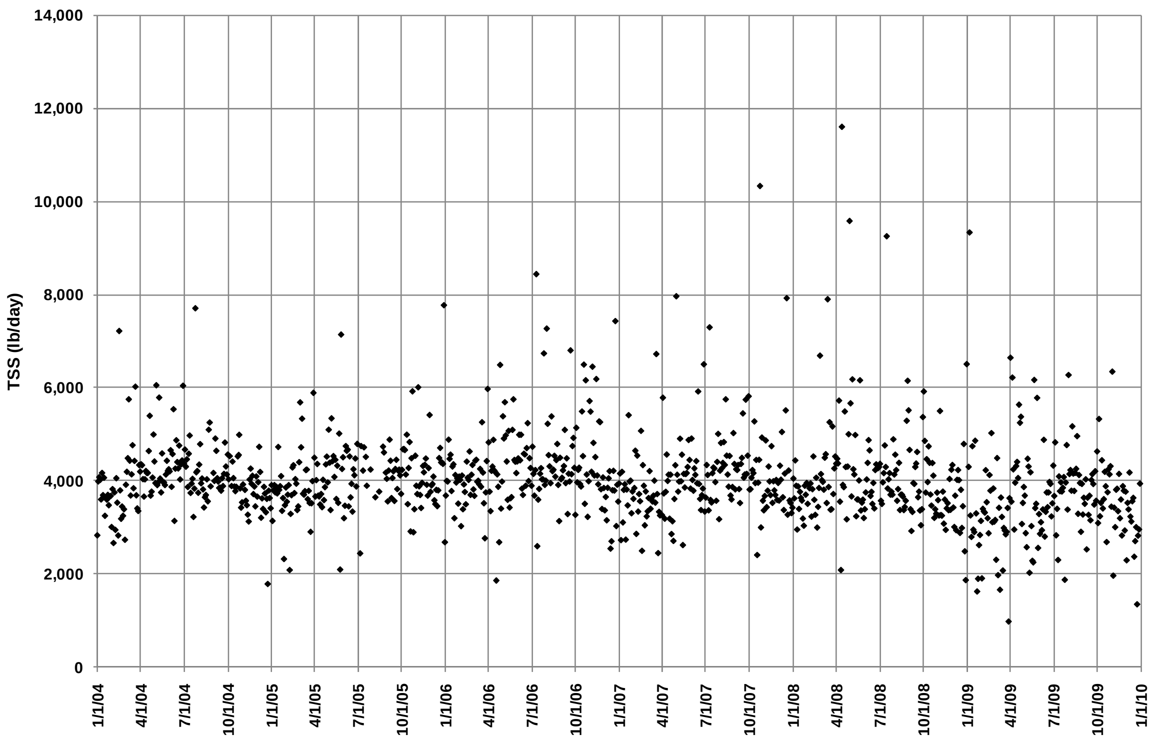


CHART 9.3: Influent TSS Load



It also appeared from the WWTP operating data that the amount of WAS generated at Ashland was much higher than at other domestic wastewater plants. The average 2009 wasting rate was about 5,700 lb WAS/day. For an average influent CBOD load of 2,680 lb/day, this represents a waste sludge production of 2.1 lb TSS/lb influent CBOD or 1.8 lb TSS/lb influent BOD_5. Based on average influent TSS loading of 3,635 lb/day, the WAS production was about 1.57 lb TSS/lb influent TSS. Oxidation ditch processes normally produce about 1.1 lb TSS/lb CBOD, 0.95 lb TSS/lb influent BOD_5, or 0.85 lb TSS/lb influent TSS. Therefore, even considering the addition of alum during part of the year, the level of sludge production at Ashland is much higher than normal, indicating possible problems with the operating data.

Given the apparent discrepancy in the amount of WAS production, the WAS produced was compared to the estimated dry solids (DS) produced from the centrifuges. A 30-day average of dry solids production from the centrifuges (3,350 lb DS/day) compared to 30-day average WAS production (5,650 lb TSS/day) represents a WAS production of about 1.7 lb TSS/lb DS hauled. Based on influent BOD and TSS loadings, the *dry solids* production was about 1.25 lb TSS/lb influent CBOD, 1.06 lb TSS/lb influent BOD_5, and 0.92 lb TSS/lb influent TSS. This amount of dry solids production is close to that normally expected in treating domestic wastewater. Thus, the WAS operating data obtained using the Royce TSS Analyzer appears to be about 1.7 times higher than normal. If the reported MCRT of 13 -17 days is adjusted to account for this difference, it appears that the actual operating MCRT at Ashland is most likely about 22 – 29 days.

Because of apparent inconsistencies and anomalies in the WWTP operating data, the spreadsheet model calculations to assess the capacity of the various process elements at Ashland were performed using influent wastewater characterization and kinetic factors from similar facilities. The flows and loadings presented in Table 9.3 (annual average and maximum month) were used as the input to the model.

(Note that the BOD loadings in Table 9.3 are expressed in terms of BOD_5 rather than CBOD, since most biological process calculations are based on BOD_5 rather than CBOD. The conversion was based on the assumption that CBOD = 0.85 BOD_5. Based on the values summarized in Table 9.2, the projected TSS:CBOD ratio at Ashland (1.3) is consistent with that seen at other plants.)

Year 2010 flows are based on an analysis of influent flow data from 2004 through 2009, using methods prescribed by DEQ guidelines (see Appendix C). The per capita flows determined for 2010 were then used to project future flows for expected populations at various milestones.

Loadings for various constituents were also analyzed based on 2004-2009 data to determine current per capita loadings. The 2010 projections shown in Table 9.2 use these per capita loading rates; the per capita loading rates are increased 10% for future years to allow for new or expanded commercial, industrial or institutional growth. This also provides a factor of safety to account for apparent discrepancies in the data analyzed. (If this increase does not in fact occur, improvements needed to handle increased loadings can be postponed.)

(The projected flows and loadings in this document differ from those developed by Carollo in a 2008 draft report [2]. This is due to the use of different population projections, different base data (2005-2007 data for Carollo analysis; 2004-2009 data for Keller analysis), and the loading factor discussed above. Compared to the Carollo 2008 draft, the values projected by Keller are lower for annual average and maximum month dry weather flows but higher for maximum month wet weather, peak day and peak hour flows. The projected BOD loadings for 2010 (no factor applied) are 2-2.5% higher than the 2008 draft values, and about 11-12% higher for 2015. The projected peak month TSS loading for 2010 (no factor applied) is 8% higher than the 2008 analysis, and about 18% higher for 2015.)

TABLE 9.3: Biological Process Projected Flows and Loadings

Year	2010	2015	2030	2060
Flow				
Annual Average (AAF) (mgd)	2.20	2.30	2.59	3.18
Maximum Month (MMWWF) (mgd)	3.60	3.76	4.24	5.20
Peak (PIF) (mgd)	10.50	10.83	11.81	13.77
BOD_5				
Annual Average (lb/day)	3,822	4,391	4,953	6,077
Maximum Month (lb/day)	4,778	5,489	6,191	7,596
TSS				
Annual Average (lb/day)	4,226	4,856	5,477	6,720
Maximum Month (lb/day)	5,283	6,070	6,846	8,400
TKN				
Annual Average (lb/day)	688	790	891	1,094
Maximum Month (lb/day)	894	1,027	1,158	1,422
NH_3-N				
Annual Average (lb/day)	389	447	504	619
Maximum Month (lb/day)	506	581	655	805
Phosphorus				
Annual Average (lb/day)	88	101	114	140
Maximum Month (lb/day)	114	131	148	182

Note: 2015 loadings include a 10% factor for new commercial growth

The results of the capacity evaluation are discussed in the following paragraphs for existing conditions (2010), as well as future conditions (2015, 2030, and 2060).

Existing Conditions (2010)

Table 9.4 summarizes the estimated operating conditions for the biological treatment process under the projected 2010 influent flow and loading conditions. The estimated operating conditions summarized in Table 9.3 assume the use of the two existing oxidation ditches and the three existing secondary clarifiers.

The 2010 operating conditions under annual average flow and loadings indicate that the existing facilities have adequate capacity for these conditions. The required MLSS concentration in the oxidation ditches, and the required aeration horsepower, indicate that there is reserve capacity under average conditions. Similarly, the projected operating conditions for the secondary clarifiers indicate that reserve capacity is available due to the low overflow rate and solids loading.

At 2010 maximum month flow and loading conditions, the MLSS in the oxidation ditches needs to be increased in order to maintain the MCRT required for nitrification at higher influent loadings and lower temperatures. With the required MLSS concentration calculated at 2,875 mg/l, the existing clarifiers are adequate for the maximum month flow of 3.60 mgd. The existing RAS pump capacity of 1,350 gpm each (3.89 mgd with two pumps) would be adequate to provide the required RAS flow for flows up to about 6.5 mgd. Flows above 6.5 mgd are projected for the peak day and peak hour in 2010. If the frequency or duration of high flows (e.g. peak weekly flows) begins to cause operational problems, larger RAS pumps (2400 gpm each) should be installed to increase RAS capacity. However, if the peak flows occur infrequently, the existing facilities (with all units in service) may be able to handle peak flows by storing excess solids in the existing clarifiers for a short duration.

Table 9.4 also summarizes the projected operating conditions with alum addition for phosphorus removal. As indicated, the MLSS concentration in the oxidation ditches must be increased to maintain the desired MCRT due to increased sludge production with alum addition and higher flow conditions. The estimated sludge production with alum addition is about 13 percent higher than under similar average operating conditions without alum.

Based on the information summarized in Table 9.4, it can be seen that the maximum month and peak flow operating conditions effectively limit the treatment capacity of the secondary treatment facilities at Ashland. With a peak flow of almost five times average, the peak flow conditions and their impact on clarifier operation become the limiting condition in assessing overall capacity.

TABLE 9.4: Projected Ashland WWTP Operation under 2010 Flows and Loadings

	Units	Annual Average	Average with Alum Addition	Maximum Month	Peak Flow
Flows and Loadings					
Flow	mgd	2.20	2.20	3.60	10.50
BOD_5	lb/day	3,822	3,822	4,778	--
TSS	lb/day	4,226	4,226	5,283	--
Oxidation Ditches					
Number of Units	--	2	2	2	--
Temperature	deg C	18.5	18.5	12.5	--
MLSS	mg/l	1,950	2,250	2,875	--
Aerobic MCRT	days	12.1	12.3	14.1	--
Overall MCRT (anoxic + aerobic)	days	15.1	15.4	17.6	--
Overall Detention Time	hr	32.3	32.2	20.7	--
Average Aeration Power	HP	147	147	186	--
Maximum Aeration Power	HP	220	220	280	--
WAS Produced	lb/day	3,781	4,291	4,798	--
Secondary Clarifiers					
Number of Clarifiers	--	3	3	3	3
Stirred SVI	ml/g	125	125	125	125
Percent RAS	%	32%	39%	56%	56%
RAS Flow	mgd	0.84	1.03	2.29	6.11
Surface Overflow Rate	gpd/sqft	205	205	320	854
Solids Loading	lb/sqft-day	4.4	5.4	12.0	32.0

Another item of concern for the Ashland WWTP is system redundancy (particularly the ability to take an oxidation ditch out of service). Based on 2010 flow numbers the plant would be required to operate at a mixed liquor concentration of 2800 mg/l (increases to 3200 mg/l in 2015 and 3600 mg/l in 2030) and a MCRT of 7 days if one oxidation ditch was taken offline. The limiting factor is the aeration capacity. The 200 HP of existing aeration capacity is not enough to meet the peak aeration demand of 218 HP (increases to 249 HP in 2015 and 281 HP in 2030). With this in mind, the plant may be able to meet permit limits during the dry season with one ditch offline even though the peak air demands are not met. However, in considering future improvement alternatives, it should be noted that the increased loading conditions are already beyond recommended operating parameters with one basin offline.

2015 Operating Conditions

The projected operating conditions under the 2015 flow and loads were evaluated. There are three conditions that lead to a recommendation to increase the plant capacity prior to 2015:

1. The first condition is the need to increase aerobic MCRT to achieve nitrification during maximum month flows. With two oxidation ditches available, operators would be required to operate at a mixed liquor concentration of approximately 3300 mg/l in order to obtain the recommended MCRT to achieve nitrification.

2. The second condition is having the ability to take an oxidation ditch offline for maintenance or other reasons. Currently there is not sufficient aeration capacity available to meet the diurnal aeration requirements even during the low flow summer months (as mentioned previously 249 HP is required in 2015 and 200 HP is available).

3. The third and final condition that is driving the need for additional capacity are peak wet weather flows. The peak flow condition of 10.8 MGD provides challenges for settling in the clarifiers (the design surface overflow rate is exceeded) and for adequate RAS pumping.

Operational adjustments such as adding alum and/or polymer during peak flows may allow the timing for the additional facilities to be extended beyond 2015. Two possible expansion alternatives were evaluated in this chapter (other alternative are evaluated in chapter 11). Table 9.5 presents the projected 2015 operating conditions assuming that one additional oxidation ditch is added. Adding one oxidation ditch provides sufficient capacity to allow a reduction in the MLSS concentration in the oxidation ditch during maximum month operating conditions and, in turn, enables the existing three secondary clarifiers to operate within their limits.

It would also be possible to utilize the two existing oxidation ditches, with higher MLSS concentrations, by adding an additional secondary clarifier. Table 9.6 summarizes the projected 2015 operating conditions with two oxidation ditches and four secondary clarifiers.

In comparing the projected operating conditions in Table 9.5 with one additional oxidation ditch, to those in Table 9.4 with one additional secondary clarifier, it can be seen that the addition of an oxidation ditch appears to have advantages over adding a secondary clarifier. The biggest advantage of adding an additional oxidation ditch is that it would significantly increase the overall reliability and redundancy of the WWTP.

As indicated in Table 9.5, the projected aeration power requirements under average influent loading conditions are 168 HP average demand and 251 HP maximum demand. While these aeration power demands can be met with two oxidations ditches in service, and one aerator out of service, it would be difficult to operate in this configuration for an extended period due to an imbalance between the loadings to each basin and the available aeration horsepower. If one aerator were out of service during maximum month loading conditions, there would not be sufficient capacity available under maximum demand periods. If an additional oxidation ditch were added, it would be possible to take one unit out of service for maintenance while continuing to use the other two units for treatment.

TABLE 9.5: Projected WWTP Operation under 2015 Flows and Loadings with a Third Oxidation Ditch

	Units	Annual Average	Average with Alum Addition	Maximum Month	Peak Flow
Flows and Loadings					
Flow	mgd	2.30	2.30	3.76	10.83
BOD_5	lb/day	4,391	4,391	5,489	--
TSS	lb/day	4,856	4,856	6,070	--
Oxidation Ditches					
Number of Units	--	3	3	3	--
Temperature	deg C	18.5	18.5	12.5	--
MLSS	mg/l	1,500	1,700	2,250	--
Aerobic MCRT	days	12.2	12.2	14.4	--
Overall MCRT (anoxic + aerobic)	days	15.2	15.2	18.0	--
Overall Detention Time	hr	46.6	46.4	29.8	--
Average Aeration Power	HP	168	168	213	--
Maximum Aeration Power	HP	251	251	319	--
WAS Produced	lb/day	4,345	4,913	5,513	--
Secondary Clarifiers					
Number of Clarifiers	--	3	3	3	3
Stirred SVI	ml/g	125	125	125	125
Percent RAS	%	23%	27%	39%	39%
RAS Flow	mgd	0.63	0.74	1.66	4.39
Surface Overflow Rate	gpd/sqft	213	214	333	880
Solids Loading	lb/sqft-day	3.3	3.9	8.7	23.0

TABLE 9.6: Projected WWTP Operation under 2015 Flows and Loadings with a Fourth Secondary Clarifier

	Units	Annual Average	Average with Alum Addition	Maximum Month	Peak Flow
Flows and Loadings					
Flow	mgd	2.30	2.30	3.76	10.83
BOD_5	lb/day	4,391	4,391	5,489	--
TSS	lb/day	4,856	4,856	6,070	--
Oxidation Ditches					
Number of Units	--	2	2	2	--
Temperature	deg C	18.5	18.5	12.5	--
MLSS	mg/l	2,250	2,550	3,300	--
Aerobic MCRT	days	12.2	12.2	14.1	--
Overall MCRT (anoxic + aerobic)	days	15.2	15.2	17.6	--
Overall Detention Time	hr	31.0	30.9	19.9	--
Average Aeration Power	HP	168	168	213	--
Maximum Aeration Power	HP	251	251	319	--
WAS Produced	lb/day	4,345	4,913	5,513	--
Secondary Clarifiers					
Number of Clarifiers	--	4	4	4	4
Stirred SVI	ml/g	125	125	125	125
Percent RAS	%	39%	47%	70%	70%
RAS Flow	mgd	1.06	1.28	2.98	7.88
Surface Overflow Rate	gpd/sqft	158	159	247	654
Solids Loading	lb/sqft-day	4.1	5.0	11.6	30.6

The second advantage of having three oxidation ditches is that it allows operation with a lower MLSS concentration to maintain the desired MCRT. While operating an activated sludge process with low MLSS is generally easier than at high MLSS, it also gives operational flexibility without overloading the secondary clarifiers. In comparing the solids loading to the secondary clarifiers under peak flow conditions with three oxidation ditches in service (Table 9.4), to that with only two oxidation ditches in service (Table 9.6), the solids loading conditions are significantly lower with lower MLSS.
In summary, the city should consider a third oxidation ditch because:

- As the CBOD loadings increase, the MLSS needs to increase proportionately to maintain the MCRT.
- When the MLSS increases to 3,300 mg/l (Table 9.6) in two ditches at projected 2015 loads, the required RAS rate increases to 70% compared to 56% under 2010 conditions. This is a 25% RAS flow increase.
- The 25% increase in required RAS flow, coupled with a 5% increase in influent flow, means the solids loading to the secondary clarifiers will increase about 31%. The 25% RAS flow increase is beyond the capacity of the existing RAS pumps. If peak flows were to last for an extended period of time, the possibility exists that solids could be washed over the clarifier weirs.
- Thus the solids loading increases from 32 lb/sqft-day to approximately 42 lb/sqft-day, which is too high (limit 36 lb/sqft-day).

Based on the results of the evaluation presented above, it is recommended that - if the City desires to keep the same treatment process to meet the future needs - one additional oxidation ditch be constructed to increase the effective biological treatment capacity of the WWTP to handle the projected flow and loads in 2015. Other options are explored in Chapter 11.

2030 Operating Conditions

The projected operating conditions under the 2030 flow and loads were evaluated assuming three oxidation ditches in operation together with the three existing secondary clarifiers. The results of this evaluation, which are summarized in Table 9.7, indicates that there should be adequate capacity to treat the projected flow and loads with all units in service. It should also be possible to take individual units out of service for maintenance during the summer.

As summarized in Table 9.7, the projected operating conditions with all units in service are conservative. The required MLSS concentration in the oxidation ditches needed to maintain the desired MCRT is low. Because of this, operational flexibility is enhanced. There would also be sufficient aeration capacity available under all operating conditions. The projected operating conditions for the secondary clarifiers, even under peak flow conditions, are very reasonable assuming the size of the RAS pumps is increased to 2,400 gpm each.

Based on the evaluation of the operating conditions, under 2030 flow and loading conditions, the WWTP oxidation ditch and clarifier process should provide sufficient biological treatment capacity to handle projected flows and loadings beyond this point in time.

TABLE 9.7: Projected WWTP Operation under 2030 Flows and Loadings

	Units	Annual Average	Average with Alum Addition	Maximum Month	Peak Flow
Flows and Loadings					
Flow	mgd	2.59	2.59	4.24	11.81
BOD_5	lb/day	4,953	4,953	6,191	--
TSS	lb/day	5,477	5,477	6,846	--
Oxidation Ditches					
Number of Units	--	3	3	3	--
Temperature	deg C	18.5	18.5	12.5	--
MLSS	mg/l	1,700	1,900	2,500	--
Aerobic MCRT	days	12.2	12.1	14.2	--
Overall MCRT (anoxic + aerobic)	days	15.3	15.2	17.7	--
Overall Detention Time	hr	42.0	41.8	26.7	--
Average Aeration Power	HP	189	189	240	--
Maximum Aeration Power	HP	283	283	360	--
WAS Produced	lb/day	4,900	5,519	6,219	--
Secondary Clarifiers					
Number of Clarifiers	--	3	3	3	3
Stirred SVI	ml/g	125	125	125	125
Percent RAS	%	27%	31%	45%	45%
RAS Flow	mgd	0.81	0.94	2.16	5.55
Surface Overflow Rate	gpd/sqft	237	238	372	957
Solids Loading	lb/sqft-day	4.3	4.9	11.3	29.0

2060 Operating Conditions

The projected operating conditions under the 2060 flow and loads were evaluated and it was determined that additional facilities will be required prior to 2060. As outlined in Table 9.8, it was determined that one additional secondary clarifier would be required to provide sufficient capacity to handle the increased flows and loads between 2030 and 2060.

It was estimated that the WWTP, with three oxidation ditches and three secondary clarifiers, should provide sufficient capacity to handle the projected flows and loads until about 2044. Therefore, prior to 2044, it will be necessary to complete construction of a fourth secondary clarifier and RAS pump station.

TABLE 9.8: Projected WWTP Operation under 2060 Flows and Loadings

	Units	Annual Average	Average with Alum Addition	Maximum Month	Peak Flow
Flows and Loadings					
Flow	mgd	3.18	3.18	5.20	13.77
BOD_5	lb/day	6,077	6,077	7,596	--
TSS	lb/day	6,720	6,720	8,400	--
Oxidation Ditches					
Number of Units	--	3	3	3	--
Temperature	deg C	18.5	18.5	12.5	--
MLSS	mg/l	2,100	2,350	3,100	--
Aerobic MCRT	days	12.3	12.3	14.3	--
Overall MCRT (anoxic + aerobic)	days	15.4	15.4	17.9	--
Overall Detention Time	hr	34.9	34.8	22.1	--
Average Aeration Power	HP	232	232	294	--
Maximum Aeration Power	HP	347	347	441	--
WAS Produced	lb/day	6,013	6,735	7,630	--
Secondary Clarifiers					
Number of Clarifiers	--	4	4	4	4
Stirred SVI	ml/g	125	125	125	125
Percent RAS	%	36%	42%	63%	63%
RAS Flow	mgd	1.29	1.51	3.62	8.96
Surface Overflow Rate	gpd/sqft	211	212	333	825
Solids Loading	lb/sqft-day	5.0	5.9	14.1	34.8

Based on the projected operating conditions for 2060, it appears that the WWTP will again be reaching its effective capacity based on using three oxidation ditches and four secondary clarifiers. Because of the uncertainty in projecting future conditions 50 years from now, together with possible changes in discharge requirements and/or changes in technology, the information presented above for 2060 should only be considered as a planning level indication of future needs.

9.3.4 UV Disinfection

Capacity

The City has four in-line UV reactors, Model 5000 as manufactured by Aquionics, Inc. The reactors are installed in the old chlorine contact chamber. The four reactors are divided between two trains with two reactors in each train; each reactor contains 8 medium pressure high intensity UV lamps. Effluent from the secondary clarifiers enters the secondary effluent discharge box, and the flow is split to the two UV trains with a motorized butterfly valve controlling the flow to each train.

Aquionics has indicated that the reactors are each rated at a capacity of 2.125 mgd at a UV transmissivity (UVT) of 65% with a dose of 24 mJ/cm^2; this results in a per train capacity of 4.25 MGD with an average dose of 48 mJ/cm^2. Based on the specified transmissivity and dose, the UV system has sufficient disinfection capacity for the year 2030 maximum month flow of 4.24 with a single train in operation, and for the year 2030 peak day flow of 8.03 mgd with both trains in operation. If the transmissivity of the effluent entering the UV system is higher, the disinfection capacity would be greater.

However, hydraulics may be the limiting factor in determining the useful capacity of the existing UV system. Though the hydraulic limit of the in-line 5000 (according to Aquionics) is 6.3 mgd (12.6 mgd through both trains), the system overall cannot handle the head loss at this flow without surcharging. Even with both UV trains in operation, flow would begin to back up in the clarifiers at about 9 mgd and could overflow the clarifier launder at about 11 mgd. To meet future peak flow conditions, the existing reactors could be replaced with larger units to expand capacity or parallel reactors could be added to provide capacity plus redundancy.

Condition

The reactors are stainless steel and are in good condition. The UV lamps have an estimated life of 8,000 hours and should be changed out as necessary to remain in good condition. The electronics (wiring, ballasts, contactors, relays) will begin to show signs of age as it approaches its design life of 15 to 20 years. In order to maintain operation of the UV system over the next 20 years, the City will need to rehabilitate the UV electronic components over the next decade (it was indicated that this is already underway).

Control System

Each of the UV trains has a magnetic flow meter, and the flow in each train is provided to the SCADA system where the total flow is calculated. The flow rate is used to control the membrane feed pumps. The current UV system does not allow the UV units to increase or decrease the lamp intensity. It is recommended that UV intensity control be included in future upgrades so the system can be flow paced to save energy.

The control system for the UV reactors was provided by Aquionics, and is in good working order. However, the existing panels do not have cooling and during the summer the panels must be left open to prevent overheating. The components of the control system have a useful life in the 15- to 20-year range. Thus it is expected that the control system will need to be replaced during the next five years. The City is also in the process of adding air conditioning to the control area of the UV building or adding air conditioned venting to the top of the control panels.

Currently, there is one control and one power panel for each UV unit for a total of eight. When the panels are replaced, there will be one power and one control panel for each train or four total. The estimated cost provided by Aquionics for new control and power panels is $25,000 per train. New panels should be required to have air conditioners.

Monitoring System

The UV control panels provide information on each reactor, including UV intensity and lamp failure. As part of the upcoming NPDES permit renewal, it is understood that DEQ will require a UVT monitor. Monitoring the actual transmissivity of the UV system may result in the ability to lower the UV dose (based on the assumption that the design value of 65% is conservative and actual transmissivity is higher). If energy savings can be realized by operating the system based on UVT, there may be an opportunity for funding the control upgrade through the Oregon Energy Trust.

Filtration after Disinfection

The UV system is installed prior to the membrane filters, which is not optimal for disinfection performance. Class A recycled water regulations require disinfection to be after filtration unless approved in writing by DEQ. Alternately, the City could relocate the UV reactors to a new vault on the effluent line following filtration. With the current flow schematic, if the filters are on line and the flow exceeds the capacity of the membrane filters, the City has to monitor the discharge from the filters and from the reaeration tanks for coliform and phosphorus.

9.3.5 Membrane Filtration

Filter Influent Pumping

The filter influent pump system consists of 4 Floway vertical turbine pumps each rated at 1,050 gpm at 52 feet of head. The system is rated at 4.54 mgd with one pump out of service. Thus the pumps exceed the current capacity of the membranes. As discussed below, as membranes are replaced with newer models and the membrane filtration capacity increases, the pumps will eventually need to be replaced with larger capacity pumps.

Membranes

Structures
The concrete structure and building housing the membrane system (membranes, blowers, pumps, and chemical feed system) are in good condition. With ongoing maintenance and coating repair or replacement, the structures would be expected to last at least another 20 years.

Membrane Capacity
The existing membrane system consists of four trains, each containing 10 cassettes with 26 modules each. Nine cassettes were initially installed in each train. The initial membranes installed in 2002 were Zenon 500c-220 membranes, with 220 square feet of membranes per module. In 2008, the City added one new cassette with Zenon 500c-250 membranes (250 square feet per module) to each train. At an average flux of 12 gallons per square foot per day (gfd), the existing system has a maximum month filtration capacity of 2.746 mgd. At a flux of 15 gfd, the system can handle a peak of 3.432 mgd.

Since the membranes are operated only during the dry weather period (May through October), the membrane system capacity is based on the effluent meeting monthly and daily phosphorus limits with dry weather flows. The existing membranes have sufficient capacity to filter 1) the entire 2010 maximum month dry weather flow of 2.7 mgd, to meet the monthly phosphorus limit of 1.6 ppd, and 2) 69% of the 2010 peak day dry weather flow of 5.0 mgd, to meet the daily phosphorus limit of 5.1 mgd (assuming clarifier effluent phosphorus ≤0.23 mg/L). In order to continue meeting these limits as flows increase, more of the flow will need to be treated to the lower phosphorus levels achievable in the membranes; as a result, less flow can be bypassed to the clarifiers and the membrane capacity will need to be increased.

<u>Membrane Condition</u>
Plant staff recently conducted an inspection of the membranes. The inspection revealed that a number of fibers in many of the cassettes are separating from the urethane potting. This is likely due to overexposure to chlorine that is applied during winter when the membranes are in storage. Approximately 25% of the membrane cassettes had more than half of the fibers loose. Since this occurred, Zenon's storage protocol has been updated to prevent this from occurring. As a result, it is possible that future membrane life expectancy could be increased. The condition of the existing membranes is presented in Table 9.9.

TABLE 9.9: 2010 Membrane Condition

Train	Cassettes	Installation Date	Percent Loose	sf/ cass.	gal/sf/d	Flow	MGD
1	1	Installed January 2008	0	220	12	68,640	
1	2	Installed 2002	5	220	12	68,640	
1	3	Installed 2002	25	220	12	68,640	
1	4	Installed 2002	0	220	12	68,640	
1	5	Installed 2002	30	220	12	68,640	
1	6	Installed 2002	0	220	12	68,640	
1	7	Installed 2002	50	220	12	68,640	
1	8	Installed 2002	0	220	12	68,640	
1	9	Installed 2002	30	220	12	68,640	
1	10	Installed 2002	50	220	12	68,640	
							0.686
2	1	Installed January 2008	0	220	12	68,640	
2	2	Installed 2002	10	220	12	68,640	
2	3	Installed 2002	5	220	12	68,640	
2	4	Installed 2002	0	220	12	68,640	
2	5	Installed 2002	5	220	12	68,640	
2	6	Installed 2002	50	220	12	68,640	
2	7	Installed 2002	50	220	12	68,640	
2	8	Installed 2002	30	220	12	68,640	
2	9	Installed 2002	50	220	12	68,640	
2	10	Installed 2002	50	220	12	68,640	
							0.686
3	1	Installed January 2008	0	220	12	68,640	
3	2	Installed 2002	0	220	12	68,640	
3	3	Installed 2002	0	220	12	68,640	
3	4	Installed 2002	0	220	12	68,640	
3	5	Installed 2002	0	220	12	68,640	
3	6	Installed 2002	0	220	12	68,640	
3	7	Installed 2002	100	220	12	68,640	
3	8	Installed 2002	100	220	12	68,640	
3	9	Installed 2002	100	220	12	68,640	
3	10	Installed 2002	100	220	12	68,640	
							0.686
4	1	Installed January 2008	0	220	12	68,640	
4	2	Installed 2002	0	220	12	68,640	
4	3	Installed 2002	0	220	12	68,640	
4	4	Installed 2002	0	220	12	68,640	
4	5	Installed 2002	0	220	12	68,640	
4	6	Installed 2002	0	220	12	68,640	
4	7	Installed 2002	0	220	12	68,640	
4	8	Installed 2002	0	220	12	68,640	
4	9	Installed 2002	0	220	12	68,640	
4	10	Installed 2002	0	220	12	68,640	
0 % Loose							**0.686**
1-49 % Loose							
50-99 % Loose							
100 % Loose						**2010 Total**	**2.746**

The City decided in August 2010 to purchase 10 new cassettes of the 500c-250 membrane modules to replace the cassettes that are in the worst condition. After the replacement (winter 2011), the membranes and their condition will be as shown in Table 9.10 and the estimated capacity will be 2.867 mgd maximum month and 3.689 mgd peak day.

TABLE 9.10: 2011 Membrane Condition

Train	Cassettes		Percent Loose	sf/ cass.	gal/sf/d	Flow	MGD
1	1	Installed January 2008	0	250	12	78,000	
1	2	Installed 2002	0	220	12	68,640	
1	3	Installed 2002	25	220	12	68,640	
1	4	Installed 2002	25	220	12	68,640	
1	5	Installed 2002	25	220	12	68,640	
1	6	Installed 2002	25	220	12	68,640	
1	7	Installed 2002	25	220	12	68,640	
1	8	Installed 2002	25	220	12	68,640	
1	9	Installed 2002	25	220	12	68,640	
1	10	Installed January 2008	0	220	12	68,640	
							0.696
2	1	Installed January 2011	0	250	12	78,000	
2	2	Installed January 2011	0	250	12	78,000	
2	3	Installed January 2011	0	250	12	78,000	
2	4	Installed January 2011	0	250	12	78,000	
2	5	Installed January 2011	0	250	12	78,000	
2	6	Installed January 2011	0	250	12	78,000	
2	7	Installed January 2011	0	250	12	78,000	
2	8	Installed January 2011	0	250	12	78,000	
2	9	Installed January 2011	0	250	12	78,000	
2	10	Installed January 2011	0	250	12	78,000	
							0.780
3	1	Installed January 2008	0	250	12	78,000	
3	2	Installed 2002	0	220	12	68,640	
3	3	Installed 2002	0	220	12	68,640	
3	4	Installed 2002	0	220	12	68,640	
3	5	Installed 2002	0	220	12	68,640	
3	6	Installed 2002	0	220	12	68,640	
3	7	Installed 2002	0	220	12	68,640	
3	8	Installed 2002	0	220	12	68,640	
3	9	Installed 2002	0	220	12	68,640	
3	10	Installed 2002	0	220	12	68,640	
							0.696
4	1	Installed January 2008	0	250	12	78,000	
4	2	Installed 2002	0	220	12	68,640	
4	3	Installed 2002	0	220	12	68,640	
4	4	Installed 2002	0	220	12	68,640	
4	5	Installed 2002	0	220	12	68,640	
4	6	Installed 2002	0	220	12	68,640	
4	7	Installed 2002	0	220	12	68,640	
4	8	Installed 2002	0	220	12	68,640	
4	9	Installed 2002	0	220	12	68,640	
4	10	Installed 2002	0	220	12	68,640	
0 % Loose							**0.696**
1-49 % Loose							
New - 0% Loose						**2015 Total**	**2.867**

The City will need to replace the membranes installed in 2002 as they wear out. This will require the City to inspect the membranes each year, observe the membrane data, assess their condition, and determine if cassettes should be replaced. Though the expected life of the membranes is between 10 and 15 years, some cassettes could last longer. An estimated replacement schedule is shown in Table 9.11. Replacements through 2018 are based on membrane age and anticipated condition, whereas later replacements are needed to address capacity to meet phosphorus limits. A memo discussing the replacement schedule is included in Appendix E.

TABLE 9.11: Suggested Membrane Cassette Replacement Schedule

Year	Train	Cassettes Replaced/Moved	Capacity, mgd Avg/Pk	Design Flow, mgd MMDF/PDDWF	Bypass PDDWF, mgd	Clarifier Eff P, mg/L
2012	1	Replace #1-10 with 500c-250 (move 3 cassettes from 2008 to train 3, leave one in train 4)	2.95 / 3.69	2.75 / 5.26	1.57	0.22
2013	3	Replace #1-10 with 500c-250 (move all 4 cassettes from 2008 to train 4)	3.06 / 3.83	2.77 / 5.38	1.55	0.22
2014	4	Replace #5-10 with 500c-250	3.12 / 3.90	2.80 / 5.51	1.61	0.21
2018	4	Replace #1-4 with 500c-250	3.12 / 3.90	2.89 / 5.72	1.82	0.19
2023	2	Replace with 340 sf membranes	3.65 / 4.57	3.01 / 5.86	1.30	0.25
2025	1	Replace with 340 sf membranes	4.17 / 5.27	3.06 / 5.92	0.65	0.44
2027	3	Replace with 340 sf membranes	4.70 / 5.87	3.11 / 5.98	0.11	-
2030	4	Replace with 340 sf membranes	5.22 / 6.53	3.18 / 6.06	-	-

When the 340 square foot membranes are installed, the header piping, valves, and permeate pumps will have to be replaced as well. Therefore it is recommended that a similar evaluation be performed prior to upgrading to the 340 square foot membranes.

Membrane Control System

The control system for the membranes was provided by Zenon, and is in good working order. The components of the control system have a useful life in the 10- to 20-year range. Thus it is expected that the control system will need to be replaced during the next ten years.

Monitoring System

Zenon provides a data tracking system called Zenotrac to extract and maintain data from the membrane system. This data can be used to evaluate the condition of the membranes, the effectiveness of the maintenance cleaning procedures, and the effectiveness of the in-place chemical cleans. A review of the existing data indicates that:

- There is no data stored for 2002, 2003, or 2004.
- For each train, there is data stored for the following periods:
  - September 20 to October 20, 2005
  - June 28 to November 14, 2006
  - April 5 to November 22, 2007
  - April 1 to December 8, 2008
  - March 31 to November 11, 2009

- For train 1, there is stored data from April 30 to July 1, 2010 and from July 28 to November 14, 2010.
- For trains 2 and 3, there is stored data from June 3, 2010 and from July 7 to November 14, 2010.
- For train 4, there is stored data from May 26, 2010 and from July 7 to November 14, 2010.

In Zenotrac, the Ashland membrane system is set up to store flux rate for each train in gallons per square feet per day (gfd), trans-membrane pressure (TMP) in psi, permeability in gfd/psi, permeate flow in gpm, permeate and reject flow in gallons, tank level in feet, and permeate pump speed in % (0 to 100 by the VFD). The measurements that are taken and recorded in Zenotrac occur just before, during, and just after each backflush. The backflushes occur approximately every 15 minutes. Thus, a great deal of data is collected for each train each day. Temperature data should be added to Zenotrac so that temperature-adjusted permeability can be calculated. This removes the effect of temperature on permeability to provide a better picture of the condition and performance of the membranes.

A preliminary review of the Zenotrac data indicates that the Ashland system has operated between a TMP of 1 to 7 psi, and that the recovery after each chemical clean has been good. There is no indication of loss of capacity. Because the Ashland flow rate is not equalized, the flow to the membranes (and correspondingly the flux across the membranes) fluctuates depending on the influent flow. Since the flux data changes due to the influent flow rate rather than membrane performance, it does not provide a good picture of how the membranes are performing. The reject flow data shows that the amount of reject flow (flow that is wasted to maintain a reasonable solids level in the reactors) has increased nearly every year from 2005 to 2009. Although the amount of reject flow is increasing, Zenon considers the current reject flow to be a reasonable percentage of the total throughput.

Zenon recommends that the operators review the data at least weekly. Keller Associates' experience is that many operators review the data for their plants daily.

Blowers and Pumps

Five 50 hp blowers, each with a capacity of 920 scfm, were installed in 2002 to provide air scour for the membranes. The membrane system also includes four variable speed permeate pumps with a capacity of 785 gpm each, two variable speed backpulse pumps (1179 gpm each), three vacuum pumps, and four variable speed reject pumps.

Chemical Storage and Feed Systems

The chemical storage and feed systems discussed in this section are the alum, sodium hypochlorite, sodium hydroxide, citric acid, and sodium bisulfite facilities affiliated with the membrane filters. Alum is added to the inlet flow to the membrane tanks to remove phosphorus by forming insoluble precipitates of aluminum phosphate.

Sodium hypochlorite (12.5%) and citric acid are added to the membranes for cleaning purposes. The citric acid facilities are labeled MC-1 cleaner on the record drawings. Either or both sodium hypochlorite or citric acid can be added to the periodic maintenance cleans programmed in as ongoing maintenance. The frequency of these cleans is determined by the performance of the membranes in terms of flux, trans-membrane pressure (TMP), and permeability. These chemicals are also added to the membrane tank during a clean-in-place.

Sodium hydroxide facilities are provided to raise the pH (if necessary) due to the possible lowering of pH by addition of alum. However, pH adjustment is not required to meet the NPDES permit limits and these facilities are not used. Sodium bisulfite facilities are provided to dechlorinate both the effluent flow after maintenance cleaning and the contents of the tank after a clean-in-place. Dechlorination has not been needed to meet the effluent permit, so these facilities are not used.

Capacity

All of the chemical storage tanks are installed inside a double contained area in the downstairs area of the filter building.

The alum tank, which stores a liquid solution containing 48.5% alum, has a volume of 8,000 gallons. The two alum feed pumps are peristaltic pumps, each with a capacity of 0.5 gpm. Alum is injected into the pipe just after the oxidation ditch.

In 2008 and 2009, the City dosed alum at an average of 85 mg/L using an alum pump flow rate of 0.24 gpm. During this period the average effluent flow was approximately 2.0 mgd, and an average of approximately 340 gpd of alum was used to achieve an average effluent total phosphorus concentration of 0.04 mg/L (0.7 ppd). Based on this usage rate, the storage tank has a capacity of 23 days storage on average. The feed pumps have sufficient capacity for effluent flows up to approximately 4 mgd; the storage tank would provide about 12 days storage at this flow rate.

The sodium hypochlorite tank has a volume of 480 gallons. The City currently purchases sodium hypochlorite in totes, and no longer uses the storage tank. The sodium hypochlorite feed pump is a metering pump rated at 8.57 gpm.

The sodium bisulfite tank has a volume of 270 gallons, and the sodium bisulfite feed pump is a metering pump rated at 1.51 gpm. The City does not currently use sodium bisulfite at the plant, as dechlorination has not been required. The sodium hydroxide tank is 270 gallons and the sodium hydroxide feed pump is a metering pump rated at 1.08 gpm. The City does not currently use sodium hydroxide at the plant, as pH adjustment has not been required.

The citric acid tank has a volume of 600 gallons. The City currently purchases citric acid in 50 lb bags, which are diluted and mixed by operations staff. The citric acid feed pump is a metering pump rated at 9.82 gpm.

The chemical feed system for sodium hypochlorite, sodium bisulfite, sodium hydroxide, and citric acid were sized by Zenon, the membrane manufacturer, and are sufficient for the membrane system through 2030. The chemical feed pumps and solenoid valves may have to be replaced in the next 10 years due to the corrosivity of the chemicals.

Condition

The chemical system is in good condition, except that the magna drives on the chemical feed pumps are not reliable. The chemical feed pumps should be replaced with a more reliable pump. The alum piping has minor leaks and will need to be replaced in a few years. The alum pump is not flow-paced. In order to provide more control of the dosing, a high quality pump with flow pacing should be installed.

Chemical Control System
The plant SCADA system controls the speed of the peristaltic pumps based on the membrane feed pump rate. The SCADA system alternates pumps, and will start the backup pump if the lead pump fails.

The Zenon control panel controls the sodium hypochlorite pump on and the MC-1 pump on during the maintenance cleans and during the clean-in-place procedures. Solenoid valves are controlled open and closed by the control panel to direct the sodium hypochlorite and citric acid to the correct membrane train, drain pump, or dip tank.

Should sodium bisulfite and/or sodium hydroxide be required, the Zenon control panel would control the respective pumps and related solenoid valves to direct the chemical to the correct membrane tank. The Zenon control panel would turn on the sodium bisulfite pump after the clean-in-place procedure to remove the chlorine from the tank, and would control the sodium hydroxide pump on after the clean-in-place procedures to raise the pH after the MC-1 clean.

Monitoring
The SCADA system monitors the level in each of the chemical tanks and provides a high and low alarm. The SCADA system monitors the pH and chlorine residual after the drain pump, and changes the sodium hydroxide pump rate to bring the pH to 7 and the sodium bisulfite pump rate to bring the chlorine residual to non-detectable.

9.3.6 Outfall

The outfall to Ashland Creek consists of 185 feet of 24-inch pipe (per record drawings, 1998 WWTP improvement project). The Department of Fish and Wildlife indicated that the existing pipe provides access to the treatment plant for fish, and that provisions should be made to block fish entry at the end of the pipe. The need for upsizing or any improvements to the existing pipe should be re-evaluated after the temperature issue is resolved (which may involve relocation of the point of discharge).

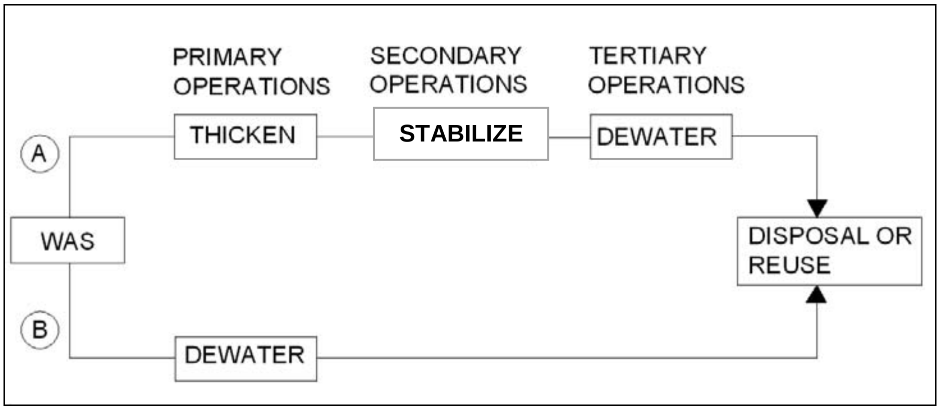
9.3.7 Solids Handling

This section will provide an evaluation of the City's current sludge handling facilities for existing and future solids generation rates. Applicable regulations were discussed in Chapter 2 which defines the treatment requirements for Class A and B biosolids. The objectives of this evaluation are to determine if current equipment and practices are adequate for managing future solids projections.

Current Sludge Management Program at the Ashland WWTP

A typical approach to treating waste activated sludge (WAS), shown in path A of Figure 9.4, includes thickening, stabilization, and dewatering prior to disposal. Path B would normally require disposal at a landfill that is permitted to accept nonstabilized biosolids. Sludge management at the Ashland WWTP includes a sludge storage tank and two centrifuges. The WAS is not thickened or stabilized prior to dewatering, but is wasted directly to the sludge storage tank prior to dewatering (basically the process shown in path B). The WAS is dewatered five days per week and then hauled to the landfill for disposal.

CHART 9.4: Generalized Sludge Process Flow Diagram



Sludge stabilization benefits a treatment facility by removing the organic and volatile portion of the WAS, which in turn reduces the odor and vector attraction parameters. Thickening is employed to increase the efficiency of the stabilization process (thicker biosolids result in less volume to treat). Initially, the WAS is approximately 1-2% solids, but thickening can increase the solids content to approximately 4-6%. After stabilization, mechanical dewatering equipment generally increases the solids content to approximately 20%. (Drying beds have the potential of generating biosolids with approximately 80-90% solids content, but would require extended detention time with warm temperatures and minimal precipitation.) The benefits of stabilization and dewatering include [3,4]:

Reduced transportation costs

- Easier handling
- Reduced requirements for supplemental bulking agents and composting amendments
- Satisfy Part 503 Rule Vector Attraction Reduction (VAR) requirements
- Meet potential landfill requirements for leachate reduction and space issues

Plant upgrades in 1998 included lime stabilization equipment, conversion of the aerobic digester to a lime stabilization/sludge storage tank and two centrifuges. The lime stabilization process was designed to stabilize the sludge to Class B standards prior to dewatering, and was used for a short period of time. However, it was discontinued since dewatered biosolids were landfilled and stabilization was not required. The WAS is dewatered five days per week and then hauled to the landfill for disposal.

Reportedly, the lime feed equipment is in good condition and could be returned to operation with minimal preparation efforts. The operators like using the centrifuges for dewatering. The centrifuges are operated three to four hours per day and there have been few maintenance issues. The solids dewatering and hauling process requires approximately 30 hours per week of staff time to operate and maintain. The cost to maintain the centrifuges has been estimated at approximately $3,750 per year.

The Ashland WWTP does not have a Sludge Management Plan (SMP). Their NPDES permit states that an SMP is not required since the City disposes of the dewatered sludge in a licensed landfill. Dewatered sludge is currently hauled to the Dry Creek landfill in White City, Oregon which is a 40-mile round trip from the Ashland WWTP. The City hauls approximately 15 tons per day, 5 days per week using a 12-yard truck.

Evaluation of Existing Sludge Management Process

As part of the master planning process, future waste activated sludge (WAS) generation rates were projected to evaluate the current sludge management program and develop future recommendations. Alum is added to the wastewater prior to the membrane filters for phosphorus removal approximately six months each year. Tables 9.12 and 9.13 show the facility design loadings and WAS flow rates projected for this evaluation:

TABLE 9.12: WAS Generation at the Ashland WWTP

Year	2010	2030	2060
Avg. Daily WAS Production (lbs./day)	3,781	4,900	6,013
Avg. Daily WAS Production w/Alum Addition (lbs./day)	4,291	5,519	6,735
Avg. Max. Month WAS Production (lbs./day)	4,798	6,219	7,630

TABLE 9.13: Projected WAS Flow Rates for Stabilization and Dewatering

Year	2010	2030	2060
Avg. Daily WAS Wasting Rate (MGD)	0.055	0.073	0.089
Avg. Daily WAS Wasting Rate w/Alum Addition (MGD)	0.064	0.082	0.099
Avg. Max. Month WAS Wasting Rate (MGD)	0.070	0.091	0.112

Note: Flowrates based on 0.8% solids concentration.

The 1998 treatment plant upgrades included installation of lime feed equipment, conversion of the aerobic digester to a covered, lime stabilization/interim sludge holding tank with an air scrubber for odor control, and construction of a new Dewatering Building with a polymer feed system, two 200-gpm centrifuges, and a truck load-out area for the dewatered sludge. Capacities of the sludge handling equipment [5] are summarized in the following table:

TABLE 9.14: 1998 Design Criteria for Existing Sludge Handling Equipment

Equipment	Design Criteria	
WAS Pumping	Pump capacity, gpm	2 ea at 175 gpm
Lime Slurry Equipment	Lime Slurry Tank, gal Lime Slurry Pumps	16,800 2 ea at 25 gpm
Sludge Stabilization Equipment	Stabilization Tank Volume, gal Stabilized Sludge Mix/Transfer Pump, gpm	6 ea at 56,000 gal 1 at 350, 1 at 500
Solids Dewatering	Centrifuge Feed Pumps Centrifuges Feed Concentration, % raw / lime stabilized Centrifuge Min. Cake Solids, % raw / lime	2 ea rated at 175 gpm 2 ea at 200 gpm max 1.0 – 2.0 / 1.3 – 2.7 20 / 27

The existing sludge handling process was evaluated using projected maximum month waste solids generation rates and a WAS concentration of 0.8%. If the City continues to dewater raw solids five days per week, the existing storage tank has adequate capacity for storage of WAS through the design year 2060.

If the City utilizes the lime stabilization system, the sludge storage tank has the capacity to treat approximately 100,000 gallons of WAS per day. This is adequate for the projected WAS flow of 91,000 gallons per day in 2030. The lime slurry tank has a storage volume of 16,800 gallons. This tank should provide a minimum of one month's storage through the year 2060 based on the lime dosing requirements to produce Class B biosolids. The lime slurry pumps are also adequately sized to meet 2060 design conditions.

The centrifuges and feed pumps were evaluated based on dewatering five days per week using a maximum feed rate of 175 gpm. At 2030 maximum month solids generation rates, two centrifuges will be required to dewater solids within an 8-hour day so there will be no system redundancy. As the City's solids generation rates increase, it is recommended that the condition of the centrifuges be evaluated for remaining useful life. If the City can continue to use the existing equipment, an additional dewatering unit should be installed to provide redundancy. If the equipment is nearing the end of its useful life, it is recommended that the City investigate several types of mechanical dewatering equipment to determine the most cost-effective approach. Given space limitations in the existing building, two larger units will likely be more feasible than three smaller units to provide the necessary system redundancy.

The cost of landfilling sludge will continue to consume a large portion of the solids handling budget. Landfill records provided by the City show the City currently disposes of 13 to 14 wet tons of solids per day, five days per week. If the City does not modify its sludge handling procedures, the amount of sludge to be disposed per day (five days per week) is estimated to increase to 18 wet tons in 2030 and 22 wet tons in 2060. At the current landfill tipping fee of $47 per ton (not considering inflation or increases in landfill tipping fees due to expansion or regulatory costs), the City's sludge disposal cost would increase to approximately $220,000 per year in 2030. The City will be required to purchase additional trucks for hauling or add on-site sludge storage, and to add staff to operate dewatering equipment.

If a new sludge management process is required in the future, placing the lime stabilization process back in use is one option. While it is possible to dewater to a higher solids concentration with lime stabilized sludge, there are several disadvantages that should be considered:

- More frequent maintenance of the centrifuges would be expected due to scaling,
- The cost of the lime will offset any savings at the landfill,
- The addition of lime will increase the weight of the solids for disposal,
- High alkalinity may render the biosolids unsuitable for land application to farm crops,
- Lime stabilized biosolids typically have decreased nutrients since ammonia-nitrogen is volatilized during the process and less organic nitrogen is available for plant use [4].

Chapter 11 will describe alternate sludge management methods for sludge stabilization, dewatering and disposal that will reduce the quantity of biosolids produced, improve the quality of the final end product, reduce disposal costs, and provide a more environmentally friendly alternative to landfilling.

9.3.8 Electrical and SCADA

Emergency Power

The emergency generator is an 875 KVA engine driven standby generator with an automatic transfer switch. The generator will provide energy to the entire plant during a power outage by providing power to MCC 1, 2, 3, 4, 6, and 7.

Power and Utility Systems

Keller Associates recommends that an energy audit be conducted at the Ashland WWTP to determine if changes at the plant could result in energy savings. The following tasks would be performed as part of the energy audit.

- Perform a brief walk-through survey of the facility to become familiar with its construction, equipment, operation, and maintenance.
- Meet with the owner and operators to learn of special problems or facility needs. Determine if any maintenance problems and/or practices may affect efficiency.
- Perform a space function analysis. Determine if efficiency may be affected by functions that differ from the original functional intent of the building.
- Description and analysis of the energy-using systems of the building, resulting from on-site observation, measurement, and engineering calculations, including:
    - Envelope
    - Lighting
    - HVAC
    - Domestic hot water
    - Conveying systems
    - Other systems
- Perform a rough estimate to determine the approximate breakdown of energy use for significant end-use categories, including weather and non-weather related uses.
- Identify low cost/no-cost changes to the facility or to operating and maintenance procedures and determine the savings that will result form these changes.
- Identify potential capital improvements for further study, and provide a summary report including initial estimates of potential costs and savings.

9.4 EVALUATION OF OPERATIONS

9.4.1 Staffing

The treatment plant is staffed 10 hours a day, 7 days a week. Most of the laboratory work and the maintenance work is done in-house. According to Ashland Public Works staff, there are an equivalent of 5.4 full-time equivalent (FTE) employees assigned to the treatment plant. Dewatering and hauling sludge utilizes 75% of one employee's time. Positions include Wastewater Treatment Plant Lead Operator and Wastewater Treatment Plant Operator/Technician.

In addition to the full-time employees designated for wastewater treatment, the organization chart for the Ashland Public Works Department shows 0.8 employee for Wastewater Treatment and Reuse (0.4 FTE collections and 0.4 FTE for treatment). The current job description for the Wastewater and Water Reuse Supervisor includes the following wastewater-related functions (this employee is also responsible for the City's storm drain system and maintenance programs):

- Supervise work crews involved in the City's wastewater treatment plant and water reuse operations, construction, pre-treatment programs, maintenance and repair of wastewater collection systems and related pump stations.
- Prepare all necessary regulatory reports.
- Review, recommend and monitor related budget and expense items.
- Keep up-to-date on federal/state regulatory requirements for wastewater treatment, collection, recycled water, biosolids storage, and recycled water applications.

A revision to this organizational structure was presented in Section 2.2 of this report. This revision involves hiring a collections system lead or supervisor who could assume the supervisory responsibilities of the collection system. This change would allow for additional time to be spent by the treatment plant supervisor in attending to the treatment plant operations and pretreatment program.

The standard for many years for evaluating WWTP staffing needs has been the 1973 EPA manual [6] entitled "Estimating Staffing for Municipal Wastewater Treatment Facilities". Data used to develop this manual included a survey of staffing levels for 35 small to large wastewater treatment facilities across the country. Staff operation and maintenance hours are projected based on the plant design capacity and treatment processes employed, with adjustments for local conditions such as plant layout, treatment level, type of effluent limits, staff training, type of waste stream treated, etc.

With the continued development of new technologies and treatment processes, the need for an update to the EPA manual has become increasingly apparent. In November 2008, the New England Interstate Water Pollution Control Commission (NEIWPCC) published "The Northeast Guide for Estimating Staffing at Publicly and Privately Owned Wastewater Treatment Plants" [7]. The staffing estimates in the NEIWPCC manual are based on a pilot study of 25 small to large treatment facilities in New England.

Both the EPA and the NEIWPCC guides were used to evaluate Ashland staffing requirements. Because it includes more current treatment processes and laboratory practices, the NEIWPCC guide is considered to provide a more accurate estimate of staffing needs at the Ashland plant. It should be noted that neither manual addresses staff needs not directly related to operation and maintenance; manhours needed for Ashland public works staff to keep current with recycled water issues, pre-treatment, and evolving federal and state regulatory requirements have been estimated separately.

Based on the above analysis (NEIWPCC staffing worksheets are included in Appendix E), the following summary of full-time employees (FTE) recommended for the existing wastewater treatment utility was developed. Manhours for sludge handling estimated by the NEIWPCC worksheet were reduced to reflect actual conditions at the Ashland plant.

▪ Treatment plant operations	3.1 FTE
▪ Treatment plant maintenance	1.6 FTE
▪ Laboratory	0.9 FTE
▪ Sludge handling	0.8 FTE
▪ Yard work	0.4 FTE
TOTAL	6.8 FTE

The City currently has 5.4 personnel designated for the wastewater treatment system, which is less than recommended in the summary above. Though the plant is well-operated at current staffing levels, this indicates that the wastewater utility is currently understaffed by a little more than one FTE. While this may reflect a higher productivity level than the 1500 manhours per person used in the guidance (32.5 hours of productive work per week, 29 days off for holidays/vacation/sick leave), it may also mean that some less pressing activities such as routine preventive maintenance are being postponed. The wastewater treatment utility would benefit from the addition of an additional full-time employee.

In addition to personnel to operate and maintain the wastewater system, engineering staff is needed to meet regulatory requirements (providing DEQ permit and funding support, staying current with recycled water issues, pre-treatment, and the evolving federal and state regulations) and to assist in administering projects proposed in the capital improvement plan. These activities are separate from the functions of Wastewater and Water Reuse Supervisor (e.g. supervision, reports, budget) that are included in standard treatment plant operations. An additional 0.5 FTE is recommended to fill this role. One additional FTE engineering position, allocated equally between the wastewater and water funds, would satisfy this requirement.

Further increases in staffing are anticipated to be needed to accommodate changes to the permit and/or treatment processes. Staffing needs for the recommended alternative are included in Chapter 12.

9.4.2 Operational Theory

The overall goal of process operation should be to operate the plant in a conservative manner to give a robust process that easily meets the discharge requirements. Effectively, this means that the plant should always be operated with a safety factor. In turn, operating conservatively results in an easier process to operate and control while ensuring it can take care of swings in influent characteristics or possible upsets. Efforts to optimize can push the plant towards the operating edge and can result in periodic violations of discharge limits or a greater chance for process upsets.

In terms of optimization, the City should select the goals to achieve. The next step is to develop programs to accomplish the selected goals. The areas most feasible for optimization are energy and chemical use, and actions may include:

- Control aerators (change speed and/or cycle aerator operation) to maintain a desired DO profile through the aeration basins.
- Control DO to minimize aerator energy use. The City may be able to reduce the DO in the basins without impacting nitrification or sludge quality.

- Adjust SRT to ensure reliable process performance in terms of nitrification and sludge/effluent quality while minimizing energy use.
- Assess possible reduction in sludge quantity by increasing SRT while also measuring increases in power use and/or changes in effluent/sludge quality.
- Develop oxidation ditch operating conditions to maximize denitrification.
- Adjust alum addition to maintain the desired effluent P and minimize alum use.
- Evaluate polymer addition for sludge dewatering in terms of quantity/cost/cake solids

Each of these areas for optimization would require the City to prepare a test protocol and data analysis plan. It is recommended that the City include selected optimization programs as part of their future operations and maintenance program.

9.4.3 Testing Practices

The City should review their laboratory procedures by making a list of all the tests that they do in house with the testing procedure used. In addition, a list of tests that are sent out should be prepared with the test method used by the laboratory. These lists can then be analyzed for compliance with the current list of EPA approved methods and equivalent approved Standard Methods for each constituent. The current list of EPA approved testing methods and approved Standard Methods can be found at:

http://water.epa.gov/scitech/swguidance/methods/methods_index.cfm

9.4.4 Safety

The City of Ashland has the basis of a safety program in place at the WWTP that is part of an overall Safety program for City's Public Works Department. The safety program at the WWTP includes several safety plans, including a Hazard Communications Plan, Laboratory Safety Plan, Chemical Hygiene Plan, Centrifuge Safety Plan, Lock-out/Tag-out Policy, Confined Space Entry Procedures, MSDS notebook, and Standard Operating Procedures for Collection System Personnel. A New Employee Health and Safety Checklist is used to provide a written record that each new employee has been given safety instructions as required for the employee's duties.

Keller Associates recommends that the safety plans be modified to develop a coordinated safety program. The Hazard Communications Plan could be developed into the overall safety plan with references to the Laboratory Safety Plan, Chemical Hygiene Plan, Centrifuge Safety Plan, Lock-out/Tag-out Policy, Confined Space Entry Procedures, MSDS notebook, and Standard Operating Procedures for Collection System Personnel. The Chemical Hygiene Plan should be modified to more closely follow Section 4 of the Laboratory Safety Plan which provides details of what is in the Chemical Hygiene Plan and to eliminate duplication of information that is already in the Laboratory Safety Plan.

Additional specific safety plans should be developed, for instance, for the membrane system (including chemicals used on the membranes), UV system, and Headworks. The New Employee Health and Safety Checklist should be updated to add the Laboratory Safety Plan, Lock-out/Tag-out Policy, Confined Space Entry Procedures, Chemical Hygiene Plan, Standard Operating Procedures for Collection System Personnel, and new plans as they are developed.

The safety plans should be edited to stream line the program and to eliminate duplication. Plant personnel should know which plan contains the safety information they are looking for and if they do not know, they should be able to find where to look in the Hazard Communications Plan. Eliminating duplication reduces the effort it takes to update the plans and reduces the potential for errors.

Keller Associates recommends that the City review and update the plans every year or two.

9.5 CARBON FOOTPRINT

To prepare for future reporting requirements (pending DEQ development of a quantification protocol), the City of Ashland requested an estimate of emitted greenhouse gases (GHG) for the city's wastewater treatment plant. A report was prepared in 2011 to quantify individual GHG emission sources associated with the facility, and discuss their likely inclusion in a GHG estimation protocol (see Appendix E). Operating conditions at the facility from 2008 to 2010 were used to estimate the annual GHG emissions. The report addressed the plant GHG emissions in four main areas:

- Biological treatment → CO_2 and N_2O emitted from the oxidation of the wastewater
- Electrical consumption (GHG emissions from generation of electricity used)
- Chemical consumption (GHG emissions from production of chemicals used)
- Solids handling, including transportation of the solids to the landfill and GHG emissions from solids decomposition.

The report estimated the annual GHG emissions from all sources listed above to be 2,690 metric tons carbon dioxide equivalent/ yr (CO_2Eq / yr). While this value exceeds the 2,500 metric ton CO_2Eq / yr reporting threshold, it includes numerous sources that would likely be attributed to other entities under the proposed reporting requirements. Assuming only on-site, non-biogenic emissions (from denitrification and solids transportation), the estimate would be much less - approximately 75 metric tons CO_2Eq / yr - and well below the reporting threshold. Nonetheless, efforts to minimize or offset energy use may be desirable if cost-effective. Where possible, the City should take advantage of grant opportunities and public-private partnerships that may become available to address carbon emissions.

9.6 SUMMARY OF EXISTING PLANT & DEFICIENCIES

The results of the plant capacity evaluation are summarized in Table 9.15. As shown in the table, the components that will need to be upgraded or replaced prior to 2030 are the manual bar screen, mechanical bar screen (due to expected wear), grit chamber, oxidation ditch (one new), RAS pumps, membrane replacement (as scheduled in Section 9.4.6), UV disinfection, and outfall piping. The components that will need to be upgraded or replaced between 2030 and 2060 are the influent pumps, centrifuge, and secondary clarifiers (one new).

Based on the expected requirements of the new NPDES permit and Table 9.14, the priorities for the treatment plant are:

Priority 1 (constructed by 2015-2020)

1A – required for permit/Agency compliance

- Effluent temperature upgrade.
- Fish screen for outfall (existing or relocated outfall).
- Add UVT monitor.

1B – recommended to address capacity and equipment condition issues

- Provide a 6-inch trash pump as a backup for the influent lift pumps. The existing influent pumps were recently redone and have sufficient capacity for current flows.
- Membrane replacement per schedule in Table 9.11 (as required).
- Additional biological capacity (options discussed in Chapter 11). Adding polymer/alum addition during peak flow event to aid settling or an aggressive I/I program to reduce the peak flows could delay the biological capacity upgrades by a few years.
- Option to meet DO limit (unless the limit is revised with the new permit and new outfall location).
- Replace RAS pumps with larger pumps

Priority 2 (constructed by 2025-2030)

- Membrane replacement per schedule (the 2023 upgrade will require piping, blower, membrane pump, and chemical treatment equipment upgrades).
- Additional UV reactors and upgrade existing panels.
- Replace membrane feed pumps.
- Membrane replacement per schedule.
- Replace mechanical bar screen.
- Upgrade or replace grit removal system.
- Replace clarifier mechanism in clarifier #2
- Replace equipment in existing oxidation ditches

Priority 3 (constructed between 2030 and 2060)

- Replace influent lift station pumps with larger capacity pumps.
- Membrane replacement per schedule.
- Add a third centrifuge.
- Add a fourth clarifier
- Replace clarifier mechanisms in clarifiers 1 and 3

Effluent temperature upgrade options will be discussed in Chapter 10 and treatment options for the rest of the Priority 1 items will be discussed in Chapter 11. The adequacy of the existing treatment plant site to meet future growth expansion will be discussed in Chapter 12.

TABLE 9.15: Summary of Treatment Capacity by Plant Process

Component	Criteria	Capacity, mgd	When Reached
Influent Pumps 3 ea 3150 gpm	63% pumped[1], 2 pumps peak hour	8.06	2045
Bar Screens Mechanical Manual	- Max V = 5 fps, flow depth 2.5 ft[1]	13.5 8.7	beyond 2030 2011
Grit Removal	Particle size removed[3] 0.21 mm (46,300 gpd/sq ft) 0.25 mm (58,000 gpd/sq ft) 0.33 mm (65,500 gpd/sq ft) 0.46 mm (87,000 gpd/sq ft)	 9.1 11.37 12.84 17.05	 2011 2023 beyond 2030 beyond 2060
Piping Hdwks-Ditch	Not submerge stop log weir in hdwks	>15.27	beyond 2060
Oxidation Ditch (2 ea)	MCRT 14 days, winter max month[4]	3.76	2015
Piping Ditch-Clarifiers	Not submerge weir in ditch[5]	12.4	beyond 2030
Clarifiers (3 ea)	Solids loading 36 ppd/sq ft peak hour	11.87	beyond 2030
RAS pumps 3 ea 1350 gpm	36-56% RAS rate, 2 pumps - pk day or - pk week	6.5 6.5	now beyond 2030
UV Disinfection	One train in operation, max month Two trains, not overflow clarifier, pk	4.25 11	2030 2020
Membrane Filtration (May 1 to November 30 each year)	2010 membranes, MMDWF 2011 membranes, MMDWF	2.75 2.87	2012 2017
Membrane Feed Pumps 4 ea 1050 gpm	3 pumps in service	4.54	2027
Alum pumps	One pump in service (0.5 gpm)	4	2025
Outfall Piping	Not submerge UV weir, peak day	11.23	beyond 2060
Sludge Storage Tank Centrifuges 2 ea 175 gpm	Weekend storage (3 days) One unit in service Max 8 hrs 5 days/wk		beyond 2060[6] 2014[6]

Notes:
1. Estimated from flow records
2. Total influent flow, assuming 63% pumped.
3. Theoretical required overflow rate for removal of particles ≥ sizes shown, Ref. [8]
4. For nitrification (as discussed in Section 9.4.3.1.1)
5. Total flow from ditch to clarifiers with RAS and recycle = 20 mgd at 12.4 mgd influent flow
6. Based on WAS production per Table 9.12

References

1. Oregon Department of Environmental Quality: *Oregon Standards for Design and Construction of Wastewater Pump Stations,* May 2001.

2. Carollo Engineers: *City of Ashland Update to WWTP Facilities Plan and Permitting Evaluation*, draft June 2008.

3. Metcalf & Eddy, Inc.: *Wastewater Engineering: Treatment and Reuse, 4th ed.*, McGraw-Hill, New York, 2003.

4. Water Environment Federation: *Design of Municipal Wastewater Treatment Plants, 5th ed., Volume 3: Solids Processing and Management*; Manual of Practice No. 8, McGraw-Hill, New York, 2010.

5. Carollo Engineers: *City of Ashland Wastewater Treatment Plant Operations & Maintenance Manual*, 1998.

6. US Environmental Protection Agency, Office of Water Program Operations: *Estimating Staffing for Municipal Wastewater Treatment Facilities*, Washington, D.C., March 1973.

7. New England Interstate Water Pollution Control Commission: *The Northeast Guide for Estimating Staffing at Publicly and Privately Owned Wastewater Treatment Plants*, Lowell, MA, November 2008.

8. Water Environment Federation: *Design of Municipal Wastewater Treatment Plants, 2nd ed., Volume I,* Manual of Practice No. 8 , 1992.

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10.0 EFFLUENT DISPOSAL ALTERNATIVES

10.1 BACKGROUND

During certain periods of the year, the wastewater effluent from the City's treatment plant accounts for a significant portion of the flow in Ashland Creek and Bear Creek. Higher effluent temperatures can raise the temperature of the creek and negatively impact aquatic habitat. DEQ will include new excess thermal load limits when the NPDES permit for the City of Ashland's wastewater treatment plant (WWTP) is renewed to address the waste load allocation in the TMDL. DEQ may also include a temperature limit to address local impacts of the thermal plume to aquatic habitat.

Current effluent temperatures have the potential to exceed allowable levels for the May through October period. Keller Associates reviewed the previous five years of temperature and flow data, and determined that there is an existing excess thermal load of approximately 44 million kcal/day (critical month is October). This is anticipated to increase to approximately 53 million kcal/day by 2030. Similar calculations by DEQ correspond to Keller Associates' calculation results [1].

Reducing the excess thermal load from the Ashland WWTP is important in meeting target downstream temperatures in Bear Creek. An evaluation of wastewater disposal options completed in 2009 [2] looked for strategies or alternatives to address the excess thermal loads. This master plan builds upon the work previously completed. Additionally, since the completion of 2009 evaluation, guidelines for evaluating the local (near field) impacts and temperature trading programs have been more fully developed by the State.

Representatives from DEQ, the City of Ashland, Keller Associates, Oregon Department of Fish and Wildlife (ODFW), and other stakeholders met on several occasions to better define the impacts of Ashland's wastewater discharge. DEQ completed a thermal plume analysis for continued discharge to Ashland Creek as well as discharge to Bear Creek. Because of concerns with near field spawning impairments, thermal shock, and migration blockage, it is unlikely that continued discharge into Ashland Creek would be permitted without first significantly cooling the effluent [1]. Relocating the outfall to Bear Creek would eliminate concerns of thermal shock and greatly mitigate other near field impacts. Based on 3D modeling completed by DEQ, a side bank discharge would allow discharge to Bear Creek without impairing spawning. However, based on historical data, there still remains a potential for migration blockage during the month of September [1].

Keller Associates' scope of work for this study was to update the evaluation for the three most promising disposal alternatives. However, because of continued interest on the part of the City and new developments, six alternatives were evaluated in more detail. The findings of this evaluation follow.

10.2 RECYCLING OPTIONS

Recycling options include those options that recycle treated wastewater. Land application of wastewater effluent during the growing season could reduce or eliminate the discharge of thermal loads to Ashland and Bear Creek during critical periods. Another benefit of recycling is that the treatment process is likely to be less affected by future changes in regulations requiring increasingly more stringent levels of treatment for discharge. For example, had the City of Ashland chosen to land apply their effluent rather than remove phosphorous via membrane filtration 10 years ago, they would not now be faced with addressing temperature concerns. As discussed in Chapter 2, additional items on the horizon that may affect future discharge requirements for the plant include 1) stricter ammonia limits, 2) Oregon Senate Bill 737, which addresses pharmaceuticals, and 3) aquatic life and human health criteria (e.g. potential copper, phthalates, and others). In addition to regulatory benefits, recycling water has the potential to offset potable water demands and make better use of available water resources.

Maintaining stream flows has been a priority to the City in the past. One of the drawbacks with any recycling alternative that involves removing the existing discharge flow from Ashland Creek is that the recycled water would not be available for use for potential downstream users or to create higher flow conditions for aquatic habitat.

From a water rights standpoint, the City of Ashland is not required to keep their effluent discharge in the creek. However, according to ORS 537.132, the following would occur if the City were to move forward with removing their flow for recycle purposes:

- The Department of Water Resources (DWR) would notify affected users if discharge from Ashland WWTP to Ashland Creek were to cease (this because Ashland has discharged for more than 5 years and the WWTP discharge may at times make up 50% or more of the flow).
- An affected downstream water right holder would need to demonstrate to DWR that the “cessation of discharge by the municipality substantially impairs the ability to satisfy a water right. . .” and if this person is successful, they would get preferential use of the recycled water.
- The City is not required to incur additional expenses (beyond a more favorable alternative) to deliver water to the affected person desiring the recycled water.

10.2.1 Option 1: Recycling Water on Imperatrice Ranch Property

The City has property north of I-5 (Imperatrice Ranch) that could be used for crop irrigation using effluent. A conveyance pipeline crossing Ashland Creek was constructed when the City was considering a project in 1997 for biosolids application, effluent storage and irrigation on the property.

Due to steep terrain and other limiting features (Talent Irrigation District canal, wetland swale), portions of the Imperatrice site are not useable for irrigation. Limiting irrigation to slopes less 20% and providing necessary buffer zones for the canal, swale and property lines provides a usable irrigation area of 412 acres for Class C effluent, or 433 acres for Class B effluent (smaller buffer to property lines) [3].

One of the primary benefits the City would realize with recycling water on the Imperatrice Ranch property is that the water rights currently used there could be transferred and used as additional water supply for the potable water system.

Two recycling options are summarized for the Imperatrice Property – Option 1A includes maximizing the total amount of water recycled on the property, and Option 1B includes recycling only the amount necessary to offset the existing water rights. Regardless of the disposal option selected by the City, Keller Associates recommends that the City work with DEQ so that future NPDES permits allow for recycling of treated effluent.

Option 1A: Maximum Recycling on Imperatrice Property

The potential for thermal shock and migration blockage in Ashland Creek would be averted by eliminating discharge from June through October, and potential salmonid spawning impairment from thermal discharges would be prevented by reducing/eliminating discharge during November and March through May. Storage volumes for this option were determined based on irrigating as much land as possible without supplemental water, and discharging excess to the creek only to the extent that impairment of salmonid spawning is avoided. This results in limited discharge during March, April and November, and discharge of stored excess during January and February when creek temperatures are low enough to easily accommodate the thermal load.

Alfalfa, pasture grass, and grass seed are potential crops; pasture grass and grass seed use more water than alfalfa and thus have lower storage requirements. Based on average net irrigation requirements and 70% irrigation efficiency, the acreage available on the Imperatrice property is sufficient to use 442 MG or 492 MG if planted to grass seed or pasture grass, respectively. Since the amount applied to crops is less than influent flows to the WWTP, the remainder would be discharged. At year 2030 flows (average 2.59 mgd), storage would be needed to provide sufficient volume during June, July and August. Additional storage volume would allow excess flows to be stored for discharge in the winter.

An irrigated area of 433 acres of pasture grass would handle (without supplemental water) up to 2.77 mgd, with a storage volume of 138 MG (see water balance in Appendix E). A total of 512 MG would be discharged to the creek from November through April. The same acreage in grass seed would handle year 2030 flows with a storage volume of 139 MG and 496 MG discharged (November through April).

The estimated project cost for Option 1A is approximately $10.8 million. Eliminating the need for phosphorus removal required for surface discharge would result in annual savings of $71,000 a year for alum. An estimated additional $100,000 potential annual savings could be realized in energy and chemical (sodium hypochlorite and citric acid) with elimination of the membrane operation. However, it is understood that the public perception may require the continued use of the membranes. If membrane operation were eliminated as part of the recycling option, the combined savings ($171,000) would more than offset the estimated $113,000 annual costs of pumping to storage on the site and from storage to irrigation. Though effluent quality would still need to be monitored with the recycling option, testing requirements (and related costs) are expected to decrease with the elimination of discharge during critical times.

Option 1B: Partial Recycling on Imperatrice Property

Keller Associates also evaluated an alternative that would recycle just enough effluent to offset the existing 424 ac-ft of irrigation rights on the Imperatrice property, and maintain the remaining flow in the stream. This scenario would allow the water right to be transferred to the City's potable water system and would also allow continued discharge to the creek. However, under this scenario, the temperature requirements of the TMDL would have to be met by employing other improvement alternatives.

To offset the 424 ac-ft water right, enough water would need to be supplied to irrigate approximately 136 acres of land. The amount of storage required would depend on how much is discharged during specific periods of time. If minimum storage were provided, then close to half of the existing discharge during July and August would be used for irrigation, while the balance would be discharged to the creek. With additional storage, discharges could be eliminated during specific periods and restricted during others to eliminate the need for additional treatment to reduce thermal and phosphorus loads for discharge. (Existing alum and membrane treatment would still be required.) This approach would require close monitoring to consistently meet the discharge limits.

If the City's primary objective is to maximize the discharge available during critical periods for aquatic habitat while offsetting the water right, this alternative could be adjusted to include increased storage during high stream flow periods and continued effluent discharge during low flow and spawning periods.

The estimated project cost for Option 1B, not including a cooling component, is approximately $5.3-8.9 million (includes 6.5-168 MG storage). Since discharge to the creek would continue, all the costs for phosphorus removal discussed above would be included in the annual operation and maintenance cost of this option. In addition, there would be the added costs (estimated $35,000/year) of pumping to storage on the site and from storage to irrigation.

10.2.2 Option 2: City-Wide Recycling

City-wide recycling (on parks, golf courses and other public spaces) of effluent was evaluated as part of the water master plan as an alternative to reduce potable water use [4]. From an implementation standpoint, Keller Associates would envision this being phased in over many years. Recycling on City property could be phased with agricultural recycling on the Imperatrice property. Since the distribution system for city-wide recycling of effluent may be extensive, the cost for implementation will exceed that of the option to apply all effluent to the Imperatrice property.

In addition, storage during shoulder seasons would still be required for temperature TMDL compliance (storage location could be at Imperatrice property).

10.3 RELOCATED DISCHARGE OPTIONS

10.3.1 Option 3: Discharge to Talent Irrigation District (TID)

This alternative would involve discharging the City's effluent into the TID irrigation system. The likely discharge location would be Talent Canal, which has a capacity of 35 to 45 cfs. According to the District, the Talent Canal services approximately 3500-4000 acres. One of the benefits of this alternative would be the reduced chemical requirements needed to remove phosphorous, because most of the water would be recycled or land applied downstream. This alternative would mitigate concerns about near field impacts to aquatic habitat, and would reduce the thermal load requirements to the extent that the effluent is reused downstream.

On October 5, 2010, representatives from Keller Associates and the City met with TID board members to further discuss this alternative. The following concerns would need to be addressed before approval could be obtained for this option:

- Real and Perceived Concerns of Receiving Effluent – The TID currently does not receive any treated effluent. The district has a number of patrons who have already expressed deep concerns about receiving Ashland's effluent.

- Not Wanting Any Additional Chemicals – downstream farmers have already fought with the district to eliminate other chemical additives for moss control in the district's canals. This concern is heightened by the number of organic farmers.
- Approval of Patrons – Because of the controversial nature of this alternative, the board indicated that they would want their patrons to weigh in on the matter, possibly even having a vote of the patrons. Educating the public, addressing their concerns, and obtaining approval at this time would require a great deal of effort with an uncertain outcome. This would also require many months to do.
- Removal of flow from Ashland Creek. ODFW has expressed a desire to keep as much flow in Ashland and Bear Creek as possible. There may also be other downstream water right impacts that would need to be addressed by removing discharge.
- Other Potential Additional Regulatory Requirements
- Additional Maintenance Requirements:
    - The district's water chemistry is very sensitive to temperature. Even a small increase in temperature or phosphorous is believed to increase the potential for moss growth in their system.
    - Receiving water during the shoulder seasons – particularly October and November – would adversely affect district operational practices. The City would need to plan on being able to store their effluent during these periods.
    - Additional fish screening may be required by DEQ. If these screens are required at outfalls, this could result in more maintenance to the district.

In addition to needing to address the above concerns, this option would also require that Ashland quantify and then mitigate excess thermal loads corresponding to the portion of flow that is not reused downstream. Given the number of issues and potential road blocks, Keller Associates recommends that this alternative not be pursued at this time. However, it may be that in the future as public perception changes and if drought conditions make the water resources more valuable, it may be beneficial to reevaluate this alternative.

10.4 OPTIONS FOR CONTINUED DISCHARGE TO ASHLAND/BEAR CREEK

10.4.1 Option 4: Cooling Tower / Heat Exchanger / Chiller

Background

A cooling tower could be used to reduce the temperature of the effluent through evaporation to reduce the effluent temperature. The primary benefit of the cooling tower alternative is it addresses the temperature requirements without concern for off-site improvements, water rights, potential reduced flows in the stream, or potential compliance schedules. However, this alternative would be an energy-consuming option because the effluent would have to be pumped to the top of the cooling tower and a large fan would be operated continuously. This option was determined to be a viable alternative by Carollo in an evaluation of disposal alternatives completed in 2009 [2]. However, as noted in the Carollo report, a cooling tower could not meet the limits all the time and a chiller would have to be added to reduce the temperature of the effluent to meet the limits during some days.

In a cooling tower, air is simultaneously drawn up through the tower in the opposite direction from the water flow. A small portion of the water is evaporated, which removes the heat from the rest of the water. Warm, moist air is discharged to the atmosphere and cooled plant effluent is discharged to the creek.

There are two types of cooling towers that would be considered for Ashland: open loop and closed loop, both using plastic media. In the open loop design, the plant effluent would be pumped to the water distribution system at the top of the cooling tower for distribution evenly across the top of the media. In the closed loop design, the plant water is kept separate from the cooling water. The advantage to the closed loop system is that the cooling water is separate from the wastewater, and anti-scaling chemicals could be added to prevent scaling in the tower without affecting the effluent water quality.

There are two types of closed loop designs. In one, the plant effluent would be pumped through coiled tubes from the top of the cooling tower to the bottom of the cooling tower. Cooling water would be pumped to the water distribution system at the top of the cooling tower for distribution evenly across the top of the media. In the second design, the layout is the same as the first except that the cooling water is put through a plate heat exchanger to further cool the cooling water. For larger systems, like that needed for Ashland, this closed loop option is less expensive.

The cooling tower would not have to be operated year-round. Its months of operation would be spring to fall. Effluent temperature limits are a daily maximum of 13 ^{o}C from October 15 to May 15, and a daily maximum of 18 ^{o}C from May 16 to October 14.

The effluent temperature regulations allow for exceedence of the effluent limits when the daily maximum temperature exceeds the 90th percentile of the last ten years of the maximum daily temperature 7-day average. Based on the last 10 years of temperature data from the Medford Airport (closest weather station to Ashland), the 90th percentile maximum daily temperature is 93.3 ^{o}F.

When the cooling tower cannot meet the effluent limit, a chiller would also need to be used to reduce the temperature of the effluent lower than can be done by evaporation alone. A chiller uses condensers and electrical energy to obtain the additional cooling required similar to a refrigerator.

The Oregon Administrative Rules (OAR) provide some relief for meeting the temperature limits with an air temperature exclusion (340-41-0028(12)(c)) and a low receiving stream flow exclusion (340-41-0028(12)(d)). The air temperature exclusion provides that effluent temperatures that exceed the limit are not considered violations when “the daily maximum air temperature exceeds the 90th percentile value of annual maximum seven-day average maximum air temperatures calculated using at least 10 years of air temperature data.”

Analysis

<u>Continuous Discharge</u>
A cooling tower can continuously cool the effluent wastewater to approximately 5^{o}F above the atmospheric wet bulb temperature. During each day the wet bulb temperature increases and decreases with the air temperature. The historical climate data for the Medford airport provided daily minimum, maximum, and average wet bulb temperature. Using this historical climate data from January 1, 1999 to August 30, 2010, the plant effluent temperatures can be calculated for the minimum, maximum, and mean wet bulb temperatures. Plots showing the estimated cooled WWTP effluent temperature at the mean, minimum and maximum wet bulb temperatures, respectively, are shown in Charts 10.1, 10.2, and 10.3.

The mean wet bulb temperature graph is based on the average effluent temperature, while the maximum wet bulb temperature graph shows the maximum daily effluent temperature, and the minimum wet bulb temperature graph shows the lowest daily effluent temperature achievable using a cooling tower. These charts show that, using only a cooling tower and continuous discharge, there would have been a significant number of temperature violations over the last 11 years. When the temperature exclusion discussed above is considered, there still would have been more the 40 violations over the last 11 years.

CHART 10.1: Calculated WWTP Effluent Temperature at the Mean Wet Bulb Temperature

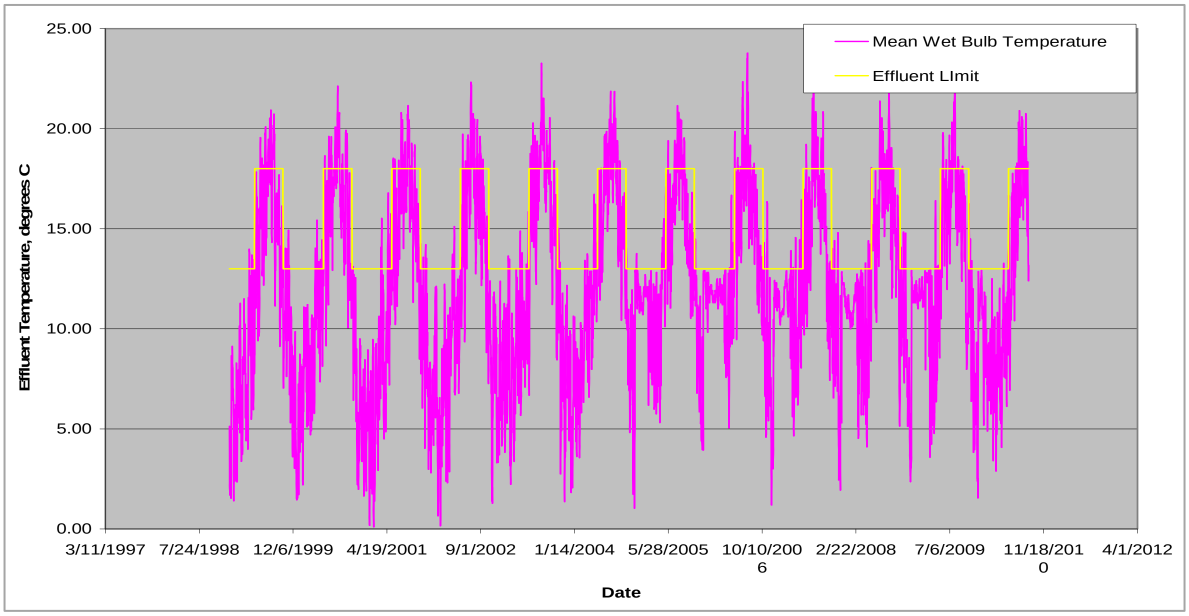


CHART 10.2: Calculated WWTP Effluent Temperature at the Minimum Wet Bulb Temperature

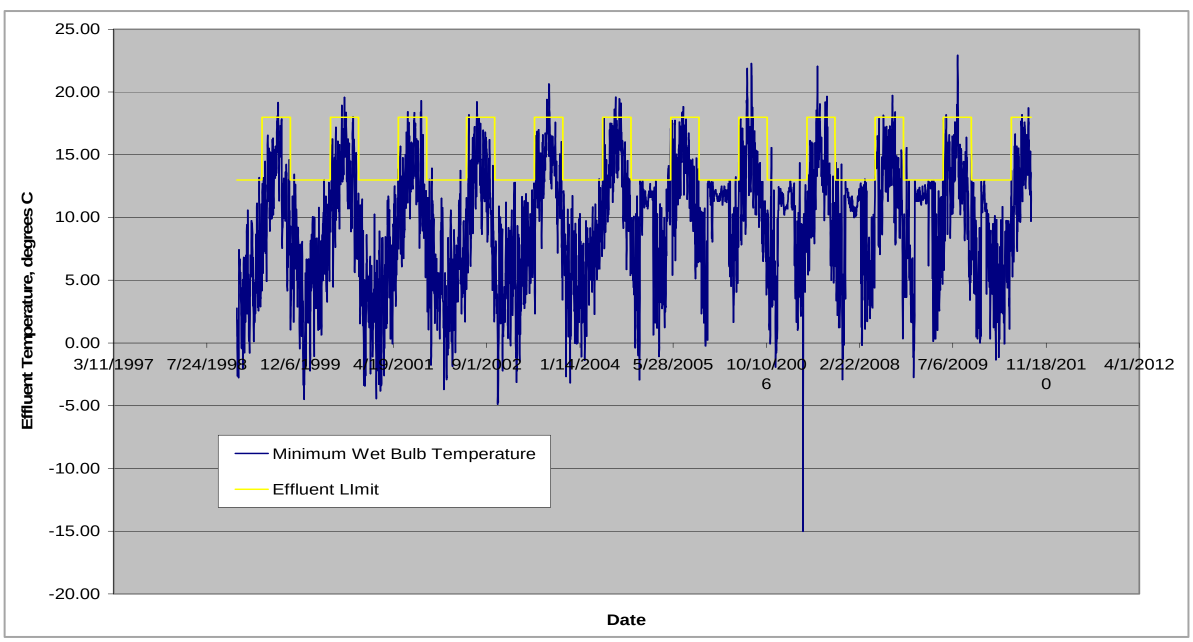
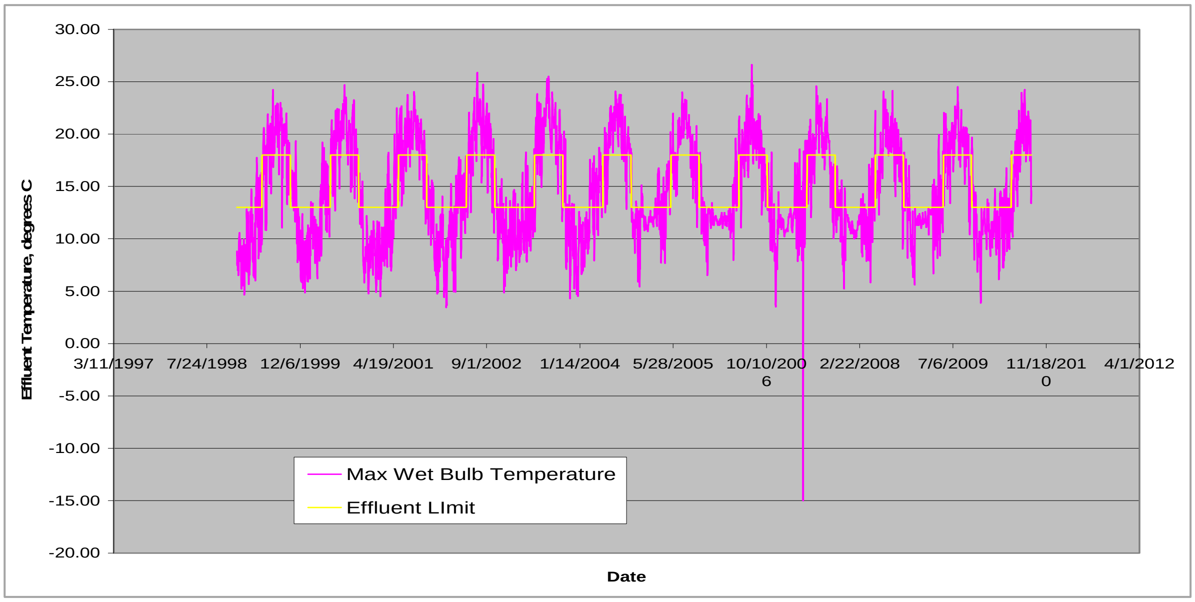


CHART 10.3: Calculated WWTP Effluent Temperature at the Maximum Wet Bulb Temperature



<u>Storage</u>
In order to meet the effluent temperature limits with a cooling tower, Keller Associates looked at using storage to cool plant effluent only during the night when the air temperatures are lower. A discharge period of 12-hour period was assumed. The storage would be sized for half the peak flow between April and October, as some of the potential violations for continuous treatment are in the shoulder periods. The estimated peak daily flow during this period is 5.5 mgd, and thus the storage tank would be sized at approximately 3.0 million gallons.

The Oregon Department of Fish and Wildlife (ODFW) has indicated that they would want the City to continue to provide continuous discharge to maintain a more uniform flows in the creeks. This would require the City to store the cooling tower effluent and discharge continuously from this tank. For planning purposes the effluent tank was also assumed to be 3.0 million gallons.

Since the cooling tower effluent would be stored, the final effluent temperature would be between the effluent at the mean and minimum wet bulb temperatures shown in Charts 10-1 and 10-2. Thus, there would still be several violations of the effluent temperature limit. The cooling tower may not meet the DEQ effluent temperature requirements all the time without additional treatment utilizing chillers to lower the effluent temperature during hot nighttime weather periods.

<u>Chiller</u>
In order to prevent any discharge temperature violations, a chiller would be needed to reduce the effluent water temperature further. A chiller would use condensers and electrical energy to obtain the cooling required. Based on the climate data analysis, the chiller may be required to reduce the effluent a further 3 ^{o}C at times. To reduce the size of the chiller, it would be installed in the effluent line from the final storage tank and thus be sized for 5.5 mgd or 3800 gpm. The preliminary sizing of the chiller is 1,500 tons. The chiller would also need to be installed in a building.

Cooling Tower and Chiller Alternative

A cooling tower/chiller alternative that would allow the City to meet the effluent temperature limits at all times would consist of the following components:

- Cooling tower inlet storage, sized to hold 12 hours of plant effluent flow from 10 AM to 10 PM during the period April 1 to October 30. The tank would hold 3.0 million gallons (50% of the peak dry weather day in 2030). For budgeting purposes, Keller Associates assumed the storage would be a concrete tank (high range) or a lined pond (low range).
- Pumps, sized to pump the daily flow from the storage tank to the cooling tower (assumes permeate pumps or filter pumps can feed the tower).
- Cooling tower, closed loop type, sized for the twice the peak dry weather day flow (7,600 gpm) in order to pump the peak day during the 12 coolest hours of the day. For budgeting purposes, Keller Associates assumed that the cooling tower would include a plate heat exchanger for the cooling water and non-chemical water treatment system for the cooling water to prevent scaling.
- Cooling tower effluent storage, sized at 3.0 million gallons; assume continuous gravity discharge at the plant influent flow rate via a motor-controlled valve. For budgeting purposes, Keller Associates assumed the storage would be a concrete tank (high range) or a lagoon (low range).
- A 1,500 ton chiller, sized to cool 3,800 gpm 3 ^{o}C, in a building (approx. 32 feet by 22 feet and 16 feet high).

The estimated capital cost for this option is $ 6,100,000 to $8,100,000, depending on the type of storage. The estimated annual O&M costs for the cooling system are approximately $200,000 (for either storage option).

The O&M challenges are:

- Scale control in tower and chiller.
- Turning cooling tower system on as temperature limit is approached and off as tower is not needed.
- Controlling the pump rates to the tower and chiller and outlet rate from the final effluent equalization tank.
- Operating chiller when needed.

10.4.2 Option 5: Trading (Shading)

Temperature trading allows for excess thermal loads to be offset by shading (from riparian vegetation) and other approaches that reduce heat loading such as constructed wetlands, flood plain restoration, and restoration of cold water refugia. In recent years, the temperature trading program has been developed more fully in the State of Oregon. In December 2009, DEQ published a guidance document *Water Quality Trading in NPDES Permits* [7]. With project protocols, verifications, and reporting procedures in place and accepted by DEQ, trading is now a viable solution for cities facing new thermal load limits like Ashland. DEQ allows for offsets in the TMDL area to apply both upstream and downstream of the point discharge. While there are few opportunities for trading in Ashland Creek, there are many opportunities to trade along Bear Creek and within the Bear Creek watershed.

In evaluating this alternative, a nonprofit organization, The Freshwater Trust, assisted in the analysis. To complete the analysis, The Freshwater Trust coordinated with and received and field verified data from DEQ's Heat Source models for Bear Creek to determine the extent of opportunities for riparian revegetation with native species to create shade and minimize solar loading in the TMDL area. In addition to using DEQ data for the analysis, The Freshwater Trust worked closely with DEQ technical staff to confirm its analysis procedures.

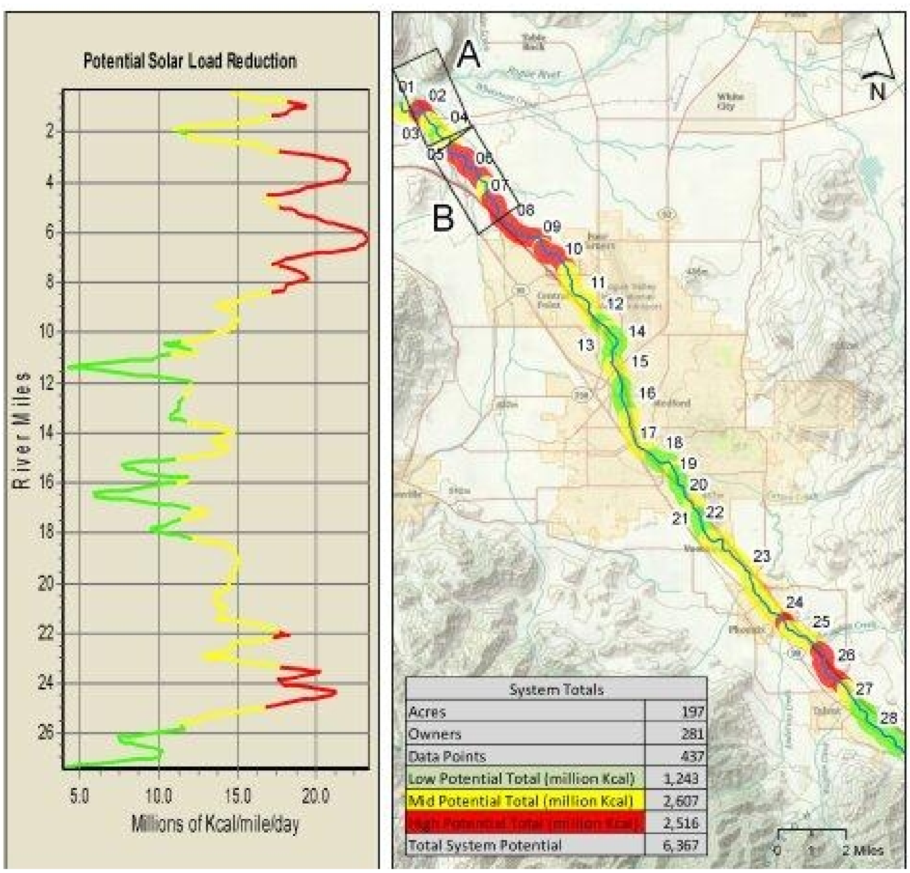
The heat source data for Bear Creek was divided into three equal interval classes: LOW, MID and HIGH, based on the difference between existing shade cover and potential shade cover. Areas with the highest potential for improvements and shade credits are designated as HIGH. The lengths and potential solar load reductions for these reaches are summarized in Table 10.1. The location of LOW, MID, and HIGH Bear Creek river stretches is further illustrated in Chart 10.4.

TABLE 10.1: Bear Creek – Heat Source Analysis Results*

Kcal Potential Category	Average of 25% Potential per mile (kcal/day)	# Miles by Potential Category	% Miles by Potential Category	Deliverable Solar Load Reductions Kcal/day (TOTAL)
LOW	3,257,325	5.16	19%	16,807,797
MID	6,818,843	16.65	61%	113,533,736
HIGH	10,141,007	5.34	20%	54,152,977
TOTALS	6,795,378 (weighted average)	27.15	100%	184,494,510

*Information provided by The Fresh Water Trust

CHART 10.4: Bear Creek – Potential Solar Load Reduction by River Mile



The Heat Source analysis showed that revegetation projects on Bear Creek will produce between 3,257,325 and 10,141,007 kcal/day per mile with a weighted average of approximately 6,800,000 kcals/day per mile. Using this weighted average, to meet the projected 2030 excess heat load of 53,000,000 kcal/day, an estimated 7.8 miles of riparian revegetation will be needed. The actual length of shading requirements will depend on the existing conditions for the reaches targeted.

With over 27 miles of riparian area, over 80% of which are in the mid to high-potential range, the data show there are sufficient revegetation opportunities along Bear Creek to meet reduction targets. For the purpose of this analysis, two conservative assumptions were made: first, the Solar Load Change actually projected is reduced by half to account for planting along only one side of the stream bank; and second, DEQ requires that the load be reduced by half again to cover risk factors of temporal loss and uncertainty.

Implementation of this alternative would occur over several years and would require a compliance period to incorporated into the City's permit. The compliance agreement would require that certain annual milestones be accomplished and that ongoing monitoring and reporting be provided. Existing protocols do not require that the projects fully mature before thermal credits are granted to the City. The City can receive thermal offset credits the same year the improvement is completed and verified.

Under this alternative, the temperature of the effluent is not cooled prior to discharge. This creates the potential for near field (local) impacts to aquatic habitat that must be accounted for. To address these concerns, Keller Associates has worked closed with regulatory agencies, the City, and other stakeholders to develop a plan that will work. Representatives from DEQ have completed computer modeling and evaluations for potential impacts to Ashland and Bear Creek. The plan presented for this alternative reflects the following improvements intended to address near field concerns:

- Continue to gather data and work with regulatory agencies and stakeholders to define impacts of newly developed treatment standards for toxins, and explore options for how those requirements may be met. Keller Associates recommends the City wait until their new permit is issued before investing significant capital in relocating the outfall.
- Relocating the outfall from Ashland Creek to Bear Creek. Keller Associates proposes that this be completed via an open channel arrangement that would convey treated wastewater to Bear Creek via a side bank discharge. Based on modeling completed by DEQ, this single improvement would alleviate all near field concerns with the exception of potential migration concerns in September (there have been a few days in the last five years that would require the effluent temperature to be lowered from 23.5C to 22.3C in September). Using an open channel conveyance could further cool the effluent via shading and interaction with shallow ground water.
- Consider modifying the existing wetland pond (Glendower pond). While this improvement may not be required to meet DEQ thermal load improvements, the wetlands could further serve to cool the effluent and improve aquatic habitat. The existing pond is too deep to encourage growth of vegetation and additional shallow groundwater interaction that would further cool the water. Creating a shallower wetland could support growth of wetland vegetation that would further cool the effluent. Additionally, ODFW has expressed a desire for off channel habitat which could be provided through properly designed wetlands. The final size of the wetlands may need to be expanded depending on a number of issues yet to be determined such as hyporheic action, shading in the channel and wetlands, and other shading

activities along Bear Creek upstream of the outfall, and additional flow and temperature data. While Glendower pond has been identified as a potential component of the improvement, it should be noted that other wetland improvements in the vicinity could also be completed to meet compliance goals.

- The Glendower pond is considered waters of the State, which could require the point of compliance to be established at the upstream end of the pond. Removing the pond from the "waters of the state" designation is possible (requires pond to be designated as a waste management area), but would require mitigation, potentially in the form of additional wetlands construction.
- Other improvements that should be added into this alternative to improve conditions for fish include: 1) removing the current outfall structure which allows fish to enter the effluent pipeline of the WWTP (and possibly be trapped), 2) constructing a fish barrier (i.e. waterfall) in the new discharge channel from the WWTP, 3) modifying the existing pond by replacing/removing inlet and outlet structures.
- It should be noted that the proposed relocation and enhancements to the Glendower pond / park area should be completed in coordination with other stakeholders including the Parks Commission, the school district (which has invested in the current pond and used the site for educational purposes), and local residents.
- It should also be noted that phasing of "near field" improvements could allow for additional flow and temperature data to be gathered and could determine the impacts of shallow ground water interaction prior to investing in construction of wetlands. Additionally, it may be that conditions may change that would reduce the near field treatment requirements for Ashland in the future. For example, there is a potential release of additional flows to Bear Creek from the Talent Irrigation District (when we met with TID in September 2010, the board mentioned the District may be required to increase flows in the future to Bear Creek to meet regulatory requirements). Finally, the City may want to consider participating with other entities to explore if a higher site-specific temperature criteria would be entertained by DEQ based on the potential biological adaptation of native fish to higher temperatures naturally occurring in this region of the state.

The Oregon DEQ has expressed support for temperature shading as a means for meeting thermal compliance at WWTPs. Other benefits of this alternative include:

- Low capital and O&M costs. On-going power costs associated with other alternatives such as cooling towers can be avoided. Costs are also spread out over the duration of the project.
- Flows remain in the stream for improved conditions for aquatic habitat during low flow periods.
- Shading along the creek also improves aquatic habitat.
- Other aesthetic and environmental benefits associated with trees.

An estimated cost for this alternative was prepared with input from The Freshwater Trust, and has an estimated net present value of approximately $2.9 million ($3.65 million spread out over 35 years). Of this cost, approximately $840,000 has been included in the budget for the outfall relocation. Actual costs could vary depending on the final sections of river that are targeted for shading and the final scope of improvements targeted for the outlet relocation and wetlands work near the treatment plant. Refer to Appendix F for a more detailed projection of annual costs for this alternative.

10.4.3 Option 6: Blending / Flow Augmentation

The concept of blending or flow augmentation involves releasing cold water upstream of the Ashland WWTP. The source of this water would be either flow from TID (ideally from lower depths of the Emigrant Dam) or from Ashland Creek. The City of Ashland is currently in the process of permanently securing an additional 600 ac-ft of additional water rights formerly belonging to the City of Talent. The purpose of this right would be to augment existing flows in Ashland Creek and/or provide additional potable water supply. One of the benefits of this alternative is that increased stream flows could improve stream conditions in Ashland and Bear Creeks.

For flow augmentation to work, the water quality and temperature conditions of the supplemental water need to be considered. This study does not include a comprehensive evaluation of these parameters. However, the City did install a temperature monitoring device in the TID system for about a week in August of 2010. Based on this temperature data, flow in the TID system already exceeded the target temperature thermal limits (18ºC) and therefore would not be able to cool Ashland's effluent to levels that met the TMDL standard. Additionally, it should be noted that if flow augmentation were used, that DEQ has indicated that they want to see information on presence of parameters in the source water for which Ashland and Bear Creeks are water quality limited (see 1992 and 2007 TMDLs) and additional parameters may be needed depending on origin of source water.

Given the need for additional potable water rights and the preference of the City to use Ashland Creek water over TID supplied water, it is unlikely that if additional Ashland Creek water rights could be supplied, that these rights would be used for flow augmentation during critical low flow conditions when they would be needed the most for flow augmentation.

While flow augmentation may help mitigate thermal impacts during certain times of the year, Keller Associates does not recommend this as a sole solution to address excess thermal loads.

10.4.4 Option 7: Hyporheic (Shallow Groundwater Mixing)

The hyporheic zone is the region where shallow ground water interacts with the surface water in a stream or river. Depending on numerous conditions (e.g., channel geometry, soil characteristics, diurnal variations, season, etc.), the hyporheic exchange can act as a buffer for river temperatures and/or as a mechanism to cool/warm river temperatures. Using a hyporheic discharge was previously recommended for future study as a disposal option for temperature control.

Implementing this process can take several forms, which can be divided into either a direct or indirect injection into the water table. Each application must satisfy the following requirements [5]:

1. Definition and maintenance of a Waste-Management Area (WMA), which defines the confines of the infiltrate influence (Chart 10.5). The WMA must be situated so that the infiltrate remains within the confines of the property and does not affect existing wells. Also, it needs to be shown that the infiltration will not contaminate the groundwater/aquifer.

2. Site/soil suitability, primarily that the hydrology of the site would permit the injection of the proposed quantity of effluent.

3. Public acceptance of the practice.

CHART 10.5: Waste-Management Area [5]

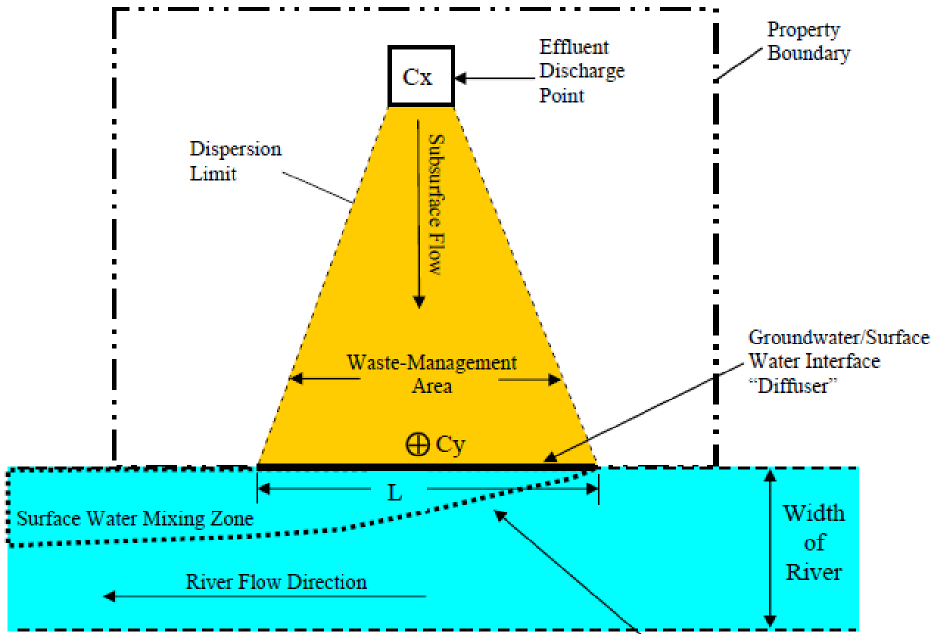


Figure 1
Plan View Schematic
Indirect Discharge Model
To Surface Water

Notes:

1. At L, provide groundwater volume, velocity and concentration of key water quality parameters for mixing zone model. These typically will be estimates based on calculations or modeling.
2. L functions as a surface water diffuser.
3. Effluent discharge in the Waste-Management Area can be in groundwater, hyporheic water, or both.
4. Limits of Waste-Management Area are determined by horizontal dispersion of the effluent in groundwater, hyporheic water or both.

Symbols:

⊕	Verification monitoring well location (i.e., *detection monitoring* well)
Cx	Concentration of key wastewater effluent parameters.
Cy	Concentration of key effluent parameters in groundwater or hyporheic water at transition to surface water for mixing zone analysis, e.g. concentration for diffuser at L.
L	Length of Waste-Management Area at receiving water body.

While the effluent temperature could conceivably be reduced through dispersion and conduction with ground water, this relationship cannot be adequately described without sufficient site data. A rough, preliminary design can be completed using semi-conservative values, which can be used as a basis to formulate site parameter investigations.

A planning level evaluation of this alternative was completed for Ashland. This section includes summary information. For more detail refer to the hyporheic evaluation in Appendix E. A preliminary evaluation of the Imperatrice property was considered. However, due to the low permeability of the Imperatice property's soil, potentially shallow soil depth, significant slope, and incomplete WMA control, the site would likely not be well suited for effluent infiltration and hyporheic exchange.

The hyporheic option could be implemented at other sites in close proximity, assuming property acquisition was a possibility. Soil maps from the National Wetland Inventory indicate substantial soil type differences in the valley, namely the presence of sandy characteristics in some areas. Sandy soils typically have a higher permeability rate, with typical values ranging from 0.13 to 12.96 in hr^{-1} for clayey sand. Over this range of values, the foot print for each MGD of effluent would be between 780 and 8 acres (assuming 15 ft of head and 300 m spacing between the river and the infiltration basin). These areas only include that needed for the WMA; due to plot dimensions, considerable additional property would likely be purchased as well.

If this option were pursued, the following phased approach should be completed in stages, obtaining more and more detailed estimates of the site characteristics, while minimizing potentially unwarranted expenditures. Initial sample planning should be based on the aforementioned design, first assessing if the City owns property that could be isolated enough to satisfy the groundwater protection requirements while providing an adequate footprint for the above design. Behind each stage is a progressively more accurate model of the ground/hyporheic water flow and the river mixing, which determines the viability of the design and directs subsequent investigations. We would recommend the following approach, each phase of which could be conducted in stages:

Phase 1 – Initial Site Assessment and Monitoring Well Installations

A preliminary assessment of the sites suitability for this approach can be completed by installing ground water monitoring wells throughout the site, as directed by the preliminary design. Placing the wells near the creek's edge as well as toward the site's boundaries will allow the wells to be used in the future for compliance testing, assuming the site is suitable. Recording soil properties and water levels in the drilling processes of the wells should provide a rough approximation of the site's geology and ground/hyporheic water state. These parameters could be used to estimate the site's infiltration capacity and subsurface conductivity. With these estimates, a rough design of the infiltration basins could be completed, balancing the need to minimize the waste-management area while maximizing the distance between the infiltration basin and the creek.

Phase 2 – Single and Multiple Well Aquifer Tests, Mixing Model Precursors

Assuming that the preliminary design completed using the estimated site parameters were viable, a more refined estimate of the site hydrology should be completed. To accomplish this task, wells should be drilled according to the predicted design, with locations in the infiltration area(s). Single well aquifer tests should then be performed to obtain actual conductivity information for the site, using the previously installed monitoring wells to observe the site's response. Using the results from these tests, the actual distribution of site conductivities can be more accurately estimated. These values can then be used to refine the previously developed model to reassess the site's viability. Tracer studies could also be used to determine ground water flow and dispersion.

The Oregon DEQ requires a mixing model analysis to be performed to determine the impact of the hyporheic exchange on the creek temperature profile, to estimate the mixing effects. To approximate these effects, the creek profile should be approximated over the range of available property, determining cross section profiles, depth, and velocity. An estimate of the hyporheic mixing capacity would also be of help. As indicated by the research of Lancaster et al.[6], if properly distanced from the creek, the injected heat should not substantially impact the creek temperature.

Phase 3 –Long Term Monitoring

Provided that the refined design was still viable, the behavior of the groundwater should be observed to determine seasonal variation and response to rainfall and creek flows. These observations would provide additional insight into the actual response of the site to real infiltration, allowing further calibration of the model and verification of the groundwater flow direction and velocity.

Phase 4 – Scaled Infiltration Test

Using a full scale design based on the estimated infiltration capacity and ground water response as a guide, a large scale infiltration test would provide a final model verification prior to full construction.

Using this approach, the capital investment required for an accurate model (which is expected for permitting [5] could be expended in stages, each of which would allow for the overall evaluation of the process, to determine if further investment is warranted.

Other Hyporheic Considerations

It should be noted that hyporheic activity can also occur through leaky wetlands. Thus some hyporheic activity could occur if the City's existing effluent outfall were relocated from Ashland Creek to Bear Creek via a channel and possible downstream wetlands.

10.5 SUMMARY AND RECOMMENDATIONS

Table 10.2 on the following page summarizes the disposal alternatives, benefits, drawbacks, and costs. Based on the available information, Keller Associates recommends that the City proceed with Option 5, Trading (Shading). Concurrent to pursuing Option 5, Keller Associates recommends that the City pursue recycling as needed to address future potable water supply needs.

TABLE 10.2: Ashland WW Disposal Option Comparison Chart

	Option	Description / Project Elements	Benefits	Drawbacks	Capital Cost	Annual Cost	Net Present Value	Comments
1A	Maximum Recycling on Imperatrice Property	Irrigate 433 acres with treated effluent. Pipeline to site, 138-166 MG storage. Shoulder season storage required / winter discharge.	Beneficial use of water. Existing water right could be used to augment potable water supply. Potential for membrane and chemical savings. Mitigate concerns of "future" more stringent regulations.	High Cost. Lower stream flows.	$10.8 M	$(58,000)	$10.1M	Savings assume that membranes are not used.
1B	Partial Recycling on Imperatrice Property -- 424 ac*ft/yr	Lower cost does includes minimum storage. Higher cost assumes more storage, and periods of no discharge.	Similar to Option 1A. Improvement could be completed later if Option 3 or 4 is pursued.	High Cost. Introduces complexities in monitoring and wastewater management. Higher O&M costs than Option 1A. Reduced stream flows available for aquatic habitat.	$5.3 – 8.9 M	$35,000	$5.8 – 9.4 M	For lower cost range, need to add cost of Option 3 or 4 to address cooling requirement
2	City Wide Recycling							
3	Discharge to TID	Pipeline to Talent Canal.	Mitigate "near field" concerns. Reduction in chemical costs.	Not a standalone solution -- still need to offset excess thermal loads. High opposition from downstream users anticipated. District concerns about chemicals. Storage for shoulder seasons likely required. Schedule and approval outside of City control.	Not Evaluated	Not Evaluated	Not Evaluated	
4	Cooling Tower	Mechanical cooling tower. Storage facilities.	Addresses temperature concerns. Allows continued discharge. Maximum control in terms of compliance schedule.	Chillers required for hottest periods. Upstream / downstream storage also required for night-time operation	$6.1 - 8.1M	$200,000	$8.6 - 11.6M	Cost range reflects use of ponds versus concrete storage reservoirs
5	Trading (Shading)	8 miles of shading. Channel to Bear Creek. Constructed wetland pond.	Lowest cost alternative. Allows continued discharge. Improved aquatic habitat and other environmental benefits.	Some uncertainty -- participating property owners to be identified; migration blockage evaluation to be completed. Potential minor additional local cooling required.	$3,645,000 spread out over 35 years		$2.9M	
6	Blending / Flow Augmentation	Blend additional water discharges from Ashland Creek or TID.	Additional stream flow.	Cannot meet temperature targets by itself. Uses water that could be used for potable water usage. Additional water quality testing may show additional water quality concerns.	Not Evaluated	Not Evaluated	Not Evaluated	Not a viable alternative.
7	Hyporheic (shallow groundwater)	Subsurface disposal of treated wastewater to shallow ground water.	Low operations costs. Simple technology.	Difficulty in locating site with suitable soils. Significant additional effort required to determine feasibility. Potential large land requirement.	Not Evaluated	Not Evaluated	Not Evaluated	

References

1. Jon Gasik: *City of Ashland Excess Thermal Load Limits*, WQ File No. 3780, ODEQ Memorandum, February 16, 2011.

2. Carollo Engineers: *City of Ashland Update to WWTP Facilities Plan and Permitting Evaluation, Temperature Management and Water Recycling*, November 2009.

3. Carollo Engineers: *City of Ashland Wastewater Treatment Plant Upgrade Project, Project B – Offsite Facilities Predesign Report*, November 1997.

4. Carollo Engineers: *City of Ashland Water Conservation & Reuse Study (WCRS) & Comprehensive Water Master Plan (CWMP), Technical Memorandum No. 7 – Recycled Water Regulations*, September 2010.

5. Oregon DEQ (2007). *Disposal of Municipal Wastewater Treatment Plant Effluent by Indirect Discharge to Surface Water via Groundwater or Hyporheic Water.Internal Management Directive.State of Oregon Department of Environmental Quality.Portland. 2007.*

6. Lancaster, S., Haggerty, R., Gregory, S., Farthing, K.T., and Biorn-Hansen, S. *Investigation of the Temperature Impact of Hyporheic Flow: Using Groundwater and Heat Flow Modeling and GIS Analyses to Evaluate Temperature Mitigation Strategies on the Willamette River, Oregon*. Final Report to Oregon Dept. Environmental Quality.Oregon State University, Corvallis. 2005.

7. Oregon DEQ (2009). *Water Quality Trading in NPDES Permits Internal Management Directive.* Portland. December 2010.

11.0 TREATMENT PLANT IMPROVEMENT ALTERNATIVES

11.1 INTRODUCTION

The feasible alternatives for treatment process improvements depend on the selected effluent disposal method and the associated effluent requirements. Requirements for agricultural recycling of effluent (on Imperatrice property) are much less stringent than for discharge to Ashland or Bear Creek. To maintain some discharge in Ashland or Bear Creek would require additional treatment capability to meet expected discharge limits. These treatment requirements could be minimized if all of the discharge were able to be land applied. A 100 percent land application program has two major obstacles: 1) some of the water is needed to sustain flow for fish in Ashland Creek, and 2) the existing City-owned property will not be large enough for 100 percent land application in the future. Land application can be used as a strategy to limit discharge to periods with less restrictive discharge limits. Alternatives without land application (recycling) would need to provide treatment that would meet all expected discharge limits year-round.

Biosolids management is also an important part of the evaluation of treatment plant improvement alternatives. While the quantity and characteristics of biosolids generated are somewhat dependent on the treatment processes utilized, the ultimate disposal method is the controlling factor in evaluating the feasibility of various biosolids handling methods. Alternates for sludge stabilization, dewatering and disposal are evaluated that may reduce the quantity of biosolids produced, improve the quality of the final end product, reduce disposal costs, and provide a more environmentally friendly alternative to landfilling.

11.2 USING AGRICULTURAL RECYCLING & CONTROLLED DISCHARGE TO OPTIMIZE

If sufficient storage were provided to optimize discharge during specific periods without restrictive discharge limits (primarily wet weather high flow periods), the need for additional treatment to reduce future phosphorus loads and to reduce near field thermal loads could be minimized.

11.3 TREATMENT ALTERNATIVES FOR YEAR-ROUND DISCHARGE

11.3.1 No Action Alternative

As discussed in Chapter 9, the existing treatment facilities have capacity limitations that will make it increasingly difficult to meet discharge permit limits as flows increase. Since the No Action alternative has the potential for periodic violations of the discharge permit, it does not represent a practical approach.

11.3.2 Reduction of Peak Flows

Reducing peak flows to the plant could delay the need for additional capacity in some of the units. Peak flows could be reduced by collection system improvements (rehabilitation) that eliminate inflow, or by adding flow equalization before or at the treatment plant (after grit removal). Collection system improvements have the advantage of decreasing wastewater flows before they reach the treatment plant, thus reducing the required capacities of influent pumping, screening and grit removal in addition to the remaining treatment components. However, it is often difficult to accurately estimate flow reductions expected from collection system rehabilitation.

Flow equalization, though not technically a treatment process, can increase the effective capacity of downstream process units by reducing extreme flow fluctuations. The equalization basin would hold peak flows and discharge at a constant (lower) rate.

TABLE 11.1: Flow Equalization

Pros	Cons
▪ Increases effective capacity of existing facilities; delays need for expansion	▪ Very large size for large flows (space requirements)
▪ Equalizes influent quality in addition to flow	▪ Aeration and mixing required
▪ May be used with any treatment alternative to minimize size of new facilities	▪ Pumping required if sufficient head unavailable for gravity flow

11.3.3 Expansion of Existing Oxidation Ditch Plant

As discussed in Chapter 9, the existing oxidation ditch system could be expanded by adding a third oxidation ditch or a fourth clarifier. Though the clarifier would take less space and would be less costly than a third oxidation ditch, it would not provide the same benefits in terms of reliability and redundancy. Therefore, expansion of the oxidation ditch plant alternative assumes construction of another oxidation ditch. With a third ditch, the existing clarifiers would be adequate beyond 2030.

The most cost-effective approach is to construct the additional ditch immediately adjacent to the existing ditches to utilize an existing wall; however, this would require expanding west into a buffer/wetlands area. City staff has commented that in order to avoid a scattered plant footprint, they would prefer that if a new oxidation ditch is constructed that it is next to the existing ditches. Therefore, wetlands mitigation would be an issue. The only other area with sufficient space for the third ditch is east of the existing facilities. The splitter boxes ahead of the oxidation ditches would need to be modified to route part of the flow to the new ditch, and additional piping would be needed to carry mixed flow to the new ditch and back to the clarifiers.

TABLE 11.2: Expansion of Existing Oxidation Ditch Plant

Pros	Cons
▪ Operator familiarity with process operation	▪ Large space requirements
▪ Allows operation at lower MLSS (activated sludge process easier to maintain, lower RAS flow required, avoids clarifier overload)	▪ Additional complexities in operations if adjacent site to west is not used
▪ Provides redundancy for aerator out of service	

11.3.4 Parallel Membrane Plant

A separate parallel membrane bioreactor (MBR) plant could be installed to treat a portion of the flows. The MBR should be sized to treat a base flow; the oxidation ditch/clarifier would handle base plus peak flows. MBR plants eliminate the need for clarifiers and operate at much higher MLSS than a conventional plant, thus reducing the size of the required footprint compared to an equivalent capacity oxidation ditch.

The MBR plant would consist of two trains for flexibility. The splitter boxes ahead of the oxidation ditches would be modified to route part of the flow to the MBR. In addition to the membrane cells, each MBR train would include an anaerobic cell, an anoxic cell, aeration cells, and post anoxic cell. Recycle pumps would be provided for each MBR train, and recycle flows would be combined with the influent flow before entering the membrane cells. Each membrane tank would have separate permeate pumps, and a single chemical treatment system would be utilized to maintain the membranes.

TABLE 11.3: Parallel MBR Plant

Pros	Cons
▪ Reliably low effluent solids independent of sludge settleability	▪ Finer screening (1-2 mm) required; will increase screenings for disposal
▪ Small footprint reduces space requirements	▪ Additional blowers required; significant energy usage (high MLSS)
▪ No additional clarifiers or tertiary filters needed	▪ O&M more complicated - dual plant with significantly different processes
▪ Existing plant can remain operational during construction	

11.3.5 Process Modifications in Existing Tankage

As discussed in Chapter 9, nitrification needed to meet year-round ammonia discharge limits is the controlling factor in evaluating the capacity of the biological process at the Ashland WWTP. Since nitrification is a function of the mean cell residence time (MCRT) or MLSS inventory in the process, process modifications that reduce the MCRT required for nitrification or increase the MLSS concentration are possible alternatives for Ashland. There are several alternatives that would utilize the existing oxidation ditch basins with modifications to accommodate increased loadings without adding tankage. These include conversion to staged aeration, an integrated fixed film/activated sludge system, or an in-ditch MBR.

Any of these process modifications would require routing all flow to one of the oxidation ditches while modifications to the other are completed. Meeting permit limits could be difficult under these conditions; the addition of alum and polymer to enhance settleability might be needed when a single ditch is in use. All of these alternatives would also require conversion of the aeration system to diffused air, necessitating the addition of blowers and a blower building, aeration piping and diffusers. Other modifications specific to a particular process are summarized in the following descriptions.

Staged Aeration

It has been demonstrated that the MCRT required for nitrification can be significantly reduced by the use of an aerobic bioreactor “with significant plug flow character”. Optimizing the plug flow character can be accomplished by increasing the number of treatment cells in series to provide staged aeration. Providing nine or more basins in series can reduce the MCRT required for nitrification by up to 30-50% [1], effectively almost doubling the capacity of the biological process. Even with a more conservative MCRT reduction, staged aeration should provide 50% additional biological process capacity.

This alternative would require construction of interior concrete walls in the oxidation ditch to form the numerous zones used for staged aeration. Bioselectors would also be incorporated to enhance settling of the mixed liquor and reduce oxygen requirements; some of the multiple cells would be anaerobic, some anoxic and some aerobic. (The ditches currently include a separate anoxic zone and an aerobic zone.) The anaerobic and anoxic cells would have mixers (submersible or vertical) only, and while the aerobic reactors would have both mixers and fine bubble diffused aeration.

TABLE 11.4: Staged Aeration

Pros	Cons
▪ Good treatment efficiency for removal of nitrogen, phosphorus and BOD	▪ Significant pumping for process recirculation requirements
▪ Operationally stable process	▪ Aeration system will need to be replaced (diffused air)
▪ Relatively low maintenance requirements	▪ Operation with single ditch during construction – possible permit violations
▪ No additional tankage needed	

Integrated Fixed Film/Activated Sludge (IFAS)

Placing fixed film media into activated sludge basins can be used to increase plant capacity at a given treatment level and/or improve treatment performance. Since additional biomass can be maintained on the fixed film, IFAS systems can increase the effective MLSS in an aeration basin by as much as 3000 mg/L [2]. This would effectively increase the MCRT.

Bioselectors would also be incorporated in the IFAS alternative to enhance settling of the mixed liquor and reduce oxygen requirements. In addition to conversion to diffused aeration, this alternative would require fine screening and construction of interior concrete walls to form selector zones and the IFAS basins. Additional walls may need to be constructed to modify the oxidation ditches in order to improve flow characteristics.

Two types of systems were explored for converting the Ashland WWTP: a fixed media system and a floating media system. Costs for both systems are comparable. The in basin equipment cost is $2.2 million for the fixed system and $1.7 million for the floating media system. The floating system requires more structural modifications to the existing tankage (removal of the center wall and construction of baffle walls). The cost used for comparison is based on the fixed system with higher equipment costs but lower structural modification costs.

TABLE 11.5: IFAS System

Pros	Cons
Provides stability of fixed film system with flexibility and treatment capability of activated sludge	Fine screening required; will increase screenings for disposal
Biomass on fixed film does not proportionately increase load to clarifier	Aeration system will need to be replaced (diffused air)
Fixed media biomass improves cold weather nitrification	Operation with single ditch during construction – possible permit violations
Reportedly generates less waste sludge than conventional systems	
No additional tankage needed	

In-Ditch MBR Plant

With operation at significantly higher MLSS concentrations than a conventional plant, a MBR facility utilizes less volume to provide the same level of treatment. The membranes provide solids separation, eliminating the need for secondary clarifiers. The MBR plant would include multiple trains for flexibility and redundancy. Recycle pumps would be provided for each MBR train, and recycle flows would be combined with the influent flow before entering the membrane cells. Each membrane tank would have separate permeate pumps, and a single chemical treatment system would be utilized to maintain the membranes. In addition to the membrane cells, each MBR train would include an anaerobic cell, an anoxic cell, aeration cells, and post anoxic cell.

This alternative would require construction of interior concrete walls in the ditches to form the selector zones and the membrane basins in multiple trains. In addition to the conversion to diffused aeration, an MBR plant would require fine screening.

Since the total volume of the two oxidation ditches (3.52 MG) would provide more than needed for an MBR plant sized for 4.24 mgd (year 2030 maximum month), only a portion of the volume would be required for the in-ditch MBR plant. In terms of volume, a single ditch could be modified to provide two trains although it is likely that a minimum of three membrane trains would be necessary from a process operation standpoint. The secondary clarifiers would no longer be needed for clarification, and could be converted to other uses. The tertiary membrane would also no longer be needed, and the space could be utilized for other purposes.

TABLE 11.6: In-Ditch MBR Plant

Pros	Cons
Reliably low effluent solids independent of sludge settleability	Fine screening (2-3 mm) required; will increase screenings for disposal
Reportedly generates less waste sludge than conventional systems	Energy usage typically 1.5-3 times conventional activated sludge
Second ditch and existing clarifier tankage available for other uses	Aeration system will need to be replaced (diffused air)
Tertiary membrane no longer needed	Operation with single ditch during construction – meeting permit limits would be a challenge
No additional tankage needed	

11.3.6 Summary of Pre-Screening

Of the treatment alternatives presented, the top three options recommended for further evaluation by Keller Associates include reduction of peak flows by equalization, expansion of existing oxidation ditch plant, and process modifications in existing tankage using staged aeration or IFAS. The technical review committee agreed with the two latter alternatives, but expressed a preference for evaluation of adding primary treatment (in the form of a fine mesh sieve) instead of evaluating a separate equalization basin. Based on this input, the following three options were evaluated in more detail with cost estimates and environmental impacts considered:

1. Constructing an additional oxidation ditch (which could be staged by initially using the shell as equalization storage);
2. Converting the existing oxidation ditches to enhanced biological treatment, through staged aeration or IFAS
3. Adding a fine mesh sieve (primary filter) to reduce loading and thus increase treatment capacity.

Expansion of the existing oxidation ditch plant by constructing an additional oxidation ditch was addressed in Section 11.3.3 and Table 11.2. Equalization was described in Section 11.3.2, with advantages/disadvantages shown in Table 11.1. Converting the existing oxidation ditches to enhanced biological treatment was covered in Section 11.3.5 plus Tables 11.4 and 11.5.

Fine mesh sieves have long been used for pretreatment in Norway, and one (Salsnes Filter™) has been developed that is capable of providing primary treatment by removing 40-70% suspended solids and 20-35% BOD. Solids removed as the wastewater flows through a fine mesh wire cloth are dewatered in the unit to 25-35% dry solids. This type of primary filter is currently installed in five plants in the United States. The plants range from 0.3 to 3.0 MGD. A primary filter was installed in eastern Idaho (Heyburn) in 2009 as part of a conversion to a BNR plant, and a pilot project has been carried out in north Idaho (Hayden). A drawback to this technology is the amount of primary solids that would be produced (approximately 3.5 cubic yards per MGD).

TABLE 11.7: Primary Filter

Pros	Cons
▪ Very small footprint	▪ Adds operational complexity (additional pumping and addition of primary solids)
▪ Low energy usage	▪ Possible blinding from grease increases maintenance requirements
▪ Second lowest life cycle cost and lowest long-term life cycle cost option	▪ Space required for pumping and odor control (possibly covered truck loading area)
▪ Would reduce biological sludge production	▪ Would reduce amount of food source for BNR operation
	▪ Higher SVI resulting in reduced clarifier performance
	▪ Limited track record in North America; most performance data from pilot studies

11.3.7 Temporary Primary Filter Option

A potential option for dealing with the wet weather peak flows would be to provide primary treatment (likely with a primary screen) followed by disinfection. A portion of the flow would bypass the secondary treatment, thus reducing the peak flows through the secondary process. This wet weather bypass is part of EPA's wet weather policy that is currently in draft form. There are other communities, including some in Oregon, that deal with wet weather flows in this manner. Of the three conditions mentioned that will lead to a need for increased plant capacity in 2015 (increased MCRT for maximum month flows, increased aeration for taking a unit offline, and peak wet weather flows), only peak wet weather flow would be addressed by an option including primary filtration, secondary bypass, and disinfection. Pros and cons of this option are listed in Table 11.8. The wet weather peak hour flow is 10.8 MGD. Biological spreadsheet modeling of the Ashland WWTP shows that the plant has the potential to treat about 9 MGD for a limited duration. Therefore a minimum reduction of about 2 MGD would be necessary to resolve the capacity issues associated with peak wet weather flows.

TABLE 11.8: Primary Treatment / Disinfection

Pros	Cons
▪ Very small footprint	▪ Adds operational complexity (additional pumping and additional treatment process)
▪ Low cost alternative	▪ Some risk since EPA's wet weather policy is only in a draft version.
	▪ Long term solution would still be needed
	▪ Additional measures to reduce I/I would be required
	▪ Does not address need for increased MCRT during max month flows and need for more aeration capacity when taking one ditch offline

11.3.8 Annualized Cost Comparison of Selected Liquid Treatment Alternatives

Table 11.9 presents the annualized cost analysis of the alternatives selected for further evaluation. This includes operation and maintenance costs based on 2030 flows as well as capital costs. Staged aeration and IFAS were evaluated separately, and using a new oxidation ditch as an equalization basin was also evaluated.

The annualized cost for the temporary primary filter option discussed above is $218,000, which is significantly less than the alternatives shown in Table 11.9. However, since this option is a temporary approach that does not address all the expansion needs, it is not included with the alternatives in Table 11.9 which are projected to satisfy the secondary treatment capacity requirements through the year 2030.

TABLE 11.9: Comparison of Costs for Selected Treatment Alternative

	Costs				
Supplier	Filter	Staged Aeration	IFAS	Additional Ditch	Additional Ditch (equalization)
Total Capital Cost (2011 dollars)	$ 5,400,000	$ 5,210,000	$ 6,540,000	$ 6,150,000	$ 4,000,000
With Odor Control	$ 300,000	$ -	$ -	$ -	$ -
Power Cost*	$ 24,000	$ 27,000	$ 27,000	$ 35,000	$ 1,000
Chemical Cost*	$ -	$ -	$ -	$ -	$ -
Labor Cost*	$ 20,000	$ 9,000	$ 12,000	$ 5,000	$ 1,000
Maintenance Cost*	$ 37,000	$ 24,000	$ 21,000	$ 14,000	$ 3,000
Water Usage Cost*	$ 4,000	$ -	$ -	$ -	$ -
Annualized Cost (4% for 20 years)	$ 419,000	$ 383,000	$ 481,000	$ 453,000	$ 294,000
Annual O & M Cost*	$ 85,000	$ 60,000	$ 60,000	$ 54,000	$ 5,000
Total Annualized Cost	**$ 504,000**	**$ 443,000**	**$ 541,000**	**$ 507,000**	**$ 299,000**

*Additional cost for 2030 operation as compared to current operation costs

All options in Table 11.9 are similar in that they provide capacity to 2030 and beyond. The table shows the alternative that is estimated to be the least expensive is to build a new ditch for initial use as an equalization basin. (Other factors that should be considered include space required, environmental impacts, and effect on operations.) By 2030 the equalization basin will need to be equipped to function as a third oxidation ditch. All options require a fourth secondary clarifier by the year 2060. Detailed capital and operation and maintenance costs can be found in Appendix F.

11.4 BIOSOLIDS HANDLING ALTERNATIVES

The City of Ashland must have a reliable means of disposal for its sludge, since it is produced on a continuous basis and there is limited existing storage on-site. The estimated amount of sludge produced is shown in Table 11.10.

TABLE 11.10: Estimated Average Annual Biosolids Produced

Description	Year	
	2010	2030
Avg. wet tons per day	10.0	12.8
Avg. dry tons per day*	2.0	2.6

*Based on avg. 19% TS from centrifuges.

Depending on the level of treatment, biosolids may be sold or given to the public as fertilizer, applied to agricultural land, or hauled to a landfill. Currently the City of Ashland disposes of their unstabilized dewatered sludge in the Dry Creek Landfill, and has adequate sludge storage and treatment facilities to manage their sludge through 2030 if this practice continues. If this option should become unavailable or if it is desired to beneficially reuse the biosolids for fertilizer, the City would be required to treat their sludge to produce Class A or B biosolids. Therefore, it is important that a backup disposal plan be identified.

11.4.1 Biosolids Disposal Options

Class B biosolids can be applied to agricultural land. There are two potential alternatives for land application: agreement with a local farmer to take the biosolids, or City purchase of farm land for a disposal site. In either case, biosolids can only be applied so as not to exceed heavy metals loads to the soil, or nitrogen and phosphorus crop uptake limits. In most cases, nitrogen loading to crops typically governs and can be used to estimate the amount of land needed.

It is anticipated the City should be able to locate a farmer to take their biosolids within 25 miles of the wastewater treatment plant. That assumption will be used in estimating a cost for this alternative. The disadvantage to this alternative is that a long-term agreement with the farmer would be needed and the City would need to work around the farmer's planting and harvesting schedule.

The second land application alternative would be for the City to purchase land for biosolids disposal. This would allow the City to grow whatever crop they desired with higher total nitrogen uptake levels, and not have to rely on a second party. In lieu of the City conducting farming operations, the City could lease the land to a farmer who would handle farming operations and assist in spreading biosolids in exchange for crop profits.

It should be noted for either land application alternative that biosolids would need to be stored from approximately November through April when farming operations cease due to cold weather. Biosolids cannot be land applied when snow is on the ground or the ground is frozen. In addition, getting on fields to apply biosolids in wet spring weather would result in rutting and damage to the field.

Class A biosolids may be used on City property, given away to farmers and citizens, or potentially sold. Though Class A biosolids have fewer use restrictions than Class B, there may be some application restrictions with Class A biosolids depending on the method selected for stabilization.

11.4.2 Biosolids Process Alternatives

Biosolids management may include thickening, stabilization and dewatering processes. Thickening is often used prior to stabilization to increase waste activated sludge (WAS) solids concentrations that are typically less than 1%. (The benefits of thickening include reductions in the volume of digester tanks and related equipment, chemical requirements, and operation and maintenance (O&M) costs.) Stabilization is provided to address EPA's 503 Rule requirements for pathogen reduction and vector attraction reduction (VAR); the Part 503 Rule offers different stabilization options to obtain Class A or B biosolids. Dewatering is used to increase the solids content and reduce the final volume of biosolids for disposal.

Table 11.11 summarizes equipment commonly used in the various steps of biosolids treatment. The biosolids treatment steps and options are discussed in following sections.

TABLE 11.11: Equipment Commonly Used in Treating Biosolids

	Thicken	Stabilize	Condition/Dewater
Equipment	Dissolved Air Flotation Tank	Aerobic Digestion	Belt Press
	Membranes	Chemical Treatment	Centrifuge
	Centrifuge	Composting	Screw Press
	Gravity Thickener	Anaerobic Digestion	Drying Bed
	Gravity-Belt Thickener	Dryers	Dryers
	Rotary Drum	Alkaline Stabilization	

Thickening

Thickening can be achieved through a variety of methods, many of which are similar to dewatering processes. Some of the more common methods are:

Dissolved Air Flotation Thickening (DAFT)
Air and WAS are pumped into a small pressurized tank simultaneously. Upon lowering the tank pressure, air bubbles float to the top of the tank carrying the heavier particles which creates a thickened WAS layer (3-6% solids) on top that can be scraped off. DAFTs typically have high power costs, can be large and may need to be enclosed in a building.

Membrane Thickening
Membranes can be used to remove water from WAS just as they are used to remove water from mixed liquor in the MBR treatment process. Removing this water effectively thickens the WAS. Because water can be removed from the WAS at a much lower rate than that used for the MBR process, old membranes can be reused for thickening.

Centrifuge
The City currently utilizes centrifuges for solids dewatering. A centrifuge can be modified to thicken by controlling the speed at which the equipment spins and the duration. Chemicals, typically polymer, are used to condition the sludge prior to thickening, with the quantity dependent on the type of polymer and waste solids, and the percent solids to be achieved. With multiple parts spinning at high velocities, centrifuges commonly cost more to maintain than other types of thickening equipment.

Gravity Thickener
A gravity thickener is similar to a clarifier. Gravity thickeners are not typically used for waste activated sludge facilities because the sludge does not flocculate and settle well due to sludge age.

Gravity-Belt Thickener
A gravity-belt thickener uses a belt to pass WAS between rollers where water is removed by compressing the belt. Polymer is typically used to aid dewatering. A variable speed pump is used to control the rate at which WAS is pumped to the belt press.

Rotary Drum Thickener
A rotary drum first mixes the WAS with polymer to condition the sludge. As the screen slowly rotates, the solids are directed up the inclined drum screen surface until they drop off of the discharge end of the cylinder, while the separated water decants through the screens. A rotary drum thickener can increase WAS concentrations from 0.5-1.0% to a solids content of 4-9%.

Currently, the City does not have a means of thickening WAS prior to lime stabilization. Due to space limitations on the site and the lack of existing tankage which could be converted to aerobic digesters, thickening does not provide any real benefits to improve the current sludge handling program and is not recommended for future implementation.

Sludge Stabilization

Sludge stabilization processes that could potentially be used at the Ashland WWTP include the following:

<u>Cannibal™ Aerobic Digestion Process</u>
The Cannibal™ digestion process is a patented, relatively new stabilization method that appears to have the potential to significantly reduce the quantity of biosolids. Initially, WAS is sent to a solids separation module (consisting of a very fine screen) that removes grit and inert material from the WAS and dewaters it to 30-40% dry solids for landfill disposal. This step eliminates up to 20% of WAS content, and the remaining material is sent to a combination of aerated digesters where tanks alternate between aerobic and anoxic conditions. These conditions allow microorganisms to consume each other, thus reducing the total amount of biosolids.

This process was installed in Ashland, Nebraska in conjunction with a vertical loop reactor treatment process similar to an oxidation ditch. To optimize solids removal in the Cannibal™ system, the vertical loop reactors are operated in series with the first tank run as an anoxic tank and the second run as an aerobic tank with the minimum amount of air supplied to maintain an aerobic environment. There have been difficulties with the rotary drum screen, freezing of airlines to the Cannibal™ tanks due to water getting into the lines, and difficulty in maintaining the process to maximize solids destruction. If the tanks do not alternate properly between aerobic and anoxic conditions, solids destruction is reduced. This process appears to be promising, but is not recommended until operation and maintenance procedures are more fully documented to ensure successful performance.

<u>The Neutralizer® (Class A biosolids) and Clean B™ (Class B biosolids) Processes</u>
These processes use a patented chemical process to stabilize solids to Class A or Class B standards. (While the processes are proprietary, the chemicals can be purchased from any chemical supplier.) The Neutralizer® requires that WAS be thickened to 4%, and then placed in a tank with a patented chemical mixture for a specified length of time. Following this chemical process, the pH is lowered and nitrate is added. The final step in the process is to raise the pH and dewater the sludge. The entire process can be completed within 8 hours. The Clean B™ process treats WAS in a 10-minute process with a patented chemical treatment.

These processes are currently being used in Florida at several facilities, and BCR Environmental has worked with EPA to obtain approval for the use of the Neutralizer process as a Class A technology. However, these processes are untested in the Pacific Northwest and the Oregon DEQ is not familiar with this technology so there would be an educational process in order to get approval of a sludge management plan.

Due to the short processing time, these stabilization processes require less space than typical stabilization methods. Another reported advantage of these processes is improved dewatering characteristics which will reduce polymer usage and increase final solids content in the dewatered biosolids. However, substantiating these claims would require a pilot study due to the variability of sludge produced at a wastewater plant. Disadvantages of the Neutralizer process compared to other Class A technologies include high capital costs, high

O&M costs, and the potential for bacterial regrowth if storing the product during the winter months (BCR does not recommend storing their product longer than three to four weeks). This technology may be a viable sludge management alternative, but more information is required before it could be recommended.

Composting

Composting can be done in windrows, aerated static piles or contained vessels. Before composting can occur, the sludge must be dewatered and mixed with bulking agents to adjust solids concentration, moisture, and the carbon to nitrogen ratio. Depending on the operation, composting can produce Class A or B biosolids.

There are three main types of composting biosolids which include windrow, aerated static pile, and in-vessel composting. All methods include mixing dewatered sludge with a bulking agent (e.g. wood chips, sawdust, leaves) before composting. Various sizes of windrows are formed depending on the windrow turning equipment. The piles are mechanically turned periodically to aerate the piles and maintain proper composting temperatures. This method is usually the least expensive, but has the potential to create odor issues when the piles are turned. Windrow composting is not recommended for the City of Ashland since the process is more difficult to control and the City would be composting unstabilized sludge with a higher potential for odors.

For the aerated static pile method, piles are formed on top of perforated piping which provides air to regulate composting temperatures. Air can be pulled through the piles using negative pressure or forced through the piles using positive pressure. The piles are covered with finished compost to prevent odors from escaping during the aeration process. If negative pressure is used for aeration, the air from the piles can be treated using a biofilter as it discharges to atmosphere. Positive pressure aeration forces the air through the pile and the finished compost layer before it reaches the atmosphere which helps remove odors. The temperature of the piles is controlled by the airflow which can be regulated by turning the blowers on and off as desired. The piles are not disturbed until the composting process is complete and the product can be moved to a curing area. The aerated static pile process is a viable sludge stabilization method for the City of Ashland.

In-vessel composting takes place within a closed system which limits the Operator's exposure to the composting sludge. However, these systems are typically the most expensive, require the most maintenance, and can only treat a fixed volume of compost at one time. They do not provide the flexibility to easily expand the composting area. For these reasons, this composting method is not recommended for further consideration.

Digestion

Digestion can be accomplished in anaerobic or aerobic processes. Anaerobic digestion involves the decomposition of organic and inorganic matter in the absence of oxygen, and produces Class B biosolids with 15-20 days solids retention times (mixed and heated). Aerobic digestion involves mixing WAS with air to allow decomposition under aerobic conditions. To meet Class B requirements with aerobic digestion, the biosolids must be retained in this aerobic environment for at least 40 days at 20°C or 60 days at 15°C [3].

The advantages of an anaerobic process include excellent volatile solids reduction, rapid response to substrate addition after long periods without feeding, and cogeneration potential using methane gas. Compared to aerobic digestion, anaerobic processes require a smaller reactor volume, have lower energy requirements, produce less digested sludge, and produce digested solids with better dewatering characteristics.

The advantages of an aerobic process include production of an odorless biologically stable end product, recovery of more of basic fertilizer values in the sludge, simple operation, and suitability for digesting nutrient-rich biosolids. Anaerobic digestion has lower capital costs than anaerobic digestion, and produces a lower BOD concentration in supernatant

Both anaerobic and aerobic digestion would benefit from the use of thickeners to reduce the size of digesters required. The low energy requirements with anaerobic digestion may offset the higher initial capital costs involved when costs are annualized. In addition to high capital cost, the main disadvantages of an anaerobic process for the Ashland WWTP are the complexity of operation, the adverse effect of lower ambient temperatures on reaction rates, treatment costs of ammonia in sidestream return, and high potential for offensive odors. In addition, the Ashland plant only produces WAS which is less compatible with anaerobic than aerobic digestion. Aerobic digestion is not recommended for further consideration due to space requirements for the basins, and high operation and maintenance costs.

<u>Dryer</u>

A dryer is one option that can produce Class A biosolids from unstabilized, dewatered biosolids in a small footprint. Dryers typically have high energy costs, but they significantly reduce the volume of solids produced. The final end product is well stabilized, and not subject to bacterial regrowth odors when stored in a dry environment before land application.

The Ashland plant could continue to store WAS in the existing sludge holding tank, and then dewater the sludge to 18%-20% solids prior to conveying the solids to a hopper for the dryer. The solids then pass through the dryer equipment to produce biosolids with a minimum 90% solids content. A covered storage area would be required to store the biosolids prior to land application.

<u>Alkaline Stabilization</u>

There are two types of alkaline stabilization. One involves liquid lime (pre-lime) stabilization per the facilities at the Ashland plant. This method of stabilization is typically used with vacuum filters or recessed plate filter presses. Though it can also be used with belt filter presses or centrifuges, this is not a common practice due to increased scaling and equipment maintenance issues. The Ashland process produces Class B biosolids, which are subject to more restrictive land application practices to meet VAR requirements.

There are also proprietary alkaline stabilization processes which produce Class A biosolids in specially designed equipment. The WAS is first dewatered to 18%-20% solids, and then quicklime and sulfamic acid are automatically added and well mixed within a plug flow reactor to provide the correct amount of time at a specified temperature and pH to meet Class A requirements. No auxiliary heat is required, as the chemical reaction produces adequate temperatures to meet the 503 regulations. The unit includes a wet scrubber to prevent odors at the equipment discharge.

A major disadvantage of the alkaline stabilization process when compared to dryers, digestion and chemical treatment, is the increased solids production (due to lime addition) which will require additional storage during the months when land application is not possible. Also, there is a potential for bacterial regrowth and odor issues if the biosolids are stored for several months before land application though this is less likely than with other chemical treatment processes due to the high initial pH.

Dewatering

Biosolids dewatering and conditioning make up the final operation in managing biosolids prior to disposal. Dewatering is a physical process in which the biosolids are condensed through various means. The dewatering process may also be assisted by chemical addition (typically polymer), which is called conditioning. Several approaches that might be employed to dewater stabilized biosolids include:

<u>Belt Filter Press</u>
A belt filter press applies pressure to the biosolids to squeeze out the water with a system of belts and rollers, producing a cake with a typical solids content of 18-22%. Although there are a variety of designs, most have the same basic configuration: polymer conditioning zone, gravity drainages zones, low pressure compression zone, and a high pressure compression zone.

<u>Centrifuge</u>
The City of Ashland currently uses centrifuges for dewatering WAS. Typically, the polymer used to assist the dewatering process is applied at a rate of 10 - 16 lbs. of polymer per dry ton of biosolids. Currently, Ashland is using over 40 pounds of polymer per dry ton of sludge to produce 18 – 20% solids concentration in dewatered solids. It is recommended that the City perform a polymer optimization study to determine if another type of polymer is more effective and would reduce the annual cost for purchasing polymer.

<u>Screw Press</u>
A screw press consists of an inclined edge wire screen basket with 0.01" openings. Liquid sludge mixed with polymer flows to the screw press where a slowly rotating screw, driven by a variable frequency drive motor, conveys the sludge upward through the inclined screen basket. A lower section of the basket serves as pre-dewatering zone where free water drains by gravity. A second section of the basket with a reduced diameter serves as pressure zone where the sludge is compressed between narrowing flights of the screw to produce a dewatered sludge cake.

Advantages and disadvantages of the belt filter press, the centrifuge, and the screw press are compared in the following table. It is recommended that the City continue to use their centrifuges until they exceed their useful life, since maintenance has been minimal and the equipment is working well. If a third unit is required due to increased flows, it is recommended that the City conduct a pilot study to determine if there is a more cost-effective method of dewatering or if the centrifuge is still the best type of equipment for Ashland.

TABLE 11.12: Dewatering Equipment Comparison

	Belt Press Filter	Centrifuge	Screw Press
Advantages	▪ Low energy requirements ▪ lower capital and operating costs ▪ Dry cake	▪ Good odor control ▪ Lower capital cost ▪ High capacity to building area ratio ▪ Dryer Cake	▪ Good odor control ▪ Minimal operator attendance ▪ Low polymer and wash water use
Disadvantages	▪ Potential odor problems ▪ Sensitive to incoming sludge feed characteristics ▪ Automatic operation generally not advised	▪ Potentially more maintenance issues ▪ Sensitive to grit ▪ Higher suspended solids content in concentrate	▪ Sensitive to incoming sludge feed characteristics

<u>Dryer</u>
Use of a dryer to evaporate the water content from the sludge can reduce the moisture content of biosolids below that achievable by conventional dewatering methods. Drying can be done by convection, conduction, radiation, or a combination. Compared to other available dewatering technologies, dryers have high capital costs and also high energy costs associated with producing the extreme heat required to burn the volatile solids. A thermal dryer was evaluated to produce Class A biosolids from dewatered, undigested sludge. Dewatering prior to drying significantly reduces energy usage and drying time, so a dryer should not be considered a substitute for other means of dewatering previously discussed.

A solar dryer was also investigated as an environmentally friendly method of dewatering. However, this technology was found to be unsuitable for undigested sludge due to large space requirements, high capital costs, the inability to regulate the drying process which is dependent on weather conditions, and the inability to produce 90% solids on a consistent basis (required to meet the vector reduction requirements in the EPA 503 regulations).

<u>Sludge Drying Beds</u>
Drying beds could be used to dewater the biosolids up to 90% dry solids, assuming extended warm temperatures and minimal precipitation. The increased solids concentration can significantly reduce hauling costs to a landfill or farm land application site. Drying beds can be solar, paved, vacuum-assisted, or sand. The downside to a drying bed is the space requirement, sensitivity to climate, and potential odor problems. This technology was not evaluated further, since the City elected to not proceed with a drying bed project in the 1990's due to public perception.

11.4.3 Summary of Pre-Screening of Biosolids Options

Of the sludge management alternatives presented, the top three options recommended for comparison by Keller Associates include continuing the current approach (no treatment, landfill disposal), thermal drying to produce Class A biosolids, and composting to produce Class A biosolids. (Sludge thickening is not currently provided and there is no room for expansion on the site to provide aerobic digestion. Therefore, neither thickening nor digestion is recommended for the Ashland WWTP at this time.) The Technical Review Committee requested that Recology Ashland be contacted regarding the operation of a biosolids composting operation with the City. The City also requested that the composting alternative be modeled around the City of Grants Pass, Oregon. Approximately ten years ago, the City of Grants Pass constructed a co-composting site which accepts green waste from the community for composting with biosolids. Based on this input, the following three options were evaluated in more detail with cost estimates and environmental impacts considered:

1. Dewater sludge using the existing centrifuges, and haul to the landfill for disposal.
2. Dewater sludge using the existing centrifuges, and compost to produce Class A biosolids for sale to commercial businesses and individuals.
3. Dewater sludge using the existing centrifuges, and dry using a thermal dryer to produce Class A biosolids for sale to commercial businesses and individuals.

Status Quo Alternative (Dewater and Landfill Sludge)

The City currently provides centrifuge dewatering with two units for redundancy. Under this alternative it is recommended that the City continue to utilize the current equipment until it needs to be replaced. There is additional space for a third unit as solids production increases. If equipment must be replaced in the future, the City should perform a pilot study to determine if other types of dewatering equipment would provide more efficient dewatering. The City currently utilizes approximately 45 lbs of polymer per ton of dry solids produced. It is recommended that the City test other types of polymer on a periodic basis to optimize dewatering operations and possibly reduce chemical costs.

TABLE 11.13: Landfill Disposal

Pros	Cons
▪ Operator familiarity with process operation	▪ Tipping fees will continue to increase disposal costs
▪ No new facilities required	▪ Practice does not decrease the quantity of solids to be handled and disposed
▪ Lowest annual cost	▪ Biosolids are not beneficially reused
	▪ Future regulations could prevent the landfill from accepting dewatered sludge

Dewatered sludge is hauled to the Dry Creek Landfill for final disposal. Sludge disposal increases methane production in the landfill for energy recovery. According to Dry Creek Landfill personnel, they perform a TCLP (toxicity characteristic leaching procedure) to verify that the material is classified as municipal solid waste, and occasionally test for metals. The landfill reportedly has a remaining life of 70 to 100 years, so this appears to be a viable disposal option unless solid waste regulations are modified in the future to prohibit sludge disposal in a landfill.

Dewater and Compost to Produce Class A Biosolids

Dewatering would occur as described in the previous option. The City would need to site and construct a new composting facility. The Dry Creek Landfill currently composts yard waste and food wastes at their site. They are not interested in accepting biosolids for composting since they currently have a certified organic product. There is evidence that personal care products and medications do not break down during the composting process. Therefore, biosolids compost may not meet the organic certification requirements. In Oregon, cities can compost biosolids under their water quality permit. The Dry Creek Landfill has a composting permit which is regulated under the solid waste permitting program. They are currently prevented from accepting all biosolids except Class A.

Recology Ashland was also contacted regarding their interest in composting biosolids. Though Recology has over 40 recycling and/or composting sites in Oregon and California, there are currently none near Ashland so a new facility would need to be constructed. This company has not composted biosolids to date due to concerns with metals, medications, and residues from personal care products. Recology typically operates facilities that accept "green waste" (i.e. grass, leaves, branches, etc.) and food wastes for composting to produce a certified organic product. They currently collect green waste and food waste and haul it to the Dry Creek Landfill for composting. They are interested in discussing partnering with the City on a biosolids composting site. However, there would be several issues to resolve in order for this partnership to be viable.

The City of Grants Pass, Oregon was contacted regarding their co-composting facility, "JO-GRO™." Their Public Works Director provided several reports describing the facility and their operation and maintenance budget. Grants Pass has a population of approximately 34,000 and produces approximately 45% more biosolids on an annual basis than the City of Ashland. (In 2000, when the facility was designed, the City produced approximately 90 dry tons of biosolids per month and biosolids production was expected to increase by 22% to 110 dry tons per month in 2010. Ashland currently produces approximately 60 dry tons per month, for a population of about 21,000.)

The City of Grants Pass uses an extended aerated static pile process, which is also recommended for the City of Ashland. The composting site, located at the Merlin Landfill, has been in operation for approximately 10 years. Wood waste and yard debris are shredded and mixed with biosolids (approximately 3 parts yard waste to 1 part sludge). The compost mixture is placed on 12" of bulking agent to aid the aeration process. The piles are then covered with a layer of finished compost or screenings. To produce Class A biosolids, the temperature in the pile must be maintained at 55 °C or higher for three days. To meet vector attraction reduction, the compost pile must achieve temperatures higher than 40°C for 14 days or longer. The JO-GRO™ Operations Manual estimated that composting would require 21 to 28 days. After the active composting period, the material is moved to the curing building where it is aerated and allowed to cure for an additional month. The cured compost material is screened to recover bulking agent and stored prior to sale.

Actual construction costs for the JO-GRO™ facility were not provided. However, the estimated construction cost in 2000 was approximately $2,000,000. Operation and maintenance costs for the site in FY2010 were $502,000. Reportedly, they sell all of their compost for $15 for the first 2 cubic yards and $18 for each additional cubic yard. Yard waste can be recycled by the public at a cost of $1.00 per cubic yard, and wood waste is accepted at a cost of $2.00 per cubic yard. This income is used to defray operating costs, with the rest of the budget subsidized by the Wastewater Department.

Composting facilities for Grants Pass consist of a green waste drop-off site with trailer, a wood waste drop-off area, a co-compost building with a biofilter, a curing building, and a storm water pond. Drainage from the composting area drains to a holding pond which is periodically pumped out and hauled to the wastewater plant for treatment. The curing building was an existing building used by the previous composting program. There would be additional capital costs for Ashland associated with obtaining a site and providing building since there are no existing buildings to be reused. However, the buildings could be smaller since sludge production is much lower than the City of Grants Pass.

TABLE 11.14: Co-Compost Site

Pros	Cons
▪ Biosolids could be sold to the public for reuse	▪ New facility would need to be sited and land purchased
▪ Beneficial reuse of biosolids as a fertilizer and soil conditioner	▪ Potential for competition with Dry Creek for yard waste
▪ Provides a primary means of disposing of biosolids which does not rely on the landfill continuing to accept biosolids	▪ Higher cost than landfilling due to new construction, operation and maintenance
▪ Lack of landfill fees and sale of product will offset composting cost	▪ Potential for odors and neighbor complaints

Dewater and Use Thermal Dryer to Produce Class A Biosolids

A thermal dryer using indirect heat to dry a continuous flow of sludge was used to evaluate this sludge management alternative. A thermal dryer would require that sludge be dewatered using the centrifuges to a minimum solids concentration of 12%. However, a higher solids concentration in the feed sludge will increase the processing capacity of the unit. The dryer system includes an odor control system. A new building would need to be constructed to house the thermal dryer and associated equipment and to provide a covered storage area for the dried biosolids. The existing dewatering building does not have enough space to install the dryer and ancillary equipment adjacent to the centrifuges. The dryer discharges approximately 150 gallons per minute of cooling water at a temperature of 170°F - 180°F. A cooling tower has also been included in the cost analysis should this water increase the plant discharge above acceptable limits. Unstabilized sludge must be dried to a minimum solids concentration of 90% to meet VAR requirements. Class A requirements are met by achieving a certain temperature for a specified length of time.

TABLE 11.15: Thermal Dryer

Pros	Cons
▪ Biosolids could be sold to the public for reuse	▪ A new facility would need to be constructed at the wastewater plant site
▪ Beneficial reuse of biosolids as a fertilizer and soil conditioner.	▪ High power costs
▪ Provides a primary means of disposing of biosolids which does not rely on the landfill continuing to accept biosolids.	▪ Odors can be an issue in cooling water.
▪ Finished product has 90% minimum solids which reduces storage and hauling costs.	▪ Cooling tower required to reduce drain water temperatures.

11.4.4 Annualized Cost Comparison of Selected Biosolids Handling Alternatives

Table 11.16 presents the life cycle cost analysis of the alternatives selected for further evaluation. This includes operation and maintenance costs as well as capital costs. Revenues generated through the sale of Class A biosolids have not been included in the cost analysis since the local market is unknown.

For cost comparison purposes, the construction cost for composting will be based on the opinion of cost for the JO-GRO™ facilities and JO-GRO™ operation and maintenance costs will be pro-rated based on sludge production. The costs for the thermal dryer alternative include one thermal dryer with a capacity of 4500 pounds per hour of sludge with a 19% solids concentration. The dryer would operate a minimum of 40 hours per week. If repairs were required on the unit, the City would need to temporarily landfill sludge as an alternate method of disposal.

TABLE 11.16: Comparison of Costs for Selected Sludge Management Alternatives

	Dewater / Landfill	Dewater / Co-Compost	Dewater / Thermal Drying
Total Capital Cost (2011 dollars)	NA	$ 2,000,000	$ 4,100,000
Electric/Natural Gas Cost			$ 66,000
Chemical Cost (polymer for dewatering)	$ 44,000	$ 44,000	$ 44,000
Startup Cost			$ 6,000
Labor Cost (dewatering, hauling off-site to landfill or compost site, etc.)	$ 84,000	$ 84,000	$ 146,000
Maintenance Cost (existing centrifuge maintenance included)	$ 4,000	$ 4,000	$ 9,000
Landfill Disposal	$ 220,000		
Composting O&M*		$ 293,000	
Annualized Cost (4% for 20 years)	NA	$ 147,000	$ 302,000
Annual O & M Cost	$ 352,000	$ 425,000	$ 271,000
Total Annualized Cost	**$ 352,000**	**$ 572,000**	**$ 573,000**
Potential Revenue from Sales of Class A Product	NA	$0 - $18/CY**	Unknown***

*Composting O&M is based on 52% of the FY 2011 JO-GRO budget and includes wages and benefits, supplies, contractual services, machinery and equipment necessary to operate the composting facility.
** JO-GRO sells their product for $15 - $18/CY.
*** Markets were not identified for dried biosolids. The City would need to develop a marketing strategy and identify customers for the end-product.

Based on the costs shown above, continuing to landfill is the least expensive alternative. Producing Class A biosolids will cost more than the “no treatment” alternative. Thermal drying and co-composting are approximately the same annualized cost. However, thermal drying facilities could be located at the existing Wastewater Treatment Plant. The benefits of composting include more product that can be sold. Composted biosolids contain more moisture and are similar in texture to soil. Dried biosolids have a granular appearance like fertilizer. One benefit of drying biosolids is that a much smaller quantity is produced which must be hauled and stored since the finished product is a minimum of 90% solids to meet Class A requirements.

Both compost and dried biosolids can be sold to generate revenue to offset the cost of sludge treatment. Ashland’s residents are able to recycle yard waste through the Dry Creek Landfill which already produces a compost product for sale to the public. Recology Ashland sells the Dry Creek compost at their transfer station. The City would need to investigate the market for biosolids compost or dried biosolids prior to implementing a program. Oak leaf compost sells for $4 per bag (1.5 cubic feet), but does not have any facility that utilizes municipal biosolids in their compost. The City of Grants Pass sells their compost at a much lower price, which may appeal to commercial businesses and residents.

References

1. Daigger, G.T. and Parker, D.S.: *Enhancing Nitrification in North American Activated Sludge Plants,* Water Science and Technology, Vol. 41, No. 9.

2. http://www.brentwoodprocess.com/ifas_main.html

3. U.S. Environmental Protection Agency, Office of Wastewater Management: *A Plain English Guide to the EPA Part 503 Biosolids Rule;* EPA/832/R-93/003, Washington, DC, 1994.

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12.0 RECOMMENDED TREATMENT PLANT IMPROVEMENTS

Improvements to address existing plant limitations were identified and prioritized in Chapter 9, disposal options were analyzed in Chapter 10, and alternatives to handle growth and expected effluent limits were evaluated in Chapter 11. This chapter provides a summary of the treatment planning objectives and recommendations to accomplish these objectives.

12.1 OBJECTIVES

12.1.1 Eliminate NPDES Permit Violations

The CBOD5, TSS, ammonia, phosphorus, and E. coli violations that have occurred were rare and appear to be related to plant operations rather than a deficiency in equipment or processes. Plant operations have previously made adjustments to better prevent future violations and as a result permit violations in the last 5 years have been minimal.

Dissolved Oxygen

As mentioned in Chapter 9, it is recommended to revisit the current DO limit with DEQ. The current limit requires supersaturation during shoulder season months. If the current limit remains in place, it will be necessary to add pure oxygen or inject air into the effluent wastewater prior to discharge to meet the DO effluent limit.

Excess Thermal Load

Trading (shading) is recommended as the alternative for complying with the temperature limit. Trading/shading can be best accomplished by entering into an agreement with an implementation organization such as The Freshwater Trust. To address local plume concerns, the City will also need to relocate their existing outfall to Bear Creek and potentially construct some local wetland improvements. It is also recommended to pursue reuse where feasible as a beneficial means of supplementing potable water demands while also lowering the thermal load output from Ashland WWTP.

12.1.2 Prevent Plant Deficiencies

Bottlenecks

The pipe from the oxidation ditch to the clarifiers reaches capacity around 2030 flows. The pipe can either be replaced with a larger pipe or a second pipe can be added when a third oxidation ditch is constructed.

Peak flow

As discussed in Chapter 11, the shell of a new oxidation ditch is the apparent best alternative to handle peak flows and should be constructed in the next 5 years. This would serve as an equalization basin during high flow events. As flows increase (prior to 2030), it can be equipped to operate as a second oxidation ditch. In the interim, the City should pursue infiltration and inflow reduction improvements in the collection system that could potentially delay necessary plant improvements related to peak flows. The I/I reduction cost evaluation may also be required if DEQ is to fund improvements required by peak flows.

12.1.3 Stay Ahead of Growth and Maintain Equipment

Ashland Creek Lift Station

The current pumps are fairly new but current peak hour flows are at 82% of design capacity. The City plans to have a portable pump on-site as a backup during peak flow conditions. The existing pumps should be replaced with larger pumps by the year 2045.

Screening

There is currently capacity to 2030. Life expectancy of the existing screen is also to 2030. A redundant automated screen is recommended for consideration as a future project.

Grit Removal

The grit removal system has sufficient capacity to 2030**.** Its useful life will likely expire around 2025 and replacing the grit system with a larger grit system is recommended at that time.

Oxidation Ditch

Replacing the aerators at the end of their useful life should be budgeted in the year 2030. (Also see 12.1.4.2 for adding a third oxidation ditch).

Secondary Clarifiers

Current capacity is to 2050 provided that a new oxidation ditch is constructed which allows plant operation at a mixed liquor concentration of less than 3000 mg/L. The clarifier internals for clarifiers 1 and 3 were installed around 2000. A life expectancy of 30 years would result in recommended replacement in 2030. Clarifier #2 mechanism was not replaced as part of the 1998 project. Plant staff have not given any indication of wear or failure for clarifier #2. Replacement as part of a 2020 improvements project would be recommended unless only minimal wear is detected.

RAS Pumps

The existing RAS pumps should be replaced with larger pumps once peak flows are commonly above 6.5 MGD. The existing peak day flow is 7.1 MGD. The existing peak week flow is 5.5 MGD. If consecutive days above 6.5 MGD were experienced plant operations would be challenged to recycle enough flow to remove solids buildup and prevent carry over. Based on this, larger RAS pumps should be planned for by 2020.

UV Disinfection

There is sufficient UV treatment capacity to 2030; however, by 2020 additional UV hydraulic capacity will be necessary to accommodate the projected flow increases.

Membrane Filtration

A membrane replacement schedule is provided in Chapter 9. The schedule is based on expanding membrane surface area to keep up with expected capacity increases as well as replacing membranes to account for the life expectancy. The membrane feed pumps will also need to be upsized by 2023 (as called for in the schedule).

Alum Feed Pump

By 2025 it is projected that additional alum feed pump capacity will be required.

12.1.4 Improve Solids Handling

The City has a desire to eventually produce a Class A biosolids. The costs associated with this venture make the existing landfill disposal option better for the short term. Due to uncertainties regarding tipping fees and the ability to continue to dispose biosolids at the landfill, the City's mid and long term plans include:

1. Continue to explore partnering opportunities with Recology and other composting ventures where a third party would manage the composting facility. (BCR Environmental was a company that was studied as a possible equipment provider and partner for composting). If these opportunities prove viable, co-composting should be reconsidered.

2. As a contingency plan, prepare to implement a thermal dryer to produce Class A biosolids within the next 10 to 20 years. The City could also explore options such as solar panels to offset energy cost associated with the thermal dryer.

12.1.5 Improve SCADA System

Improving the existing plant SCADA system to more operator friendly windows and automatic data logging is recommended to add plant staff in better tracking plant operation parameters. A preliminary design report which provides specific design guidelines should be performed prior to implementing the improvements.

12.2 SELECTED TREATMENT IMPROVEMENTS

The improvements summarized above are depicted in Figure 12.1 (Appendix A) with recommended timing (priorities). The major improvement in the short term is the construction of the shell of a third oxidation ditch. This option is developed in the figure at a concept level.

12.2.1 Description

The recommended alternative is to build the outer shell of a new oxidation ditch, and use it as an equalization basin until additional treatment capacity is needed. It is anticipated that the equalization basin will need to be equipped to function as a third oxidation ditch after 2030. An additional clarifier and centrifuge will also be needed in the future. Other factors to be considered include space required, environmental impacts, and effect on operations.

12.2.2 Land Availability

The wastewater treatment plant site is space constrained. Proceeding with the recommended option of constructing the outer shell of a third oxidation ditch is dependent on being able to obtain land adjacent to the existing oxidation ditches from the Parks Department. If land cannot be obtained from the Parks Department, it is recommended that the City proceed with staged aeration as the next best option.

12.2.3 Environmental Impacts

Most of the improvements will be constructed within the existing plant site, and thus will have minimal environmental impacts since all the ground on the site has previously been disturbed. However, some of the priority improvements will be located outside the current property boundaries, including the outfall relocation, fish screen, and third oxidation ditch (Priority1), and the fourth clarifier (Priority 3). These improvements involve several Water Resource Protection Zones as shown on the City's Official Water Resources Map, including Ashland Creek and a section of Bear Creek north of the Bear Creek Greenway, both designated as Riparian Corridors; a Locally Significant Wetland (W1) along the western boundary of the treatment plant; and Possible Wetlands (PW) east of the treatment plant.

Relocating the outfall (with fish screen) to Bear Creek will involve work in the designated Riparian Corridor, which has potential temporary impacts to water quality and fish. Use of BMPs would be required to prevent adverse impacts to the creek water quality, and coordination with ODFW would be necessary to appropriately schedule the outfall construction in order to minimize adverse impacts to the fish population.

Construction of the third oxidation ditch where shown on Figure 12.1 would encroach on a designated Locally Significant Wetlands area, triggering the need for a permit and mitigation process. The mitigation process would require a survey of delineated wetland boundaries, and a determination by the Oregon State Department of Lands that project alternatives that would minimize or avoid encroaching on wetlands have been adequately explored. Compensatory mitigation would then be considered in order to allow placement of fill or excavation in the wetland. For compensatory mitigation, project impacts on wetland acreage and functions would be evaluated to determine the amount of in-kind wetlands replacement needed to offset the loss of wetlands. At a minimum, one acre of replacement wetlands would be needed for every acre lost.

On the other hand, environmental impacts associated with constructing the fourth clarifier where shown on Figure 12.1 would be minimized by adjusting the final location to avoid encroaching on the Possible Wetlands area as identified in the field. Some temporary impacts could be expected during construction due to the need to cross the Bear Creek Greenway to install connecting piping, but no long-term impacts are anticipated.

12.2.4 Operation Theory

Initially only the outer shell of the third oxidation ditch would be constructed, and the ditch would be used as an equalization basin for large flow events (5-year storm). The existing flow splitting structure ahead of the anoxic basins can be modified to automatically send peak flows to the equalization basin. Once the peak event has passed, the water stored in the equalization basin can be pumped back to the splitter box. Though the large flow events are not frequent, they have the potential to wash solids over the clarifier weirs and cause a permit violation. Using the equalization basin to remove peaks will also allow the plant to meet treatment requirements during maximum month flows until the ditch internals are constructed as part of Priority 2 improvements. Finally, if it is necessary to take a ditch offline for maintenance, the equalization basin can be used during low flow summer months to equalize the flow to the remaining online ditch.

12.2.5 Staffing

Additional manhours will be required for operation and maintenance of the additional treatment units proposed for the recommended alternative. Based on the NEIWPCC staffing criteria in Chapter 9, the total wastewater treatment staff needed will increase to 7.5 people when the third clarifier and centrifuge go online as part of Priority 2 improvements. Since the wastewater utility appears to be currently understaffed (Chapter 9), it is recommended that staffing be increased by 1 person in the next 5 years, with an additional 0.5 staff added within the next 20 years.

12.3 SUMMARY & RECOMMENDATIONS

12.3.1 Construction Phasing Plan

The items outlined in this chapter are intended to provide a plan that allows the City to meet its wastewater treatment goals and objectives. Figure 12.1 provides general timeframes for construction of the various recommended improvements. As previously discussed, the first phase would include construction of the oxidation ditch shell for use as an equalization basin. The next phase would involve installation of all necessary mechanical equipment to convert the shell to a fully functional oxidation ditch treatment process.

12.3.2 Cost Estimates

Cost estimates for the recommended improvements illustrated on Figure 12.1, along with the associated timeframe for implementing the improvements, are included in the Capital Improvement Plan in Chapter 13. Detailed cost estimates are included in Appendix F.

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13.0 CAPITAL IMPROVEMENT PLAN

This chapter summarizes the costs for the recommended improvements. For a more detailed description of the recommended improvements, refer to Chapter 8 (wastewater collection system) and Chapter 12 (wastewater treatment).

13.1 BASIS FOR ESTIMATE OF PROBABLE COST

Costs presented in this report are based on 2011 regional cost data. They are also conceptual level costs which can be highly variable due to material shortages, bidding climate, and unforeseen economic climate such as inflation rate, etc. In a conceptual level report such as this, costs can normally vary by -30%/+50% or more depending on the economic climate and conditions.

13.1.1 Construction Costs

As indicated, costs estimated are reflective of construction costs in the Northwest and include a 15% allowance for mobilization/demobilization, overhead and profit.

13.1.2 Contingencies

Cost contingencies have been assumed at 30% of construction costs for this report.

13.1.3 Engineering Costs

Engineering Costs have been assumed at 18% of construction costs for this report.

13.1.4 Administrative Costs

Legal and administrative costs have been assumed at 2% of construction costs for this report.

13.2 SUMMARY OF COSTS

Table 13.1 presents a summary of future costs in order of priority. The need for each improvement varies for reasons including: compliance with the City's discharge permit, including new regulations; achieving capacity necessary to accommodate growth; and replacing worn/old equipment. Priority 1 improvements are expected to be required from 2012 to 2020, Priority 2 from 2020 to 2030, and Priority 3 are projected requirements beyond 2030. However, the City should recognize that flexibility in the completion of many of these improvements may be warranted. For example, the City should consider accelerating pipeline projects if they can be coordinated with roadway improvements. Similarly, changes in flows and efforts to reduce infiltration and inflow may allow for some improvements to be postponed.

TABLE 13.1: City of Ashland Wastewater Improvements Opinion of Probable Cost

ID#	Item	Primary Purpose	Total Estimated Cost	Growth Apportionment		City's Estimated Portion
				%	Cost	
Priority 1 Improvements (2012 - 2020)						
Wastewater Treatment						
1	Outfall Relocation / Fish Screen	Compliance	$ 856,000	15%	$ 128,400	$ 727,600
2	Shading (Capital Cost + first 6 years of O&M)	Compliance	$ 1,646,000	15%	$ 246,900	$ 1,399,100
3	UVT Monitor	Compliance	Completed	0%	$ -	$ -
4	Backup (Portable) Pump	Capacity	$ 60,000	0%	$ -	$ 60,000
5	Membrane Replacement (two trains)	Replacement	$ 1,248,000	0%	$ -	$ 1,248,000
6	Oxidation Ditch Shell	Capacity	$ 4,000,000	39%	$ 1,560,000	$ 2,440,000
7	RAS Pump Replacement	Capacity	$ 90,000	20%	$ 18,000	$ 72,000
8	Wastewater Master Plan Update	Update	$ 125,000	100%	$ 125,000	$ -
9	Wastewater Facility Plan	Financing	$ 35,000	50%	$ 17,500	$ 17,500
Wastewater Collection System						
1A	18" and 24" Parallel Trunkline Along Creek	Capacity	$ 1,248,000	70%	$ 873,600	$ 374,400
1B	15" Main Along Mountain Ave	Capacity	$ 118,000	25%	$ 29,500	$ 88,500
1C	Oak St. 24" Trunkline	Capacity	$ 40,000	15%	$ 6,000	$ 34,000
1D	A St 15" Main	Capacity	$ 522,000	10%	$ 52,200	$ 469,800
1E	12" Main Along Railroad	Capacity	$ 275,000	57%	$ 156,750	$ 118,250
1F	12" Siskiyou Blvd Main	Capacity	$ 73,000	46%	$ 33,580	$ 39,420
1G	Miscellaneous Upgrades	Various	$ 335,000	10%	$ 33,500	$ 301,500
1H	Portable Flow Meters	Operations	$ 60,000	0%	$ -	$ 60,000
1J	Storm Water Inflow Study (2012 - 2013)	Capacity	$ 60,000	0%	$ -	$ 60,000
	Total Priority 1 Improvements		**$ 10,791,000**		**$ 3,280,930**	**$ 7,510,070**
Priority 2 Improvements (by 2020 - 2030)						
Wastewater Treatment						
1	Membrane Replacement (Larger Membranes)	Capacity/ Replacement	$ 4,659,000	40%	$ 1,863,600	$ 2,795,400
2	Membrane Feed Pumps & Piping Replacement	Capacity	$ 507,000	80%	$ 405,600	$ 101,400
3	Additional UV Reactors & Upgrade Control Panels	Capacity	$ 351,000	100%	$ 351,000	$ -
4	Mechanical Bar Screen Replacement	Replacement	$ 496,000	20%	$ 99,200	$ 396,800
5	Grit Removal System Replacement	Replacement	$ 801,000	20%	$ 160,200	$ 640,800
6	Oxidation Ditch Internals	Capacity	$ 2,150,000	100%	$ 2,150,000	$ -
7	Existing Oxidation Ditch Equipment Replacement	Replacement	$ 1,551,000	0%	$ -	$ 1,551,000
8	Clarifier Mechanism Replacement	Replacement	$ 324,000	0%	$ -	$ 324,000
9	Replace Ashland Creek Lift Station Pumps with Larger Pumps	Capacity	$ 353,000	80%	$ 282,400	$ 70,600
8	Wastewater Master Plan Update	Update	$ 125,000	100%	$ 125,000	$ -
9	Biosolids Disposal (assumes thermal dryer)	Various	$ 4,100,000	20%	$ 820,000	$ 3,280,000
Wastewater Collection System						
2A	12" Pipeline on Nevada Street	Capacity	$ 217,000	38%	$ 82,460	$ 134,540
2B	8" Slope Correction on Walker Ave.	Operations	$ 168,000	28%	$ 47,040	$ 120,960
2C	12" Pipeline on Wightman St.	Capacity	$ 172,000	66%	$ 113,520	$ 58,480
2D	Miscellaneous Upgrades	Various	$ 739,000	10%	$ 73,900	$ 665,100
	Total Priority 2 Improvements		**$ 16,713,000**		**$ 6,573,920**	**$ 10,139,080**

TABLE 13.1: City of Ashland Wastewater Improvements
Opinion of Probable Cost *(Continued)*

ID#	Item	Primary Purpose	Total Estimated Cost	Growth Apportionment		City's Estimated Portion
				%	Cost	
Future Improvements (beyond 2030) or Development Related Improvements						
Wastewater Treatment						
1	Additional Centrifuge	Capacity	$ 817,000	100%	$ 817,000	$ -
2	Clarifier Mechanism Replacement (2)	Replacement	$ 646,000	0%	$ -	$ 646,000
3	Additional Clarifier	Capacity	$ 1,773,000	100%	$ 1,773,000	$ -
Wastewater Collection System						
3A	Rogue Valley Hwy 99 Collection, Lift Station, & Pressure Main (assumes City provides service)	Growth	$ 2,545,000	100%	$ 2,545,000	$ -
3B	Upsize Costs for Future Expansion	Growth	$ 18,000	100%	$ 18,000	$ -
	Total Priority 3 Improvements		**$ 5,799,000**		**$ 5,153,000**	**$ 646,000**
TOTAL WASTEWATER IMPROVEMENTS COSTS (rounded)			**$ 33,303,000**		**$ 15,007,850**	**$ 18,295,150**

13.3 OTHER ANNUAL COSTS

In addition to the capital improvement costs presented in the previous section, Keller Associates recommends the following be accounted for in setting annual budgets:

- Additional staffing needs (refer to Sections 8.2.3 and 9.5.1): additional $195,000/year for 2.5 additional full time equivalent employees (collection system lead foreman, treatment plant operator, and 0.5 FTE for regulatory compliance).
- Additional collection system replacement / rehabilitation needs (refer to Section 3.4.1 of this report): City should eventually budget an additional $637,000/year (either to be contracted out or completed using City crews). To minimize rate impacts, this program may not fully be funded until after 2022 when the existing wastewater loans are retired.
- Additional annual operations and maintenance costs will be required to maintain the shading improvements: anticipated to cost approximately $55,000/year for years 6-10, and closer to $39,000/year for years 11-20.
- Other additional annual operation and maintenance costs are associated with Priority 1 improvements (relocation of the outfall, larger RAS pumps, backup lift station pump, and equalization basin): the additional operations and maintenance costs for these improvements are anticipated at close to $26,000/year, most of which is associated with increased power usage of the RAS pumps.
- Short-lived assets (pumps, equipment, etc.): equates to an average of approximately $93,500/year, of which approximately $29,700/year is attributed to future facilities that will be added over the 20-year planning period.

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14.0 FINANCIAL PLAN

Appendix G contains a detailed financial plan. This Chapter is a summary of the plan. The financial plan includes three new loans to pay for the Priority 1 Capital Improvements discussed in Chapter 13. To pay for these improvements, the City will have to increase sewer rates and the sewer system development charge (SDC). Assumptions for increases in operating revenues and expenditures are addressed in this chapter. Recommendations to update the SDC are published in a separate report.

14.1 FINANCIAL FORECAST

Table 14.1 shows a summary of the financial forecast. It is organized into four sections: Operating Activities, Capital & Related Activities, Investment Activities, and Beginning & Ending Cash. Table 14.2 shows the schedule of capital improvements which have been inflated from the 2012 cost estimates to costs in the year of construction at the rate of 3.5% per year.

14.1.1 Cash Flows from Operating Activities

Operating activities include revenues from sewer rates and expenditures for recurring costs such as personnel, materials and services, and repair and maintenance. Revenues increase due to economic growth and rate increases. Expenditures increase due to additions in staff and the escalating cost of labor, materials and supplies.

Operating Revenues

As shown in Table 14.3, the sewer rates are scheduled to increase at 10% per year for all customers over a 5-year period, as shown in Table 14.3. Beginning in FY 2016-17, rates will be adjusted as necessary to meet the escalating costs of operations and capital improvements.

Each year the City will evaluate the utility's financial performance during the previous year and decide whether to follow or modify the planned rate increases. Changes in the construction schedule, financing costs, operating costs, or revenues from rates and SDCs may require the City to modify the planned rate increases. For example, economic growth that exceeds the forecast growth rate will result in increased revenue from both SDCs and rates, which will allow for a decrease in the sewer rate adjustment. Alternatively, if costs escalate faster than forecast, there may be a need to increase rates more rapidly.

Operating Expenditures

Operating Expenditures are forecast to increase due to inflation, new employees, and new materials and services costs.

Personal Services
Personal services (personnel costs) are forecast to increase at the rate of 6.7% per year due to inflation and the addition of new staff. Since FY 2006, personal services have increased an average 5.8% per year. The forecast also includes the addition of a 1/2 full time equivalent (FTE) regulatory compliance officer at $39,000 per year beginning in FY 2012-13, and another 1 FTE collection system supervisor at an annual cost of $78,000 beginning in FY 2013-14. The cost of these additional FTE's will also increase with inflation at 5.5% per year.

<u>Materials & Services</u>

Materials & services are forecast to increase an average of 6.2% per year. Since FY 2006 these costs have increased an average of 9.4% per year. The forecast includes an additional $55,000 per year beginning in FY 2015 to pay for the annual costs of maintaining the Bear Creek shading project, and two increases of $13,000 per year to cover the annual cost of operating and maintaining the new pumping facilities—one in FY 2016 and another in FY 2017 . In addition to these costs, inflation will increase all materials and services costs 6% per year.

14.1.2 Cash Flows from Capital & Related Activities

These activities include all revenues dedicated to capital improvements (SDCs, Food & Beverage Tax, and Loan Proceeds), and all expenses associated with Capital Improvements, and Debt Service on loans used to make capital improvements. These cash flows fluctuate significantly, as the City borrows money for constructing the improvements.

The forecast includes financing for three new capital projects:

- WWTP Replacement Membranes—$1.248 million; FY 2011-12
  - *Lender:* Oregon DEQ SRF (Department of Environmental Quality, State Revolving Fund loan program)
  - *Terms:* $1.248 million; 2.72% interest rate; 12-year term; annual debt service ~$123,300, to begin FY 2015(Note: A decision by the lender to reduce the interest rate from 2.72% to 1.0% is pending. If granted, the reduction would result in annual debt service savings of approximately $12,900 per year.)
- WWTP Outfall Replacement & Bear Creek Shading—$2.752 million; FY 2011-12
  - *Lender:* Oregon DEQ SRF
  - *Terms:* $2.4 million; 1.0% interest rate; 20-year term; annual debt service ~$123,300, to begin FY 2015
- Oxidation Ditch Shell—$5.75 million; FY 2015-16
  - *Lender:* Municipal Bond Market, Full Faith & Credit Pledge
  - *Terms:* $5.75 million (includes associated improvements and bond issuance costs); 5.0% interest rate; 25-year term; annual debt service ~$408,000, to begin FY 2017(Note: The City also will apply for an SRF loan (20 year term at 3.32%) which, if successful, will reduce the annual debt service to approximately $396,100—a savings of $11,900 per year.)

The remaining projects will be paid from cash reserves in the wastewater funds.

TABLE 14.1: Financial Forecast

FY ending June 30	Estimate 2011 2012	Forecast 2012 2013	2013 2014	2014 2015	2015 2016	2016 2017	2017 2018	2018 2019	2019 2020	Avg Ann % ∆
Cash Flows from Operating Activities										
Revenues										
Customer Receipts	$ 3,558,000	$3,918,000	$3,959,000	$ 4,370,000	$4,829,000	$ 5,338,000	$5,901,000	$6,500,000	$6,859,000	8.2%
Miscellaneous	14,250									
Total Revenues	$ 3,572,250	$3,918,000	$3,959,000	$ 4,370,000	$4,829,000	$ 5,338,000	$5,901,000	$6,500,000	$6,859,000	8.2%
Expenditures										
Personal Services	$ 922,420	$1,012,000	$1,146,000	$ 1,209,000	$1,275,000	$ 1,345,000	$1,419,000	$1,497,000	$1,579,000	6.7%
Materials & Services	2,324,366	2,464,000	2,612,000	2,824,000	3,006,000	3,199,000	3,391,000	3,594,000	3,810,000	6.2%
Tax Equivalents	297,670	313,400	316,700	349,600	386,300	427,000	472,100	520,000	548,700	7.6%
Total Expenditures	$ 3,544,456	$3,789,400	$4,074,700	$ 4,382,600	$4,667,300	$ 4,971,000	$5,282,100	$5,611,000	$5,937,700	6.4%
Net Cash From Operating Activities	$ 27,794	$ 128,600	($ 115,700)	($ 12,600)	$ 161,700	$ 367,000	$ 618,900	$ 889,000	$ 921,300	43.8%
Cash Flows from Capital & Related Activities										
System Development Charges	$ 56,392	$ 64,000	$ 160,000	$ 224,000	$ 288,000	$ 320,000	$ 324,000	$ 324,100	$ 324,200	
Food & Beverage Tax	1,703,000	1,703,000	1,729,000	1,755,000	1,781,000	1,808,000	1,835,000	1,863,000	1,891,000	
Capital Expenditures	(555,650)	(1,315,000)	(928,000)	(2,852,000)	(3,346,000)	(3,242,000)	(725,000)	(600,000)	(600,000)	
Long-term Debt										
Loan Proceeds	995,000	445,000	1,214,000	541,000	5,987,000	216,000				
Principal	(1,155,318)	(1,176,086)	(1,201,892)	(1,385,937)	(1,424,894)	(1,579,446)	(1,688,598)	(1,744,783)	(1,801,420)	
Interest	(535,836)	(516,718)	(503,985)	(467,588)	(714,681)	(673,106)	(621,627)	(557,441)	(491,004)	
Loan Costs (DEQ)	(4,975)	(7,200)	(13,270)	(15,209)	(15,613)	(15,896)	(14,813)	(13,711)	(12,589)	
Net Cash From Capital Activities	$ 502,613	($ 803,004)	$ 455,853	($ 2,200,734)	$2,554,812	($ 3,166,448)	($ 891,037)	($ 728,835)	($ 689,814)	
Cash Flows from Investing Activities										
Interest	$ 17,944	$ 29,700	$ 28,300	$ 19,200	$ 21,900	$ 21,700	$ 6,500	$ 6,000	$ 8,000	
Net Cash From Investing Activities	$ 17,944	$ 29,700	$ 28,300	$ 19,200	$ 21,900	$ 21,700	$6,500	$ 6,000	$ 8,000	
Net Change In Cash & Equivalents	$ 548,351	($ 644,704)	$ 368,453	($ 2,194,134)	$2,738,412	($2,777,748)	($ 265,637)	$ 166,165	$ 239,486	
Cash & Equivalents, Beginning	$ 2,743,939	$3,292,290	$2,647,586	$ 3,016,039	$ 821,905	$3,560,317	$ 782,569	$ 516,932	$ 683,096	
Cash & Equivalents, Ending	$ 3,292,290	$2,647,586	$3,016,039	$ 821,905	$3,560,317	$ 782,569	$ 516,932	$ 683,096	$ 922,582	

TABLE 14.2: Priority 1 Capital Improvements Schedule – Inflated at 3.5% per Year

Opinion of Probable Costs	Forecast Cost^	2011 2012	2012 2013	2013 2014	2014 2015	2015 2016	2016 2017	2017 2018
Wastewater Treatment Plant								
Membrane System	$ 1,248,000†		$ 624,000		$ 624,000			
Outfall Relocation	1,047,000		21,000	$ 22,000	23,000	$ 94,000	$ 436,000	$ 451,000
Shading	1,875,000		261,000	351,000	528,000	489,000	134,000	112,000
Backup Portable Lift Station Pump	64,000		64,000					
Oxidation Ditch Shell	4,735,000			246,000	511,000	1,955,000	2,023,000	
RAS Pumps	109,000						109,000	
Wastewater Master Plan Update	155,000						76,000	79,000
Facilities Plan	37,000		37,000					
Total Treatment	$ 9,270,000	$ 0	$ 1,007,000	$ 619,000	$ 1,686,000	$ 2,538,000	$ 2,778,000	$ 642,000
Collection System Improvements								
18" and 24" Parallel Trunkline Along Bear Creek	$ 1,434,000		$ 26,000	$ 110,000	$ 638,000	$ 660,000		
15" Main Along Mountain Ave	134,000		3,000	10,000	121,000			
Oak St. 24" Trunkline	46,000		1,000	4,000	41,000			
A St 15" Main	621,000		11,000	46,000	107,000		$ 457,000	
12" Main Along Railroad	316,000		6,000	24,000	140,000	146,000		
12" Siskiyou Blvd Main	92,000					2,000	7,000	$ 83,000
Miscellaneous Upgrades	492,000	$ 125,000	133,000	115,000	119,000			
Portable Flow Meters	64,000		64,000					
Storm Water Inflow Study	64,000		64,000					
Total Collection	$ 3,263,000	$ 125,000	$ 308,000	$ 309,000	$ 1,166,000	$ 808,000	$ 464,000	$ 83,000
Priority 1 Total – Adjusted	$ 12,533,000	$ 125,000	$ 1,315,000	$ 928,000	$ 2,852,000	$ 3,346,000	$ 3,242,000	$ 725,000

^ Costs are inflated at the rate of 3.5% per year based on *ENR's* long-run average.

† This project cost is fixed by negotiation with the supplier, and does not increase due to inflation.

14.1.3 Cash Flows from Investing Activities

The City invests idle cash in interest bearing securities. These revenues from interest fluctuate with cash balances and the rate of return on the investments.

14.1.4 Cash & Equivalents

Net Change in Cash & Equivalents is the annual sum of all current-year cash flows. These are added (or subtracted) from *Cash & Equivalents, Beginning*—which are carried over from the previous year—and result in *Cash & Equivalents, Ending* at the end of each fiscal year. Beginning and ending cash fluctuate with loan and construction activity.

14.2 SUMMARY

As Table 14.1 shows, Cash & Equivalents decrease to less than $1.0 million in FY 2017through FY 2019. This level of cash reserve is the minimum the City should expect to have for this utility. The sewer system s a complex mechanical system that is susceptible to unexpected breakdowns. A reasonable cash reserve would be $1.0 to $1.5 million. Oregon statutes do not require a specific level of cash reserves for sewer utilities, but experience by two government agencies provide some guidance:

- *Oregon Public Utility Commission.* The Oregon PUC regulates the user rates of privately-owned utilities, and allows cash reserves equal to 1/12th of operating expenditures plus cash reserves required by lenders.

- *U.S. Department of Agriculture, Rural Development.* USDA Rural Development, which finances many of the rural municipal utilities, allows cash reserves equal to 1/4th of operating expenditures plus cash reserves required by lenders.

The City's current major lender does not require a reserve, because the City made a Full Faith & Credit pledge to repay the loan. The two proposed loans from DEQ (membrane replacement and outfall/shading) will require a reserve equal to 12.5% of average annual debt service, plus a debt coverage ratio of 1.05.[1] The reserve on the two proposed loans amounts to about $123,290. The proposed loan for the oxidation ditch will also require the same reserve (and debt coverage ratio) if DEQ is the lender, and the reserve for this 3rd loan would therefore be $176,100. Assuming DEQ is the lender for all three of the proposed projects—and assuming operating expenditures in FY 2018 are $5,282,100, as forecast—the sewer utility's cash reserves should be between $959,700 (Oregon PUC standard) and $1,620,000 (USDA standard).

These measures simply illustrate that Ashland's wastewater utility is taking financial risks because cash reserves dip below $1.0 million in FY 2017 through FY 2019. Even in these years the City will meet its loan obligations, but will have very little available for emergency repairs. Also, if the City finances the oxidation ditch project through the municipal bond market rather than DEQ, the rate increases will likely have to occur sooner than forecast.

[1] DEQ has a sliding scale between the reserve and the debt coverage ratio. If the City reserves 100% of ½ of average annual debt service, the debt coverage ratio is 1.05. A reserve of 25% of ½ of average annual debt service requires a ratio of 1.35.

TABLE 14.3: Sewer Rates & Forecast Rate Increases

Customer Class	Current Rates^	2012 2013	2013 2014	2014 2015	2015 2016	2016 2017	2017 2018	2018 2019	2019 2020
% Rate Increase (per year)		10.0%	10.0%	10.0%	10.0%	10.0%	10.0%	5.0%	5.0%
Residential†									
Base	$ 18.70	$20.60	$22.70	$25.00	$27.50	$30.30	$33.30	$35.00	$36.80
Commodity	$ 2.80	$ 3.08	$ 3.39	$ 3.73	$ 4.10	$ 4.51	$ 4.96	$ 5.21	$ 5.47
Commercial									
Base	$ 19.54	$21.50	$23.70	$26.10	$28.70	$31.60	$34.80	$36.50	$38.30
Commodity	$ 3.11	$ 3.42	$ 3.76	$ 4.14	$ 4.55	$ 5.01	$ 5.51	$ 5.79	$ 6.08

^ Base Rates are rounded to the nearest $0.10/month. Commodity Rates are per 100 cubic feet, rounded to the nearest $0.01.

† Most residential customers pay only the base rate, which includes 400 cubic feet of water consumption.

14.3 OTHER FINANCING OPTIONS

The City has nearly completed financing for two of three most immediate construction projects – replacing the membranes at the WWTP, and moving the outfall and shading the receiving Bear Creek. Because of its cost, the City will have to finance the oxidation ditch. Because of the unpredictability of the state and federal financing programs, our forecast assumes this third project will be financed through the municipal bond market, which is likely more expensive than state and federal financing options. The following funding programs are available at this time. The sources of funding may change over time, however, and should be re-evaluated in the future as the need for financing approaches.

14.3.1 Oregon Department of Environmental Quality, State Revolving Fund (DEQ SRF)

DEQ SRF has provided financing for the first two projects on the capital improvements list. The program relies on annual Congressional appropriations and on repayment of loans made to municipalities in prior years. In a typical year, the SRF funds about 10% of the eligible projects. (In the past 5 years, DEQ lent between $29.5 million and $59.3 million a year for 8 to 19 projects per year.) Modifications to the SRF program are in progress, and may result in changes that make the program attractive for Ashland. If so, the modifications will also result in increased competition for the funds.

14.3.2 Oregon Infrastructure Finance Authority (IFA)

IFA administers two state-funded programs and the rural portion of one federal program. Two IFA programs provide both loans and grants for eligible capital improvements—the *Water/Wastewater Financing Program* and the *Special Public Works Fund.* Both of these programs are funded by State lottery revenues and loans the IFA obtains from the municipal bond market, which are then re-lent to municipalities at subsidized rates and terms. The interest rate on these loans is lower than Ashland would likely get for a direct loan through the municipal bond market because (1) the State has a better credit rating, and (2) the State pays all the closing costs of issuing the bonds, which can range from 1% to 2% of the amount borrowed. The loans are controlled by the Oregon Legislature and contain several provisions and limitations that need to be addressed if Ashland proceeds with applications to either program. Eligible projects can receive loans up to $10 million for a term not to exceed the lesser of 25 years or the expected life of the assets being financed.

The grants portions of the IFA programs differ:

- *Water/Wastewater Financing Program.* This program provides up to $750,000 in grants for qualifying applicants based on two factors: financial need (which is determined by a needs analysis conducted by IFA), and median household income which must be less than the statewide average. In 2007 the latest year data are available, Jackson County's MHI was below the State's MHI.

- *Special Public Works Fund.* This program offers up to $500,000 in grants (but not more than 85% of the total project cost) for qualifying projects, which must either create or retain "traded-sector" jobs. (These are jobs in businesses selling goods or services in markets for which national or international competition exists, as determined by IFA.) The ratio of project cost (loan and grant) to jobs created or retained cannot exceed $5,000 per job.

- *Community Development Block Grant (CDBG).* The U.S. Department of Housing and Urban Development (HUD) administers this program. HUD distributes the funding to metropolitan governments or to designated state agencies for rural communities—in this case the Oregon IFA. Because Ashland is designated a metropolitan city, it receives funding directly from HUD. The City will have to balance its own set of priorities for CDBG funds—*i.e.,* sewer projects will have to compete with other City priorities for the available funds. The funds are for projects that must meet 3 criteria: (1) the project must benefit low- and moderate-income households; (2) aid in the prevention or elimination of slums or blight; and (3) meet an urgent need that poses a serious and immediate threat to the health or welfare of the community.

14.3.3 U.S. Economic Development Administration (EDA)

The US EDA provides loans and grants for qualified public works projects, including sewer. Congress has been appropriating between $50 million and $70 million annually for EDA's western region (the states of Alaska, California, Hawaii, Idaho, Montana, Oregon, and Washington) for projects that retain or create jobs. The program funds up to 50% of the project cost, and the balance must come from local matching funds. Projects must show a very strong connection to jobs, and competition from the larger states in the 7-state region is very strong. Ashland's planned sewer projects are not likely to meet EDA's criteria. In the past, the maximum funding awarded to a project was $2.5 million. In practice, Ashland would have to have an existing private employer of significant employment or a potential new employer that need a significant sewer improvement to compete for this funding.

14.3.4 Bonneville Power Association, Energy Smart Industrial Program (BPA ESI)

BPA sponsors this ESI program—recently extended through September 30, 2013—to promote energy-efficient industrial improvements. The sewer utility qualifies for the program as an industrial customer of Ashland's Municipal Electric Utility, which is served by the BPA.

ESI performs an energy audit of the facility and drafts a report that lists opportunities to reduce electricity consumption. Recommendations include capital improvements and changes in operations and maintenance that would result in reduced electricity consumption. Eligible projects also include previously-planned improvements identified by ESI as energy-efficient.

Once the energy-savings measures have been implemented, the City receives a credit of $0.25/kWh saved in the first year of operation—or up to 70% of the total project cost. (For example, last year the City of La Center, Washington, received $215,000 in cash rebates from ESI for upgrading its wastewater treatment plant from an SBR technology to a MBR technology.) ESI does not charge for the audit and report, and the City is under no obligation to accept or implement ESI's recommendations.

14.4 RECOMMENDATIONS

We recommend that the City continue to apply to the SRF program for the oxidation ditch and smaller projects. If it does not score highly for the SRF program, the City's next best source of low-cost loans and possibly grants is from IFA's *Water/Wastewater Financing Program*. Internally, the City can decide if one or more of the planned sewer projects are appropriate for CDBG funding. Also, since the MBR technology Ashland uses is energy intensive relative to other treatment technologies, we recommend that the City request an energy audit from ESI. ESI will review this Master Plan and if appropriate conduct an on-site analysis of the City's wastewater system. Again, the City will get credit for energy efficient engineering already completed, and the City is under no obligation to accept or implement ESI's recommendations.

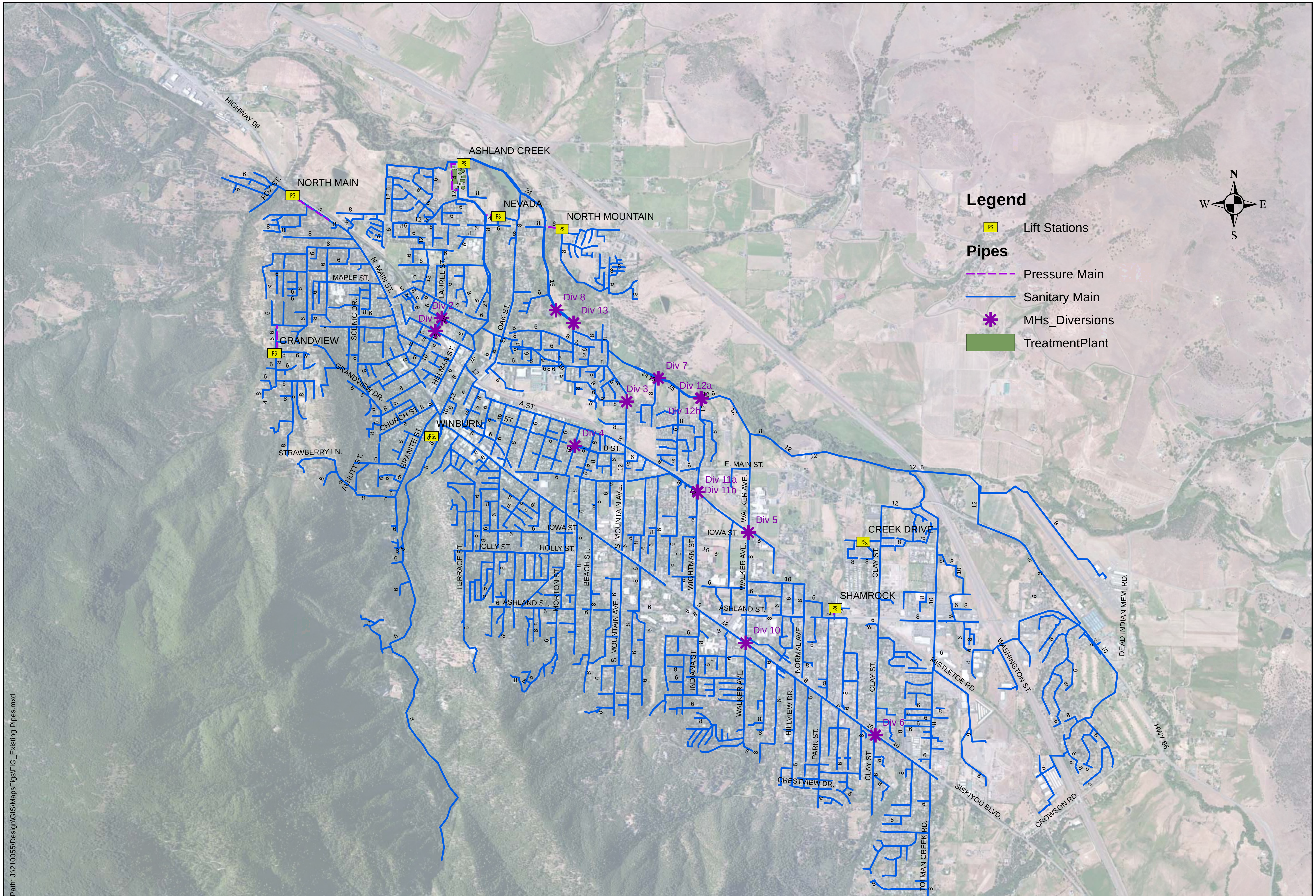
This financial plan can only be accomplished with sewer rate increases. We recommend increasing the sewer rates incrementally at 10% per year over a 5-year period to avoid rate shock to customers, and to provide time for the economy to recover from recession. Each year the City can re-evaluate the sewer utility's financial position relative to this forecast and as necessary adjust the next proposed rate increase.

APPENDIX A

FIGURES

- FIGURE 3.1: EXISTING COLLECTION SYSTEM
- FIGURE 3.2: SEWER BASINS
- FIGURE 5.1: EXISTING SYSTEM CAPACITY
- FIGURE 6.1: STUDY AREA
- FIGURE 8.1: MASTER PLAN
- FIGURE 12.1: FUTURE WWTP EXPANSION

KELLER associates



KELLER associates

Prepared for:

CITY of ASHLAND, OREGON

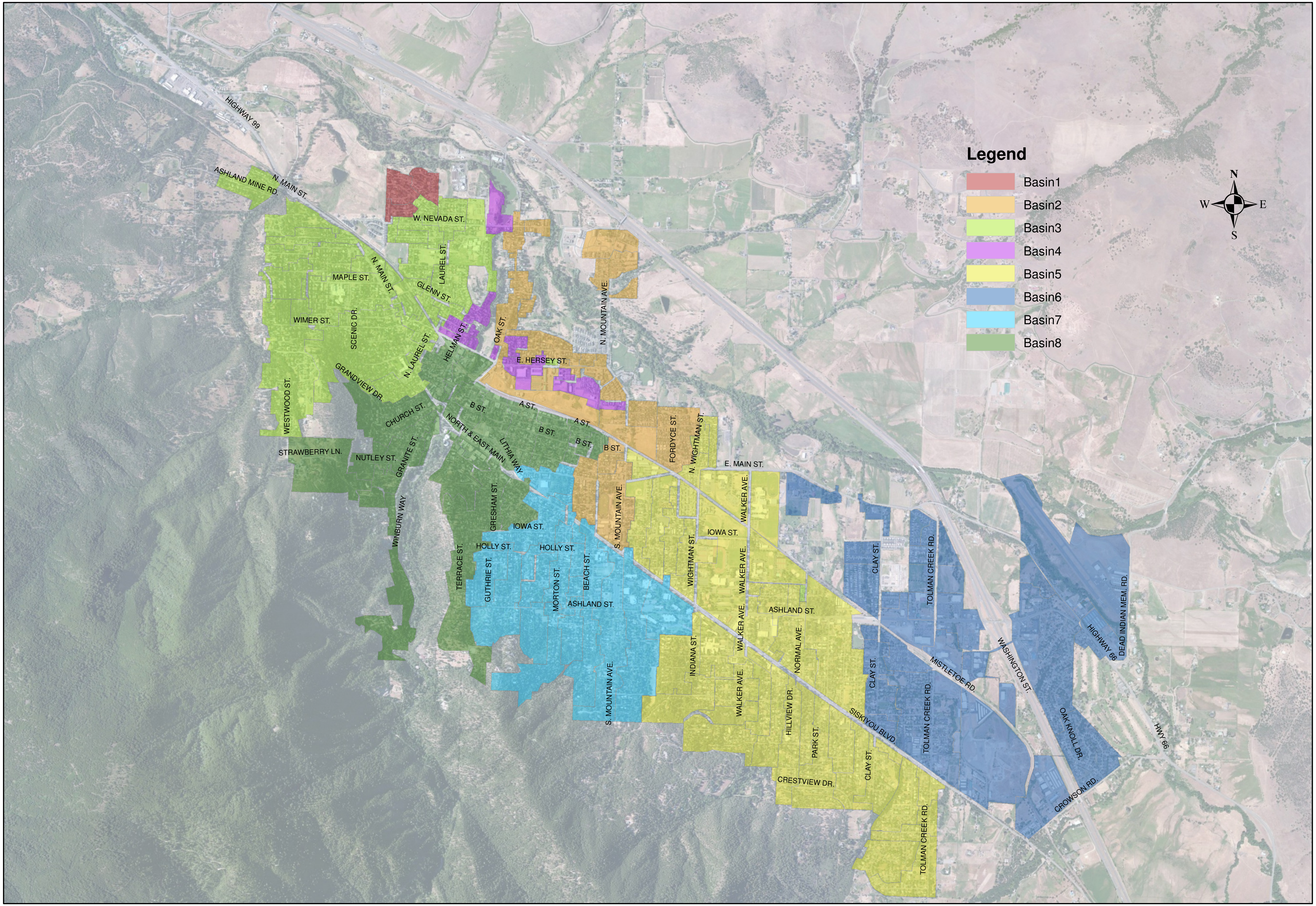
WASTEWATER SYSTEM MASTER PLAN

Title:

EXISTING COLLECTION SYSTEM

Figure:

3.1

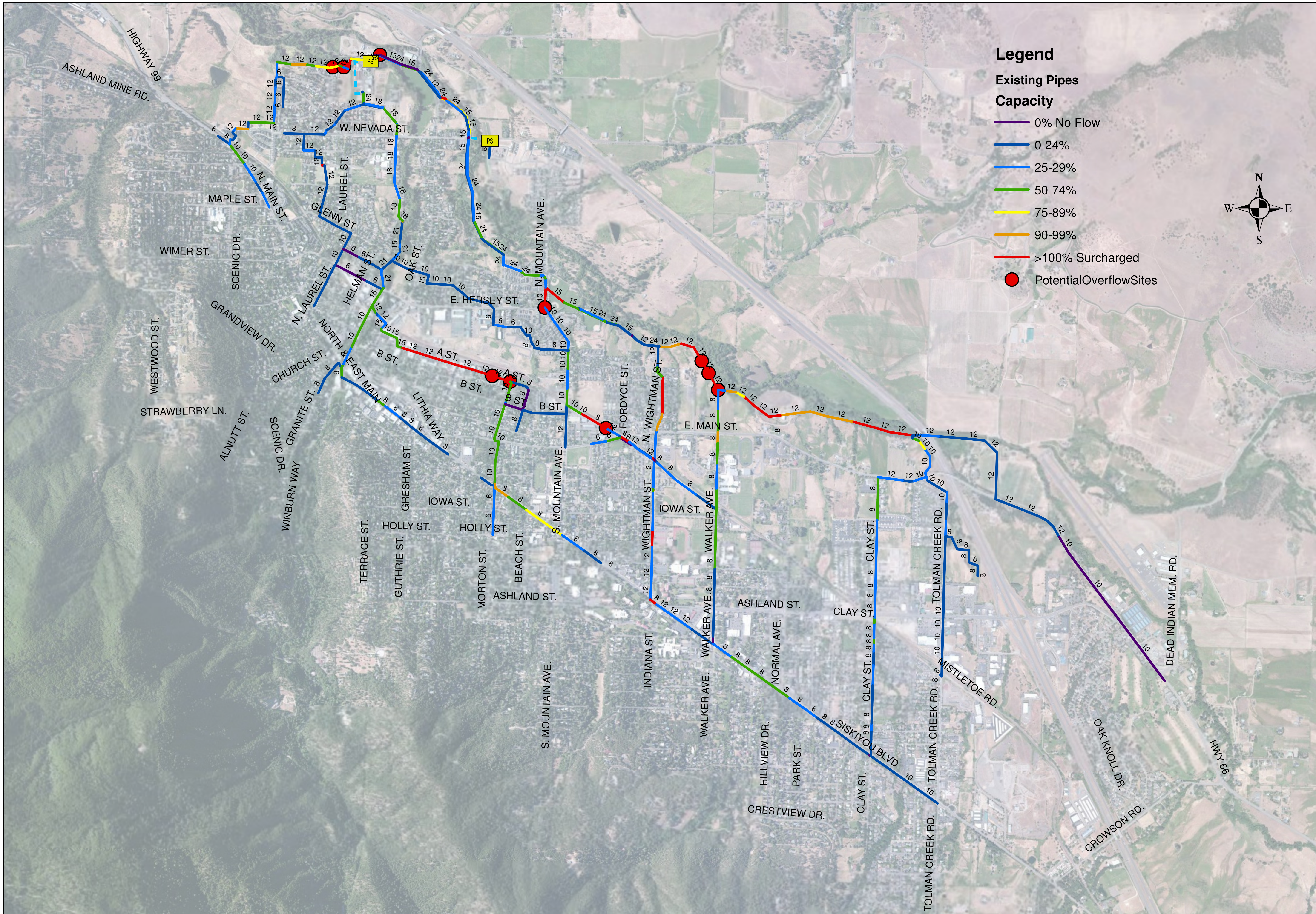


Prepared for:
CITY of ASHLAND, OREGON

WASTEWATER SYSTEM MASTER PLAN

Title:
SEWER BASINS

Figure:
3.2



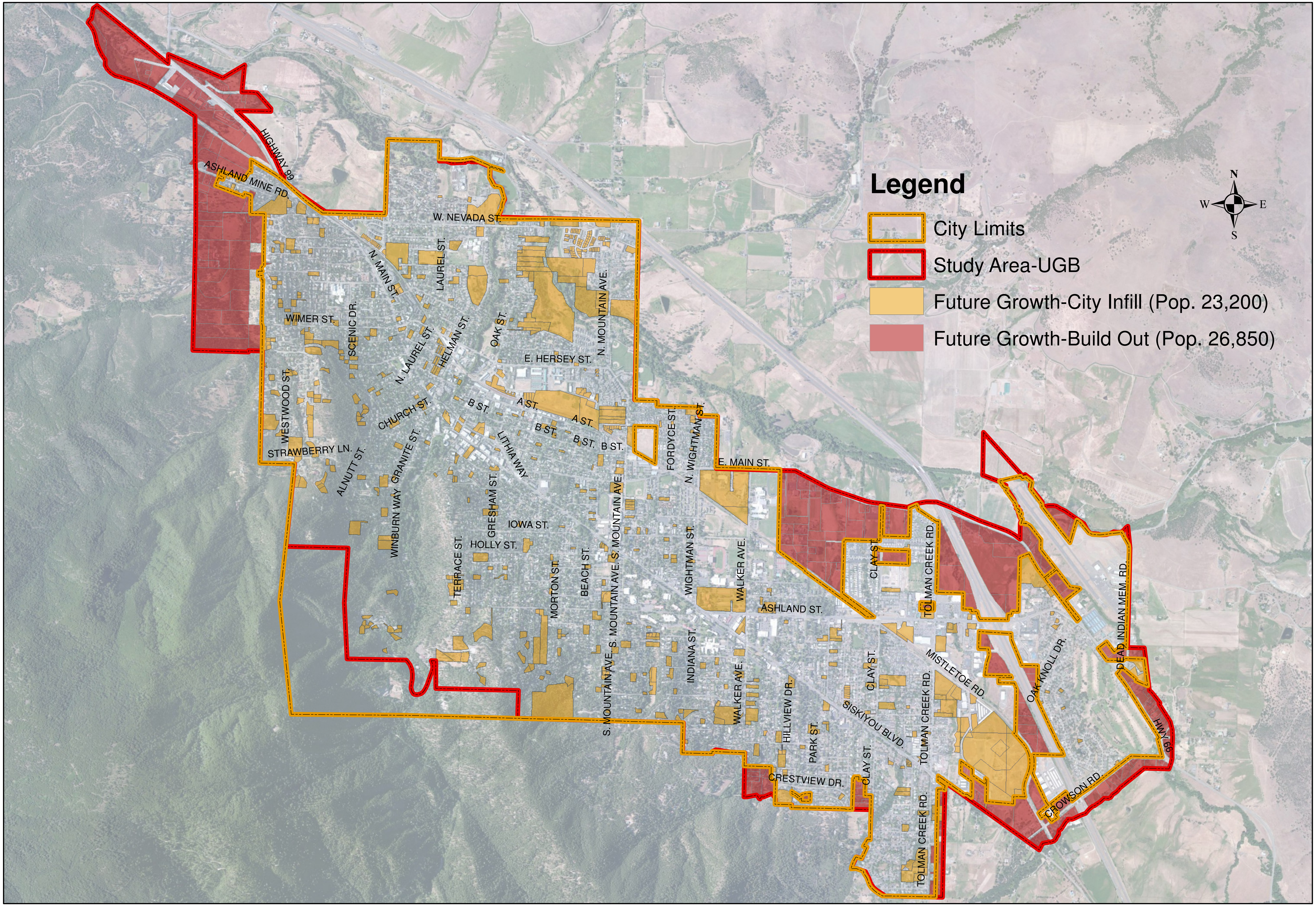
KELLER associates

Prepared for:
CITY of ASHLAND, OREGON

WASTEWATER SYSTEM MASTER PLAN

Title:
EXISTING SYSTEM CAPACITY

Figure:
5.1



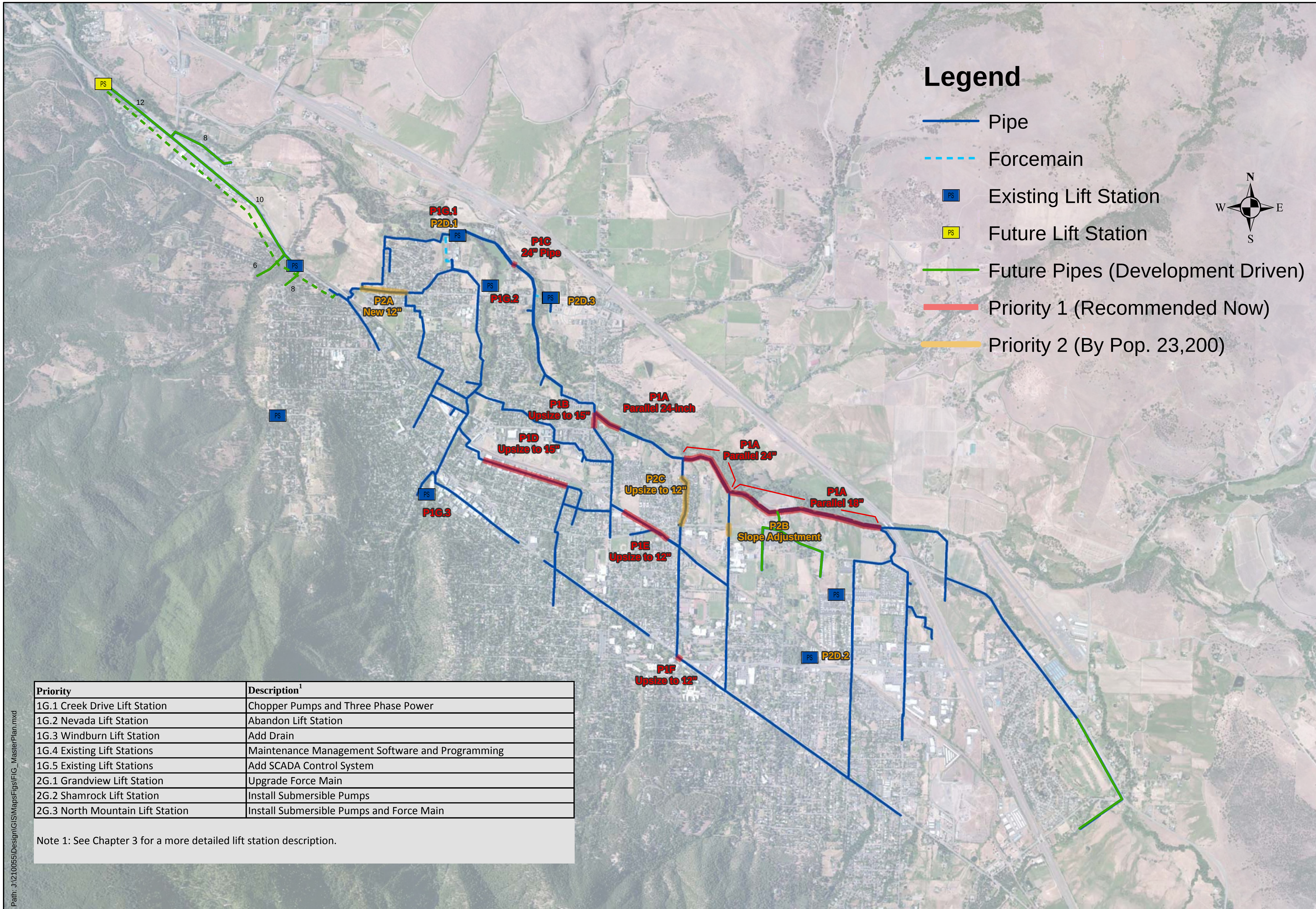
Prepared for:

CITY of ASHLAND, OREGON

WASTEWATER SYSTEM MASTER PLAN

Title: STUDY AREA

Figure: 6.1



Priority	Description[1]
1G.1 Creek Drive Lift Station	Chopper Pumps and Three Phase Power
1G.2 Nevada Lift Station	Abandon Lift Station
1G.3 Windburn Lift Station	Add Drain
1G.4 Existing Lift Stations	Maintenance Management Software and Programming
1G.5 Existing Lift Stations	Add SCADA Control System
2G.1 Grandview Lift Station	Upgrade Force Main
2G.2 Shamrock Lift Station	Install Submersible Pumps
2G.3 North Mountain Lift Station	Install Submersible Pumps and Force Main

Note 1: See Chapter 3 for a more detailed lift station description.

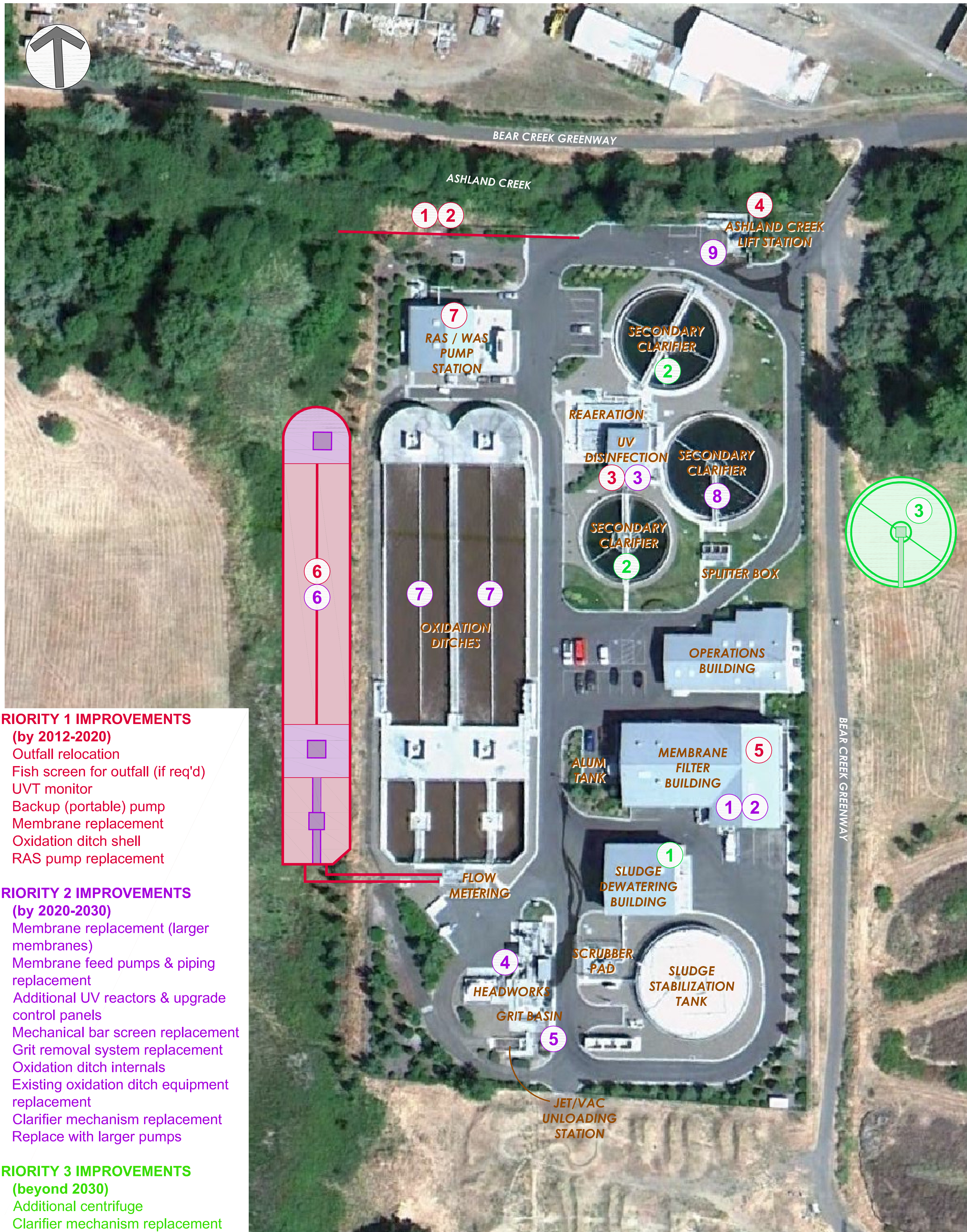
KELLER associates

Prepared for: CITY of ASHLAND, OREGON

Title: WASTEWATER SYSTEM MASTER PLAN

MASTER PLAN

Figure: 8.1



PRIORITY 1 IMPROVEMENTS (by 2012-2020)

1 Outfall relocation
2 Fish screen for outfall (if req'd)
3 UVT monitor
4 Backup (portable) pump
5 Membrane replacement
6 Oxidation ditch shell
7 RAS pump replacement

PRIORITY 2 IMPROVEMENTS (by 2020-2030)

1 Membrane replacement (larger membranes)
2 Membrane feed pumps & piping replacement
3 Additional UV reactors & upgrade control panels
4 Mechanical bar screen replacement
5 Grit removal system replacement
6 Oxidation ditch internals
7 Existing oxidation ditch equipment replacement
8 Clarifier mechanism replacement
9 Replace with larger pumps

PRIORITY 3 IMPROVEMENTS (beyond 2030)

1 Additional centrifuge
2 Clarifier mechanism replacement
3 Additional clarifier

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Figure:	Title:		Prepared for:
12.1	FUTURE WWTP EXPANSION	WASTEWATER SYSTEM MASTER PLAN	CITY of ASHLAND, OREGON

KELLER associates

APPENDIX B

EXISTING SYSTEM DATA

- NPDES PERMIT
- LIFT STATION DATA
- PIPELINE CONDITION EVALUATION

KELLER associates

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NPDES Permit

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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:
Ashland, City of
20 East Main Street
Ashland, Oregon 97520

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	001	R.M. 0.25
Reclaimed Water Reuse	002	

FACILITY TYPE AND LOCATION:
Oxidation Ditch
Ashland STP
1/4 Mile NW of Nevada St. & Oak St.
Ashland

Treatment System Class: Level IV
Collection System Class: Level III
EPA REFERENCE NO: OR-002073-7

RECEIVING STREAM INFORMATION:
Basin: Southern Oregon Coastal
Sub-Basin: Middle Rogue

Receiving Stream: Ashland Creek
LLID: 1227202422154 - 0.25 - D
County: Jackson

Issued in response to Application Nos. 988564 received 10/02/2000 and 985027 received 12/06/2002.
This permit is issued based on the land use findings in the permit record.

Michael H. Kortenhof, Water Quality Manager
Western Region

May 27, 2004
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 to waters of the state is prohibited, including discharge to 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 - August 31 and November 1 – November 30:

Parameter	Average Effluent Concentrations (mg/L) Monthly	Weekly	Daily	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
$CBOD_5$	10	15		120	210	380
TSS	10	15		96	180	480
Ammonia (see note 2)	0.52		1.2			
Phosphorus				1.6		5.1

(2) September 1 – October 31:

Parameter	Average Effluent Concentrations (mg/L) Monthly	Weekly	Daily	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
$CBOD_5$	4	5		77	120	250
TSS	10	15		96	180	480
Ammonia (see note 2)	0.52		1.2			
Phosphorus				1.6		5.1

(3) December 1 - April 30:

Parameter	Average Effluent Concentrations (mg/L) Monthly	Weekly	Daily	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
$CBOD_5$	25	40		400	920	1500
TSS	30	45		400	920	1500
Ammonia (See note 2)	0.80		1.8			

* Average dry weather design flow to the facility equals 2.3 MGD. Mass load limits have been individually assigned.

(4)

Other parameters (year-round except as noted)	Limitations
E. coli Bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL. (See Note 1)
pH	Shall be within the range of 6.5 - 8.5
$CBOD_5$ and TSS Removal Efficiency	Shall not be less than 85% monthly average for $CBOD_5$ and 85% monthly for TSS.
Dissolved Oxygen (Oct 15 though May 15)	Shall not be less than 9.0 mg/L
Excess Thermal Load (Oct 15 though May 15)	Shall not exceed 78 million kcals/day (See Note 3)
Excess Thermal Load (May 16 though Oct 14)	Shall not exceed 38 million kcals/day (See Note 3)

(5) Except as provided for in OAR 340-045-0080, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-041-0365, except in the following defined temperature mixing zone:

The allowable temperature mixing zone is that portion of Ashland Creek which allows for mixing of the treated effluent with 25 percent of the stream flow.

(6) Raw sewage discharges are prohibited to waters of the State from November 1 through May 21, except during a storm event greater than the one-in-five-year, 24-hour duration storm, and from May 22 through October 31, except during a storm event greater than the one-in-ten-year, 24-hour duration storm.

(7) 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.

(8) Chlorine and chlorine compounds shall not be used as a disinfecting agent of the treated effluent and no chlorine residual shall be allowed in the discharged effluent due to chlorine used for maintenance purposes.

b. Reclaimed Wastewater Outfall 002

(1) No discharge to state waters is permitted. All reclaimed water shall be distributed on land, for dissipation by evapotranspiration and controlled seepage by following sound irrigation practices so as to prevent:

a. Prolonged ponding of treated reclaimed water on the ground surface;

b. Surface runoff or subsurface drainage through drainage tile;

c. The creation of odors, fly and mosquito breeding or other nuisance conditions;

d. The overloading of land with nutrients, organics, or other pollutant parameters; and,

e. Impairment of existing or potential beneficial uses of groundwater.

(2) Prior to land application of the reclaimed water, it shall receive at least level IV treatment as defined in OAR 340-055 to:

(a) Reduce Total Coliform to a seven-day median of 2.2 organisms per 100 mL and a maximum of 23 organisms per 100 mL.

(b) Reduce turbidity to a 24-hour mean of 2 Nephelometric Turbidity Units (NTUs) with no more than five percent of the samples during a 24-hour period exceeding 5 NTUs.

(3) Irrigation shall conform to the reclaimed water use plan once approved by the Department. The plan shall contain a description of the design of the proposed reclamation system and shall clearly indicate the means for compliance with OAR 340-041-0055.

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)

NOTES:

1. 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.

2. The ammonia limits were calculated using the EPA Gold Book Criteria and are considered interim limits. DEQ is in the process of adopting the EPA 1999 ammonia criteria. Upon approval by the EPA, the following limits will automatically be applied to the discharge without a permit modification:

Parameter	Average Effluent Concentrations (mg/L)		
	Monthly	Weekly	Daily
Ammonia (May 1 - November 30)	1.2		2.4
Ammonia (December 1 – April 30)	2.0		3.3

3. The Excess Thermal Load limits are interim limits that were calculated using the average dry weather design flow and an estimated maximum weekly effluent temperature. The Department also calculated water quality based Excess Thermal Limits using projected estimations of the worst case conditions. These water quality based Excess Thermal Limits will become effective 55 months after permit issuance, unless modified as described below:

Other parameters	Limitations
Excess Thermal Load (Oct 15 though May 15)	Shall not exceed 2.8 million kcals/day
Excess Thermal Load (May 16 though Oct 14)	Shall not exceed 21 million kcals/day

The Department recognizes that the estimation of critical stream flow conditions are based on minimal information and that additional stream flow information is needed to provide a more accurate estimate. Schedule B, condition 1.d. requires the Permittee to collect this additional stream flow information. Schedule C, condition 1 also allows time to implement thermal reduction activities and requires the Permittee to provide better estimates of the critical low flow conditions. Upon receipt of this additional information, the Department intends to recalculate the Excess Thermal Loads, re-open this permit, and modify the allowable thermal load.

The Permittee has chosen riparian improvements as a portion of their thermal reduction program. This permit may be re-opened, and the maximum allowable thermal load modified, when more accurate effluent temperature data becomes available or when a water quality credit trading plan is authorized by the Department.

In addition, upon approval of a Total Maximum Daily Load for temperature for this sub-basin, this permit may be re-opened and new temperature and/or thermal load limits assigned.

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

Influent samples and measurements are taken just before the grit basin. All samples for toxics are taken in the same location.

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 sampling locations are the following:

- When using the membrane filtration system, effluent samples and measurements are taken from membrane building effluent well.
- When the membrane filtration system is not in use, effluent samples and measurements are taken from the re-aeration chamber just downstream of the UV disinfection system.

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Semi-Annual	Verification
$CBOD_5$	2/Week	Composite
TSS	2/Week	Composite
pH	3/Week	Grab
E. coli	2/Week	Grab (See Note 1)
UV Radiation Intensity	Daily	Reading (See Note 3)
Pounds Discharged ($CBOD_5$ and TSS)	2/Week	Calculation
Average Percent Removed ($CBOD_5$ and TSS)	Monthly	Calculation
Ammonia (NH3-N)	2/Week	Composite
Nutrients		
TKN, NO2+NO3-N, Total Phosphorus, ortho phosphorus	2/Week (May 1- Nov 30) Monthly (Dec 1 – Apr 30)	24-hour Composite
Toxics:		
Whole Effluent Toxicity (WET) test (See Note 2)	Semi-annually	See Schedule D condition 3
Priority Pollutant Scan	3 per year	See Schedule D condition 4

b. Treated Effluent Outfall 001 (continued)

Item or Parameter	Minimum Frequency	Type of Sample
Other Parameters:		
Dissolved Oxygen	2/Week (Oct 15 – May 15)	Grab
Temperature, Daily Max	Daily	Monitor (See Note 4)
Effluent Temperature, Average of Daily Maximums (See Note 4)	Weekly	Calculation
Excess Thermal Load (See Note 4)	Weekly	Calculation (See Note 5)

c. Reclaimed Wastewater Outfall 002

The reclaimed water sampling locations shall be as specified in the Department approved reclaimed water use plan.

Item or Parameter	Minimum Frequency	Type of Sample
Quantity Irrigated (inches/acre)	Daily	Measurement
Flow Meter Calibration	Annually	Verification
Quantity Chlorine Used	Daily	Measurement
Chlorine Residual	Daily	Grab
pH	2/Week	Grab
Total Coliform	Daily	Grab
Turbidity	Hourly	Measurement
Nutrients (TKN, NO_2+NO_3-N, NH_3, Total Phosphorus)	Quarterly	Grab

d. Ashland Creek Monitoring

Item or Parameter	Minimum Frequency	Type of Sample
Flow (upstream)	Daily	Measurement
Dissolved Oxygen (surface water)	2/month	Grab
Intergravel Dissolved Oxygen	1/year (See note 6)	Study

e. Sludge Management

Item or Parameter	Minimum Frequency	Type of Sample
Quantity of sludge disposed	Daily	Pounds of sludge disposed

2. **Reporting Procedures**

a. Monitoring results shall be reported on approved forms. The reporting period is the calendar month. Reports must be submitted to the Department's Western Region - Medford office by the 15th day of the following month.

b. State monitoring reports shall identify the name, certificate classification and grade level of each principal operator designated by the permittee as responsible for supervising the wastewater collection and treatment systems during the reporting period. Monitoring reports shall also identify each system classification as found on page one of this permit.

c. Monitoring reports shall also include a record of the quantity and method of use of all sludge removed from the treatment facility and a record of all applicable equipment breakdowns and bypassing.

3. **Report Submittals**

a. The permittee shall have in place a program to identify and reduce inflow and infiltration into the sewage collection system. An annual report shall be submitted to the Department by February 1 each year which details sewer collection maintenance activities that reduce inflow and infiltration. The report shall state those activities that have been done in the previous year and those activities planned for the following year.

b. For any year in which biosolids are land applied or used as land fill cover, 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).

c. By no later than January 15 of each year, the permittee shall submit to the Department an annual report describing the effectiveness of the reclaimed water system to comply with approved reclaimed water use plan, the rules of Division 55, and the limitations and conditions of this permit applicable to reuse of reclaimed water.

NOTES:

1. *E. coli* monitoring must be conducted according to any of the following test procedures as specified in **Standard Methods for the Examination of Water and Wastewater, 19th Edition**, or according to any test procedure that has been authorized and approved in writing by the Director or an authorized representative:

Method	Reference	Page	Method Number
mTEC agar, MF	Standard Methods, 18th Edition	9-29	9213 D
NA-MUG, MF	Standard Methods, 19th Edition	9-63	9222 G
Chromogenic Substrate, MPN	Standard Methods, 19th Edition	9-65	9223 B
Colilert QT	Idexx Laboratories, Inc.		

2. Beginning no later than January 1, 2005, the permittee shall conduct Whole Effluent Toxicity (WET) testing for a period of one (1) year in accordance with the frequency specified above. If the WET 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 WET testing will be required during this permit cycle. Note that WET test results will be required along with the next NPDES permit renewal application.

3. The intensity of UV radiation passing through the water column will affect the systems ability to kill organisms. To track the reduction in intensity, the UV disinfection system must include a UV intensity meter with a sensor located in the water column at a specified distance from the UV bulbs. This meter will measure the intensity of UV radiation in mWatts-seconds/cm2. The daily UV radiation intensity shall be determined by reading the meter each day. If more than one meter is used, the daily recording will be an average of all meter readings each day.

4. Temperature shall be continuously monitored with a maximum of 20 minute increments. The maximum value recorded during a 24 hour period shall be reported on the monthly reports. In the event the continuous temperature recorder malfunctions, Permittee shall record grab measurements at one-hour intervals. Instrumentation malfunctions shall be noted on the monthly reports.

5. Calculated as follows:
(Weekly average of daily maximum effluent temperatures in °C - applicable stream temperature standard) X (Weekly average of daily flow in MGD) X 3.785 = Excess Thermal Load, in Million Kcals/day.

6. The City is not required to perform the IGDO studies in condition 1.d. until the Department has provided the City with a written procedure that has been reviewed and accepted by the National Marine Fisheries Service and the Oregon Department of Fish and Wildlife.

SCHEDULE C

Compliance Schedules and Conditions

1. By May 27, 2006, the permittee shall complete the thermal reduction measures recommended in the Wastewater Treatment Plant Temperature Management Plan (April 2002). These measures are as follows:

   MP-1: The City will develop a market evaluation and water recycling plan. The planning process will include a public education component about the water quality of Ashland's effluent, a market survey, opportunities to increase stream flow by offsetting existing irrigation demand, and the development of infrastructure needs and costs to meet existing and future market demand for recycled water.

   MP-4: The City will develop and implement a riparian corridor improvement plan for Ashland Creek. The plan would include temperature modeling to predict the benefits of modifying the riparian corridor, identification of stream reaches that need improvement, and the development of effective and needed modifications. In addition to improving temperature, the planning would focus on improving both in-stream and riparian habitat, reducing flooding, and improving aesthetics.

   Bay May 27, 2007, the permittee shall submit a report detailing the effectiveness of measures MP-1 and MP-4. The report shall include information collected on Ashland Creek, including daily stream flow and temperatures. The report shall also provide estimates of critical low flows. If the water quality based excess thermal limits in Schedule A, Note 3 are not achieved, the report shall include an evaluation of the cost effectiveness of additional temperature reduction measures and a selected preferred alternative. Upon Department review and approval, the permittee shall implement the preferred alternative.

2. The permittee is expected to meet the compliance dates which have been established in this schedule. Either prior to or no later than fourteen 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. Prior to increasing thermal load (flow or temperature) beyond the current permit limitations, the Permittee shall notify the Department and apply for and be issued a permit modification allowing the increase.

2. The facility's sludge is currently disposed of in a Department approved landfill as a solid waste (either in a landfill cell or is used as interim cover). Disposal must be in accordance with OAR Chapter 340, Division 93. Proper waste monitoring would be prescribed by the landfill in accordance with that rule. Monitoring and reporting as biosolids is not required under this permit.

3. **Whole Effluent Toxicity (WET) Testing**

   a. The permittee shall conduct whole effluent toxicity 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).

   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 biotoxicity. 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 six 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 test, are using for all toxicity tests required by the Department.

h. Reopener

(1) If WET testing indicates acute and/or chronic toxicity, the Department may reopen and modify this permit to include new limitations and/or conditions as determined by the Department to be appropriate, and in accordance with procedures outlined in Oregon Administrative Rules, Chapter 340, Division 45.

4. The permittee shall perform chemical analysis of its effluent for the specific toxic pollutants listed in Appendix J, Table 2 of 40 CFR Part 122. The effluent samples shall be 24-hour daily composites, except where sampling volatile compounds. For volatile compounds, six (6) discrete samples (not less than 100 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.

5. The permittee shall meet the requirements for use of reclaimed water under Division 55, including the following:

a. All reclaimed water shall be managed in accordance with the approved Reclaimed Water Use Plan. No substantial changes shall be made in the approved plan without written approval of the Department.

b. No reclaimed water shall be released by the permittee to another person, as defined in Oregon Revised Statute (ORS) 468.005, for use unless there is a valid contract between the permittee and that person that meets the requirements of OAR 340-055-0015(9).

c. The permittee shall notify the Department within 24 hours if it is determined that the treated effluent is being used in a manner not in compliance with OAR 340-055. When the Department offices are not open, the permittee shall report the incident of noncompliance to the Oregon Emergency Response System (Telephone Number 1-800-452-0311).

d. No reclaimed water shall be made available to a person proposing to recycle unless that person certifies in writing that they have read and understand the provisions in these rules. This written certification shall be kept on file by the sewage treatment system owner and be made available to the Department for inspection.

6. 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 <u>and</u> 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 7.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 7.b. above.

7. The permittee shall notify the DEQ Western Region - Medford Office (phone: (541) 776-6010) 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.

8. 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.

9. Permittee shall not accept septage without written approval from the Department.

NPDES GENERAL CONDITIONS
(SCHEDULE F)

SECTION A. STANDARD CONDITIONS

1. Duty to Comply

The permittee must comply with all conditions of this permit. Any permit noncompliance constitutes a violation of Oregon Revised Statutes (ORS) 468B.025 and is grounds for enforcement action; for permit termination, suspension, or modification; or for denial of a permit renewal application.

2. Penalties for Water Pollution and Permit Condition Violations

Oregon Law (ORS 468.140) allows the Director to impose civil penalties up to $10,000 per day for violation of a term, condition, or requirement of a permit.

In addition, a person who unlawfully pollutes water as specified in ORS 468.943 or ORS 468.946 is subject to criminal prosecution.

3. Duty to Mitigate

The permittee shall take all reasonable steps to minimize or prevent any discharge or sludge use or disposal in violation of this permit which has a reasonable likelihood of adversely affecting human health or the environment. In addition, upon request of the Department, the permittee shall 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 shall be submitted at least 180 days before the expiration date of this permit.

The Director 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, suspended, 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; or

c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge.

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

6. Toxic Pollutants

The permittee shall comply with any applicable effluent standards or prohibitions established under Section 307(a) of the Clean Water Act 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

The issuance of this permit does not convey any property rights of any sort, or any exclusive privilege.

8. Permit References

Except for effluent standards or prohibitions established under Section 307(a) of the Clean Water Act 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.

SECTION B. OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

1. Proper Operation and Maintenance

The permittee shall at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) which 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 which 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 shall, 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 shall not be 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 include nonuse of singular or multiple units or processes of a treatment works when the nonuse is insignificant to the quality and/or quantity of the effluent produced by the treatment works. 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.

(2) "Severe property damage" means substantial physical damage to property, damage to the treatment facilities or treatment processes 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 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 judgement to prevent a bypass which 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 Director may approve an anticipated bypass, after considering its adverse effects and any alternatives to bypassing, when the Director 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, it shall submit prior written notice, if possible at least ten days before the date of the bypass.

(2) Unanticipated bypass. The permittee shall 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. No determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance, is 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 shall 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 Event

For purposes of this permit, A Single Operational Event which leads to simultaneous violations of more than one pollutant parameter shall be treated as a single violation. A single operational event is an exceptional incident which 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 event 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 event 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 overflows. Overflows 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.

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

d. 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.

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 shall 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 shall be disposed of in such a manner as to prevent any pollutant from such materials from entering public waters, causing nuisance conditions, or creating a public health hazard.

SECTION C. MONITORING AND RECORDS

1. Representative Sampling

Sampling and measurements taken as required herein shall be representative of the volume and nature of the monitored discharge. All samples shall 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 shall not be changed without notification to and the approval of the Director.

2. Flow Measurements

Appropriate flow measurement devices and methods consistent with accepted scientific practices shall be selected and used to ensure the accuracy and reliability of measurements of the volume of monitored discharges. The devices shall 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 shall 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 shall, upon conviction, be punished by a fine of not more than $10,000 per violation, or by imprisonment for not more than two years, or by 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 shall be summarized each month on a Discharge Monitoring Report form approved by the Department. The reports shall 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 136 or as specified in this permit, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the Discharge Monitoring Report. Such increased frequency shall 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 shall be recorded unless otherwise specified in this permit.

7. Averaging of Measurements

Calculations for all limitations which require averaging of measurements shall 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 shall retain records of all monitoring information, including all calibration and maintenance records of 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 Director at any time.

9. Records Contents

Records of monitoring information shall 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 shall allow the Director, or an authorized 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 shall comply with Oregon Administrative Rules (OAR) 340, Division 52, "Review of Plans and Specifications". Except where exempted under OAR 340-52, no construction, installation, or modification involving disposal systems, treatment works, sewerage systems, or common sewers shall be commenced until the plans and specifications are submitted to and approved by the Department. The permittee shall 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 shall give advance notice to the Director of any planned changes in the permitted facility or activity which 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 shall be transferred to a third party without prior written approval from the Director. The permittee shall 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 shall be submitted no later than 14 days following each schedule date. Any reports of noncompliance shall 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 shall report any noncompliance which may endanger health or the environment. Any information shall 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 shall be called. Outside of normal business hours, the Department shall be contacted at 1-800-452-0311 (Oregon Emergency Response System).

A written submission shall also be provided within 5 days of the time the permittee becomes aware of the circumstances. If the permittee is establishing an affirmative defense of upset or bypass to any offense under ORS 468.922 to 468.946, and in which case if the original reporting notice was oral, delivered written notice must be made to the Department or other agency with regulatory jurisdiction within 4 (four) calendar days. The written submission shall 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 shall be included as information which must be reported within 24 hours under this paragraph:

a. Any unanticipated bypass which exceeds any effluent limitation in this permit.

b. Any upset which exceeds any effluent limitation in this permit.

c. Violation of maximum daily discharge limitation for any of the pollutants listed by the Director in this permit.

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 shall report all instances of noncompliance not reported under General Condition D.4 or D.5, at the time monitoring reports are submitted. The reports shall 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 shall furnish to the Department, within a reasonable time, any information which the Department may request to determine compliance with this permit. The permittee shall 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 failed to submit any relevant facts in a permit application, or submitted incorrect information in a permit application or any report to the Department, it shall promptly submit such facts or information.

8. Signatory Requirements

All applications, reports or information submitted to the Department shall be signed and certified in accordance with 40 CFR 122.22.

9. Falsification of Information

A person who supplies the Department with false information, or omits material or required information, as specified in ORS 468.953 is subject to criminal prosecution.

10. Changes to Indirect Dischargers - **[Applicable to Publicly Owned Treatment Works (POTW) only]**

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.

11. Changes to Discharges of Toxic Pollutant - **[Applicable to existing manufacturing, commercial, mining, and silvicultural dischargers only]**

The permittee must notify the Department as soon as they know or have reason to believe of the following:

a. That any activity has occurred or will occur which would result in the discharge, on a routine or frequent basis, of any toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels:

(1) One hundred micrograms per liter (100 μg/L);

(2) Two hundred micrograms per liter (200 μg/L) for acrolein and acrylonitrile; five hundred micrograms per liter (500 μg/L) for 2,4-dinitrophenol and for 2-methyl-4,6-dinitrophenol; and one milligram per liter (1 mg/L) for antimony;

(3) Five (5) times the maximum concentration value reported for that pollutant in the permit application in accordance with 40 CFR 122.21(g)(7); or

(4) The level established by the Department in accordance with 40 CFR 122.44(f).

b. That any activity has occurred or will occur which would result in any discharge, on a non-routine or infrequent basis, of a toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels":

(1) Five hundred micrograms per liter (500 μg/L);

(2) One milligram per liter (1 mg/L) for antimony;

(3) Ten (10) times the maximum concentration value reported for that pollutant in the permit application in accordance with 40 CFR 122.21(g)(7); or

(4) The level established by the Department in accordance with 40 CFR 122.44(f).

SECTION E. DEFINITIONS

1. BOD means five-day biochemical oxygen demand.

2. TSS means total suspended solids.

3. mg/L means milligrams per liter.

4. kg means kilograms.

5. m^3/d means cubic meters per day.

6. MGD means million gallons per day.

7. Composite sample means a sample formed by collecting and mixing discrete samples taken periodically and based on time or flow.

8. FC means fecal coliform bacteria.

9. Technology based permit effluent limitations means technology-based treatment requirements as defined in 40 CFR 125.3, and concentration and mass load effluent limitations that are based on minimum design criteria specified in OAR 340-41.

10. CBOD means five day carbonaceous biochemical oxygen demand.

11. Grab sample means an individual discrete sample collected over a period of time not to exceed 15 minutes.

12. Quarter means January through March, April through June, July through September, or October through December.

13. Month means calendar month.

14. Week means a calendar week of Sunday through Saturday.

15. Total residual chlorine means combined chlorine forms plus free residual chlorine.

16. The term "bacteria" includes but is not limited to fecal coliform bacteria, total coliform bacteria, and E. coli bacteria.

17. POTW means a publicly owned treatment works.

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LIFT STATION DATA

INVENTORY

PUMP CURVES

PUMP TESTS

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Ashland, OR

Wastewater Collection Master Plan

Lift Station Inventory

Est.: 10/13/11 | Last Rev: 10/20/11
By: KTK | By: KTK
per City comments

	Ashland Cr	Creek Dr	Grandview	Nevada	N. Main	N. Mountain	Shamrock	Windburn
Pump Station								
Year Constructed	2000		redesign for 2011	oldest	2007	pumps ~15yr old	1972	1999-new pumps
Wet Well Dimensions	12' dia. x 23' D	5' dia x 19' D	3'W x 15'L x 12'D	6'W x 6'L x 4'D	6' dia. x 14' D	6' dia. x 13'D	4'W x 5'L x 9'D	5' dia. x 10' D
Type	Triplex, submersible	Duplex, submersible	Duplex, Dry Well	Duplex, vacuum prime	Duplex, Dry Well	Duplex, suction lift, self prime	Duplex, Dry Well	Duplex, submersible
Pump Type	Variable Speed, non-clog	Constant Speed, non-clog	Constant Speed, non-clog	Constant Speed, non-clog	Constant Speed, non-clog	Constant Speed,	Constant Speed, non-clog	Constant Speed, non-clog
Pump Capacity	2,200 (single) @ 82' TDH 5,080 (any 2) 8890 (all 3)	150gpm @ 20' TDH (each)	800gpm @ 42' TDH (each)	82gpm @ 24' TDH (each)	490gpm @ 49' TDH (each)	Test: 395 (single) 529 (both) Curve: ~520gpm @ 23' TDH (each)	100gpm @ 42' TDH (each)	150gpm @ 17' TDH (each)
KA Pump Test (Y/N)	Y	N	N	N	N	Y	N	N
Pump Power, Phase	75 Hp	1.5 Hp, 1Ø	20 Hp	3 Hp, 3Ø	10 Hp, 3Ø	7.5 Hp, 3Ø	5 Hp	3 Hp, 3Ø
Pump Speed	1550 RPM	1150 RPM	1165 RPM	1775 RPM	1735 RPM	870 RPM		1750 RPM
Pump Mfgr(s)	WEMCO	Myers	Perless	Cornell	Flygt	HYDR-O-MATIC	Chicago	Flygt
Pump Suction, Discharge	8", 6"	4", 3"	6", 6"	4", 4"	4", 4"	6", 6"	4", 4"	4", 4"
Level Control Type	Utrasonic (Milltronics)	Float Switch	Float	Float	Multi trode	Float (Milltronics not hooked up)	Float (Milltronics not hooked up)	Mercury Float
Alarm Control	Float Switch	Float Switch	Float	Float	ultrasonic	Float	Float	Float
Flow meter (Y/N)	Y (Krohne)	N	N	N	Y	Y	N	N
Pressure Gauge/Port	Y / Y	N / N	N / Y	N / Y	N / N	N / Y	N / Y	N / N
Overflow Piping	None	to MH 11CB-018	None	None	None	None	None	None
Auxiliary Power Type	Standby Gen.	NA	None	None	None	None	None	Standby Gen.
Transfer Switch	Auto	NA	Manual	Manual	Manual	Manual	Manual	Manual
Alarm Telemetry Type	Radio	Audible and Flashing Light, Radio	SCADA	SCADA	SCADA	SCADA	SCADA	SCADA
Bypass Provisions	Y	N/A	N	N	Y	N	N	Y
Force Main								
Diameter, Material, Total Length	18" , 890'	4" D.I., 32'	6" Steel, 400'+	4" galv.,	4" A.C., 600'+	6"	4"	4" D.I., 90' (55' to high pt)
Discharge Manhole	WWTP Headworks	05CD-008	05CD-008	O4BA-006	05AC-012	04AC-020	15AA-025	09BB-001
Air/Vacc Valves	None	None	None	None	None	None	None	None
Sulfide Control System	None	None	None	None	None	None	None	None

data from previous 2005 study (confirmed w/ City 10/18/11)

red text needs updated

unknown / unsure

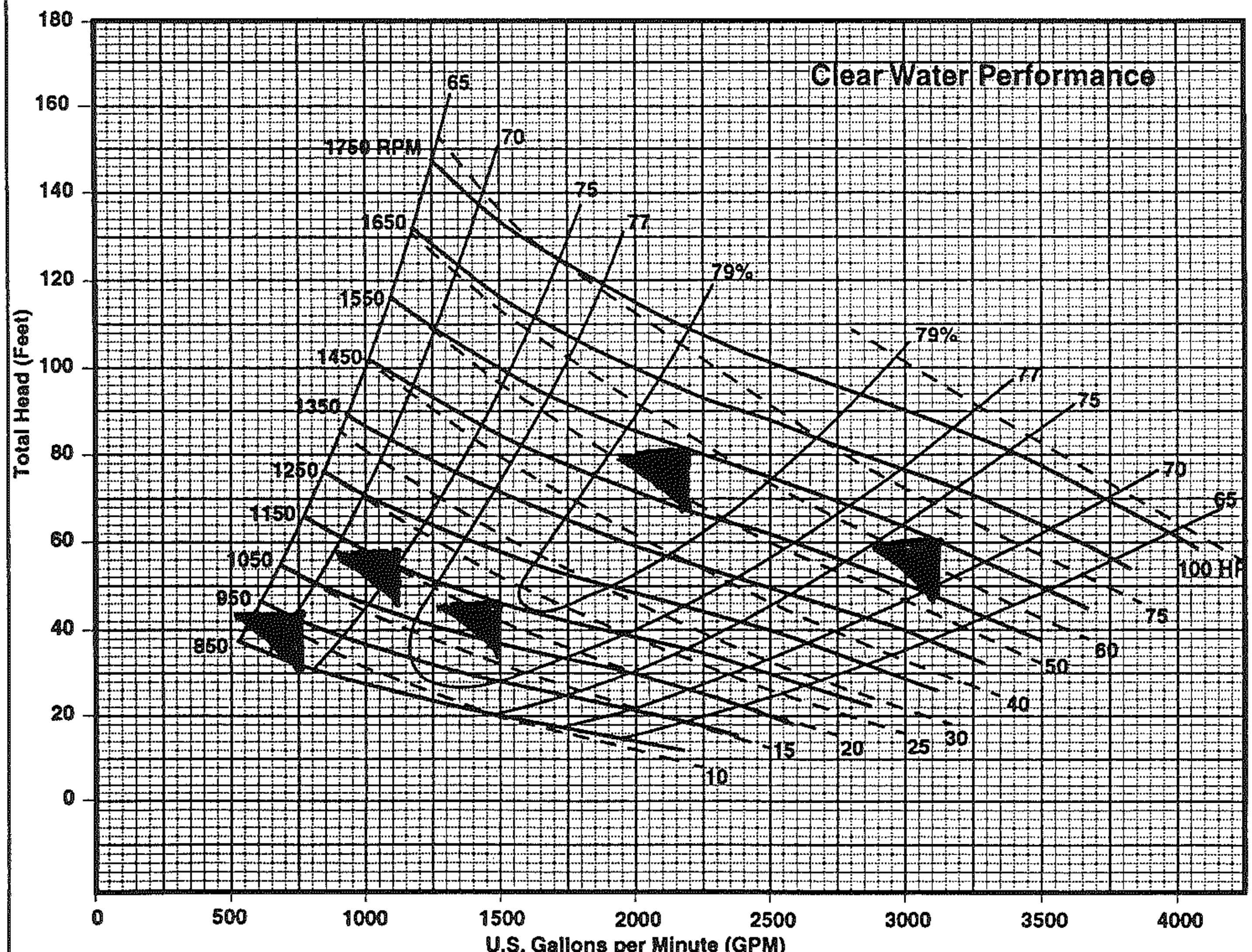
blank: site vist/City supplied

From Engr Report ROMTEC 2007

From Engr Report Otak, Inc 1999

WEMCO-Hidrostal PUMP

EnviroTech
PUMPSYSTEMS
A Weir Group company
440 W. 800 S. Salt Lake City, Utah 84110
Phone: (801) 359-8731 Fax: (801) 355-9303



8.0" Suction
6.0" Discharge
Variable RPM
13-7/8" Impeller Diameter
4.50" Max Sphere Size

$$BHP = \frac{GPM \times FT \times SG}{3960 \times Efficiency}$$

The brake horsepower and efficiency shown is for pumps with properly installed and lubricated packing. Pumps with mechanical seals will require additional horsepower and the factory may be contacted for these values. Certified tests are performed using Hydraulic Institute acceptance level A.

3/22/99

Supersedes P25-D112 dated 3/1/90

P25-D112

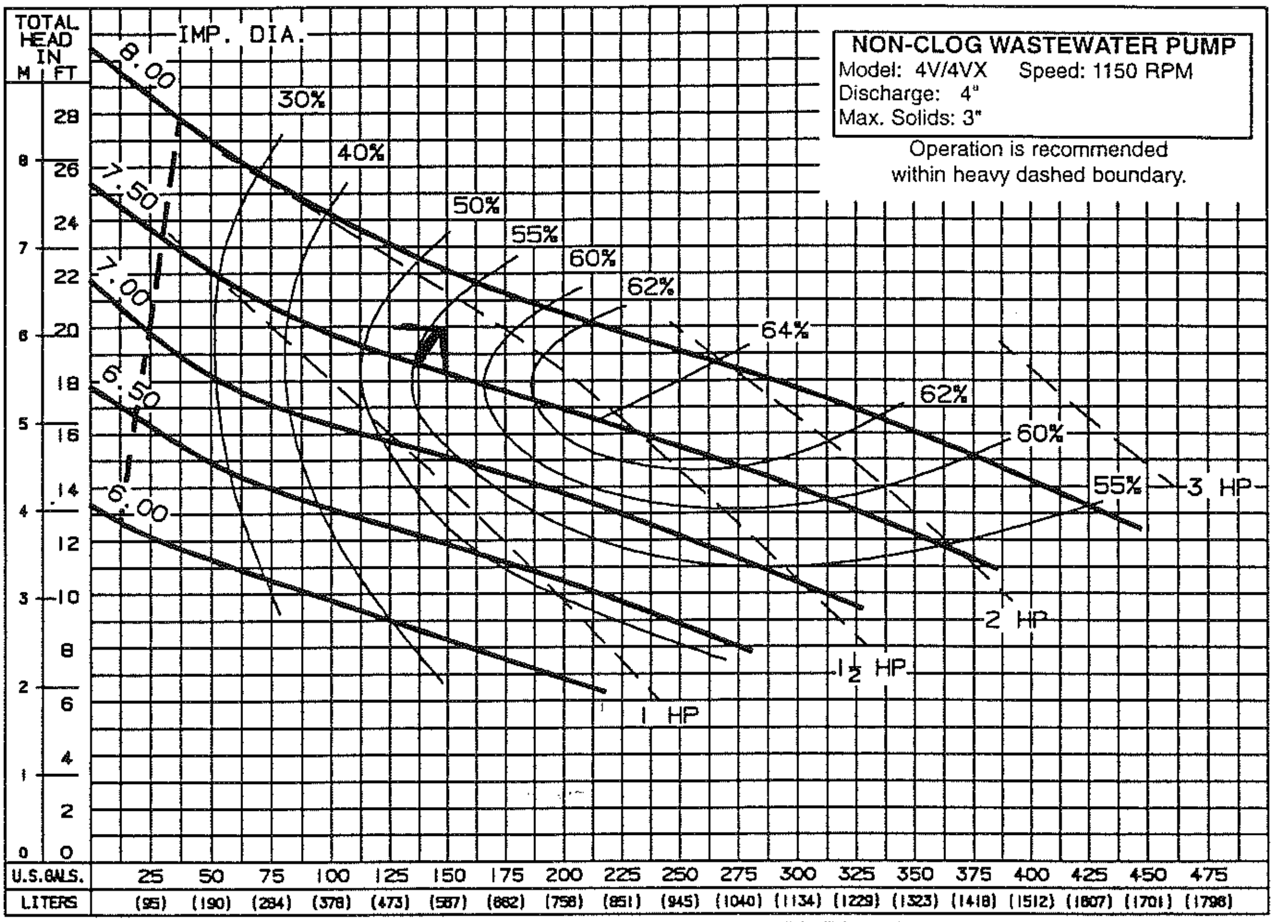
F6K-S

Ashland Creek Pump Curve

S/N (s) 99X22699, -700, -701

Pump Performance

Creek Drive Pump Curve



Pump performance is based on clear water (1.0 specific gravity @ 68°F) and pump fluid end (hydraulic) efficiency. Motor data based on 40°C ambient temperature.

Available Models		Motor Electrical Data											
Standard	Explosion Proof	HP	Volts	Phase	Start Amps	Run Amps	Service Factor Amps	Run KW	Service Factor KW	Start KVA	Run KVA	NEC Code Letter	Service Factor
4V10M6-21	4VX10M6-21	1	230	1	35	9	10.8	1.5	1.9	8.1	2.1	K	1.2
4V10M6-03	4VX10M6-03	1	200	3	23.8	7.4	8.9	1.8	2.3	8.3	2.6	K	1.2
4V10M6-23	4VX10M6-23	1	230	3	20.7	6.4	7.8	1.8	2.3	8.3	2.6	K	1.2
4V10M6-43	4VX10M6-43	1	460	3	10.4	3.2	3.9	1.8	2.3	8.3	2.6	K	1.2
4V10M6-53	4VX10M6-53	1	575	3	8.3	2.6	3.1	1.8	2.3	8.3	2.6	K	1.2
4V15M6-21	4VX15M6-21	1.5	230	1	42	11	13.2	1.9	2.4	9.7	2.5	H	1.2
4V15M6-03	4VX15M6-03	1.5	200	3	33.4	9.8	11.8	2.2	2.8	11.6	3.3	J	1.2
4V15M6-23	4VX15M6-23	1.5	230	3	29	8.5	10.2	2.2	2.8	11.6	3.3	J	1.2
4V15M6-43	4VX15M6-43	1.5	460	3	14.5	4.2	5.1	2.2	2.8	11.6	3.3	J	1.2
4V15M6-53	4VX15M6-53	1.5	575	3	11.6	3.3	4	2.2	2.8	11.6	3.3	J	1.2
4V20M6-21	4VX20M6-21	2	230	1	60	18	21	2.8	3.5	19.5	4.2	H	1.2
4V20M6-03	4VX20M6-03	2	200	3	56	12	14.5	2.4	3.6	19.5	4.2	L	1.2
4V20M6-23	4VX20M6-23	2	230	3	49	10.5	12.6	2.4	3.6	19.5	4.2	L	1.2
4V20M6-43	4VX20M6-43	2	460	3	24.5	5.2	6.3	2.4	3.6	19.5	4.2	L	1.2
4V20M6-53	4VX20M6-53	2	575	3	19.6	4.2	5	2.4	3.6	19.5	4.2	L	1.2
4V30M6-21		3	230	1	60	21	21	3.8	3.8	13.8	4.8	H	1.0
4V30M6-03		3	200	3	56	16.8	16.8	3.8	3.8	19.5	5.6	H	1.0
4V30M6-23		3	230	3	49	14	14	3.8	3.8	19.5	5.6	H	1.0
4V30M6-43		3	460	3	24.5	7	7	3.8	3.8	19.5	5.6	H	1.0
4V30M6-53		3	575	3	19.6	5.6	5.6	3.8	3.8	19.5	5.6	H	1.0

Motor Efficiencies and Power Factor									
		Motor Efficiency %				Power Factor %			
HP	Phase	Service Factor Load	100% Load	75% Load	50% Load	Service Factor Load	100% Load	75% Load	50% Load
1	1	59.5	58	53	44.5	75	72	66	58
1	3	64	61.5	55.5	46	75.5	71	62	48.5
1.5	1	56	59	55	47	80	77	73	67.5
1.5	3	68	67	63.5	56	69.5	66	59.5	50
2	1	61	59	54	45.5	73	68	60	51
2	3	71	69	64	54	71.5	58.5	51	43
3	1	60	60	60	54	78	78	71	60
3	3	73	73	70.5	64	69	69	62	51

Myers® F. E. Myers, 1101 Myers Parkway, Ashland, Ohio 44805-1969
419/289-1144 • FAX: 419/289-6658 • TLX: 98-7443

K3615 3/95

Printed in U.S.A.

4NNDL Curves

NEVADA STREET

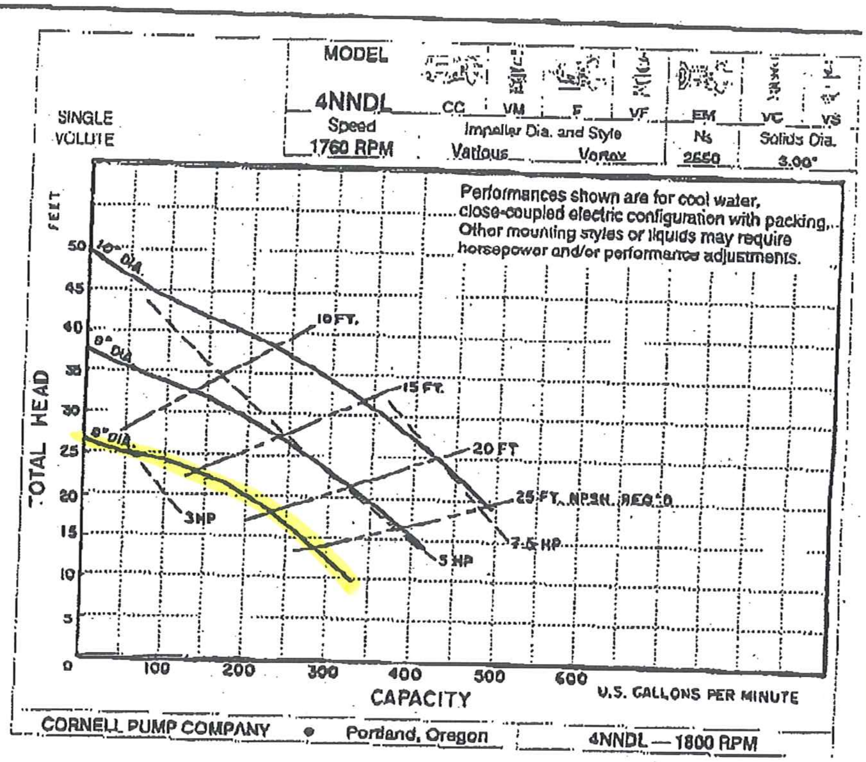
Model 4NNDL
1800 and 1200 RPM
60 Hertz
Vortex Impeller

1800 RPM

Performances shown are for cool water, close-coupled electric configuration with packing. Other mounting styles or liquids may require horsepower and/or performance adjustments.

Feet x .305 = Meters
Inches x 25.4 = Millimeters
GPM x .227 = Cubic Meters/Hour
GPM x 3.785 = Liters/Minute
HP x .746 = KW

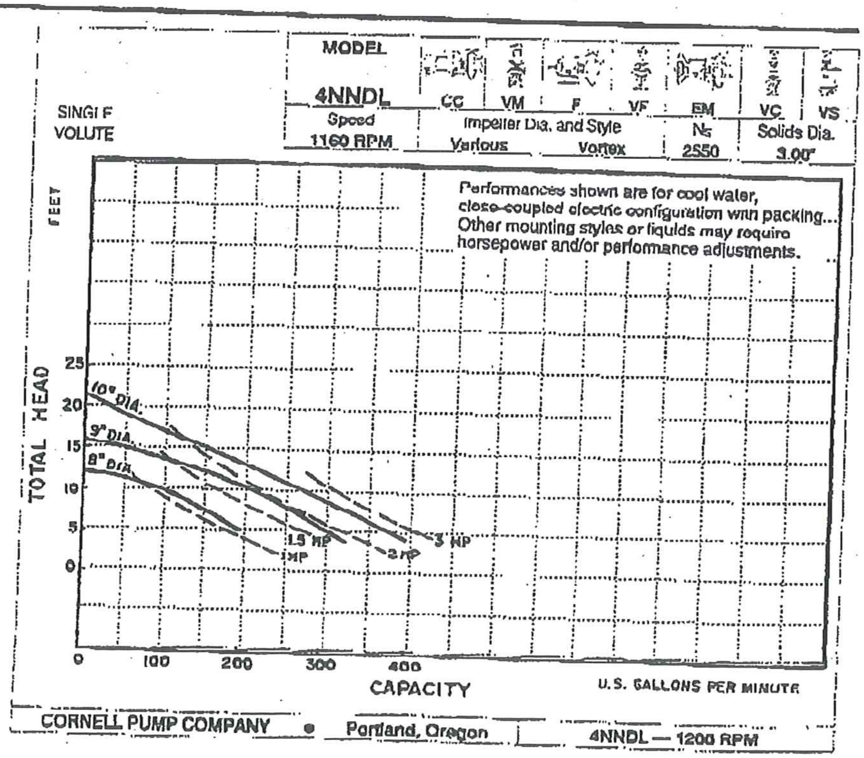
Repair note for 3hp 1775 RPM



1200 RPM

Performances shown are for cool water, close-coupled electric configuration with packing. Other mounting styles or liquids may require horsepower and/or performance adjustments.

Feet x .305 = Meters
Inches x 25.4 = Millimeters
GPM x .227 = Cubic Meters/Hour
GPM x 3.785 = Liters/Minute
HP x .746 = KW



REVISED

PERFORMANCE CURVE

FLYGT	PERFORMANCE CURVE	PRODUCT: NP3127.090	TYPE: HT
DATE: 2007-08-16	PROJECT: NORTH MAIN	CURVE NO: 63-488-00-3702	ISSUE: 1

	1/1-LOAD	3/4-LOAD	1/2-LOAD
POWER FACTOR	0.89	0.87	0.81
EFFICIENCY	83.5 %	85.0 %	84.5 %
MOTOR DATA	---	---	---

RATED POWER	10	hp
STARTING CURRENT ...	128	A
RATED CURRENT ...	25	A
RATED SPEED	1735	rpm
TOT.MOM.OF INERTIA ...	0.054	kgm2
NO. OF BLADES	2	

IMPELLER DIAMETER: 215 mm

MOTOR #	STATOR	REV
21-12-4AL	12Y//	11

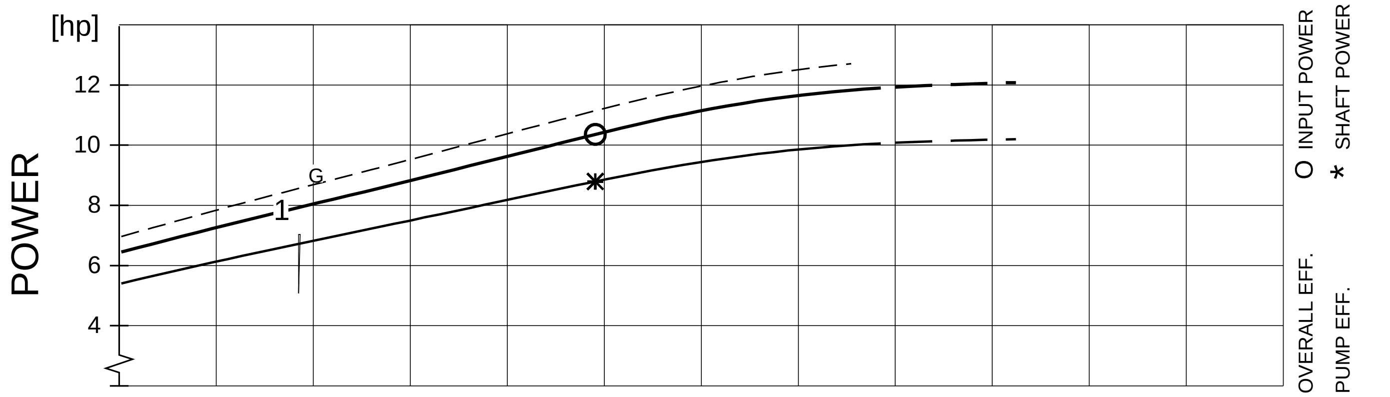
FREQ.	PHASES	VOLTAGE	POLES
60 Hz	3	230 V	4

GEARTYPE	RATIO
---	---

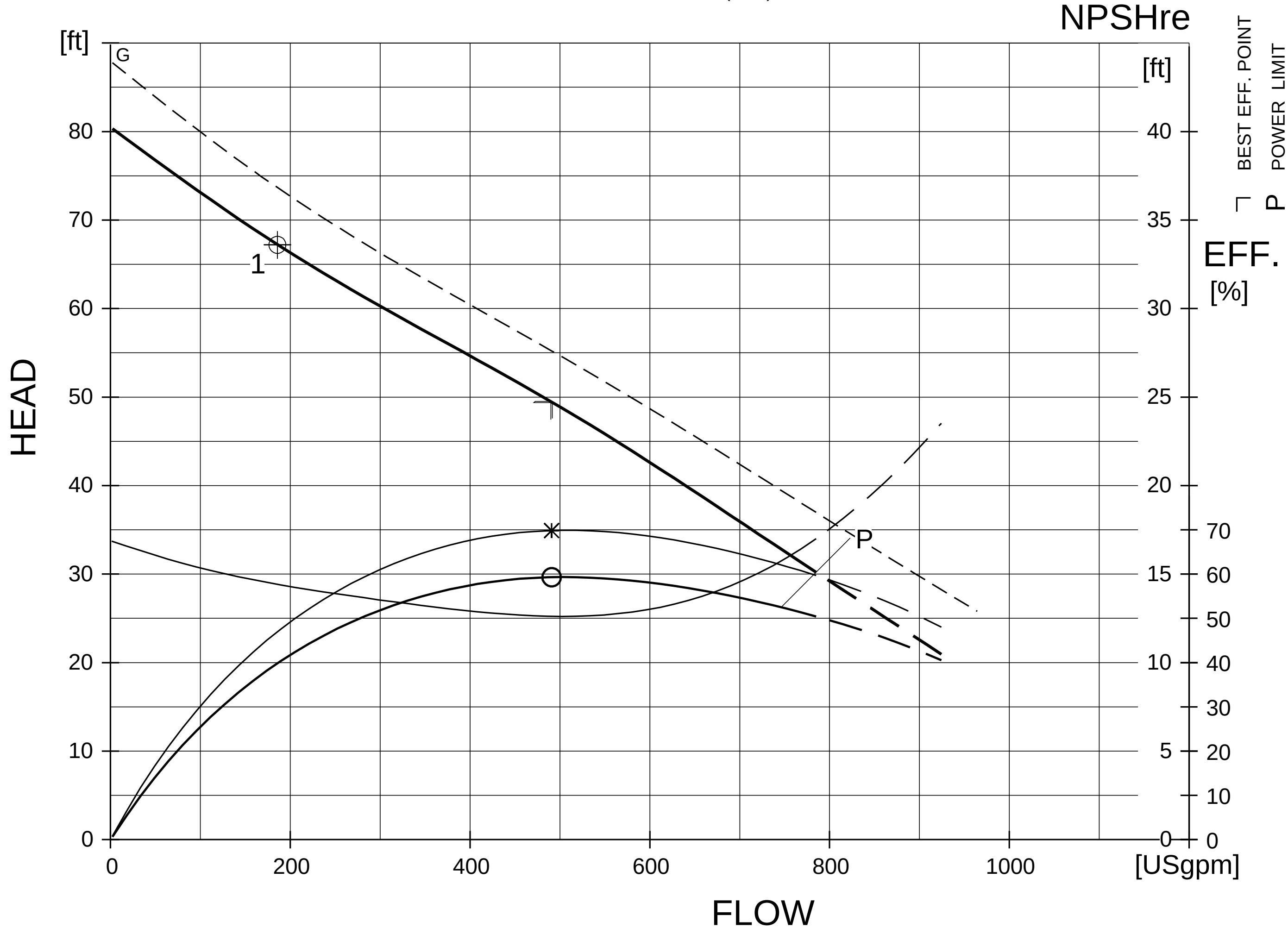
COMMENTS

INLET/OUTLET: -/ 4 inch

IMP. THROUGHLET: ---



DUTY-POINT	FLOW[USgpm]	HEAD[ft]	POWER [hp]	EFF. [%]	NPSHre[ft]	GUARANTEE
1	186	67.2	7.90 (6.71)	39.7 (46.9)	14.4	
B.E.P.	491	49.4		59.3 (69.8)	12.6	HI level A



FLYPS3.1.5.9 (20060531)

NPSHre = NPSH3% + min. operational margin

Performance with clear water and ambient temp 40 °C

GUARANTEE BETWEEN LIMITS (G) ACC. TO

HI level A

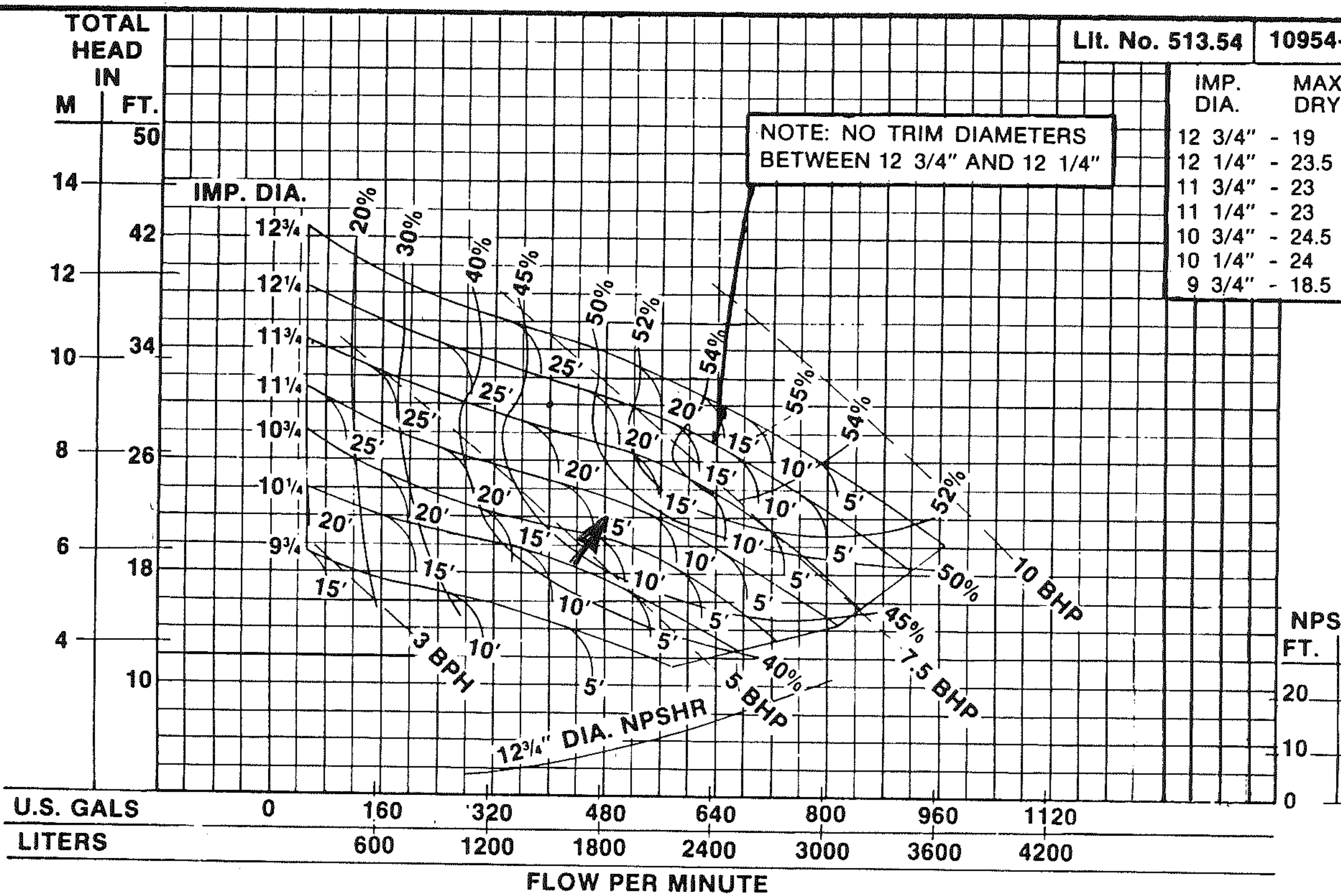
FIXED RPM. VARIOUS DIAMETERS

MODEL: 60MPD SELF PRIMER - MAX. SOLIDS: 4" SPHERE - 870 RPM

Lit. No. 513.54 | 10954-000-1

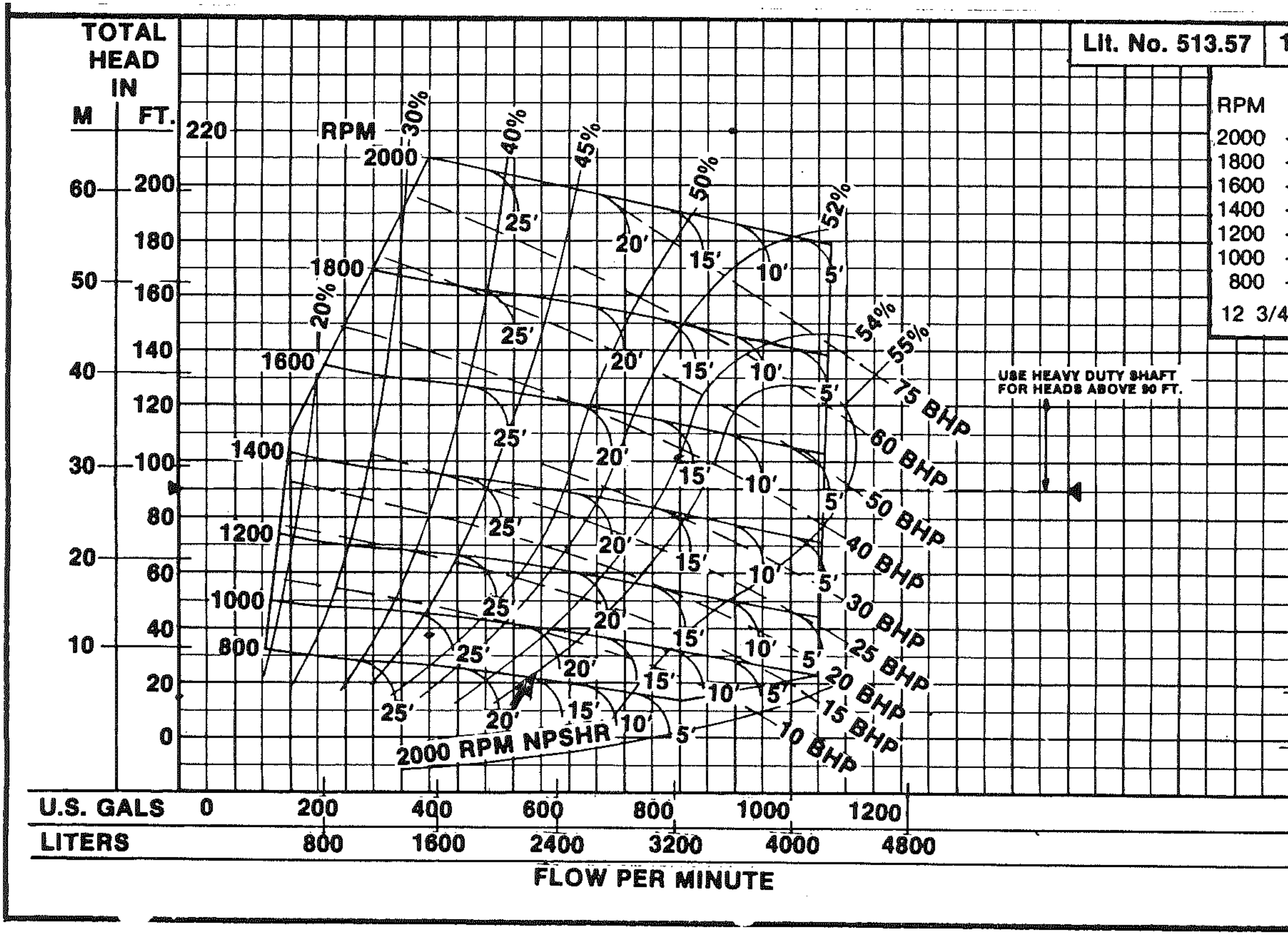
IMP. DIA.	MAX. VERT. DRY PRIME
12 3/4"	- 19 FT.
12 1/4"	- 23.5
11 3/4"	- 23
11 1/4"	- 23
10 3/4"	- 24.5
10 1/4"	- 24
9 3/4"	- 18.5 FT.

NOTE: NO TRIM DIAMETERS BETWEEN 12 3/4" AND 12 1/4"



North Mountain Pump Curve

12 3/4" IMPELLOR VARIOUS R.P.M.'s



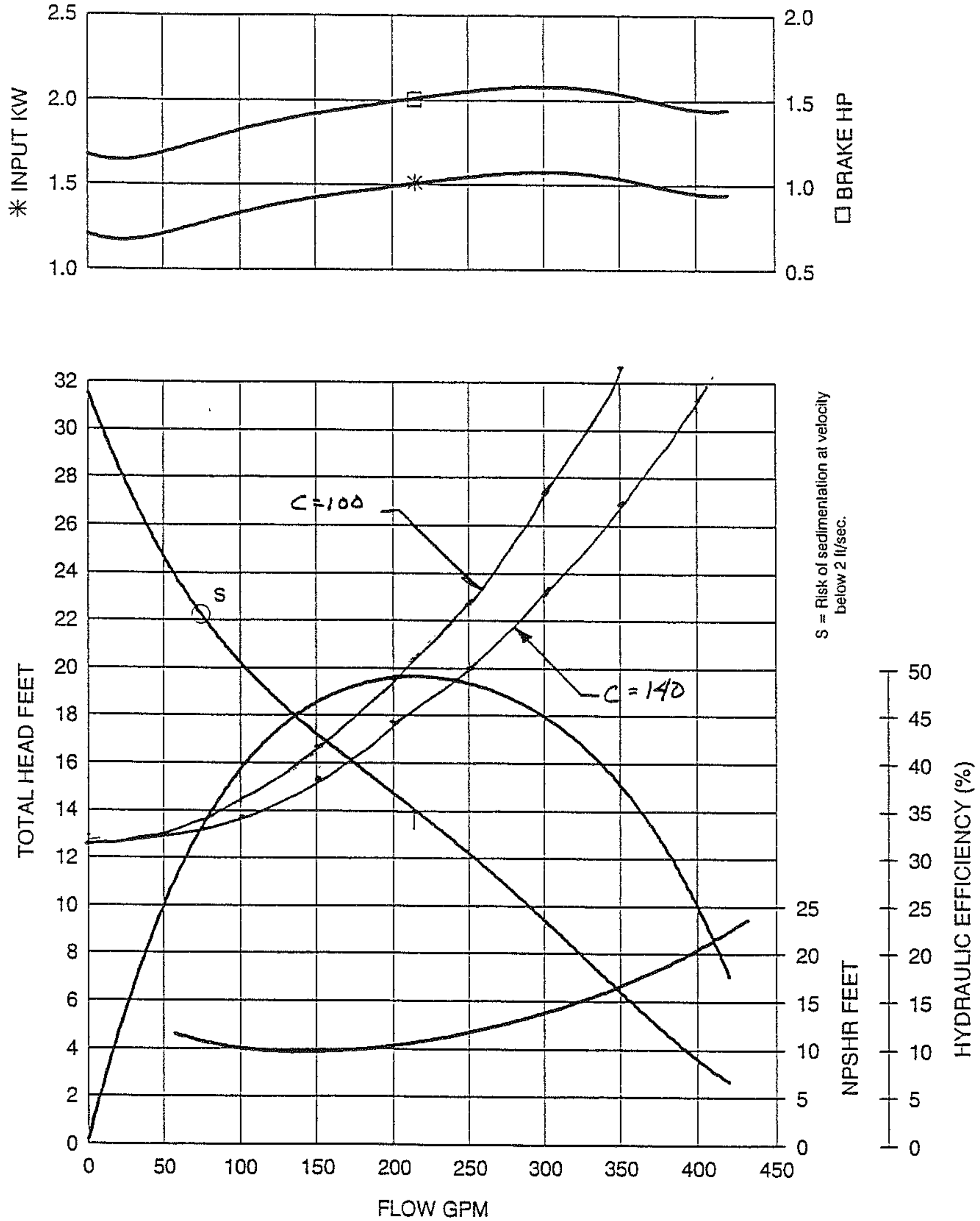
RPM	MAX. VERT. DRY PRIME
2000	- 25 FT.
1800	- 24
1600	- 20
1400	- 12.8
1200	- 15.4
1000	- 12.0
800	- 20.8 FT.

12 3/4 impeller dia.

North Mountain Pump Curve

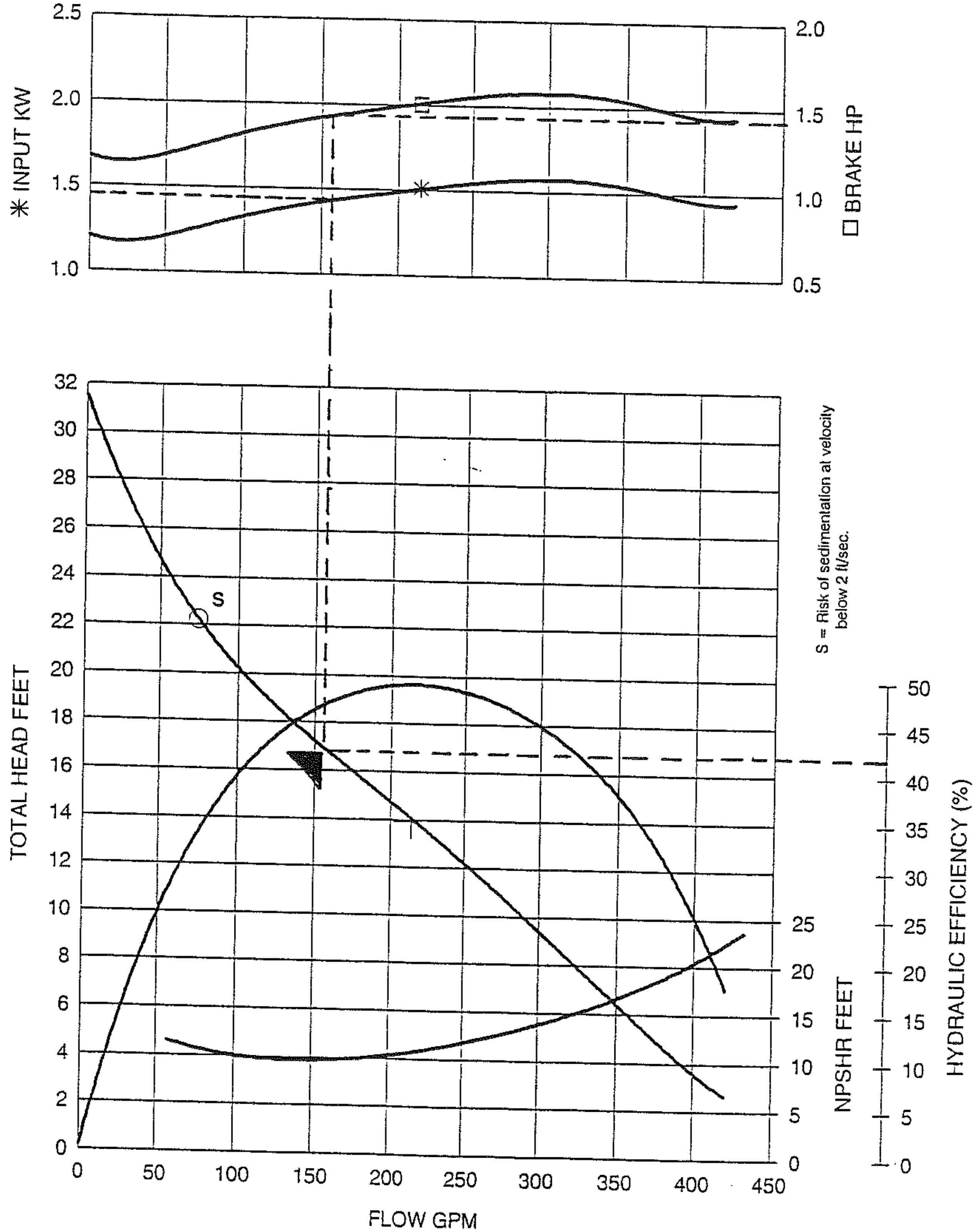
Windburn Pump Curve

CONFIG.		C-3085 438 Impeller	SECTION	PAGE
CP/CS			3	7
PHASE	VANES		SUPERSEDES	ISSUED
3	1		6/94	2/96



Windburn Pump Curve

CONFIG.		C-3085 438 Impeller	SECTION	PAGE
CP/CS			3	7
PHASE	VANES		SUPERSEDES	ISSUED
3	1		6/94	2/96



FLYGT

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Pipeline Condition Evaluation

Condition Summary

TV Log Review - Photos

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City of Ashland
Comprehensive Sanitary Sewer Master Plan

Sewer Pipeline Condition Evaluation

Number of Various Defects

Pipeline ID	Size	Material	Length (ft)	Structural Problems: Cracks: Small	Medium	Large	Misaligned / Broken Joints: Misaligned 1	Misaligned 4	Small Crack	Medium Crack	Large Crack	Moderate offset	Severe offset	Exposed Gasket	Laterals: Intruding Service 1"-3"	Intruding Service 3+"	Lateral w/Defect 4	Lateral w/Defect 7	Light Roots in Service	Medium Roots in Service	Heavy Roots in Service	Gasket
		Weighting Factor		1	4	7	1	4	1	4	7	3	9	4	4	7	4	7	1	4	7	4
2842	8	PVC/Con	226																			
2707	8	PVC	104																			
2223	8	Con	336									1		1	1	1						
2714	8	PVC	96																			
2772	8	con	176					1						1								
2877	15	con	218				1			1												
2962	8	pvc	18																			
5166	8	pvc/con	65					2						1								
5415	24	pvc	393																			
2961	8	pvc	48																			
2950	8	pvc	44																			
3315	6	clay	67				3					1										
5178	15	con	539																			
2678	15	con	369																			
2258	8	clay	406					1					2						1			
2334	6	clay	270				2	2				1										
2349	6	clay	493	1			8	2			1		1						3			
2343	6	clay	157																2	0	1	
2345	6	clay	434	2	2		1		1								1		2			
2230	10	con	267						1	1												
2309	6	con	401				1												2			
2328	6	clay	411	8	3	1	3	4	1	1	1						2	1	3			
2977	10	clay	371	1	2		1	1									2		2			
2518	6	clay	364				1	2	1								1			2		
6080	6	clay	142				1	1											3			
2614	8	clay	250				1										1					

Pipeline ID	Oval Structure, Collapsed Structure and Structure Defects: Past Repair	Broken (small)	Broken (large)	Small Hole	Large Hole	Small Sag	Medium Sag	Large Sag	Pipe deteriorated (point) - light	Pipe deteriorated (point) - medium	Pipe deteriorated (point) - heavy	Oval < 5%	Collapsed	Light Overall Pipe Deterioration	Medium Overall Pipe Deterioration	Heavy Overall Pipe Deterioration	Root Problems: Light Root Instruction	Medium Root Intrusion	Heavy Root Intrusion	I/I Issues: Light I/I	Medium I/I	Heavy I/I
Weighting Factor	2	4	7	4	7	1	4	7	1	4	7	3	9	1	3	7	1	3	7	1	5	9
2842	1					1	2					1										
2707																						
2223	1																					
2714						1																
2772						1																
2877	1				1	3							1						1			
2962						1																
5166	1																					
5415						2																
2961																						
2950																						
3315	1																					
5178						3												2				
2678						3											1	1				
2258	2							2									2					
2334	2					1																
2349					1	2											4					
2343						1											3	1				
2345				2	1												3					
2230							1										6	1		4		
2309	1					1		1									25	3	4			
2328	1					1											5					
2977		2															1	2				
2518	2																5					
6080	1																4					
2614																	14					

Pipeline ID	Sewer Condition Rankings: Structural Rating	Root Rating	I/I Rating	Overall	City Rankings: Service	Structural	Overall	Comments
2842	64	0	0		80	80	0	repair all sags [pvc/con light flow
2707	0	0	0		100	100	0	
2223	24	0	0		60	80	acceptable	flow medium
2714	4	0	0		0	100	ocr?	light flow
2772	18	0	0		80	60	0	No cap / CO / MH at pipe end
2877	140	10	0		40	40	0	collapsed pipe on top big root mass 12" hole in pipe [high flow may need to up size] pvc/con
2962	4	0	0		60	100	0	line must drop down to main
5166	63	0	0		60	80	0	con/pvc to pvc/con big offset
5415	8	0	0		60	100	ocr?	lines in good shape medium flow
2961	0	0	0		60	100	ocr?	lines in good shape
2950	0	0	0		60	100	ocr?	lines in good shape light flow
3315	35	0	0		80	80	0	clay to pvc deviates to left light flow
5178	12	3	0		0	0	0	pipe in good shape medium flow needs root control flow light roots
2678	12	3	0		40	100	ocr?	pipe in good shape medium flow needs root control
2258	89	1	0		40	80	acceptable	from 264 to 405 large sags[line need regraded [needs replaced medium flow]
2334	34	0	0		60	80	acceptable	pipe in ok shape misaligned bells pvc to clay repairs light flow
2349	58	2	0		80	20	marginal	lots misaligned bells lot of fall [pipe bursting]light flow [runs to long max 400 feet]
2343	21	11	0		0	100	marginal	roots in services and in bells light flow
2345	72	2	0		80	20	marginal	light roots small sag medium flow [pipe bursting]
2230	22	10	4		0	80	orc failed	bells are starting to leak cracks in pipe long sag medium flow
2309	42	46	0		0	80	orc failed	light flow needs root control and grease control
2328	67	4	0		0	40	failed	light flow pvc/con 6"to8" poor shape
2977	51	6	0		0	80	failed	heavy flow needs up size poor shape
2518	34	4	0		60	80	ocr acceptable	pipe in ok shape light flow
6080	25	8	0		40	60	marginal	lots of roots light flow
2614	6	17	0		80	80	acceptable	lots of roots light flow

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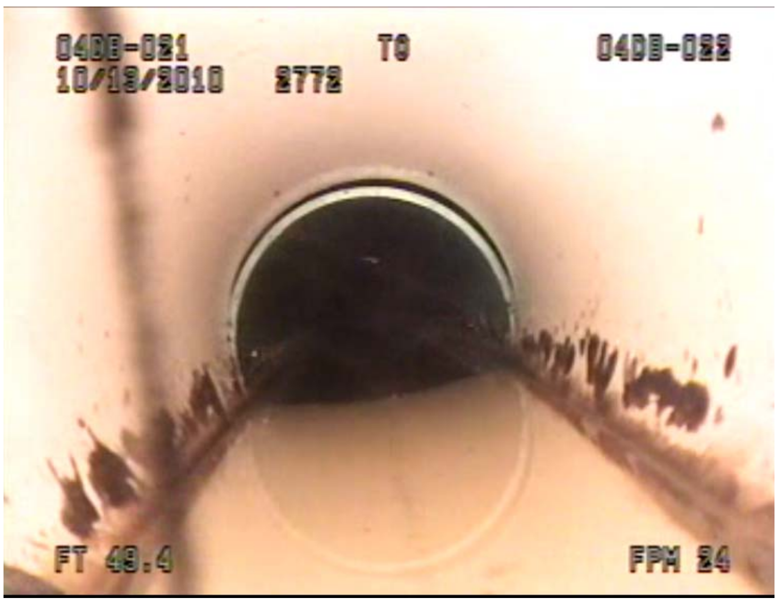
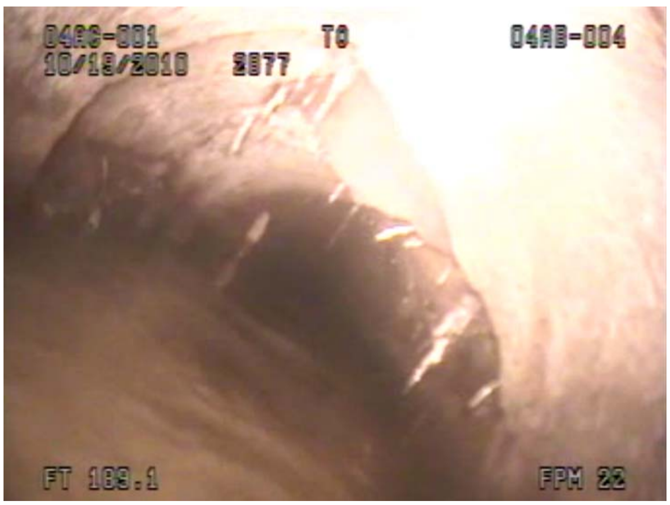
City of Ashland
Comprehensive Sanitary Sewer Master Plan
TV Log Review

Pipe # 2714 -- Needs cap or manhole or clean out



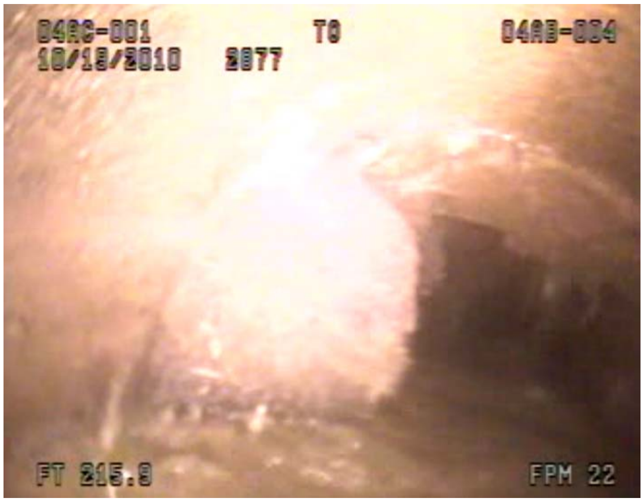
Pipe # 2772 -- Misaligned joint; small sag

Pipe # 2877 Con/pvc transition; misaligned bell



Pipe# 2877 Rock in pipe; broken pipe

Pipe # 2877 Root mass; broken pipe



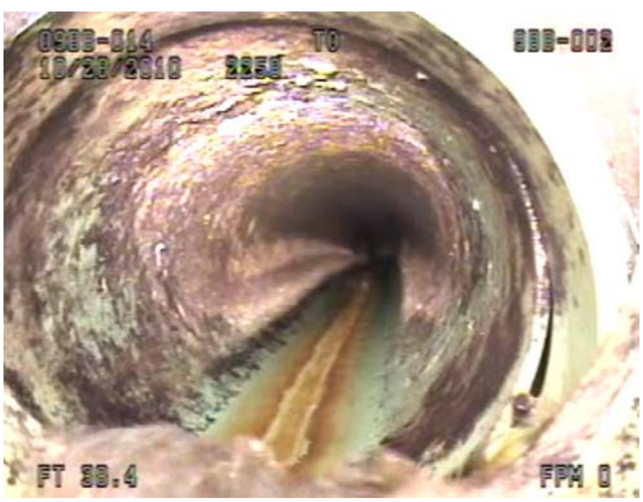
Pipe # 2877 Root mass; collapsed pipe

Pipe # 5166 Con/pvc transition; significant joint offset

Pipe# 5166 Pvc/con offset

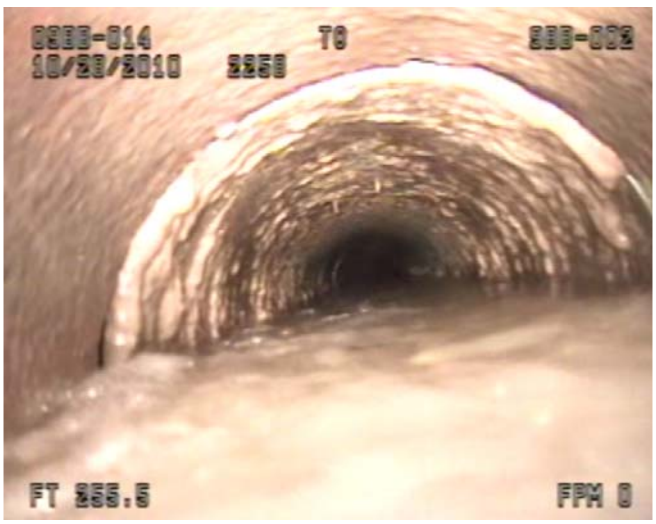
Pipe# 2678 Roots

Pipe# 2258 Misaligned con/pvc

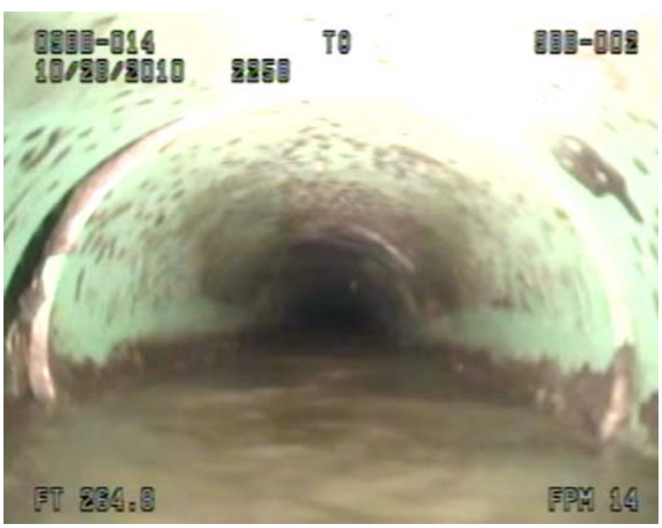


Pipe# 2258 Offset service

Pipe # 2258 Offset service; roots



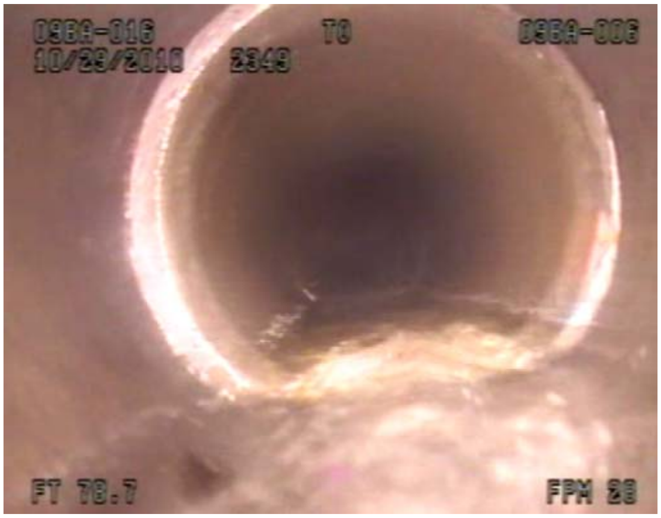
Pipe # 2258 Flow 25% to 50% of pipe



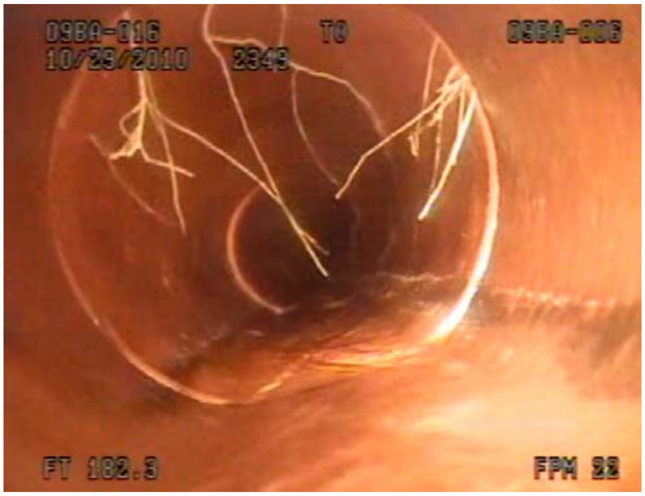
Pipe # 2258 Misaligned pipe

Pipe # 2334 Misaligned bell

Pipe # 2349 Misaligned service



Pipe # 2349 Misaligned bell



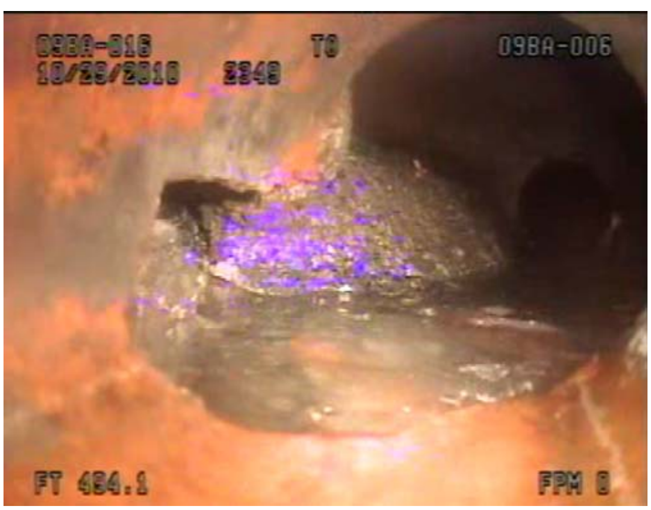
Pipe # 2349 Roots

Pipe # 2349 Roots in service

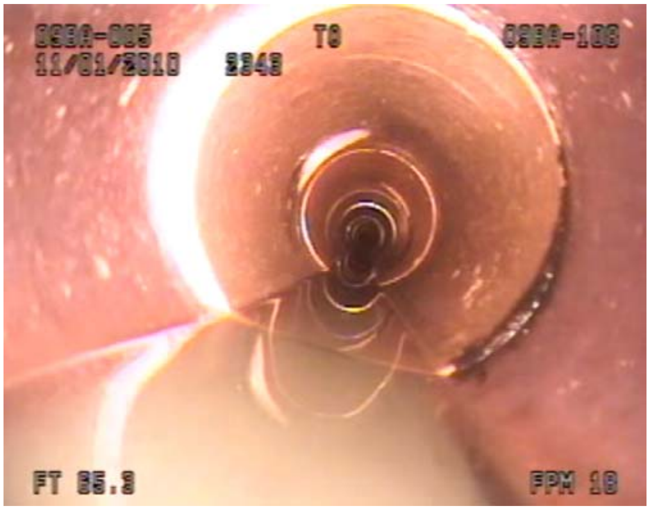
Pipe # 2349 Large crack at bell

Pipe# 2349 Small sag

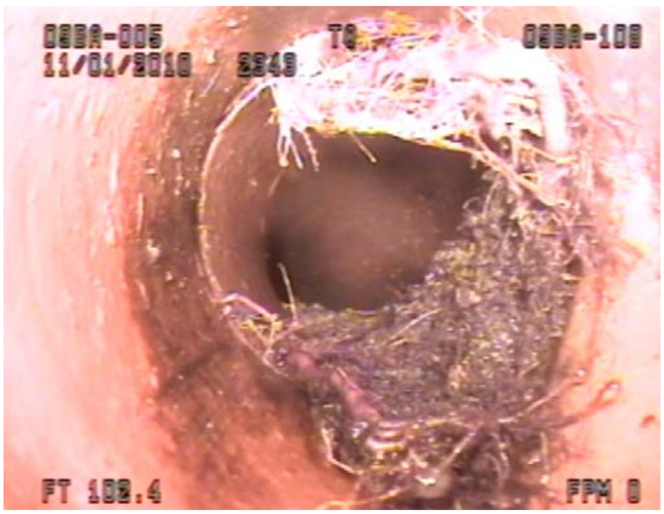
Pipe # 2349 Misaligned bell



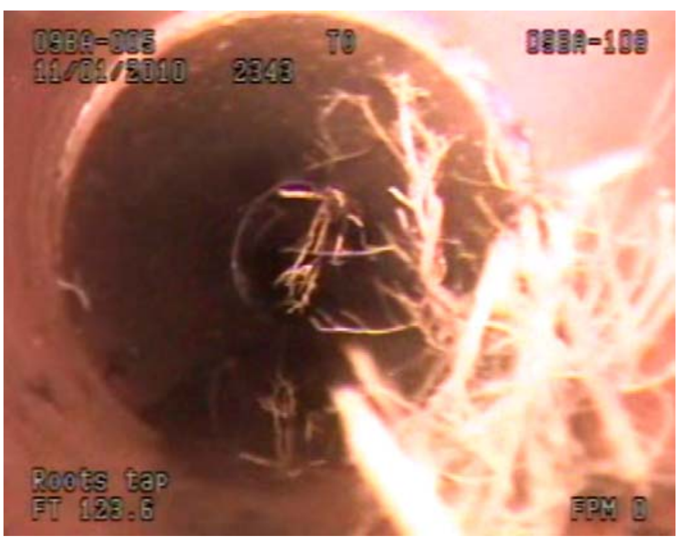
Pipe # 2349 Bottom of pipe missing



Pipe # 2343 Small sag, misaligned joint



Pipe # 2343 Roots in service



Pipe # 2343 Roots in bell

Pipe # 2343 Roots in unused service

Pipe # 2343 Roots in service

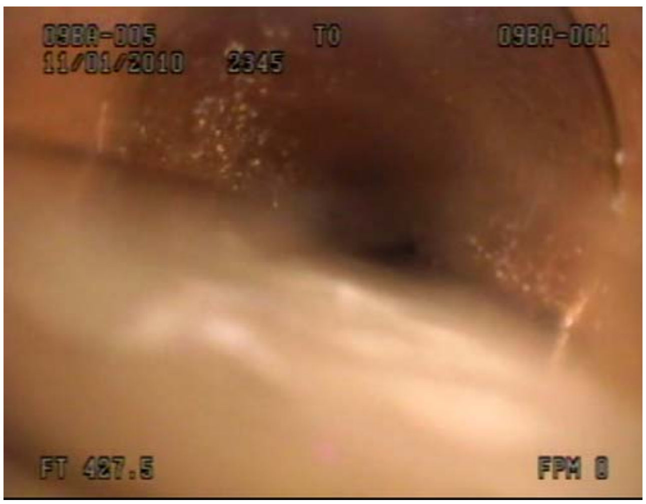
Pipe # 2345 Longitudinal crack

Pipe # 2345 Roots at bell

Pipe # 2345 Service offset; roots

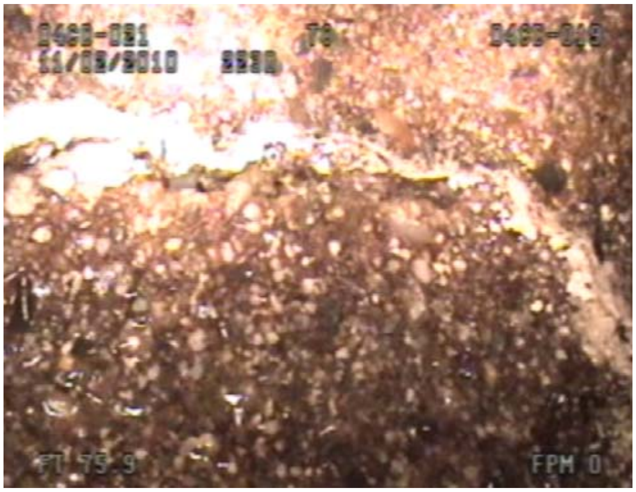
Pipe # 2345 Multiple cracks

Pipe# 2345 Service crack and hole



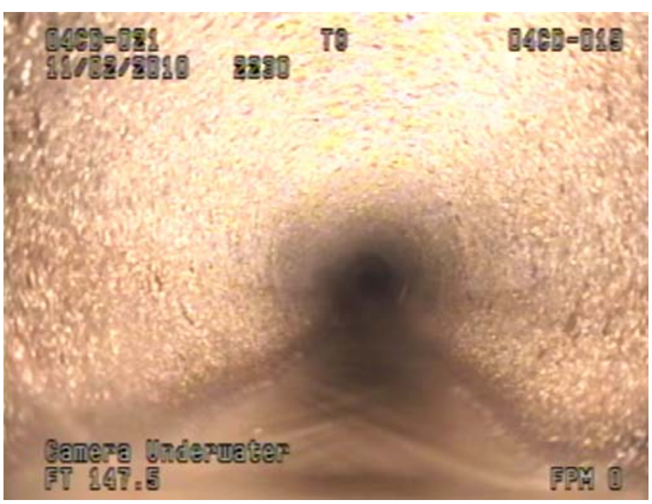
Pipe# 2345 Large sag from 427' to end

Pipe# 2230 Crack with infiltration

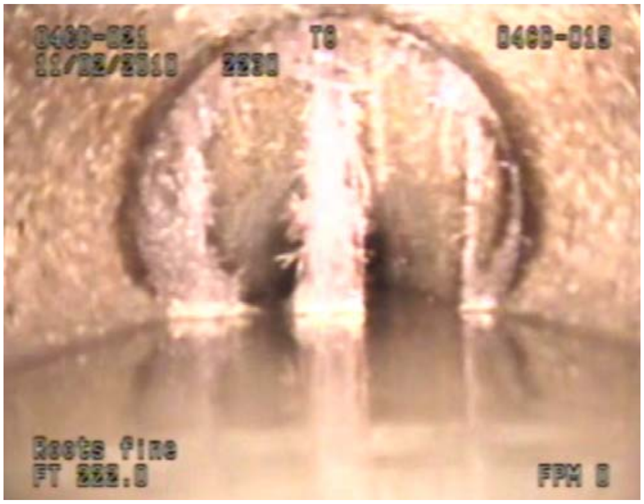


Pipe # 2230 Crack with infiltration

Pipe# 2230 Medium sag from 117 feet 146 feet

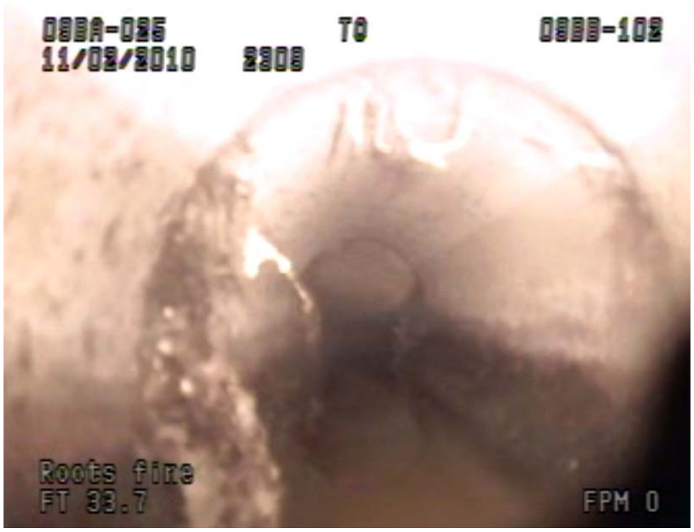


Pipe # 2230 Line no sag



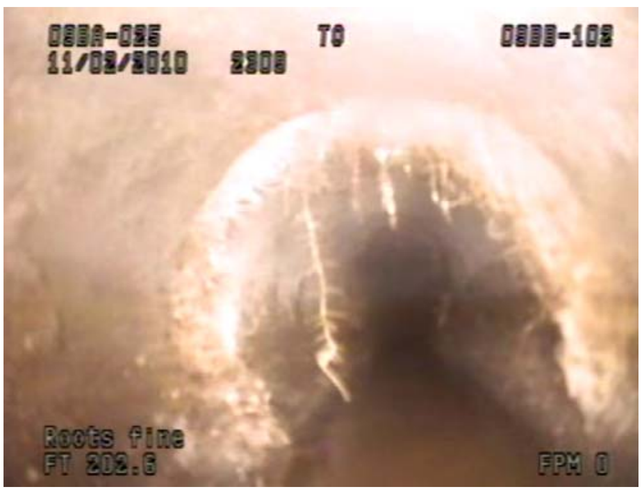
Pipe # 2230 Roots at bell

Pipe# 2309 Misaligned pipe



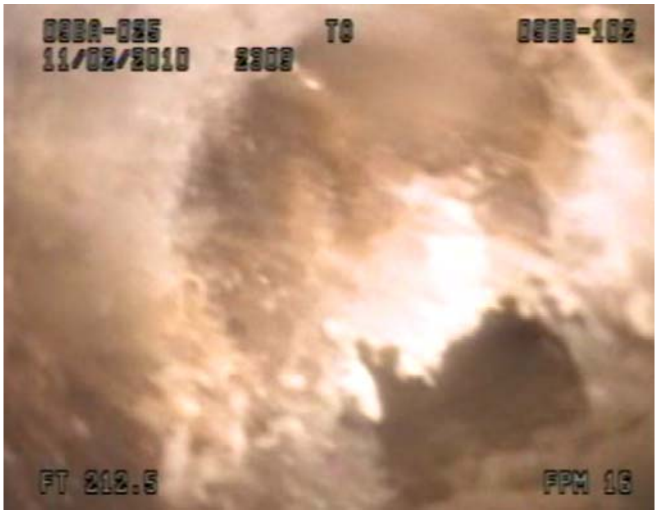
Pipe # 2309 Roots at bell

Pipe # 2309 Roots & grease buildup

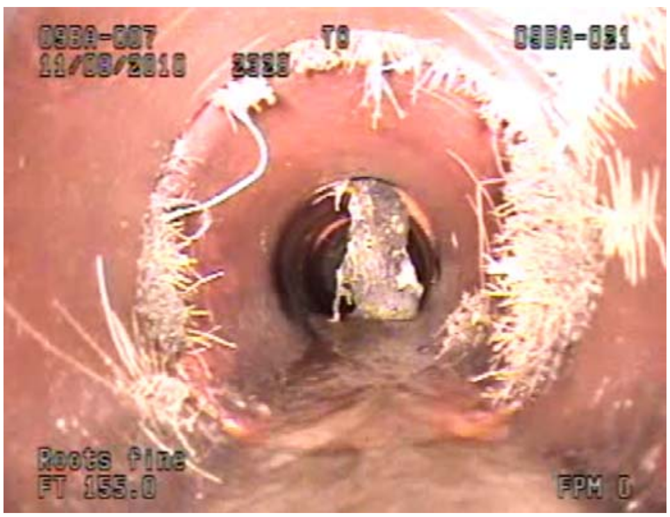


Pipe# 2309 Root at bell

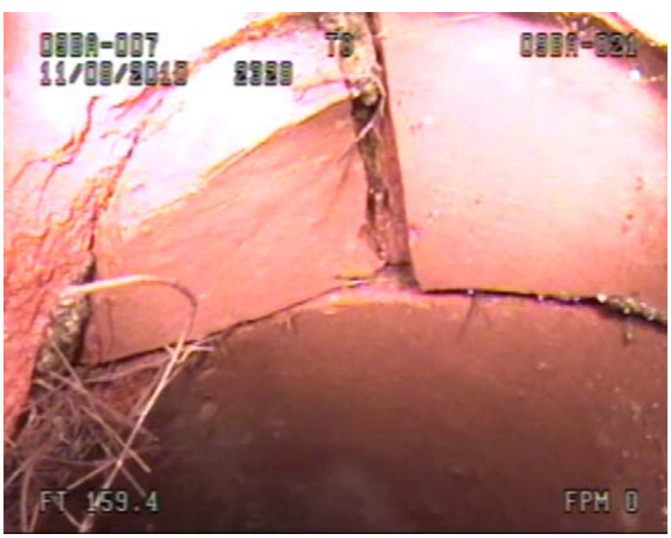
Pipe# 2309 Root mass; grease buildup



Pipe # 2309 Root mass

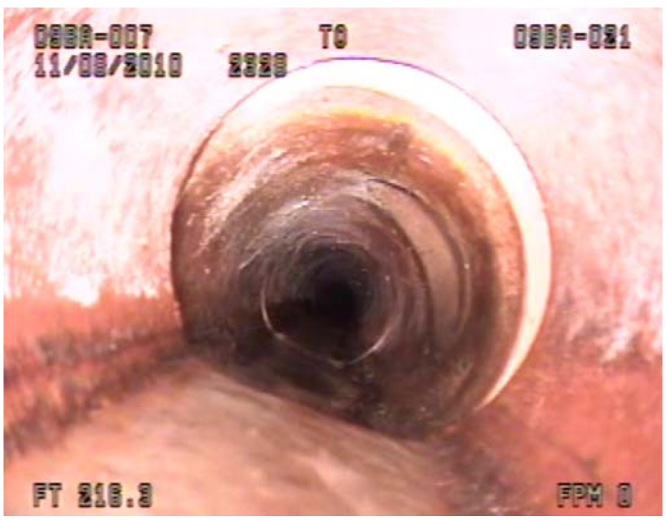


2328 Root at bell



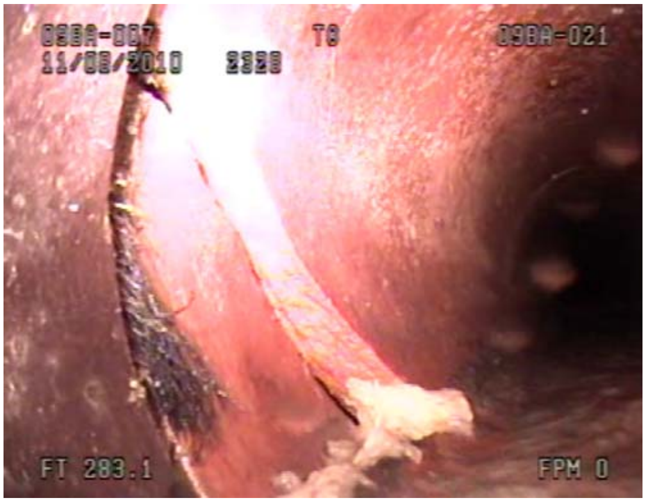
Pipe# 2328 Roots service; crack in service

Pipe# 2328 Longitudinal cracking

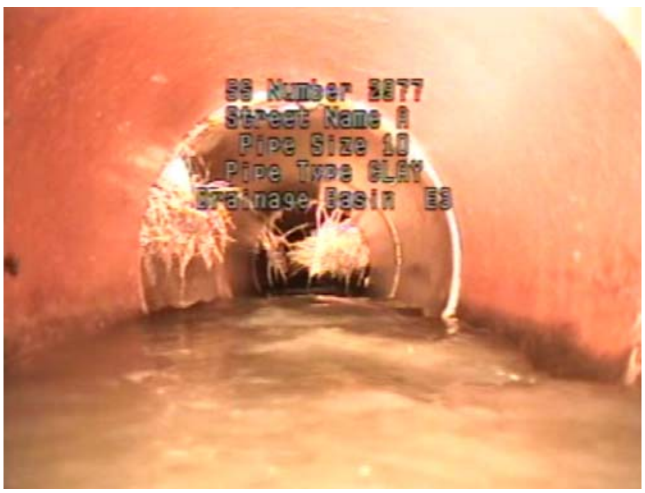


Pipe# 2328 Offset

Pipe# 2328 Crack with missing pipe and roots

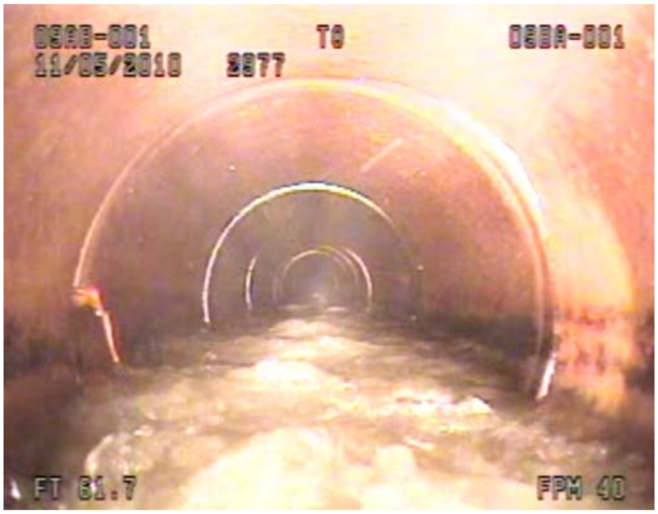


2328 Crack at joint with roots



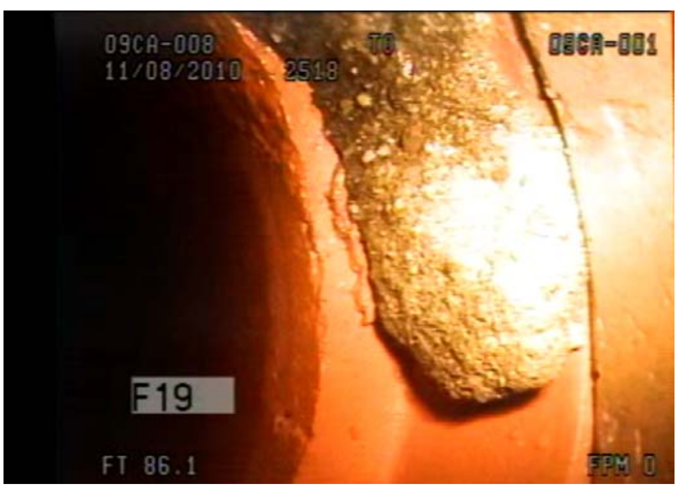
Pipe# 2977 Root mass

Pipe# 2977 Longitudinal crack

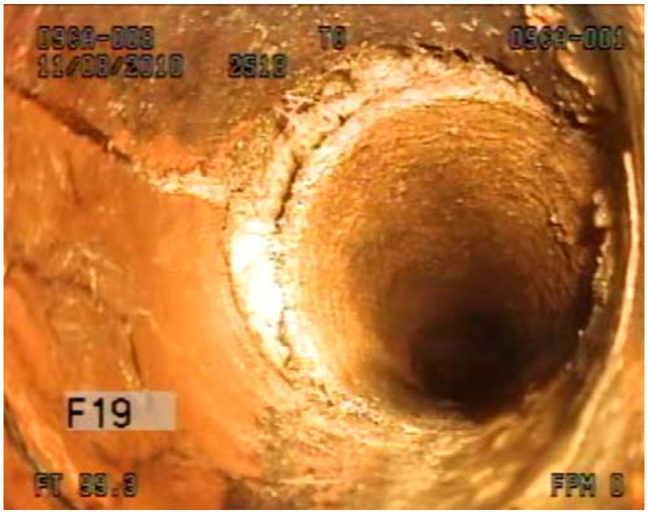


Pipe# 2977 Crack

Pipe# 2977 Flow high



Pipe# 2518 Misaligned bell



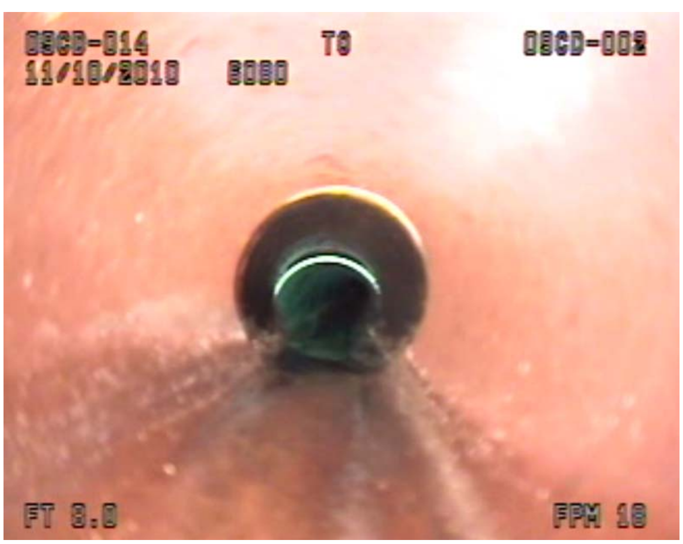
Pipe# 2518 Crack in service

Pipe# 2518 Offset service

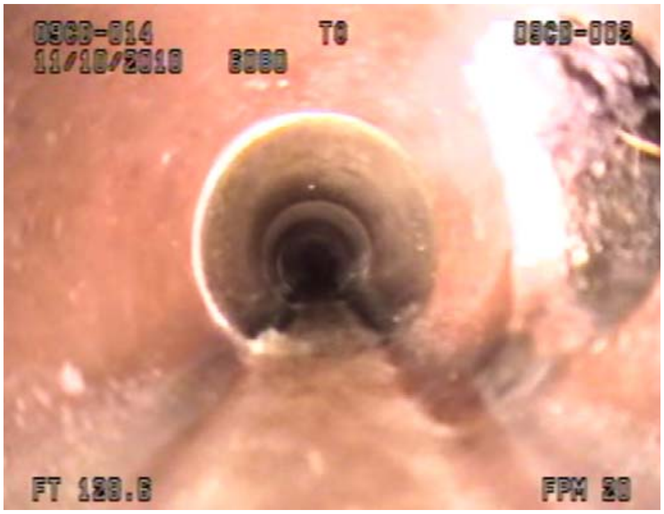
Pipe# 2518 Roots in services

2518 Clay to pvc misaligned

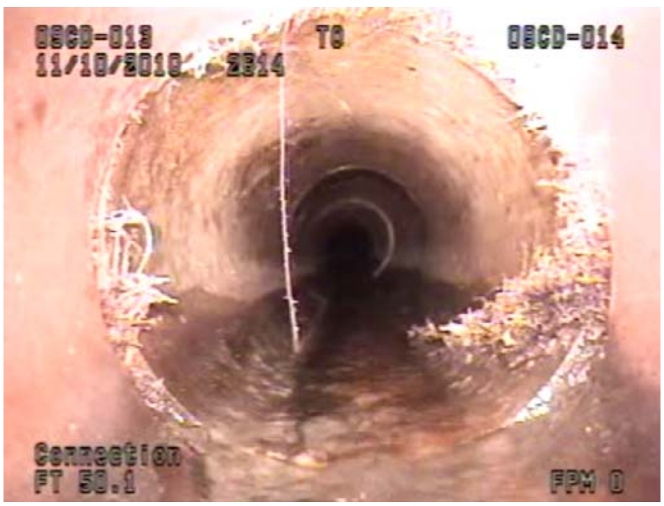
Pipe# 6080 Misaligned pipe



Pipe # 6080 Misaligned pipe



Pipe# 6080 Roots service



Pipe# 2614 Roots

Pipe# 2614 Roots in bell

APPENDIX C
FLOW DATA

- WWTP INFLUENT & PRECIPITATION DATA
- WATER USAGE ANALYSIS
- DESIGN FLOW METHOD
- SUMMARY OF FLOW MONITORING DATA

KELLER associates

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WWTP Influent & Precipitation Data

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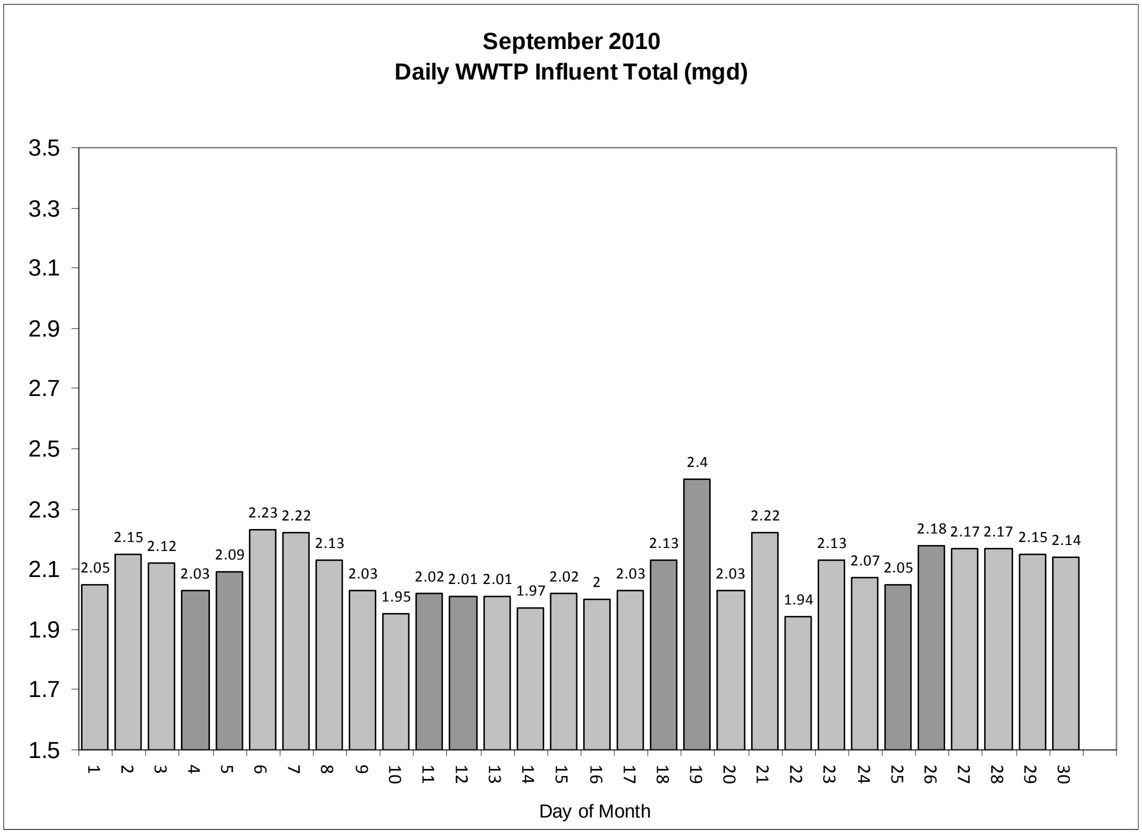
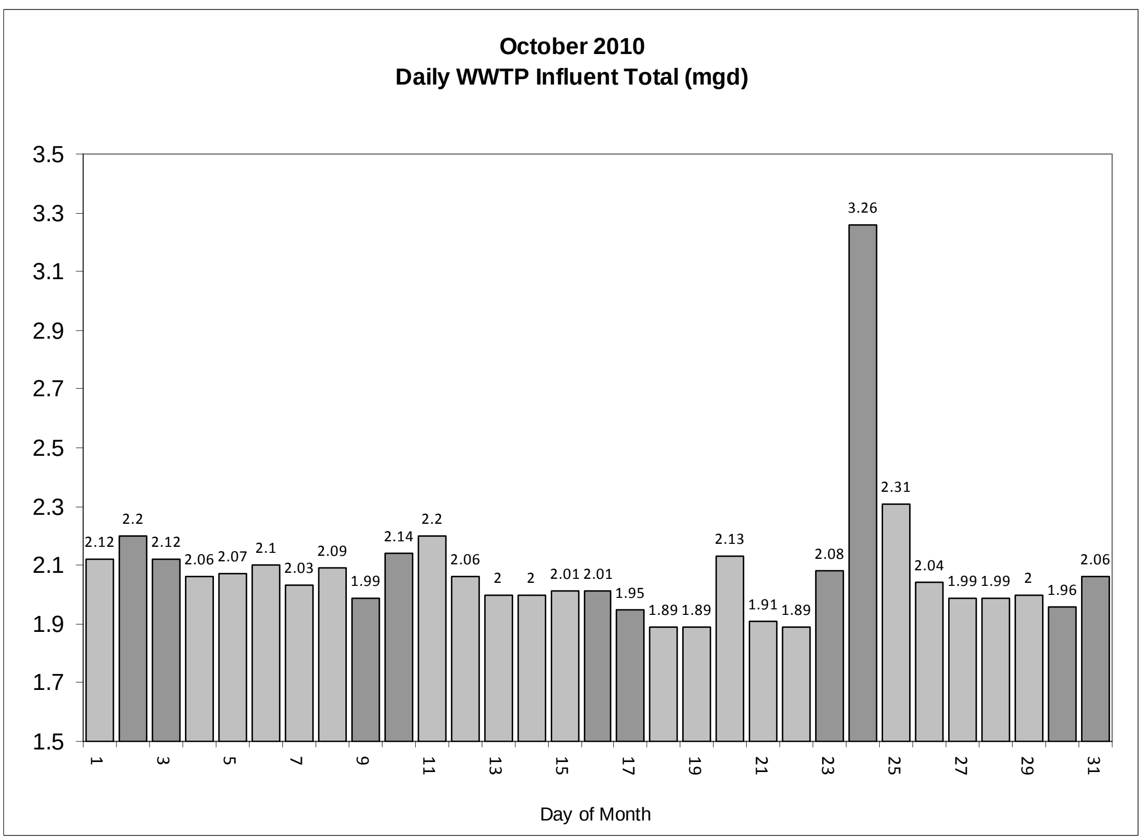
Ashland, OR
WWTP Influent Flows
during Flow Monitoring Period

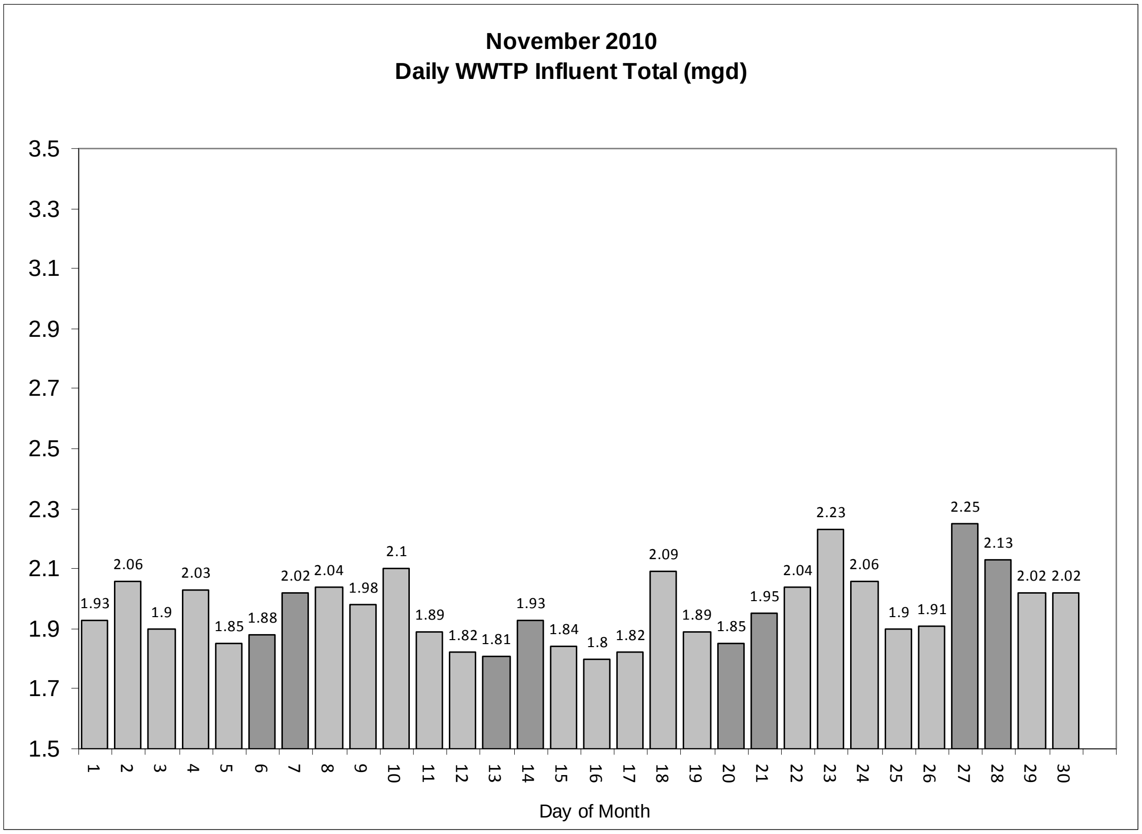
Day	Sep-10	Oct-10	Nov-10	Dec-10	Jan-11
	MGD	MGD	MGD	MGD	MGD
1	2.05	2.12	1.93	2.03	2.20
2	2.15	2.2	2.06	2.84	2.25
3	2.12	2.12	1.9	2.35	2.25
4	2.03	2.06	2.03	2.08	2.18
5	2.09	2.07	1.85	2.05	2.10
6	2.23	2.1	1.88	2.02	2.10
7	2.22	2.03	2.02	1.94	2.03
8	2.13	2.09	2.04	1.98	2.08
9	2.03	1.99	1.98	1.98	2.05
10	1.95	2.14	2.1	1.97	2.02
11	2.02	2.2	1.89	2.02	2.02
12	2.01	2.06	1.82	1.95	2.03
13	2.01	2	1.81	2.12	2.53
14	1.97	2	1.93	2.48	2.27
15	2.02	2.01	1.84	2.23	2.30
16	2	2.01	1.8	2.12	3.49
17	2.03	1.95	1.82	2.2	2.95
18	2.13	1.89	2.09	2.56	2.62
19	2.4	1.89	1.89	2.64	2.44
20	2.03	2.13	1.85	2.43	2.28
21	2.22	1.91	1.95	2.22	2.18
22	1.94	1.89	2.04	2.23	2.15
23	2.13	2.08	2.23	2.09	2.12
24	2.07	3.26	2.06	2.03	2.07
25	2.05	2.31	1.9	1.96	2.09
26	2.18	2.04	1.91	2.23	2.06
27	2.17	1.99	2.25	2.2	1.99
28	2.17	1.99	2.13	2.74	2.03
29	2.15	2	2.02	2.84	2.01
30	2.14	1.96	2.02	2.53	2.02
31		2.06		2.43	1.93

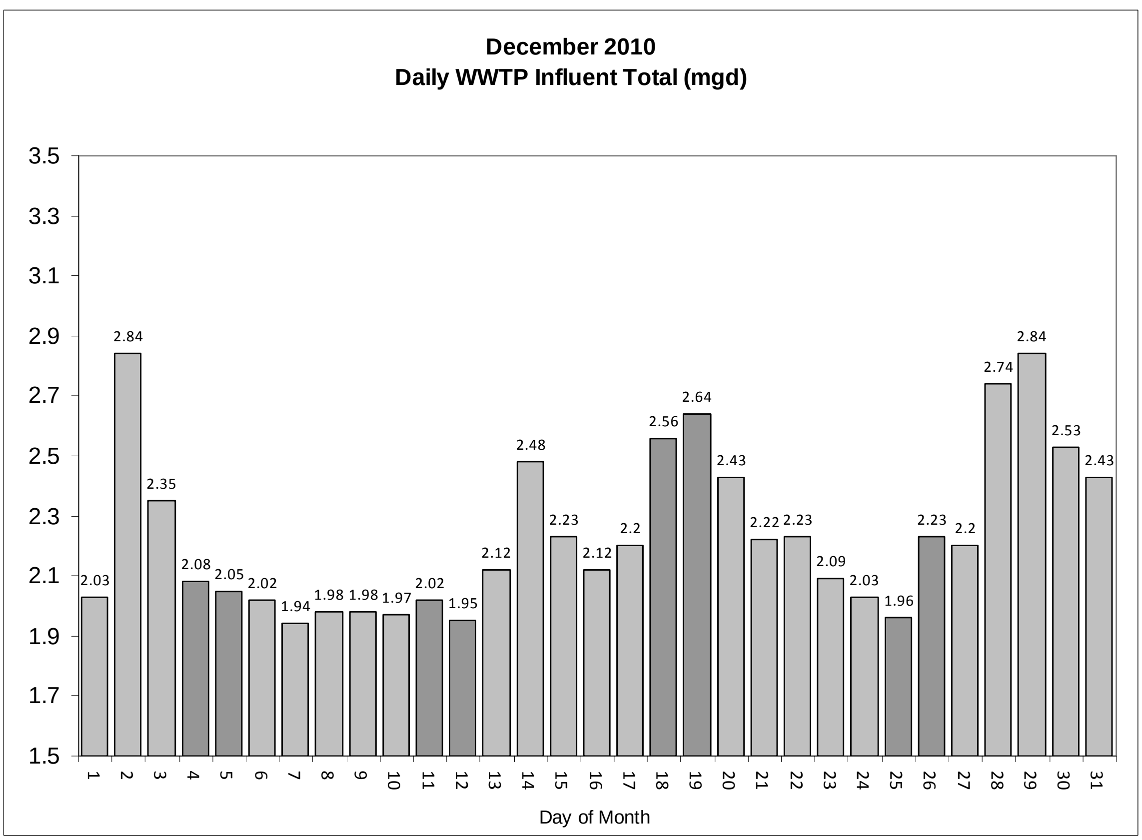
Ashland, OR
WWTP Influent Flows
during Flow Monitoring Period



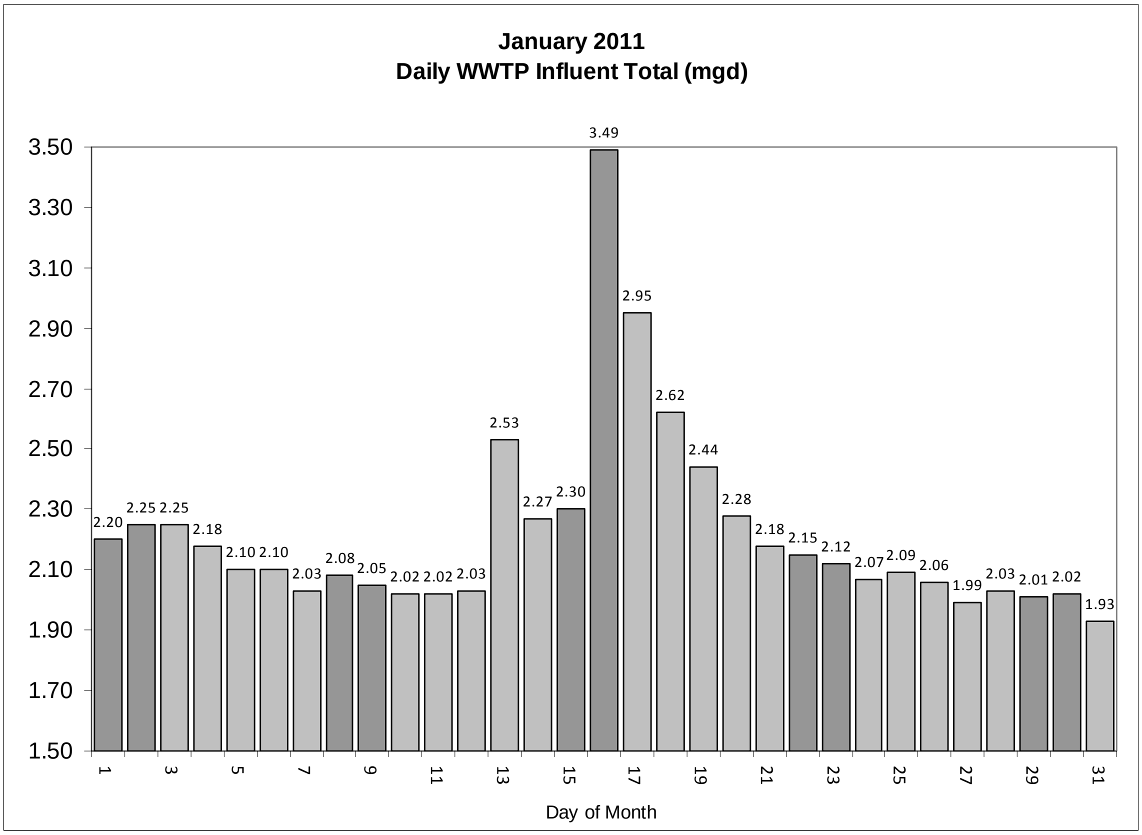


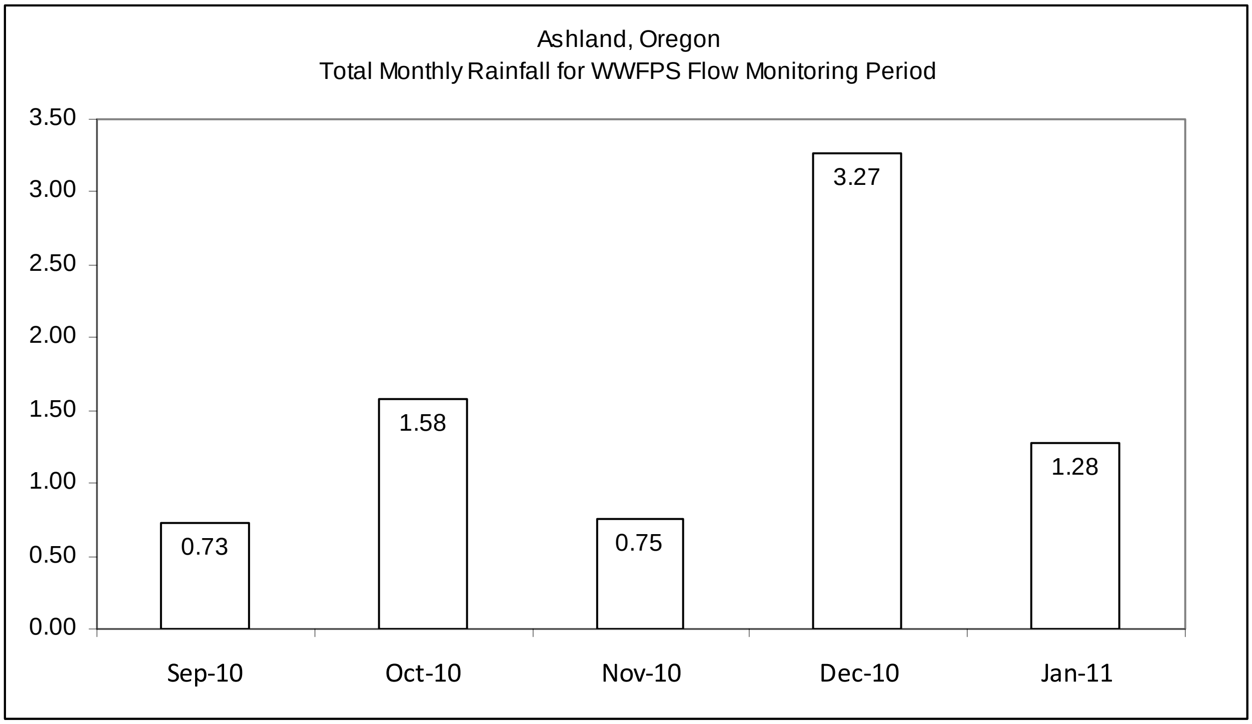
Ashland, OR
WWTP Influent Flows
during Flow Monitoring Period



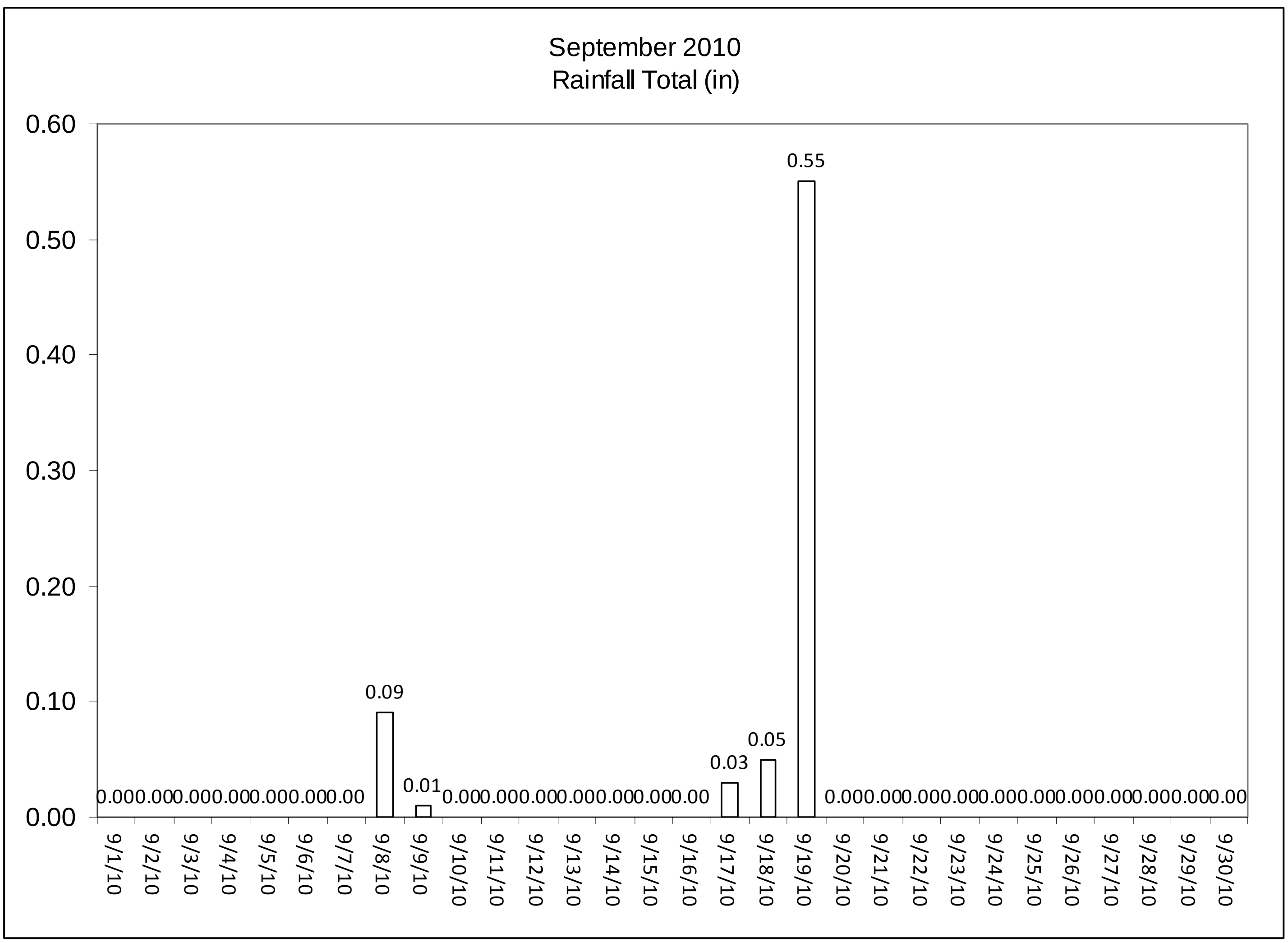


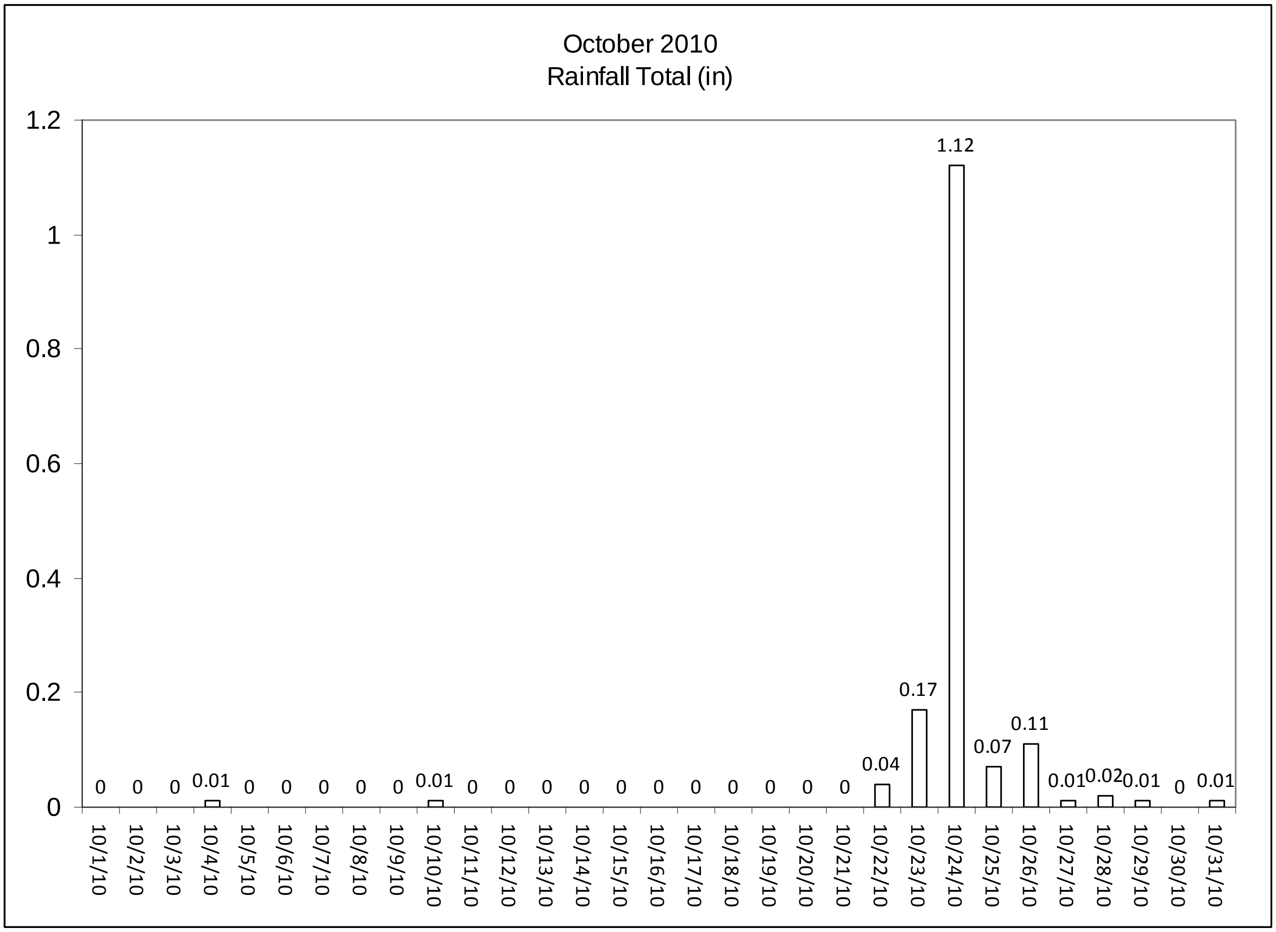
Ashland, OR
WWTP Influent Flows
during Flow Monitoring Period



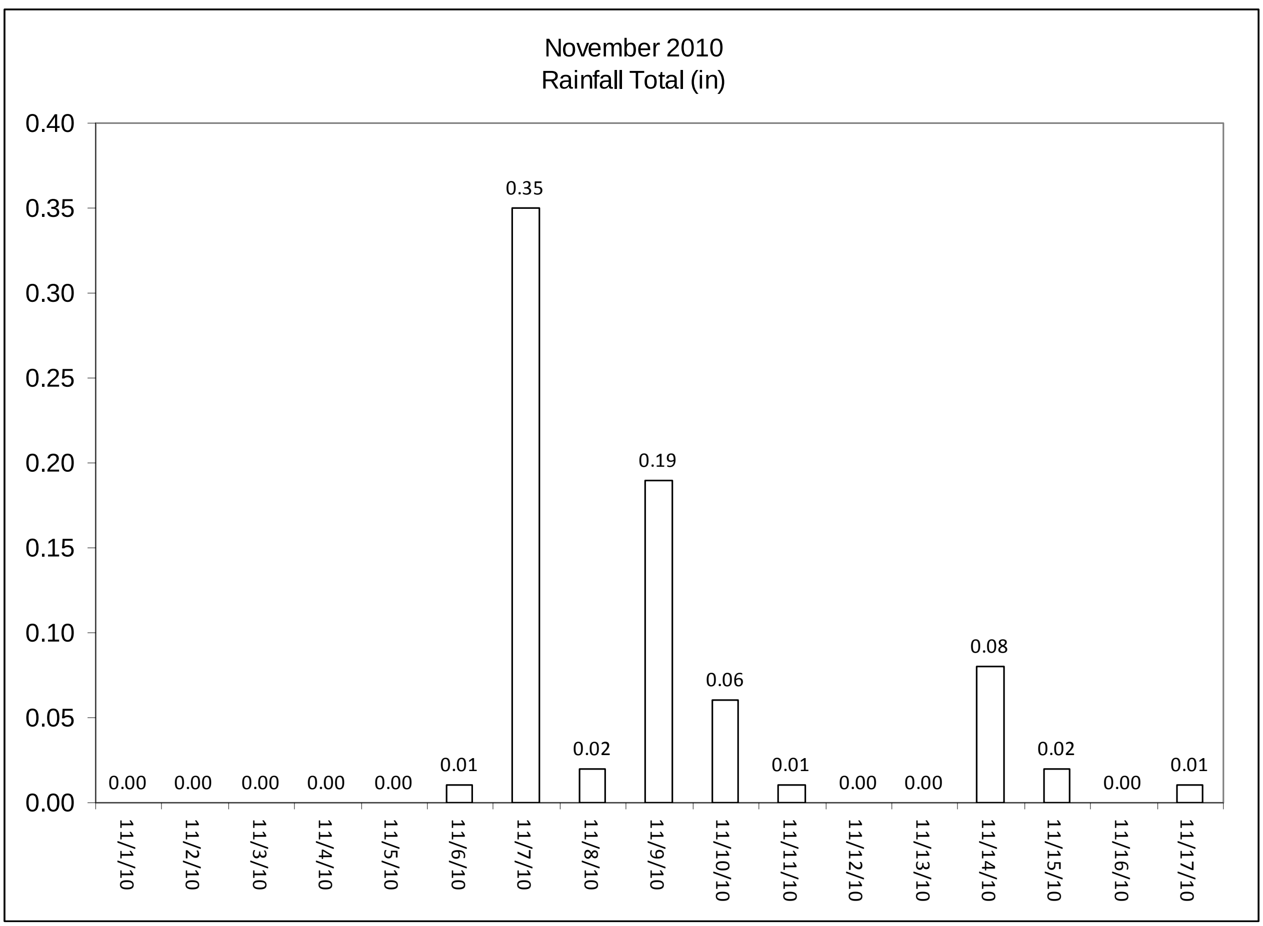


Note: November data only covers Nov 1-17

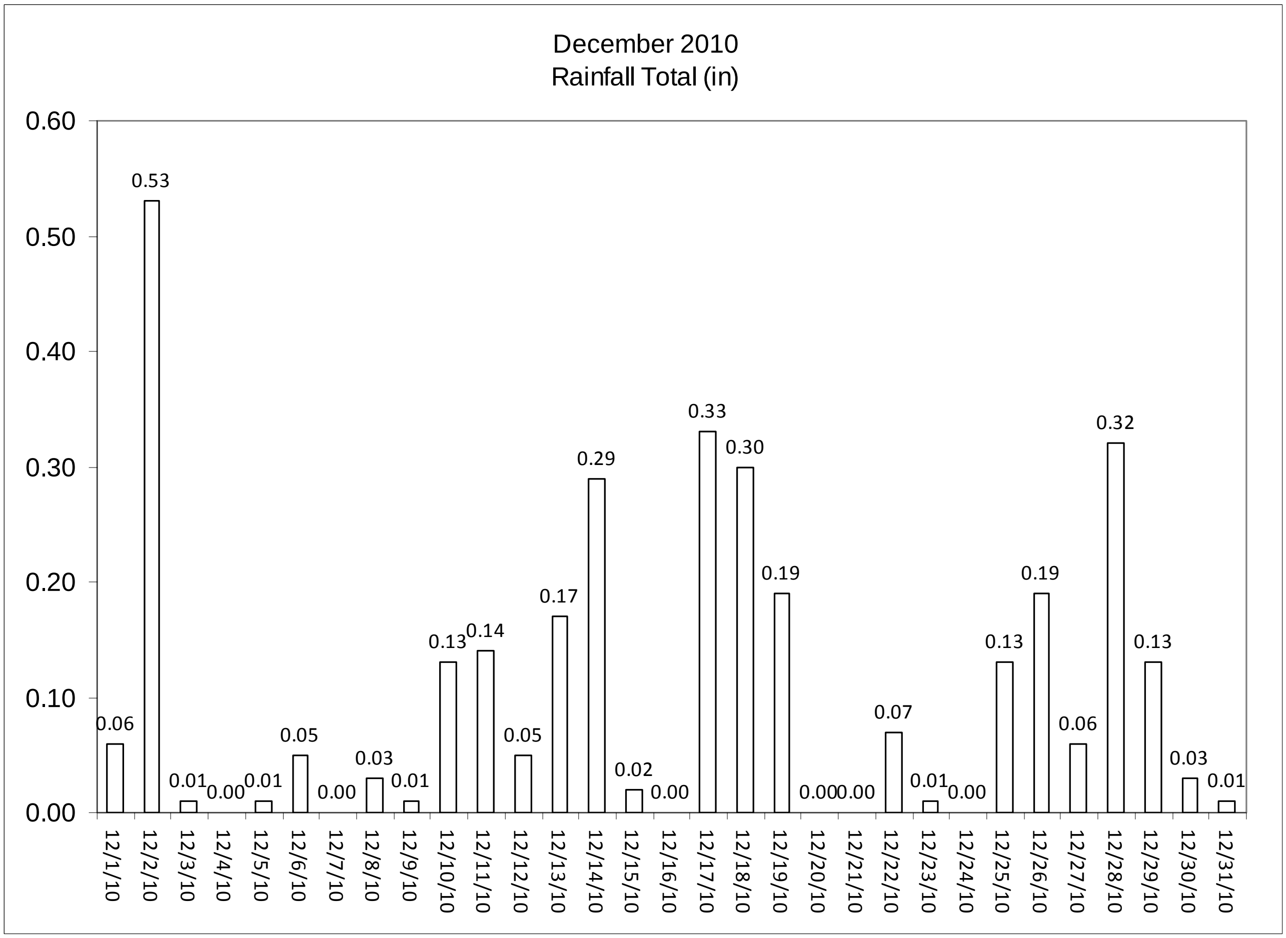




October 2010
Rainfall Total (in)
1.2
1
0.8
0.6
0.4
0.2
0
0 0 0 0.01 0 0 0 0 0 0.01 0 0 0 0 0 0 0 0 0 0 0 0.04 0.17 1.12 0.07 0.11 0.01 0.02 0.01 0 0.01
10/1/10 10/2/10 10/3/10 10/4/10 10/5/10 10/6/10 10/7/10 10/8/10 10/9/10 10/10/10 10/11/10 10/12/10 10/13/10 10/14/10 10/15/10 10/16/10 10/17/10 10/18/10 10/19/10 10/20/10 10/21/10 10/22/10 10/23/10 10/24/10 10/25/10 10/26/10 10/27/10 10/28/10 10/29/10 10/30/10 10/31/10



November 2010
Rainfall Total (in)
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0.00 0.00 0.00 0.00 0.00 0.01 0.35 0.02 0.19 0.06 0.01 0.00 0.00 0.08 0.02 0.00 0.01
11/1/10 11/2/10 11/3/10 11/4/10 11/5/10 11/6/10 11/7/10 11/8/10 11/9/10 11/10/10 11/11/10 11/12/10 11/13/10 11/14/10 11/15/10 11/16/10 11/17/10


December 2010
Rainfall Total (in)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.06
0.53
0.01
0.00
0.01
0.05
0.00
0.03
0.01
0.13
0.14
0.05
0.17
0.29
0.02
0.00
0.33
0.30
0.19
0.00
0.00
0.07
0.01
0.00
0.13
0.19
0.06
0.32
0.13
0.03
0.01
12/1/10
12/2/10
12/3/10
12/4/10
12/5/10
12/6/10
12/7/10
12/8/10
12/9/10
12/10/10
12/11/10
12/12/10
12/13/10
12/14/10
12/15/10
12/16/10
12/17/10
12/18/10
12/19/10
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12/21/10
12/22/10
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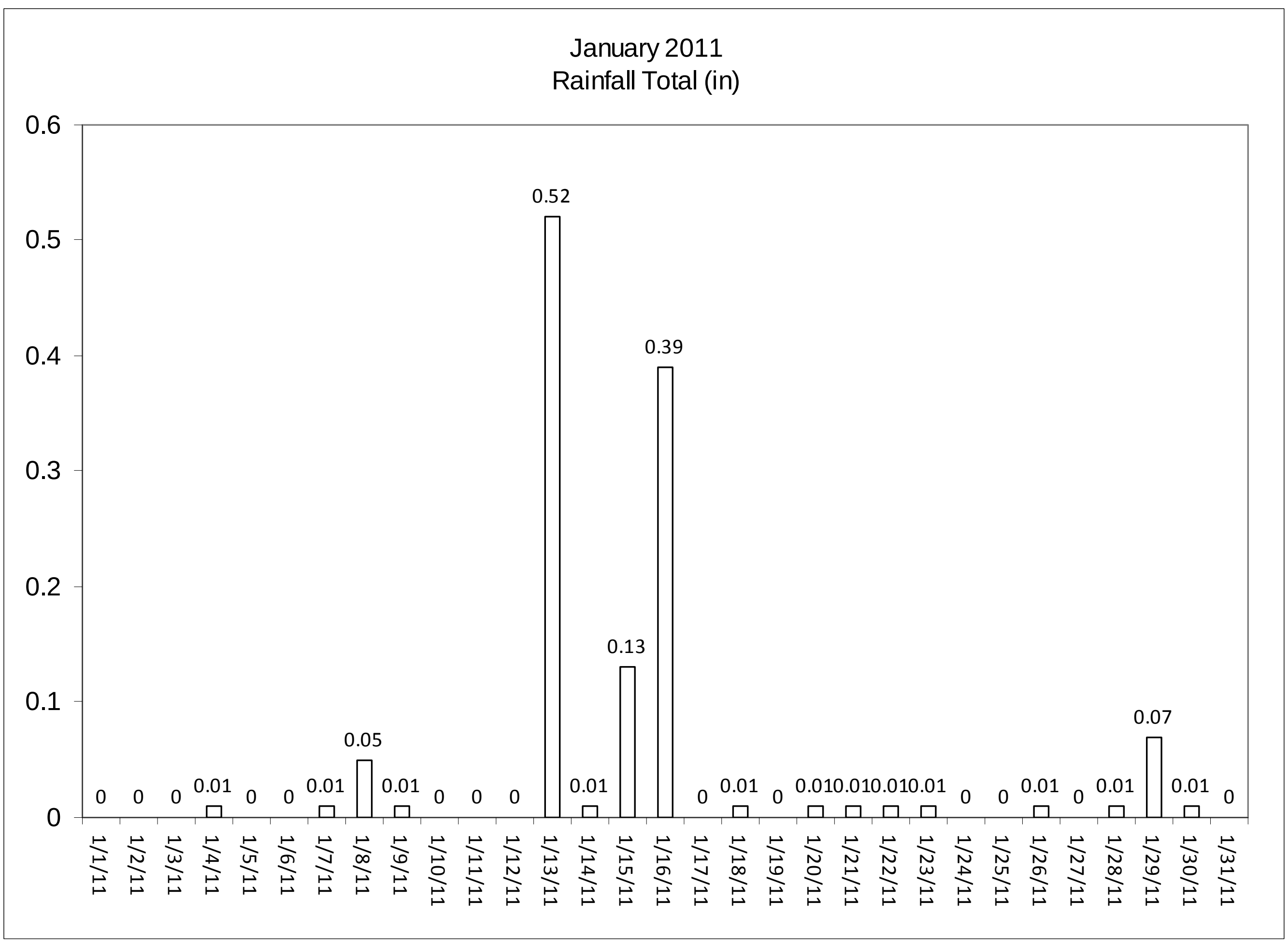
12/28/10
12/29/10
12/30/10
12/31/10


January 2011
Rainfall Total (in)
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0
0
0.01
0
0
0.01
0.05
0.01
0
0
0
0.52
0.01
0.13
0.39
0
0.01
0
0.01
0.01
0.01
0.01
0
0
0.01
0
0.01
0.07
0.01
0
1/1/11
1/2/11
1/3/11
1/4/11
1/5/11
1/6/11
1/7/11
1/8/11
1/9/11
1/10/11
1/11/11
1/12/11
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1/30/11
1/31/11

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Water Usage Analysis

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SEWER **2009-2010**

Updated 7/9/10 Months in report 0

	COMMERCIAL,GOVERNMENTAL, MUNICIPAL							MULTI-FAMILY RESIDENTIAL						RESIDENTIAL				
09-10	Customer Count	Usage (CF)	Revenue Usage	Revenue Non-Usage	Average Usage	Average Cost		Customer Count	Usage Calc	Revenue	Average Cost/Cust	Average Cost/Unit		Customer Count	Usage	Revenue	Average Cost	
July	614	2,702,899	75,716.94	483.08	4,402	124.10		578	2,695,957	61,084	105.68	105.68		7,049	10,852,415	142,680	20.24	
Aug	608	3,143,691	84,368.89	483.08	5,171	139.56		574	2,968,157	60,999	106.27	106.27		6,942	13,001,197	142,405	20.51	
Sept	611	2,969,185	80,542.24	483.08	4,860	132.61		580	2,570,946	61,228	105.57	105.57		7,026	10,258,833	142,637	20.30	
Oct	614	2,370,509	70,000.48	483.08	3,861	114.79		581	1,886,297	59,494	102.40	102.40		7,017	6,679,007	142,485	20.31	
Nov	615	1,858,890	59,935.07	483.08	3,023	98.24		579	1,327,719	59,404	102.60	102.60		6,945	3,795,588	141,320	20.35	
Dec	614	1,574,007	53,766.10	483.08	2,564	88.35		582	1,395,751	59,446	102.14	102.14		6,956	2,803,138	141,649	20.36	
Jan	617	1,362,066	48,046.46	483.08	2,208	78.65		578	1,346,141	59,308	102.61	102.61		6,933	3,545,491	141,004	20.34	
Feb	614	1,380,237	48,359.62	483.08	2,248	79.55		578	1,265,174	59,316	102.62	102.62		6,955	3,120,695	141,329	20.32	
Mar	617	1,387,954	48,929.78	395.46	2,250	79.94		581	1,232,495	59,341	102.14	102.14		6,970	3,034,853	140,928	20.22	
April	616	1,463,203	50,727.70	395.46	2,375	82.99		579	1,245,276	57,780	99.79	99.79		6,855	3,328,192	137,151	20.01	
May	615	1,496,072	51,175.47	335.19	2,433	83.76		576	1,251,092	57,859	100.45	100.45		6,945	3,519,916	141,119	20.32	
June	618	1,926,288	62,621.39	336.51	3,117	101.87		586	1,592,357	58,334	99.55	99.55		7,036	4,960,779	136,707	19.43	
	7,373	23,635,001	734,190.14	5,327.26	38,509	100.30	0	6,952	20,777,362	713,593	102.65	#DIV/0!	0	83,629	68,900,104	1,691,414	20.23	0

Average December - February

	# Accounts	Usage (CF)	% of Total	Usage/ Account
Commercial	615	1,438,770	24.3%	2,339
Multi-Family	579	1,335,689	22.5%	2,306
Residential	6,948	3,156,441	53.2%	454
Total	8,142	5,930,900	100%	

TWO YEAR AVERAGE

	% of Total
Commercial	**24%**
Residential / Multifamily	**76%**

#

SEWER **2008-2009**

Updated 7/7/09 Months in report 0

	COMMERCIAL,GOVERNMENTAL, MUNICIPAL							MULTI-FAMILY RESIDENTIAL						RESIDENTIAL				
08-09	Customer Count	Usage	Revenue Usage	Revenue Non-Usage	Average Usage	Average Cost		Customer Count	Usage Calc	Revenue	Average Cost/Cust	Average Cost/Unit		Customer Count	Usage	Revenue	Average Cost	
July	603	3,035,304	67,589.31	520.85	5,034	112.95		576	2,635,063	48,875	84.85	#DIV/0!		6,974	11,125,631	116,185	16.66	
Aug	521	5,264,310	66,741.62	526.48	10,114	129.24		844	4,376,407	48,212	57.12	#DIV/0!		11,924	11,000,000	109,656	9.20	
Sept	777	3,386,138	72,035.89	537.05	4,358	93.40		879	3,925,724	50,173	57.08	#DIV/0!		8,822	11,454,250	119,635	13.56	
Oct	610	2,548,899	60,757.05	537.32	4,179	100.48		579	2,190,367	50,242	86.77	#DIV/0!		6,942	8,004,880	119,904	17.27	
Nov	610	2,074,089	53,059.10	537.32	3,400	87.86		576	1,641,464	50,375	87.46	#DIV/0!		6,914	4,922,481	119,406	17.27	
Dec	608	1,598,922	45,792.19	537.32	2,630	76.20		576	1,472,013	50,366	87.44	#DIV/0!		6,939	3,600,939	119,323	17.20	
Jan	616	1,507,258	44,094.05	537.32	2,447	72.45		575	1,462,765	50,341	87.55	#DIV/0!		6,979	3,630,021	119,747	17.16	
Feb	621	1,381,777	40,879.97	537.32	2,225	66.69		576	1,407,875	50,411	87.52	#DIV/0!		6,982	3,063,083	119,558	17.12	
Mar	615	1,417,380	41,000.40	529.38	2,305	67.53		579	1,388,756	50,371	87.00	#DIV/0!		6,953	3,130,725	119,297	17.16	
April	608	1,471,199	41,998.93	452.19	2,420	69.82		577	1,427,557	50,677	87.83	#DIV/0!		7,007	3,567,275	117,879	16.82	
May	611	1,704,378	52,279.77	431.85	2,789	86.27		574	1,557,014	55,714	97.06	#DIV/0!		6,997	4,726,582	127,948	18.29	
June	610	2,264,819	66,665.45	483.08	3,713	110.08		574	2,048,589	61,103	106.45	#DIV/0!		7,089	7,853,098	141,941	20.02	
	7,410	27,654,473	652,893.73	6,167.48	45,613	88.95	0	7,485	25,533,594	616,860	82.41	#DIV/0!	0	90,522		1,450,479	16.02	0

Average December - February

	# Accounts	Usage (CF)	% of Total	Usage/ Account
Commercial	615	1,495,986	23.5%	2,432
Multi-Family	576	1,447,551	22.7%	2,515
Residential	6,967	3,431,348	53.8%	493
Total	8,157	6,374,884	100%	

DESIGN FLOW METHOD

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CURRENT DESIGN FLOWS

In calculating current design flows, Keller Associates used the Oregon DEQ method described in "Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon MMDWF, MMWWF, PDAF, and PDIF." Design Flows were calculated from daily total WWTP flows and precipitation from 2005-2009. Brief discussions on how this method was applied for each parameter are given below.

Average Dry-Weather Flow (ADWF)
ADWF is the average daily flow for the period of May – October. An ADWF was calculated for each year of data and the year 2007 result selected as the design flow to coincide with other design points selected for reasons discussed below in the MMDWF subsection.

Max Month Dry-Weather Flow (MMDWF10)
Oregon DEQ outlines that May is typically the max month for the dry-weather period of May – October. The DEQ method for calculating MMDWF is to graph the Jan-May average daily flows of the most recent year against total precipitation for the month. A trend line is fitted to the data and then MMDWF read from the trend line at a precipitation equal to the May 90% precipitation probability value (3.16) published in "Climatology of the United States No. 20, 1971-2000" (CLIM20).

When graphs of the Jan-May values for each year of data were compared it was observed that the most recent year (2009) was the driest (lowest flows) and had the lowest correlation factor. Years 2007 and 2006 were the wettest (highest flows) and the year 2007 trend line was nearest the trend line of all data points 2005-2009 combined (see Figure C-1). Therefore, year 2007 was selected as the most representative year for the data available.

Average Wet-Weather Flow (AWWF)
Calculations for AWWF are not outlined by DEQ; however AWWF was calculated as the average daily flow for the period of January – April for each year of data. The year 2007 result was selected as design flow to coincide with other design points selected for reasons discussed above in the MMDWF subsection.

Max Month Wet-Weather Flow (MMWWF5)
Oregon DEQ outlines that January is typically the max month for the wet-weather period of January – April. The DEQ method for calculating MMWWF is to enter the graph of Jan-May average daily flows vs. monthly precipitation and read MMWWF from the trend line at a precipitation equal to the January 80% precipitation probability value (3.72) published in CLIM20 (see Figure C-2).

Peak Daily Average Flow (PDAF5)
Oregon DEQ outlines that the PDAF typically corresponds to the 5-year storm event and, therefore, is calculated as the flow resulting from a 5-year storm during a period of high groundwater (Jan-April). The DEQ method for determining PDAF is to plot daily plant

flow against daily precipitation for large storm events over several years only using data during wet-weather seasons when ground water is high. A trend line is fitted to the points and then PDAF read from the trend line at the 5-year, 24-hour storm event level. For the Ashland calculation, storm events were selected based on a precipitation greater than 0.50 inches and WWTP flow greater than 2.0 MGD for the period Jan-April, 2005-2009 (see Figure C-3 and Table C-1). Several “fringe” storm events (4 total) from late December and early May were included because these events were part of multiple days of wet weather and/or corresponded to high flow events at the plant (>4.0 MGD). The 5-year, 24-hour storm value for Ashland was selected from NOAA isopluvial maps for the state (Atlas 2, Volume X, Figure 26) and equaled 2.5 inches.

Peak Week Flow (PWkF)
Calculations for Peak Week Flow are not outlined by DEQ, however, are useful in some design calculations. A 7-day average flow was calculated for every day using the 7 previous days of data (rolling average). PWkF was then calculated as the max of all weekly (7-day) rolling averages. Except for 2005, the PWkF always occurred during the Wet Weather Season and, typically, in January. The year 2007 result was selected as the design flow to coincide with other design points selected for reasons discussed above in the MMDWF subsection.

Peak Instantaneous Flow (PIF)
Oregon DEQ allows several options in calculating PIF. The first is to examine flow charts recorded during high-flow days, preferably during a 5-year storm event if available, and select the peaking factor applied to the AWWF to calculate the PIF. The second option is extrapolation utilizing the other Design Flow values. This includes plotting Annual Average Daily Flow (AADF), PDAF, and MMWWF on a logarithmic probability graph, plotting a trend line, and reading the PIF off the trend line at the given probability of a PIF event (0.011%). Utilizing this method for Ashland results in a PIF = 11.0 MGD (see Figure C-4). This is a relatively high value when compared to recent flow data and was initially considered unrepresentative. Therefore, the first method of selecting a peaking factor from available hourly data was also utilized.

Hourly SCADA data for the plant was analyzed for selected dates in 2008 & 2009. The dates, flows, peaking factors, and precipitation are listed in Table 4.4 in the Wastewater Master Plan. The dates were selected based on days with a correlation between peak WWTP flow and peak precipitation events as observed from graphs of daily flow & precipitation data (see Figures C-5 through C-9). Hourly SCADA data was not available prior to September 2007.

Upon reviewing the data, it was observed that a Peak Instantaneous flow of at least 10 MGD was sustained on January 4, 2008. With this information, and considering that a 5-year storm event was not experienced within the available data, the extrapolated value of 10.5 MGD was determined plausible and accepted as the PIF. Select days of hourly data is included in Figures C-10 through C-14.

PROJECTED DESIGN FLOWS

The calculated current design flows were projected forward utilizing population numbers developed in the Ashland Comprehensive Plan. Specifically, for Average and Max Month parameters the current flow was divided by the current estimated 2010 population to derive a gallon per capita per day (gpcd). The gpcd was then multiplied by the projected population in each year to yield the projected design flow. For Peak Week, Peak Day, and Peak Hour a slightly lower gpcd was applied to growth beyond the calculated 2010 values. This accounts for newer, more water tight components of new installations, and reduction of I/I through continuing reduction efforts by the City.

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Graph #1a

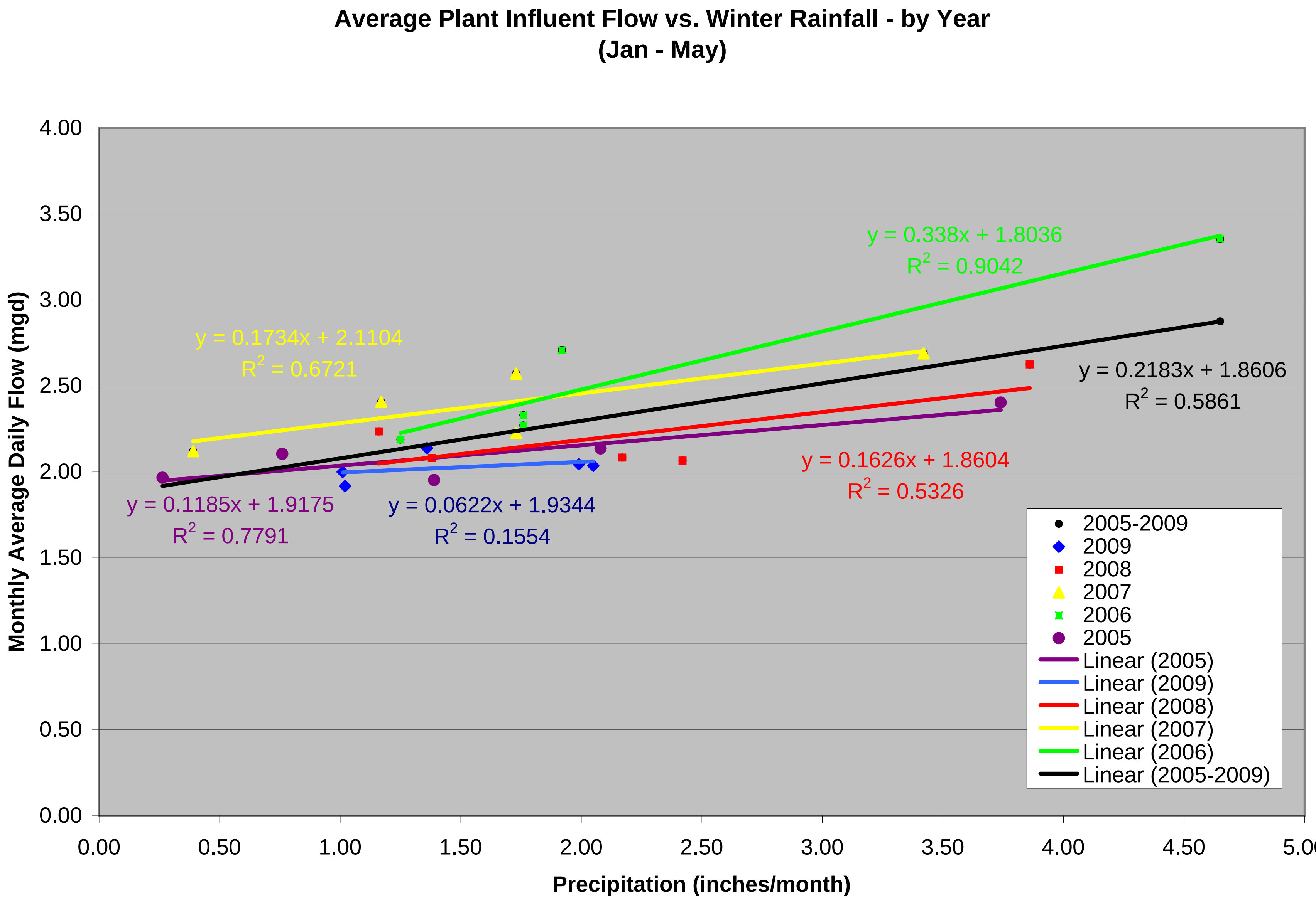


Figure C-1

P:\210055\Design\Spreadsheets\Flow Analysis 2004-2009_DMRwPrecip.xls
9/2/2010

Ashland WW

Table C-1
Monthly Precipitation vs. Flows

Graph #1a

	Precipitation	Flow
Mo/Yr	(in)	MGD
Jan-05	0.76	2.10
Feb-05	0.26	1.97
Mar-05	1.39	1.95
Apr-05	2.08	2.14
May-05	3.74	2.40
Jan-06	4.65	3.35
Feb-06	1.92	2.71
Mar-06	1.76	2.27
Apr-06	1.76	2.33
May-06	1.25	2.19
Jan-07	1.73	2.57
Feb-07	3.42	2.69
Mar-07	1.17	2.41
Apr-07	1.73	2.23
May-07	0.39	2.12
Jan-08	3.86	2.62
Feb-08	1.16	2.24
Mar-08	2.42	2.07
Apr-08	1.38	2.08
May-08	2.17	2.08
Jan-09	1.99	2.04
Feb-09	1.02	1.92
Mar-09	2.05	2.04
Apr-09	1.01	2.00
May-09	1.36	2.14

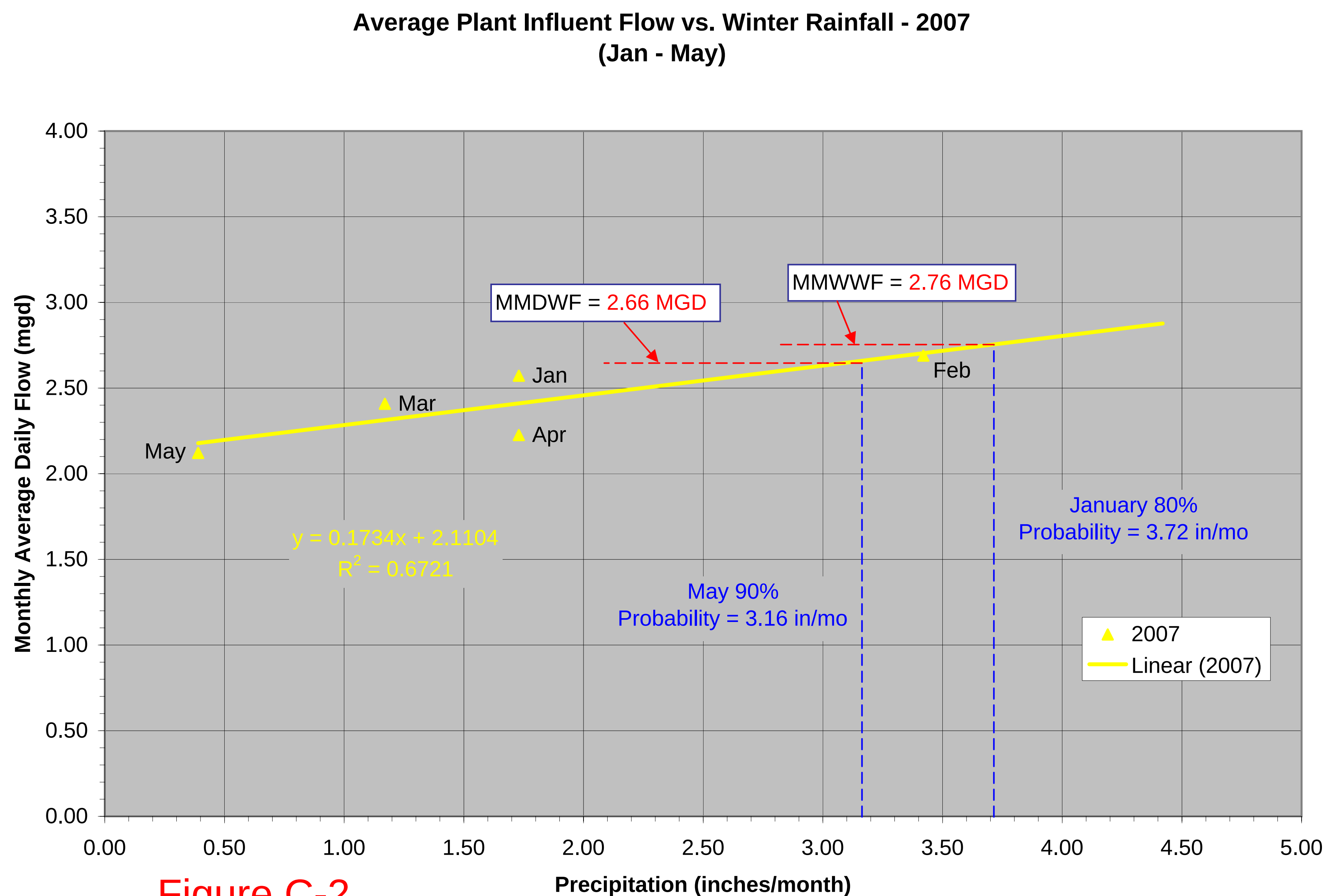

Average Plant Influent Flow vs. Winter Rainfall - 2007
(Jan - May)
Monthly Average Daily Flow (mgd)
Precipitation (inches/month)
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
MMDWF = 2.66 MGD
MMWWF = 2.76 MGD
Jan
Feb
Mar
Apr
May
y = 0.1734x + 2.1104
R2 = 0.6721
May 90%
Probability = 3.16 in/mo
January 80%
Probability = 3.72 in/mo
2007
Linear (2007)

Figure C-2

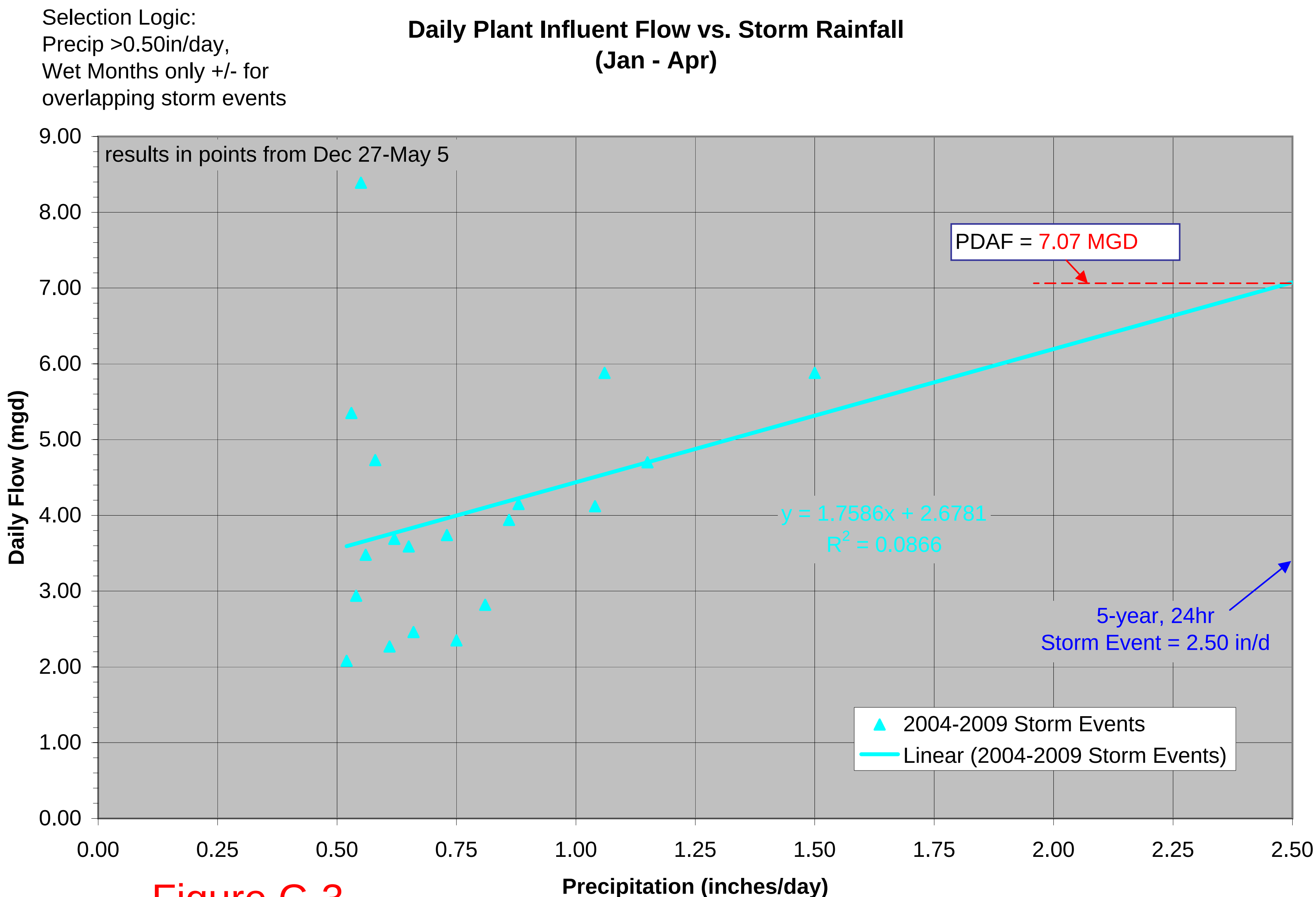

Selection Logic:
Precip >0.50in/day,
Wet Months only +/- for
overlapping storm events
Daily Plant Influent Flow vs. Storm Rainfall
(Jan - Apr)
results in points from Dec 27-May 5
PDAF = 7.07 MGD
y = 1.7586x + 2.6781
R2 = 0.0866
5-year, 24hr
Storm Event = 2.50 in/d
2004-2009 Storm Events
Linear (2004-2009 Storm Events)
Daily Flow (mgd)
Precipitation (inches/day)
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50

Figure C-3

Table C-2
Peak Precipitation/Flow Events

Precip >0.50 in Wet Months	Precipitation	Flow
Mo/Yr	(in)	MGD
January 1, 2004	0.65	3.59
February 17, 2004	1.15	4.70
February 18, 2004	0.62	3.69
April 28, 2005	0.81	2.82
May 5, 2005	0.66	2.46
December 30, 2005	0.55	8.39
December 31, 2005	1.50	5.88
January 1, 2006	0.53	5.35
January 21, 2006	0.56	3.48
February 1, 2006	0.58	4.73
December 27, 2006	1.04	4.12
January 4, 2007	0.88	4.15
February 22, 2007	0.86	3.94
January 4, 2008	1.06	5.88
January 5, 2008	0.73	3.74
January 28, 2008	0.75	2.35
January 2, 2009	0.54	2.94
January 25, 2009	0.61	2.27
April 10, 2009	0.52	2.08

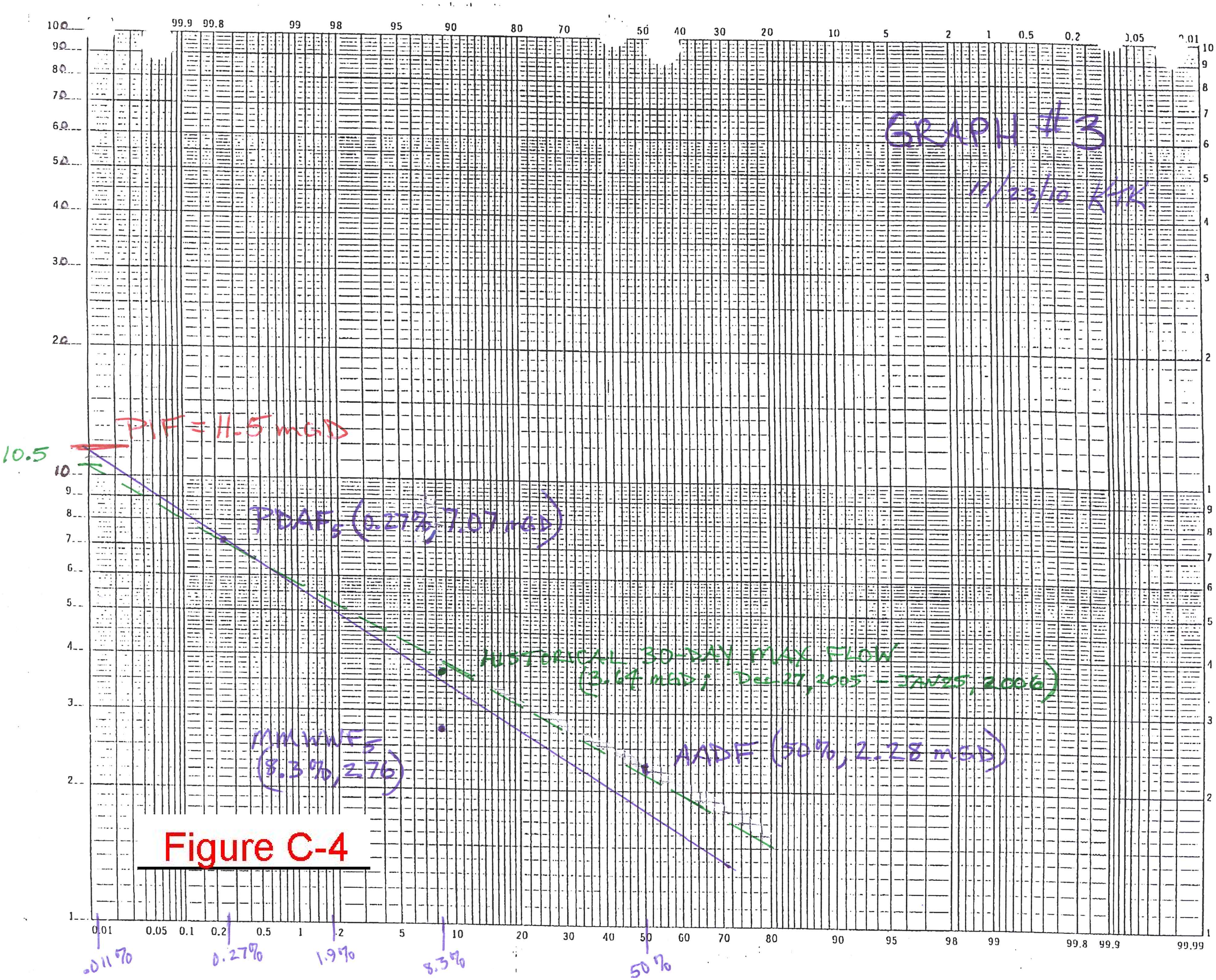

GRAPH #3
11/23/10 KTR
PIF = 11.5 mGD
10.5
PDAF5 (0.27%, 7.07 mGD)
HISTORICAL 30-DAY MAX FLOW
(3.64 mGD; Dec 27, 2005 - JAN 25, 2006)
MMWWF5
(8.3%, 2.76)
AADF (50%, 2.28 mGD)
.011%
0.27%
1.9%
8.3%
50%

Figure C-4

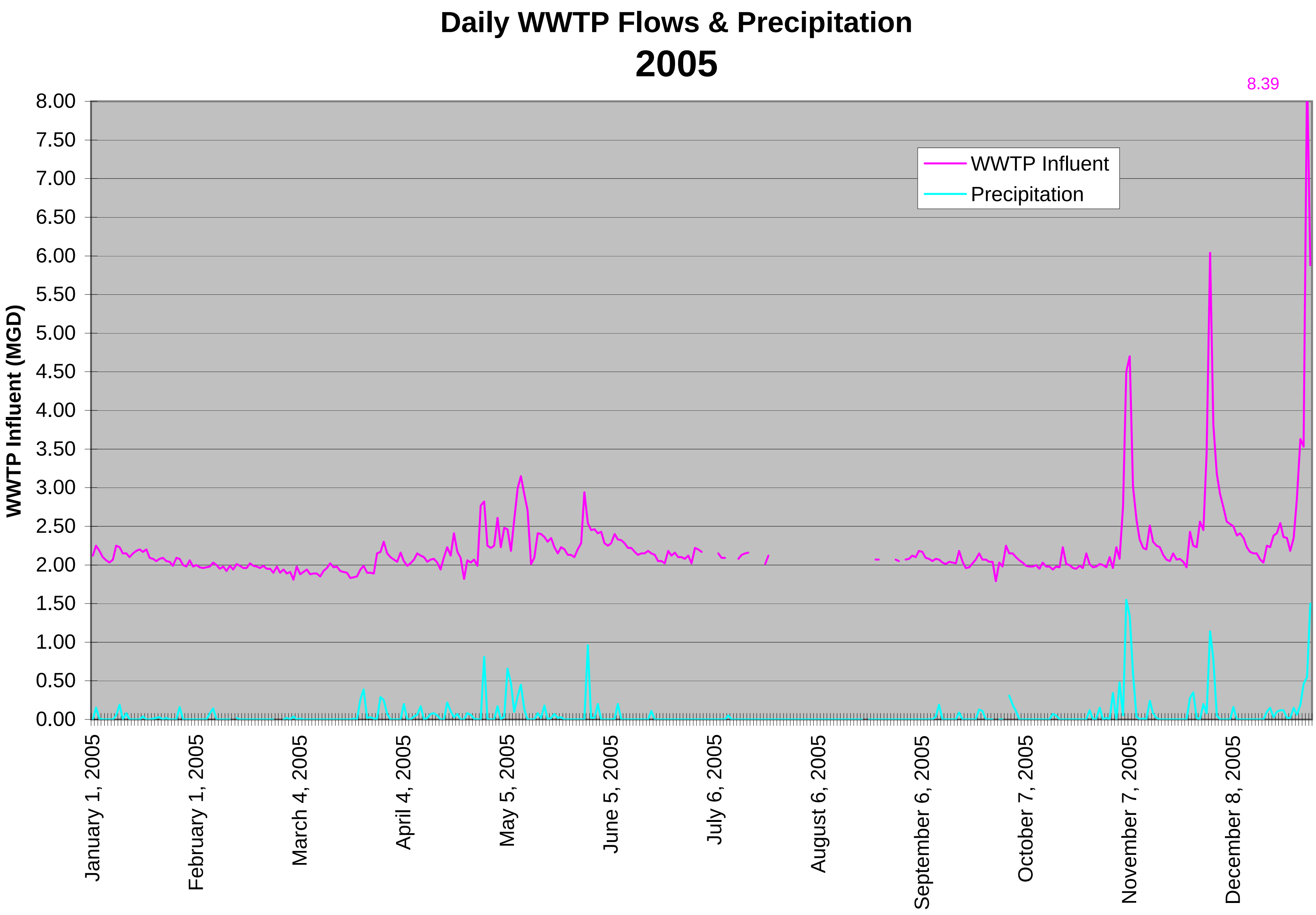
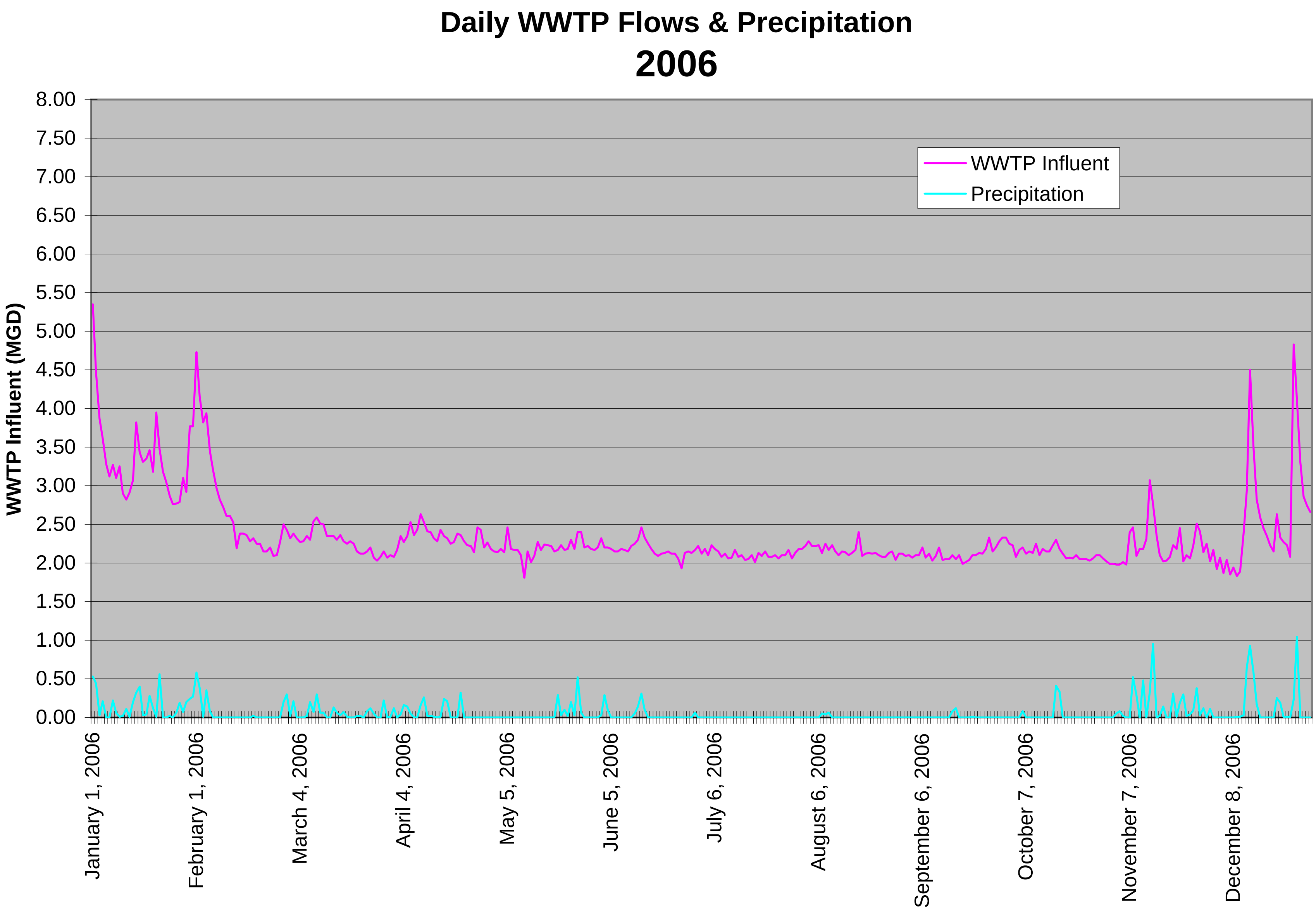

Daily WWTP Flows & Precipitation
2005
8.39
WWTP Influent
Precipitation
WWTP Influent (MGD)
8.00
7.50
7.00
6.50
6.00
5.50
5.00
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
January 1, 2005
February 1, 2005
March 4, 2005
April 4, 2005
May 5, 2005
June 5, 2005
July 6, 2005
August 6, 2005
September 6, 2005
October 7, 2005
November 7, 2005
December 8, 2005

Figure C-5



Daily WWTP Flows & Precipitation
2006
WWTP Influent
Precipitation
WWTP Influent (MGD)
8.00
7.50
7.00
6.50
6.00
5.50
5.00
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
January 1, 2006
February 1, 2006
March 4, 2006
April 4, 2006
May 5, 2006
June 5, 2006
July 6, 2006
August 6, 2006
September 6, 2006
October 7, 2006
November 7, 2006
December 8, 2006

Figure C-6

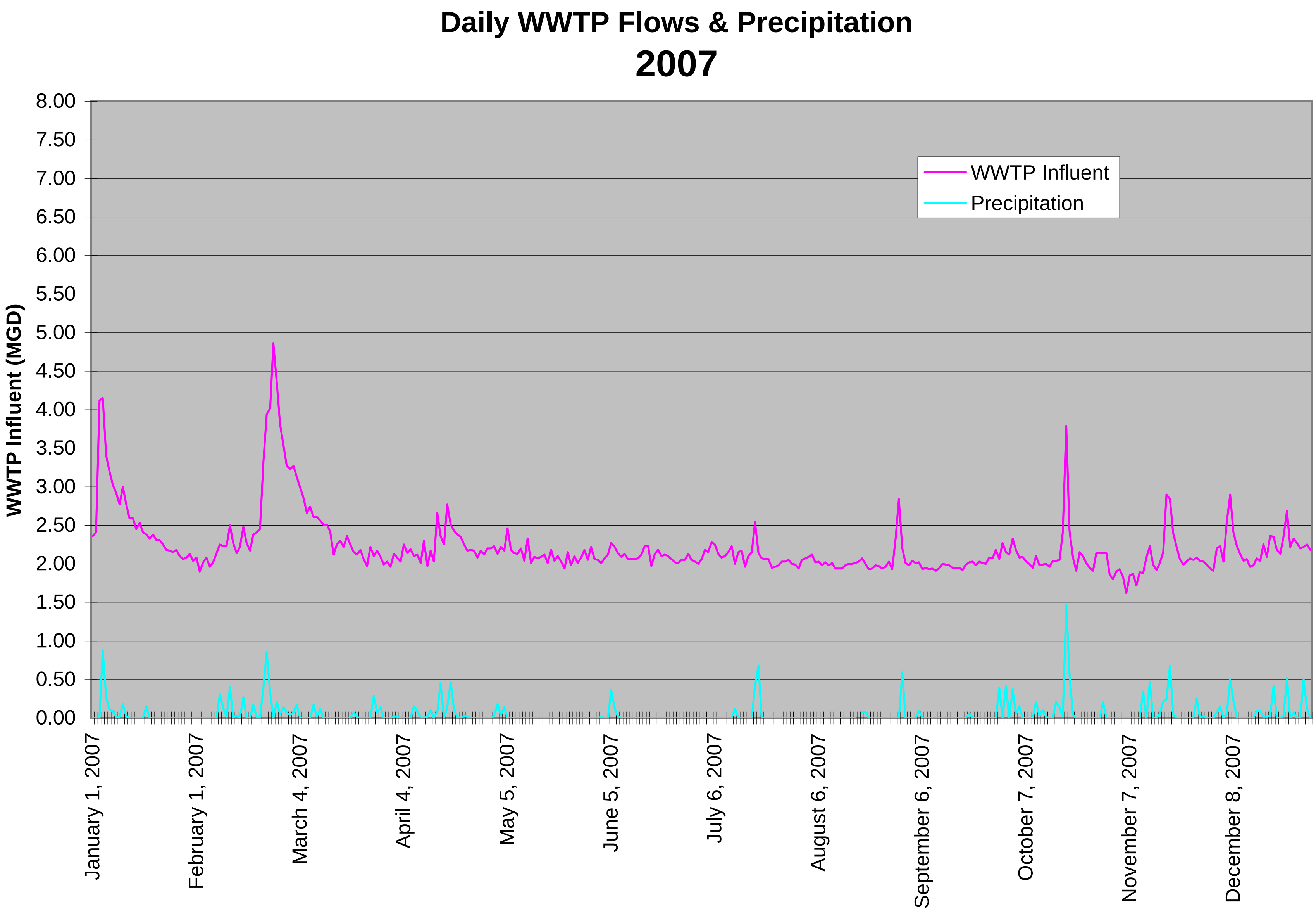

Daily WWTP Flows & Precipitation
2007
WWTP Influent (MGD)
8.00
7.50
7.00
6.50
6.00
5.50
5.00
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
WWTP Influent
Precipitation
January 1, 2007
February 1, 2007
March 4, 2007
April 4, 2007
May 5, 2007
June 5, 2007
July 6, 2007
August 6, 2007
September 6, 2007
October 7, 2007
November 7, 2007
December 8, 2007

Figure C-7

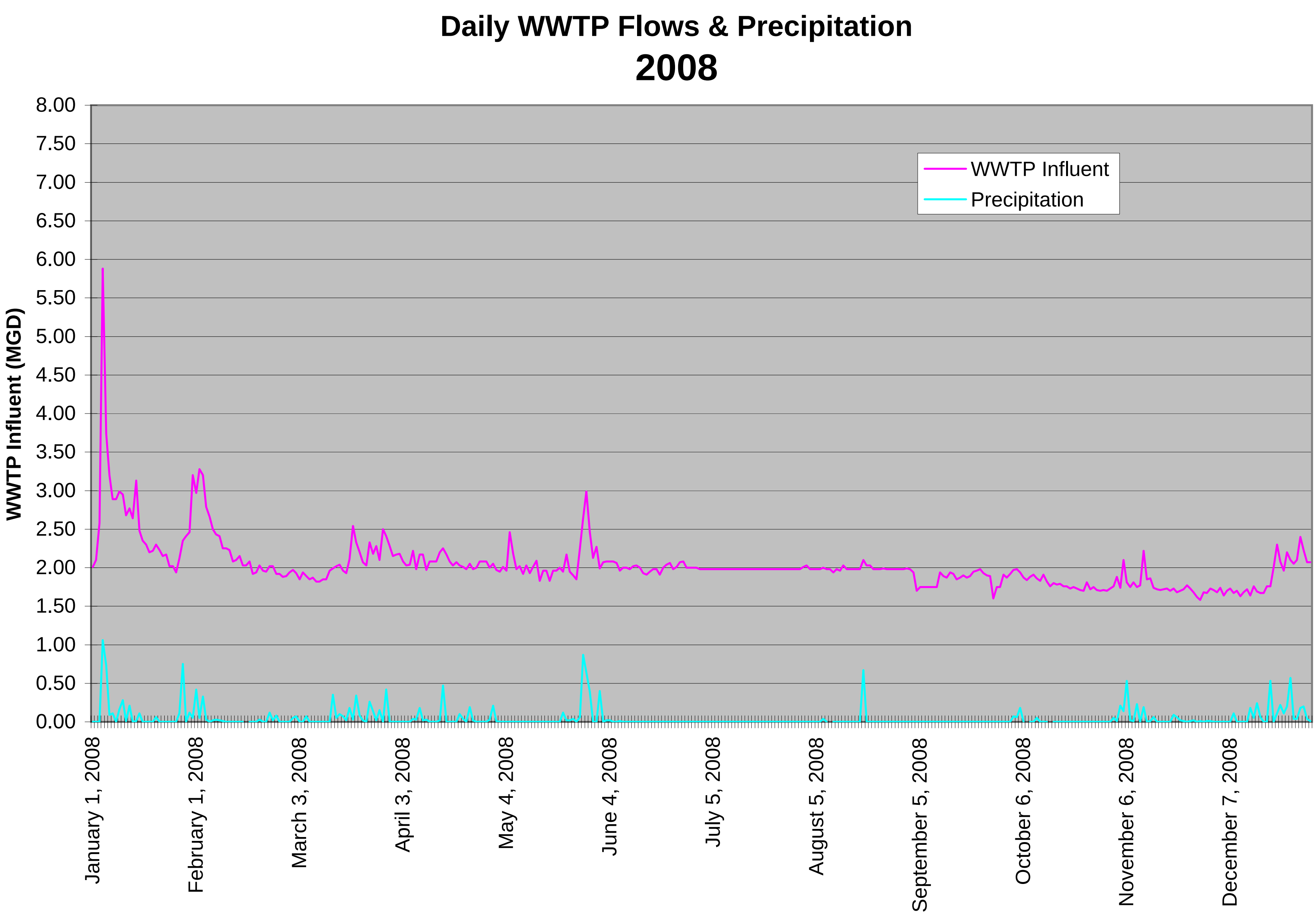

Daily WWTP Flows & Precipitation
2008
WWTP Influent (MGD)
WWTP Influent
Precipitation
8.00
7.50
7.00
6.50
6.00
5.50
5.00
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
January 1, 2008
February 1, 2008
March 3, 2008
April 3, 2008
May 4, 2008
June 4, 2008
July 5, 2008
August 5, 2008
September 5, 2008
October 6, 2008
November 6, 2008
December 7, 2008

Figure C-8

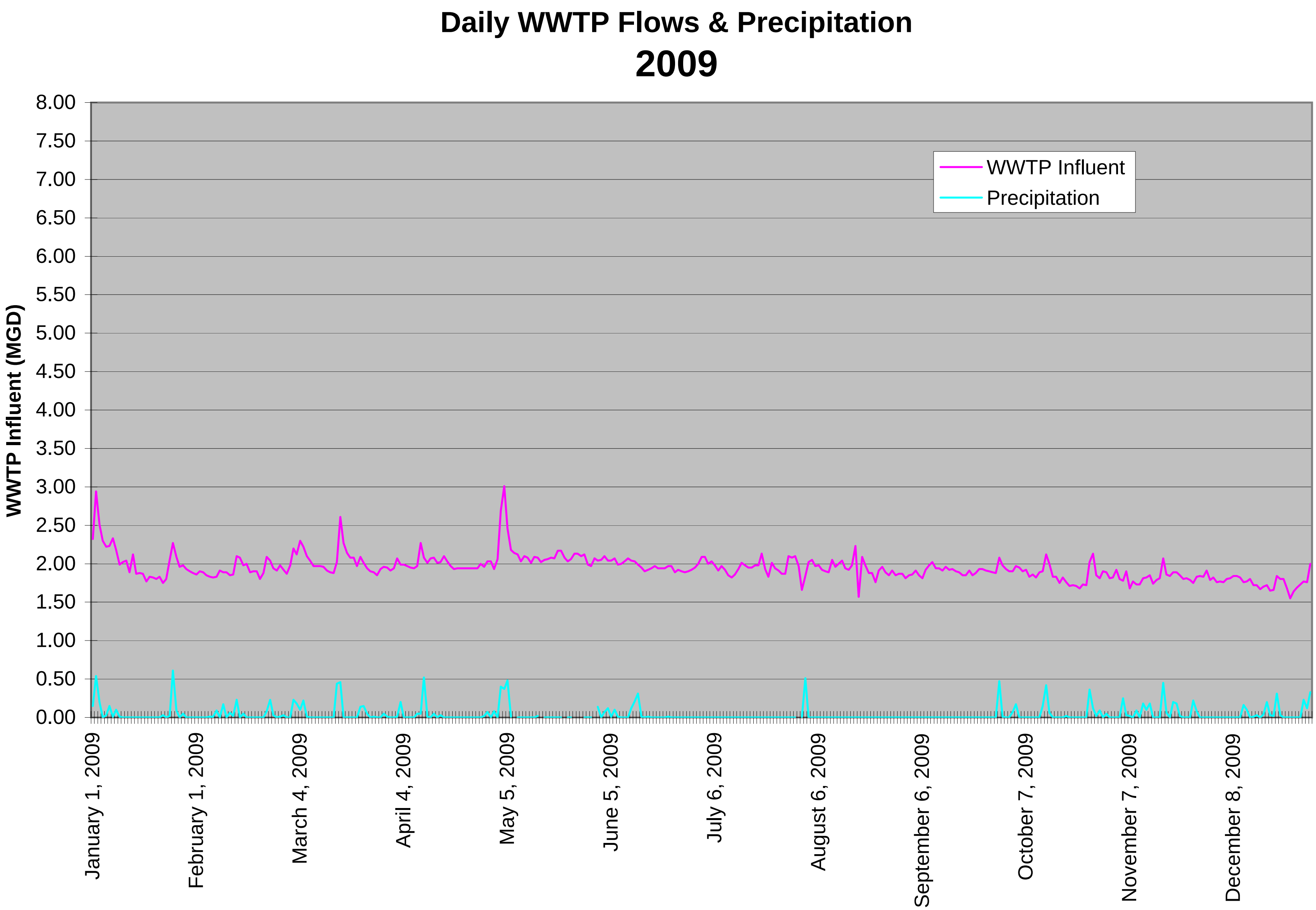
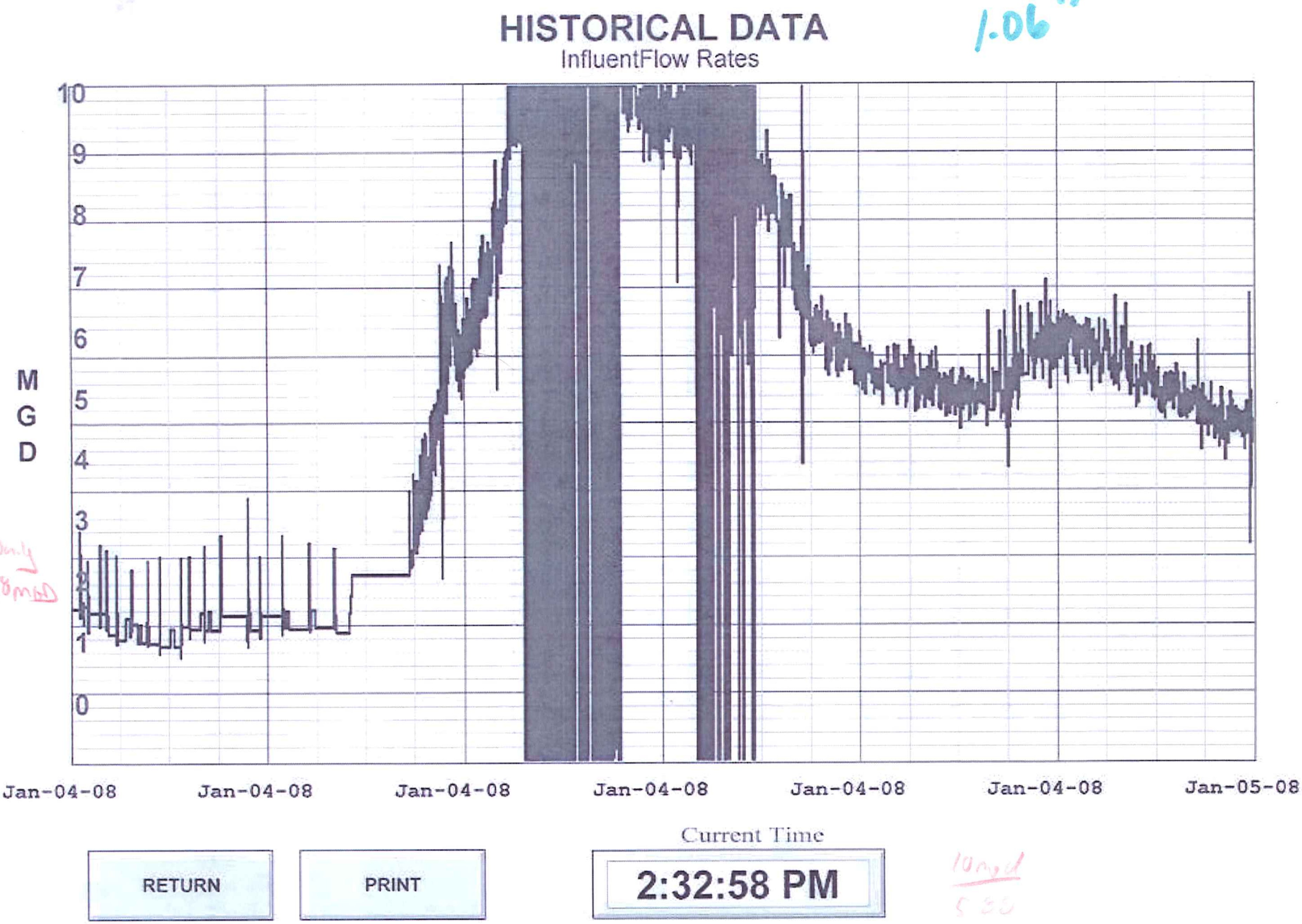

Daily WWTP Flows & Precipitation
2009
8.00
7.50
7.00
6.50
6.00
5.50
5.00
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
WWTP Influent (MGD)
WWTP Influent
Precipitation
January 1, 2009
February 1, 2009
March 4, 2009
April 4, 2009
May 5, 2009
June 5, 2009
July 6, 2009
August 6, 2009
September 6, 2009
October 7, 2009
November 7, 2009
December 8, 2009

Figure C-9



1.06"

Ave Daily
5.88 MGD

10 mgd
5.88
= 1.7

Figure C-10

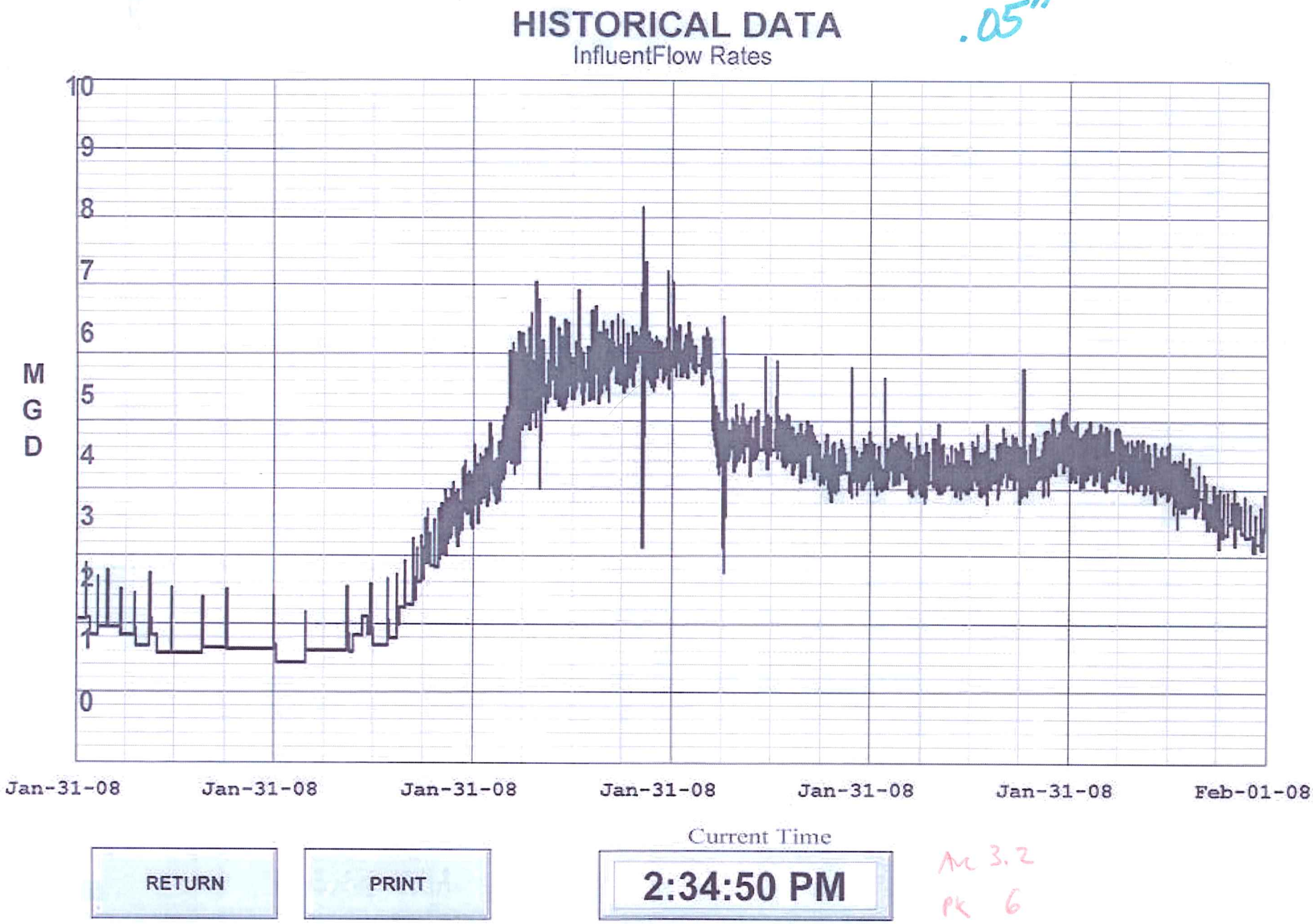
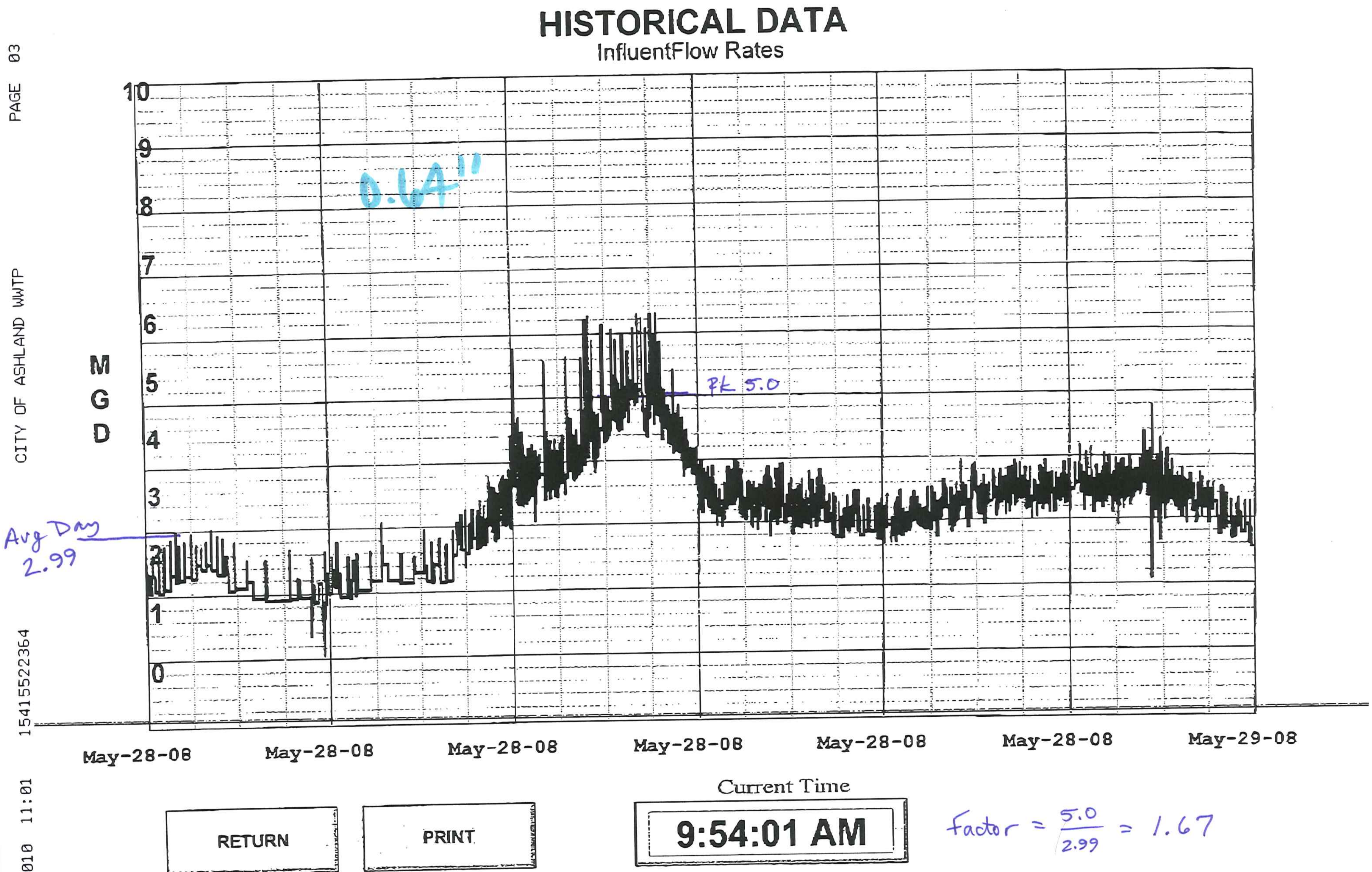


Figure C-11

.05"

Av 3.2
Pk 6
1.9



Factor $= \frac{5.0}{2.99} = 1.67$

Figure C-12

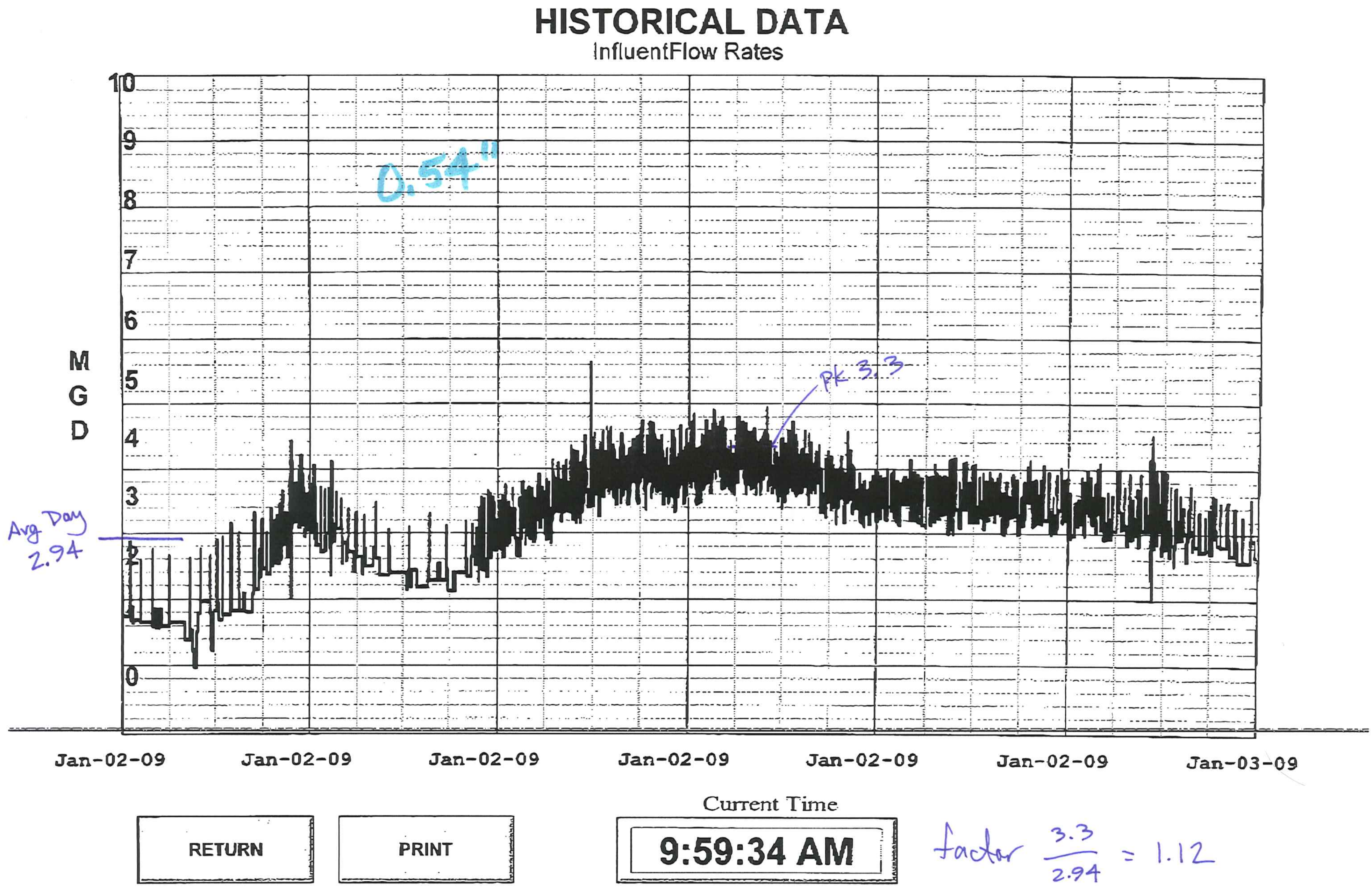
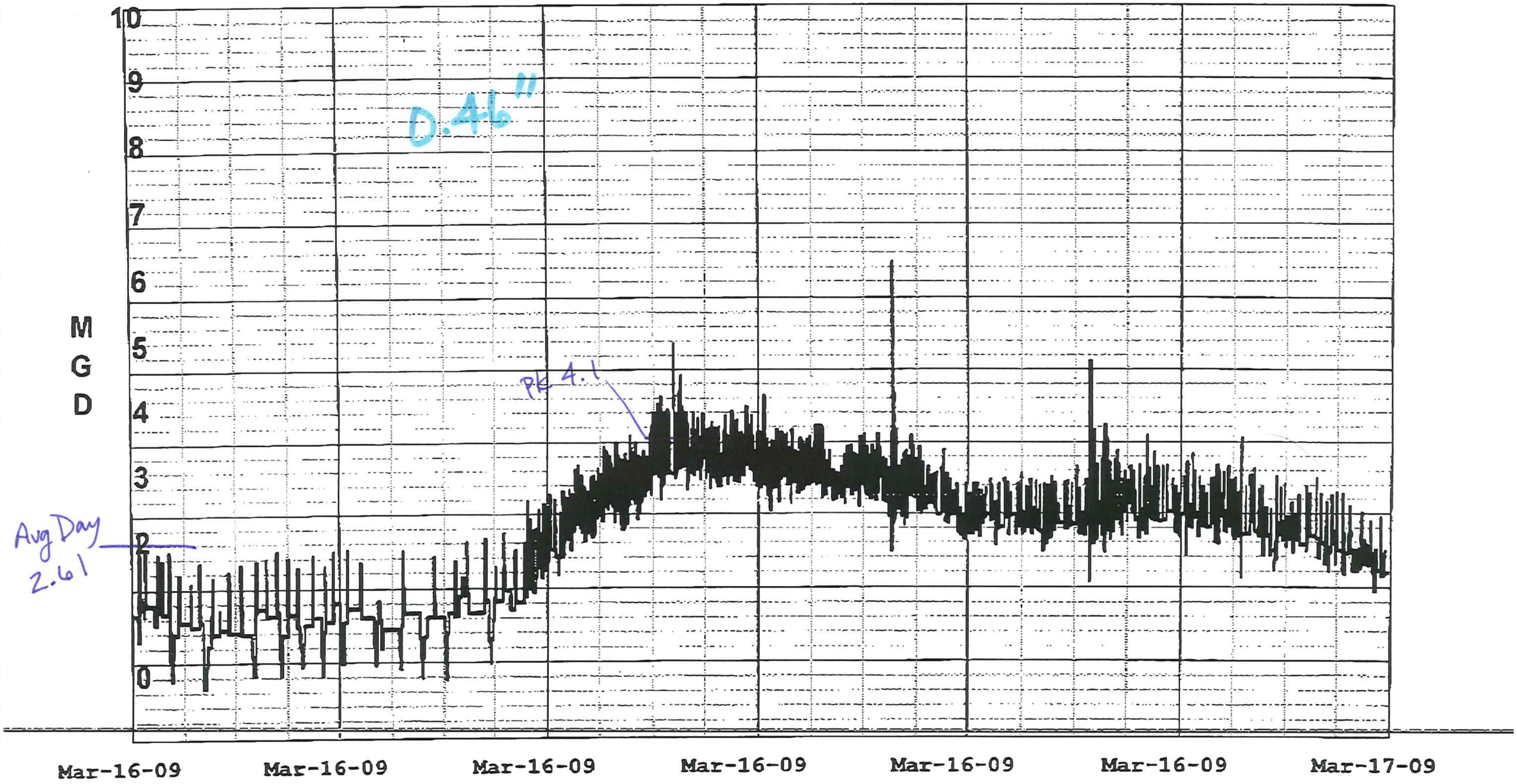

HISTORICAL DATA
InfluentFlow Rates
M G D
10
9
8
7
6
5
4
3
2
0
0.54"
PK 3.3
Avg Day 2.94
Jan-02-09
Jan-02-09
Jan-02-09
Jan-02-09
Jan-02-09
Jan-02-09
Jan-03-09
RETURN
PRINT
Current Time
9:59:34 AM
factor 3.3/2.94 = 1.12

Figure C-13

HISTORICAL DATA

InfluentFlow Rates



RETURN

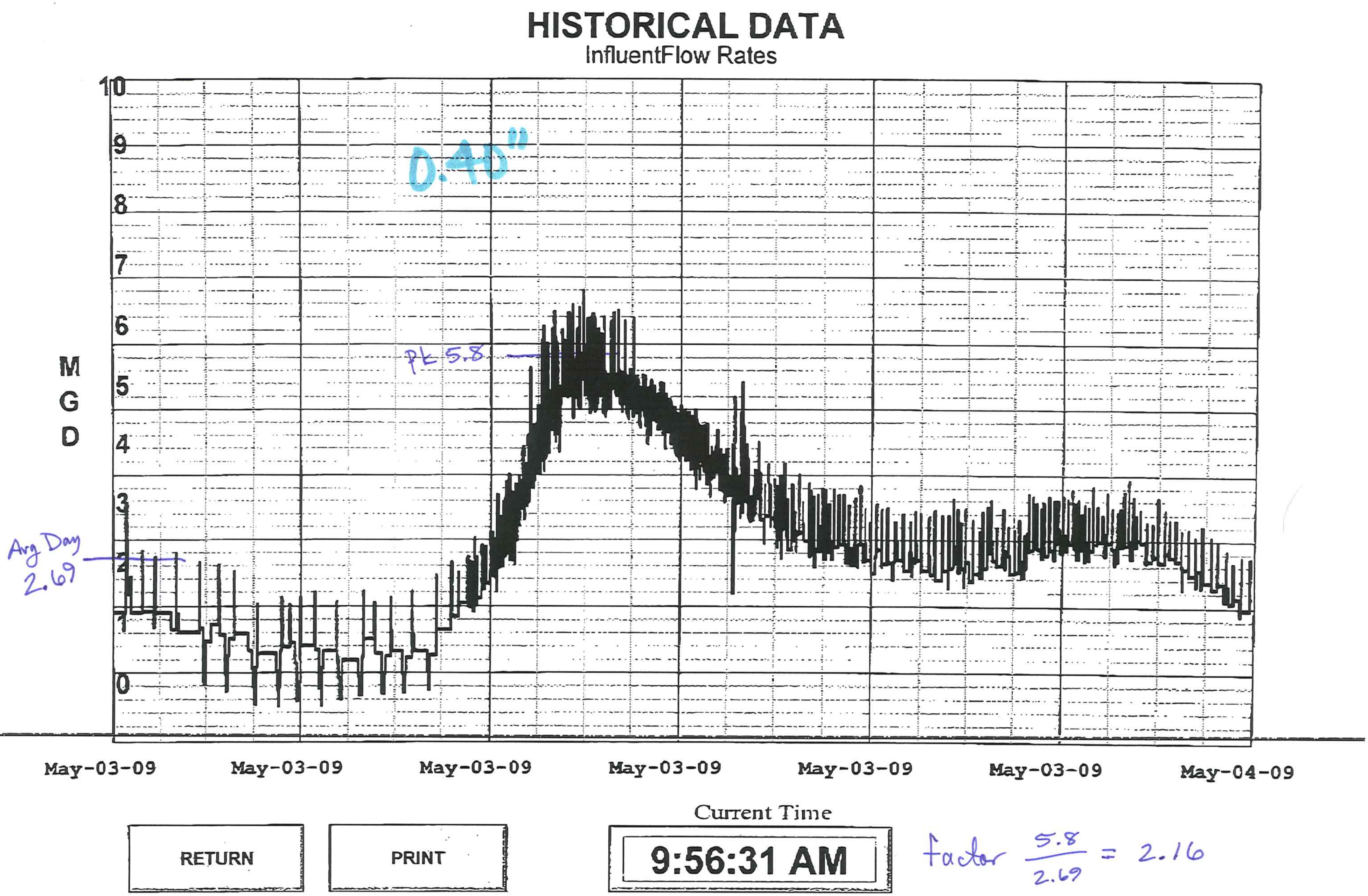
PRINT

Current Time

10:00:42 AM

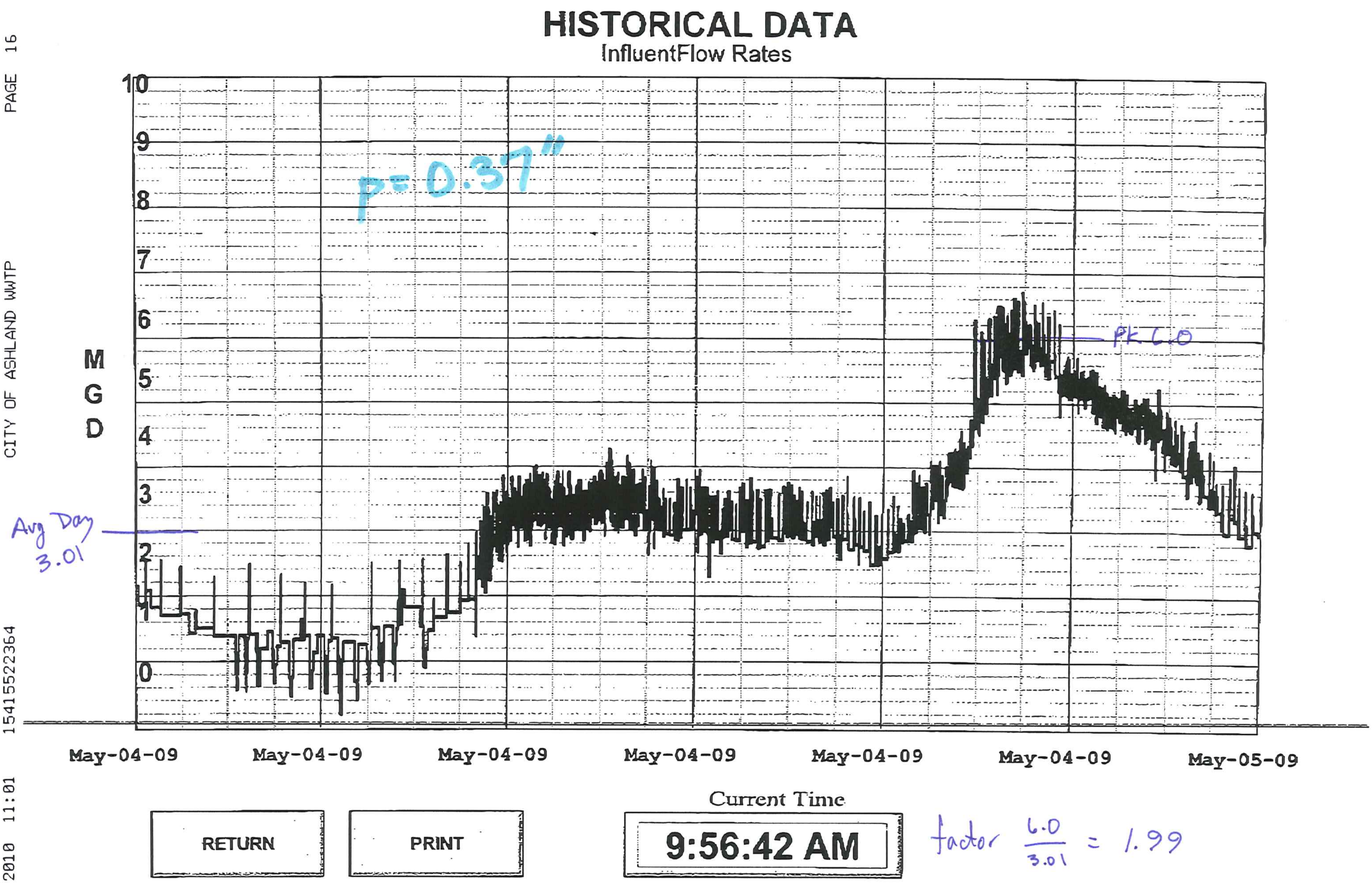
factor $\frac{4.1}{2.61} = 1.57$

Figure C-14



HISTORICAL DATA
InfluentFlow Rates
M G D
10
9
8
7
6
5
4
3
2
1
0
0.40"
Pk 5.8
Avg Day 2.69
May-03-09
May-03-09
May-03-09
May-03-09
May-03-09
May-03-09
May-04-09
RETURN
PRINT
Current Time
9:56:31 AM
factor 5.8/2.69 = 2.16

Figure C-15



factor $\frac{6.0}{3.01} = 1.99$

Figure C-16

Summary of Flow Monitoring Data

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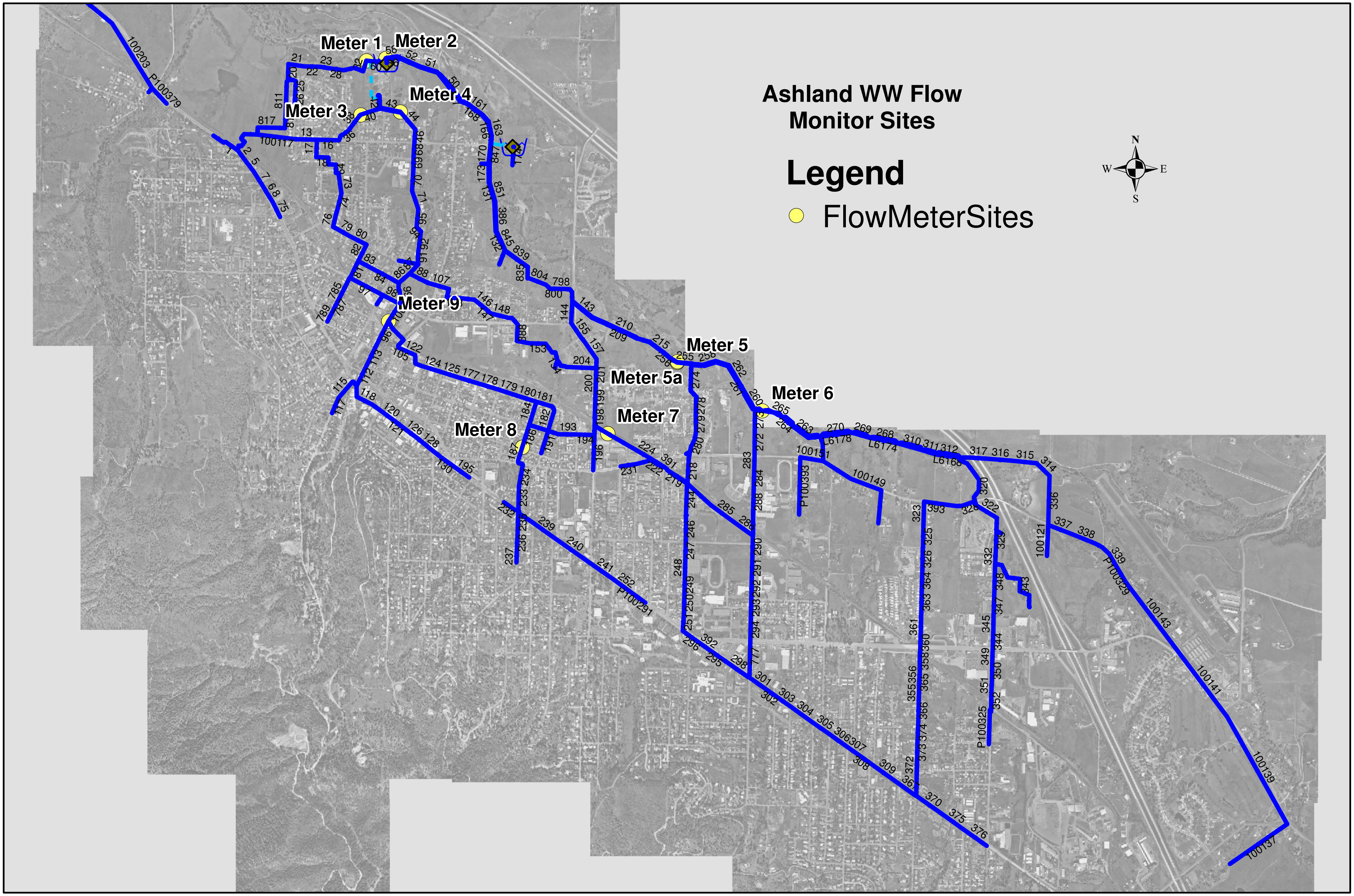
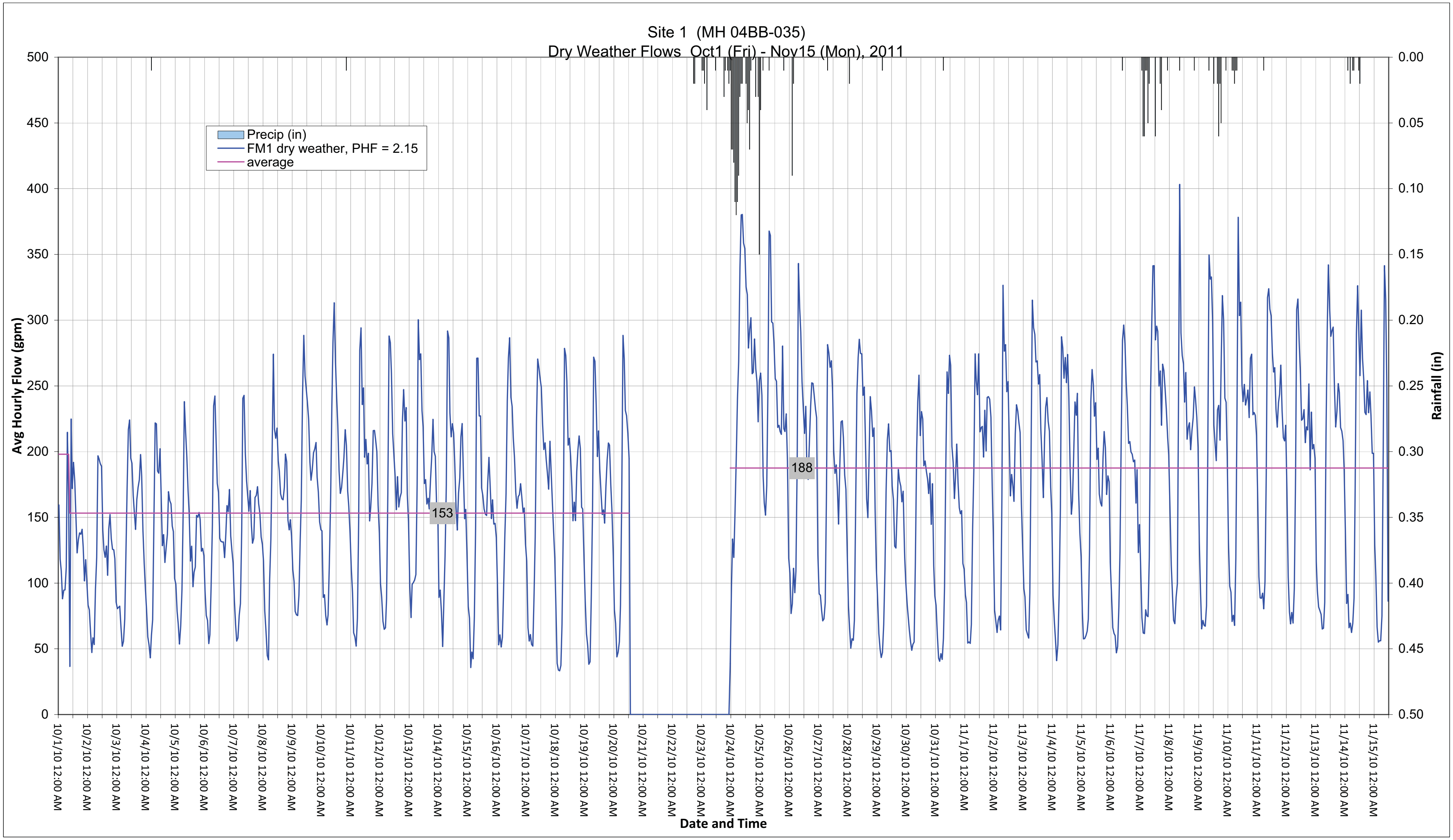


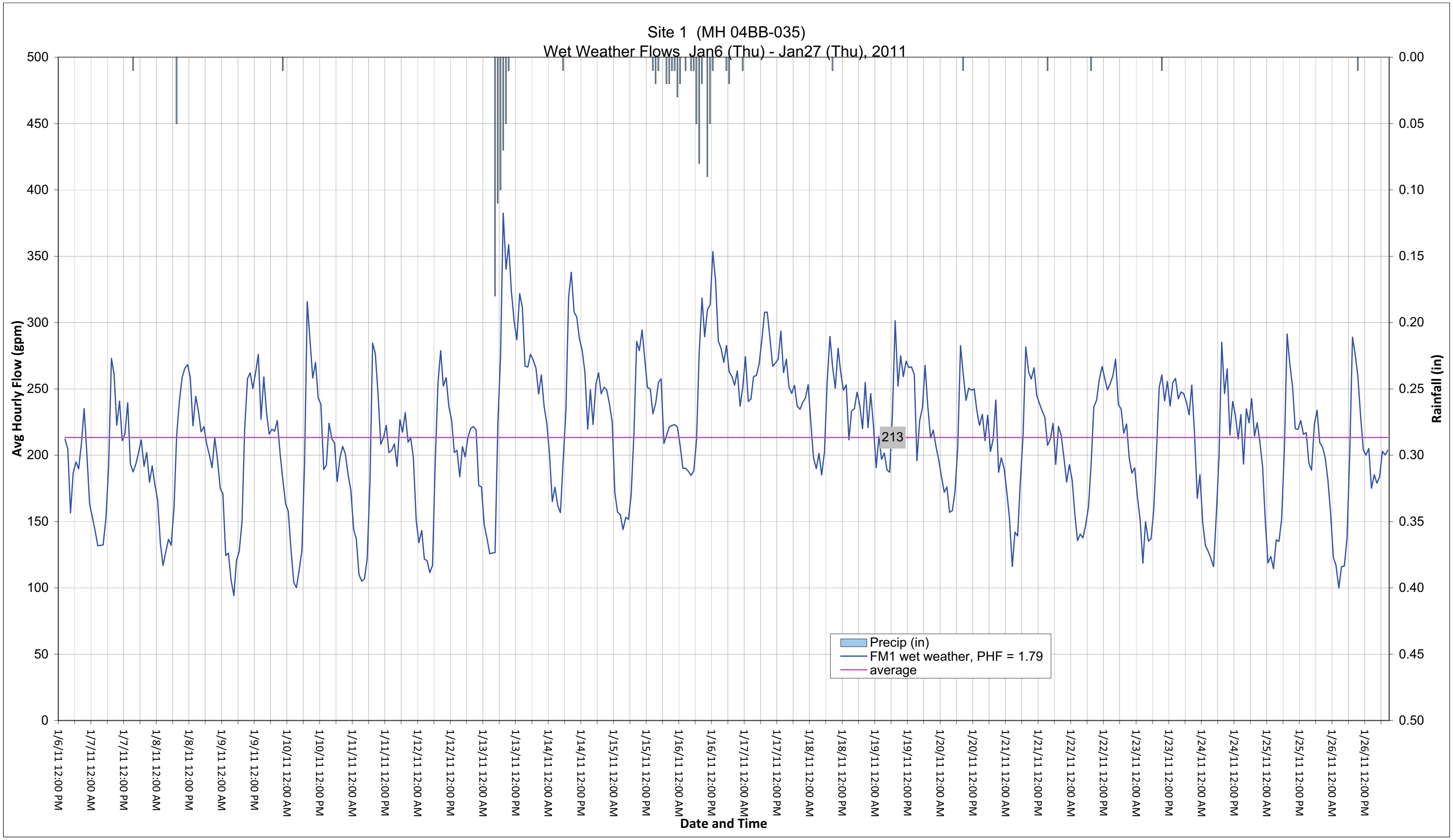
Path: J:\210055\Design\KellerCollectionModel\AshlandModel_Current.mxd Date: 10/13/2011 Scale 1:25,000

Ashland, OR
Flow Monitoring Sites/Meter Locations

Flow Meter		**9/7 - 9/17/10**	**9/17 - 10/1/10**	**10/1 - 11/15/10**	**12/17 - 1/6/2011**	**1/6 - 1/19/11**	**1/19 - 1/27/11**
1	Site:	6	6	1	6	1	1
2	Site:	9	5	9	5	9	9
3	Site:	8	5a	4	5a	4	8
4	Site:	7	7	2	7	2	3



Site 1 (MH 04BB-035)
Dry Weather Flows Oct1 (Fri) - Nov15 (Mon), 2011
Precip (in)
FM1 dry weather, PHF = 2.15
average
153
188
Avg Hourly Flow (gpm)
Rainfall (in)
Date and Time



Site 1 (MH 04BB-035)
Wet Weather Flows Jan6 (Thu) - Jan27 (Thu), 2011
Avg Hourly Flow (gpm)
Rainfall (in)
Date and Time
Precip (in)
FM1 wet weather, PHF = 1.79
average
213
500
450
400
350
300
250
200
150
100
50
0
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
1/6/11 12:00 PM
1/7/11 12:00 AM
1/7/11 12:00 PM
1/8/11 12:00 AM
1/8/11 12:00 PM
1/9/11 12:00 AM
1/9/11 12:00 PM
1/10/11 12:00 AM
1/10/11 12:00 PM
1/11/11 12:00 AM
1/11/11 12:00 PM
1/12/11 12:00 AM
1/12/11 12:00 PM
1/13/11 12:00 AM
1/13/11 12:00 PM
1/14/11 12:00 AM
1/14/11 12:00 PM
1/15/11 12:00 AM
1/15/11 12:00 PM
1/16/11 12:00 AM
1/16/11 12:00 PM
1/17/11 12:00 AM
1/17/11 12:00 PM
1/18/11 12:00 AM
1/18/11 12:00 PM
1/19/11 12:00 AM
1/19/11 12:00 PM
1/20/11 12:00 AM
1/20/11 12:00 PM
1/21/11 12:00 AM
1/21/11 12:00 PM
1/22/11 12:00 AM
1/22/11 12:00 PM
1/23/11 12:00 AM
1/23/11 12:00 PM
1/24/11 12:00 AM
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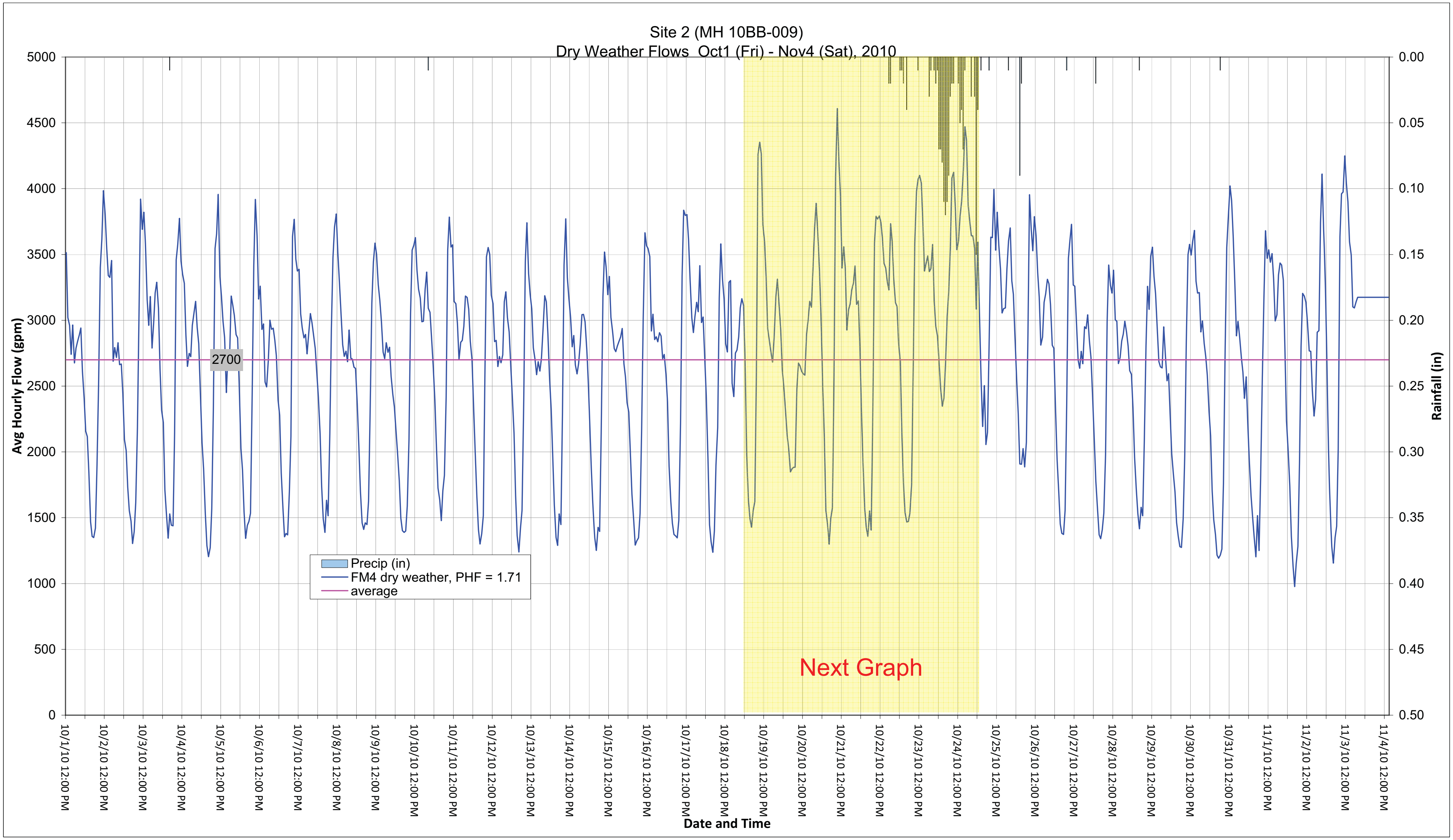
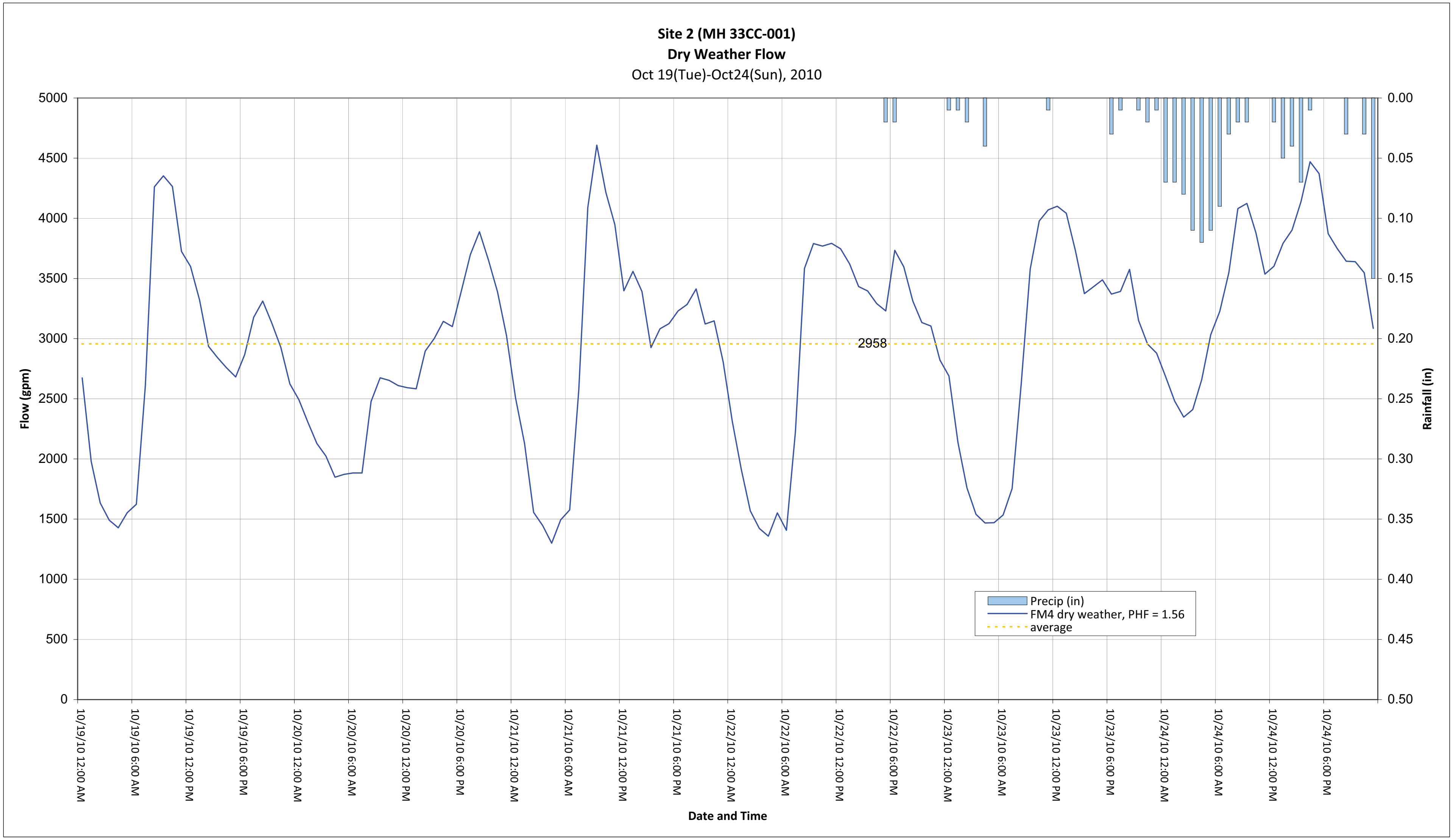
1/26/11 12:00 AM
1/26/11 12:00 PM



Site 2 (MH 10BB-009)
Dry Weather Flows Oct1 (Fri) - Nov4 (Sat), 2010
Avg Hourly Flow (gpm)
Rainfall (in)
Date and Time
Precip (in)
FM4 dry weather, PHF = 1.71
average
2700
Next Graph



Site 2 (MH 33CC-001)
Dry Weather Flow
Oct 19(Tue)-Oct24(Sun), 2010
Flow (gpm)
Rainfall (in)
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
2958
Precip (in)
FM4 dry weather, PHF = 1.56
average
10/19/10 12:00 AM
10/19/10 6:00 AM
10/19/10 12:00 PM
10/19/10 6:00 PM
10/20/10 12:00 AM
10/20/10 6:00 AM
10/20/10 12:00 PM
10/20/10 6:00 PM
10/21/10 12:00 AM
10/21/10 6:00 AM
10/21/10 12:00 PM
10/21/10 6:00 PM
10/22/10 12:00 AM
10/22/10 6:00 AM
10/22/10 12:00 PM
10/22/10 6:00 PM
10/23/10 12:00 AM
10/23/10 6:00 AM
10/23/10 12:00 PM
10/23/10 6:00 PM
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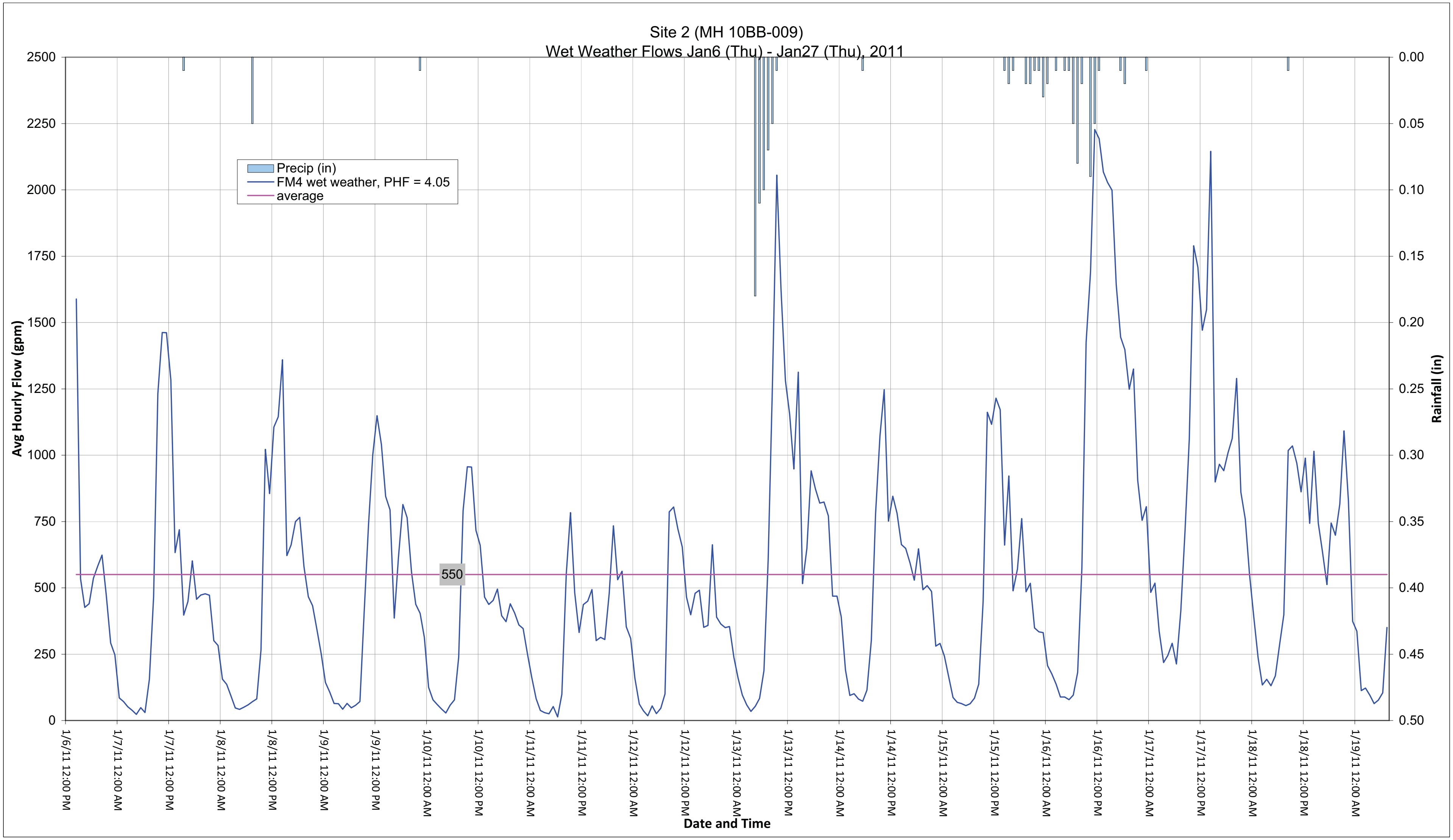
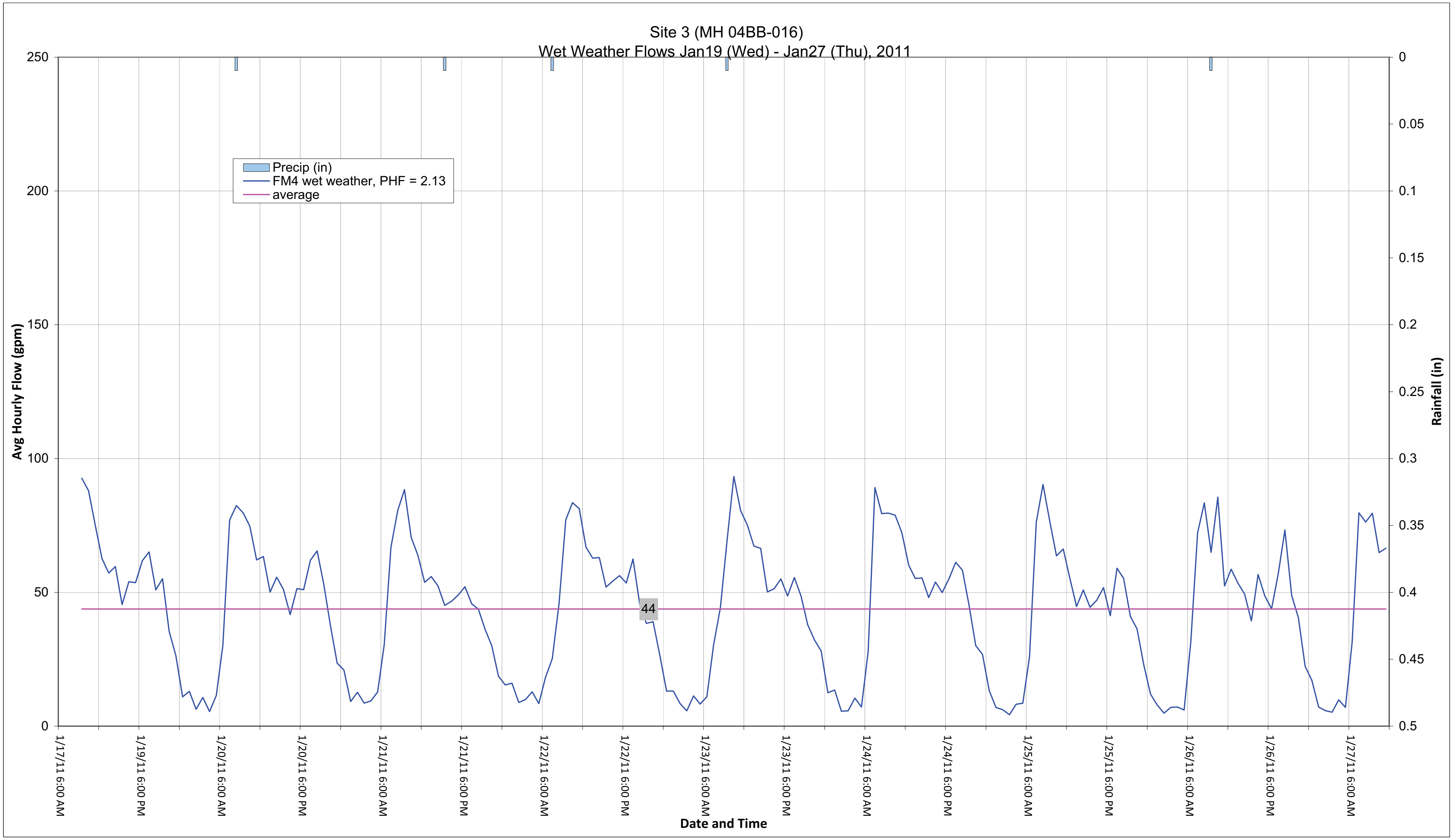
10/24/10 6:00 AM
10/24/10 12:00 PM
10/24/10 6:00 PM
Date and Time



Site 2 (MH 10BB-009)
Wet Weather Flows Jan6 (Thu) - Jan27 (Thu), 2011
Precip (in)
FM4 wet weather, PHF = 4.05
average
550
Avg Hourly Flow (gpm)
Rainfall (in)
2500
2250
2000
1750
1500
1250
1000
750
500
250
0
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
1/6/11 12:00 PM
1/7/11 12:00 AM
1/7/11 12:00 PM
1/8/11 12:00 AM
1/8/11 12:00 PM
1/9/11 12:00 AM
1/9/11 12:00 PM
1/10/11 12:00 AM
1/10/11 12:00 PM
1/11/11 12:00 AM
1/11/11 12:00 PM
1/12/11 12:00 AM
1/12/11 12:00 PM
1/13/11 12:00 AM
1/13/11 12:00 PM
1/14/11 12:00 AM
1/14/11 12:00 PM
1/15/11 12:00 AM
1/15/11 12:00 PM
1/16/11 12:00 AM
1/16/11 12:00 PM
1/17/11 12:00 AM
1/17/11 12:00 PM
1/18/11 12:00 AM
1/18/11 12:00 PM
1/19/11 12:00 AM
Date and Time


Site 3 (MH 04BB-016)
Wet Weather Flows Jan19 (Wed) - Jan27 (Thu), 2011
Precip (in)
FM4 wet weather, PHF = 2.13
average
Avg Hourly Flow (gpm)
Rainfall (in)
Date and Time
44
250
200
150
100
50
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
1/17/11 6:00 AM
1/19/11 6:00 PM
1/20/11 6:00 AM
1/20/11 6:00 PM
1/21/11 6:00 AM
1/21/11 6:00 PM
1/22/11 6:00 AM
1/22/11 6:00 PM
1/23/11 6:00 AM
1/23/11 6:00 PM
1/24/11 6:00 AM
1/24/11 6:00 PM
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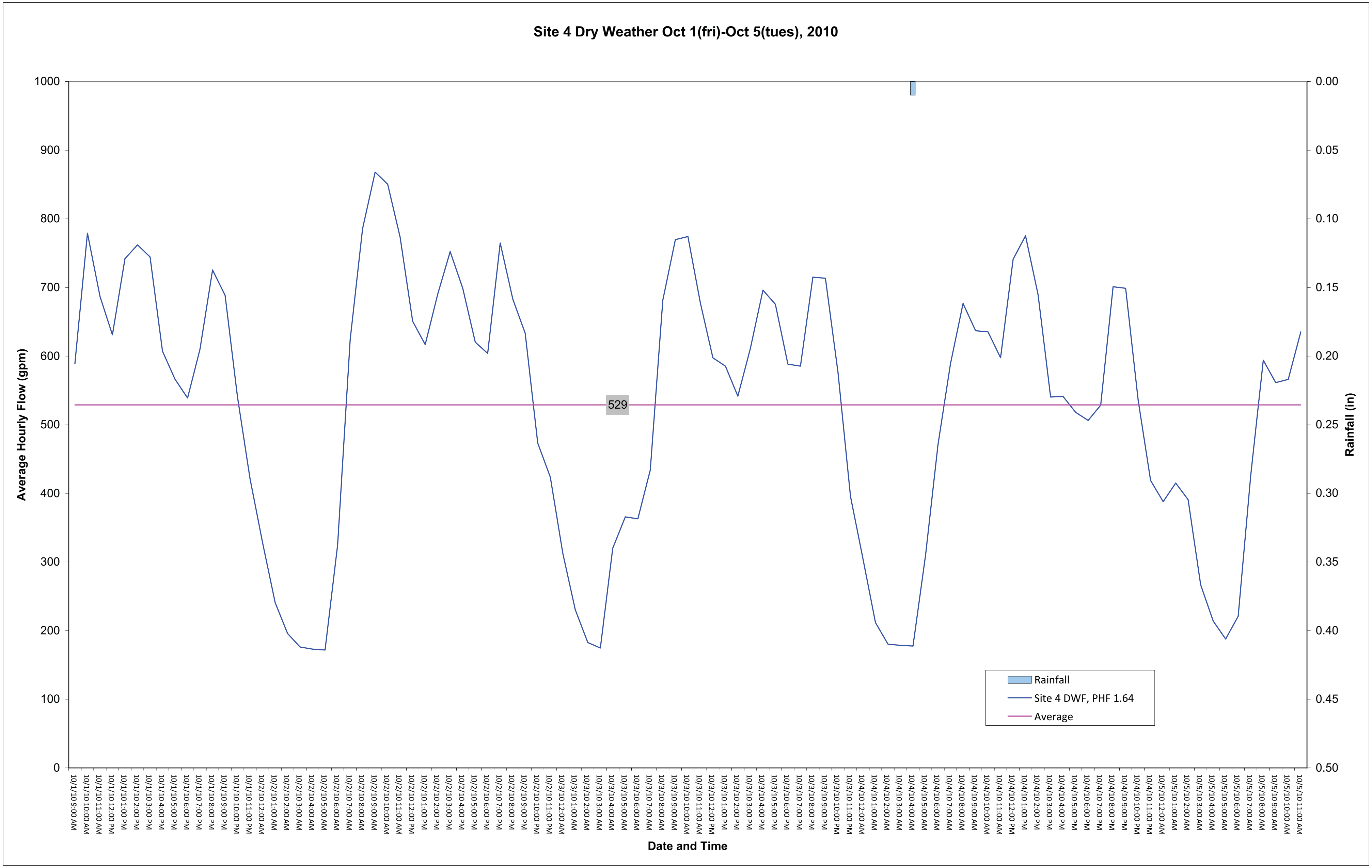
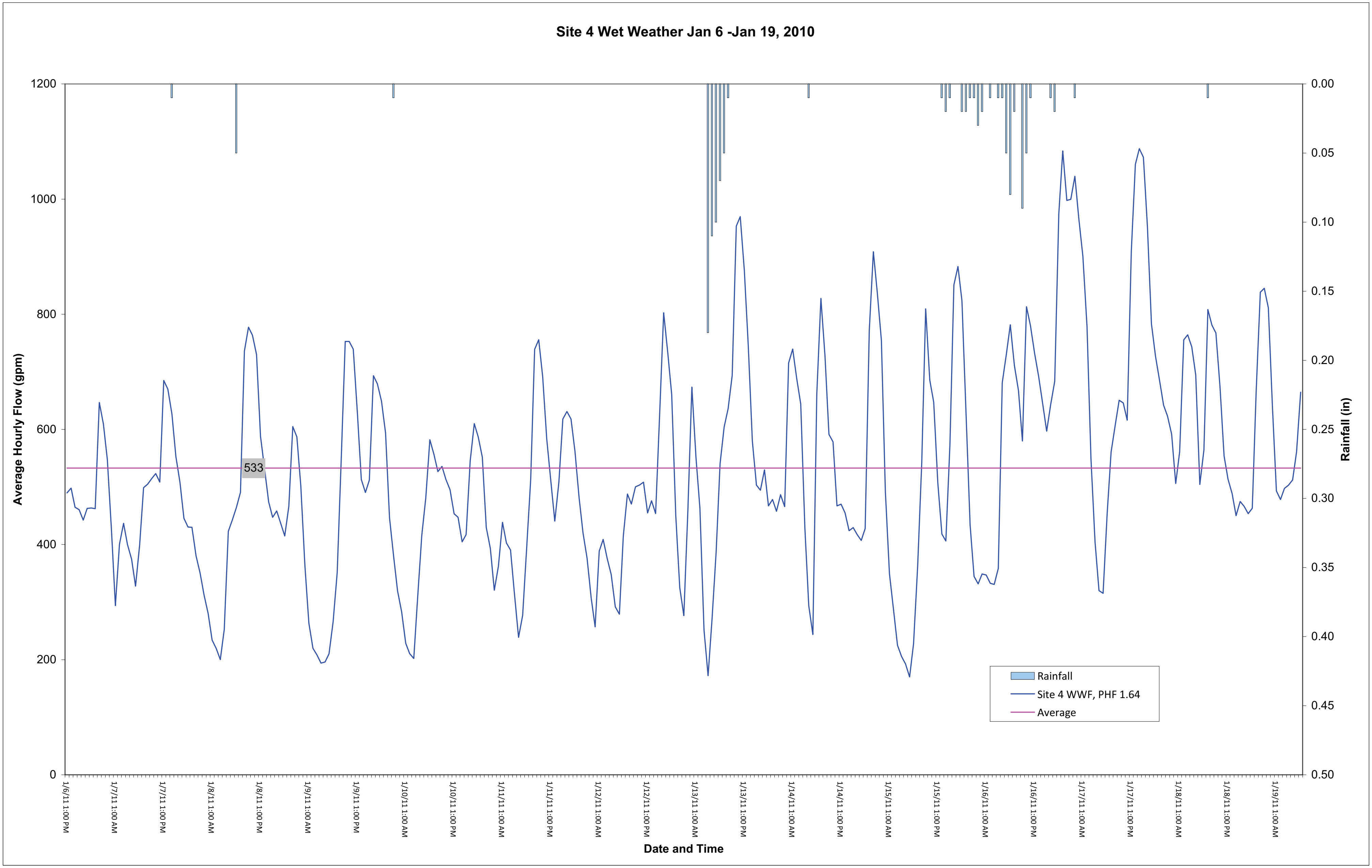
1/26/11 6:00 AM
1/26/11 6:00 PM
1/27/11 6:00 AM



Site 4 Dry Weather Oct 1(fri)-Oct 5(tues), 2010
Average Hourly Flow (gpm)
Rainfall (in)
Date and Time
529
Rainfall
Site 4 DWF, PHF 1.64
Average



Site 4 Wet Weather Jan 6 -Jan 19, 2010
Average Hourly Flow (gpm)
Rainfall (in)
Date and Time
533
Rainfall
Site 4 WWF, PHF 1.64
Average
0
200
400
600
800
1000
1200
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
1/6/11 1:00 PM
1/7/11 1:00 AM
1/7/11 1:00 PM
1/8/11 1:00 AM
1/8/11 1:00 PM
1/9/11 1:00 AM
1/9/11 1:00 PM
1/10/11 1:00 AM
1/10/11 1:00 PM
1/11/11 1:00 AM
1/11/11 1:00 PM
1/12/11 1:00 AM
1/12/11 1:00 PM
1/13/11 1:00 AM
1/13/11 1:00 PM
1/14/11 1:00 AM
1/14/11 1:00 PM
1/15/11 1:00 AM
1/15/11 1:00 PM
1/16/11 1:00 AM
1/16/11 1:00 PM
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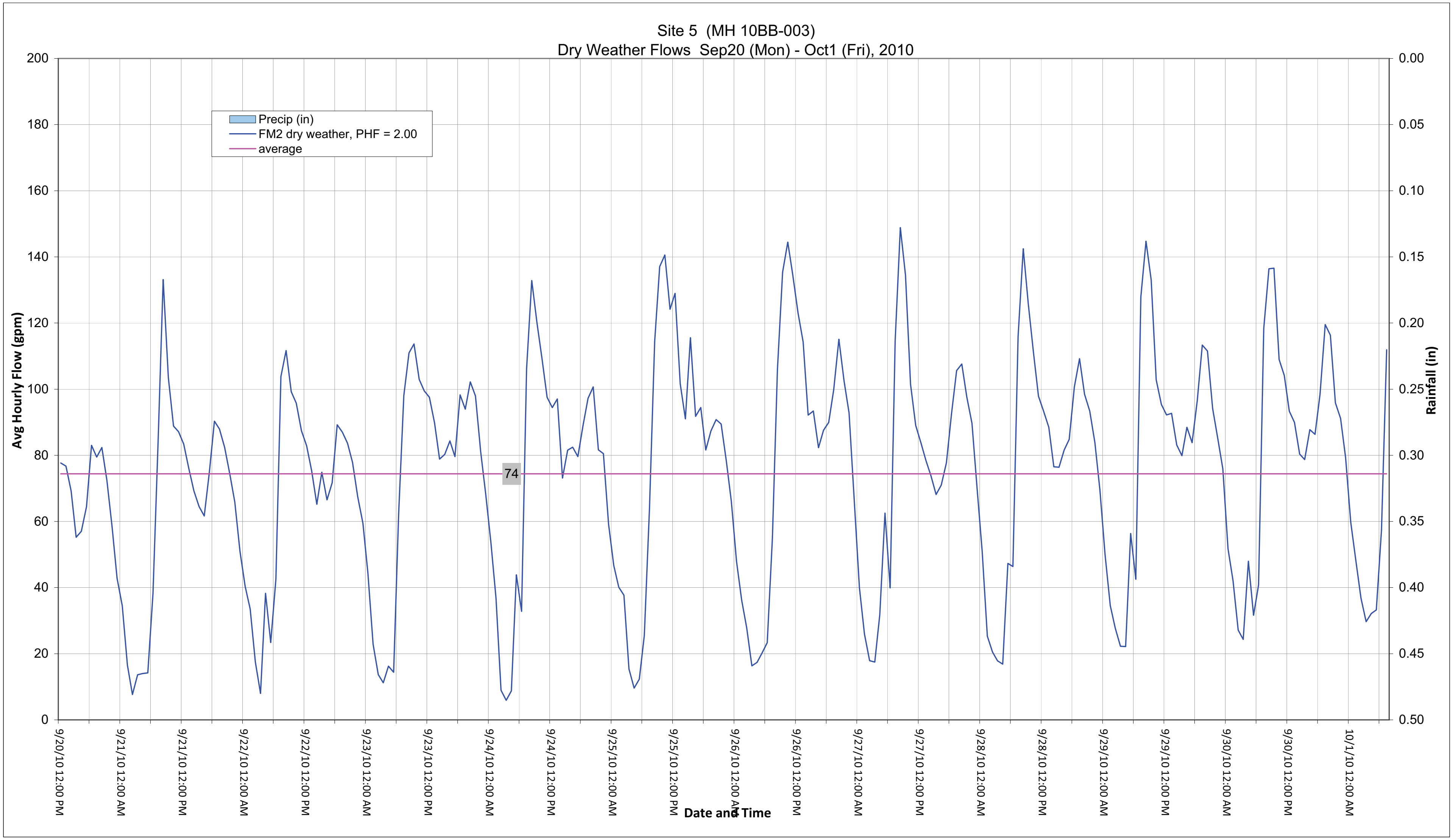
1/18/11 1:00 AM
1/18/11 1:00 PM
1/19/11 1:00 AM



Site 5 (MH 10BB-003)
Dry Weather Flows Sep20 (Mon) - Oct1 (Fri), 2010
Precip (in)
FM2 dry weather, PHF = 2.00
average
74
Avg Hourly Flow (gpm)
Rainfall (in)
Date and Time
0
20
40
60
80
100
120
140
160
180
200
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
9/20/10 12:00 PM
9/21/10 12:00 AM
9/21/10 12:00 PM
9/22/10 12:00 AM
9/22/10 12:00 PM
9/23/10 12:00 AM
9/23/10 12:00 PM
9/24/10 12:00 AM
9/24/10 12:00 PM
9/25/10 12:00 AM
9/25/10 12:00 PM
9/26/10 12:00 AM
9/26/10 12:00 PM
9/27/10 12:00 AM
9/27/10 12:00 PM
9/28/10 12:00 AM
9/28/10 12:00 PM
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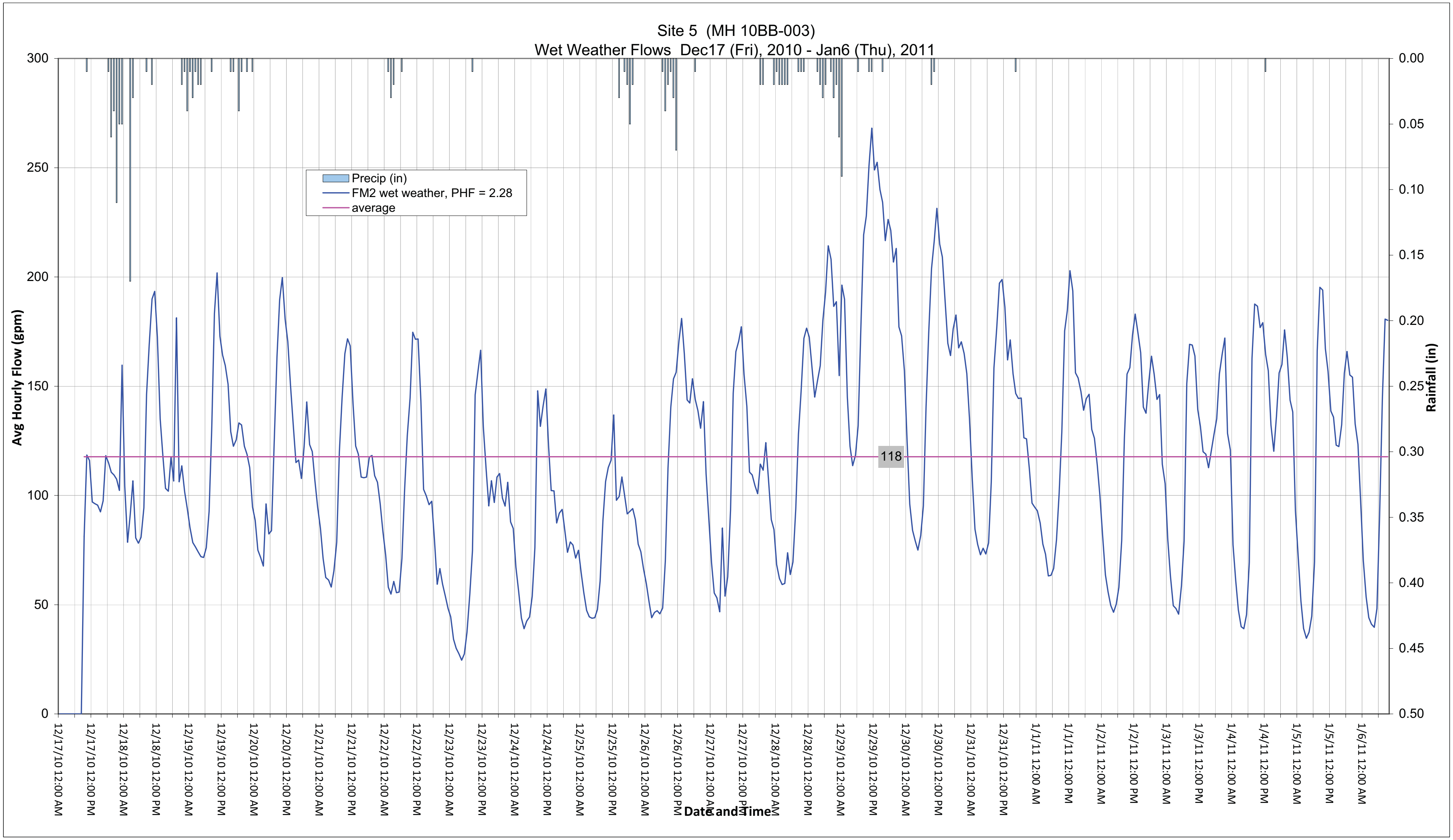
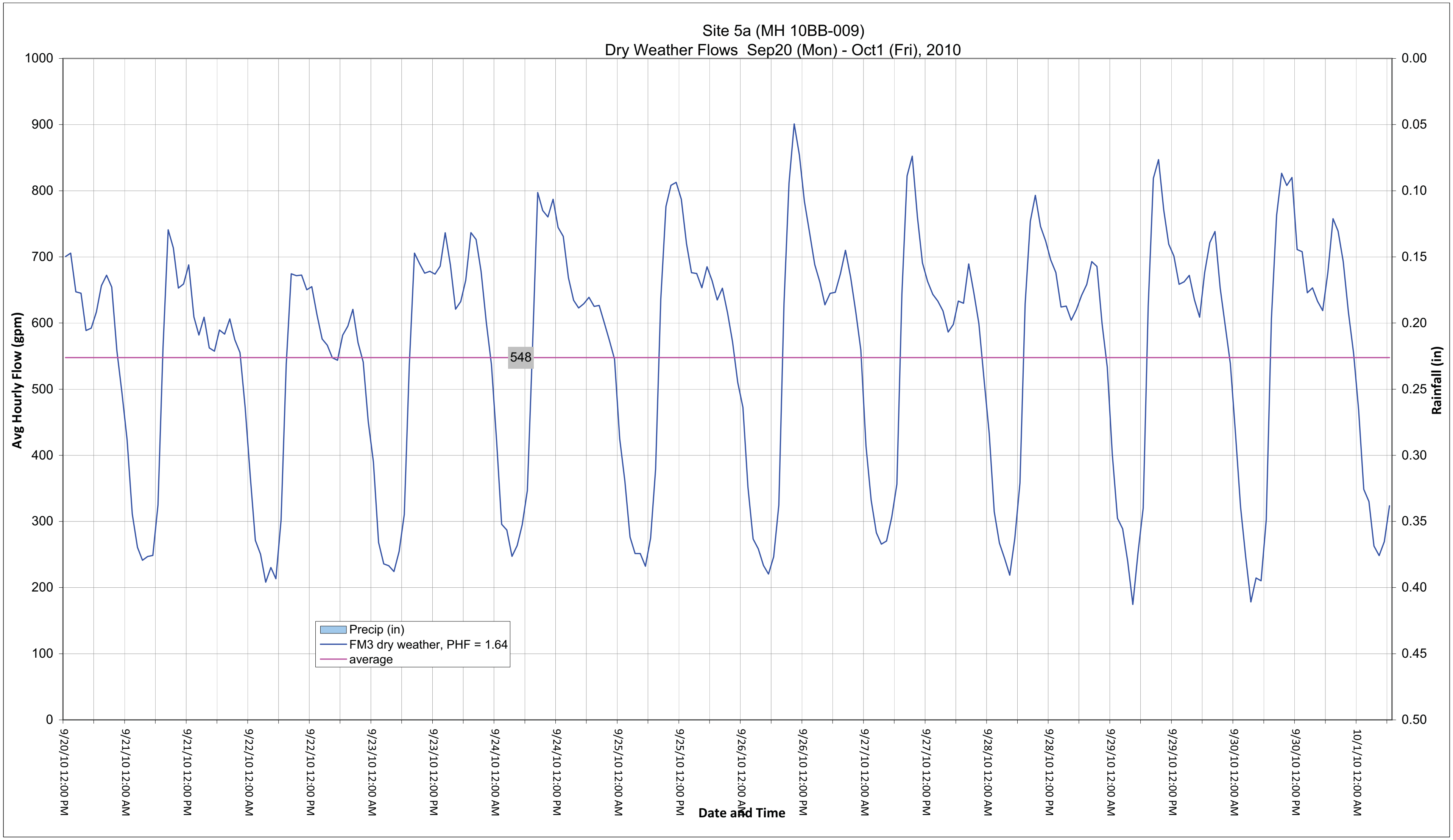
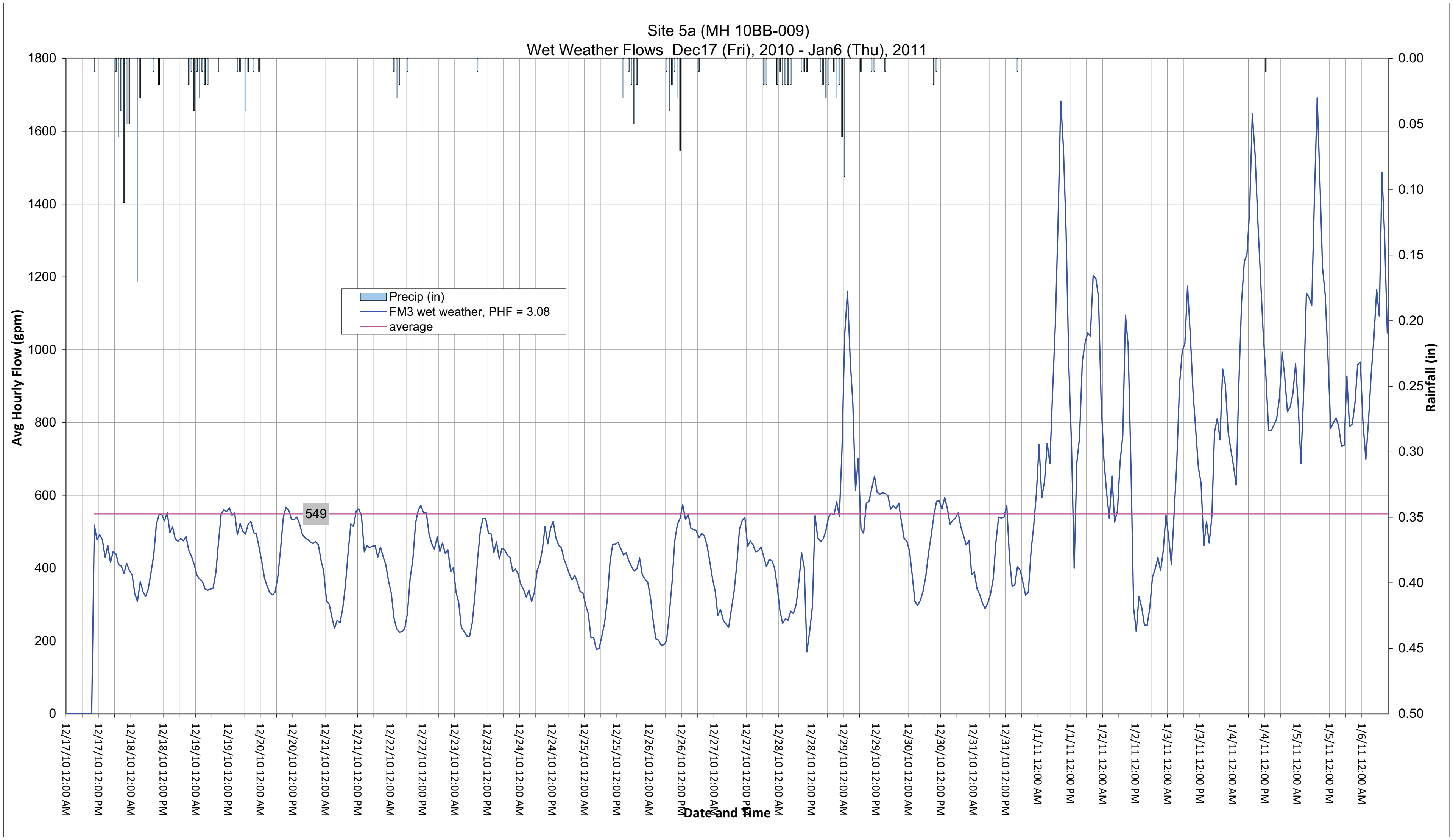
10/1/10 12:00 AM



Site 5 (MH 10BB-003)
Wet Weather Flows Dec17 (Fri), 2010 - Jan6 (Thu), 2011
Precip (in)
FM2 wet weather, PHF = 2.28
average
118
Avg Hourly Flow (gpm)
Rainfall (in)
Date and Time


Site 5a (MH 10BB-009)
Dry Weather Flows Sep20 (Mon) - Oct1 (Fri), 2010
Avg Hourly Flow (gpm)
Rainfall (in)
1000
900
800
700
600
500
400
300
200
100
0
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
548
Precip (in)
FM3 dry weather, PHF = 1.64
average
9/20/10 12:00 PM
9/21/10 12:00 AM
9/21/10 12:00 PM
9/22/10 12:00 AM
9/22/10 12:00 PM
9/23/10 12:00 AM
9/23/10 12:00 PM
9/24/10 12:00 AM
9/24/10 12:00 PM
9/25/10 12:00 AM
9/25/10 12:00 PM
9/26/10 12:00 AM
9/26/10 12:00 PM
9/27/10 12:00 AM
9/27/10 12:00 PM
9/28/10 12:00 AM
9/28/10 12:00 PM
9/29/10 12:00 AM
9/29/10 12:00 PM
9/30/10 12:00 AM
9/30/10 12:00 PM
10/1/10 12:00 AM
Date and Time



Site 5a (MH 10BB-009)
Wet Weather Flows Dec17 (Fri), 2010 - Jan6 (Thu), 2011
Avg Hourly Flow (gpm)
Rainfall (in)
Date and Time
Precip (in)
FM3 wet weather, PHF = 3.08
average
549
1800
1600
1400
1200
1000
800
600
400
200
0
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
12/17/10 12:00 AM
12/17/10 12:00 PM
12/18/10 12:00 AM
12/18/10 12:00 PM
12/19/10 12:00 AM
12/19/10 12:00 PM
12/20/10 12:00 AM
12/20/10 12:00 PM
12/21/10 12:00 AM
12/21/10 12:00 PM
12/22/10 12:00 AM
12/22/10 12:00 PM
12/23/10 12:00 AM
12/23/10 12:00 PM
12/24/10 12:00 AM
12/24/10 12:00 PM
12/25/10 12:00 AM
12/25/10 12:00 PM
12/26/10 12:00 AM
12/26/10 12:00 PM
12/27/10 12:00 AM
12/27/10 12:00 PM
12/28/10 12:00 AM
12/28/10 12:00 PM
12/29/10 12:00 AM
12/29/10 12:00 PM
12/30/10 12:00 AM
12/30/10 12:00 PM
12/31/10 12:00 AM
12/31/10 12:00 PM
1/1/11 12:00 AM
1/1/11 12:00 PM
1/2/11 12:00 AM
1/2/11 12:00 PM
1/3/11 12:00 AM
1/3/11 12:00 PM
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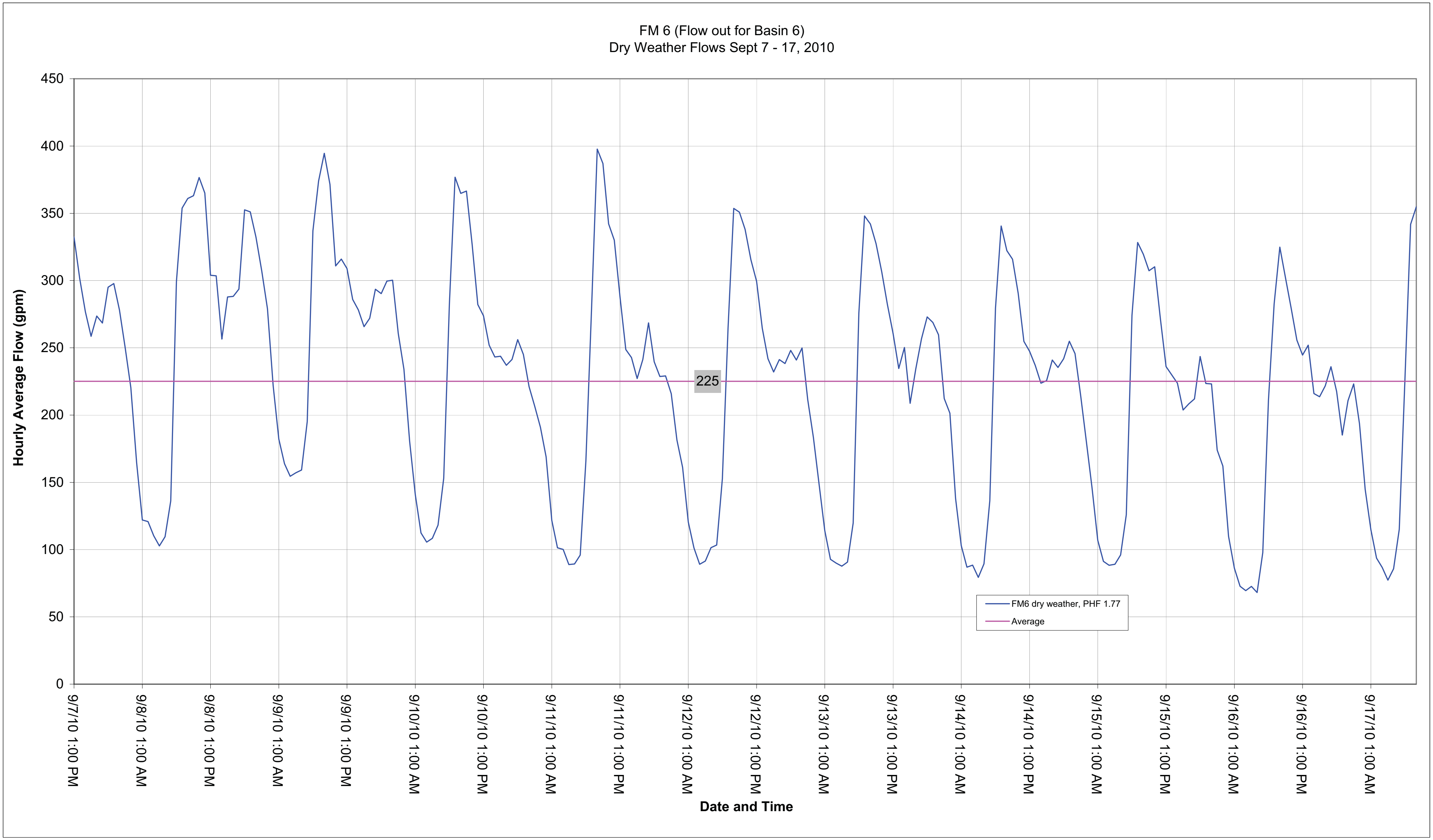
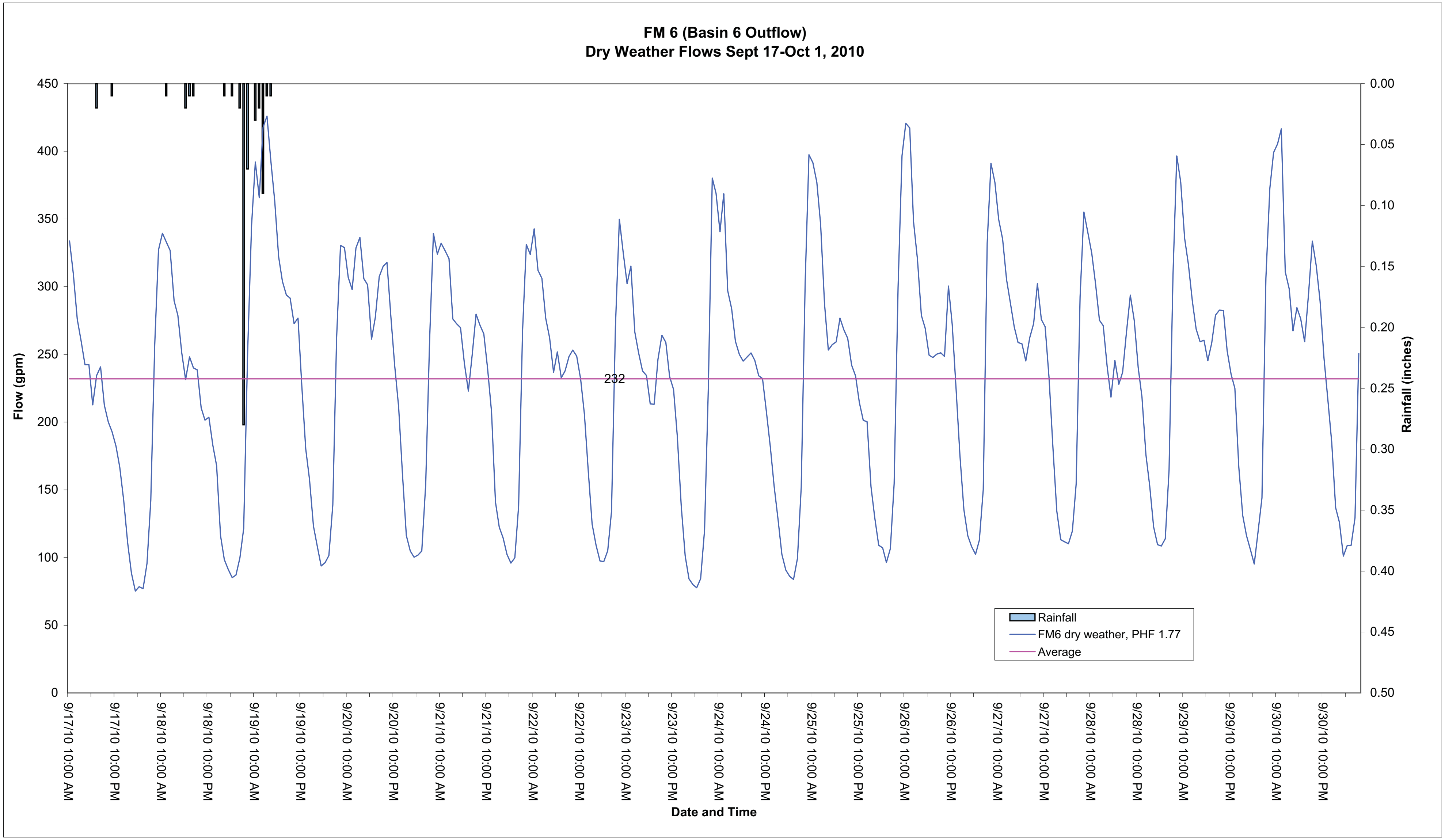
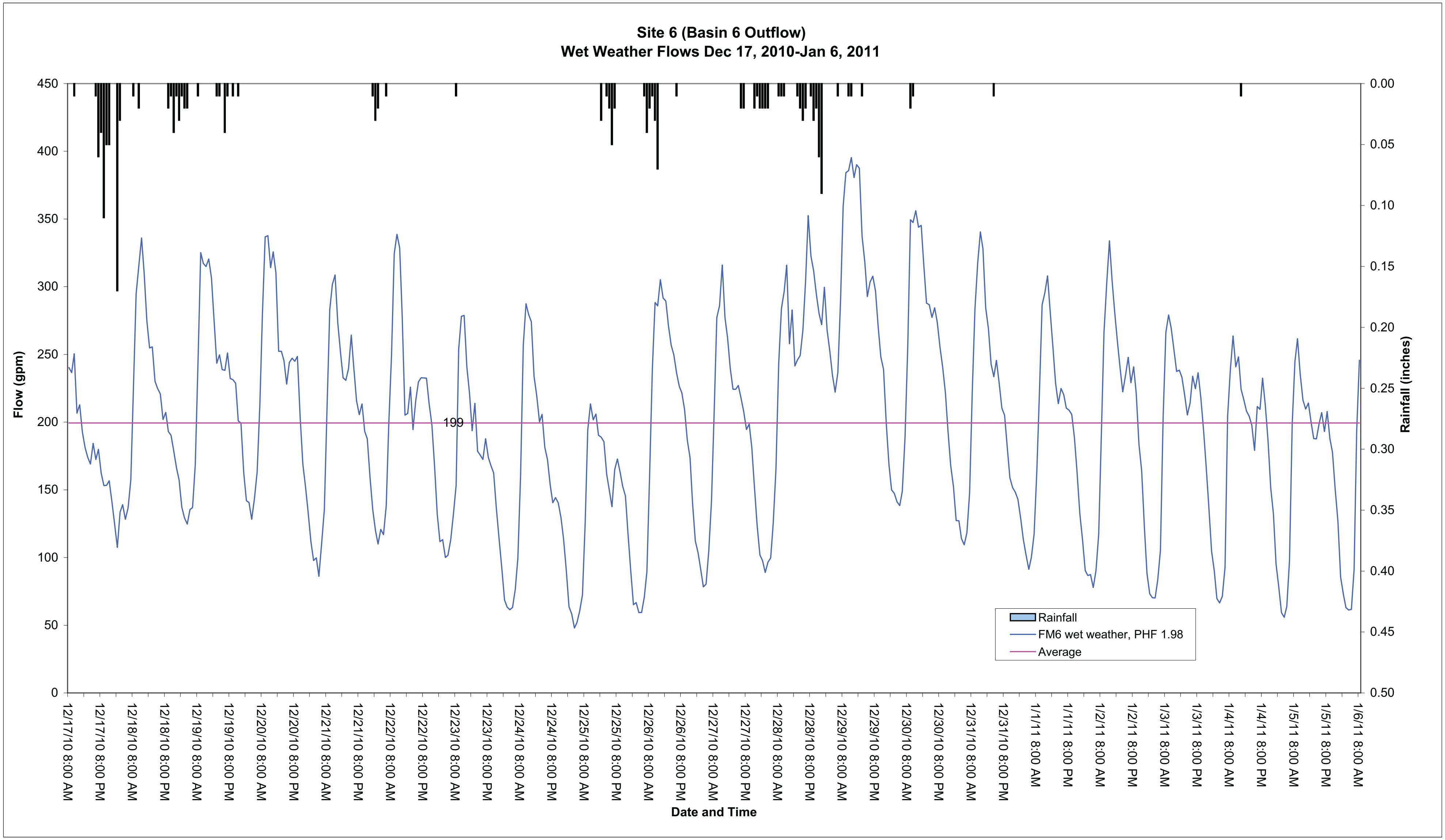
1/5/11 12:00 PM
1/6/11 12:00 AM



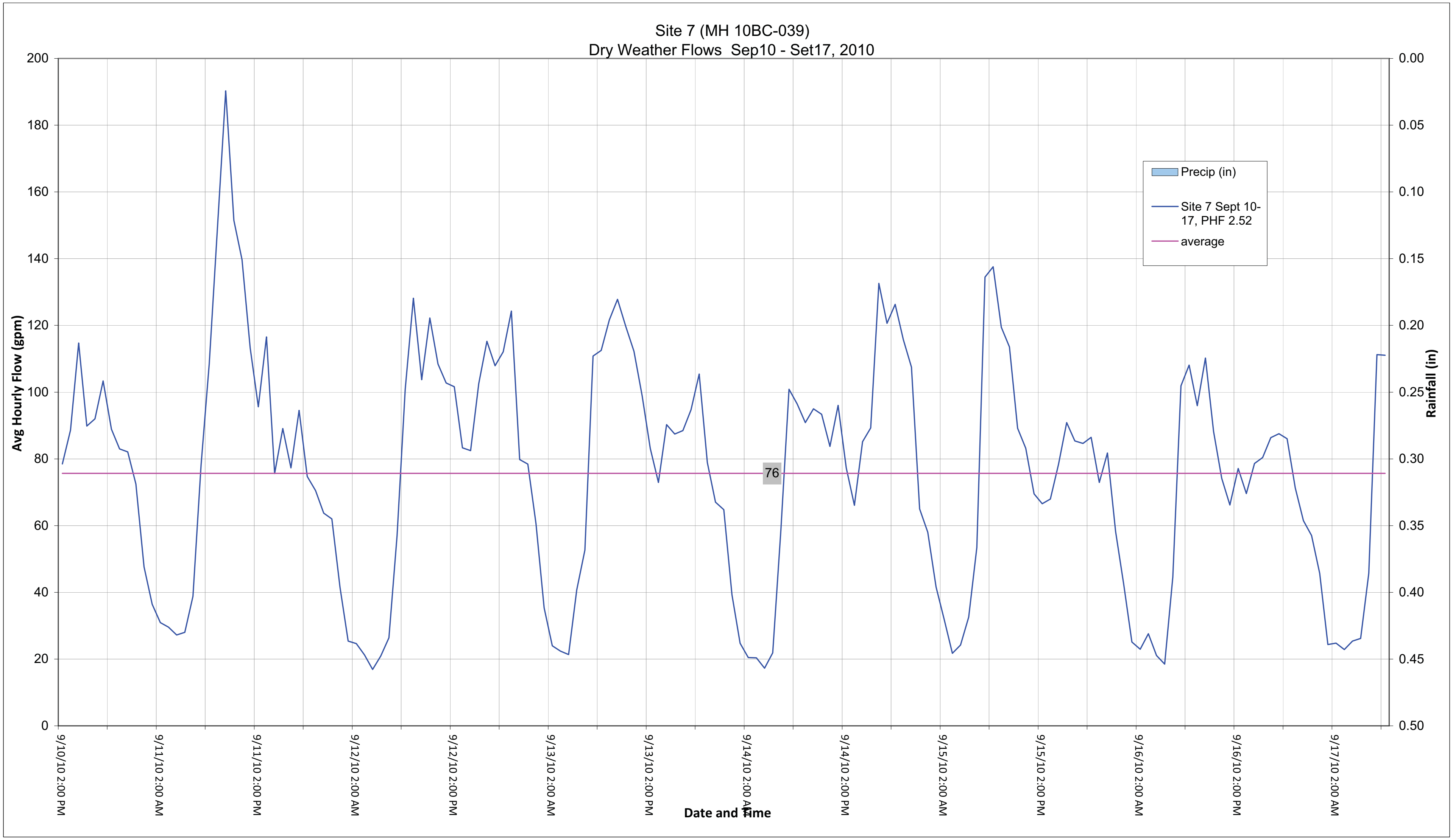
FM 6 (Flow out for Basin 6)
Dry Weather Flows Sept 7 - 17, 2010
Hourly Average Flow (gpm)
450
400
350
300
250
200
150
100
50
0
225
FM6 dry weather, PHF 1.77
Average
9/7/10 1:00 PM
9/8/10 1:00 AM
9/8/10 1:00 PM
9/9/10 1:00 AM
9/9/10 1:00 PM
9/10/10 1:00 AM
9/10/10 1:00 PM
9/11/10 1:00 AM
9/11/10 1:00 PM
9/12/10 1:00 AM
9/12/10 1:00 PM
9/13/10 1:00 AM
9/13/10 1:00 PM
9/14/10 1:00 AM
9/14/10 1:00 PM
9/15/10 1:00 AM
9/15/10 1:00 PM
9/16/10 1:00 AM
9/16/10 1:00 PM
9/17/10 1:00 AM
Date and Time



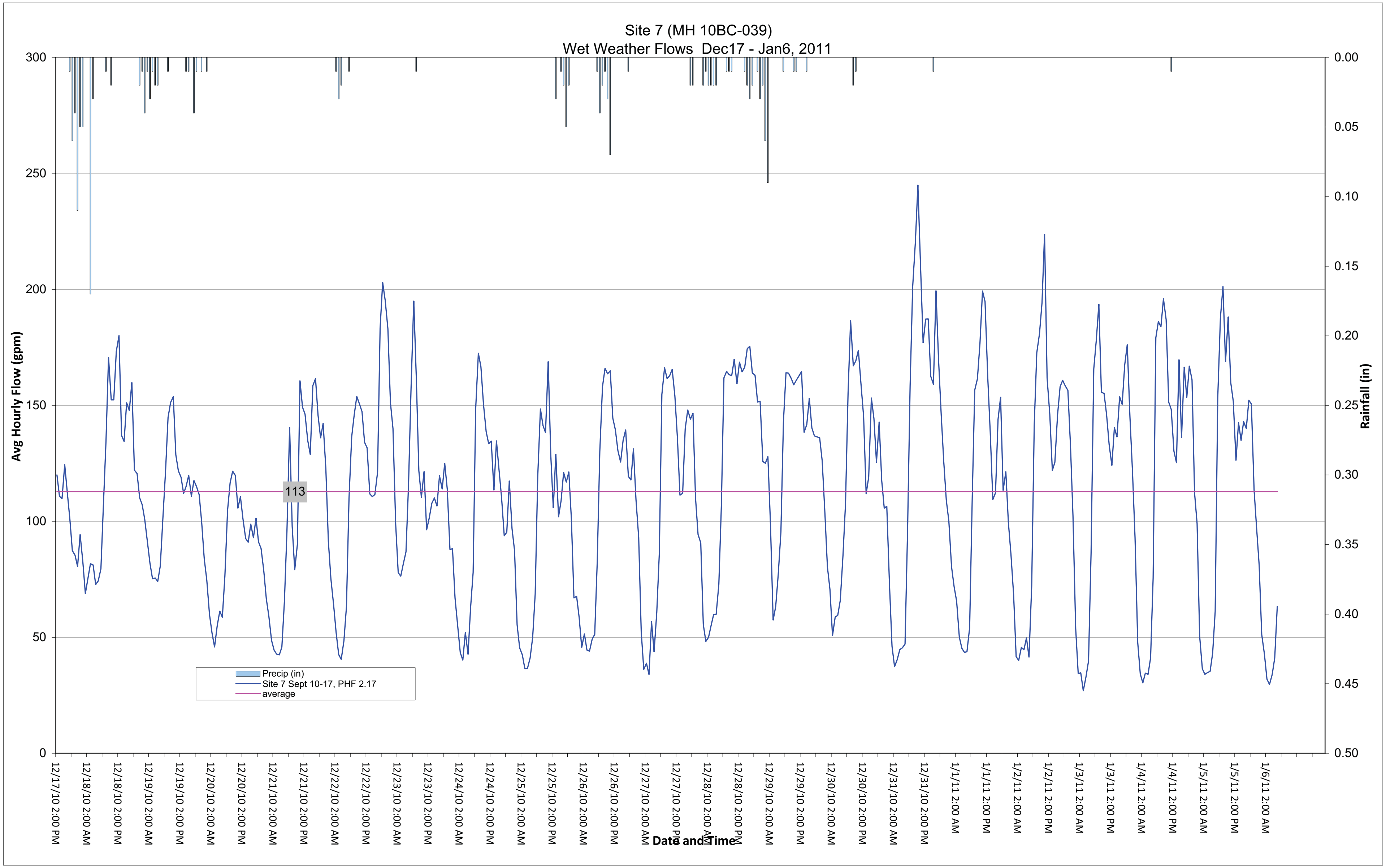
FM 6 (Basin 6 Outflow)
Dry Weather Flows Sept 17-Oct 1, 2010
Flow (gpm)
Rainfall (inches)
Date and Time
232
Rainfall
FM6 dry weather, PHF 1.77
Average
0
50
100
150
200
250
300
350
400
450
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
9/17/10 10:00 AM
9/17/10 10:00 PM
9/18/10 10:00 AM
9/18/10 10:00 PM
9/19/10 10:00 AM
9/19/10 10:00 PM
9/20/10 10:00 AM
9/20/10 10:00 PM
9/21/10 10:00 AM
9/21/10 10:00 PM
9/22/10 10:00 AM
9/22/10 10:00 PM
9/23/10 10:00 AM
9/23/10 10:00 PM
9/24/10 10:00 AM
9/24/10 10:00 PM
9/25/10 10:00 AM
9/25/10 10:00 PM
9/26/10 10:00 AM
9/26/10 10:00 PM
9/27/10 10:00 AM
9/27/10 10:00 PM
9/28/10 10:00 AM
9/28/10 10:00 PM
9/29/10 10:00 AM
9/29/10 10:00 PM
9/30/10 10:00 AM
9/30/10 10:00 PM



Site 6 (Basin 6 Outflow)
Wet Weather Flows Dec 17, 2010-Jan 6, 2011
Flow (gpm)
Rainfall (inches)
Date and Time
199
Rainfall
FM6 wet weather, PHF 1.98
Average



Site 7 (MH 10BC-039)
Dry Weather Flows Sep10 - Set17, 2010
Avg Hourly Flow (gpm)
Rainfall (in)
Date and Time
Precip (in)
Site 7 Sept 10-17, PHF 2.52
average
76
200
180
160
140
120
100
80
60
40
20
0
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
9/10/10 2:00 PM
9/11/10 2:00 AM
9/11/10 2:00 PM
9/12/10 2:00 AM
9/12/10 2:00 PM
9/13/10 2:00 AM
9/13/10 2:00 PM
9/14/10 2:00 AM
9/14/10 2:00 PM
9/15/10 2:00 AM
9/15/10 2:00 PM
9/16/10 2:00 AM
9/16/10 2:00 PM
9/17/10 2:00 AM



Site 7 (MH 10BC-039)
Wet Weather Flows Dec17 - Jan6, 2011
Avg Hourly Flow (gpm)
Rainfall (in)
Date and Time
113
Precip (in)
Site 7 Sept 10-17, PHF 2.17
average
0
50
100
150
200
250
300
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
12/17/10 2:00 PM
12/18/10 2:00 AM
12/18/10 2:00 PM
12/19/10 2:00 AM
12/19/10 2:00 PM
12/20/10 2:00 AM
12/20/10 2:00 PM
12/21/10 2:00 AM
12/21/10 2:00 PM
12/22/10 2:00 AM
12/22/10 2:00 PM
12/23/10 2:00 AM
12/23/10 2:00 PM
12/24/10 2:00 AM
12/24/10 2:00 PM
12/25/10 2:00 AM
12/25/10 2:00 PM
12/26/10 2:00 AM
12/26/10 2:00 PM
12/27/10 2:00 AM
12/27/10 2:00 PM
12/28/10 2:00 AM
12/28/10 2:00 PM
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12/29/10 2:00 PM
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12/30/10 2:00 PM
12/31/10 2:00 AM
12/31/10 2:00 PM
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1/2/11 2:00 PM
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1/3/11 2:00 PM
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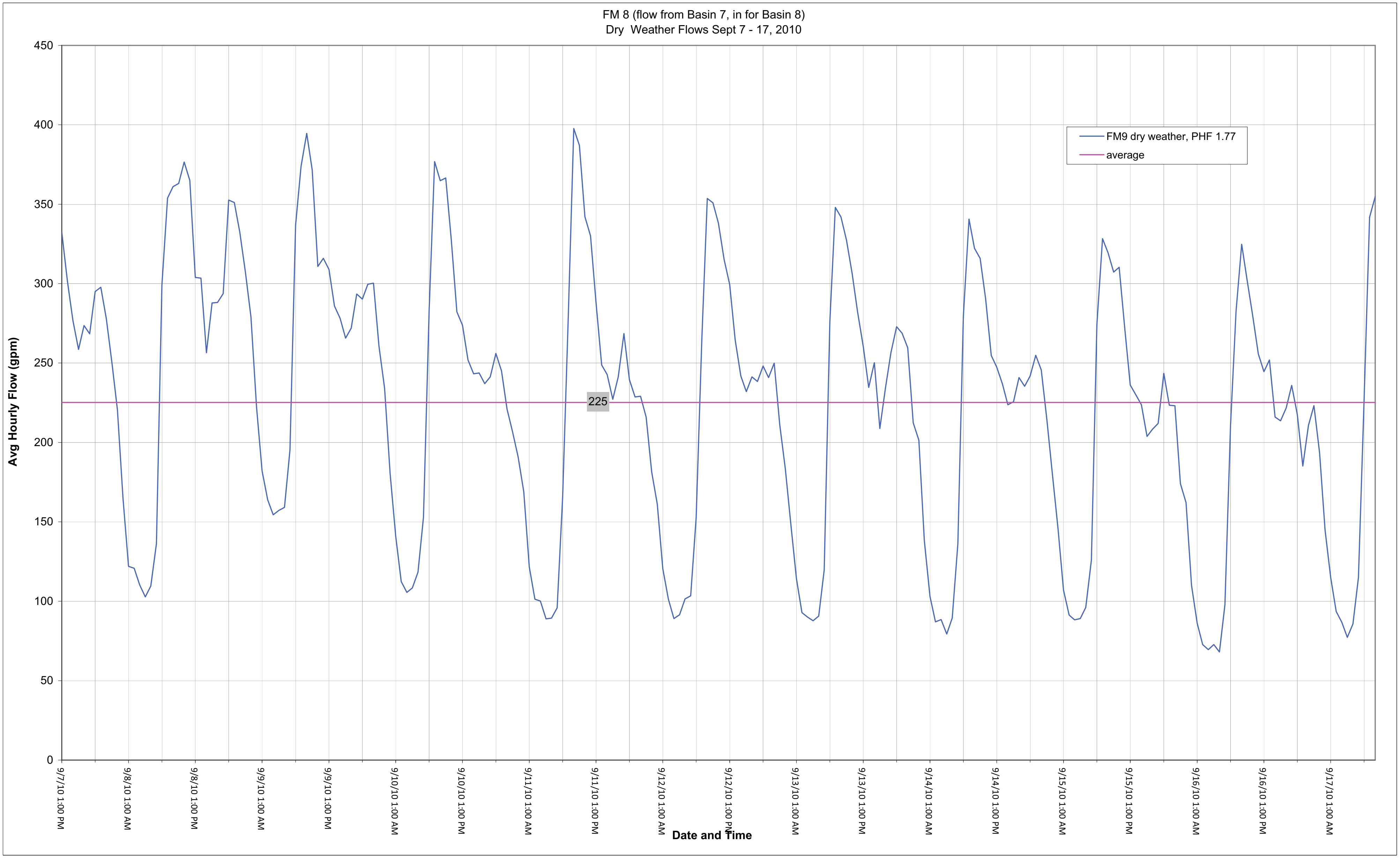
1/5/11 2:00 PM
1/6/11 2:00 AM



FM 8 (flow from Basin 7, in for Basin 8)
Dry Weather Flows Sept 7 - 17, 2010
Avg Hourly Flow (gpm)
450
400
350
300
250
200
150
100
50
0
FM9 dry weather, PHF 1.77
average
225
9/7/10 1:00 PM
9/8/10 1:00 AM
9/8/10 1:00 PM
9/9/10 1:00 AM
9/9/10 1:00 PM
9/10/10 1:00 AM
9/10/10 1:00 PM
9/11/10 1:00 AM
9/11/10 1:00 PM
9/12/10 1:00 AM
9/12/10 1:00 PM
9/13/10 1:00 AM
9/13/10 1:00 PM
9/14/10 1:00 AM
9/14/10 1:00 PM
9/15/10 1:00 AM
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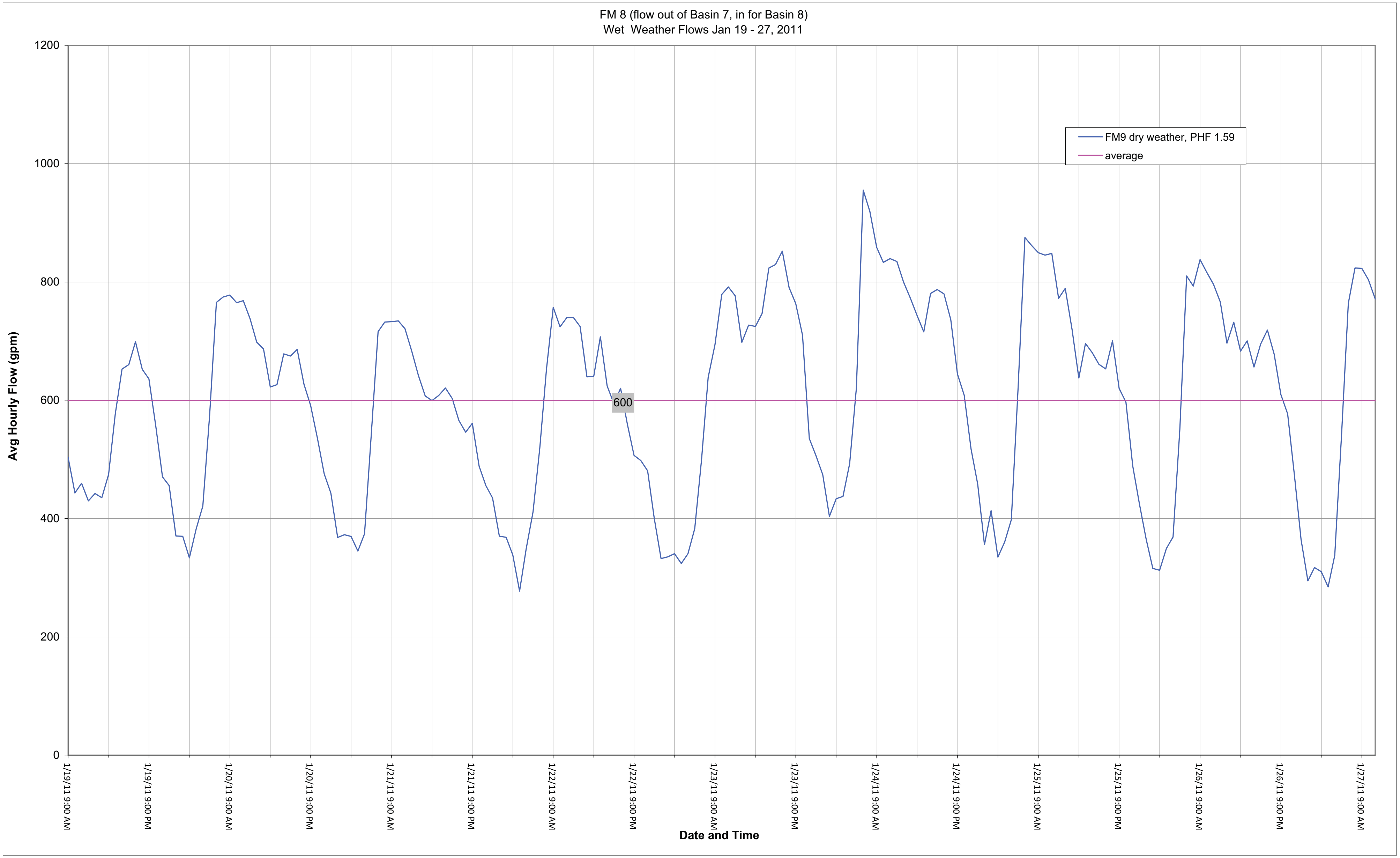
9/16/10 1:00 AM
9/16/10 1:00 PM
9/17/10 1:00 AM
Date and Time


FM 8 (flow out of Basin 7, in for Basin 8)
Wet Weather Flows Jan 19 - 27, 2011
FM9 dry weather, PHF 1.59
average
600
Avg Hourly Flow (gpm)
0
200
400
600
800
1000
1200
Date and Time
1/19/11 9:00 AM
1/19/11 9:00 PM
1/20/11 9:00 AM
1/20/11 9:00 PM
1/21/11 9:00 AM
1/21/11 9:00 PM
1/22/11 9:00 AM
1/22/11 9:00 PM
1/23/11 9:00 AM
1/23/11 9:00 PM
1/24/11 9:00 AM
1/24/11 9:00 PM
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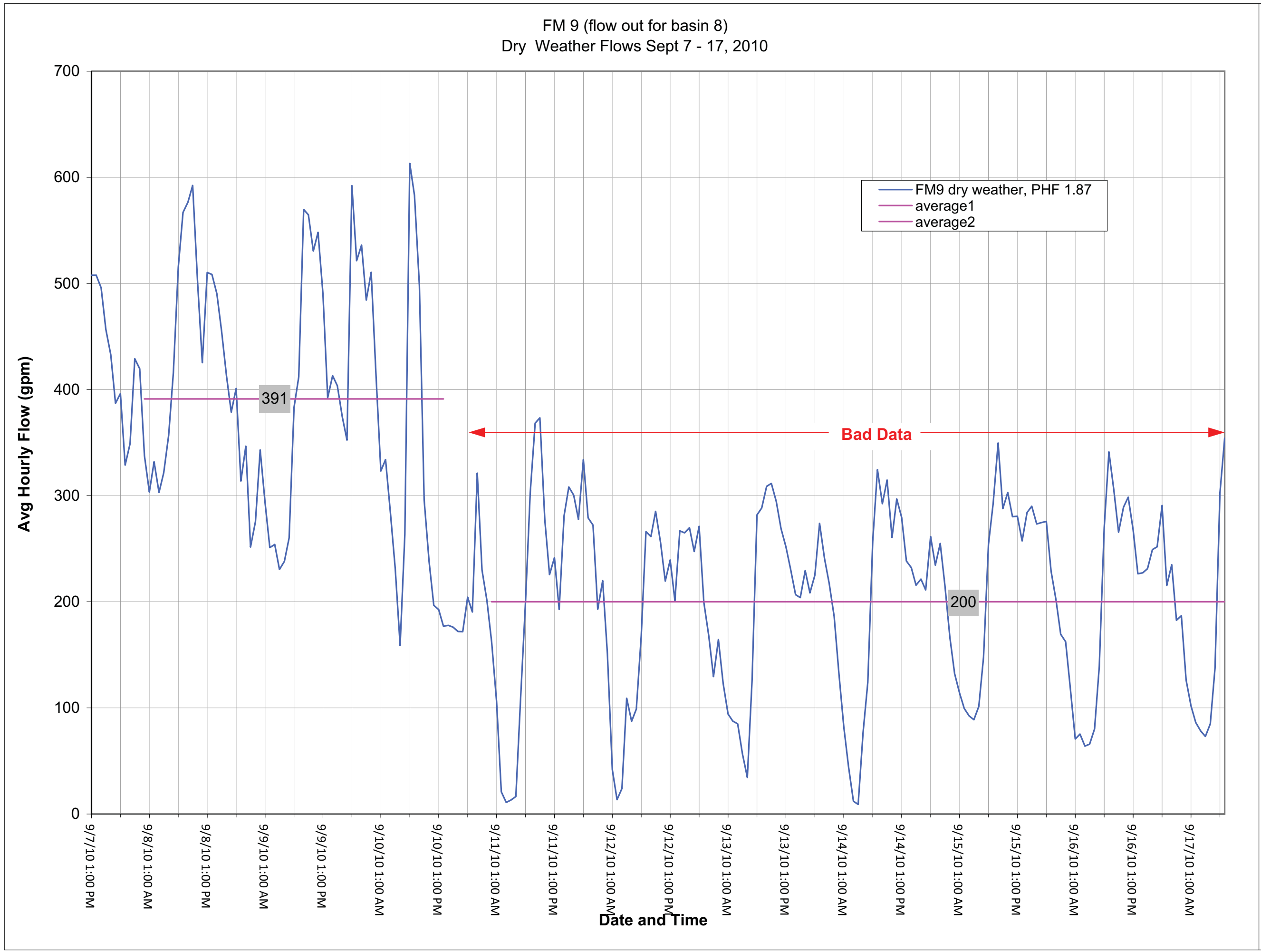
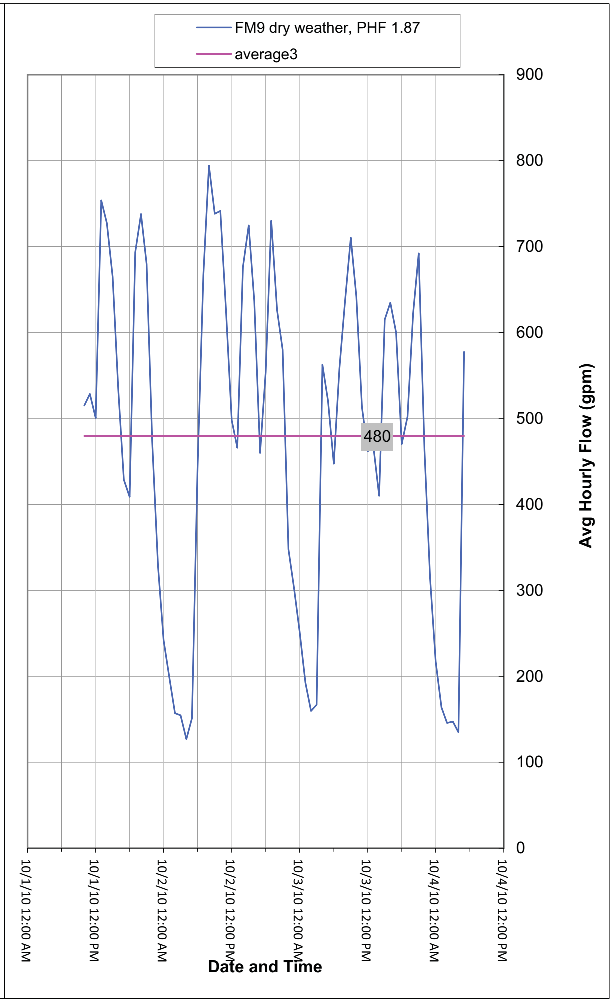
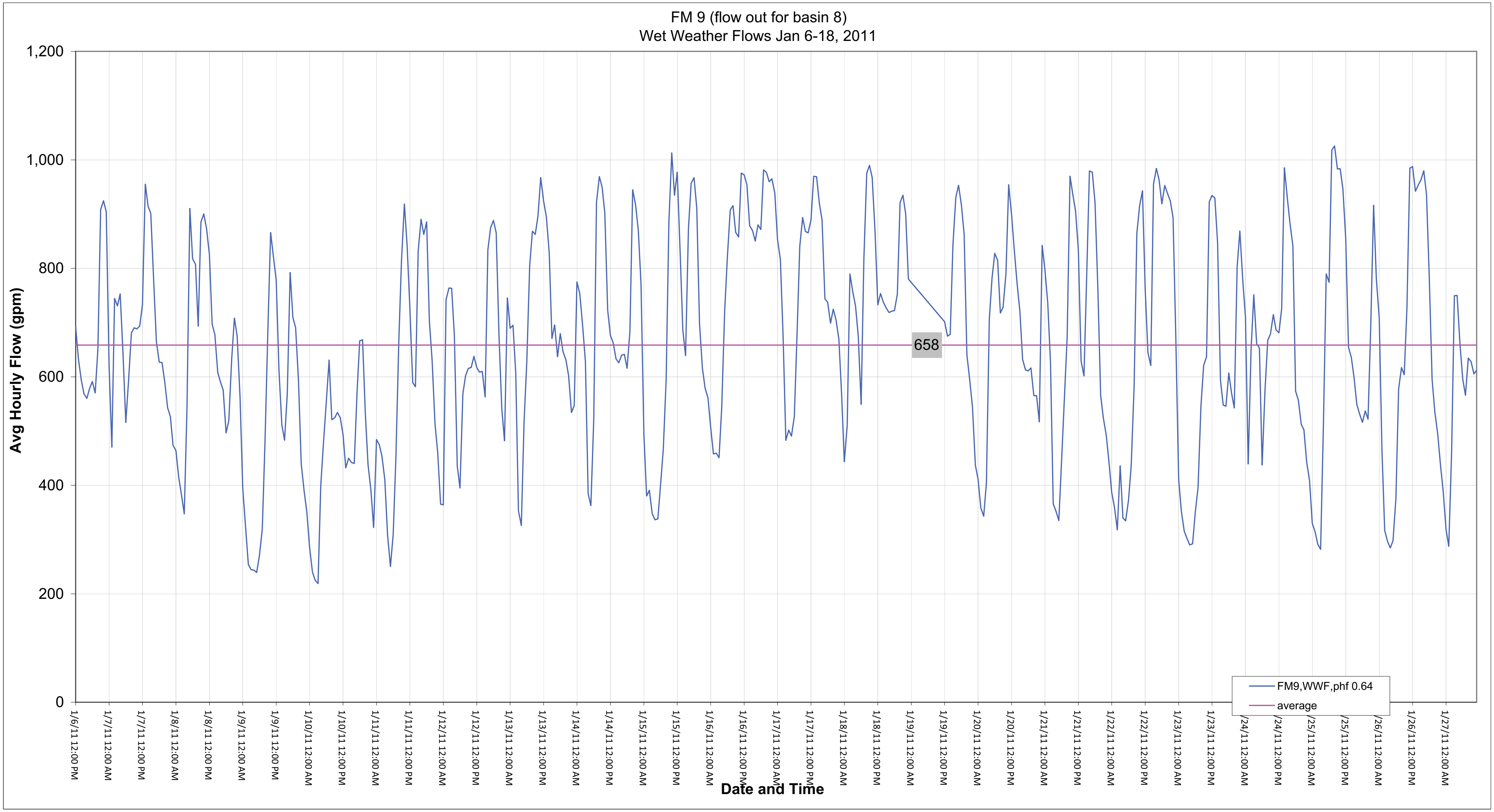
1/27/11 9:00 AM


FM 9 (flow out for basin 8)
Dry Weather Flows Sept 7 - 17, 2010
Avg Hourly Flow (gpm)
700
600
500
400
300
200
100
0
FM9 dry weather, PHF 1.87
average1
average2
391
200
Bad Data
9/7/10 1:00 PM
9/8/10 1:00 AM
9/8/10 1:00 PM
9/9/10 1:00 AM
9/9/10 1:00 PM
9/10/10 1:00 AM
9/10/10 1:00 PM
9/11/10 1:00 AM
9/11/10 1:00 PM
9/12/10 1:00 AM
9/12/10 1:00 PM
9/13/10 1:00 AM
9/13/10 1:00 PM
9/14/10 1:00 AM
9/14/10 1:00 PM
9/15/10 1:00 AM
9/15/10 1:00 PM
9/16/10 1:00 AM
9/16/10 1:00 PM
9/17/10 1:00 AM
Date and Time

FM9 dry weather, PHF 1.87
average3
900
800
700
600
500
400
300
200
100
0
480
Avg Hourly Flow (gpm)
10/1/10 12:00 AM
10/1/10 12:00 PM
10/2/10 12:00 AM
10/2/10 12:00 PM
10/3/10 12:00 AM
10/3/10 12:00 PM
10/4/10 12:00 AM
10/4/10 12:00 PM
Date and Time


FM 9 (flow out for basin 8)
Wet Weather Flows Jan 6-18, 2011
Avg Hourly Flow (gpm)
1,200
1,000
800
600
400
200
0
658
FM9,WWF,phf 0.64
average
Date and Time
1/6/11 12:00 PM
1/7/11 12:00 AM
1/7/11 12:00 PM
1/8/11 12:00 AM
1/8/11 12:00 PM
1/9/11 12:00 AM
1/9/11 12:00 PM
1/10/11 12:00 AM
1/10/11 12:00 PM
1/11/11 12:00 AM
1/11/11 12:00 PM
1/12/11 12:00 AM
1/12/11 12:00 PM
1/13/11 12:00 AM
1/13/11 12:00 PM
1/14/11 12:00 AM
1/14/11 12:00 PM
1/15/11 12:00 AM
1/15/11 12:00 PM
1/16/11 12:00 AM
1/16/11 12:00 PM
1/17/11 12:00 AM
1/17/11 12:00 PM
1/18/11 12:00 AM
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1/23/11 12:00 PM
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1/24/11 12:00 PM
1/25/11 12:00 AM
1/25/11 12:00 PM
1/26/11 12:00 AM
1/26/11 12:00 PM
1/27/11 12:00 AM

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APPENDIX D

MODEL CONSTRUCTION & SCENARIO RESULTS

- DRY CALIBRATION DATA
- EXISTING & FUTURE CAPACITY ANALYSIS (MODEL RESULTS)

KELLER associates

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Dry Calibration Data

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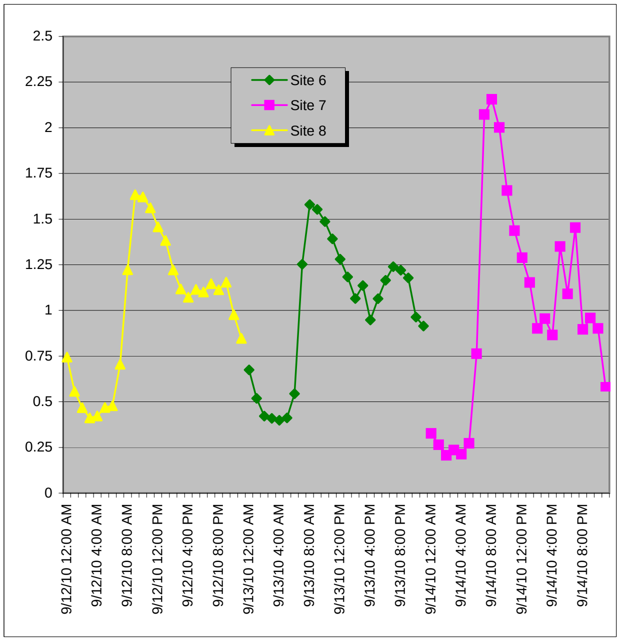
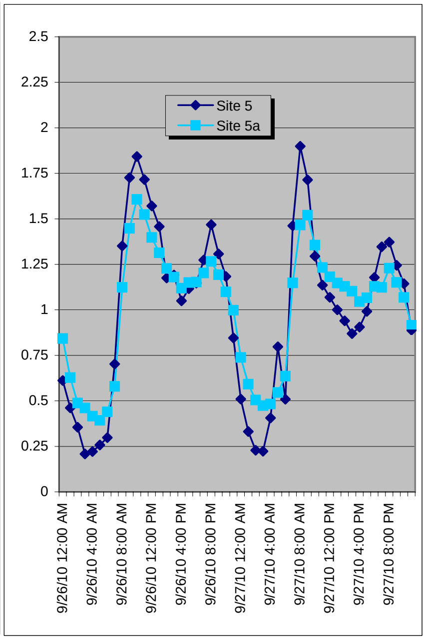
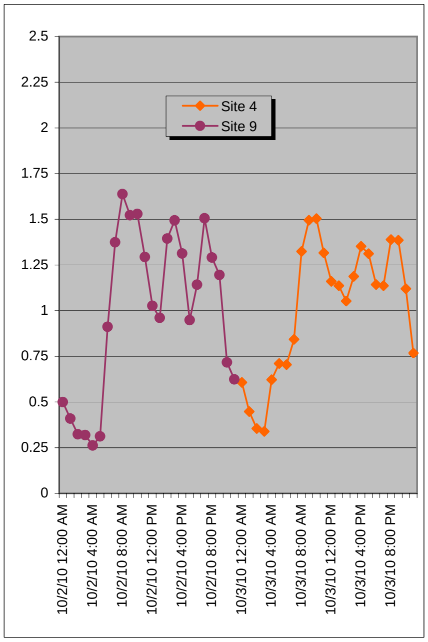
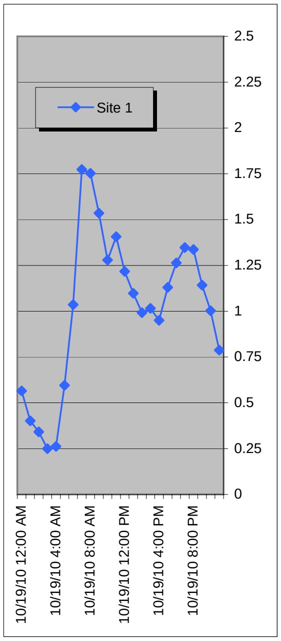
City of Ashland, OR
WW Model Calibration

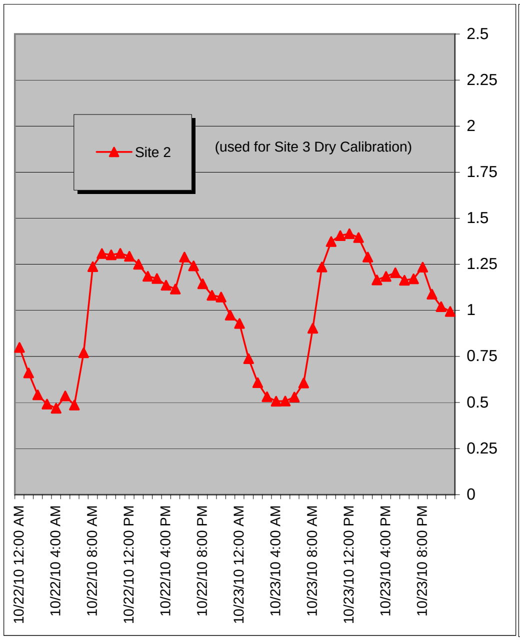
Est: 6/1/11 Updated: 6/7/ Updated: 8/3/12
By: KTK By: KTK By: KTK

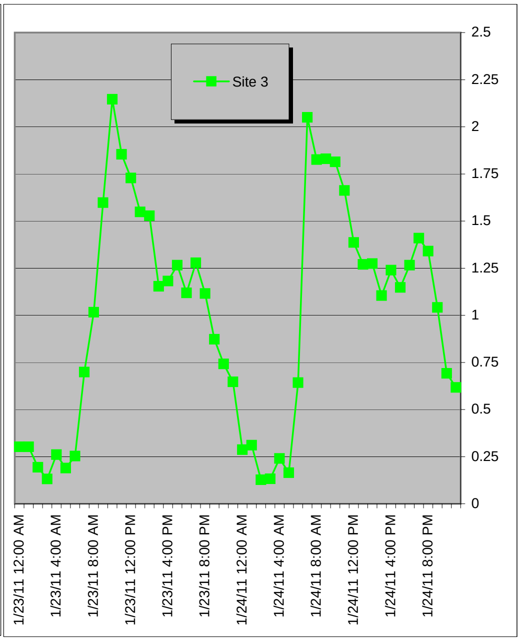












Dry Calibration Basin: Site 1 - Basin 1

copied from Flow Data File:
"Basin1_FlowMonitoring_NEW.xls" at J:\210055\Design\KellerCollectionModel\ModelCalibration\Basin_1

WWTP Typ. Dry Day Avg = 1458 gpm
WWTP Calibration Day Avg = 1313 gpm
Normalizing Factor Calc = 1.11

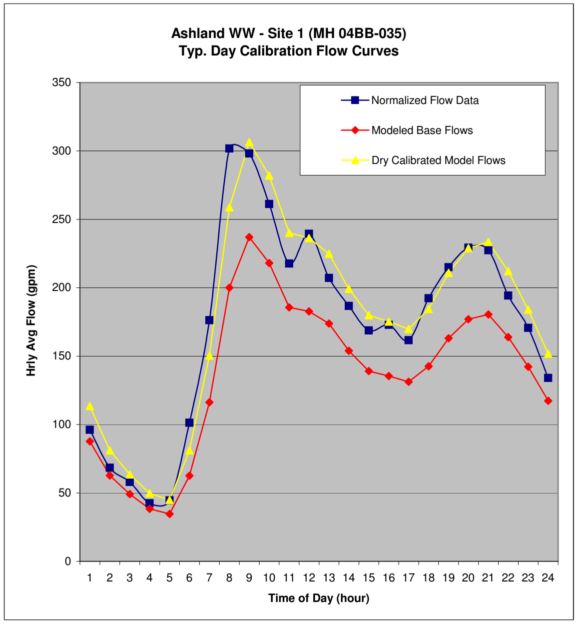
Normalizing Factor = 1.11

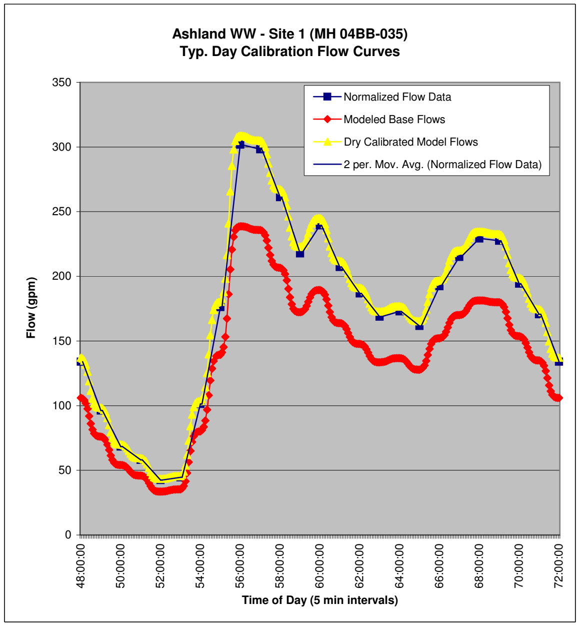
Load Shift Factor = 1.3 | 1.3

Typ Day Used to Calibrate Model

Date	Time	Hrly Avg Flow New mislabeled Site 3 (gpm)	Normalized Flow Data (gpm)		Modeled Base Flows (gpm) 8/3/2011	Modeled Base Flows (gpm) 8/3/2011			Dry Calibrated Model Flows (gpm) 8/3/2011	Dry Calibrated Model Flows (gpm) 8/3/2011	
19-Oct-10	0:00	86.645	96.18		87.73	87.73			113.48	113.48	
19-Oct-10	1:00	61.612	68.39		62.66	62.66			81.05	81.05	
19-Oct-10	2:00	52.313	58.07		49.11	49.11			63.53	63.53	
19-Oct-10	3:00	38.256	42.46		38.40	38.40			49.67	49.67	
19-Oct-10	4:00	40.159	44.58		34.61	34.61			44.77	44.77	
19-Oct-10	5:00	91.243	101.28		62.63	62.63			81.01	81.01	
19-Oct-10	6:00	158.773	176.24		116.29	116.29			150.43	150.43	
19-Oct-10	7:00	271.892	301.80	MAX	200.00	200.00			258.72	258.72	
19-Oct-10	8:00	268.65	298.20		236.97	236.97	MAX		306.53	306.53	MAX
19-Oct-10	9:00	235.324	261.21		218.00	218.00			282.00	282.00	
19-Oct-10	10:00	196.153	217.73		185.61	185.61			240.10	240.10	
19-Oct-10	11:00	215.715	239.44		182.69	182.69			236.33	236.33	
19-Oct-10	12:00	186.612	207.14		173.79	173.79			224.81	224.81	
19-Oct-10	13:00	168.221	186.73		153.98	153.98			199.18	199.18	
19-Oct-10	14:00	152.095	168.83		139.05	139.05			179.87	179.87	
19-Oct-10	15:00	155.748	172.88		135.49	135.49			175.26	175.26	
19-Oct-10	16:00	145.595	161.61		131.30	131.30			169.84	169.84	
19-Oct-10	17:00	173.264	192.32		142.65	142.65			184.53	184.53	
19-Oct-10	18:00	193.686	214.99		163.06	163.06			210.93	210.93	
19-Oct-10	19:00	206.556	229.28		176.94	176.94			228.88	228.88	
19-Oct-10	20:00	204.911	227.45		180.46	180.46			233.44	233.44	
19-Oct-10	21:00	175.072	194.33		163.91	163.91			212.03	212.03	
19-Oct-10	22:00	153.726	170.64		142.27	142.27			184.03	184.03	
19-Oct-10	23:00	120.725	134.00		117.28	117.28			151.71	151.71	
								Factor			
	MAX	271.892	301.80	MAX	236.97	236.97		1.273588	306.53	306.53	
	AVG	156.3728	173.57	AVG	137.29	137.29		1.264313	177.59	177.59	
	MIN	38.256	42.46	MIN	34.61	34.61		1.227057	44.77	44.77	

		(gpm)	Modeled Base Flows (gpm)	Model Moving Hrly Avg (gpm)	Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)
48:00:00	**120.73**	**134.00**	106.01	114.91	137.128	148.65
48:05:00	117.89		105.45	112.60	136.403	145.65
48:10:00	115.05		104.22	110.37	134.815	142.77
48:15:00	112.21		101.82	108.29	131.71	140.08
48:20:00	109.37		97.56	106.31	126.203	137.52
48:25:00	106.53		91.89	104.32	118.87	134.95
48:30:00	103.69		86.12	102.23	111.405	132.24
48:35:00	100.85		81.50	99.99	105.429	129.34
48:40:00	98.01		78.55	97.62	101.607	126.28
48:45:00	95.17		77.02	95.19	99.631	123.13
48:50:00	92.33		76.38	92.71	98.798	119.93
48:55:00	89.49		76.15	90.22	98.508	116.71
49:00:00	**86.645**	**96.17595**	76.09	87.73	98.425	113.48
49:05:00	84.56		75.67	85.25	97.887	110.27
49:10:00	82.47		74.77	82.79	96.719	107.10
49:15:00	80.39		73.01	80.39	94.438	103.99
49:20:00	78.30		69.88	78.09	90.393	101.01
49:25:00	76.21		65.72	75.90	85.007	98.19
49:30:00	74.13		61.48	73.85	79.523	95.53
49:35:00	72.04		58.08	71.90	75.134	93.01
49:40:00	69.96		55.91	70.01	72.327	90.57
49:45:00	67.87		54.79	68.16	70.875	88.17
49:50:00	65.78		54.32	66.32	70.263	85.79
49:55:00	63.70		54.15	64.49	70.05	83.42
50:00:00	**61.61**	**68.39**	54.11	62.66	69.989	81.05
50:05:00	60.84		53.95	60.85	69.782	78.71
50:10:00	60.06		53.61	59.08	69.347	76.43
50:15:00	59.29		52.95	57.41	68.499	74.27
50:20:00	58.51		51.79	55.90	66.997	72.32
50:25:00	57.74		50.25	54.62	64.996	70.65
50:30:00	56.96		48.67	53.55	62.959	69.27
50:35:00	56.19		47.41	52.66	61.329	68.12





Dry Calibration Basin: Site 2 - Basin 2

copied from Flow Data File:
"Basin2_FlowMonitoring.xls" at J:\210055\Design\KellerCollectionModel\ModelCalibration\Basin_2

WWTP Typ. Dry Day Avg = 1458 gpm
WWTP Calibration Day Avg = 1313 gpm
Normalizing Factor Calc = 1.11

Normalizing Factor = 0.26428571

Load Shift Factor = upstream c upstream changes

Date	Time	Hrly Avg Flow (gpm)		Reduced & Normalized Flow Data (gpm)		Modeled Base Flows (gpm) 5/19/2011	Modeled Base Flows (gpm) 5/19/2011		Dry Calibrated Model Flows (gpm) 6/6/2011	Dry Calibrated Model Flows (gpm) 6/6/2011	
22-Oct-10	0:00	2313.134	Typ Day Used to Calibrate Model	611.33		701.79	701.79		791.98	791.98	
22-Oct-10	1:00	1913.788		505.79		582.01	582.01		648.12	648.12	
22-Oct-10	2:00	1568.714		414.59		472.38	472.38		523.65	523.65	
22-Oct-10	3:00	1423.083		376.10		364.68	364.68		404.80	404.80	
22-Oct-10	4:00	1358.443		359.02		311.76	311.76		351.00	351.00	
22-Oct-10	5:00	1551.489		410.04		286.63	286.63		323.24	323.24	
22-Oct-10	6:00	1406.996		371.85		278.09	278.09		317.95	317.95	
22-Oct-10	7:00	2230.035		589.37		297.91	297.91		373.31	373.31	
22-Oct-10	8:00	3583.509		947.07		426.09	426.09		591.37	591.37	
22-Oct-10	9:00	3790.212		1001.70		709.75	709.75		911.21	911.21	
22-Oct-10	10:00	3769.466		996.22		923.27	923.27		1,134.98	1,134.98	
22-Oct-10	11:00	3792.196		1002.22	MAX	1,063.04	1,063.04	MAX	1,265.57	1,265.57	MAX
22-Oct-10	12:00	3746.348		990.11		1,042.75	1,042.75		1,230.30	1,230.30	
22-Oct-10	13:00	3620.885		956.95		982.14	982.14		1,151.88	1,151.88	
22-Oct-10	14:00	3432.833		907.25		879.09	879.09		1,032.66	1,032.66	
22-Oct-10	15:00	3396.073		897.53		820.59	820.59		952.94	952.94	
22-Oct-10	16:00	3291.213		869.82		772.77	772.77		901.94	901.94	
22-Oct-10	17:00	3230.434		853.76		748.81	748.81		870.78	870.78	
22-Oct-10	18:00	3734.019		986.85		734.35	734.35		884.44	884.44	
22-Oct-10	19:00	3596.726		950.56		745.03	745.03		886.98	886.98	
22-Oct-10	20:00	3313.25		875.64		823.43	823.43		983.04	983.04	
22-Oct-10	21:00	3133.78		828.21		810.85	810.85		943.23	943.23	
22-Oct-10	22:00	3104.781		820.55		799.04	799.04		927.92	927.92	
22-Oct-10	23:00	2822.448		745.93		799.27	799.27	Factor	918.70	918.70	
	MAX	3792.196		1002.22	MAX	1063.04	1063.04	0.942786	1265.57	1265.57	
	AVG	2880.16063		761.19	AVG	682.31	682.31	1.115594	805.08	805.08	
	MIN	1358.443	BAD DATA	359.02	MIN	278.09	278.09	1.291022	317.95	317.95	

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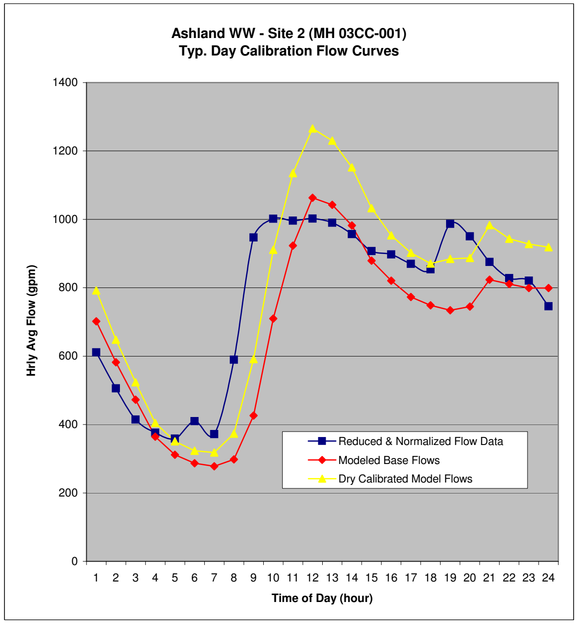
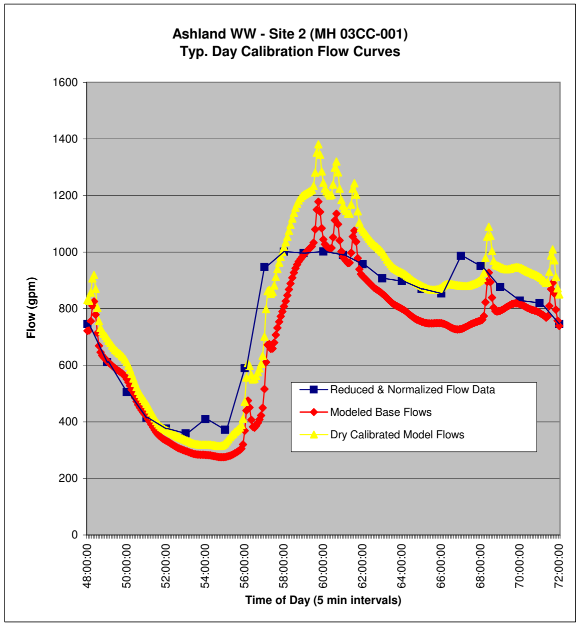
Time	Hrly Avg Flow	(gpm)	Modeled Base Flows (gpm)	Model Moving Hrly Avg (gpm)	Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)
48:00:00	**2822.45**	**745.93**	722.05	794.26	830.782	912.27
48:05:00	2780.01		723.24	789.71	828.386	906.08
48:10:00	2737.56		756.02	788.28	857.18	902.78
48:15:00	2695.12		810.01	791.74	907.21	904.14
48:20:00	2652.68		825.24	796.13	918.92	906.24
48:25:00	2610.23		779.28	793.60	870.13	901.29
48:30:00	2567.79		714.08	780.70	802.835	885.86
48:35:00	2525.35		668.64	762.15	755.92	864.74
48:40:00	2482.91		646.09	744.85	732.297	844.85
48:45:00	2440.46		634.94	731.36	720.206	828.81
48:50:00	2398.02		627.53	720.51	711.642	815.47
48:55:00	2355.58		621.20	710.69	703.661	803.26
49:00:00	**2313.134**	**611.328271**	615.24	701.79	695.406	791.98
49:05:00	2279.86		609.47	692.31	686.761	780.18
49:10:00	2246.58		603.89	679.63	678.008	765.25
49:15:00	2213.30		598.56	662.01	669.609	745.45
49:20:00	2180.02		593.53	642.70	661.943	724.03
49:25:00	2146.74		588.81	626.83	655.167	706.12
49:30:00	2113.46		584.37	616.02	649.246	693.32
49:35:00	2080.18		580.17	608.65	644.007	684.00
49:40:00	2046.90		576.07	602.81	639.122	676.23
49:45:00	2013.62		571.72	597.55	634.052	669.05
49:50:00	1980.35		566.53	592.46	628.095	662.09
49:55:00	1947.07		559.85	587.35	620.535	655.16
50:00:00	**1913.79**	**505.79**	551.16	582.01	610.836	648.12
50:05:00	1885.03		540.30	576.25	598.818	640.79
50:10:00	1856.28		527.58	569.89	584.742	633.01
50:15:00	1827.52		513.62	562.81	569.26	624.65
50:20:00	1798.76		499.25	554.95	553.246	615.59
50:25:00	1770.01		485.20	546.32	537.598	605.80
50:30:00	1741.25		472.13	536.96	523.091	595.28
50:35:00	1712.49		460.46	526.99	510.248	584.14





Dry Calibration Basin: Site 3 - Basin 3

copied from Flow Data File: "Basin3_FlowMonitoring_NEW.xls" at J:\210055\Design\KellerCollectionModel\ModelCalibration\Basin_3

WWTP Typ. Dry Day Avg = 1458 gpm
WWTP Calibration Day Avg = 1458 gpm
Normalizing Factor Calc = 1.00

Normalizing Factor = 1

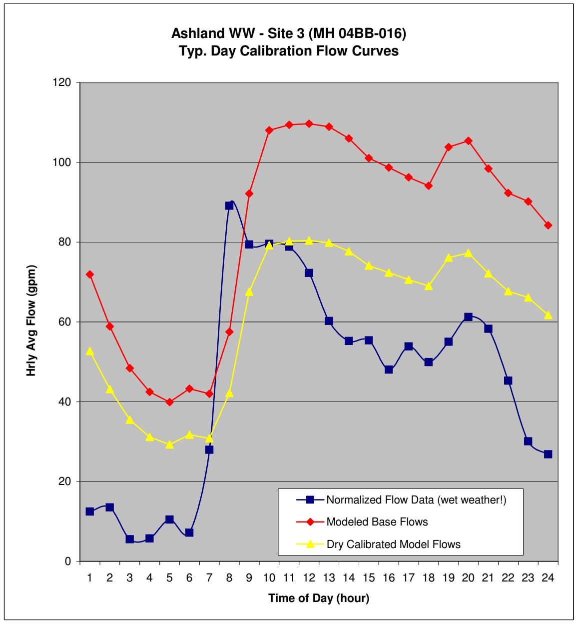
Load Shift Factor = 0.73 | 0.73

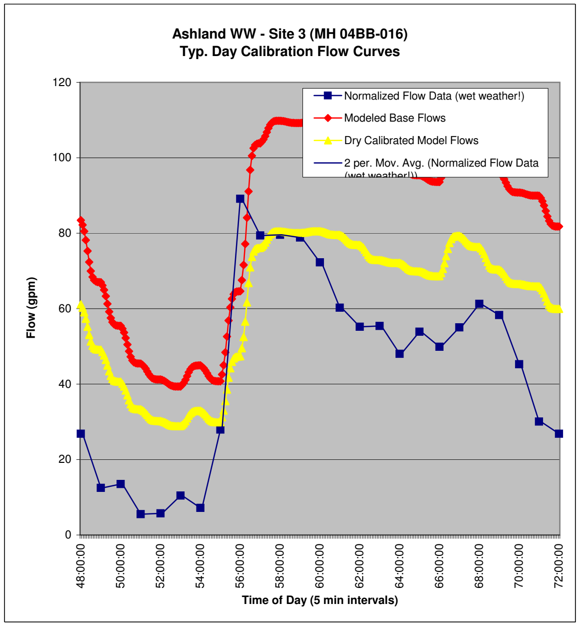
NOTE: matched averages not peak because Flow Data is wet weather

Typ Day Used to Calibrate Model

Date	Time	Hrly Avg Flow New mislabeled Site 1 (gpm)	Normalized Flow Data (wet weather!) (gpm)		8/3/2011 Modeled Base Flows (gpm)	8/3/2011 Modeled Base Flows (gpm)		8/3/2011 Dry Calibrated Model Flows (gpm)	8/3/2011 Dry Calibrated Model Flows (gpm)	
24-Jan-11	0:00	12.48479	12.48		71.90	71.90		52.71	52.71	
24-Jan-11	1:00	13.50774	13.51		58.89	58.89		43.17	43.17	
24-Jan-11	2:00	5.532592	5.53		48.42	48.42		35.50	35.50	
24-Jan-11	3:00	5.740728	5.74		42.49	42.49		31.15	31.15	
24-Jan-11	4:00	10.45593	10.46		39.92	39.92		29.26	29.26	
24-Jan-11	5:00	7.170009	7.17		43.29	43.29		31.74	31.74	
24-Jan-11	6:00	27.95112	27.95		42.01	42.01		30.80	30.80	
24-Jan-11	7:00	89.1247	89.12	MAX	57.53	57.53		42.18	42.18	
24-Jan-11	8:00	79.40293	79.40		92.18	92.18		67.58	67.58	
24-Jan-11	9:00	79.58524	79.59		108.04	108.04		79.21	79.21	
24-Jan-11	10:00	78.86311	78.86		109.40	109.40		80.20	80.20	
24-Jan-11	11:00	72.25788	72.26		109.68	109.68	MAX	80.41	80.41	MAX
24-Jan-11	12:00	60.25043	60.25		108.94	108.94		79.87	79.87	
24-Jan-11	13:00	55.21067	55.21		105.99	105.99		77.71	77.71	
24-Jan-11	14:00	55.40195	55.40		101.08	101.08		74.11	74.11	
24-Jan-11	15:00	48.03116	48.03		98.72	98.72		72.37	72.37	
24-Jan-11	16:00	53.87186	53.87		96.26	96.26		70.57	70.57	
24-Jan-11	17:00	49.91484	49.91		94.12	94.12		69.00	69.00	
24-Jan-11	18:00	55.02222	55.02		103.85	103.85		76.14	76.14	
24-Jan-11	19:00	61.2542	61.25		105.39	105.39		77.27	77.27	
24-Jan-11	20:00	58.29149	58.29		98.44	98.44		72.17	72.17	
24-Jan-11	21:00	45.27375	45.27		92.34	92.34		67.70	67.70	
24-Jan-11	22:00	30.06246	30.06		90.21	90.21		66.13	66.13	
24-Jan-11	23:00	26.83656	26.84		84.21	84.21		61.74	61.74	
							Factor			
	MAX	89.1247	89.12	MAX	109.68	109.68	0.812588	80.41	80.41	
	AVG	45.06243	45.06	AVG	83.47	83.47	0.539858	61.19	61.19	
	MIN	5.532592	5.53	MIN	39.92	39.92	0.138604	29.26	29.26	

Time		(gpm)	Modeled Base Flows (gpm)	Model Moving Hrly Avg (gpm)	Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)
48:00:00	**26.84**	**26.84**	83.43	83.72	61.165	61.37
48:05:00	25.64		82.20	83.19	60.265	60.99
48:10:00	24.44		80.51	82.62	59.021	60.57
48:15:00	23.25		78.18	81.98	57.314	60.10
48:20:00	22.05		75.28	81.22	55.19	59.54
48:25:00	20.86		72.36	80.31	53.047	58.88
48:30:00	19.66		69.98	79.27	51.302	58.11
48:35:00	18.46		68.41	78.13	50.149	57.28
48:40:00	17.27		67.56	76.94	49.533	56.41
48:45:00	16.07		67.20	75.72	49.265	55.51
48:50:00	14.88		67.07	74.50	49.171	54.61
48:55:00	13.68		67.03	73.27	49.143	53.71
49:00:00	**12.48479**	**12.4847949**	67.02	71.90	49.134	52.71
49:05:00	12.57		66.15	70.56	48.498	51.73
49:10:00	12.66		64.96	69.27	47.621	50.78
49:15:00	12.74		63.32	68.03	46.418	49.87
49:20:00	12.83		61.27	66.86	44.92	49.02
49:25:00	12.91		59.21	65.76	43.41	48.21
49:30:00	13.00		57.53	64.73	42.18	47.45
49:35:00	13.08		56.43	63.73	41.367	46.72
49:40:00	13.17		55.83	62.75	40.933	46.01
49:45:00	13.25		55.58	61.78	40.744	45.29
49:50:00	13.34		55.49	60.82	40.677	44.59
49:55:00	13.42		55.46	59.85	40.658	43.88
50:00:00	**13.51**	**13.51**	55.45	58.89	40.651	43.17
50:05:00	12.84		54.70	57.93	40.102	42.47
50:10:00	12.18		53.67	56.99	39.344	41.78
50:15:00	11.51		52.25	56.07	38.304	41.11
50:20:00	10.85		50.48	55.17	37.01	40.45
50:25:00	10.18		48.70	54.30	35.705	39.81
50:30:00	9.52		47.25	53.44	34.642	39.18
50:35:00	8.86		46.30	52.60	33.94	38.56





Dry Calibration Basin: Site 4 - Basin 4

copied from Flow Data File:
"Site4.xls" at J:\210055\Design\KellerCollectionModel\ModelCalibration\Basin_4

WWTP Typ. Dry Day Avg = 1458 gpm
WWTP Calibration Day Avg = 1472 gpm
Normalizing Factor Calc = 0.99

Normalizing Factor = 0.99

Load Shift Factor = upstream changes to Site 9

Typ Day Used to Calibrate Model

Date	Time	Hrly Avg Flow (gpm)	Normalized Flow Data (gpm)		5/19/2011 Modeled Base Flows (gpm)	5/19/2011 Modeled Base Flows (gpm)		6/2/2011 Dry Calibrated Model Flows (gpm)	6/2/2011 Dry Calibrated Model Flows (gpm)	
3-Oct-10	0:00	312.494	309.37		343.18	343.18		351.08	351.08	
3-Oct-10	1:00	230.526	228.22		285.25	285.25		295.02	295.02	
3-Oct-10	2:00	182.65	180.82		224.24	224.24		230.32	230.32	
3-Oct-10	3:00	174.683	172.94		194.97	194.97		197.90	197.90	
3-Oct-10	4:00	320.184	316.98		180.06	180.06		182.64	182.64	
3-Oct-10	5:00	365.755	362.10		189.31	189.31		191.63	191.63	
3-Oct-10	6:00	362.953	359.32		323.56	323.56		294.78	294.78	
3-Oct-10	7:00	433.883	429.54		501.51	501.51		429.29	429.29	
3-Oct-10	8:00	681.591	674.78		663.55	663.55		576.24	576.24	
3-Oct-10	9:00	769.665	761.97		762.53	762.53		701.73	701.73	
3-Oct-10	10:00	774.167	766.43	MAX	813.75	813.75	MAX	778.61	778.61	MAX
3-Oct-10	11:00	677.69	670.91		767.46	767.46		746.09	746.09	
3-Oct-10	12:00	597.549	591.57		668.85	668.85		666.60	666.60	
3-Oct-10	13:00	585.302	579.45		603.37	603.37		608.35	608.35	
3-Oct-10	14:00	541.697	536.28		666.01	666.01		645.43	645.43	
3-Oct-10	15:00	611.317	605.20		719.91	719.91		670.67	670.67	
3-Oct-10	16:00	696.206	689.24		684.40	684.40		635.36	635.36	
3-Oct-10	17:00	675.616	668.86		581.90	581.90		555.97	555.97	
3-Oct-10	18:00	588.175	582.29		574.54	574.54		552.55	552.55	
3-Oct-10	19:00	585.332	579.48		669.58	669.58		621.29	621.29	
3-Oct-10	20:00	715.115	707.96		684.50	684.50		637.09	637.09	
3-Oct-10	21:00	713.3	706.17		637.22	637.22		601.11	601.11	
3-Oct-10	22:00	576.468	570.70		526.22	526.22		520.02	520.02	
3-Oct-10	23:00	395.219	391.27		409.78	409.78		416.03	416.03	
							Factor			Factor
	MAX	774.167	766.43	**MAX**	813.75	813.75	0.941846	778.61	778.61	0.98435
	AVG	523.64738	518.41	**AVG**	528.15	528.15	0.981556	504.41	504.41	1.02776
	MIN	174.683	172.94	**MIN**	180.06	180.06	0.960448	182.64	182.64	0.946853

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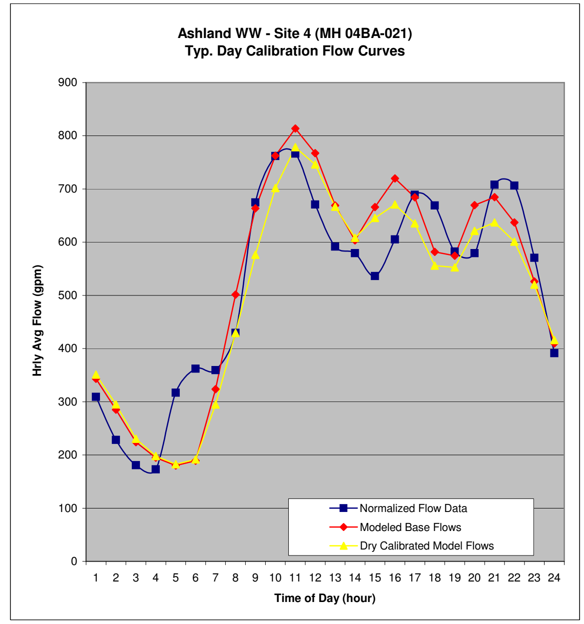
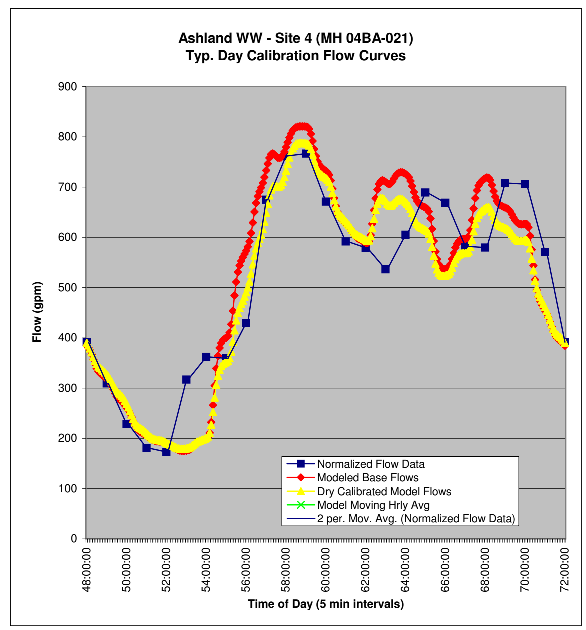
Time		(gpm)	Modeled Base Flows (gpm)	Model Moving Hrly Avg (gpm)	Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)
48:00:00	**395.22**	**391.27**	384.39	404.46	390.208	410.73
48:05:00	388.33		380.27	399.38	384.955	405.68
48:10:00	381.43		374.90	394.60	378.866	400.89
48:15:00	374.54		367.14	390.01	371.286	396.26
48:20:00	367.64		357.37	385.37	362.672	391.60
48:25:00	360.75		347.42	380.52	354.418	386.77
48:30:00	353.86		339.14	375.42	347.778	381.75
48:35:00	346.96		333.23	370.11	343.084	376.60
48:40:00	340.07		329.26	364.72	339.83	371.42
48:45:00	333.18		326.37	359.32	337.176	366.28
48:50:00	326.28		323.78	353.97	334.392	361.24
48:55:00	319.39		321.08	348.70	331.108	356.31
49:00:00	**312.494**	**309.36906**	318.21	343.18	327.38	351.08
49:05:00	305.66		314.92	337.73	323.174	345.93
49:10:00	298.83		310.66	332.38	318.288	340.88
49:15:00	292.00		304.61	327.17	312.275	335.96
49:20:00	285.17		297.13	322.15	305.57	331.21
49:25:00	278.34		289.62	317.34	299.256	326.61
49:30:00	271.51		283.42	312.69	294.209	322.15
49:35:00	264.68		278.92	308.17	290.545	317.77
49:40:00	257.85		275.66	303.70	287.667	313.42
49:45:00	251.02		272.76	299.23	284.639	309.04
49:50:00	244.19		269.44	294.70	280.616	304.56
49:55:00	237.36		265.33	290.06	275.229	299.90
50:00:00	**230.53**	**228.22**	260.55	285.25	268.75	295.02
50:05:00	226.54		255.29	280.28	261.676	289.89
50:10:00	222.55		249.35	275.17	254.275	284.56
50:15:00	218.56		242.25	269.98	246.508	279.08
50:20:00	214.57		234.30	264.74	238.791	273.51
50:25:00	210.58		226.70	259.50	231.992	267.91
50:30:00	206.59		220.59	254.26	226.799	262.29
50:35:00	202.60		216.32	249.05	223.25	256.68





Dry Calibration Basin: Site 5 - Basin 5

copied from Flow Data File:
"Basin5_FlowMonitoring.xls" at J:\210055\Design\KellerCollectionModel\ModelCalibration\Basin_5

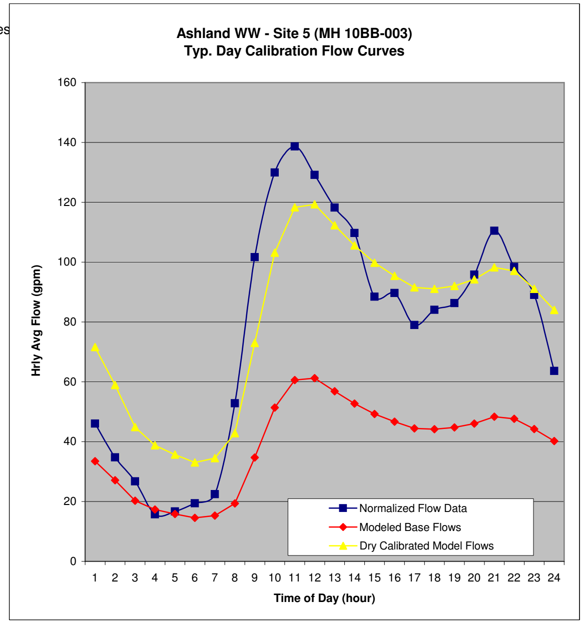
WWTP Typ. Dry Day Avg =	1458	gpm
WWTP Calibration Day Avg =	1514	gpm
Normalizing Factor Calc =	0.96	
Normalizing Factor =	0.96	

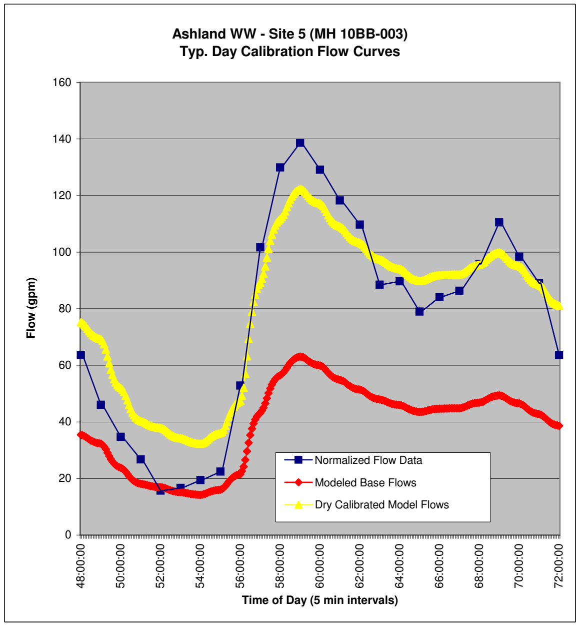
Load Shift Factor = 1.5 & upstream Div & Basin 6 changes

Typ Day Used to Calibrate Model

Date	Time	Hrly Avg Flow (gpm)	Normalized Flow Data (gpm)		5/19/2011 Modeled Base Flows (gpm)	5/19/2011 Modeled Base Flows (gpm)		6/6/2011 Dry Calibrated Model Flows (gpm)	6/6/2011 Dry Calibrated Model Flows (gpm)	
26-Sep-10	0:00	47.959	46.04		33.50	33.50		71.58	71.58	
26-Sep-10	1:00	36.18	34.73		27.11	27.11		58.92	58.92	
26-Sep-10	2:00	27.859	26.74		20.27	20.27		44.86	44.86	
26-Sep-10	3:00	16.359	15.70		17.35	17.35		38.78	38.78	
26-Sep-10	4:00	17.361	16.67		15.84	15.84		35.68	35.68	
26-Sep-10	5:00	20.225	19.42		14.57	14.57		33.04	33.04	
26-Sep-10	6:00	23.37	22.44		15.30	15.30		34.50	34.50	
26-Sep-10	7:00	55.035	52.83		19.38	19.38		42.85	42.85	
26-Sep-10	8:00	105.9	101.66		34.69	34.69		73.01	73.01	
26-Sep-10	9:00	135.369	129.95		51.40	51.40		103.17	103.17	
26-Sep-10	10:00	144.455	138.68	MAX	60.58	60.58		118.28	118.28	
26-Sep-10	11:00	134.498	129.12		61.21	61.21	MAX	119.31	119.31	MAX
26-Sep-10	12:00	123.178	118.25		56.83	56.83		112.27	112.27	
26-Sep-10	13:00	114.286	109.71		52.73	52.73		105.57	105.57	
26-Sep-10	14:00	92.156	88.47		49.27	49.27		99.78	99.78	
26-Sep-10	15:00	93.417	89.68		46.70	46.70		95.41	95.41	
26-Sep-10	16:00	82.29	79.00		44.46	44.46		91.53	91.53	
26-Sep-10	17:00	87.559	84.06		44.19	44.19		91.05	91.05	
26-Sep-10	18:00	89.924	86.33		44.76	44.76		92.05	92.05	
26-Sep-10	19:00	99.834	95.84		46.02	46.02		94.22	94.22	
26-Sep-10	20:00	115.103	110.50		48.37	48.37		98.22	98.22	
26-Sep-10	21:00	102.489	98.39		47.65	47.65		97.04	97.04	
26-Sep-10	22:00	92.764	89.05		44.20	44.20		91.08	91.08	
26-Sep-10	23:00	66.29	63.64		40.21	40.21		84.02	84.02	
							Factor			Factor
	MAX	144.455	138.68	**MAX**	61.21	61.21	2.265708	119.31	119.31	1.162354
	AVG	80.16083	76.95	**AVG**	39.02	39.02	1.971942	80.26	80.26	0.958823
	MIN	16.359	15.70	**MIN**	14.57	14.57	1.077869	33.04	33.04	0.475322

Time		(gpm)	Modeled Base Flows (gpm)	Model Moving Hrly Avg (gpm)	Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)
48:00:00	**66.29**	**63.64**	35.43	39.62	75.228	82.94
48:05:00	64.76		35.22	39.04	74.867	81.88
48:10:00	63.23		34.97	38.48	74.417	80.85
48:15:00	61.71		34.62	37.93	73.761	79.85
48:20:00	60.18		34.20	37.40	72.949	78.87
48:25:00	58.65		33.78	36.87	72.136	77.90
48:30:00	57.12		33.41	36.35	71.412	76.93
48:35:00	55.60		33.09	35.83	70.804	75.96
48:40:00	54.07		32.84	35.31	70.311	74.99
48:45:00	52.54		32.64	34.80	69.924	74.02
48:50:00	51.01		32.50	34.28	69.633	73.05
48:55:00	49.49		32.39	33.76	69.429	72.07
49:00:00	**47.959**	**46.04064**	32.32	33.50	69.293	71.58
49:05:00	46.98		31.90	33.22	68.594	71.06
49:10:00	46.00		31.27	32.91	67.444	70.47
49:15:00	45.01		30.32	32.56	65.565	69.79
49:20:00	44.03		29.12	32.13	63.151	68.97
49:25:00	43.05		27.92	31.64	60.692	68.02
49:30:00	42.07		26.86	31.10	58.486	66.94
49:35:00	41.09		25.97	30.51	56.624	65.76
49:40:00	40.11		25.25	29.87	55.114	64.50
49:45:00	39.12		24.69	29.21	53.931	63.16
49:50:00	38.14		24.27	28.53	53.043	61.78
49:55:00	37.16		23.97	27.82	52.421	60.36
50:00:00	**36.18**	**34.73**	23.79	27.11	52.021	58.92
50:05:00	35.49		23.43	26.41	51.384	57.49
50:10:00	34.79		22.98	25.72	50.525	56.08
50:15:00	34.10		22.35	25.05	49.222	54.72
50:20:00	33.41		21.57	24.42	47.593	53.42
50:25:00	32.71		20.79	23.83	45.961	52.19
50:30:00	32.02		20.10	23.26	44.516	51.03
50:35:00	31.33		19.53	22.73	43.306	49.92





Dry Calibration Basin: Site 5a - Basin 5

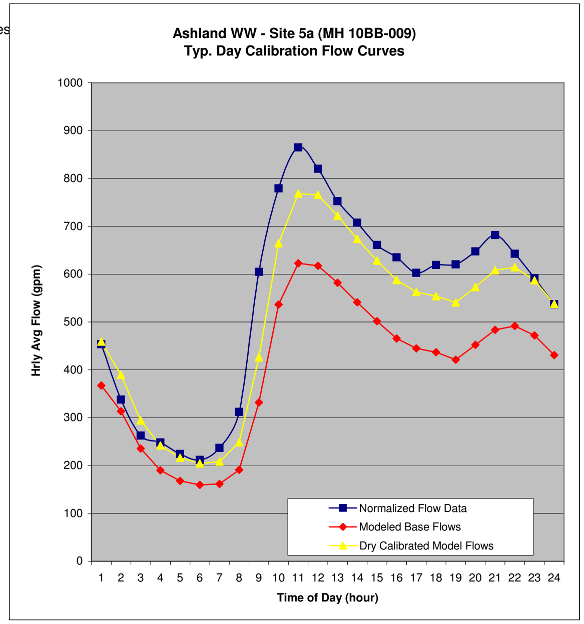
copied from Flow Data File:
"Basin5_FlowMonitoring.xls" at J:\210055\Design\KellerCollectionModel\ModelCalibration\Basin_5

WWTP Typ. Dry Day Avg = 1458 gpm
WWTP Calibration Day Avg = 1514 gpm
Normalizing Factor Calc = 0.96

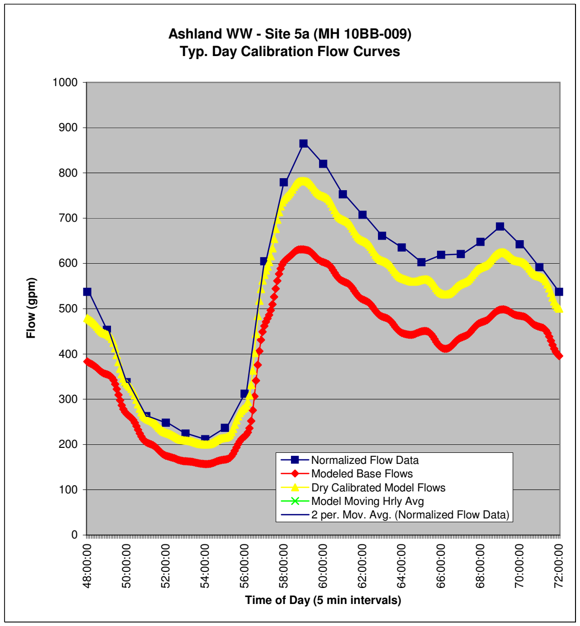
Normalizing Factor = 0.96

Load Shift Factor = 1.5 & upstream Div & Basin 6 changes

Date	Time	Hrly Avg Flow (gpm)	Typ Day Used to Calibrate Model	Normalized Flow Data (gpm)		5/19/2011 Modeled Base Flows (gpm)	5/19/2011 Modeled Base Flows (gpm)			6/6/2011 Dry Calibrated Model Flows (gpm)	6/6/2011 Dry Calibrated Model Flows (gpm)	
26-Sep-10	0:00	472.335		453.44		367.17	367.17			458.97	458.97	
26-Sep-10	1:00	351.769		337.70		313.52	313.52			389.45	389.45	
26-Sep-10	2:00	273.557		262.61		235.49	235.49			294.09	294.09	
26-Sep-10	3:00	258.491		248.15		190.10	190.10			241.71	241.71	
26-Sep-10	4:00	233.427		224.09		167.91	167.91			216.17	216.17	
26-Sep-10	5:00	220.494		211.67		159.70	159.70			204.22	204.22	
26-Sep-10	6:00	246.538		236.68		161.70	161.70			208.17	208.17	
26-Sep-10	7:00	324.932		311.93		190.90	190.90			248.37	248.37	
26-Sep-10	8:00	629.994		604.79		331.93	331.93			426.05	426.05	
26-Sep-10	9:00	811.786		779.31		536.61	536.61			664.40	664.40	
26-Sep-10	10:00	900.996		864.96	MAX	622.69	622.69	MAX		768.21	768.21	MAX
26-Sep-10	11:00	854.196		820.03		617.23	617.23			765.56	765.56	
26-Sep-10	12:00	783.959		752.60		581.79	581.79			722.00	722.00	
26-Sep-10	13:00	736.674		707.21		541.00	541.00			673.62	673.62	
26-Sep-10	14:00	688.389		660.85		501.58	501.58			628.04	628.04	
26-Sep-10	15:00	661.687		635.22		465.49	465.49			587.59	587.59	
26-Sep-10	16:00	627.616		602.51		444.92	444.92			562.48	562.48	
26-Sep-10	17:00	644.731		618.94		436.91	436.91			553.95	553.95	
26-Sep-10	18:00	646.408		620.55		421.02	421.02			540.82	540.82	
26-Sep-10	19:00	674.603		647.62		452.14	452.14			572.96	572.96	
26-Sep-10	20:00	710.037		681.64		483.62	483.62			608.41	608.41	
26-Sep-10	21:00	669.166		642.40		491.62	491.62			614.51	614.51	
26-Sep-10	22:00	616.438		591.78		471.99	471.99			587.10	587.10	
26-Sep-10	23:00	559.425		537.05		430.80	430.80	Factor		537.80	537.80	Factor
	MAX	900.996		864.96	MAX	622.69	622.69	1.389075		768.21	768.21	1.125935
	AVG	566.5687		543.91	AVG	400.74	400.74	1.357245		503.11	503.11	1.081087
	MIN	220.494		211.67	MIN	159.70	159.70	1.325473		204.22	204.22	1.036497



Time	Value	(gpm)	Modeled Base Flows (gpm)	Model Moving Hrly Avg (gpm)	Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)
48:00:00	**559.43**	**537.05**	383.37	424.49	479.381	530.22
48:05:00	552.17		380.64	418.06	476.525	522.54
48:10:00	544.91		378.62	411.64	474.222	514.91
48:15:00	537.65		376.73	405.39	471.755	507.47
48:20:00	530.40		374.53	399.43	468.662	500.37
48:25:00	523.14		371.81	393.89	464.814	493.69
48:30:00	515.88		368.59	388.82	460.381	487.48
48:35:00	508.62		365.19	384.25	455.867	481.73
48:40:00	501.37		362.04	380.11	451.853	476.38
48:45:00	494.11		359.41	376.34	448.675	471.35
48:50:00	486.85		357.42	372.84	446.363	466.56
48:55:00	479.59		355.99	369.53	444.773	461.94
49:00:00	**472.335**	**453.4416**	355.00	367.17	443.7	458.97
49:05:00	462.29		353.53	364.91	441.753	456.07
49:10:00	452.24		351.51	362.65	438.779	453.11
49:15:00	442.19		348.00	360.25	433.399	449.92
49:20:00	432.15		342.11	357.55	424.718	446.26
49:25:00	422.10		333.43	354.35	412.63	441.91
49:30:00	412.05		322.24	350.49	397.847	436.70
49:35:00	402.00		309.70	345.86	382.088	430.55
49:40:00	391.96		297.42	340.48	367.397	423.51
49:45:00	381.91		286.69	334.42	355.158	415.72
49:50:00	371.86		278.15	327.81	345.81	407.34
49:55:00	361.82		271.88	320.81	339.148	398.54
50:00:00	**351.77**	**337.70**	267.55	313.52	334.658	389.45
50:05:00	345.25		264.21	306.07	330.969	380.22
50:10:00	338.73		261.44	298.57	327.532	370.95
50:15:00	332.22		258.29	291.09	323.11	361.76
50:20:00	325.70		253.98	283.75	316.92	352.77
50:25:00	319.18		248.10	276.64	308.769	344.12
50:30:00	312.66		240.70	269.84	299.023	335.88
50:35:00	306.15		232.49	263.41	288.723	328.10



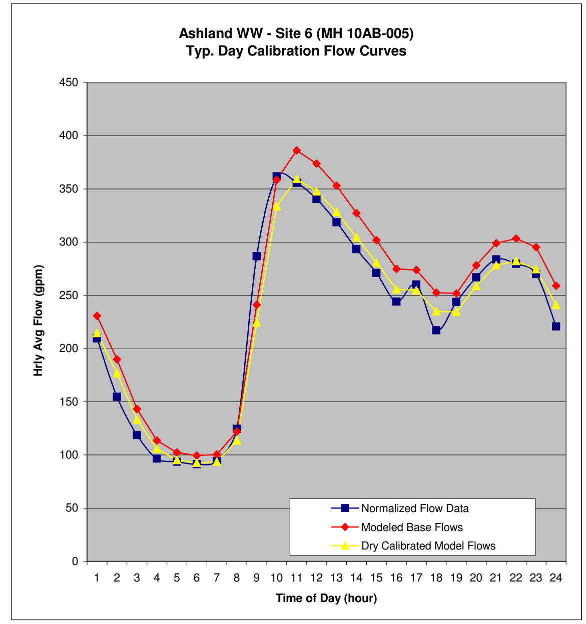
Dry Calibration Basin: Site 6 - Basin 6

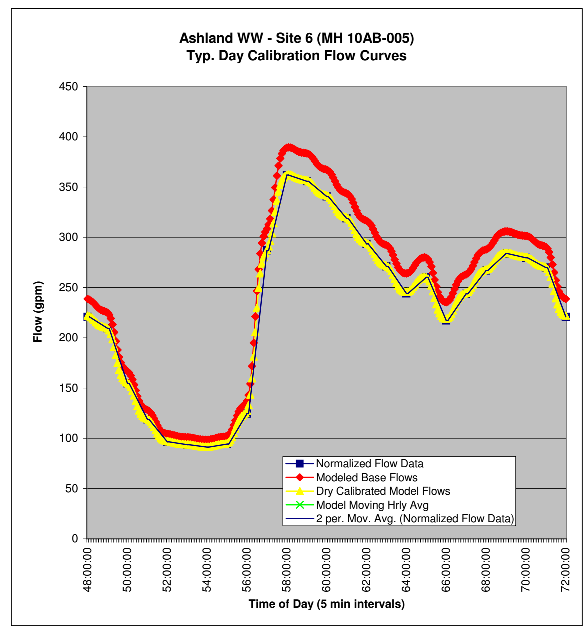
copied from Flow Data File:			
"Basin6_FlowMonitoring.xls"	at	J:\210055\Design\KellerCollectionModel\ModelCalibration\Basin_6	
	WWTP Typ. Dry Day Avg =	1458	gpm
	WWTP Calibration Day Avg =	1396	gpm
	Normalizing Factor Calc =	1.04	
	Normalizing Factor =	1.04	

Date	Time	Hrly Avg Flow (gpm)	Normalized Flow Data (gpm)		Modeled Base Flows (gpm) 5/19/2011	Modeled Base Flows (gpm) 5/19/2011		Dry Calibrated Model Flows (gpm) 5/27/2011	Dry Calibrated Model Flows (gpm) 5/27/2011	
						Load Shift Factor =		0.93	0.93	
13-Sep-10	0:00	201.397	209.45		230.66	230.66		214.65	214.65	
13-Sep-10	1:00	148.596	154.54		189.82	189.82		176.65	176.65	
13-Sep-10	2:00	114.162	118.73		143.19	143.19		133.26	133.26	
13-Sep-10	3:00	92.839	96.55		113.54	113.54		105.66	105.66	
13-Sep-10	4:00	90.017	93.62		102.24	102.24		95.15	95.15	
13-Sep-10	5:00	87.75	91.26		99.45	99.45		92.55	92.55	
13-Sep-10	6:00	90.698	94.33		100.52	100.52		93.55	93.55	
13-Sep-10	7:00	119.735	124.52		121.93	121.93		113.47	113.47	
13-Sep-10	8:00	275.844	286.88		241.05	241.05		224.33	224.33	
13-Sep-10	9:00	347.95	361.87	MAX	358.54	358.54		333.67	333.67	
13-Sep-10	10:00	342.003	355.68		386.20	386.20	MAX	359.40	359.40	MAX
13-Sep-10	11:00	327.48	340.58		373.73	373.73		347.80	347.80	
13-Sep-10	12:00	306.512	318.77		353.03	353.03		328.54	328.54	
13-Sep-10	13:00	282.162	293.45		327.21	327.21		304.51	304.51	
13-Sep-10	14:00	260.605	271.03		301.79	301.79		280.85	280.85	
13-Sep-10	15:00	234.668	244.05		274.71	274.71		255.65	255.65	
13-Sep-10	16:00	250.213	260.22		273.81	273.81		254.81	254.81	
13-Sep-10	17:00	208.792	217.14		252.47	252.47		234.95	234.95	
13-Sep-10	18:00	234.415	243.79		251.82	251.82		234.35	234.35	
13-Sep-10	19:00	256.676	266.94		278.20	278.20		258.90	258.90	
13-Sep-10	20:00	272.919	283.84		299.02	299.02		278.27	278.27	
13-Sep-10	21:00	268.812	279.56		303.34	303.34		282.29	282.29	
13-Sep-10	22:00	259.598	269.98		295.18	295.18		274.70	274.70	
13-Sep-10	23:00	212.313	220.81		258.93	258.93		240.97	240.97	
							Factor			
	MAX	347.95	361.87	MAX	386.20	386.20	0.937	359.40	359.40	
	AVG	220.2564	229.07	AVG	247.10	247.10	0.927021	229.96	229.96	
	MIN	87.75	91.26	MIN	99.45	99.45	0.917658	92.55	92.55	

Typ Day Used to Calibrate Model

Time		(gpm)	Modeled Base Flows (gpm)	Model Moving Hrly Avg (gpm)	Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)
48:00:00	**212.31**	**220.81**	238.65	254.77	222.094	237.09
48:05:00	211.40		237.88	250.83	221.37	233.42
48:10:00	210.49		236.92	247.24	220.482	230.09
48:15:00	209.58		235.62	244.14	219.274	227.20
48:20:00	208.67		233.97	241.56	217.736	224.79
48:25:00	207.76		232.12	239.47	216.014	222.85
48:30:00	206.86		230.33	237.77	214.349	221.27
48:35:00	205.95		228.84	236.34	212.957	219.94
48:40:00	205.04		227.74	235.09	211.935	218.77
48:45:00	204.13		227.00	233.94	211.249	217.70
48:50:00	203.22		226.49	232.85	210.775	216.69
48:55:00	202.31		225.97	231.79	210.294	215.71
49:00:00	**201.397**	**209.45288**	225.02	230.66	209.408	214.65
49:05:00	197.00		223.00	229.42	207.526	213.50
49:10:00	192.60		219.23	227.94	204.019	212.13
49:15:00	188.20		213.32	226.09	198.519	210.40
49:20:00	183.80		205.47	223.71	191.213	208.19
49:25:00	179.40		196.57	220.75	182.929	205.43
49:30:00	175.00		187.93	217.21	174.887	202.14
49:35:00	170.60		180.70	213.20	168.159	198.41
49:40:00	166.20		175.39	208.84	163.218	194.35
49:45:00	161.80		171.84	204.24	159.92	190.07
49:50:00	157.40		169.58	199.50	157.815	185.66
49:55:00	153.00		168.07	194.68	156.409	181.17
50:00:00	**148.60**	**154.54**	166.77	189.82	155.199	176.65
50:05:00	145.73		165.09	185.00	153.632	172.16
50:10:00	142.86		162.45	180.26	151.179	167.76
50:15:00	139.99		158.52	175.70	147.521	163.51
50:20:00	137.12		153.37	171.36	142.728	159.47
50:25:00	134.25		147.55	167.27	137.316	155.67
50:30:00	131.38		141.92	163.44	132.068	152.10
50:35:00	128.51		137.20	159.81	127.68	148.72





Dry Calibration Basin: Site 7 - Basin 5

copied from Flow Data File: "Site7_FlowMonitoring.xls" at J:\210055\Design\KellerCollectionModel\ModelCalibration\Basin_5

WWTP Typ. Dry Day Avg = 1458 gpm
WWTP Calibration Day Avg = 1368 gpm
Normalizing Factor Calc = 1.07

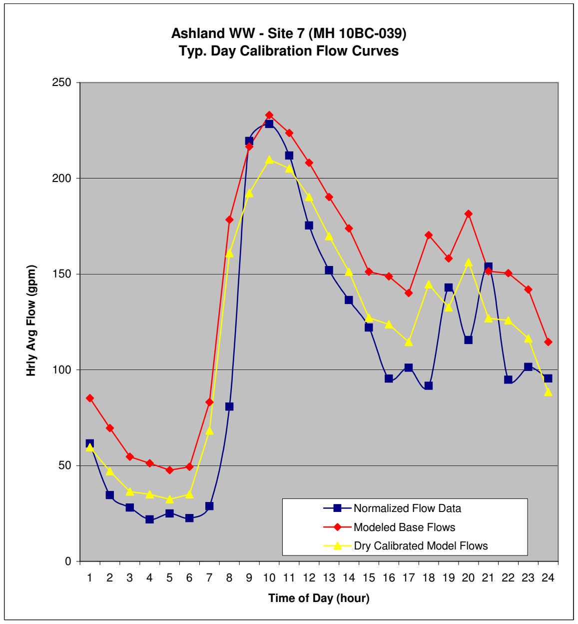
#2

Normalizing Factor = 1.07

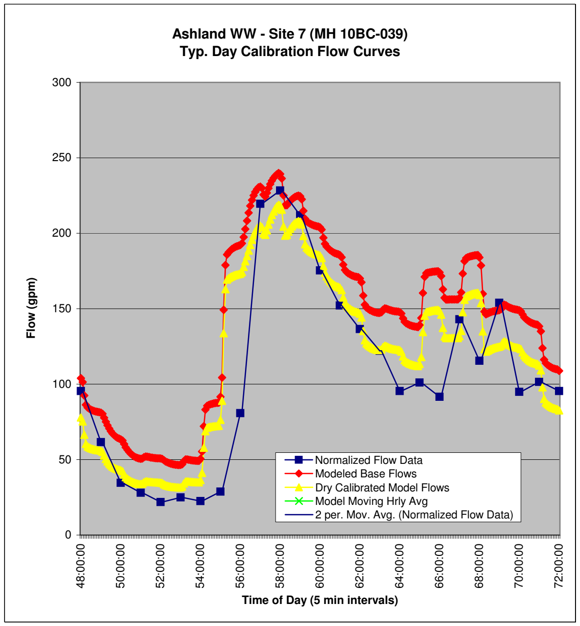
change patternLoad Shift Factor = 1.5 & Div changes

Typ Day Used to Calibrate Model

Date	Time	Hrly Avg Flow (gpm)	Normalized Flow Data (gpm)		Modeled Base Flows (gpm) 6/1/2011	Modeled Base Flows (gpm) 6/1/2011		Factor	Dry Calibrated Model Flows (gpm) 6/6/2011	Dry Calibrated Model Flows (gpm) 6/6/2011		Factor
14-Sep-10	0:00	57.584	61.61		85.19	85.19			59.59	59.59		
14-Sep-10	1:00	32.356	34.62		69.64	69.64			47.05	47.05		
14-Sep-10	2:00	26.214	28.05		54.61	54.61			36.39	36.39		
14-Sep-10	3:00	20.462	21.89		51.26	51.26			35.03	35.03		
14-Sep-10	4:00	23.358	24.99		47.64	47.64			32.39	32.39		
14-Sep-10	5:00	21.047	22.52		49.39	49.39			34.99	34.99		
14-Sep-10	6:00	26.925	28.81		83.14	83.14			68.29	68.29		
14-Sep-10	7:00	75.518	80.80		178.39	178.39			160.92	160.92		
14-Sep-10	8:00	205.157	219.52		216.56	216.56			192.25	192.25		
14-Sep-10	9:00	213.391	228.33	MAX	233.07	233.07	MAX		209.79	209.79	MAX	
14-Sep-10	10:00	198.072	211.94		223.67	223.67			205.17	205.17		
14-Sep-10	11:00	163.915	175.39		208.13	208.13			190.24	190.24		
14-Sep-10	12:00	142.15	152.10		190.26	190.26			169.76	169.76		
14-Sep-10	13:00	127.55	136.48		173.90	173.90			151.24	151.24		
14-Sep-10	14:00	114.069	122.05		151.26	151.26			127.14	127.14		
14-Sep-10	15:00	89.215	95.46		148.81	148.81			123.79	123.79		
14-Sep-10	16:00	94.479	101.09		140.12	140.12			114.52	114.52		
14-Sep-10	17:00	85.646	91.64		170.34	170.34			144.63	144.63		
14-Sep-10	18:00	133.617	142.97		158.17	158.17			132.60	132.60		
14-Sep-10	19:00	107.973	115.53		181.48	181.48			156.20	156.20		
14-Sep-10	20:00	143.822	153.89		151.50	151.50			126.95	126.95		
14-Sep-10	21:00	88.662	94.87		150.52	150.52			125.82	125.82		
14-Sep-10	22:00	94.858	101.50		141.99	141.99			116.38	116.38		
14-Sep-10	23:00	89.232	95.48		114.59	114.59			88.42	88.42		
	MAX	213.391	228.33	MAX	233.07	233.07		0.979641	209.79	209.79		1.08836
	AVG	98.96967	105.90	AVG	140.57	140.57		0.753359	118.73	118.73		0.891915
	MIN	20.462	21.89	MIN	47.64	47.64		0.45958	32.39	32.39		0.675995



Time		(gpm)	Modeled Base Flows (gpm)	Model Moving Hrly Avg (gpm)	Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)
48:00:00	**89.23**	**95.48**	103.95	112.00	77.911	85.82
48:05:00	86.59		101.33	110.11	75.306	83.93
48:10:00	83.96		92.42	108.13	66.436	81.95
48:15:00	81.32		86.34	105.83	60.429	79.67
48:20:00	78.68		84.53	103.47	58.724	77.33
48:25:00	76.05		83.77	101.11	58.066	75.02
48:30:00	73.41		83.14	98.76	57.532	72.72
48:35:00	70.77		82.61	96.41	57.092	70.44
48:40:00	68.13		82.19	94.07	56.741	68.16
48:45:00	65.50		81.85	91.72	56.469	65.89
48:50:00	62.86		81.60	89.38	56.265	63.63
48:55:00	60.22		81.43	87.09	56.121	61.42
49:00:00	**57.584**	**61.61488**	81.13	85.19	55.845	59.59
49:05:00	55.48		80.39	83.45	55.142	57.91
49:10:00	53.38		77.76	82.23	52.676	56.76
49:15:00	51.28		75.03	81.28	50.388	55.92
49:20:00	49.17		72.72	80.30	48.74	55.09
49:25:00	47.07		70.63	79.21	47.369	54.20
49:30:00	44.97		68.83	78.01	46.25	53.26
49:35:00	42.87		67.32	76.74	45.352	52.28
49:40:00	40.77		66.10	75.40	44.633	51.27
49:45:00	38.66		65.14	74.01	44.07	50.24
49:50:00	36.56		64.42	72.58	43.643	49.19
49:55:00	34.46		63.92	71.12	43.338	48.12
50:00:00	**32.36**	**34.62**	63.42	69.64	42.969	47.05
50:05:00	31.84		62.63	68.16	42.271	45.97
50:10:00	31.33		60.25	66.70	40.082	44.93
50:15:00	30.82		58.05	65.29	38.322	43.92
50:20:00	30.31		56.46	63.93	37.324	42.97
50:25:00	29.80		55.09	62.64	36.528	42.07
50:30:00	29.29		53.91	61.39	35.833	41.20
50:35:00	28.77		52.93	60.19	35.25	40.36



Dry Calibration Basin: Site 8 - Basin 7

copied from Flow Data File:
"Basin8_FlowMonitoring.xls" at J:\210055\Design\KellerCollectionModel\ModelCalibration\Basin_8

WWTP Typ. Dry Day Avg = 1458 gpm
WWTP Calibration Day Avg = 1396 gpm
Normalizing Factor Calc = 1.04

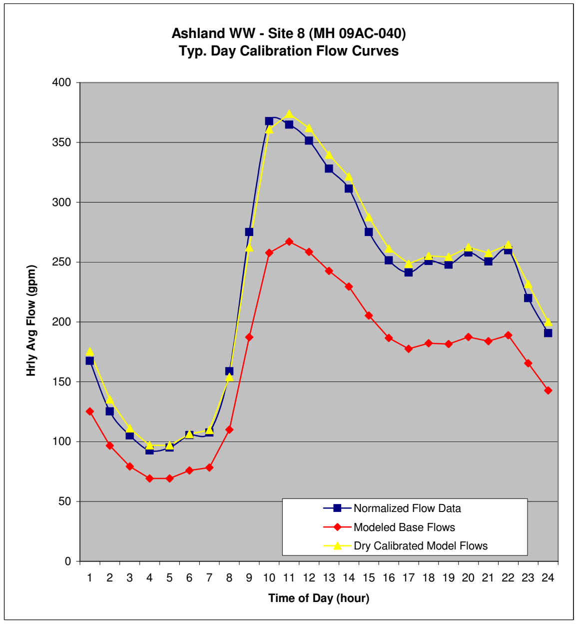
Normalizing Factor = 1.04

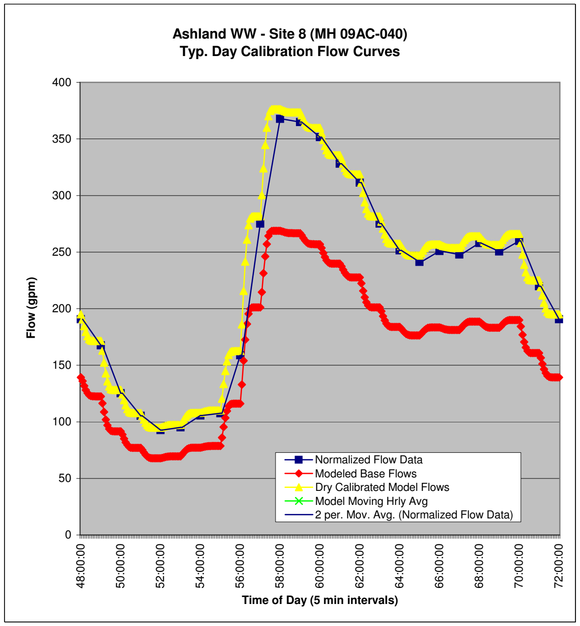
Load Shift Factor = 1.4 | 1.4

Typ Day Used to Calibrate Model

Date	Time	Hrly Avg Flow (gpm)	Normalized Flow Data (gpm)		Modeled Base Flows (gpm) 5/19/2011	Modeled Base Flows (gpm) 5/19/2011		Factor	Dry Calibrated Model Flows (gpm) 5/20/2011	Dry Calibrated Model Flows (gpm) 5/20/2011	
12-Sep-10	0:00	161.134	167.58		125.24	125.24			175.37	175.37	
12-Sep-10	1:00	120.488	125.31		96.63	96.63			135.31	135.31	
12-Sep-10	2:00	101.204	105.25		79.33	79.33			111.08	111.08	
12-Sep-10	3:00	89.106	92.67		69.24	69.24			96.95	96.95	
12-Sep-10	4:00	91.45	95.11		69.24	69.24			96.95	96.95	
12-Sep-10	5:00	101.472	105.53		75.90	75.90			106.29	106.29	
12-Sep-10	6:00	103.443	107.58		78.40	78.40			109.78	109.78	
12-Sep-10	7:00	152.702	158.81		110.00	110.00			154.03	154.03	
12-Sep-10	8:00	264.604	275.19		187.33	187.33			262.31	262.31	
12-Sep-10	9:00	353.59	367.73	MAX	257.81	257.81			361.01	361.01	
12-Sep-10	10:00	350.853	364.89		267.07	267.07	MAX		373.98	373.98	MAX
12-Sep-10	11:00	338.007	351.53		258.56	258.56			362.06	362.06	
12-Sep-10	12:00	315.419	328.04		242.59	242.59			339.70	339.70	
12-Sep-10	13:00	299.354	311.33		229.57	229.57			321.47	321.47	
12-Sep-10	14:00	264.517	275.10		205.41	205.41			287.63	287.63	
12-Sep-10	15:00	241.857	251.53		186.67	186.67			261.40	261.40	
12-Sep-10	16:00	232.003	241.28		177.60	177.60			248.69	248.69	
12-Sep-10	17:00	241.26	250.91		182.27	182.27			255.24	255.24	
12-Sep-10	18:00	238.347	247.88		181.56	181.56			254.24	254.24	
12-Sep-10	19:00	248.082	258.01		187.40	187.40			262.42	262.42	
12-Sep-10	20:00	240.873	250.51		184.02	184.02			257.68	257.68	
12-Sep-10	21:00	249.941	259.94		188.90	188.90			264.51	264.51	
12-Sep-10	22:00	211.422	219.88		165.50	165.50			231.74	231.74	
12-Sep-10	23:00	183.273	190.60		142.81	142.81			199.98	199.98	
	MAX	353.59	367.73	**MAX**	267.07	267.07		1.376894	373.98	373.98	
	AVG	216.4334	225.09	**AVG**	164.54	164.54		1.367969	230.41	230.41	
	MIN	89.106	92.67	**MIN**	69.24	69.24		1.338485	96.95	96.95	

Time		(gpm)	Modeled Base Flows (gpm)	Model Moving Hrly Avg (gpm)	Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)
48:00:00	**183.27**	**190.60**	139.33	141.38	195.108	197.98
48:05:00	181.43		136.01	140.12	190.449	196.21
48:10:00	179.58		131.83	138.89	184.597	194.49
48:15:00	177.74		128.16	137.65	179.454	192.76
48:20:00	175.89		125.42	136.37	175.622	190.96
48:25:00	174.05		123.62	135.04	173.109	189.09
48:30:00	172.20		122.79	133.65	171.94	187.15
48:35:00	170.36		122.55	132.25	171.601	185.19
48:40:00	168.51		122.51	130.85	171.544	183.23
48:45:00	166.67		122.50	129.45	171.539	181.26
48:50:00	164.82		122.50	128.05	171.539	179.30
48:55:00	162.98		122.50	126.64	171.539	177.34
49:00:00	**161.134**	**167.57936**	122.50	125.24	171.539	175.37
49:05:00	157.75		116.39	123.61	162.986	173.08
49:10:00	154.36		108.72	121.68	152.241	170.39
49:15:00	150.97		101.98	119.50	142.8	167.33
49:20:00	147.59		96.95	117.13	135.764	164.01
49:25:00	144.20		93.66	114.63	131.151	160.52
49:30:00	140.81		92.13	112.07	129.004	156.94
49:35:00	137.42		91.68	109.50	128.382	153.34
49:40:00	134.04		91.61	106.93	128.278	149.73
49:45:00	130.65		91.60	104.35	128.269	146.12
49:50:00	127.26		91.60	101.78	128.268	142.52
49:55:00	123.88		91.60	99.20	128.268	138.91
50:00:00	**120.49**	**125.31**	91.60	96.63	128.268	135.31
50:05:00	118.88		88.70	94.32	124.211	132.08
50:10:00	117.27		85.06	92.35	119.113	129.31
50:15:00	115.67		81.86	90.67	114.633	126.97
50:20:00	114.06		79.48	89.22	111.296	124.93
50:25:00	112.45		77.92	87.90	109.107	123.09
50:30:00	110.85		77.19	86.66	108.088	121.35
50:35:00	109.24		76.98	85.43	107.793	119.63





Dry Calibration Basin: Site 9 - Basin 8

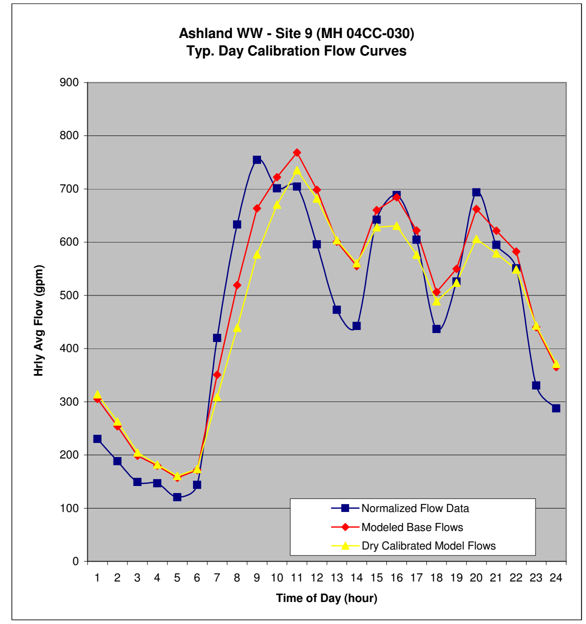
copied from Flow Data File: "Basin8_FlowMonitoring.xls" at J:\210055\Design\KellerCollectionModel\ModelCalibration\Basin_8

WWTP Typ. Dry Day Avg = 1458 gpm
WWTP Calibration Day Avg = 1528 gpm
Normalizing Factor Calc = 0.95

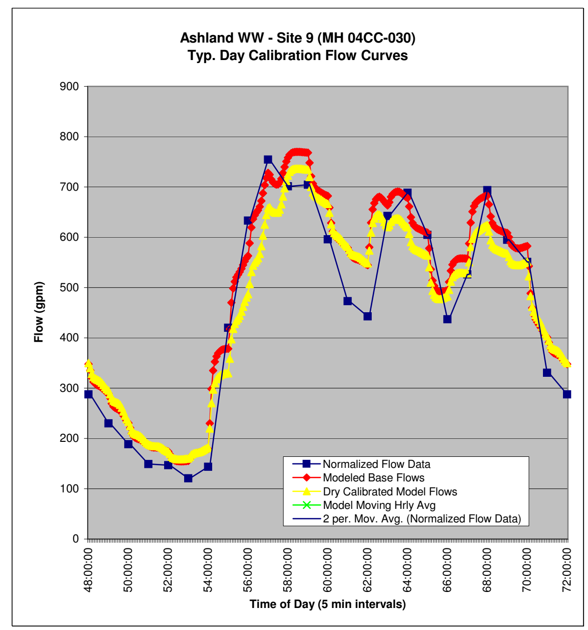
Normalizing Factor = 0.95
Load Shift Factor = 0.73 0.73

Typ Day Used to Calibrate Model

Date	Time	Hrly Avg Flow (gpm)	Normalized Flow Data (gpm)		Pipe 96	Pipe 103	5/19/2011 Modeled Base Flows (gpm)	5/19/2011 Modeled Base Flows (gpm)		Pipe 96	Pipe 103	6/2/2011 Dry Calibrated Model Flows (gpm)	6/2/2011 Dry Calibrated Model Flows (gpm)	
2-Oct-10	0:00	242.175	230.07		138.10	167.19	305.29	305.29		101.24	212.90	314.15	314.15	
2-Oct-10	1:00	198.331	188.41		112.76	141.00	253.76	253.76		82.67	180.22	262.89	262.89	
2-Oct-10	2:00	156.952	149.10		89.63	108.96	198.59	198.59		65.71	138.81	204.52	204.52	
2-Oct-10	3:00	154.611	146.88		86.02	93.92	179.95	179.95		63.07	118.79	181.86	181.86	
2-Oct-10	4:00	126.909	120.56		72.13	84.75	156.89	156.89		52.88	107.67	160.55	160.55	
2-Oct-10	5:00	151.297	143.73		82.60	89.15	171.75	171.75		60.56	112.96	173.52	173.52	
2-Oct-10	6:00	442.167	420.06		228.43	122.29	350.72	350.72		167.47	141.32	308.79	308.79	
2-Oct-10	7:00	666.567	633.24		357.01	162.27	519.27	519.27		261.74	177.45	439.19	439.19	
2-Oct-10	8:00	794.322	754.61	MAX	433.69	229.76	663.45	663.45		317.95	258.98	576.93	576.93	
2-Oct-10	9:00	738.158	701.25		413.36	308.81	722.17	722.17		303.05	367.52	670.56	670.56	
2-Oct-10	10:00	741.593	704.51		411.74	356.76	768.51	768.51	MAX	301.86	433.27	735.13	735.13	MAX
2-Oct-10	11:00	627.187	595.83		355.16	343.35	698.50	698.50		260.38	421.45	681.83	681.83	
2-Oct-10	12:00	497.832	472.94		284.18	316.92	601.10	601.10		208.35	395.24	603.59	603.59	
2-Oct-10	13:00	465.847	442.55		260.66	294.43	555.09	555.09		191.10	369.26	560.36	560.36	
2-Oct-10	14:00	676.083	642.28		363.13	297.04	660.17	660.17		266.23	361.70	627.93	627.93	
2-Oct-10	15:00	724.68	688.45		399.68	284.54	684.22	684.22		293.02	337.87	630.89	630.89	
2-Oct-10	16:00	636.543	604.72		358.80	263.32	622.12	622.12		263.05	313.14	576.19	576.19	
2-Oct-10	17:00	459.908	436.91		265.91	240.44	506.35	506.35		194.95	293.88	488.83	488.83	
2-Oct-10	18:00	553.822	526.13		302.09	247.82	549.92	549.92		221.47	302.19	523.67	523.67	
2-Oct-10	19:00	730.266	693.75		395.23	266.99	662.22	662.22		289.76	316.34	606.09	606.09	
2-Oct-10	20:00	626.152	594.84		353.97	267.56	621.53	621.53		259.51	319.34	578.85	578.85	
2-Oct-10	21:00	579.887	550.89		324.85	257.61	582.46	582.46		238.16	310.59	548.75	548.75	
2-Oct-10	22:00	347.873	330.48		206.95	233.21	440.16	440.16		151.72	291.49	443.22	443.22	
2-Oct-10	23:00	302.599	287.47		170.77	194.49	365.25	365.25		125.19	246.14	371.33	371.33	
									Factor					Factor
	MAX	794.322	754.61	MAX			768.51	768.51	0.981912			735.13	735.13	1.026489
	AVG	485.0734	460.82	AVG			493.31	493.31	0.934139			469.57	469.57	0.981373
	MIN	126.909	120.56	MIN			156.89	156.89	0.768475			160.55	160.55	0.750929



Time		(gpm)			Modeled Base Flows (gpm)	Model Moving Hrly Avg (gpm)			Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)
48:00:00	**302.60**	**287.47**	168.09	179.531	347.62	361.66	123.233	226.486	349.72	367.61
48:05:00	297.56		158.755	176.742	335.50	358.03	116.389	223.385	339.77	363.84
48:10:00	292.53		145.66	174.834	320.49	353.67	106.788	221.43	328.22	359.51
48:15:00	287.49		139.082	173.46	312.54	348.89	101.966	220.183	322.15	354.86
48:20:00	282.46		136.487	172.291	308.78	343.95	100.064	219.237	319.30	350.09
48:25:00	277.42		135.233	171.223	306.46	338.93	99.145	218.383	317.53	345.29
48:30:00	272.39		134.73	170.062	304.79	333.94	98.775	217.295	316.07	340.55
48:35:00	267.35		134.571	168.389	302.96	329.04	98.659	215.368	314.03	335.97
48:40:00	262.32		134.533	165.864	300.40	324.26	98.631	212.132	310.76	331.56
48:45:00	257.28		134.526	162.685	297.21	319.55	98.626	207.854	306.48	327.25
48:50:00	252.25		134.525	159.44	293.97	314.86	98.625	203.382	302.01	322.98
48:55:00	247.21		134.525	156.665	291.19	310.16	98.625	199.514	298.14	318.68
49:00:00	**242.175**	**230.06625**	134.525	154.631	289.16	305.29	98.625	196.67	295.30	314.15
49:05:00	238.52		127.752	152.509	280.26	300.68	93.659	194.265	287.92	309.83
49:10:00	234.87		118.25	151.073	269.32	296.42	86.693	192.766	279.46	305.76
49:15:00	231.21		113.477	150.053	263.53	292.33	83.194	191.826	275.02	301.83
49:20:00	227.56		111.594	149.187	260.78	288.34	81.814	191.113	272.93	297.97
49:25:00	223.91		110.684	148.328	259.01	284.38	81.147	190.376	271.52	294.14
49:30:00	220.25		110.319	147.093	257.41	280.43	80.879	189.034	269.91	290.29
49:35:00	216.60		110.204	144.702	254.91	276.43	80.795	185.981	266.78	286.35
49:40:00	212.95		110.176	140.542	250.72	272.29	80.774	180.364	261.14	282.22
49:45:00	209.29		110.171	134.973	245.14	267.95	80.771	172.682	253.45	277.80
49:50:00	205.64		110.17	129.118	239.29	263.39	80.77	164.528	245.30	273.07
49:55:00	201.98		110.17	124.046	234.22	258.65	80.77	157.436	238.21	268.08
50:00:00	**198.33**	**188.41**	110.17	120.316	230.49	253.76	80.77	152.214	232.98	262.89
50:05:00	194.88		103.778	117.177	220.96	248.81	76.083	148.354	224.44	257.59
50:10:00	191.43		94.81	115.226	210.04	243.87	69.509	146.104	215.61	252.27
50:15:00	187.99		90.305	114.007	204.31	238.94	66.206	144.858	211.06	246.94
50:20:00	184.54		88.529	113.109	201.64	234.01	64.904	144.072	208.98	241.62
50:25:00	181.09		87.67	112.337	200.01	229.09	64.274	143.431	207.71	236.30
50:30:00	177.64		87.325	111.471	198.80	224.21	64.021	142.584	206.61	231.02
50:35:00	174.19		87.216	110.127	197.34	219.41	63.942	140.981	204.92	225.87



Dry Calibration Basin: WWTP Influent

entered manually to mimic
influent SCADA printouts from City

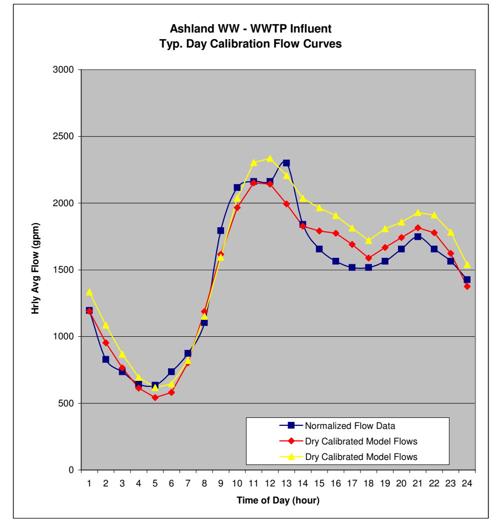
WWTP Typ. Dry Day Avg = 1458 gpm
WWTP Calibration Day Avg = 1583 gpm
Normalizing Factor Calc = 0.92

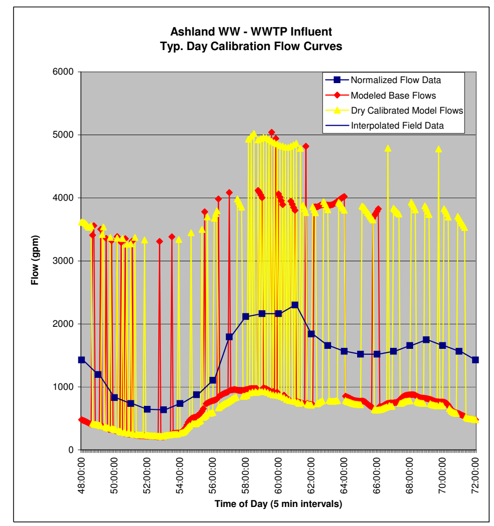
Normalizing Factor = 0.92

					WWTP Influent by Summing 3 gravity lines - ignore LS spikes!						Pipe 200000		WWTP Influent by Summing 3 gravity lines - ignore LS spikes!						Pipe 200000	
								5/19/2011	5/19/2011		5/19/2011					8/3/2011	8/3/2011		8/3/2011	
								NA			NA	ıift Factor =				changes at Site 5			changes at Site 5	
Date	Time	Estimated Hrly Avg Flow (gpm)	Normalized Flow Data (gpm)		Pipe 100001	Pipe 42	Pipe 384	Dry Calibrated Model Flows (gpm)	Dry Calibrated Model Flows (gpm)		Modeled Base Flows (gpm)		Pipe 100001	Pipe 42	Pipe 384	Dry Calibrated Model Flows (gpm)	Dry Calibrated Model Flows (gpm)		Dry Calibrated Model Flows (gpm)	
19-Aug-08	0:00	1300	1196.00		770.68	0.00	418.97	1,189.65	1,189.65		934.11		907.64	0.00	426.14	1,333.78	1,333.78		2,252.68	
19-Aug-08	1:00	900	828.00		613.58	0.00	339.67	953.25	953.25		1,105.19		730.69	0.00	354.57	1,085.26	1,085.26		1,126.64	
19-Aug-08	2:00	800	736.00		496.82	0.00	268.37	765.20	765.20		1,038.15		587.93	0.00	280.02	867.95	867.95		1,294.74	
19-Aug-08	3:00	700	644.00		385.38	0.00	227.73	613.11	613.11		484.74		455.40	0.00	238.51	693.91	693.91		759.81	
19-Aug-08	4:00	690	634.80		331.97	0.00	211.15	543.12	543.12		470.59		395.72	0.00	221.35	617.07	617.07		221.35	
19-Aug-08	5:00	800	736.00		332.29	0.00	248.81	581.10	581.10		510.70		401.46	0.00	242.61	644.07	644.07		500.71	
19-Aug-08	6:00	950	874.00		377.95	0.00	425.51	803.46	803.46		425.51		464.65	0.00	355.55	820.19	820.19		610.44	
19-Aug-08	7:00	1200	1104.00		504.71	0.00	682.75	1,187.46	1,187.46		940.95		625.56	0.00	525.92	1,151.47	1,151.47		1,040.49	
19-Aug-08	8:00	1950	1794.00		750.88	0.00	865.49	1,616.37	1,616.37		1,391.80		897.78	0.00	696.63	1,594.41	1,594.41		1,478.02	
19-Aug-08	9:00	2300	2116.00		1,018.04	0.00	948.69	1,966.73	1,966.73		949.56		1,208.68	0.00	830.31	2,038.99	2,038.99		1,859.93	
19-Aug-08	10:00	2350	2162.00		1,180.42	0.00	971.02	2,151.44	2,151.44	MAX	2,001.53		1,403.54	0.00	900.76	2,304.30	2,304.30		2,597.19	
19-Aug-08	11:00	2350	2162.00		1,209.25	0.00	933.01	2,142.27	2,142.27		1,879.50		1,457.48	0.00	877.04	2,334.52	2,334.52	MAX	2,899.46	
19-Aug-08	12:00	2500	2300.00	MAX	1,171.58	0.00	822.09	1,993.67	1,993.67		2,622.77		1,415.08	0.00	791.36	2,206.44	2,206.44		2,811.14	
19-Aug-08	13:00	2000	1840.00		1,089.45	0.00	737.94	1,827.39	1,827.39		1,080.96		1,310.91	0.00	725.15	2,036.06	2,036.06		2,185.83	
19-Aug-08	14:00	1800	1656.00		1,009.44	0.00	782.01	1,791.44	1,791.44		3,613.70		1,213.54	0.00	750.98	1,964.53	1,964.53		2,560.75	
19-Aug-08	15:00	1700	1564.00		936.22	0.00	838.64	1,774.86	1,774.86		3,942.62	MAX	1,128.12	0.00	777.89	1,906.00	1,906.00		2,065.72	
19-Aug-08	16:00	1650	1518.00		889.14	0.00	800.97	1,690.11	1,690.11		801.85		1,072.42	0.00	740.59	1,813.01	1,813.01		741.43	
19-Aug-08	17:00	1650	1518.00		873.77	0.00	714.39	1,588.17	1,588.17		1,734.40		1,053.79	0.00	666.91	1,720.70	1,720.70		2,984.02	MAX
19-Aug-08	18:00	1700	1564.00		949.59	0.00	717.64	1,667.23	1,667.23		981.13		1,137.96	0.00	669.69	1,807.65	1,807.65		1,274.80	
19-Aug-08	19:00	1800	1656.00		919.70	0.00	821.91	1,741.61	1,741.61		821.90		1,115.18	0.00	741.59	1,856.77	1,856.77		1,767.95	
19-Aug-08	20:00	1900	1748.00		970.88	0.00	844.80	1,815.68	1,815.68		844.79		1,172.34	0.00	756.73	1,929.07	1,929.07		2,309.11	
19-Aug-08	21:00	1800	1656.00		998.55	0.00	779.11	1,777.66	1,777.66		779.11		1,200.63	0.00	709.07	1,909.70	1,909.70		1,556.35	
19-Aug-08	22:00	1700	1564.00		967.87	0.00	655.00	1,622.87	1,622.87		655.00		1,156.76	0.00	625.33	1,782.09	1,782.09		2,174.20	
19-Aug-08	23:00	1550	1426.00		863.86	0.00	511.75	1,375.61	1,375.61		511.75		1,031.69	0.00	507.69	1,539.38	1,539.38		1,524.08	
										Factor		Factor						Factor		Factor
	MAX	2500	2300.00	MAX	1209.25	0.00	971.02	2151.44	2151.44	1.069051	3942.62	0.583369	1457.48	0.00	900.76	2334.52	2334.52	0.985213	2984.02	0.770773
	AVG	1585	1458.20	AVG	817.17	0.00	648.64	1465.81	1465.81	0.994808	1271.76	1.146598	981.04	0.00	600.52	1581.56	1581.56	0.922003	1691.53	0.862058
	MIN	690	634.80	MIN	331.97	0.00	211.15	543.12	543.12	1.168807	425.51	1.491862	395.72	0.00	221.35	617.07	617.07	1.028737	221.35	2.867855

Typ Day Used to Calibrate Model

Time		(gpm)				Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)	Modeled Base Flows (gpm)				Dry Calibrated Model Flows (gpm)	Calibrated Model Moving Hrly Avg (gpm)	Dry Calibrated Model Flows (gpm)
48:00:00	**1550.00**	**1426.00**	815.46	0	478.934	1,294.39	1,361.39	478.93	968.13	0	480.294	1,448.43	1,523.93	3,615.69
48:05:00	1529.17		814.50	0	468.094	1,282.59	1,347.38	468.09	965.28	0	468.589	1,433.86	1,508.64	3,611.83
48:10:00	1508.33		844.00	0	460.438	1,304.44	1,336.52	460.43	992.43	0	460.593	1,453.03	1,496.40	3,607.68
48:15:00	1487.50		894.20	0	451.445	1,345.65	1,330.72	451.44	1,039.93	0	452.194	1,492.12	1,489.11	3,590.38
48:20:00	1466.67		905.65	0	439.685	1,345.34	1,326.68	439.68	1,048.33	0	442.192	1,490.52	1,483.49	3,563.90
48:25:00	1445.83		855.02	0	426.647	1,281.67	1,319.05	426.64	994.41	0	431.75	1,426.16	1,474.22	3,537.69
48:30:00	1425.00		783.81	0	414.692	1,198.51	1,305.94	414.69	920.02	0	422.509	1,342.53	1,459.44	3,533.36
48:35:00	1404.17		732.292	0	405.447	1,137.74	1,288.86	405.45	865.799	0	415.479	1,281.28	1,440.67	3,556.42
48:40:00	1383.33		705.105	0	399.13	1,104.24	1,269.81	3,407.53	836.613	0	410.618	1,247.23	1,419.96	421.088
48:45:00	1362.50		691.082	0	394.948	1,086.03	1,249.97	3,557.68	821.232	0	407.16	1,228.39	1,398.48	407.16
48:50:00	1341.67		681.898	0	391.85	1,073.75	1,229.87	402.39	811.097	0	404.177	1,215.27	1,376.80	404.177
48:55:00	1320.83		674.11	0	389.052	1,063.16	1,209.79	389.05	802.497	0	401.011	1,203.51	1,355.19	401.01
49:00:00	**1300**	**1196**	666.52	0	386.21	1,052.73	1,189.65	386.21	794.04	0	397.451	1,191.50	1,333.78	397.451
49:05:00	1266.67		658.657	0	377.982	1,036.64	1,169.16	377.98	785.134	0	388.519	1,173.65	1,312.10	388.52
49:10:00	1233.33		650.438	0	372.064	1,022.50	1,145.66	3,502.27	775.583	0	382.287	1,157.87	1,287.50	382.29
49:15:00	1200.00		641.972	0	365.121	1,007.09	1,117.45	375.59	765.545	0	375.729	1,141.27	1,258.27	3,419.47
49:20:00	1166.67		633.107	0	356.16	989.27	1,087.78	356.16	754.848	0	368.035	1,122.88	1,227.63	3,537.25
49:25:00	1133.33		623.78	0	346.344	970.12	1,061.81	346.34	743.45	0	360.124	1,103.58	1,200.75	370.69
49:30:00	1100.00		614.50	0	337.407	951.90	1,041.26	3,358.25	732.09	0	353.187	1,085.27	1,179.31	353.187
49:35:00	1066.67		606.08	0	330.47	936.55	1,024.50	340.57	721.81	0	347.866	1,069.67	1,161.68	347.87
49:40:00	1033.33		598.998	0	325.556	924.55	1,009.52	325.55	713.184	0	343.944	1,057.13	1,145.83	343.94
49:45:00	1000.00		593.038	0	321.882	914.92	995.27	321.88	705.875	0	340.58	1,046.46	1,130.67	3,371.70
49:50:00	966.67		587.422	0	318.458	905.88	981.28	3,322.96	698.834	0	336.792	1,035.63	1,115.70	346.93
49:55:00	933.33		581.225	0	314.579	895.80	967.33	324.63	690.86	0	331.92	1,022.78	1,100.64	331.92
50:00:00	**900.00**	**828.00**	573.70	0	310.062	883.77	953.25	310.06	681.05	0	325.904	1,006.96	1,085.26	325.904
50:05:00	891.67		564.355	0	303.113	867.47	939.15	303.11	668.96	0	317.132	986.09	1,069.63	317.132
50:10:00	883.33		553.049	0	296.693	849.74	924.76	3,385.15	654.623	0	308.939	963.56	1,053.44	3,362.08
50:15:00	875.00		540.144	0	289.467	829.61	909.97	299.79	638.562	0	300.46	939.02	1,036.59	310.671
50:20:00	866.67		526.11	0	281.116	807.22	894.79	281.11	621.44	0	291.611	913.05	1,019.10	291.61
50:25:00	858.33		511.553	0	272.535	784.09	879.29	3,294.79	604.072	0	283.179	887.25	1,001.07	283.179
50:30:00	850.00		497.398	0	265.004	762.40	863.50	275.11	587.527	0	276.13	863.66	982.60	3,355.13
50:35:00	841.67		484.541	0	259.315	743.86	847.44	259.31	572.783	0	270.953	843.74	963.78	281.25





EXISTING & FUTURE CAPACITY ANALYSIS

(MODEL RESULTS)

EXISTING CAPACITY FIGURE D.1

INCREASED MAINTENANCE RECOMMENDATIONS

FUTURE CAPACITY SNAPSHOTS

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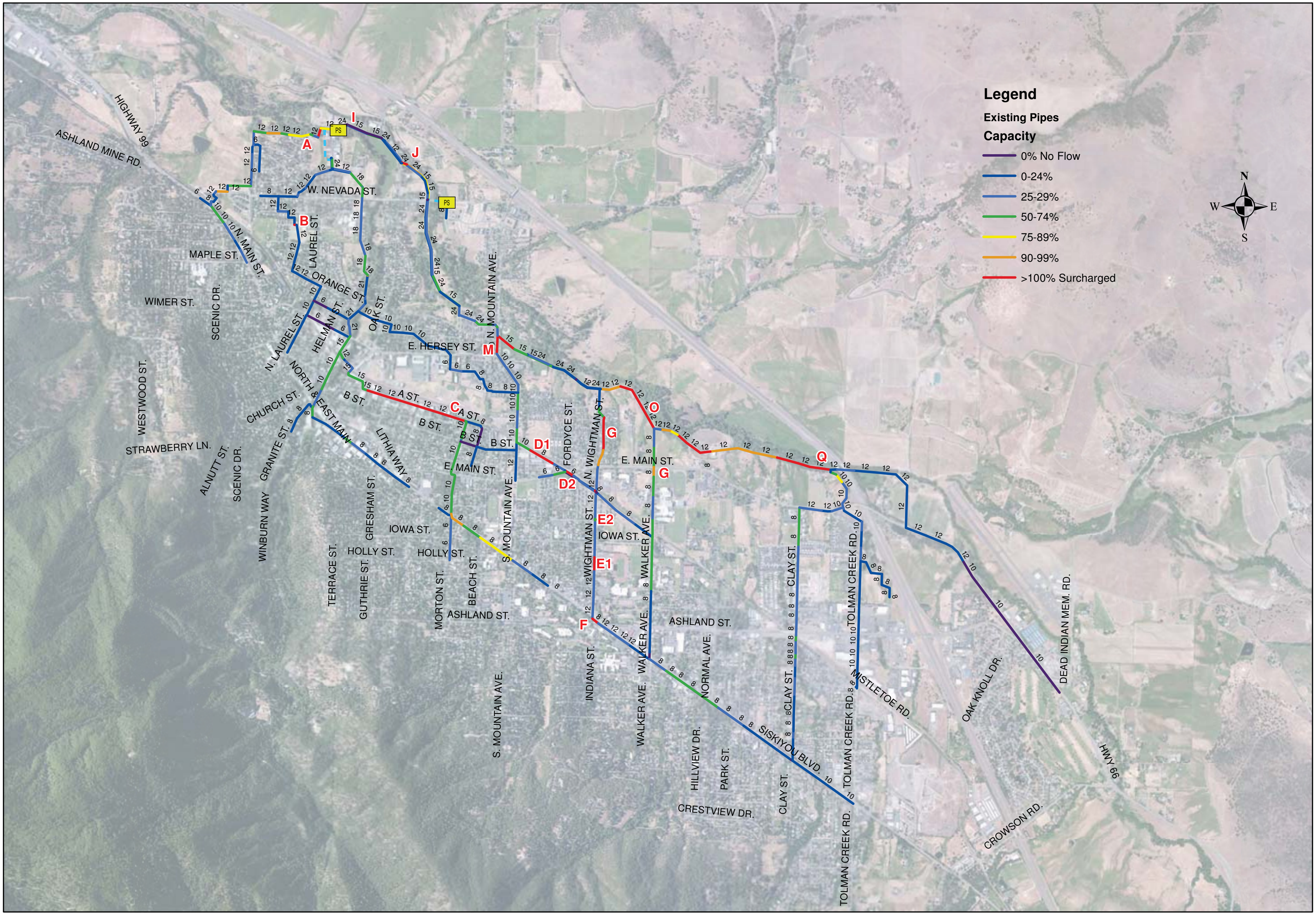


KELLER associates

Prepared for:
CITY of ASHLAND, OREGON

WASTEWATER SYSTEM MASTER PLAN

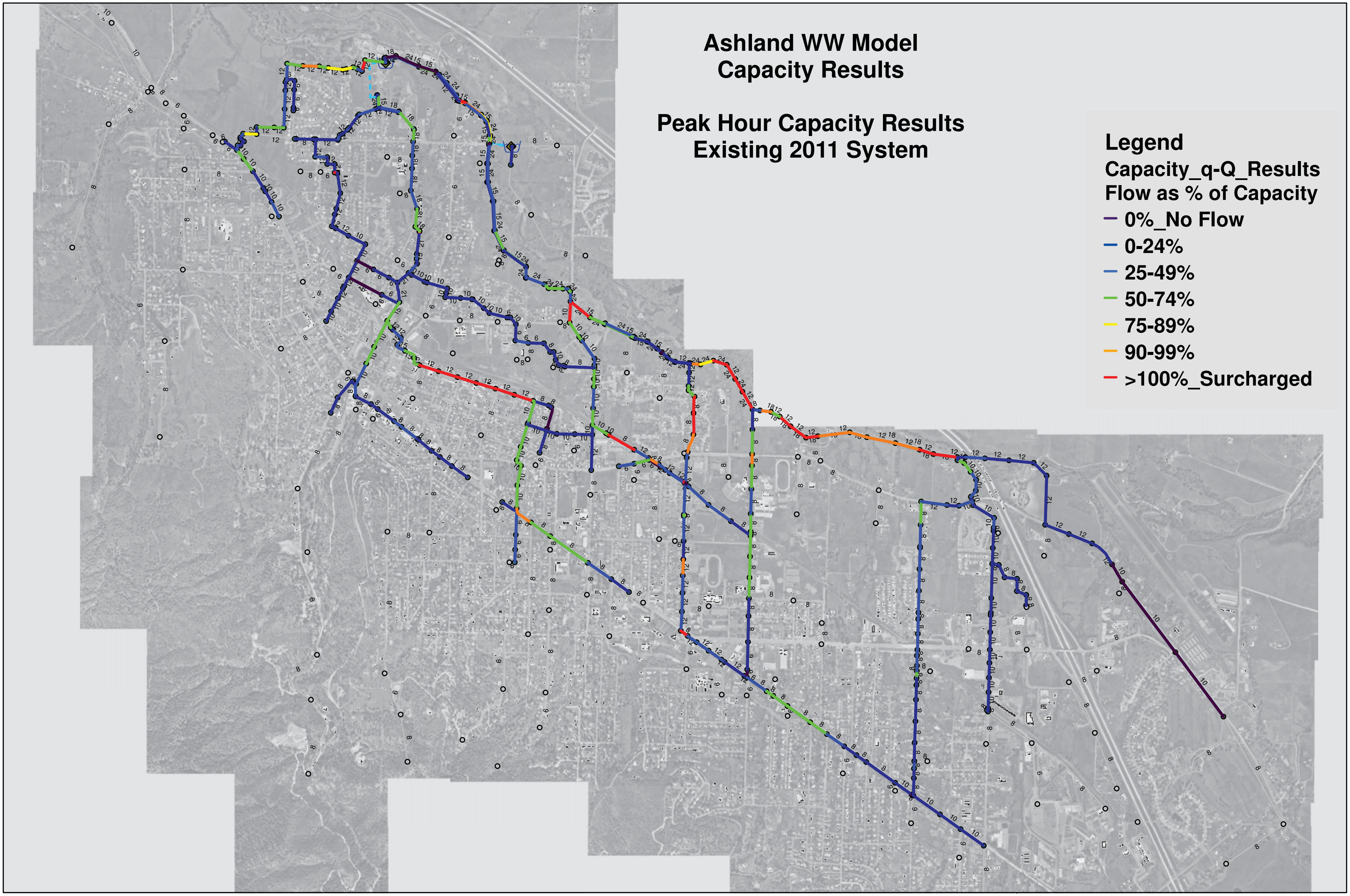
Title:
POTENTIAL OVERFLOWS

Figure:
D.1

Ashland, OR
Wastewater System Master Plan
Schedule of Manholes and Pipes
Recommended for Increased Maintenance

Potential Overflow Sites				Bottlenecks / Localized Surcharged Pipes						
Map ID	**Location**	**MHID**		**Map ID**	**Location**	**Upstream MH**	**to**	**Downstream MH**	**Size**	**Design Issue**
a	East of Ash.Cr. LS	4BB-005 4BB-039								
				b	Randy St.	4BC-005	to	4BC-006	12"	flat
c	"A" St.	9AB-015 9AB-013								
d1	RR near Main St.	10BC-042		d2	RR & Main St.	10BC-042 10BC-023	to to	10BC-040 10BC-008	8"	bottleneck between 10" & 12"
				e1 e2	Wightman St.	10CD-013 10CA-003	to to	10CA-011 10CA-012	12" 8"	low slope bottleneck in 12"
				f	Siskiyou Blvd. & Wightman St.	10CD-004	to	10CD-005	8"	bottleneck in 12"
				g	Wightman St.	10BD-013	to	10BA-018	8"	bottleneck in 12"
				h	Walker Ave.	10AC-003	to	10AC-002	8"	flat
i	West of Ash. Cr. LS	33CC-007								
				j	Bear Cr. - Oak St.	4BA-011	to	4BA-014	15"	bottleneck in 24"
m	N. Mountain Ave.	4DD-024		m	N. Mountain Ave.	4DD-024	to	4DD-008	10"	flat
o	Bear Cr. - Walker Ave.	10AB-004 10BA-029 10BA-028								
				q	Bear Cr. - I-5	11BC-006	to	11BC-005	12"	flat

k,l,n,p *(not used)*



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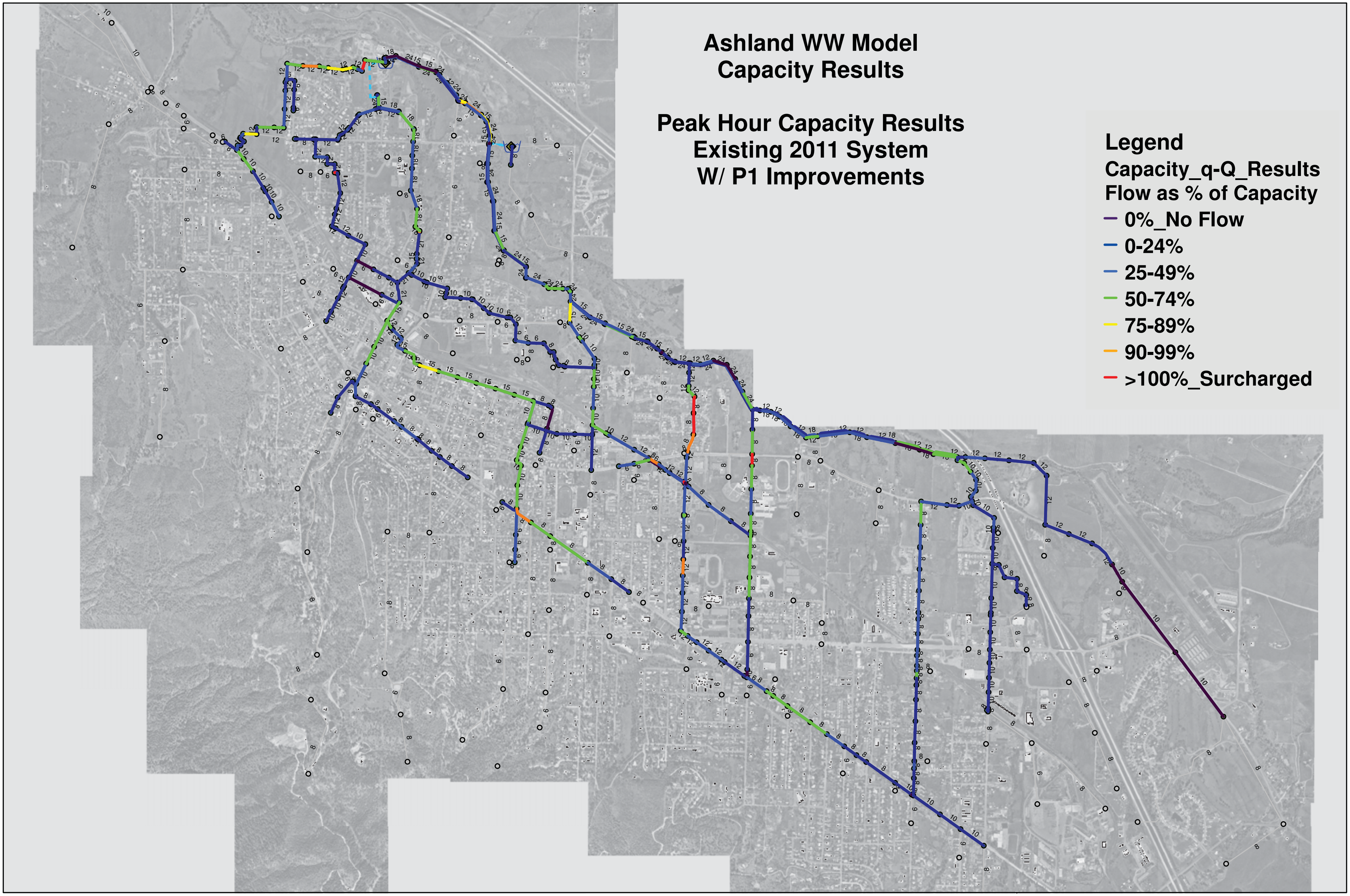

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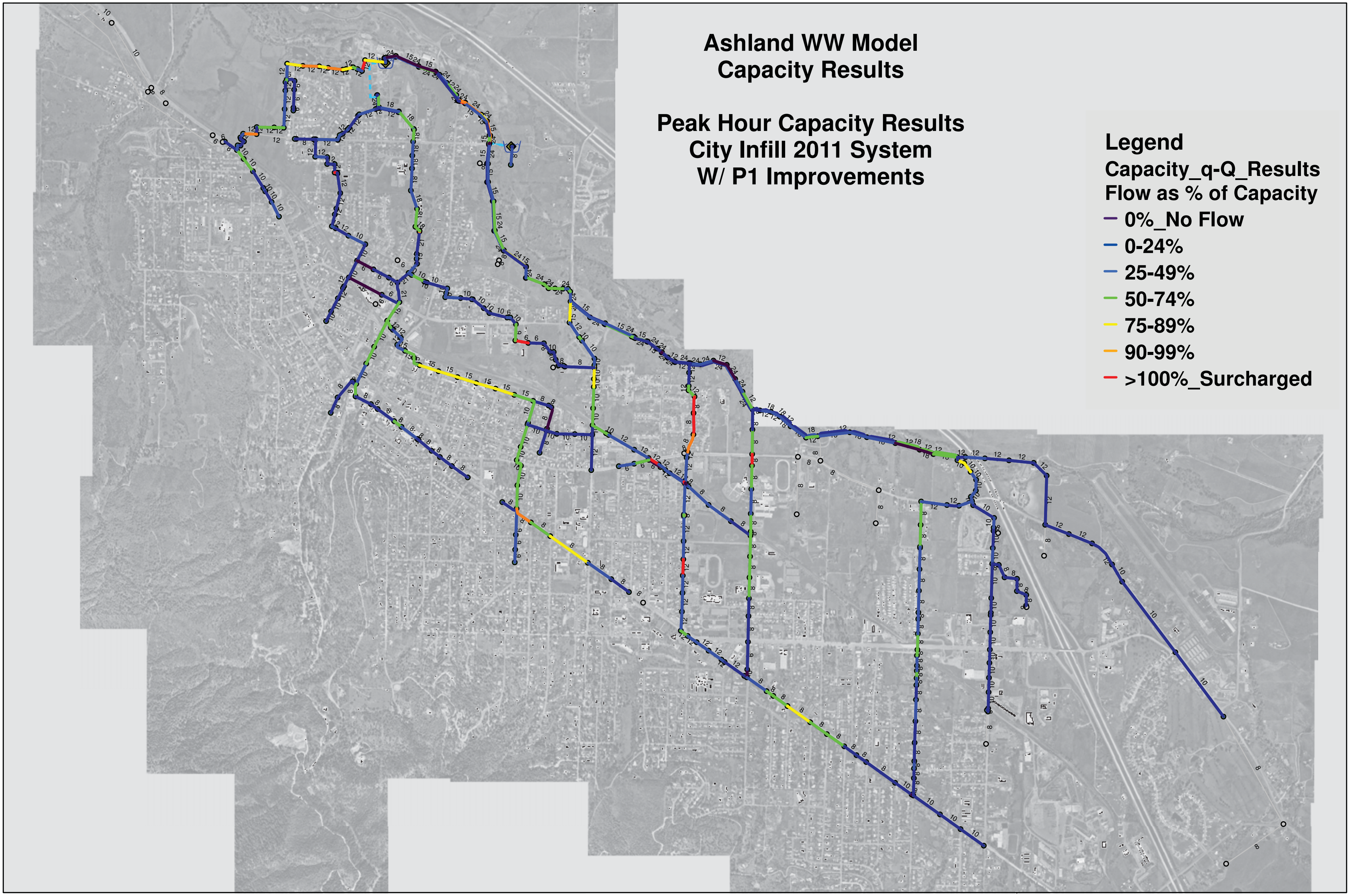
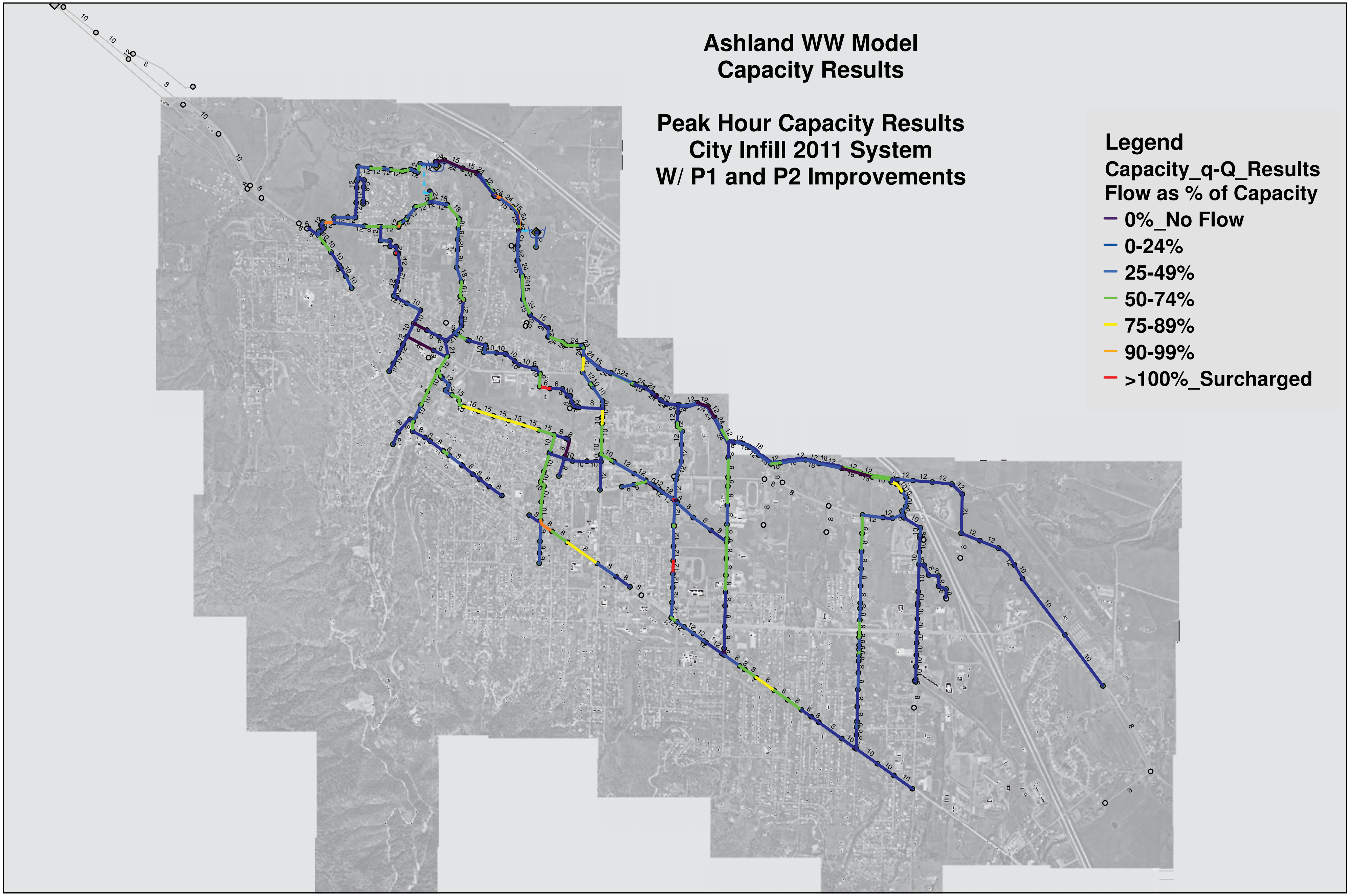
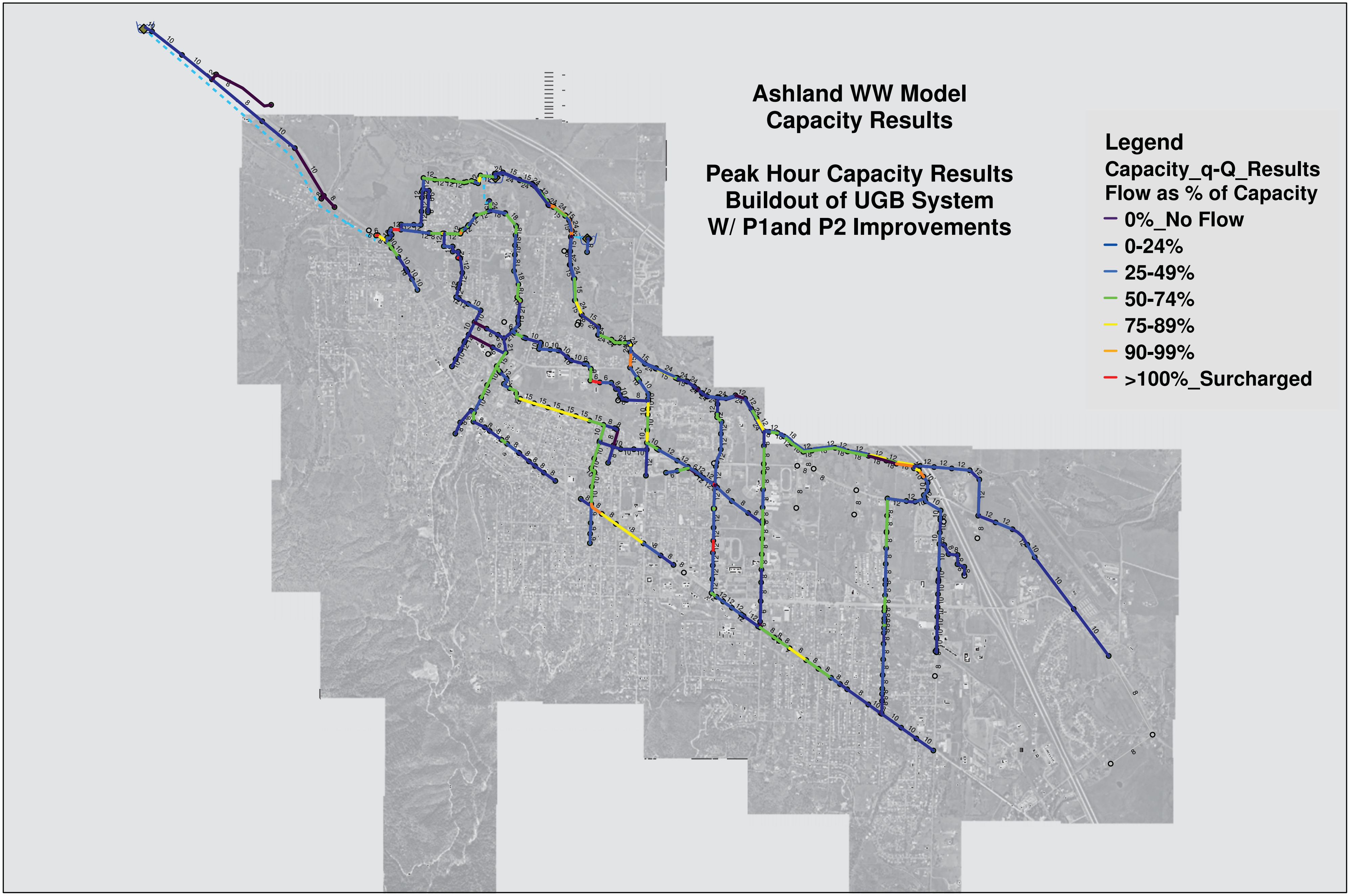
Path: J:\210055\Design\KellerCollectionModel\KellerCollectionModel\AshlandModel_Current.mxd Date: 9/26/2011 Scale 1:25,000





Path: J:\210055\Design\KellerCollectionModel\KellerCollectionModel\AshlandModel_Current.mxd   Date: 9/26/2011   Scale 1:25,000



Path: J:\210055\Design\KellerCollectionModel\KellerCollectionModel\AshlandModel_Current.mxd Date: 9/26/2011 Scale 1:25,000

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APPENDIX E

Treatment Evaluation Support Data

- **Membrane Replacement Tech Memo**
- **Staffing Analysis**
- **Recycle/Reuse Analysis**
- **Hyporheic Evaluation**
- **Carbon Footprint Evaluation**

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Membrane Replacement

Technical Memorandum

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CITY OF ASHLAND
OREGON

Wastewater Treatment Plant Membrane Replacement Options

FINAL MEMORANDUM

September 13, 2010

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210055-000

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131 SW 5th Ave.
Meridian, ID 83642
208-288-1992

FINAL MEMORANDUM

Date: SEPTEMBER 13, 2010

To: SCOTT FLEURY AND MIKE FAUGHT, CITY OF ASHLAND

From: LARRY RUPP

Subject: **CITY OF ASHLAND WWTP MEMBRANE REPLACEMENT OPTIONS**

This memorandum summarizes the review of the membrane replacement options for the Ashland Wastewater Treatment Plant (WWTP). The need for this review is due to the concern with the condition of the existing membranes and the need for additional capacity. Below is a summary of items reviewed in providing our recommendation to the City of Ashland.

1. Review the existing membrane system.
2. Review the membrane inspection spreadsheet provided by Ashland staff.
3. Perform a hydraulic analysis to determine other components that may need to be replaced if a higher capacity membrane is installed.
4. Review membrane replacement options from other potential suppliers to determine if a feasible option exists that will fit within the existing system.
5. Review membrane replacement options from Zenon.
6. Provide a recommendation for membrane replacement.

In addition to the steps listed above, an attempt was made to review the historical operational data (cleaning intervals, permeability, and turbidity) that are available on ZenoTrac. To date, the ZenoTrac data has not been reviewed. The review is pending based on feedback from Zenon. Depending on the data available, it could be used to recommend a higher or lower flux rate and to better determine if the existing membranes are approaching their existing useful life.

EXISTING MEMBRANE SYSTEM

The existing membranes have been in operation since May of 2002. Since the original commissioning, an additional 10 % capacity was added in January 2008 by installing membranes similar to the original membranes in the remaining basin area. Current flows indicate the need for additional capacity is approaching. Another concern is the life of the existing membranes. Additionally, the City has a price guarantee for membrane replacement that is due to expire April 4, 2011.

Plant staff conducted an inspection of the membranes (See summary of results in Table 1). The inspection revealed that a number of fibers are separating from the urethane potting. This is likely due to over exposure of chlorine. Approximately 25% of the membrane cassettes have more than half of the fibers loose.

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Table 1 – Condition of Existing Membrane Cassettes

Table 1 - Condition of Esisting Membrane Cassettes

Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose	sf/cass.	g/sf/d	flow	MGD
1	1	Installed January 2008																										0	220	12	68,640	
1	2	Installed 2002																										5	220	12	68,640	
1	3	Installed 2002																										25	220	12	68,640	
1	4	Installed 2002																										0	220	12	68,640	
1	5	Installed 2002																										30	220	12	68,640	
1	6	Installed 2002																										0	220	12	68,640	
1	7	Installed 2002																										50	220	12	68,640	
1	8	Installed 2002																										0	220	12	68,640	
1	9	Installed 2002																										30	220	12	68,640	
1	10	Installed 2002																										50	220	12	68,640	
Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose				0.686
2	1	Installed January 2008																										0	220	12	68,640	
2	2	Installed 2002																										10	220	12	68,640	
2	3	Installed 2002																										5	220	12	68,640	
2	4	Installed 2002																										0	220	12	68,640	
2	5	Installed 2002																										5	220	12	68,640	
2	6	Installed 2002																										50	220	12	68,640	
2	7	Installed 2002																										50	220	12	68,640	
2	8	Installed 2002																										30	220	12	68,640	
2	9	Installed 2002																										50	220	12	68,640	
2	10	Installed 2002																										50	220	12	68,640	
Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose				0.686
3	1	Installed January 2008																										0	220	12	68,640	
3	2	Installed 2002																										0	220	12	68,640	
3	3	Installed 2002																										0	220	12	68,640	
3	4	Installed 2002																										0	220	12	68,640	
3	5	Installed 2002																										0	220	12	68,640	
3	6	Installed 2002																										0	220	12	68,640	
3	7	Installed 2002																										100	220	12	68,640	
3	8	Installed 2002																										100	220	12	68,640	
3	9	Installed 2002																										100	220	12	68,640	
3	10	Installed 2002																										100	220	12	68,640	
																																0.686
Trains	Cassettes																											Percent Loose				
4	1	Installed January 2008																										0	220	12	68,640	
4	2	Installed 2002																										0	220	12	68,640	
4	3	Installed 2002																										0	220	12	68,640	
4	4	Installed 2002																										0	220	12	68,640	
4	5	Installed 2002																										0	220	12	68,640	
4	6	Installed 2002																										0	220	12	68,640	
4	7	Installed 2002																										0	220	12	68,640	
4	8	Installed 2002																										0	220	12	68,640	
4	9	Installed 2002																										0	220	12	68,640	
4	10	Installed 2002																										0	220	12	68,640	
0 % Loose																																0.686
1-49 % Loose																																
50-99 % Loose																															2008 Total	2.746
100 % Loose																																
																															2002 Total	2.471

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Another consideration is the life expectancy of the membrane fibers. With Ashland being one of the first tertiary membrane installations, data to determine the life expectancy is not available. Membrane life can vary widely depending on the operating conditions, chemical exposure, membrane materials, and other factors. Loss of permeability even after cleanings is typically an indication of the need to replace membranes. A life expectancy of 10 years is not uncommon for wastewater. For the Ashland WWTP, an even longer life expectancy could be realized as the membranes are only operated 7 months a year.

HYDRAULIC ANALYSIS

The membrane system was modeled hydraulically using spreadsheet calculations. Initially the limiting factor hydraulically is the permeate pumps which have a capacity of 1.13 mgd for a total capacity of 4.5 mgd. The permeate piping is also designed to handle approximately 4.5 mgd. Any membrane capacity expansion beyond 4.5 mgd should include a replacement of both permeate pumps and piping.

POTENTIAL MEMBRANE SUPPLIERS

In addition to the original membrane supplier (GE/Zenon), Koch Membrane Systems (Puron) also provides cassettes/modules that are made to replace GE/Zenon membranes. A proposal from Koch for membrane replacement at the Ashland WWTP is included in Appendix A. The cost for Koch membranes is approximately $6.00 to $6.50 per $ft.^2$. This is slightly less when compared to GE/Zenon replacement cost shown in Table 2. If this option is pursued, it is recommended that further detailed design level evaluation be completed in order to verify compatibility and identify any required modifications to the existing system.

GE/ZENON REPLACEMENT OPTIONS

GE/Zenon currently manufactures three feasible options for replacing the membranes at the Ashland WWTP. In order of the least to most capacity the options are ZW500C-250, ZW500D-340, and ZW500D-440 with the last three numbers corresponding to the amount of membrane surface area per module. Appendix B contains a copy of GE/Zenon's proposed scope of replacement and budgetary pricing. Table 2 summarizes the GE/Zenon's options. Options 4 and 5 are not recommended at this time as they require a major upgrade of the membrane system. Using the ZW500D-340 would add very little capacity because the current configuration will only fit 20 modules per cassette versus the existing 26 modules per cassette. Similarly the ZW500D-440 option would only be necessary if peak flows require membrane treatment.

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Table 2 - Membrane Replacement Options Cost Summary

	Quoted Price	# of Modules	SF/Modules	$/SF
2W500C-2505	$1.75 Million	936	250	$7.48
2W500D-3405*	$2.1 - 2.5 Million	800	340	$7.72 - 9.19
2W500D-4405*	$2.3 - 2.7 Million	752	440	$6.95 - 8.16

* Includes cost for modifying blower capacity

Figure 1 shows a recommended replacement schedule. This schedule is based on the maximum month flow and population projections to date (@ 1.66% growth rate) and may require revision as these projections are finalized with the master plan. The details of the membrane replacement are shown in Tables 3-5. For example Table 3 shows a recommendation for replacing modules and moving other existing modules. The result shown in Table 3 are obtained by replacing the damaged cassettes in trains 1, 2, and 3 with new cassettes and moving the good cassettes from train 1 and train 2 to train 3. The result is membranes in the worst condition are replaced and there are two trains of new membranes.

Figure 1 - Projected Membrane Replacement Schedule

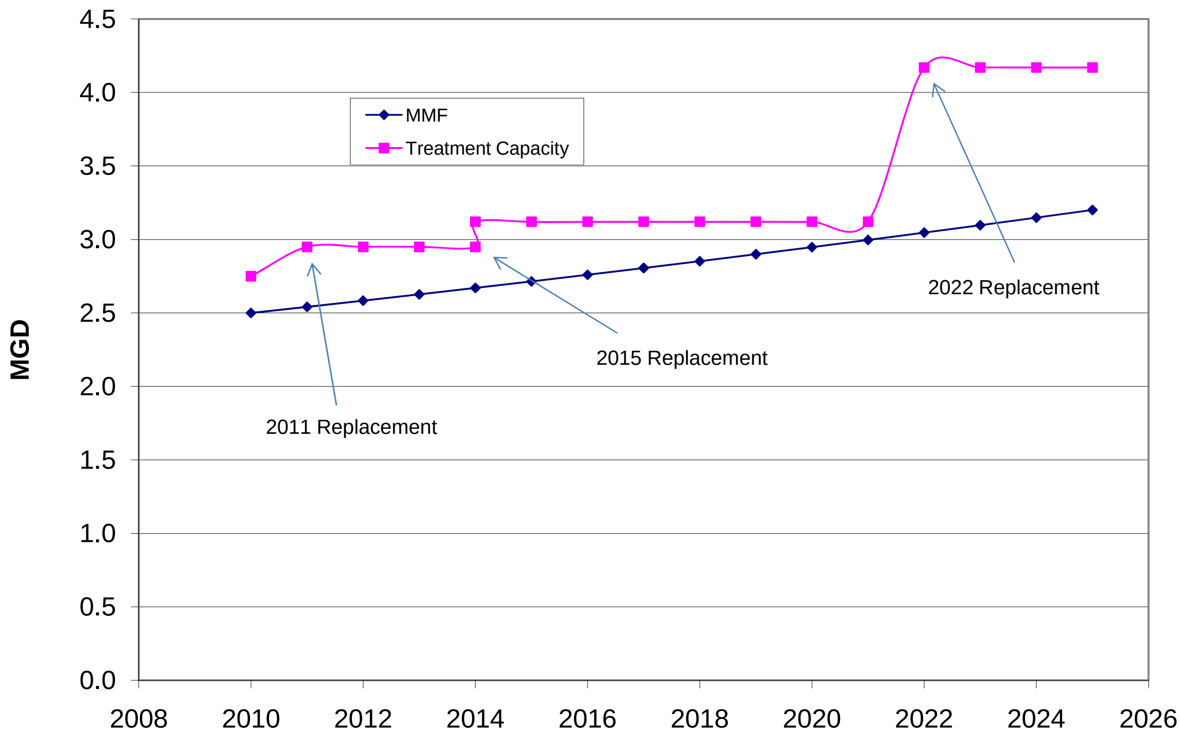

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Table 3 – Phase 1 Replacement

Table 3 - Phase I Replacement

Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose	sf/cass.	g/sf/d	flow	MGD
1	1	Installed January 2008																									0	250	12	78,000	
1	2	Installed January 2011																									0	250	12	78,000	
1	3	Installed January 2011																									0	250	12	78,000	
1	4	Installed January 2011																									0	250	12	78,000	
1	5	Installed January 2011																									0	250	12	78,000	
1	6	Installed January 2011																									0	250	12	78,000	
1	7	Installed January 2011																									0	250	12	78,000	
1	8	Installed January 2011																									0	250	12	78,000	
1	9	Installed January 2011																									0	250	12	78,000	
1	10	Installed January 2011																									0	250	12	78,000	
Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose				0.780
2	1	Installed January 2008																									0	250	12	78,000	
2	2	Installed January 2011																									0	250	12	78,000	
2	3	Installed January 2011																									0	250	12	78,000	
2	4	Installed January 2011																									0	250	12	78,000	
2	5	Installed January 2011																									0	250	12	78,000	
2	6	Installed January 2011																									0	250	12	78,000	
2	7	Installed January 2011																									0	250	12	78,000	
2	8	Installed January 2011																									0	250	12	78,000	
2	9	Installed January 2011																									0	250	12	78,000	
2	10	Installed January 2011																									0	250	12	78,000	
Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose				0.780
3	1	Installed January 2008																									0	250	12	78,000	
3	2	Installed 2002																									0	220	12	68,640	
3	3	Installed 2002																									0	220	12	68,640	
3	4	Installed 2002																									0	220	12	68,640	
3	5	Installed 2002																									0	220	12	68,640	
3	6	Installed 2002																									0	220	12	68,640	
3	7	Installed 2002																									0	220	12	68,640	
3	8	Installed 2002																									0	220	12	68,640	
3	9	Installed 2002																									0	220	12	68,640	
3	10	Installed 2002																									0	220	12	68,640	
																																0.696
Trains	Cassettes																											Percent Loose				
4	1	Installed January 2008																									0	250	12	78,000	
4	2	Installed 2002																									0	220	12	68,640	
4	3	Installed 2002																									0	220	12	68,640	
4	4	Installed 2002																									0	220	12	68,640	
4	5	Installed 2002																									0	220	12	68,640	
4	6	Installed 2002																									0	220	12	68,640	
4	7	Installed 2002																									0	220	12	68,640	
4	8	Installed 2002																									0	220	12	68,640	
4	9	Installed 2002																									0	220	12	68,640	
4	10	Installed 2002																									0	220	12	68,640	
0 % Loose																																0.696
1-49 % Loose																																
50-99 % Loose																																2.952
100 % Loose																																
Moved from one train to another																																
New																																

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Table 4 – Phase 2 Replacement

Table 4 - Phase 2 Replacement

Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose	sf/cass.	g/sf/d	flow	MGD
1	1	Installed January 2008																									0	250	12	78,000	
1	2	Installed January 2011																									0	250	12	78,000	
1	3	Installed January 2011																									0	250	12	78,000	
1	4	Installed January 2011																									0	250	12	78,000	
1	5	Installed January 2011																									0	250	12	78,000	
1	6	Installed January 2011																									0	250	12	78,000	
1	7	Installed January 2011																									0	250	12	78,000	
1	8	Installed January 2011																									0	250	12	78,000	
1	9	Installed January 2011																									0	250	12	78,000	
1	10	Installed January 2011																									0	250	12	78,000	
Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose				0.780
2	1	Installed January 2008																									0	250	12	78,000	
2	2	Installed January 2011																									0	250	12	78,000	
2	3	Installed January 2011																									0	250	12	78,000	
2	4	Installed January 2011																									0	250	12	78,000	
2	5	Installed January 2011																									0	250	12	78,000	
2	6	Installed January 2011																									0	250	12	78,000	
2	7	Installed January 2011																									0	250	12	78,000	
2	8	Installed January 2011																									0	250	12	78,000	
2	9	Installed January 2011																									0	250	12	78,000	
2	10	Installed January 2011																									0	250	12	78,000	
Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose				0.780
3	1	Installed January 2008																									0	250	12	78,000	
3	2	Installed January 2015																									0	250	12	78,000	
3	3	Installed January 2015																									0	250	12	78,000	
3	4	Installed January 2015																									0	250	12	78,000	
3	5	Installed January 2015																									0	250	12	78,000	
3	6	Installed January 2015																									0	250	12	78,000	
3	7	Installed January 2015																									0	250	12	78,000	
3	8	Installed January 2015																									0	250	12	78,000	
3	9	Installed January 2015																									0	250	12	78,000	
3	10	Installed January 2015																									0	250	12	78,000	
																																0.780
Trains	Cassettes																											Percent Loose				
4	1	Installed January 2008																									0	250	12	78,000	
4	2	Installed January 2015																									0	250	12	78,000	
4	3	Installed January 2015																									0	250	12	78,000	
4	4	Installed January 2015																									0	250	12	78,000	
4	5	Installed January 2015																									0	250	12	78,000	
4	6	Installed January 2015																									0	250	12	78,000	
4	7	Installed January 2015																									0	250	12	78,000	
4	8	Installed January 2015																									0	250	12	78,000	
4	9	Installed January 2015																									0	250	12	78,000	
4	10	Installed January 2015																									0	250	12	78,000	
0 % Loose																																0.780
1-49 % Loose																																
50-99 % Loose																																3.120
100 % Loose																																
Moved from one train to another																																
New																																

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Table 5 - Phase 3 Replacement

Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose	sf/cass.	g/sf/d	flow	MGD
1	1	Installed January 2022																									0	340	12	261,120	
1	2	Installed January 2022																									0	340	12	261,120	
1	3	Installed January 2022																									0	340	12	261,120	
1	4	Installed January 2022																									0	340	12	261,120	
1	5	Installed January 2022																									0	340	12	261,120	
1	6																											0		12	0	
1	7																											0		12	0	
1	8																											0		12	0	
1	9																											0		12	0	
1	10																											0		12	0	
Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose				1.306
2	1	Installed January 2022																									0	340	12	261,120	
2	2	Installed January 2022																									0	340	12	261,120	
2	3	Installed January 2022																									0	340	12	261,120	
2	4	Installed January 2022																									0	340	12	261,120	
2	5	Installed January 2022																									0	340	12	261,120	
2	6																											0		12	0	
2	7																											0		12	0	
2	8																											0		12	0	
2	9																											0		12	0	
2	10																											0		12	0	
Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose				1.306
3	1	Installed January 2011																									0	250	12	78,000	
3	2	Installed January 2015																									0	250	12	78,000	
3	3	Installed January 2015																									0	250	12	78,000	
3	4	Installed January 2015																									0	250	12	78,000	
3	5	Installed January 2015																									0	250	12	78,000	
3	6	Installed January 2015																									0	250	12	78,000	
3	7	Installed January 2015																									0	250	12	78,000	
3	8	Installed January 2015																									0	250	12	78,000	
3	9	Installed January 2015																									0	250	12	78,000	
3	10	Installed January 2015																									0	250	12	78,000	
																																0.780
Trains	Cassettes																											Percent Loose				
4	1	Installed January 2011																									0	250	12	78,000	
4	2	Installed January 2015																									0	250	12	78,000	
4	3	Installed January 2015																									0	250	12	78,000	
4	4	Installed January 2015																									0	250	12	78,000	
4	5	Installed January 2015																									0	250	12	78,000	
4	6	Installed January 2015																									0	250	12	78,000	
4	7	Installed January 2015																									0	250	12	78,000	
4	8	Installed January 2015																									0	250	12	78,000	
4	9	Installed January 2015																									0	250	12	78,000	
4	10	Installed January 2015																									0	250	12	78,000	
0 % Loose																																0.780
1-49 % Loose																																
50-99 % Loose																																4.171
100 % Loose																																
Moved from one train to another																																
New																																

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The future recommendations account for future flows and should be re-evaluated before proceeding with membrane replacement. The membrane replacement recommendations are based on both meeting capacity objective and on an expected membrane life of 10 to 15 years. Another approach that could be employed would be to replace based on capacity and continue to use the membranes until a drop in performance is noticed. This approach would result in pushing the membranes to the edge of their useful life versus replacing membranes based on expected useful life. Table 6 shows the membrane replacement for phase I if this philosophy is employed.

It should be noted that the recommendations are based on being able to treat maximum month flows. Peak hour flows would bypass the tertiary membrane system. For the purposes of meeting the effluent phosphorus limit treating maximum month flows appears adequate for the near future. Due to the phosphorus limit being load based, the amount of flow that can be bypassed while still meeting the limit will decrease as overall flows increase. As the flow increase requires lower and lower effluent phosphorus concentrations, other options should be explored for meeting the phosphorus limit. If the City desires to treat all flows including peaks, additional membrane capacity will be required. This should be considered as temperature options which may require membrane treated effluent are explored and finalized.

MEMBRANE REPLACEMENT RECOMMENDATION SUMMARY

It is understood that the chosen membrane replacement option which meets the City's budget is as shown in Table 6. This is a less-conservative approach than replacing membranes in both train 1 and train 2. However, with the outstanding temperature issue, which could affect the membrane design and use, this approach is more fiscally conservative. Once a direction is known on the temperature issue, it is recommended to revisit replacing train 2 within the next year or two, as shown in Table 3.

Based on the review conducted, it is recommended to provide 9 new ZW500C- 250 cassettes to train (as shown in Table 6). The estimated replacement cost based on information provided by GE/Zenon is $400,000. This will allow the City to accomplish the following:

- Replace damaged cassettes
- Take advantage of the replacement price guarantee for one-quarter the membranes
- Upgrade the capacity
- Decision on majority of membrane replacement can be made after knowing the affect of the meeting the future temperature limit on tertiary treatment

In addition, the good cassettes should be moved to train 3, as shown in Table 6, to replace those that have fibers separated from the potting.

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Table 6 – Phase 1b Replacement Option

Table 6 - Phase 1b Replacement Option

Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose	sf/cass.	g/sf/d	flow	MGD
1	1	Installed January 2008																										0	250	12	78,000	
1	2	Installed 2002																										0	220	12	68,640	
1	3	Installed 2002																										0	220	12	68,640	
1	4	Installed 2002																										0	220	12	68,640	
1	5	Installed 2002																										0	220	12	68,640	
1	6	Installed 2002																										0	220	12	68,640	
1	7	Installed 2002																										0	220	12	68,640	
1	8	Installed 2002																										0	220	12	68,640	
1	9	Installed 2002																										0	220	12	68,640	
1	10	Installed 2002																										0	220	12	68,640	
Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose				0.696
2	1	Installed January 2008																										0	250	12	78,000	
2	2	Installed January 2011																										25	250	12	78,000	
2	3	Installed January 2011																										25	250	12	78,000	
1	2	Installed January 2011																										25	250	12	78,000	
2	5	Installed January 2011																										25	250	12	78,000	
1	3	Installed January 2011																										25	250	12	78,000	
1	5	Installed January 2011																										25	250	12	78,000	
2	8	Installed January 2011																										25	250	12	78,000	
1	7	Installed January 2011																										25	250	12	78,000	
1	9	Installed January 2011																										25	250	12	78,000	
Trains	Cassettes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	Percent Loose				0.780
3	1	Installed January 2008																										0	250	12	78,000	
3	2	Installed 2002																										0	220	12	68,640	
3	3	Installed 2002																										0	220	12	68,640	
3	4	Installed 2002																										0	220	12	68,640	
3	5	Installed 2002																										0	220	12	68,640	
3	6	Installed 2002																										0	220	12	68,640	
1	4	Installed 2002																										0	220	12	68,640	
1	6	Installed 2002																										0	220	12	68,640	
1	8	Installed 2002																										0	220	12	68,640	
2	4	Installed 2002																										0	220	12	68,640	
																																0.696
Trains	Cassettes																											Percent Loose				
4	1	Installed January 2008																										0	250	12	78,000	
4	2	Installed 2002																										0	220	12	68,640	
4	3	Installed 2002																										0	220	12	68,640	
4	4	Installed 2002																										0	220	12	68,640	
4	5	Installed 2002																										0	220	12	68,640	
4	6	Installed 2002																										0	220	12	68,640	
4	7	Installed 2002																										0	220	12	68,640	
4	8	Installed 2002																										0	220	12	68,640	
4	9	Installed 2002																										0	220	12	68,640	
4	10	Installed 2002																										0	220	12	68,640	
0 % Loose																																0.696
1-49 % Loose																																
50-99 % Loose																																2.867
100 % Loose																																
Moved from one train to another																																
New																																

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Reasons for proceeding with the option shown in Table 3 in the next two years include:

- Some damaged cassettes are not replaced
- The option shown in Table 3 better fits a long-term replacement schedule when considering expected membrane life

It is recommended that this evaluation be performed again prior to future membrane replacement (targeted for 2015) to determine if the replacement shown in table 4 is still the best option. If peak capacity is to be met or once the max month capacity exceeds 4.0 mgd, new piping, pumps, and blower modifications will be required. At that time, converting to a higher capacity membrane would also be necessary.

STAFFING ANALYSIS

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ASHLAND WWTP MANHOURS per NEIWPCC charts (1.0-5.0 mgd)

Chart 1 Operation	*manhrs*	*# staff*
Preliminary Treat	320	
Ox Ditch w BNR	3200	
Phos Removal *6 mos*	160	
MBR *6 mos*	80	
Plant reuse water	32	
UV Disinfection	320	
Wet Odor Control	320	
Dry Odor Control	160	
Post aeration *estimated*	150	
	4742	3.16

Chart 2 Maintenance	*manhrs*	*# staff*
Mech screen (1)	80	
Gravity grit (1)	48	
Chem add'n (2)	64	
Circular clarifiers (3)	480	
Pumps	250	
Mech mixers (2)	64	
Blowers *(4) 6 mos*	128	
MBR (*40 cartridges) 6mos*	640	
Centrifuge (2)	96	
UV (8 racks)	256	
Activated carbon (2)	320	
Probes (2)	64	
	2490	1.66

Chart 3 Lab	*manhrs*	*# staff*
	260	
	104	
	15	
	6	
	39	
	208	
	120	
	10	
	312	
	91	
	156	
	1321	0.88

	NEIWPCC	*Actual (from Staffing sprdsht)*
Biosolids	960	639
	1280	485
	2240	**1124**
		see totals below

Chart 5 Yardwork	580	0.39

	NEIWPCC	Adjusted sludge handling	
TOTAL MANHRS	**11,373**	**10,257**	
	7.6	6.8	*1500 mh/person/yr*

CHART 1 (One-Plus Shift)
BASIC AND ADVANCED OPERATIONS AND PROCESSES

	Flow						
Process	0.25-0.5 mgd	0.5-1.0 mgd	1.0-5.0 mgd	5.0-10.0 mgd	10.0-20.0 mgd	>20 mgd	Total Hours for Plant
Preliminary Treatment	160	160	320	640	960	1280	320
Primary Clarification (mult. by # of units)	160	160	160	320	320	320	
Activated Sludge	640	1280	1920	1920-2560	2560-3200	7680	
Activated Sludge w/BNR	960	1920	2560	2880-3840	3840-7680	8960	
Rotating Biological Contactor	320	480-960	960-1920	1920	X	X	
Sequencing Batch Reactor (per tank)	320	320	320	320	320	320	
Extended Aeration (w/o primary)	800	1600	2560	X	X	X	
Extended Aeration w/BNR	1120	2240	3200	X	X	X	
Pure Oxygen Facility	X	X	X	2560-3200	3200	5760	
Pure Oxygen Facility w/BNR	X	X	X	3200-4800	4800	7680	
Trickling Filter	320	320	640	960	1280	2560	
Oxidation Ditch (w/o primary)	800	1600	2560	X	X	X	
Oxidation Ditch w/BNR	1120	2240	3200	X	X	X	3200
Aeration Lagoon	480	480	480	X	X	X	
Stabilization Pond	320	320	320	X	X	X	
Innovative Alternative Technologies	640	960	X	X	X	X	
Nitrification	80	80	160	160	320	640	
Denitrification	80	80	160	160	320	640	
Phosphorus Removal (Biological)	80	80	160	160	320	640	

Continued on page 36

CHART 1 (One-Plus Shift) *continued*
BASIC AND ADVANCED OPERATIONS AND PROCESSES

	Flow						
Process	**0.25-0.5 mgd**	**0.5-1.0 mgd**	**1.0-5.0 mgd**	**5.0-10.0 mgd**	**10.0-20.0 mgd**	**>20 mgd**	**Total Hours for Plant**
Phosphorus Removal (Chemical/Physical)	80	160	320	640	960	1280	x 0.5 = 160
Membrane Processes	80	80	160	160	320	320	x 0.5 = 80
Cloth Filtration	80	80	160	160	160	160	
Granular Media Filters (Carbon, sand, anthracite, garnet)	160	320	320	480	480	960	
Water Reuse	80	80	160	160	160	160	
Plant Reuse Water	32	32	32	48	80	80	32
Chlorination	160	160	320	320	320	320	
Dechlorination	160	160	320	320	320	320	
Ultraviolet Disinfection	160	160	320	320	320	320	320
Wet Odor Control (mult. by # of systems)	160	160	320	320	320	320	320
Dry Odor Control (mult. by # of systems)	80	80	160	160	160	160	160
Septage Handling	160	160	320	320	320	320	
Post Aeration			150				150
TOTAL							4742

- Activated Sludge process includes RAS and WAS pumping.
- Secondary Clarification has been built into basic operations processes.

CHART 2 (One-Plus Shift)
MAINTENANCE

	Flow							
Activity	0.25-0.5 mgd	0.5-1.0 mgd	1.0-5.0 mgd	5.0-10.0 mgd	10.0-20.0 mgd	>20 mgd	Multiply by	Total Hours for Plant
Manually Cleaned Screens	80	80	80	80	160	320	# of screens	
Mechanically Cleaned Screens	80	80	80	320	960	1280	# of screens	x1 = 80
Mechanically Cleaned Screens with grinders/ washer/compactors	80	160	320	640	1280	1600	# of screens	
Comminutors/ Macerators	80	80	80	160	240	320	# of units	
Aerated Grit Chambers	32	32	80	160	240	320	# of chambers	
Vortex Grit Removal	32	32	80	160	240	320	# of units	
Gravity Grit Removal	32	32	48	64	80	160	# of units	x1 = 48
Additional Process Tanks	32	32	32	32	32	32	# of tanks	
Chemical Addition (varying dependent upon degree of treatment)	32	32	32	32-96	96-192	256	# of chemicals added for processes	x2 = 64
Circular Clarifiers	80	80	160	160	240	320	# of clarifiers	x3= 480
Chain and Flight Clarifiers	80	80	160	160	240	320	# of clarifiers	
Traveling Bridge Clarifiers	X	X	X	X	240	320	# of clarifiers	
Squircle Clarifiers	80	80	160	160	240	320	# of clarifiers	
Pumps	100	100	250	500	750	1500	X	250
Rotating Biological Contactor	48	48	80	80	X	X	# of trains	
Trickling Filters	48	48	48	80	128	160	# of TFs	
Sequencing Batch Reactor	48	48	48	80	128	160	# of tanks	
Mechanical Mixers	32	32	32	32	48	64	# of mixers	x2= 64
Aeration Blowers	64	64	64	64	96	128	# of blowers	x4x0.5= 128
Membrane Bioreactor	32	32	32	64	96	128	# of cartridges	x40 x.5= 640

Continued on page 38

CHART 2 (One-Plus Shift) *continued*
MAINTENANCE

	Flow							
Activity	**0.25-0.5 mgd**	**0.5-1.0 mgd**	**1.0-5.0 mgd**	**5.0-10.0 mgd**	**10.0-20.0 mgd**	**>20 mgd**	**Multiply by**	**Total Hours for Plant**
Subsurface Disposal System	32	32	32	32	96	128	# of systems	
Groundwater Discharge	32	32	32	32	48	64	X	
Aerobic Digestion	32	32	32	32	48	64	# of digesters	
Anaerobic Digestion	X	64	64	96	192	320	# of digesters	
Gravity Thickening	32	32	32	32	96	128	# of basins	
Gravity Belt Thickening	48	48	48	80	128	160	# of belts	
Belt Press	48	48	48	80	128	160	# of presses	
Mechanical Dewatering (Plate Frame and Centrifuges)	48	48	48	80	128	160	# of units	x2 = 96
Dissolved Air Floatation	X	32	32	32	96	128	# of units	
Chlorination (gas)	32	32	32	64	96	128	X	
Chlorination (liq.)	64	64	64	96	144	192	X	
Dechlorination (gas)	32	32	32	64	96	128	X	
Dechlorination (liq.)	64	64	64	96	144	192	X	
Ultraviolet	32	32	32	48	80	96	# of racks	x8 = 256
Biofilter	160	160	160	160	160	160	# of units	
Activated Carbon HDWKS	160	160	160	240	240	320	# of units	x2 = 320
Wet Scrubbers	X	X	X	48	80	96	# of units	
Microscreens	32	32	32	48	80	96	# of screens	
Pure Oxygen	X	X	X	64	96	128	# of units	
Final Sand Filters	64	64	64	64	96	192	# of units	
Probes/ Instrumentation/ Calibration	32	32	32	32	32	32	# of probes in-line	x2 = 64
TOTAL								
								2498

CHART 3 (One-Plus Shift)
LABORATORY OPERATIONS

		How often are tests run?			
Test Required by Permit	Testing Time (hrs.)	Tested Weekly X 52	Tested Monthly X 12	Tested Quarterly X 4	Annual Hours
Acidity	0.75				
Alkalinity, total	0.75				
Biochemical Oxygen Demand (BOD)	2.5	x2 260			
Chemical Oxygen Demand (COD)	2.5				
Chloride	0.5				
Chlorine, Total Residual	0.25				
Coliform, Total, Fecal, E.Coli	1.0	x2 104			
Dissolved Oxygen (DO) Oct 15 - May 15 Ash. Ck 2/Mo	0.25	x2 15	x2 6		
Hydrogen Ion (pH)	0.25	x3 39			
Metals	3.0				
Toxicity	2.0				
Ammonia	2.0	x2 208			
Total Nitrogen May 1 - Nov. 30	2.0	x2 120			
Oil and Grease	3.0				
Total and Dissolved Phosphorus Dec. 1 - Apr 30	2.0		10		
Solids, Total, Dissolved, and Suspended	3.0	x2 312			
Specific Conductance	0.25				
Sulfate	1.0				
Surfactants	1.0				
Temperature	0.25	x7 91			
Total Organic Carbon (TOC)	0.25				
Turbidity	0.25				
Bacteriological Enterococci	1.0				
Lab QA/QC Program	1.0				
Process Control Testing	3.0	156			
Sampling for Contracted Lab Services	0.25				
Sampling for Monitoring Groundwater Wells	0.5				
TOTAL		1321			

- Sampling time is built into testing time estimates.

CHART 4 (One-Plus Shift)
BIOSOLIDS/SLUDGE HANDLING

	Flow					
Process	0.25-0.5 mgd	0.5-1.0 mgd	1.0-5.0 mgd	5.0-10.0 mgd	10.0-20.0 mgd	>20 mgd
Belt Press	320	960	1920	2560	2560	2560/shift
Plate & Frame Press	320	480	960	2560	2560	2560
Gravity Thickening	80	80	160	160	320	320
Gravity Belt Thickening	80	80	160	160	320	640
Rotary Press	80	80	160	160	320	640
Dissolved Air Floatation	X	160	160	320	320	320
Alkaline Stabilization	80	80	80	80	80	80
Aerobic Digestion	160	160	160	320	480	640
Anaerobic Digestion	80	80	160	480	800	1280
Centrifuges	320	320	960	2560	2560	2560
Composting	320	640-960	1280	2560	2560	2560/shift
Incineration	X	X	X	X	7680	7680
Air Drying – Sand Beds	160	160	X	X	X	X
Land Application	80	160	160	X	X	X
Transported Off-Site for Disposal	80	320	1280	2560	2560	2560
Static Dewatering	320	320	X	X	X	X
TOTAL			2240			

CHART 5 (One-Plus Shift)
YARDWORK

	Size of Plant			
Work Done	Small	Average	Large	Total Hours for Plant
Janitorial/Custodial Staff	100	200	400	200
Snow Removal	60	120	400	—
Mowing	100	120	400	120
Vehicle Maintenance (per vehicle)	25	25	25	100
Facility Painting	60	80	160	80
Rust Removal	60	80	160	80
TOTAL				580

Recycle/Reuse Analysis

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		ASHLAND LAND APPLICATION - Pasture, 42" application							
Month	Influent WW, gpd	WW, ac-ft	Stg. Pond Gain (Loss), ac-ft	Applied WW, ac-ft	Discharged WW, ac-ft	Net Storage Change, ac-ft	Cumulative Storage, ac-ft		
Oct	2,590,052	246.41	0.91	62.89	0	184.43	222.43		
Nov	2,637,659	242.84	5.52	0	55.70	192.66	415.09	*0.605*	mgd max dischg
Dec	2,937,578	279.47	7.13	0	279.47	7.13	**422.22**		
Jan	3,219,050	306.25	5.55	0	521.52	-209.73	212.49	*dischg stored + influent*	
Feb	2,991,433	257.05	2.79	0	472.33	-212.49	0.00	*dischg stored + influent*	
Mar	2,796,290	266.03	0.80	0	133.67	133.16	133.16	*1.405*	mgd max dischg
Apr	2,772,352	255.24	-2.53	117.53	107.72	27.46	160.62	*1.17*	mgd max dischg
May	2,804,849	266.84	-5.46	194.85	0	66.53	227.15		
Jun	2,680,310	246.77	-9.17	265.99	0	-28.39	198.77		
Jul	2,632,392	250.43	-13.42	371.14	0	-134.13	64.64		
Aug	2,633,574	250.55	-10.54	304.65	0	-64.64	0.00		
Sep	2,557,777	235.49	-5.21	192.79	0	37.49	37.48	*excess to storage*	
		3103.35	*-23.64*	*1509.83*	*1570.40*		*422.22*	*Max storage*	
AADF, mgd	*2.77*						138	MG	
		size	30	433	ac				
			ac	41.8	in				

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HYPORHEIC EVALUATION

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<u>Ashland WWTP Hyporheic Cooling Option Evaluation</u>

Ashland, OR

Prepared for Keller Associates

Nick Guho

November 12, 2010

Background

The 2007 Bear Creek TMDL limits the temperature to a maximum of 13°C for October 15 to May 15 and 18°C for May 16 to October 14. Cumulative anthropogenic impacts are allowed to exceed these limits by at most 0.3°C (termed the Human Use Allowance, HUA), with specific sources on the creek receiving portions of that total. Specifically, the Ashland wastewater treatment plant (WWTP) is permitted a maximum HUA of 0.1°C above the numerically calculated biological limit. Currently, the Ashland WWTP exceeds this allotment in the summer and fall.

The purpose of this memorandum is to:

1. Review the available literature on the utilization of hyporheic exchange for temperature remediation.
2. Estimate the potential viability of this option
3. Present a recommendation for further investigation of this option.

Literature Review

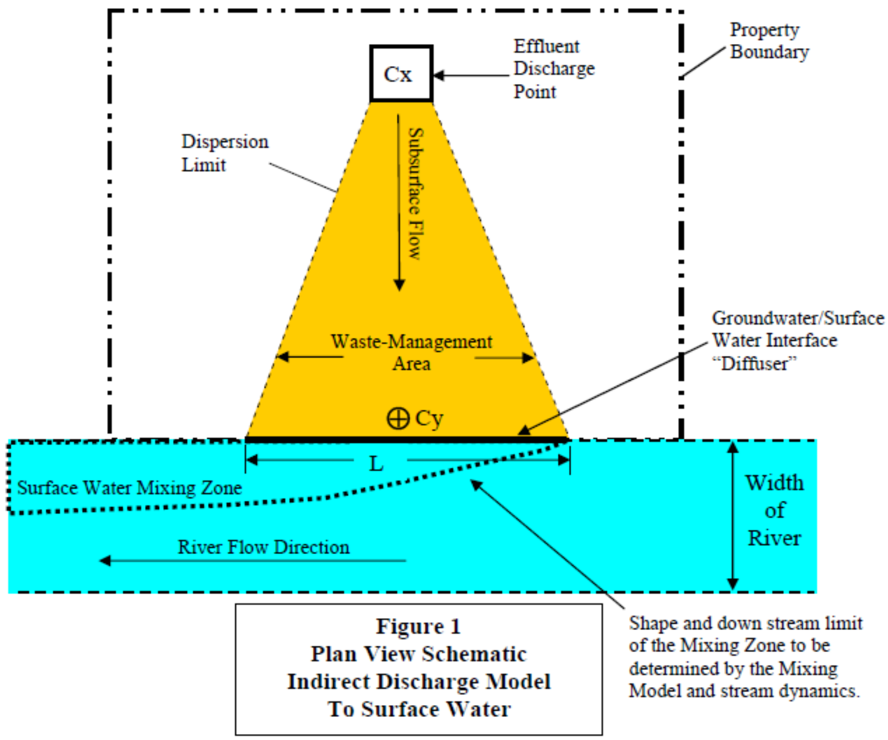
The hyporheic zone is the region where shallow ground water interacts with the surface water in a stream or river. The understanding of the behavior of this zone and its utilization for temperature reduction are relatively new and active areas of research; as such, literature on the topic is limited. Despite this novelty, the Oregon DEQ (2007) recognizes the indirect discharge of effluent to surface water via hyporheic exchange as a viable temperature control alternative. Therefore, the design and implementation of this process is based on site-specific calibration of temperature flow models.

Depending on numerous conditions (e.g., channel geometry, soil characteristics, diurnal variations, season, etc.), the hyporheic exchange can act as a buffer for river temperatures and/or as a mechanism to cool/warm river temperatures (Arrigoni et al., 2008). This behavior has been evaluated via modeling as a means to reduce the anthropogenic impact on receiving waters; typically through the modification of river geometry to increase mixing, thereby reducing temperature (Burkholder et al., 2008; Seedang et al., 2008).

Considerations and Regulations

Implementing this process can take several forms, which can be divided into either a direct or indirect injection into the water table. Each application must satisfy the following requirements (Oregon DEQ 2007):

1. Definition and maintenance of a Waste-Management Area (WMA), which defines the confines of the infiltrate influence (Figure 1). The WMA must be situated so that the infiltrate remains within the confines of the property and does not affect existing wells. Also, it needs to be shown that the infiltration will not contaminate the groundwater/aquifer.
2. Site/soil suitability, primarily that the hydrology of the site would permit the injection of the proposed quantity of effluent.
3. Public acceptance of the practice.



Notes:

1. At L, provide groundwater volume, velocity and concentration of key water quality parameters for mixing zone model. These typically will be estimates based on calculations or modeling.
2. L functions as a surface water diffuser.
3. Effluent discharge in the Waste-Management Area can be in groundwater, hyporheic water, or both.
4. Limits of Waste-Management Area are determined by horizontal dispersion of the effluent in groundwater, hyporheic water or both.

Symbols:

⊕	Verification monitoring well location (i.e., *detection monitoring* well)
Cx	Concentration of key wastewater effluent parameters.
Cy	Concentration of key effluent parameters in groundwater or hyporheic water at transition to surface water for mixing zone analysis, e.g. concentration for diffuser at L.
L	Length of Waste-Management Area at receiving water body.

Figure 1 - Waste-Management Area (Oregon DEQ 2007)

While the effluent temperature could conceivably be reduced through dispersion and conduction with ground water, this relationship cannot be adequately described without sufficient site data. A rough, preliminary design can be completed using semi-conservative values, which can be used as a basis to formulate site parameter investigations.

Proposed Implementation& Imperatice Property Evaluation

The City of Ashland owns more than 800 acres (referred to as the Imperatice Property) north of the WWTP which have been suggested as a possible location for the installation of an indirect effluent injection into the hyporheic zone of Bear Creek. While the location of the Imperatice Property is desirable for this option, several issues, as described below, would have to be addressed for this implementation.

As noted above, containment of the WMA within property owned or controlled by a local government or special district is typically required by the DEQ; also, it must be shown that the WMA will not impact existing and potential future water supply wells (Oregon DEQ 2007). The Imperatice property ends approximately 600-1600 ft from Bear Creek, with the interstate, Eagle Mill Rd, and the Lower East Side Lateral irrigation ditch separating the property line from Bear Creek. Additionally, there would appear to be a residence located between Eagle Mill Rd and the irrigation ditch, indicating a possible change in zoning for the area. To satisfy the DEQ requirements, ownership/control of the property between the Imperatice property and Bear Creek would need to be addressed. The irrigation ditch between Bear Creek and the Imperatice Property may also pose a challenge as it may intercept the infiltrated effluent, depending on the ground water behavior.

With regard to the suitability of Imperatice property's geology, the NRCS soil type is defined as Carney Clay, which has a permeability rate of 0.06 in hr^{-1}which is considered extremely slow (Carollo Engineers 1997). Soil depth over rock varies across the site, with estimates ranging from 3 to over 9 ft (Carollo Engineers 1997). Assuming the infiltration rate was equal to the permeability rate (i.e., all of the effluent applied infiltrated), each MGD discharged would require 26 acres for the infiltration basin. Dimensions of the basin, and hence an estimate of the site footprint, depend on the capacity of the soil to convey the flow to the river (i.e., assuming all of the infiltrate were conveyed to the river, knowing site parameters like hyporheic water location and soil depth would allow an estimate of the maximum flow per foot parallel to the river). To approximate this capacity, the Darcy equation was employed, with a headloss of 15ft (assumed that the loss from the conductivity would result in a hydraulic grade-line from the surface to the ground water level, as indicated in historical records from a well near the site) (Oregon WRD 1996) and the aforementioned permeability rate of 0.06 in hr^{-1}. The maximum flow rate per foot is the product of the head and the permeability, which would be 13.5 gal day^{-1} ft^{-1}. Assuming this maximum conductivity, the minimum length of the infiltration basin (parallel to the creek) would be over 14 miles for each MGD.

Due to the low permeabilityof the Imperatice Property's soil, potentially shallow soil depth, significant slope, and incomplete WMA control, the site would likely not be well suited for effluent infiltration and hyporheic exchange. The option could be implemented at other sites in close proximity, assuming property acquisition was a possibility. Soil maps from the National Wetland Inventory indicate substantial soil type differences in the valley, namely the presence of sandy characteristics in some areas. Sandy soils typically have a higher permeability rate, with typical values ranging from 0.13 to 12.96 in hr^{-1} for Clayey Sand (Coduto 1999). Over this range of values, the foot print for each MGD of effluent would be between 780 and 8 acres (assuming 15 ft of head and 300 m spacing between the river and the infiltration basin). These areas only include that needed for the WMA; due to plot dimensions, considerable additional property would likely be purchased as well.

If this option were pursued, the following phased approach should be completed in stages, obtaining more and more detailed estimates of the site characteristics, while minimizing potentially unwarranted expenditures. Initial sample planning should be based on the aforementioned design, first assessing if the City owns property that would be isolated enough to satisfy the groundwater protection requirements (Oregon DEQ 2007) while providing an adequate footprint for the above design. Behind each stage is a progressively more accurate model of the ground/hyporheic water flow and the river mixing, which determines the viability of the design and directs subsequent investigations. I would recommend the following approach, each phase of which could be conducted in stages:

Phase 1 – Initial Site Assessment and Monitoring Well Installations

A preliminary assessment of the sites suitability for this approach can be completed by installing ground water monitoring wells throughout the site, as directed by the preliminary design. Placing the wells near the creek's edge as well as toward the site's boundaries will allow the wells to be used in the future for compliance testing, assuming the site is suitable. Recording soil properties and water levels in the drilling processes of the wells should provide a rough approximation of the site's geology and ground/hyporheic water state. These parameters could be used to estimate the site's infiltration capacity and subsurface conductivity. With these estimates, a rough design of the infiltration basins could be completed, balancing the need to minimize the waste-management area while maximizing the distance between the infiltration basin and the creek.

Phase 2 – Single and Multiple Well Aquifer Tests, Mixing Model Precursors

Assuming that the preliminary design completed using the estimated site parameters was viable, a more refined estimate of the site hydrology should be completed. To accomplish this task, wells should be drilled according to the predicted design, with locations in the infiltration area(s). Single well aquifer tests should then be performed to obtain actual conductivity information for the site, using the previously installed monitoring wells to observe the site's response. Using the results from these tests, the actual distribution of site conductivities can be more accurately estimated. These values can then be used to refine the previously developed model to reassess the site's viability. Tracer studies could also be used to determine ground water flow and dispersion.

The Oregon DEQ requires a mixing model analysis to be performed to determine the impact of the hyporheic exchange on the creek temperature profile, to estimate the mixing effects. To approximate these effects, the creek profile should be approximated over the range of available property, determining cross section profiles, depth, and velocity. Also, an estimate of the hyporheic mixing capacity would also be of help. As indicated by the research of Lancaster et al. 2005, if properly distanced from the creek, the injected heat should not substantially impact the creek temperature.

Phase 3 –Long Term Monitoring

Provided that the refined design was still viable, the behavior of the groundwater should be observed to determine seasonal variation and response to rainfall and creek flows. These observations would provide additional insight into the actual response of the site to real infiltration, allowing further calibration of the model and verification of the groundwater flow direction and velocity.

Phase 4 – Scaled Infiltration Test

Using a full scale design based on the estimated infiltration capacity and ground water response as a guide, a large scale infiltration test would provide a final model verification prior to full construction.

Using this approach, the capital investment required for an accurate model (which is expected for permitting (Oregon DEQ 2007)) could be expended in stages, each of which would allow for the overall evaluation of the process, to determine if further investment is warranted.

References

Arrigoni, A.S., Poole, G.C., Mertes, L.A.K., O'Daniel, S.J., Woessner, W.W., Thomas, S.A. (2008). *Buffered, lagged, or cooled? Disentangling hyporheic influences on temperature cycles in stream channels*. Water Resources Research, 44.

Burkholder, B.K., Grant, G.E., Haggerty, R., Khangaonkar, T., Wampler, P.J. (2008).*Influence of hyporheic flow and geomorphology on temperature of a large, gravel-bed river, Clackamas River, Oregon, USA*. Hydrological Processes. 22, 941-953.

Carollo Engineers. (1997). Project B – Offsite Facilities Predesign Report. Final Report to the City of Ashland.Carollo Engineers, Portland

Carollo Engineers. (2009). *Update to WWTP Facilities Plan and Permitting Evaluation: Temperature Management and Water Recylcing*. Final Report to City of Ashland.Carollo Engineers, Portland

Coduto, D.P. (1999). *Geotechnical Engineering: Principles and Practices*. Prentice Hall, Upper Saddle River

Lancaster, S., Haggerty, R., Gregory, S., Farthing, K.T., and Biorn-Hansen, S., (2005). *Investigation of the Temperature Impact of Hyporheic Flow: Using Groundwater and Heat Flow Modeling and GIS Analyses to Evaluate Temperature Mitigation Strategies on the Willamette River, Oregon*. Final Report to Oregon Dept. Environmental Quality.Oregon State University, Corvalis.

Oregon DEQ.(2007). *Disposal of Municipal Wastewater Treatment Plant Effluent by Indirect Discharge to Surface Water via Groundwater or Hyporheic Water*.Internal Management Directive.State of Oregon Department of Environmental Quality.Portland.

Oregon WRD.(1996). *Hydrograph for State Well JACK 20246*.http://www1.wrd.state.or.us/groundwater/obswells/htmls/well_JACK020246.html. Salem, OR. October 15, 2010.

Seedang, S., Fernald, A.G., Adams, R.M., Landers, D.H., (2008).*Economic Analysis of Water Temperature Reduction Practices in a Large River Floodplain: an Exploratory Study of the Willamette River, Oregon*. River Research and Applications. 24, 941-959.

CARBON FOOTPRINT EVALUATION

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Ashland WWTP Carbon Footprint Estimation Report

Ashland, OR

Prepared for Keller Associates

Nick Guho

January 31, 2011

Introduction

The anthropogenic generation of greenhouse gasses (GHGs) has recently become an issue as environmental protection becomes more and more contentious. Direct GHGs, including carbon dioxide (CO_2), methane(CH_4), and nitrous oxide (N_2O), while naturally occurring,have dramatically increased in the atmosphere since the industrial revolution[1].

In the most recent US Green House Gas Inventory Report, the US Environmental Protection Agency estimated that in the year 2008 wastewater treatment generated approximately 29.2 million metric tons carbon dioxide equivalents (CO_2Eq) or 0.42%of the total anthropogenic GHG emissions for the US[1]. Following concerns over environmental quality and to reflect federal reporting requirements, the Oregon DEQ recently instituted mandatory GHG emission reporting for facilities with 2,500 metric tons CO_2Eq or more annual emissions; wastewater treatment facilities are temporarily exempt from reporting, pending the adoption of a quantification protocol[2]. To prepare for future reporting requirements, the City of Ashland has requested a carbon footprint estimation (i.e., estimation of emitted GHGs)to be completed for the city's wastewater treatment plant (the facility).

Fundamental to the quantification of GHG emissions for a given process is determining the boundary within which emissions are attributed to the operation in question. While there is currently no uniform protocol for municipal wastewater treatment plants (WWTPs), several approaches described in literature as well as semi-related processes were assessed and a final approach to consider on-site, upstream, and downstream emissions. Under this boundary, operating conditions at the facility from 2008 to 2010 were used to estimate the annual GHG emissions to be 2,690 metric tons CO_2Eq / yr. While this value exceeds the 2,500 metric ton CO_2Eq / yr reporting threshold, it includes numerous sources that would likely be attributed to other entities under the proposed reporting requirements. Assuming only on-site, non-biogenic emissions, the estimate would be much less at approximately 75 metric tons CO_2Eq / yr.

The purpose of this memorandum is to:

1. Provide a brief background for quantifying GHG emissions and description of the facility.
2. Quantify and discuss individual GHG emission sources associated with the facility according to the on-site LCA/GHG estimation procedure.

3. Present the compiled individual emissions (based on their likely inclusion in a GHG estimation protocol) and discuss as relevant to future reporting requirements.

GHGQuantification: Methods and Wastewater Treatment

While numerous gasses are known to contribute to the greenhouse effect, only carbon dioxide (CO_2), methane (CH_4), and nitrous oxide (N_2O) are naturally occurring. The other contributing gases are artificially produced fluorinated compounds including hydrofluorocarbons, fluorocarbons, and sulfur hexafluoride. GHGs directly emitted by municipal WWTPs will be limited to those that occur naturally. Contributions to the greenhouse effect vary between compounds; as CO_2 is the most common, the impact of other gasses are normalized to its effect and expressed as CO_2 equivalents (CO_2Eq). According to the EPA and IPCC, the effect of CH_4 and N_2O are 21 and 310 kg CO_2Eq / kg, respectively[1, 3].

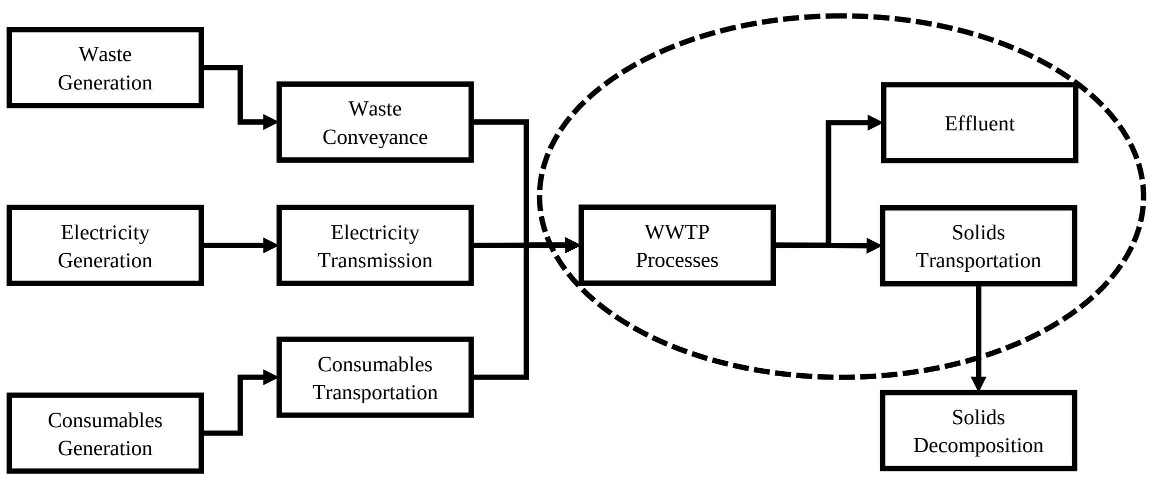

Figure 1 –Wastewater Treatment Plant GHG Emission Process Considerations. The circled emissions indicate those likely to be included in reporting requirements.

Quantifying GHG emissions for a given facility is predicated on the determination of the boundaries within which emissions will be quantified and attributed to the operation in question. At the moment, there are several competing practices with regard to the placement of these boundaries, with the differences primarily attributed to differences in the goals of each given study or assessment. The first approach is facility specific and accounts for environmental impacts both upstream and downstream in the supply chain in addition to the onsite impacts[4-6]. This would be characterized by including all of the impacts from all of the processes shown in Figure 1. The goal of this holistic approach is to quantify what effects different processes have and allow for a metric to assess future improvements. Life cycle assessments (LCAs) are the primary approach for this quantification, with the U.S. EPA’s Tool for the Reduction and Assessment of Chemical and Other Environmental Impacts (TRACI) being the approached used herein [3]. The second type is an aggregated assessment of the impacts of all facilities in a geographic area. In this approach, impacts are assigned to the facility that generates them, reducing double counting; only those processes within the dashed ellipse in Figure 1 would be counted. For this report, GHG emissions were quantified first using a broad impact boundary, including the generation of electricity, disposal of solids, etc, while also breaking out contributions for varied boundaries (i.e., only onsite generation for potential use in the Oregon DEQ reporting).

Ashland Wastewater Treatment Plant

The City of Ashland operates a tertiary treatment facility (Figure 2) to effectively remove carbon, nitrogen, and phosphorus from the influent wastewater. Primary treatment consists of a grit basin and a mechanical bar screen. Of note, aside from the potentially included electrical demand, GHG emissions from headworks processes are insignificant[6]. Secondary treatment includes an oxidation ditch with an initial anoxic zone to achieve some level of denitrification, followed by secondary clarification. This stage of the process emits CO_2 and N_2O, with CH_4 emissions considered negligible as microbiallyconducive anaerobic treatment conditions are not imposed at the facility; these emissions are described below in the "Biogenic GHG Emissions" section. Waste solids are stored in a lime stabilization tank prior to dewatering and disposal. Solids GHG emissions are estimated below in "Solids Disposal".

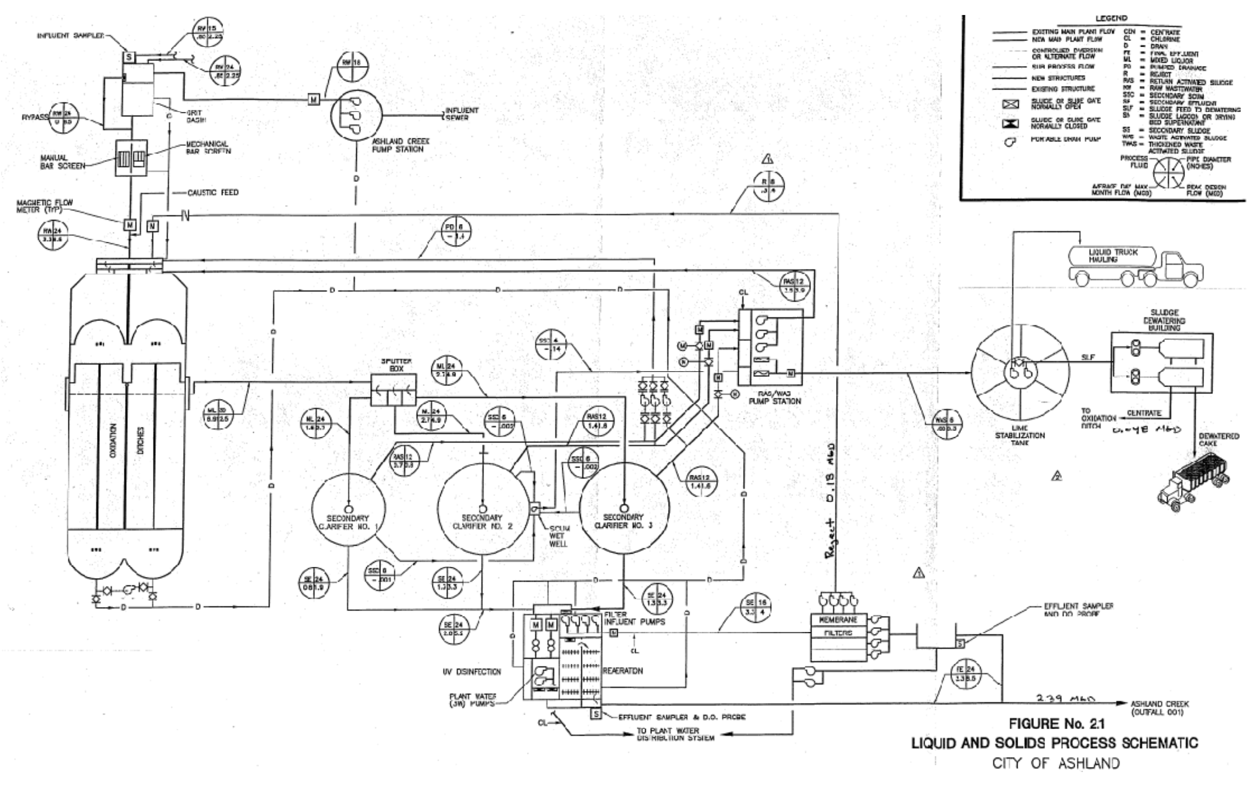

Figure 2 - City of Ashland WWTP Process Schematic

While carbon and nitrogen are removed biologically, phosphorus is removed chemically through the addition of alum followed by membrane filtration. Prior to membrane filtration, the secondary effluent is disinfected via ultraviolet light and reaerated. Chemicals are used throughout the facility and each is addressed in the "Chemical Consumption Section". Electricity for all processes was considered together in "Electricity Consumption".

Biogenic GHG Emissions

The CO_2 generated through the oxidation of wastewater nutrients is often not included in the GHG tabulation, as the process is simply an acceleration of the natural carbon cycle[4, 5]. This GHG contribution, termed biogenic, is simply a cycling of existing CO_2 in the atmosphere and does not constitute a net contribution; therefore, many protocols outlined in literature do not include this contribution. The argument could similarly be made for N_2O, which is consistently included in quantifications yet is a byproduct of denitrification, a fundamental process in the nitrogen cycle. As this extension has not been applied in literature and with the potential to temporally sequester CO_2 and NH_3 through emerging technologies (e.g., PHA storage and controlled struvite precipitation) these contributions were approximated for the facility[6].

To quantify the CO_2 emitted through the oxidation of the influent wastewater, the bioenergetics approach outlined in Rittmann and McCarty [7]and Tchobanoglous et al.[8] was employed to estimate the following theoretical stoichiometric relationship for the three biological redox reactions, wherein $C_{10}H_{19}O_3N$ and $C_5H_7O_2N$ are the typical composite chemical formulas for domestic wastewater and bacterial cells, respectively.

Aerobic Oxidation of COD:

$$1.0{\cdot}C_{10}H_{19}O_3N + 4.46{\cdot}O_2 + 0.61{\cdot}HCO_3^- + 0.61{\cdot}NH_4^+ \rightarrow 1.61{\cdot}C_5H_7O_2N + 5.39{\cdot}H_2O + 2.57{\cdot}CO_2$$

$Y = 0.94$ kg CO_2/ kg COD oxidized with O_2

Anoxic Oxidation of COD:

$$1.0{\cdot}C_{10}H_{19}O_3N + 3.71{\cdot}NO_3^- + 3.71{\cdot}H^+ + 0.57{\cdot}NH_4^+ \rightarrow 1.57{\cdot}C_5H_7O_2N + 7.28{\cdot}H_2O + 2.71{\cdot}CO_2 + 0.57{\cdot}HCO_3^- + 1.86{\cdot}N_2$$

$Y = 0.99$ kg CO_2/ kgCOD oxidized with NO_3^-

Aerobic Oxidation of NH_4^+:41

$$1.0{\cdot}NH_4^+ + 1.72{\cdot}O_2 + 0.16{\cdot}CO_2 + 0.04{\cdot}HCO_3^- \rightarrow 0.04{\cdot}C_5H_7O_2N + 0.92{\cdot}H_2O + 1.92{\cdot}H^+ + 0.96{\cdot}NO_3^-$$

$Y = 0.51$ kg CO_2/ kgNH_4^+-N oxidized

Using the theoretical COD equivalence for the domestic wastewater formula of 1.99 g COD / g solids, the stoichiometric ratios were used to approximate the CO_2 produced from the oxidation of the substrate (Y) above. Computing a simple numeric average of the influent CBOD loading provided in the DMR from 2004 - 2009 yields an average of 498,000 kg CBOD / yr. Conservatively assuming that all influent COD is oxidized by nitrate, the CO_2 evolved from substrate oxidation would be approximately 488 metric tons CO_2Eq/ yr.

As indicated in the ammonia oxidation equation above, the autotrophic reaction fixes CO_2 to supply the carbon for cell growth. In a holistic evaluation of the facility, the carbon fixed in this process would be liberated in the sludge decomposition at the landfill; however, in the facility specific protocol, the liberation of the CO_2 through decomposition would likely be reported by the landfill[4]. While influent ammonia is not recorded in the DMR, if a typical influent concentration of 25 mg NH_3-N / Lis assumed, the average loading would be 74,500 kg NH_3-N / yr. Discounting the ammonia required for heterotroph growth, the CO_2 temporally sequestered would amount to 32.7 metric tons CO_2Eq / yr.

N_2O emissions are a byproduct of denitrification with no direct stoichiometric relationship formally established[5]. Therefore, the estimation of N_2O emissions from a WWTP is based on empirical data, with intentionally nitrifying and denitrifying municipal WWTPs generating 7 g N_2O / capita / yrand those not producing 3.2 g N_2O / capita / yr[1].With the City's population of approximately 21,000, the facility generates approximately 147 kg N_2O / yr, which equates to 45.6 metric tons CO_2Eq/ yr.

<u>ElectricityConsumption</u>

Depending on the goal of the GHG emission quantification protocol (i.e., process specific or inter-industry census) the GHG emissions associated with electricity generation and delivery can either be attributed to the producer of the electricity or the consumer. The protocols presented in literature consider this emission part of the footprint of the facility [4, 5] while the national censuses keep this emission tied to the producer. As both approaches are valid, the facility's share of this emission was quantified.

In the years 2008-2009, the monthly energy consumption of the facility varied between 205 and 450MWh, with an average consumption of 302MWh. Annualconsumption totaled 3,590 and 3,660MWh for 2008 and 2009 respectively, with 2010 through September consuming 2,600MWh. Assuming the average monthly consumption was 302MWh, the average yearly consumption would be approximately 3,620MWh.

Using the EPA's eGRID online utility[9], the normalized GHG emissions for the WECC Northwest eGRIDsubregion were inflated by a grid loss factor of 5.33% and applied to the facility's average annual consumption. The result is an equivalent GHG emission of 1,560 metric tons CO_2Eq annually. The constituents, CO_2, CH_4, and N_2O, contributed 99.4, 0.04, and 0.51% (by CO_2 Eq), respectively.

<u>Chemical Consumption</u>

The sludge stabilization, chemical phosphorus removal, and effluent reuse practices at the facilityrequire chemical addition, which in turn exert GHG emission burdens. At this time, GHG emission estimation for specific chemical production is not widely available. Without these data, the emissions from the production of these compounds cannot be accurately quantified; additionally, these emissions would likely be associated with the chemical manufacturer and not the consumer (the facility) under a reporting format. However, with the facility-centric approach these emissions are relevant and are therefore approximated using available GHG emissions for similar chemical production as surrogates, where possible.

While specific chemical GHG emissions are not yet available, approximate values can be presented for those chemicals of which the precursors are readily available. For example, aluminum sulfate (alum) utilized during the dry weather months to precipitate phosphorus is produced by digesting bauxite (aluminum ore) with sulfuric acid. While the GHG emissions from the conversion of bauxite to alum are not readily available, those associated with bauxite mining, refining to alumina, and transportation are[10]. These values were used as an approximation of the GHG emissions from alum production. An additional consideration, the majority of the world's bauxite is mined from Australia and several South American countries, resulting in additional GHG emissions from transportation[10]. Between 2008 and 2010, the facility used an average of 410 ton alum (diluted to 48.5% (w/w)) / yr, which is equivalent to92.6 metric tons $Al_2(SO_4)_3$ / yr which corresponds to approximately 78,200 kg CO_2Eq / yr.

Similarly, the production of sodium hypochlorite (NaClO), which is used at the facility to chlorinate the effluent recycled for onsite use (i.e., irrigation, wash down, process maintenance, etc.), is most often produced by electrolysis of a brine (NaCl) solution[10]. While GHG data for NaClO production is not explicitly available, GHG data is available for NaCl purification, which is accomplished through a similar process. Between 2008 and 2010, the facility used an average of 2530 gal NaClO/ yr. Assuming aNaClO concentration of 15%, the resulting mass is 1440 kg NaClO / yr. Adjusting the weight basis (i.e., 1.270 kg NaClO/ kgNaCl), the GHG emissions from the production of NaClO can be approximated as that of 1130 kg NaCl / yr, which emits 0.121 kg CO_2Eq / kg NaCl (assuming average U.S. energy emissions)[10]. The production of NaClO would therefore produce approximately 137 kg CO_2Eq / yr.

Bioxide®, which is added to reduce the odors generated from the storage of the wasted sludge, is 60 % (w/w)ammonium calcium nitrate ($5{\cdot}Ca(NO_3)_2{\cdot}NH_4NO_3{\cdot}10H_2O$)[11], which dissociates to nitrate in solution to provide an electron acceptor preferred over sulfate, thereby reducing hydrogen sulfide emissions. The GHG emissions from ammonium calcium nitrate production have been approximated as those associated with fertilizer production, which is conventionally based on natural gas[10]. For ease of conversion, it was assumed that the fertilizer was pure ammonium nitrate, resulting in a mass conversion, on an N basis,of 1.025 kg $5{\cdot}Ca(NO_3)_2{\cdot}NH_4NO_3{\cdot}10H_2O$ / kg NH_4NO_3. Between 2008 and 2010, the facility consumed approximately 14,000 gal of Bioxide®, which is approximately 23,800 kg $5{\cdot}Ca(NO_3)_2{\cdot}NH_4NO_3{\cdot}10H_2O$ / yr. The resulting GHG emissions for Bioxide® production at 0.584 kg CO_2Eq / kg fertilizer are approximately 13,900 kg CO_2Eq / yr[10].

A polymer is added to the sludge prior to centrifuging to improve solids separation. Typical polymers used for sludge conditioning are proprietary anionic compounds. As a result, no surrogate chemical compound could be found and without which GHG emissions could not be approximated.

For the three chemicals approximated above, it is important to note the scope of the estimated emissions. As the specific production locations and distribution paths are not known, the estimates include GHG emissions for the production of the chemical surrogates only. More exact GHG emission estimates would require data from the actual chemical production process and transportation route from production, through distribution, and to the facility.

<u>Solids Disposal</u>
Currently, solids are disposed of daily by trucking to the Dry Creek Landfill outside White City, OR[12]. GHG emissions occur through the transportation and the decomposition of these solids at the landfill itself. Solids degradation in the landfill environment generates both CO_2 and CH_4 in approximately a 1:1 weight ratio[1]. Recently, the Dry Creek Landfill has installed a biogas capture and energy generation system, to generate electricity from the methane produced at the landfill [13]. The new facility operates two 20 cylinder engines that drive two 1.6 MWh generators[13].

As above, the inclusion of each of these processes depends on the scope of the GHG protocol. If a holistic approach, centered on the facility is adopted, then inclusion of the GHG emitted from the transport, decomposition, and subsequent conversion to electricity would be included. However, if the aggregated approach evaluating several facilities independently is adopted, then the GHGs emitted from the transportation of the solids to the landfill would likely be the only solids disposal emission attributed to the facility; the decomposition and combustion would be under the filing of the receiving landfill. As landfills are required to report GHG emissions under the current DEQ guidelines[2], these emissionswill likely not be included in future reporting.

The GHG generated from the transportation of the solids were estimated assuming daily trips [12], a round trip distance of 54.4 miles[14], an average heavy duty vehicle fuel efficiency of 6 mile / gallon[15], and a fuel source of diesel. Using these assumptions and the vehicle emissions recommended in [16, 17], the GHG production for solids transportation would be approximately 29.1 metric tons CO_2Eq/ yr.

Annual solids production at the facility from 2008-2010 is very consistent, averaging 3,350 ton / yr. The carbon quantity in the solids, and hence the carbon dioxide and methane that would be generated through decomposition can be approximated through a carbon balance using the stoichiometric relationships derived above. Using this approach, 247 metric tons of biomass are generatedannually. Assuming a 1:1 CO_2 to CH_4 ratio, and 85% biogas capture at the landfill , 72.4 metric tons CH_4/ yr are burned to generate electricity. Using the energy and emission conversion factors, 136 MWh / yrcan be credited using the electricity emissions outlined above [18].

Discussion

As discussed above, GHG emission quantification is dependent on the boundary for the process under consideration. There are two main categories for GHG protocols, the holistic, facility-specific approach and the inter-industry census. The first represents a comprehensive quantification of the GHG emissions that can be attributed to the facility both on and offsite - up and down stream. This approach is valuable when using GHG emissions as a metric to compare the current operation to another (i.e., LCA). The second approach is primarily used to satisfy regulatory requirements for regional emissions, as is the case with the pending DEQ reporting. This report estimated the GHG emissions using the holistic approach, as the census protocol's results are inclusive; both are summarized below.

Table 1 - Estimated GHG Emissions for the City of Ashland, OR WWTP

Total Estimated GHG Emissions Ashland OR[1]

Process	QTY	Unit	kg CO_2 / yr	kg N_2O /yr	kg CH4 / yr	1000 kg CO_2 Eq / yr	% Total
Biological Treatment							
BOD Oxidation[2]	497,611	kg BOD / yr	467,754	-	-	468	17%
Autotrophic CO2 Sequestration	64,114	kg NH_3-N / yr	(32,730)	-	-	-32.7	-1%
Denitrification N_2O	7	g / capita / yr	-	147.0	-	45.6	2%
Electricity Consumption	3,600	MWh / yr	#######	25.6	32.9	1561	58%
Chemicals Consumed							
Alum Production[3]	92,562	kg Alum/yr	74,062	13.3	-	78.2	3%
Sodium Hypochlorie Production[4]	2,530	gal NaClO / yr	137	-	-	0.14	0%
Bioxide®[5]	7,174	;al Bioxide® / y	13,900	-	-	13.9	1%
Polymer[6]	4,600	gal Polymer / yı	-	-	-	0.00	0%
Solids Handling							
Solids Transportation	19,856	miles / yr	29,082	0.10	0.12	29.1	1%
Solids Decomposition	3,355	ton / yr	240,186	-	13,133	516	19%
Biogas Capture and Energy	74,420	kg CH_4 /yr	13,595	(1.25)	(0.97)	13.3	0%
		Total:	#######	185	13,165	2692	

Notes:
1. Highlighted items are those likely to be included in the scope to be reported to the DEQ.
2. Calculated conservatively assuming anoxic oxidation.
3. GHG emissions approximated from aluminum production.
4. GHG emissions approximated from sodium chloride purification.
5. GHG emissions approximated from fertilizer production.
6. GHG emissions could not be approximated without a similar compound reference.

As shown, the total annual GHG emissions for the WWTP isapproximately 2690 metric tons CO_2Eq /yr. This includes the CO_2 and N_2O emitted from the oxidation of the wastewater, the GHG emissions from the generation of the facilities electricity, the GHG emissions from the transportation of the solids to the landfill, and the GHG emissions from the biomass degradation. Also included are estimates of the GHG emissions from the alum, sodium hypochlorite, and Bioxide® used at the facility. The chemical emissions estimates are based on surrogate chemicals that follow a similar production path (i.e., alumina for alum, sodium chloride for sodium hypochlorite, and fertilizer for Bioxide®). The alum estimate includes transportation emissions as the primary source of bauxite is Australia; the transportation emissions for the other two chemicals were not approximated. With regard to the proposed reporting requirements of the Oregon DEQ, the emissions from the facility that would likely be included in such a report (i.e., the denitrification-derived N_2O and the solids transportation emissions) of 74.7 metric tons CO_2Eq / yr are well below the threshold 2,500 metric tons CO_2Eq/ yr. That being said, the final protocol may include some of the other emissions listed above, in which case, reporting may be required.

References

1. U.S. EPA, *Inventory of U.S. Greenhouse Gas Emissions and Sinks: 1990-2008*. 2010: Washington, DC.
2. Oregon DEQ. *Greenhouse Gas Reporting*. 2010 [cited 2011 01/21/2011]; Available from: http://www.deq.state.or.us/aq/climate/reporting.htm.
3. U.S. EPA. *Tools for the Reduction and Assessment of Chemical and Other Environmental Impacts (TRACI)*. 2010 12/29/2010 [cited 2010 10/14/2011]; Available from: http://www.epa.gov/nrmrl/std/sab/traci/.
4. Foley, J., de Haas, D., Hartley, K., Lant, P., *Comparative life cycle inventories of alternative wastewater treatment systems.* Water Research, 2010. **44**: p. 1654-1666.
5. Sahely, H.R., et al., *Comparison of on-site and upstream greenhouse gas emissions from Canadian municipal wastewater treatment facilities.* Journal of Environmental Engineering and Science, 2006. **5**(5): p. 405-415.
6. Monteith, H.D., et al., *A rational procedure for estimation of greenhouse-gas emissions from municipal wastewater treatment plants.* Water Environment Research, 2005. **77**(4): p. 390-403.
7. Rittmann, B.E. and P.L. McCarty, *Environmental Biotechnology: Principles and Applications*. McGraw-Hill Series in Water Resources and Environmental Engineering, ed. G. Tchobanoglous. 2001, Boston: McGraw Hill.
8. Tchobanoglous, G., F.L. Burton, and H.D. Stensel, *Wastewater Engineering: Treatment and Reuse*. 4 ed. 2003, Boston: McGraw Hill.
9. U.S. EPA. *Emissions & Generation Resource Integrated Database (eGrid)*. 2011 01/20/2011 [cited 2010 12/09/10]; Available from: http://cfpub.epa.gov/egridweb/index.cfm.
10. National Renewable Energy Laboratory, *U.S. Life-Cycle Inventory Database*. 2008, Alliance for Sustainable Energy, LLC.
11. Siemens (2010) *Material Safety Data Sheet - Bioxide*.
12. City of Ashland. *City of Ashland, Oregon - Public Works - Wastewater - Wastewater Treatment Plant*. 2010 01/07/2011 [cited 2010 12/09/10]; Available from: http://www.ashland.or.us/Page.asp?NavID=8494.
13. Rouge Disposal & Recycling Inc. *About The Dry Creek Landfill*. 2010 [cited 2010 12/18/2011]; Available from: http://www.roguedisposal.com/about_contact.htm.
14. Google Inc. *Google maps*. 2011 [cited 2010 12/18/2010]; Available from: http://maps.google.com/.
15. U.S. EPA, *Greenhouse Gas Emissions from the U.S. Transportation Sector 1990-2003*. 2006: Washington, D.C.
16. U.S. EPA, *Update of Methane and Nitrous Oxide Emission Factors for On-Highway Vehicles*. 2004: Washington, D.C.
17. U.S. EPA. *Emission Facts: Average Carbon Dioxide Emissions Resulting from Gasoline and Diesel Fuel*. 2010 11/14/2010 [cited 2010 12/18/2011]; Available from: http://www.epa.gov/otaq/climate/420f05001.htm.
18. U.S. DOE. *U.S. Energy Information Administration - Independent Statistcis and Analysis*. 2011 01/15/2011 [cited 2011 01/20/2011]; Available from: http://www.eia.doe.gov/ask/environment_faqs.asp.

APPENDIX F
Cost Estimates

- Collection System Costs
- Treatment System Costs
  - WWTP Improvements
  - Shading/Outfall Relocation
  - Treatment Alternatives
- Short-Lived Assets
- 6-Year Capital Improvements Plan
- Rogue Valley Sewer Disposal Option

KELLER associates

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Collection System Cost Estimates

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CITY OF ASHLAND
Wastewater Facility Planning Study
Collection System Capital Improvement Plan

ID#	Item	Est. Project Cost		% Allocated to Growth	$ to Growth	
Priority 1 Improvements (2011-2013)						
1A	18" and 24" Parallel Trunkline Along Creek	$	1,248,000	70%	$	873,600
1B	15" Main Along Mountain Ave	$	118,000	25%	$	29,500
1C	Oak St. 24" Trunkline	$	40,000	15%	$	6,000
1D	A St 15" Main	$	522,000	10%	$	52,200
1E	12" Main Along Railroad	$	275,000	57%	$	156,750
1F	12" Siskiyou Blvd Main	$	73,000	46%	$	33,580
1G	Miscellaneous Lift Station Upgrades	$	335,000	10%	$	33,500
1H	Portable Flow Meters	$	60,000	0%	$	-
1I	Storm Water Inflow Study	$	60,000	0%	$	-
	Total Priority 1 Improvements	**$**	**2,731,000**	**43%**	**$**	**1,185,130**
Priority 2 Improvements (by 2020)						
2A	12" Pipeline on Nevada Street	$	217,000	38%	$	82,460
2B	8" Slope Correction on Walker Ave.	$	168,000	28%	$	47,040
2C	12" Wightman St.	$	172,000	66%	$	113,520
2D	Miscellaneous Lift Station Upgrades	$	739,000	10%	$	73,900
	Total Priority 2 Improvements	**$**	**1,296,000**	**24%**	**$**	**316,920**
Priority 3 IExpansion (by UGB Build-out)						
3A	Rogue Valley Hwy 99 Collection, Lift Station, & Pressure Main	$	2,545,000	100%	$	2,545,000
3B	Upsize Costs for Future Expansion	$	18,000	100%	$	18,000
	Total Priority 3 Improvements	**$**	**2,563,000**	**100%**	**$**	**2,563,000**
	TOTAL COLLECTION SYSTEM COSTS (rounded)	**$**	**4,027,000**			

* All costs in 2011 Dollars. Costs include engineering and contingencies.
* * Costs assume open cut construction. Alternative technologies (i.e. pipe bursting) should be explored during pre-design phase.

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City of Ashland, OR
WW Collection CIP Unit Price List

ITEM	UNIT	UNIT PRICE*
PVC Pipe (Gravity)		
8" Pipe - Excavation, Backfill	LF	$50
10" Pipe - Excavation, Backfill	LF	$55
12" Pipe - Excavation, Backfill	LF	$60
15" Pipe - Excavation, Backfill	LF	$65
18" Pipe - Excavation, Backfill	LF	$90
21" Pipe - Excavation, Backfill	LF	$100
24" Pipe - Excavation, Backfill	LF	$115
36" Pipe - Excavation, Backfill	LF	$135
PVC Pipe (Pressure)		
4" Pressure Pipe - Excavation, Backfill	LF	$35
6" Pressure Pipe - Excavation, Backfill	LF	$40
8"Pressure Pipe - Excavation, Backfill	LF	$45
PVC Pipe (Gravity) Upsize Costs		
10" Pipe - Excavation, Backfill (upsize from 8")	LF	$5
12" Pipe - Excavation, Backfill (upsize from 8")	LF	$10
Remove Old Pipe - 8" thru 18"	LF	$5
Connect/Reconnect Pipes at Manholes - 8" thru 21"	EA	$1,500
Connect/Reconnect Pipes at Manholes - 24" thru 36"	EA	$3,000
Manhole 48" - 8" thru 18" pipe	EA	$3,000
Manhole 54" - 21" thru 24" pipe	EA	$3,500
Manhole 60" - 30" thru 36" pipe	EA	$4,000
Reconnect Services	LF	$10
Existing Utility Protection	LF	$4
Traffic Control	LS	varies
Rock Excavation	LF	$35
Bore Short Length (<60feet) - incl casing & carrier pipe	LF	$600
Bore Long Length (>100feet) - incl casing & carrier pipe	LF	$450
Canal/Creek Crossing - incl. casing & carrier pipe	LS	$15,000
Easement	LF	$25
Bypass Piping Setup - 8" thru 24" gravity	EA	$5,000
Bypass Pipe and Pump Operation - 8" thru 24" gravity	LF	$10
1/2 Lane Pavement Repair	LF	$30
Full Lane Pavement Repair	LF	$60
Control Density Backfill	LF	$40
Gravel Road Repair	LF	$7
Miscellaneous Surface Repair	LF	$5
6' Chain Link Security Fencing (add $1000 per gate)	LF	$20
Mobilization - Percent of Item Cost Sum	%	5%
Contingency - % of construction costs	%	30%
Engineering and CMS - % of construction costs	%	18%

* Costs in 2011 Dollars

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Ashland, OR
Wastewater System Collection Improvements

Item	Unit	Unit Price	Estimated Quantity	Item Cost (Rounded)	Total Cost (Rounded)
Priority 1					
1A: 18" and 24" Parallel Trunkline Along Creek					
18" Pipe - Excavation, Backfill	LF	$90	4,160	$374,400	
24" Pipe - Excavation, Backfill	LF	$115	2,370	$272,550	
Manhole 48" - 8" thru 18" pipe	EA	$3,000	14	$42,000	
Manhole 54" - 21" thru 24" pipe	EA	$3,500	8	$28,000	
Connect/Reconnect Pipes at Manholes - 8" thru 21'	EA	$1,500	4	$6,000	
Connect/Reconnect Pipes at Manholes - 24" thru 36'	EA	$3,000	4	$12,000	
Miscellaneous Surface Repair	LF	$5	4,530	$22,650	
Gravel Road Repair	LF	$7	2,000	$14,000	
Traffic Control	LS	$3,000	1	$3,000	
Canal/Creek Crossing - incl. casing & carrier pipe	LS	$15,000	1	$15,000	
Subtotal					*$789,600*
Mobilization - Percent of Item Cost Sum	%	5%		$39,480	
Total Construction Costs					*$829,080*
Contingency - % of construction costs	%	30%		$248,724	
Permitting	LS			$20,000	
Engineering and CMS - % of construction costs	%	18%		$149,234	
Total Project Cost (rounded)					***$1,248,000***
1B: 15" Main Along Mountain Ave					
15" Pipe - Excavation, Backfill	LF	$65	395	$25,675	
Connect/Reconnect Pipes at Manholes - 8" thru 21'	EA	$1,500	2	$3,000	
Bypass Piping Setup - 8" thru 24" gravity	EA	$5,000	1	$5,000	
Bypass Pipe and Pump Operation - 8" thru 24" gravity	LF	$10	395	$3,950	
Existing Utility Protection	LF	$4	395	$1,580	
Reconnect Services	LF	$10	395	$3,950	
1/2 Lane Pavement Repair	LF	$30	395	$11,850	
Control Density Backfill	LF	$40	395	$15,800	
Traffic Control	LS	$5,000	1	$5,000	
Subtotal					*$75,805*
Mobilization - Percent of Item Cost Sum	%	5%		$3,790	
Total Construction Costs					*$79,595*
Contingency - % of construction costs	%	30%		$23,879	
Engineering and CMS - % of construction costs	%	18%		$14,327	
Total Project Cost (rounded)					***$118,000***
1C: Oak St. 24" Trunkline					
24" Pipe - Excavation, Backfill	LF	$115	115	$13,225	
Connect/Reconnect Pipes at Manholes - 24" thru 36'	EA	$3,000	2	$6,000	
Miscellaneous Surface Repair	LF	$5	115	$575	
Traffic Control	LS	$3,000	1	$3,000	
Subtotal					*$22,800*
Mobilization - Percent of Item Cost Sum	%	5%		$1,140	
Total Construction Costs					*$23,940*
Contingency - % of construction costs	%	30%		$7,182	
Engineering and CMS - % of construction costs	%	35%		$8,379	
Total Project Cost (rounded)					***$40,000***
1D: A St 15" Main					
15" Pipe - Excavation, Backfill	LF	$65	2,200	$143,000	
Manhole 48" - 8" thru 18" pipe	EA	$3,000	7	$21,000	
Connect/Reconnect Pipes at Manholes - 8" thru 21'	EA	$1,500	2	$3,000	
Bypass Piping Setup - 8" thru 24" gravity	EA	$5,000	1	$6,000	
Bypass Pipe and Pump Operation - 8" thru 24" gravity	LF	$10	800	$8,000	
Existing Utility Protection	LF	$4	2,200	$8,800	
Reconnect Services	LF	$10	2,200	$22,000	
1/2 Lane Pavement Repair	LF	$30	2,200	$66,000	
Control Density Backfill	LF	$40	395	$15,800	
Traffic Control	LS	$7,000	1	$7,000	
Rock Excavation	LF	$35	1,000	$35,000	
Subtotal					*$335,600*
Mobilization - Percent of Item Cost Sum	%	5%		$16,780	
Total Construction Costs					*$352,380*
Contingency - % of construction costs	%	30%		$105,714	
Engineering and CMS - % of construction costs	%	18%		$63,428	
Total Project Cost (rounded)					***$522,000***

Item	Unit	Unit Price	Estimated Quantity	Item Cost (Rounded)	Total Cost (Rounded)
1E: 12" Main Along Railroad					
12" Pipe - Excavation, Backfill	LF	$60	1,350	$81,000	
Manhole 48" - 8" thru 18" pipe	EA	$3,000	4	$12,000	
Connect/Reconnect Pipes at Manholes - 8" thru 21"	EA	$1,500	2	$3,000	
Bypass Piping Setup - 8" thru 24" gravity	EA	$5,000	1	$5,000	
Bypass Pipe and Pump Operation - 8" thru 24" gravity	LF	$10	800	$8,000	
Reconnect Services	LF	$10	1,350	$13,500	
Existing Utility Protection	LF	$4	1,350	$5,400	
1/2 Lane Pavement Repair	LF	$30	1,350	$40,500	
Traffic Control	LS	$8,000	1	$8,000	
Subtotal					*$176,400*
Mobilization - Percent of Item Cost Sum	%	5%		$8,820	
Total Construction Costs					*$185,220*
Contingency - % of construction costs	%	30%		$55,566	
Engineering and CMS - % of construction costs	%	18%		$33,340	
Total Project Cost (rounded)					***$275,000***
1F: 12" Siskiyou Blvd Main					
12" Pipe - Excavation, Backfill	LF	$60	200	$12,000	
Connect/Reconnect Pipes at Manholes - 8" thru 21"	EA	$1,500	2	$3,000	
Bypass Piping Setup - 8" thru 24" gravity	EA	$5,000	1	$5,000	
Bypass Pipe and Pump Operation - 8" thru 24" gravity	LF	$10	200	$2,000	
Reconnect Services	LF	$10	200	$2,000	
Existing Utility Protection	LF	$4	200	$800	
Full Lane Pavement Repair	LF	$60	200	$12,000	
Control Density Backfill	LF	$40	200	$8,000	
Traffic Control	LS	$2,000	1	$2,000	
					$46,800
Mobilization - Percent of Item Cost Sum	%	5%		$2,340	
Total Construction Costs					*$49,140*
Contingency - % of construction costs	%	30%		$14,742	
Engineering and CMS - % of construction costs	%	18%		$8,845	
Total Project Cost (rounded)					***$73,000***
1G.1: Misc Upgrades - Creek Drive LS Upgrades					
Chopper Pumps	EA	$16,000	2	$32,000	
Three Phase Power	LS	$25,000	1	$25,000	
Subtotal					*$57,000*
1G.2: Misc Upgrades - Abandon Nevada LS & Oak Street Rehabilitation					
Abandon LS and Oak Street Rehab Project (portion of work completed by City)	LS	$95,000	1	$95,000	
Subtotal					*$95,000*
1G: Miscellaneous Upgrades					
1G.1 Creek Drive Lift Station Chopper Pumps and Three Phase Power	LS	$57,000	1	$57,000	
1G.2 Abandon Nevada Lift Station	LS	$95,000	1	$95,000	
1G.3 Add Drain at Windburn Lift Station	LS	$3,500	1	$3,500	
1G.4 Maintenance Management Software & Programming Upgrades	LS	$10,000	1	$10,000	
1G.5 Add SCADA Control System - All Lift Stations	LS	$50,000	1	$50,000	
Subtotal					*$215,500*
Mobilization - Percent of Item Cost Sum	%	5%		$10,775	
Total Construction Costs					*$226,275*
Contingency - % of construction costs	%	30%		$67,883	
Engineering and CMS - % of construction costs	%	18%		$40,730	
Total Project Cost (rounded)					***$335,000***
			Total Priority 1 Cost (rounded)		**$2,611,000**

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Item	Unit	Unit Price	Estimated Quantity	Item Cost (Rounded)	Total Cost (Rounded)
Priority 2					
2A: 12" Pipeline on Nevada St.					
12" Pipe - Excavation, Backfill	LF	$35	1,150	$40,250	
Manhole 48" - 8" thru 18" pipe	EA	$2,000	4	$8,000	
Connect/Reconnect Pipes at Manholes - 8" thru 21'	EA	$1,500	2	$3,000	
Existing Utility Protection	LF	$4	1,150	$4,600	
Reconnect Services	LF	$10	1,150	$11,500	
1/2 Lane Pavement Repair	LF	$20	1,150	$23,000	
Control Density Backfill	LF	$40	1,150	$46,000	
Traffic Control	LS	$3,000	1	$3,000	
Subtotal					*$139,350*
Mobilization - Percent of Item Cost Sum	%	5%		$6,968	
Total Construction Costs					*$146,318*
Contingency - % of construction costs	%	30%		$43,895	
Engineering and CMS - % of construction costs	%	18%		$26,337	
Total Project Cost (rounded)					***$217,000***
2B: 8" Slope Correction on Walker Ave.					
8" Pipe - Excavation, Backfill	LF	$50	670	$33,500	
Connect/Reconnect Pipes at Manholes - 8" thru 21'	EA	$1,500	2	$3,000	
Bypass Piping Setup - 8" thru 24" gravity	EA	$5,000	1	$5,000	
Bypass Pipe and Pump Operation - 8" thru 24" gravity	LF	$10	670	$6,700	
Reconnect Services	LF	$10	670	$6,700	
Existing Utility Protection	LF	$4	670	$2,680	
1/2 Lane Pavement Repair	LF	$30	670	$20,100	
Control Density Backfill	LF	$40	670	$26,800	
Traffic Control	LS	$3,000	1	$3,000	
Subtotal					*$107,480*
Mobilization - Percent of Item Cost Sum	%	5%		$5,374	
Total Construction Costs					*$112,854*
Contingency - % of construction costs	%	30%		$33,856	
Engineering and CMS - % of construction costs	%	18%		$20,314	
Total Project Cost (rounded)					***$168,000***
2C: 12" Main Wightman St.					
12" Pipe - Excavation, Backfill	LF	$35	1,300	$45,500	
Manhole 48" - 8" thru 18" pipe	EA	$2,000	4	$8,000	
Connect/Reconnect Pipes at Manholes - 8" thru 21'	EA	$1,500	2	$3,000	
Bypass Piping Setup - 8" thru 24" gravity	EA	$5,000	1	$5,000	
Bypass Pipe and Pump Operation - 8" thru 24" gravity	LF	$10	800	$8,000	
Reconnect Services	LF	$10	1,300	$13,000	
Existing Utility Protection	LF	$4	1,300	$5,200	
1/2 Lane Pavement Repair	LF	$20	1,300	$26,000	
Control Density Backfill	LF	$40	1,300	$52,000	
Traffic Control	LS	$5,000	1	$5,000	
Subtotal					*$170,700*
Mobilization - Percent of Item Cost Sum	%	5%		$8,535	
Total Construction Costs					*$179,235*
Contingency - % of construction costs	%	30%		$53,771	
Engineering and CMS - % of construction costs	%	18%		$32,262	
Total Project Cost (rounded)					***$172,000***
2D.1: Misc Upgrades - Grandview LS Force Main Replacement					
6" Pressure Pipe - Excavation, Backfill	LF	$40	720	$28,800	
Bypass Piping Setup - 8" thru 24" gravity	EA	$5,000	1	$5,000	
Bypass Pipe and Pump Operation - 8" thru 24" gravity	LF	$10	720	$7,200	
Existing Utility Protection	LF	$4	720	$2,880	
1/2 Lane Pavement Repair	LF	$30	720	$21,600	
Control Density Backfill	LF	$40	720	$28,800	
Traffic Control	LS	$3,000	1	$3,000	
Subtotal					*$97,280*
2D.2: Misc Upgrades - Shamrock LS Upgrades					
Replace with Submersible Pumps	LS	$50,000	1	$50,000	
Bypass Piping Setup - 8" thru 24" gravity	EA	$5,000	1	$5,000	
Bypass Pipe and Pump Operation - 8" thru 24" gravity	LF	$10	220	$2,200	
Subtotal					*$57,200*
2D.3: Misc Upgrades - North Mountain LS Upgrades					
Replace with Submersible Pumps, Standardize	LS	$250,000	1	$250,000	
4" Pressure Pipe - Excavation, Backfill	LF	$35	400	$14,000	
Bypass Piping Setup - 8" thru 24" gravity	EA	$5,000	1	$5,000	
Bypass Pipe and Pump Operation - 8" thru 24" gravity	LF	$10	400	$4,000	
Existing Utility Protection	LF	$4	300	$1,200	
Miscellaneous Surface Repair	LF	$5	300	$1,500	
Bore Long Length (>100feet) - incl casing & carrier pipe	LF	$450	100	$45,000	
Subtotal					*$320,700*

Item	Unit	Unit Price	Estimated Quantity	Item Cost (Rounded)	Total Cost (Rounded)
2D: Miscellaneous Upgrades					
2D.1 Grandview Lift Station Force Main Upgrade	LS	$97,300	1	$97,300	
2D.2 Shamrock Lift Station Upgrades	LS	$57,200	1	$57,200	
2D.3 North Mountain Lift Station & Force Main Upgrades	LS	$320,700	1	$320,700	
Subtotal					*$475,200*
Mobilization - Percent of Item Cost Sum	%	5%		$23,760	
Total Construction Costs					*$498,960*
Contingency - % of construction costs	%	30%		$149,688	
Engineering and CMS - % of construction costs	%	18%		$89,813	
Total Project Cost (rounded)					***$739,000***
Total Priority 2 Cost (rounded)					**$1,296,000**

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Item	Unit	Unit Price	Estimated Quantity	Item Cost (Rounded)	Total Cost (Rounded)
Priority 3					
3A: Rogue Valley Hwy 99 Lift Station					
Abandon N. Main LS - pull pumps, plug pipes, fill wet well, etc	LS	$5,000	1	$5,000	
10" Pipe - Excavation, Backfill (upsize from 8")	LF	$5	4,170	$20,850	
12" Pipe - Excavation, Backfill (upsize from 8")	LF	$10	2,310	$23,100	
New Lift Station - wet well, pumps, elec. etc.	LS	$600,000	1	$600,000	
6" Pressure Pipe - Excavation, Backfill	LF	$40	8,270	$330,800	
1/2 Lane Pavement Repair	LF	$30	8,270	$248,100	
Control Density Backfill	LF	$40	8,270	$330,800	
Traffic Control	LS	$10,000	1	$10,000	
Bore Long Length (>100feet) - incl casing & carrier pipe	LF	$450	120	$54,000	
Canal/Creek Crossing - incl. casing & carrier pipe	LS	$15,000	1	$15,000	
Subtotal					*$1,637,650*
Mobilization - Percent of Item Cost Sum	%	5%		$81,883	
Total Construction Costs					*$1,719,533*
Contingency - % of construction costs	%	30%		$515,860	
Engineering and CMS - % of construction costs	%	18%		$309,516	
Total Project Cost (rounded)					***$2,545,000***
3B: Future System Expansion					
10" Pipe - Excavation, Backfill (upsize from 8")	LF	$5	2,300	$11,500	
Subtotal					*$11,500*
Mobilization - Percent of Item Cost Sum	%	5%		$575	
Total Construction Costs					*$12,075*
Contingency - % of construction costs	%	30%		$3,623	
Engineering and CMS - % of construction costs	%	18%		$2,174	
Total Project Cost (rounded)					***$18,000***
Total Priority 3 Cost (rounded)					**$2,563,000**

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TREATMENT SYSTEM COST ESTIMATES

RECOMMENDED IMPROVEMENTS

SHADING/OUTFALL RELOCATION

TREATMENT COST ALTERNATIVES

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Priority1 - Membrane Replacement

Description	Estimate
Membranes	$960,000
Installation (by City)	$0
Construction Pretotal	$960,000
Mobilization, Contractor OH&P (15%)	$0
Subtotal	$960,000
Contingency (30%)	$288,000
Total Construction Estimate	**$1,248,000**
Engineering, Legal, Administration (20%)	NA
TOTAL PROJECT COST	**$1,248,000**

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Priority2 - Membrane Replacement

Description	Estimate
Membranes	$2,500,000
Installation	$375,000
Construction Pretotal	$2,875,000
Mobilization, Contractor OH&P (15%)	$431,250
Subtotal	$3,306,250
Contingency (30%)	$991,900
Total Construction Estimate	**$4,299,000**
Engineering, Legal, Administration (20%)	359,800
TOTAL PROJECT COST	**$4,658,800**

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Priority2 - Membrane Permeate Piping and Pumps

Description	Estimate
PermeatePiping Replacement	$120,000
Permeate Pump Replacement	$125,000
Installation	$36,750
Construction Pretotal	$281,750
Mobilization, Contractor OH&P (15%)	$42,263
Subtotal	$324,013
Contingency (30%)	$97,300
Total Construction Estimate	**$422,000**
Engineering, Legal, Administration (20%)	84,400
TOTAL PROJECT COST	**$506,400**

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Priority2 - UV Addition

Description	Estimate
UV Unit	$140,000
Piping	$30,000
Control Panel	$25,000
Installation	$36,750
Construction Pretotal	$195,000
Mobilization, Contractor OH&P (15%)	$29,250
Subtotal	$224,250
Contingency (30%)	$67,300
Total Construction Estimate	**$292,000**
Engineering, Legal, Administration (20%)	58,400
TOTAL PROJECT COST	**$350,400**

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Priority 2 - Mechanical Bar Screen Replacement

Description	Estimate
Screen	$150,000
Washer/Compactor	$40,000
Misc. piping	$7,500
Structural Modifications	$11,800
Electrical / I & C	$38,000
Installation	$28,500
Construction Pretotal	$275,800
Mobilization, Contractor OH&P (15%)	$41,370
Subtotal	$317,170
Contingency (30%)	$95,200
Total Construction Estimate	**$413,000**
Engineering, Legal, Administration (20%)	82,600
TOTAL PROJECT COST	**$495,600**

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Grit Removal Replacement

Description	Estimate
Grit Pumps	$30,000
Grit Chamber	$286,000
Misc. piping	$7,500
Structural Modifications	$11,800
Electrical / I & C	$63,200
Installation	$47,400
Construction Pretotal	$445,900
Mobilization, Contractor OH&P (15%)	$66,885
Subtotal	$512,785
Contingency (30%)	$153,900
Total Construction Estimate	**$667,000**
Engineering, Legal, Administration (20%)	133,400
TOTAL PROJECT COST	**$800,400**

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Priority 2 Oxidation Ditch Equipment Replacement

Description	Estimate
Ditch Equipment	$720,000
Installation	$144,000
Construction Pretotal	$864,000
Mobilization, Contractor OH&P (15%)	$129,600
Subtotal	$993,600
Contingency (30%)	$298,100
Total Construction Estimate	**$1,292,000**
Engineering, Legal, Administration (20%)	258,400
TOTAL PROJECT COST	**$1,550,400**

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Priority 2 Clarifier Equipment Replacement

Description	Estimate
Clarifier Equipment	$150,000
Installation	$30,000
Construction Pretotal	$180,000
Mobilization, Contractor OH&P (15%)	$27,000
Subtotal	$207,000
Contingency (30%)	$62,100
Total Construction Estimate	**$270,000**
Engineering, Legal, Administration (20%)	54,000
TOTAL PROJECT COST	**$324,000**

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Priority 2 Ashland Creek Pump Replacement

Description	Estimate
Pumps	$140,000
Electrical Upgrades	$28,000
Installation	$28,000
Construction Pretotal	$196,000
Mobilization, Contractor OH&P (15%)	$29,400
Subtotal	$225,400
Contingency (30%)	$67,700
Total Construction Estimate	**$294,000**
Engineering, Legal, Administration (20%)	58,800
TOTAL PROJECT COST	**$352,800**

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Priority 3 Additional Centrifuge

Description	Estimate
Centrifuge	$350,000
Electrical Upgrades	$35,000
Installation	$70,000
Construction Pretotal	$455,000
Mobilization, Contractor OH&P (15%)	$68,250
Subtotal	$523,250
Contingency (30%)	$157,000
Total Construction Estimate	**$680,250**
Engineering, Legal, Administration (20%)	136,100
TOTAL PROJECT COST	**$816,350**

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Priority 2 Clarifier Equipment Replacement

Description	Estimate
Clarifier Equipment	$300,000
Installation	$60,000
Construction Pretotal	$360,000
Mobilization, Contractor OH&P (15%)	$54,000
Subtotal	$414,000
Contingency (30%)	$124,200
Total Construction Estimate	**$538,200**
Engineering, Legal, Administration (20%)	107,700
TOTAL PROJECT COST	**$645,900**

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Priority 3 Additional Centrifuge

Description	Estimate
Clarifier	$988,000
Construction Pretotal	$988,000
Mobilization, Contractor OH&P (15%)	$148,200
Subtotal	$1,136,200
Contingency (30%)	$340,900
Total Construction Estimate	**$1,477,100**
Engineering, Legal, Administration (20%)	295,500
TOTAL PROJECT COST	**$1,772,600**

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Cost Analysis Summary

Restoration Compliance Alternative - City of Ashland

Estimated project scope

Project sites (owners) - approximate		44.00
Project area (acres)		56.72
Buffer Width (feet)		60.00
Project length (miles)		7.80
Direct cost per mile restoration	$	89,270
Total cost per mile of credit generation	$	441,700

The Freshwater Trust™

Prepared for the City of Ashland

CONTACT: David Primozich, Director of Ecosystem Services
(503) 434-8033, primozich@thefreshwatertrust.org

Estimated price per kcal credit

Kilocalories required by TMDL		53,000,000
Avg. avail. Kcal/mile		6,795,378
Miles required for offsets		7.80
Average $ per kcal credit	$	0.04885

Credit Generation Costs			~ % of cost
Restoration direct project costs	$	696,304	20%
Contingency (25% of restoration direct costs)	$	174,076	5%
Credit calculation & project management	$	255,976	7%
Certification, verification & registration	$	178,892	5%
Overhead (insurance, occupancy, etc.)	$	62,044	2%
Financing	$	82,038	2%
SUBTOTAL - CAPITAL COSTS	**$**	**1,449,329**	
Maintenance and monitoring	$	370,500	11%
Landowner payments (for acreage converted to conservation)	$	769,236	22%
SUBTOTAL - O&M	**$**	**1,139,736**	
Relocation of Outfall (see table to right)	*$*	*855,938*	
TOTAL	**$**	**3,445,003**	***75%***
Initital Modeling & Assessment	*$*	*200,000*	***Present Value***
GRAND TOTAL	**$**	**3,645,003**	**$ 2,921,404**

Outfall Relocation/Ashland Pond Enhancement		
Surveys (site/cultural resources)	$	20,000
Groundwater study (cooling)	$	40,000
Channel/wetlands design	$	65,000
Permit coordination	$	15,000
Construction (plant internal)	$	50,000
Construction (external)	$	212,500
Construction contingency	$	285,313
Revegetation	$	30,000
Project management & coordination	$	116,500
Overhead (inurance, occupancy etc.)	$	21,625
TOTAL OUTFALL RELOCATION	**$**	**855,938**

Cost Analysis

Restoration Compliance Alternative - City of Ashland

Estimated project scope

Project sites (owners)	44.00
Project area (acres)	56.72
Buffer Width (feet)	60.00
Project length (miles)	7.80
Cost per mile restoration	$89,270

Maintenance/Monitoring Cost per mile

Year 0-2	$ 11,000
Year 3-4	$ 6,000
Year 5-10	$ 1,000
Year 11-25	$ 500

Present Value Calculation:

Rate	4.750%

Project:

City of Ashland Wastewater Treatment Plant

Trading Ratios

0-120M Kcals	2:1
121M-214M Kcals	1.5:1
215M-324M Kcals	1.5:1

Recruitment, Construction and Maintenance, and Management Costs

Year	Year No.	Miles	CAPITAL COSTS								O&M		TOTAL	PRESENT VALUE	Ashland Billing (Capital)	Ashland Billing (O&M)	Kcals (RUNNING TOTAL)
			Direct Project Costs	Contingency (25%)	Credit Calculation	Project Management*	Certification + Verification	Registration	Overhead**	Financing (6%)	Maint/Monitor	Landowner Payments***					
2011	0	0.00	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	0
2012	1	1.50	$133,905	$33,476	$1,500	$33,476	$1,473	$15,493	$8,369	$13,662	$0	$4,364	$245,717	$234,575	$241,354	$4,364	10,200,000
2013	2	1.80	$160,686	$40,171	$1,800	$43,471	$1,767	$18,592	$10,868	$16,641	$16,500	$9,600	$320,097	$291,725	$293,997	$26,100	22,400,000
2014	3	2.50	$223,174	$55,794	$2,500	$63,054	$2,455	$25,822	$15,763	$23,314	$36,300	$16,873	$465,048	$404,609	$411,876	$53,173	39,400,000
2015	4	2.00	$178,540	$44,635	$2,000	$55,895	$1,964	$20,658	$13,974	$19,060	$56,300	$22,691	$415,715	$345,287	$336,724	$78,991	53,000,000
2016	5		$0	$0	$0	$13,860	$0	$0	$3,465	$1,040	$69,300	$22,691	$110,355	$87,503	$18,365	$91,991	
2017	6		$0	$0	$0	$9,860	$982	$0	$2,465	$798	$49,300	$25,527	$88,933	$67,319	$14,105	$74,827	
2018	7		$0	$0	$0	$6,060	$1,178	$0	$1,515	$525	$30,300	$25,527	$65,106	$47,048	$9,278	$55,827	
2019	8		$0	$0	$0	$3,560	$1,636	$0	$890	$365	$17,800	$25,527	$49,779	$34,341	$6,452	$43,327	
2020	9		$0	$0	$0	$1,560	$1,309	$0	$390	$196	$7,800	$25,527	$36,782	$24,224	$3,455	$33,327	
2021	10		$0	$0	$0	$1,560	$0	$0	$390	$117	$7,800	$25,527	$35,394	$22,253	$2,067	$33,327	
2022	11		$0	$0	$0	$1,560	$982	$0	$390	$176	$7,800	$28,364	$39,271	$23,571	$3,108	$36,164	
2023	12		$0	$0	$0	$1,410	$1,178	$0	$353	$176	$7,050	$28,364	$38,531	$22,078	$3,117	$35,414	
2024	13		$0	$0	$0	$1,230	$1,636	$0	$308	$190	$6,150	$28,364	$37,878	$20,720	$3,364	$34,514	
2025	14		$0	$0	$0	$980	$1,309	$0	$245	$152	$4,900	$28,364	$35,950	$18,773	$2,686	$33,264	
2026	15		$0	$0	$0	$780	$0	$0	$195	$59	$3,900	$28,364	$33,297	$16,600	$1,034	$32,264	
2027	16		$0	$0	$0	$780	$4,582	$0	$195	$333	$3,900	$31,200	$40,990	$19,508	$5,890	$35,100	
2028	17		$0	$0	$0	$780	$5,236	$0	$195	$373	$3,900	$31,200	$41,684	$18,939	$6,584	$35,100	
2029	18		$0	$0	$0	$780	$5,236	$0	$195	$373	$3,900	$31,200	$41,684	$18,080	$6,584	$35,100	
2030	19		$0	$0	$0	$780	$5,060	$0	$195	$362	$3,900	$31,200	$41,497	$17,183	$6,397	$35,100	
2031	20		$0	$0	$0	$780	$0	$0	$195	$59	$3,900	$31,200	$36,134	$14,283	$1,034	$35,100	
2032	21		$0	$0	$0	$780	$4,582	$0	$195	$333	$3,900	$34,036	$43,827	$16,539	$5,890	$37,936	
2033	22		$0	$0	$0	$780	$5,236	$0	$195	$373	$3,900	$34,036	$44,520	$16,039	$6,584	$37,936	
2034	23		$0	$0	$0	$780	$5,236	$0	$195	$373	$3,900	$34,036	$44,520	$15,311	$6,584	$37,936	
2035	24		$0	$0	$0	$780	$5,060	$0	$195	$362	$3,900	$34,036	$44,333	$14,556	$6,397	$37,936	
2036	25		$0	$0	$0	$780	$0	$0	$195	$59	$3,900	$34,036	$38,970	$12,215	$1,034	$37,936	
2037	26		$0	$0	$0	$780	$4,582	$0	$195	$333	$3,900	$36,873	$46,663	$13,963	$5,890	$40,773	
2038	27		$0	$0	$0	$630	$5,236	$0	$158	$361	$3,150	$29,782	$39,317	$11,231	$6,385	$32,932	
2039	28		$0	$0	$0	$450	$5,236	$0	$113	$348	$2,250	$21,273	$29,670	$8,091	$6,147	$23,523	
2040	29		$0	$0	$0	$200	$5,060	$0	$50	$319	$1,000	$9,455	$16,083	$4,187	$5,628	$10,455	
2041	30		$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	
2042	31		$0	$0	$0	$0	$4,582	$0	$0	$275	$0	$0	$4,857	$1,152	$4,857	$0	
2043	32		$0	$0	$0	$0	$5,236	$0	$0	$314	$0	$0	$5,551	$1,257	$5,551	$0	
2044	33		$0	$0	$0	$0	$5,236	$0	$0	$314	$0	$0	$5,551	$1,200	$5,551	$0	
2045	34		$0	$0	$0	$0	$5,060	$0	$0	$304	$0	$0	$5,363	$1,107	$5,363	$0	
		7.80	**$696,304**	**$174,076**	**$7,800**	**$248,176**	**$98,326**	**$80,566**	**$62,044**	**$82,038**	**$370,500**	**$769,236**	**$2,589,066**	**$1,865,466**	**$1,449,329**	**$1,139,736**	
			26.89%	6.72%	0.30%	9.59%	3.80%	3.11%	2.40%	3.17%	14.31%	29.71%	100.00%				
												Initial Modeling & Assessment	*$200,000*	*$200,000*			
												Grand Total	**$2,789,066**	**$2,065,466**			

* Project Management is calculated based on 20% of Direct Project Costs + Contingency

** Overhead is calculated based on Direct Project Costs + Contingency + Maintenance/Monitoring @ 5%.

*** Assumes $400/acre, with periodic escalation

**** Final accounting/reconciliation for compliance schedule. May occur sooner if kilocalorie requirment met ahead of schedule.

Restoration Implemention Cost Worksheet

Bear Creek Heat Source Analysis Results										
Kcal Potential Category	Average of 25% Potential per mile (kcal/day)	Excess Load (kcal/day)	# Miles by Potential Category	% Miles by Potential Category	Deliverable Solar Load Reductions Kcal/day (TOTAL)	% OF TOTAL	PROJECT MILES	COST PER MILE	TOTAL	AVG PER MILE
low	3,257,325	44,000,000	5.16	19%	16,807,797		1.48	$ 76,441	$ 113,309	
mid	6,818,843	44,000,000	16.65	61%	113,533,736		4.78	$ 89,738	$ 429,223	
high	10,141,007	44,000,000	5.34	20%	54,152,977		1.53	$ 100,207	$ 153,720	
TOTALS	6,795,378	= WEIGHTED AVG	27.15	100%	184,494,510	100.00%	7.80		$ 696,252	$ 89,269.75

GRAND TOTALS		= WEIGHTED AVG				TOTAL MILES	7.80		$ -	$ -

Category	UNIT TYPE	UNIT #	RATE	LOW		UNIT #	RATE	MID		UNIT #	RATE	HIGH	
				ACRE	MILE (60' BUFFER)			ACRE	MILE (60' BUFFER)			ACRE	MILE (60' BUFFER
Design	Labor	40.00	45.00	$ 1,800	$ 1,800	40.00	45.00	$ 1,800	$ 1,800	40.00	45.00	$ 1,800	$ 1,800
Site Assessment & Baseline Monitoring	Labor	4.00	45.00	$ 180	$ 1,309	4.00	45.00	$ 180	$ 1,309	4.00	45.00	$ 180	$ 1,309
Site Prep	Labor	10.00	20.00	$ 200	$ 1,400	20.00	20.00	$ 400	$ 2,908	30.00	20.00	$ 600	$ 4,362
	Materials			$ 1,000	$ 7,000			$ 2,000	$ 14,540			$ 3,000	$ 21,810
Plants and Planting	Plants	1,600.00	1.00	$ 1,600	$ 11,200	1,600.00	1.00	$ 1,600	$ 11,632	1,600.00	1.00	$ 1,600	$ 11,632
	Plant Protection	1,600.00	1.00	$ 1,600	$ 11,200	1,600.00	1.00	$ 1,600	$ 11,632	1,600.00	1.00	$ 1,600	$ 11,632
	Labor	1,600.00	2.50	$ 4,000	$ 28,000	1,600.00	2.50	$ 4,000	$ 29,080	1,600.00	2.50	$ 4,000	$ 29,080
Local Proj Mgmt	Labor			$ 2,076	$ 14,532			$ 2,316	$ 16,837			$ 2,556	$ 18,582
TOTALS				$ 12,456	$ 76,441			$ 11,580	$ 89,738			$ 12,780	$ 100,207

Item	Units	Cost Each	Quantity	Unit Total	Subtotal	Item Total
Capital Costs						
Sitework						
Sitework Subtotal						*$ 101,760*
Pump Station						$ 340,000
FilterBuilding						$ 420,000
Mechanical						
Filter Equipment	LS	$ 1,020,000	1	$ 1,020,000		
Utility Water	LS	$ 35,000	1	$ 35,000		
Valves	EA	$ 4,000	2	$ 8,000		
Pumps	Ea	$ 25,000	3	$ 75,000		
Conveyance Equipment	LS	$ 160,000	1	$ 160,000		
Equipment Installation	%	$ 1,138,000	25%	$ 284,500		
Taxes	%	$ 1,138,000	6%	$ 68,280		
Piping	ft	$ 110	120	$ 13,200		
Misc. Concrete	ft	$ 150	120	$ 18,000		
					$1,681,980	
Electrical	LS	$ 21,000	1	$ 353,216		
					$ 353,216	
Subtotal						*$ 2,035,196*
Electrical Site Work	LS	$ 25,000	1	$ 25,000		
Electrical Site Subtotal						*$ 25,000*
Controls Site Work						
Instrumentation	LS	$ 5,000	1	$ 5,000		
Ductbank	LS	$ 10,000	1	$ 10,000		
Panels	LS	$ 5,000	1	$ 5,000		
Programming	LS	$ 10,000	1	$ 10,000		
Controls Subtotal						*$ 30,000*
Treatment System Construction Pretotal	---			**$ 2,090,000**		**$ 2,950,000**
Mobilization & Contractor OH&P	---		15%	$ 442,500		
Subtotal						**$ 3,390,000**
Contingency	---		30%	$ 1,017,000		
Subtotal						**$ 4,410,000**
Engineering			18.0%			$ 793,800
Administration and Wetlands Mitigation			2.0%			$ 188,200
Subtotal						**$ 5,400,000**
Total Capital	---					**$ 5,400,000**

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Item	Units	Cost Each	Quantity	Unit Total	Subtotal	Item Total
Capital Costs						
Blower Building						*$ 450,000*
Oxidation Ditches						
Structural						
Concrete Walls	cy	$ 850	780	$ 663,000		
Reinforcing Center Wall	cy	$ 850	180	$ 153,000		
Concrete Hanging Slabs	cy	$ 1,200	62	$ 74,400		
					$ 890,400	
Mechanical						
Aeration Equipment	LS	$ 139,286	1	$ 139,286		
Blowers (5600 SCFM) Turbo	LS	$ 124,000	3	$ 372,000		
Blower Air Piping	EA	$ 150,000	1	$ 150,000		
Valves	Ea	$ 4,500	2	$ 9,000		
Mixers	Ea	$ 7,000	5	$ 35,000		
Equipment Installation	%	$ 705,286	25%	$ 176,321		
Taxes	%	$ 705,286	6%	$ 42,317		
Air Pipe Runs	FT	$ 180	500	$ 90,000		
Misc. Concrete	FT	$ 150	500	$ 75,000		
					$1,088,924	
Electrical	LS	$ 31,500	1	$ 415,658		
					$ 415,658	
Oxidation Ditches Subtotal						*$ 2,394,982*
Electrical Site Work	LS	$ 25,000	1	$ 25,000		
Electrical Site Subtotal						*$ 25,000*
Controls Site Work						
Instrumentation	LS	$ 5,000	1	$ 5,000		
Ductbank	LS	$ 10,000	1	$ 10,000		
Panels	LS	$ 5,000	1	$ 5,000		
Programming	LS	$ 10,000	1	$ 10,000		
Controls Subtotal						*$ 30,000*
Treatment System Construction Pretotal	---			**$ 2,450,000**		**$ 2,900,000**
Mobilization & Contractor OH&P	---		15%	$ 435,000		
Subtotal						**$ 3,340,000**
Contingency	---		30%	$ 1,002,000		
Subtotal						**$ 4,340,000**
Engineering			18.0%			$ 781,200
Administration			2.0%			$ 86,800
Subtotal						**$ 5,210,000**
Total Capital	---					**$ 5,210,000**

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Ashland WWTP Improvements
IFAS in Existing Ditch

Item	Units	Cost Each	Quantity	Unit Total	Subtotal	Item Total
Capital Costs						
Blower Building						*$ 450,000*
Oxidation Ditches						
Structural						
Concrete Walls	cy	$ 850	39	$ 33,150		
					$ 33,150	
Mechanical						
IFAS Equipment	LS	$ 1,088,500	1	$ 1,088,500		
Blowers (8000 SCFM) Turbo	LS	$ 124,000	4	$ 496,000		
Blower Air Piping	LS	$ 150,000	1	$ 150,000		
Valves	Ea	$ 4,500	6	$ 27,000		
Mixers	Ea	$ 7,000	4	$ 28,000		
Chemical Feed System	LS	$ 50,000	1	$ 50,000		
Equipment Installation	%	$ 1,789,500	25%	$ 447,375		
Taxes	%	$ 1,789,500	6%	$ 107,370		
Air Pipe Runs	ft	$ 180	500	$ 90,000		
Misc. Concrete	ft	$ 150	500	$ 75,000		
					$2,559,245	
Electrical/Control	LS	21%	1	$ 544,403		
					$ 544,403	
Oxidation Ditches Subtotal						*$ 3,136,798*
Electrical Site Work	LS	$ 25,000	1	$ 25,000		
Electrical Site Subtotal						*$ 25,000*
Controls Site Work						
Instrumentation	LS	$ 5,000	1	$ 5,000		
Ductbank	LS	$ 10,000	1	$ 10,000		
Panels	LS	$ 5,000	1	$ 5,000		
Programming	LS	$ 10,000	1	$ 10,000		
Controls Subtotal						*$ 30,000*
Treatment System Construction Pretotal	---			**$ 3,190,000**		**$ 3,640,000**
Mobilization & Contractor OH&P	---		15%	$ 546,000		
Subtotal						**$ 4,190,000**
Contingency	---		30%	$ 1,257,000		
Subtotal						**$ 5,450,000**
Engineering			18.0%			$ 981,000
Administration			2.0%			$ 109,000
Subtotal						**$ 6,540,000**
Total Capital	---					**$ 6,540,000**

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Item	Units	Cost Each	Quantity	Unit Total	Subtotal	Item Total
Capital Costs						
Sitework						
Sitework Subtotal						*$ 297,700*
Oxidation Ditches						
Rock Excavation	cy	$ 60	3333	$ 200,000		
					$ 200,000	
Structural						
Foundation Slab	cy	$ 500	716	$ 358,000		
Concrete Walls	cy	$ 850	1560	$ 1,326,189		
Concrete Hanging Slabs	cy	$ 1,200	260	$ 311,731		
Grating	sq ft	$ 10	52	$ 520		
Railing	ft	$ 15	404	$ 6,060		
Stairs	ft	$ 40	24	$ 960		
					$2,003,460	
Mechanical						
Equipment	LS	$ 359,375	1	$ 359,375		
Utility Water	LS	$ 10,000	1	$ 10,000		
Valves	EA	$ 4,000	2	$ 8,000		
Slide Gates	Ea	$ 4,500	2	$ 9,000		
Weir	Ea	$ 1,000	2	$ 2,000		
Equipment Installation	%	$ 388,375	25%	$ 97,094		
Taxes	%	$ 388,375	6%	$ 23,303		
20" ML	ft	$ 110	220	$ 24,200		
Misc Piping	ft	$ 90	230	$ 20,680		
Misc. Concrete	ft	$ 150	900	$ 134,934		
4" UW	ft	$ 65	80	$ 5,200		
Hose Bibbs	ea	$ 250	4	$ 1,000		
					$ 694,785	
Electrical	LS	$ 31,500	1	$ 78,750		
					$ 78,750	
Oxidation Ditches Subtotal						*$ 2,976,995*
Electrical Site Work	LS	$ 25,000	1	$ 25,000		
Electrical Site Subtotal						*$ 25,000*
Controls Site Work						
Instrumentation	LS	$ 12,500	1	$ 12,500		
Ductbank	LS	$ 25,000	1	$ 25,000		
Panels	LS	$ 12,500	1	$ 12,500		
Programming	LS	$ 25,000	1	$ 25,000		
Controls Subtotal						*$ 75,000*
Treatment System Construction Pretotal	---			**$ 3,080,000**		**$ 3,370,000**
Mobilization & Contractor OH&P	---		15%	$ 505,500		
Subtotal						**$ 3,880,000**
Contingency	---		30%	$ 1,164,000		
Subtotal						**$ 5,040,000**
Engineering			18.0%			$ 907,200
Administration and Wetlands Mitigation			2.0%			$ 200,800
Subtotal						**$ 6,150,000**
Total Capital	---					**$ 6,150,000**

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Item	Units	Cost Each	Quantity	Unit Total	Subtotal	Item Total
Capital Costs						
Sitework						
Sitework Subtotal						*$ 195,111*
Oxidation Ditches						
Rock Excavation	cy	$ 60	3333	$ 200,000		
					$ 200,000	
Structural						
Foundation Slab	cy	$ 500	716	$ 358,000		
Concrete Walls	cy	$ 850	1282	$ 1,089,511		
Concrete Hanging Slabs	cy	$ 1,200	0	$ -		
Grating	sq ft	$ 10	52	$ 520		
Railing	ft	$ 15	404	$ 6,060		
Stairs	ft	$ 40	24	$ 960		
					$1,455,051	
Mechanical						
Equipment	LS	$ 359,375		$ 84,000		
Utility Water	LS	$ 10,000		$ -		
Valves	EA	$ 4,000		$ -		
Slide Gates	Ea	$ 4,500		$ -		
Weir	Ea	$ 1,000		$ -		
Equipment Installation	%	$ 84,000	25%	$ 21,000		
Taxes	%	$ 84,000	6%	$ 5,040		
20" ML	ft	$ 110	220	$ 24,200		
Misc Piping	ft	$ 90	230	$ 20,680		
Misc. Concrete	ft	$ 150	900	$ 134,934		
4" UW	ft	$ 65	80	$ 5,200		
Hose Bibbs	ea	$ 250	4	$ 1,000		
					$ 296,054	
Electrical	LS	$ 31,500	0	$ -		
					$ -	
Oxidation Ditches Subtotal						*$ 1,951,105*
Electrical Site Work	LS	$ 25,000	1	$ 25,000		
Electrical Site Subtotal						*$ 25,000*
Controls Site Work						
Instrumentation	LS	$ 12,500	0	$ -		
Ductbank	LS	$ 25,000	0	$ -		
Panels	LS	$ 12,500	0	$ -		
Programming	LS	$ 25,000	0	$ -		
Controls Subtotal						*$ -*
Treatment System Construction Pretotal	---			**$ 1,980,000**		**$ 2,170,000**
Mobilization & Contractor OH&P	---		15%	$ 325,500		
Subtotal						**$ 2,500,000**
Contingency	---		30%	$ 750,000		
Subtotal						**$ 3,250,000**
Engineering			18.0%			$ 585,000
Administration and Wetlands Mitigation			2.0%			$ 165,000
Subtotal						**$ 4,000,000**
Total Capital	---					**$ 4,000,000**

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Item	Units	Cost Each	Quantity	Unit Total	Subtotal	Item Total
Capital Costs						
Sitework						
Sitework Subtotal						*$ 41,413*
Pump Station						$ 170,000
FilterBuilding						$ 420,000
Mechanical						
Filter Equipment	LS	$ 340,000	1	$ 340,000		
Utility Water	LS	$ 35,000	1	$ 35,000		
Valves	EA	$ 4,000	2	$ 8,000		
Pumps	Ea	$ 25,000	3	$ 75,000		
Conveyance Equipment	LS	$ 53,333	1	$ 53,333		
Equipment Installation	%	$ 458,000	25%	$ 114,500		
Taxes	%	$ 458,000	6%	$ 27,480		
Piping	ft	$ 110	120	$ 13,200		
Misc. Concrete	ft	$ 150	120	$ 18,000		
					$ 684,513	
Electrical	LS	$ 21,000	1	$ 143,748		
					$ 143,748	
Subtotal						*$ 828,261*
Electrical Site Work	LS	$ 25,000	1	$ 25,000		
Electrical Site Subtotal						*$ 25,000*
Controls Site Work						
Instrumentation	LS	$ 5,000	1	$ 5,000		
Ductbank	LS	$ 10,000	1	$ 10,000		
Panels	LS	$ 5,000	1	$ 5,000		
Programming	LS	$ 10,000	1	$ 10,000		
Controls Subtotal						*$ 30,000*
Treatment System Construction Pretotal	---			**$ 880,000**		**$ 1,510,000**
Mobilization & Contractor OH&P	---		15%	$ 226,500		
Subtotal						**$ 1,740,000**
Contingency	---		30%	$ 522,000		
Subtotal						**$ 2,260,000**
Engineering			18.0%			$ 406,800
Administration and Wetlands Mitigation			2.0%			$ 145,200
Subtotal						**$ 2,820,000**
Total Capital	---					**$ 2,820,000**

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Short-Lived Assets

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6-Year Capital Improvement Plan

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UPDATED (11/15/11)

Ashland Wastewater Short Lived Assets (2011)

Equipment Description	Replacement Items	Unit Cost	Frequency (Yr)	Annual Cost
Collection System Pump Stations				
6 Lift Stations	Pumps / SCADA replacement	$ 180,000	15	$ 12,000
(Excludes Ashland Creek and Nevada Lift Stations)				
Wastewater Treatment Plant Facilities				
Ashland Creek Lift Station	Pumps	$ 175,000	15	$ 11,667
Headworks				
	Grit Classifier	$ 30,000	15	$ 2,000
	Grit Pumps	$ 25,000	15	$ 1,667
	Screening Compactor	$ 125,000	15	$ 8,333
	Flowmeter	$ 10,000	15	$ 667
Oxidation Ditch	Aerators	$ 200,000	15	$ 13,333
	Anoxic Mixers	$ 25,000	15	$ 1,667
RAS Pumps		$ 90,000	15	$ 6,000
WAS Pumps		$ 40,000	15	$ 2,667
Membrane Filtration System	Blowers	$ 20,000	15	$ 1,333
	Permeate Pumps	$ 80,000	15	$ 5,333
	Backpulse Pumps	$ 20,000	15	$ 1,333
	Vacuum Pumps	$ 9,000	15	$ 600
	Drain Pump	$ 10,000	15	$ 667
	Reject Pumps	$ 8,000	15	$ 533
	Membrane Feed Pumps	$ 20,000	15	$ 1,333
	Chemical Pumps	$ 18,000	10	$ 1,800
	Air Compressor	$ 6,000	15	$ 400
	Alum Pumps	$ 9,000	10	$ 900
	No. 4 Water Pumps	$ 15,000	15	$ 1,000
Backup Portable Pump		*$ 60,000*	*15*	*$ 4,000*
Scum Pumps		$ 5,000	15	$ 333
UV Lamp Replacement		$ 5,000	10	$ 500
Re-Aeration Blowers		$ 55,000	15	$ 3,667
EQ Basin Improvements	*Submersible Pumps*	*$ 50,000*	*15*	*$ 3,333*
	Check Valve Replacement	*$ 20,000*	*15*	*$ 1,333*
Utility Water System	Utility Water Pumps	$ 15,000	15	$ 1,000
Solids Handling Improvements	Centrifuge Feed Pumps	$ 30,000	15	$ 2,000
	Polymer Feed Pumps	$ 15,000	10	$ 1,500
Electrical/SCADA	PLC / Instrumentation Replacements	$ 200,000	15	$ 13,333
	Biofilter Media	$ 5,000	2	$ 2,500
Total Annual Cost for Short-Lived Assets:				**$ 108,733**

Future Facilities Items Shown in Blue Text (Itallics) *$ 8,667*

UPDATED (1/23/12)

City of Ashland

Updated 6-year CIP

	Cost	Opinion of Probable Costs (2011 Dollars) 2012	2013	2014	2015	2016	2017
Wastewater Treatment Plant							
Outfall Relocation	$ 856,000	$ 20,000	$ 20,000	$ 20,000	$ 80,000	$ 358,000	$ 358,000
Shading	$ 1,646,000	$ 246,000	$ 320,000	$ 465,000	$ 416,000	$ 110,000	$ 89,000
Backup Portable Lift Station Pump	$ 60,000	$ 60,000					
Membrane System	$ 1,248,000		$ 624,000	$ 624,000			
Oxidation Ditch Shell	$ 4,000,000		$ 224,000	$ 450,000	$ 1,663,000	$ 1,663,000	
RAS Pumps	$ 90,000				$ 90,000		
Facilities Plan	$ 35,000	$ 35,000					
Wastewater Master Plan Update	$ 125,000					$ 62,500	$ 62,500
Collection System Improvements							
18" and 24" Parallel Trunkline Along Creek	$1,248,000	$24,960	$99,840	$561,600	$561,600		
15" Main Along Mountain Ave	$118,000	$2,360	$9,440	$106,200			
Oak St. 24" Trunkline	$40,000	$800	$3,200	$36,000			
A St 15" Main	$522,000	$10,440	$41,760	$94,000		$375,800	
12" Main Along Railroad	$275,000	$5,500	$22,000	$123,500	$124,000		
12" Siskiyou Blvd Main	$73,000				$1,460	$5,840	$65,700
Miscellaneous Upgrades	$335,000	$170,000	$165,000				
Portable Flow Meters	$60,000	$60,000					
Storm Water Inflow Study	$60,000	$60,000					
	$ 10,791,000	$ 695,060	$ 1,529,240	$ 2,480,300	$ 2,936,060	$ 2,575,140	$ 575,200

Rogue Valley Sewer Disposal Option

Cost Estimates

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Item	Unit	Unit Price	Estimated Quantity	Item Cost (Rounded)	Wet Industry Based Total Cost (Rounded)	ERU Based Total Cost (Rounded)
Medford Disposal Option						
X1: Lift Station at WWTP & Force Main to Talent						
Equalization Basin (assumes converting existing basins to storage)	LS	$1,000,000	1	$1,000,000		
New Lift Station - wet well, pumps, elec. etc.	LS	$1,500,000	1	$1,500,000		
30" Pipe - Excavation, Backfill	LF	$125	24,500	$3,062,500		
Full Lane Pavement Repair	LF	$60	14,344	$860,640		
Control Density Backfill	LF	$40	14,344	$573,760		
Traffic Control	LS	$50,000	1	$50,000		
Wrights Creek Crossing - incl. casing & carrier pipe	LS	$15,000	1	$15,000		
Subtotal					*$7,061,900*	*$7,061,900*
Mobilization - Percent of Item Cost Sum	%	5%		$353,095		
Total Construction Costs					*$7,414,995*	*$7,414,995*
Contingency - % of construction costs	%	30%		$2,224,499		
Engineering and CMS - % of construction costs	%	18%		$1,334,699		
Total Project Cost (rounded)					***$10,975,000***	***$10,975,000***
X2: SDC Connection to Regional System						
ST-SDC Fees - Medford Treatment Plant - Flat Rate Method	EDU	$1,212	13,000	$15,756,000		←
ST-SDC Fees - Medford Treatment Plant - Flow/Load Method	LS	$8,809,360	1	$8,809,360	←	
Flow (max month)	Gal	$1.95	3,600,000	$7,020,000		
BOD (avg day)	ppd	$353	2,680	$946,040		
TSS (avg day)	ppd	$232	3,635	$843,320		
I-SDC Fees - Rogue Valley Sewer Interceptor Capital Expansion (assumed amount per RVS buys ~4.8 MGD peak flow)	LS	$5,000,000	1	$5,000,000	←	←
Total Project Cost (rounded)					***$13,810,000***	***$20,756,000***
X3: Abandon Existing WWTP						
Abandon WWTP Structures - fill/demolish (assumes 1/2 of previous study, inflated)	LS	$192,000	1	$192,000		
24" Pipe - Excavation, Backfill (Gravity Trans., old Headworks to new LS)	LF	$115	1,000	$115,000		
Subtotal					*$307,000*	*$307,000*
Mobilization - Percent of Item Cost Sum	%	5%		$15,350		
Total Construction Costs					*$322,350*	*$322,350*
Contingency - % of construction costs	%	30%		$96,705		
Engineering and CMS - % of construction costs	%	18%		$58,023		
Total Project Cost (rounded)					***$478,000***	***$478,000***
Total Current Project Cost (rounded)					**$25,263,000**	**$32,209,000**
X4: SDC Connection - Additional for 2030 Growth						
ST-SDC Fees - Medford Treatment Plant - Flat Rate Method	EDU	$1,212	2,315	$2,805,780		←
ST-SDC Fees - Medford Treatment Plant - Flow/Load Method	LS	$1,566,108	1	$1,566,108	←	
Flow	Gal	$1.95	640,000	$1,248,000		
BOD	ppd	$353	476	$168,185		
TSS	ppd	$232	646	$149,924		
I-SDC Fees - Rogue Valley Sewer Interceptor Capital Expansion (estimated)	LS	$850,000	1	$850,000	←	←
Total Project Cost (rounded)					***$2,417,000***	***$3,655,780***
Total Future Project Cost (rounded)					**$27,680,000**	**$35,865,000**

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APPENDIX G

Financial Analysis

KELLER associates

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City of Ashland, Oregon

WASTEWATER UTILITY:
FINANCIAL HISTORY & FORECAST

Prepared by:

ECONOMIC & FINANCIAL ANALYSIS

1409 Franklin Street ▪ Suite 201
Vancouver, WA 98660

March 2012

ECONOMIC & FINANCIAL ANALYSIS

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CONTENTS

TABLES

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INTRODUCTION

This report presents a 7-year financial history and an 8-year financial forecast of the wastewater utility. The report addresses the City's need to construct approximately $13 million of capital improvements over the next 5 to 6 years, and to cover annual operating and maintenance expenses. The report contains a Cash Flow History, a Cash Flow Forecast, and two appendices showing future debt service and detailed forecast assumptions.

The City's fiscal year spans half of two calendar years, from July 1 through June 30. For ease of reference, we refer to the fiscal year in this report as the second of the two calendar years—for example, FY 2012 is the fiscal year beginning July 1, 2011 and ending June 30, 2012.

CASH FLOW HISTORY

Table 1 shows a 7-year cash flow history for the sewer utility, modified to detail the various sources of revenues—user fees, bond proceeds, SDCs, interest earnings, miscellaneous—and operating and capital-related expenses. Six years of history are audited results; FY 2012 is a projection based on actual revenues and expenditures for the first half of FY 2012. The statement includes 4 cash flow activities that clearly associate revenues and related expenses—Operations, Capital-Related, Investments, and accumulated Cash & Equivalents. Think of these activities as cascading waterfalls: any net cash from operations can be used for capital (but not vice versa), and net cash from capital and investments goes into Cash & Equivalents.

In the following subsections, we address each activity in Table 1 and summarize the conclusions in the last subsection.

Table 1 Wastewater Utility Cash Flow History

FY ending June 30	Audited History 2005 2006	2006 2007	2007 2008	2008 2009	2009 2010	2010 2011	Estimate 2011 2012	2006-2012 Avg Ann % Δ
CASH FLOWS FROM OPERATING ACTIVITIES								
Revenues								
Customer Receipts	$2,709,655	$2,463,081	$2,598,788	$2,705,970	$3,129,132	$3,358,725	$3,558,000	4.5%
Miscellaneous	2,449	74,088	3,167	110	5	0	14,250	29.4%
Total Revenues	$2,712,104	$2,537,169	$2,601,955	$2,706,080	$3,129,137	$3,358,725	$3,572,250	4.6%
Expenditures								
Personal Services	652,425	784,308	775,407	869,678	831,611	821,874	922,420	5.8%
Materials & Services	1,326,042	2,201,942	2,320,863	1,415,675	2,154,359	2,285,776	2,324,366	9.4%
Tax Equivalents	181,819	186,986	195,225	255,982	317,840	272,658	297,670	8.2%
Total Expenditures	$2,160,286	$3,173,236	$3,291,495	$2,541,335	$3,303,810	$3,380,308	$3,544,456	8.3%
Net Cash From Operating Activities	$551,818	($636,067)	($689,540)	$164,745	($174,673)	($21,583)	$27,794	-49.8%
CASH FLOWS FROM CAPITAL & RELATED ACTIVITIES								
System Development Charges	410,910	208,554	127,224	75,843	55,945	68,956	56,392	
Food & Beverage Tax	1,480,566	1,594,280	1,566,868	1,495,164	1,583,807	1,592,897	1,703,000	
Capital Expenditures	(966,378)	(155,329)	(519,817)	(355,062)	85,111	(371,568)	(555,650)	
Long-term Debt								
Loan Proceeds						324,400	995,000	
Principal	(957,257)	(990,373)	(1,024,634)	(760,081)	(1,110,644)	(2,225,002)	(1,155,318)	
Interest	(961,028)	(793,546)	(757,648)	(716,790)	(689,200)	313,560	(535,836)	
Loan Costs (DEQ)							(4,975)	
Net Cash From Capital & Related Activities	($993,187)	($136,414)	($608,007)	($260,926)	($74,981)	($296,757)	$502,613	
CASH FLOWS FROM INVESTING ACTIVITIES								
Interest	170,856	239,195	178,212	44,060	42,501	22,475	17,944	
Net Cash From Investing Activities	$170,856	$239,195	$178,212	$44,060	$42,501	$22,475	$17,944	
Net Change In Cash & Equivalents	($270,513)	($533,286)	($1,119,335)	($52,121)	($207,153)	($295,865)	$548,351	
CASH & EQUIVALENTS, Beginning	5,222,212	4,951,699	4,418,413	3,299,078	3,246,957	3,039,804	2,743,939	-10.7%
CASH & EQUIVALENTS, Ending	$4,951,699	$4,418,413	$3,299,078	$3,246,957	$3,039,804	$2,743,939	$3,292,290	-6.8%

Operating Activities

Cash Flows From Operating Activities in the preceding table show that operating expenses have been growing faster (8.3% per year) than revenues from sewer rates (4.5%). As a result, Net Cash from Operating Activities was negative in 4 of the past 7 years. Operating Revenues are primarily from sewer rates. Some miscellaneous revenues related to operations also come into the fund. These amounts are negligible. Operating Expenses for sewer collection and treatment include Personal Services (personnel), Materials & Services, and Tax Equivalents (franchise fees).

Revenues from sewer rates have increased in response to growth in the customer base and increases in the sewer rates. The City increased rates 10% in FY 2007, 3% in FY 2008, 20% in FY 2009, 9% in FY 2010, and 6% in FY 2011. The average annual rate increases were 9% over the 6-year period, but revenues increased only an average of 4.5% per year. In other words, sales of sewer services decreased over the period.

Personal Services have increased at 5.8% per year since FY 2007.

Materials & Services fluctuate significantly from year-to-year owing to changes in operating costs and repair and replacement of minor capital equipment. Overall, the average increase has been about 9.4% per year.

Tax Equivalents are franchise fees the City charges its sewer utility. The City Council sets the fee at about 10% of Customer Receipts and allocates 8% of the 10% to the General Fund, and up to 2% of the 10% to the Transportation Fund. The actual percentage varies from 6.7% in FY 2006 to between 8% and 10% in more recent years. Over the years, the City Council has varied the franchise fee for transportation.

Net Cash From Operating Activities for utilities with outstanding revenue bonds should be equal to or greater than the amount of annual debt services plus required debt coverage. Although Ashland’s sewer utility has outstanding bonds, it is different for two reasons. First, the City secured repayment of the sewer debt with a Full Faith & Credit Pledge, and these bonds do not usually require debt coverage. Second, the City imposes a Food & Beverage Tax and dedicates these revenues to repayment of the sewer debts. In any case, however, Net Cash From Operating Activities should be positive in order to cover the fluctuations in operating costs from year-to-year and to pay for cash acquisition of some capital improvements.

Capital & Related Activities

Cash Flows From Capital & Related Activities include all revenues dedicated to paying debt service and all expenses associated with making capital improvements. SDC Revenues, Bond Proceeds, and revenues from the Food & Beverage Tax are all included in this activity and are restricted to capital improvements and debt service (interest expense and principal on outstanding debts).

The City’s sewer utility has two outstanding debts. The first is a loan from Bank of America, which the City borrowed on "Full Faith & Credit" to make both sewer (30%) and water (70%) capital improvements. The sewer utility’s share of annual debt service is approximately $29,000. The interest rate varies from about 5.0% to over 7.0% per year, and will be fully repaid in FY 2024. The City uses cash in the sewer utility to pay debt service.

The second loan is only for the sewer system and is also a Full Faith & Credit obligation from U.S. Bank, with about $13.8 million remaining to be repaid through FY 2022. Interest varies from 3.5% to 4.0% per year and annual debt service is approximately $1.63 million. The City pledged to use revenues from the Food & Beverage Tax to pay debt service.[1] When this source is inadequate, other sewer revenues (SDCs, cash reserves) are used to complete the annual debt service.

In the past 7 years, total revenues from capital-related activities have been less than the sum of annual debt service and capital expenditures. In addition, only in FY 2009 did the Food & Beverage Tax revenues exceed debt service. In all other years, the tax revenues were less than annual debt service. In addition, revenues from System Development Charges (SDC) have been less than Capital Expenditures. As a result, net income from capital-related activities has been negative.

Investing Activities & Cash Reserves

Cash Flows From Investing Activities are earnings from idle cash the City invests. This source of revenue has decreased from $170,856 in FY 2006 to $22,475 in FY 2011 a projected $17,944 at the end of FY 2012. This trend results for two reasons. First, interest is earned on the idle cash represented by the beginning and ending Cash & Equivalents, and these reserves have decreased from nearly $5 million at the end of FY 2006 to about $2.7 million at the end of FY 2011. Second, the earnings on investments have decreased, from approximately 5.0% in FY 2006 to less than 1% in FY 2012. Earnings are calculated on the average of beginning and ending Cash & Equivalents.

Cash & Equivalents has decreased from about $5 million in FY 2006 to about $2.7 million in FY 2011, and an estimated $3.3 million at the end of FY 2012. This decrease is the result of negative cash flows in operating and capital-related activities.

Conclusions

The utility has been spending more than it has been receiving from all sources of revenue—sewer rates, taxes, SDCs, and interest earnings. Ashland's economy has also been struggling along with the rest of Nation to recover from a recession. Sewer rate revenues, tax revenues, and SDC revenues have all been reduced from historical trends. In the next chapter, we present a forecast that addresses the above cash shortfall and present a financial plan to pay for higher operating costs and for the planned capital improvements.

CASH FLOW FORECAST

The City plans to make about $11 million in Priority 1 capital improvements over the next 5 years. Inflation will increase the cost of these improvements in 3 to 5 years when they are scheduled to be built to a sum of nearly $13 million. To pay for these improvements, the following financial plan will have the City borrow about $9.478 million of the $13 million, and use cash reserves to pay for the remaining $3.055 million. The City also expects operating costs to increase an average of 6.4% per year. To pay these capital and operating costs, the City will have to increase sewer rates. These changes and a set of economic assumptions shown in Appendix A are reflected in the following 8-year financial forecast.

[1] The City allocates revenues from the Food & Beverage Tax 1% for administration of the tax. The remaining 99% is allocated 79.2% to repayment of the debt service on this bond (80% of the balance), and 19.8% to parks and recreation (20% of the balance).

Table 2 Wastewater Utility Cash Flow Forecast

	Estimate	Forecast								
	2011	2012	2013	2014	2015	2016	2017	2018	2019	Avg Ann
FY ending June 30	2012	2013	2014	2015	2016	2017	2018	2019	2020	% Δ
CASH FLOWS FROM OPERATING ACTIVITIES										
Revenues										
Customer Receipts	3,558,000	3,918,000	3,959,000	4,370,000	4,829,000	5,338,000	5,901,000	6,500,000	6,859,000	8.2%
Miscellaneous	14,250									
Total Revenues	$3,572,250	$3,918,000	$3,959,000	$4,370,000	$4,829,000	$5,338,000	$5,901,000	$6,500,000	$6,859,000	8.2%
Expenditures										
Personal Services	922,420	1,012,000	1,146,000	1,209,000	1,275,000	1,345,000	1,419,000	1,497,000	1,579,000	6.7%
Materials & Services	2,324,366	2,464,000	2,612,000	2,824,000	3,006,000	3,199,000	3,391,000	3,594,000	3,810,000	6.2%
Tax Equivalents	297,670	313,400	316,700	349,600	386,300	427,000	472,100	520,000	548,700	7.6%
Total Expenditures	$3,544,456	$3,789,400	$4,074,700	$4,382,600	$4,667,300	$4,971,000	$5,282,100	$5,611,000	$5,937,700	6.4%
Net Cash From Operating Activities	$27,794	$128,600	($115,700)	($12,600)	$161,700	$367,000	$618,900	$889,000	$921,300	43.8%
CASH FLOWS FROM CAPITAL & RELATED ACTIVITIES										
System Development Charges	56,392	64,000	160,000	224,000	288,000	320,000	324,000	324,100	324,200	
Food & Beverage Tax	1,703,000	1,703,000	1,729,000	1,755,000	1,781,000	1,808,000	1,835,000	1,863,000	1,891,000	
Capital Expenditures	(555,650)	(1,315,000)	(928,000)	(2,852,000)	(3,346,000)	(3,242,000)	(725,000)	(600,000)	(600,000)	
Long-term Debt										
Loan Proceeds	995,000	445,000	1,214,000	541,000	5,987,000	216,000				
Principal	(1,155,318)	(1,176,086)	(1,201,892)	(1,385,937)	(1,424,894)	(1,579,446)	(1,688,598)	(1,744,783)	(1,801,420)	
Interest	(535,836)	(516,718)	(503,985)	(467,588)	(714,681)	(673,106)	(621,627)	(557,441)	(491,004)	
Loan Costs (DEQ)	(4,975)	(7,200)	(13,270)	(15,209)	(15,613)	(15,896)	(14,813)	(13,711)	(12,589)	
Net Cash From Capital Activities	$502,613	($803,004)	$455,853	($2,200,734)	$2,554,812	($3,166,448)	($891,037)	($728,835)	($689,814)	
CASH FLOWS FROM INVESTING ACTIVITIES										
Interest	17,944	29,700	28,300	19,200	21,900	21,700	6,500	6,000	8,000	
Net Cash From Investing Activities	$17,944	$29,700	$28,300	$19,200	$21,900	$21,700	$6,500	$6,000	$8,000	
Net Change In Cash & Equivalents	$548,351	($644,704)	$368,453	($2,194,134)	$2,738,412	($2,777,748)	($265,637)	$166,165	$239,486	
CASH & EQUIVALENTS, Beginning	2,743,939	3,292,290	2,647,586	3,016,039	821,905	3,560,317	782,569	516,932	683,096	
CASH & EQUIVALENTS, Ending	$3,292,290	$2,647,586	$3,016,039	$821,905	$3,560,317	$782,569	$516,932	$683,096	$922,582	

Operating Activities

Revenues increase in the forecast due to growth in the sale of sewer services and due to sewer rate increases. Our forecast shows an increase in sewer rate revenues because of a slow growth rate (between 0.1% and 0.5% per year) and an average annual increase of 10% in the sewer rate (Table 3). These assumptions are shown in Appendix A. The growth rate also affects SDC revenues in Cash Flows From Capital & Related Activities.

Table 3 Current & Forecast Sewer Rates

		Forecast							
Customer Class	Current Rates^	2012 2013	2013 2014	2014 2015	2015 2016	2016 2017	2017 2018	2018 2019	2019 2020
% Rate Increase (per year)		10.00%	10.00%	10.00%	10.00%	10.00%	10.00%	5.00%	5.00%
Residential Rates†									
Base	$18.70	$20.60	$22.70	$25.00	$27.50	$30.30	$33.30	$35.00	$36.80
Commodity ($/ccf)	$2.80	$3.08	$3.39	$3.73	$4.10	$4.51	$4.96	$5.21	$5.47
Commercial Rates									
Base	$19.54	$21.50	$23.70	$26.10	$28.70	$31.60	$34.80	$36.50	$38.30
Commodity ($/ccf)	$3.11	$3.42	$3.76	$4.14	$4.55	$5.01	$5.51	$5.79	$6.08

^ Base Rates are rounded to the nearest $0.10/month. Commodity Rates are rounded to the nearest $0.01.
† Most residential customers pay only the base rate, which includes 400 cubic feet of water consumption.

Operating Expenditures increase at 6.4% per year.

Personal Services include the addition of ½ full time equivalent (FTE) in FY 2013 at a cost of $39,000/year, and a 1.0 FTE in FY 2014 at a cost of $78,000/year. These new FTEs and the existing FTEs are forecast to increase 5.5% per year. Overall, total Personal Services are forecast to increase 6.7% per year.

Materials & Services fluctuate substantially from year to year and have averaged 9.0% per year. This has been due to changes in operations and inflation. In the forecast, we assume inflation will increase all Materials & Services costs 6% per year. Also, these costs will increase because of additional pumping facilities that use more electricity, and for care of trees and other landscaping installed to reduce temperatures below the sewer outfall in Bear Creek—$55,000/year in FY 2015, $13,000/year in FY 2016, and an additional $13,000/year in FY 2017. Inflation and these additional costs results in an average annual increase of 6.2%.

In this forecast, we assume the City will assess the **Tax Equivalent** at 8% of customer receipts. Customer receipts increase due to growth and sewer rate increases. Over the forecast period, this cost is forecast to increase an average of 7.6% per year.

Capital & Related Activities

In 2012 dollars, the City plans to construct approximately $11 million of improvements over the next 5 years (Table 4). Assuming an average 3.5% rate of inflation per year in construction costs, the City will have to raise nearly $13 million over the next 5 years to pay for the improvements (Table 5). Since the City has only about $2 million in cash reserves, most of which is being held to secure existing debts, the City will have to borrow most of the $13 million. Our forecast projects the following loans and terms. The debt service on these and the existing loans is listed in Appendix B.

Membrane Replacement

To complete the immediate need to replace membranes in the existing WWTP at a cost of $2.157 million, the sewer utility used $909,000 from its own Equipment Reserve Fund, and will borrow the remaining $1.248 million from the State of Oregon Department of Environmental Quality Revolving Loan Fund (DEQ SRF). The SRF loan will have a 12-year term, an interest rate of 2.72%, and an administrative fee equal to 0.5% of the outstanding loan balance. The membranes have a life of 10 to 15 years.

Outfall Relocation & Shading

The $2.752 million Outfall Relocation & Shading project will be financed through the SRF for a term of 20 years. This project qualifies for an interest rate of 1% through the U.S. EPA's Sponsorship Program, which funds projects that improve the environment. The City will take up to 6 years to construct this entire project and will have to maintain the trees it plants for at least 3 years after planting. DEQ disburses the SRF loan proceeds as construction is completed and charges interest on the is disbursed amount. After 3 years, the DEQ converts the disbursed amount into a loan that has a 20-year term. This project is forecast to result in two separate 20-year loans over a 6-year construction period.

Oxidation Ditch & Remaining Projects

The remaining projects will be financed in FY 2015 with the use of cash reserves and a loan of approximately $5.75 million. We assume the City will issue a revenue bond with a term of 25 years at 5.0% interest.

A decision is pending by the U.S. EPA as to whether the SRF loan for Membrane Replacement can be combined with the loan for the Outfall & Shading project, which has an interest rate of 1%. If approved, the interest rate for the Membrane project will be reduced from 2.72% to 1%, and the annual debt service would decrease from approximately $123,300 to $110,900—or an annual savings of $12,400.

If the $5.75 million loan for the remaining projects were also to qualify for the Sponsorship Program at a term of 20 years and 1% interest, the combined annual debt service on all SRF loans would be reduced from approximately $352,100 to $306,100—or an annual savings of $46,000.

The amount the City borrows may be more or less depending on actual economic growth and increases in operating costs. Our forecast projects the loans will be obtained at the higher interest rates described above. However, as construction approaches, the choice of lenders and lending options will have to be re-evaluated.

Table 4 Priority 1 Capital Improvements—2012 $'s

FY 2012-2018	Cost (2012 $)	2011 2012	2012 2013	2013 2014	2014 2015	2015 2016	2016 2017	2017 2018
Wastewater Treatment Plant								
Outfall Relocation	856,000		20,000	20,000	20,000	80,000	358,000	358,000
Shading	1,646,000		246,000	320,000	465,000	416,000	110,000	89,000
Backup Portable Lift Station Pump	60,000		60,000					
Membrane System	1,248,000		624,000		624,000			
Oxidation Ditch Shell	4,000,000			224,000	450,000	1,663,000	1,663,000	
RAS Pumps	90,000						90,000	
Wastewater Master Plan Update	125,000						62,500	62,500
Facilities Plan	35,000		35,000					
Total Treatment	$8,060,000	$0	$985,000	$564,000	$1,559,000	$2,159,000	$2,283,500	$509,500
Collection System Improvements								
18" and 24" Parallel Trunkline Along Bear Creek	1,248,000		24,960	99,840	561,600	561,600		
15" Main Along Mountain Ave	118,000		2,360	9,440	106,200			
Oak St. 24" Trunkline	40,000		800	3,200	36,000			
A St 15" Main	522,000		10,440	41,760	94,000		375,800	
12" Main Along Railroad	275,000		5,500	22,000	123,500	124,000		
12" Siskiyou Blvd Main	73,000					1,460	5,840	65,700
Miscellaneous Upgrades	335,000	125,000	125,000	105,000	105,000			
Portable Flow Meters	60,000		60,000					
Storm Water Inflow Study	60,000		60,000					
Total Collection	$2,731,000	$125,000	$289,060	$281,240	$1,026,300	$687,060	$381,640	$65,700
PRIORITY 1 TOTAL – 2012 $'s	$10,791,000	$125,000	$1,274,060	845,240	2,585,300	2,846,060	2,665,140	575,200

Source: Keller & Associates, February 2012

Table 5 Priority 1 Capital Improvements—Adjusted for Inflation

FY ending June 30	Forecast Cost^	2011 2012	2012 2013	2013 2014	2014 2015	2015 2016	2016 2017	2017 2018
Wastewater Treatment Plant								
Membrane System†	$1,248,000†		$624,000		$624,000			
Outfall Relocation	1,047,000		21,000	22,000	23,000	94,000	436,000	$451,000
Shading	1,875,000		261,000	351,000	528,000	489,000	134,000	112,000
Backup Portable Lift Station Pump	64,000		64,000					
Oxidation Ditch Shell	4,735,000			246,000	511,000	1,955,000	2,023,000	
RAS Pumps	109,000						109,000	
Wastewater Master Plan Update	155,000						76,000	79,000
Facilities Plan	37,000		37,000					
Total Treatment	$9,270,000	$0	$1,007,000	$619,000	$1,686,000	$2,538,000	$2,778,000	$642,000
Collection System Improvements								
18" and 24" Parallel Trunkline Along Bear Creek	$1,434,000		$26,000	$110,000	$638,000	$660,000		
15" Main Along Mountain Ave	134,000		3,000	10,000	121,000			
Oak St. 24" Trunkline	46,000		1,000	4,000	41,000			
A St 15" Main	621,000		11,000	46,000	107,000		457,000	
12" Main Along Railroad	316,000		6,000	24,000	140,000	146,000		
12" Siskiyou Blvd Main	92,000					2,000	7,000	83,000
Miscellaneous Upgrades	492,000	125,000	133,000	115,000	119,000			
Portable Flow Meters	64,000		64,000					
Storm Water Inflow Study	64,000		64,000					
Total Collection	$3,263,000	$125,000	$308,000	$309,000	$1,166,000	$808,000	$464,000	$83,000
PRIORITY 1 TOTAL – Adjusted	$12,533,000	$125,000	$1,315,000	$928,000	$2,852,000	$3,346,000	$3,242,000	$725,000

^ Costs are inflated at the rate of 3.5% per year based on *ENR's* long-run average.

† This project cost is fixed by negotiation with the supplier, and does not increase due to inflation.

Other Financing Options

This forecast is based on two identified lenders: the SRF program and the municipal bond market. Two other lenders may be available to the City and the City may be eligible for one other source of cash that it can use for capital improvements.

State of Oregon, Infrastructure Finance Authority (IFA).

The IFA borrows from the municipal bond market and revenues from the State's lottery to relend to municipalities at subsidized interest rates and terms. The interest rate on these loans is lower than Ashland would likely get in the municipal bond market for two reasons: the State has a better credit rating, and it pays all the closing costs of issuing the bonds, which can range from 1% to 2% of the amount borrowed. Loans are up to $10 million for a term not to exceed 25 years or the expected life of the assets being financed, whichever is less. They are controlled by the Oregon Legislature and contain several provisions and limitations that need to be addressed if Ashland proceeds with application to either program. The IFA also has administers 3 grant programs:

- The **Water/Wastewater Financing Program** can provide up to $750,000 in grants for qualifying applicants based on two factors: (1) financial need, which is based on a needs analysis conducted by IFA, and (2) median household income, which must be less that the 2007 statewide average (the latest available data).

- The **Special Public Works Fund** offers grants up to $500,000 (not to exceed 85% of the total project cost) for projects that either create or retain "trade-sector" jobs. Grants are for 85% of the total project cost, up to a maximum of $500,000. The jobs must be in the business of selling goods or services in nationally- or internationally-competitive markets, as determined by IFA. The ratio of project cost to jobs created or retained cannot exceed $5,000 per job.

- The **Community Development Block Grant (CBDG)** is a federal program funded by the U.S. Department of Housing & Urban Development (HUD), which receives an annual appropriation from Congress for projects that benefit low- and moderate-income households. The IFA administers grants for non-metropolitan cities and rural areas. Urban cities such as Ashland receive these grants directly from HUD. In general, all CDBG projects must meet 3 criteria: (1) the project must benefit low- and moderate-income individuals; (2) aid in the prevention or elimination of slums or blight; and (3) meet an urgent need that poses a serious and immediate threat to the health or welfare of the community. These funds are limited, and the City will have to determine whether CDBG funds would be best utilized for sewer or other City projects.

U.S. Economic Development Administration (EDA).

The EDA provides loans and grants for qualified public works projects, including sewer. Congress has been appropriating between $50 million and $70 million annually for EDA's western region (the states of Alaska, California, Hawaii, Idaho, Montana, Oregon, and Washington) for projects that retain or create jobs. The program funds up to 50% of the project cost (the historic maximum awarded to a single project is $2.5 million), and the balance must come from local matching funds. Projects must show a very strong connection to jobs, and competition from the larger states in the 7-state region is very strong. Ashland's planned sewer projects are not likely to meet EDA's criteria unless a large employer depends on a sewer construction project to remain in Ashland or to locate to Ashland.

BPA Energy Smart Industrial Program (ESI).

This program, which was recently extended through September 30, 2013, is sponsored by the Bonneville Power Administration (BPA) to promote energy-efficient industrial improvements. The sewer utility qualifies for the program as an industrial customer of Ashland's Municipal Electric Utility, which is served by the BPA. ESI performs an energy audit of the facility and drafts a report that lists opportunities to reduce electricity consumption. Recommendations include capital improvements and changes in operations and maintenance that would result in reduced electricity consumption. Eligible projects also include previously-planned improvements identified by ESI as energy-efficient. Once the energy-savings measures have been implemented, the City receives a credit of $0.25/kWh saved in the first year of operation—or up to 70% of the total project cost. (For example, last year the City of La Center, Washington, received $215,000 in cash rebates from ESI for upgrading its wastewater treatment plant from an SBR technology to a MBR technology.) ESI does not charge for the audit and report, and the City is under no obligation to accept or implement ESI's recommendations.

Financial Performance

As shown above in Table 2, Cash & Equivalents decrease to less than $1.0 million in FY 2017 through FY 2019. This level of cash reserve is the minimum the City should expect to have for this utility. The sewer system is a complex mechanical system that is susceptible to unexpected breakdowns. A reasonable cash reserve would be $1.0 to $1.5 million. There are two components to a reserve: (a) fluctuations in operating expenditures and for modest capital repair and replacement; and (b) reserves required by lenders.

Oregon statutes do not require a specific level of cash reserves for sewer operating expenditures. Two other government agencies provide some guidance. The Oregon Public Utility Commission (PUC) regulates the user rates of privately-owned utilities, and allows operating reserves equal to 1/12th of operating expenditures plus cash reserves required by lenders. USDA Rural Development—that finances many capital improvement projects for cities with populations of less than 10,000—allows cash reserves equal to 1/4th of operating expenditures plus cash reserves required by lenders.

Assuming operating expenditures in FY 2018 are $5,282,100 as forecast, Ashland's sewer utility should have an operating cash reserve between $440,175 (equal to the Oregon PUC standard) and $1,320,525 (USDA's standard). In addition, the new lenders-DEQ's SRF program and the municipal bond market—will require reserves for the loans. The SRF loans require a reserve equal to 50% of 50% of the annual debt service, which amounts to $39,000 for the Membrane loan and $33,250 for the Effluent Cooling Project. If the City uses its revenue bond authority, the loan from the municipal bond market will require a reserve equal to average annual debt service, or $352,000. If the City uses it is Full Faith and Credit pledge with the municipal bond market the reserve is likely to be zero. The sewer utility's current Full Faith & Credit loans do not have reserve requirements.

The sum of the operating reserves and the loan reserves ranges from $1.09 million (PUC standard) to $1.74 million (USDA standard).

Table 6 shows two additional measures of financial performance. The available revenue to pay annual debt service is Net Cash From Operating Activities and revenue from the Food & Beverage Tax. In FY year 2014 through FY 2017 the available revenue is less than annual debt service. As a result, Cash & Equivalents would decrease if not for the proceeds of new loans. Six years of 10% per year rate increases and increases in the Food & Beverage Tax revenues eventually produce available revenues that equal or exceed annual debt service. The second measure is the debt coverage ratio. This ratio equals:

$$\frac{\text{Net Cash from Operations} + \text{Food \& Beverage Tax} + \text{Interest Earnings}}{\text{Total Annual Debt Service}}$$

When the ratio equals 1, the sum of the revenues just equals annual debt service. A general rule of thumb is that the ratio equal or exceeds 1.25. In other words, the City collects sufficient revenues to pay all of its operating costs, all debt service, and about 25% in excess of debt service. By the end of the forecast period, FY 2019-20, the ratio increases to 1.23.

These measures simply illustrate that Ashland's wastewater utility is taking financial risks because cash reserves dip below $1.0 million in FY 2017 through FY 2019. These risks may be reduced if the economy recovers more rapidly than forecast, inflation increases operating costs less than forecast. Also, the City can either increase sewer rates faster than forecast or delay some capital improvements.

Table 6 Financing Performance Measures

	Estimate	Forecast							
	2011	2012	2013	2014	2015	2016	2017	2018	2019
FY ending June 30	2012	2013	2014	2015	2016	2017	2018	2019	2020
Net Cash from Operating, F&B Tax	$1,730,794	$1,831,600	$1,613,300	$1,742,400	$1,942,700	$2,175,000	$2,453,900	$2,752,000	$2,812,300
Debt Service	(1,691,154)	(1,692,804)	(1,705,877)	(1,853,525)	(2,139,575)	(2,252,552)	(2,310,224)	(2,302,224)	(2,292,424)
Difference	$39,640	$138,796	($92,577)	($111,125)	($196,875)	($77,552)	$143,676	$449,776	$519,876
Coverage--All, w/Tax Revenues	1.02	1.08	0.95	0.94	0.91	0.97	1.06	1.20	1.23

Conclusions

The scheduling of capital improvements and assumed rates of inflation for capital and for operating costs drive the need to increase sewer rates. Economic growth and the demand for sewer services also affect the annual revenues from sewer rates, the City’s primary source of revenue to own and operate the sewer system.

We developed a schedule of capital improvements over the next 5 years that minimizes sewer rate increases for the City's customers. Each year, the City Council must review the sewer utility's financial performance in the previous year to determine whether the scheduled rate increase is adequate to cover costs in the next year. If growth occurs more rapidly, costs increase less rapidly, or capital improvements are delayed, our forecast of rate increases may be reduced. Conversely, if revenues do not increase as rapidly or costs increase more rapidly than forecast, our forecast of rate increases may have to be increased.

APPENDIX A: FORECAST ASSUMPTIONS & RATE INCREASES

History

FY ending June 30	2006 2007	2007 2008	2008 2009	2009 2010	2010 2011	2011 2012
Customer Receipts						
% Δ Y-to-Y	-9.1%	5.5%	4.1%	15.6%	7.3%	5.9%
% Average Annual % Δ	-9.5%	-2.1%	0.0%	3.6%	4.3%	4.5%
Annual Growth Rate	-9.1%	-3.4%	-2.7%	-1.6%	-1.6%	-3.5%
Avg Annual % Increase – month-weight	0.0%	9.20%	6.96%	17.56%	9.13%	9.75%
Cumulative % Increase	-9.10%	-12.17%	-14.50%	-15.90%	-17.28%	-20.16%
SDCs – Number of New EDUs	52	32	19	14	17	14
Food & Beverage Tax						
% Δ Y-to-Y	7.7%	-1.7%	-4.6%	5.9%	0.6%	6.9%
Avg Ann % Δ	7.4%	2.8%	0.3%	1.7%	1.5%	2.3%
Interest Earnings						
% Δ Y-to-Y	5.1%	4.6%	1.3%	1.4%	0.8%	0.6%
Personal Services						
% Δ Y-to-Y	20%	-1%	12%	-4%	-1%	12%
Avg Ann % Δ	18.4%	8.6%	9.6%	6.1%	4.6%	5.8%
New FTEs						
Materials & Services						
% Δ Y-to-Y	66%	5%	-39%	52%	6%	2%
Avg Ann % Δ	50.7%	28.0%	2.2%	12.1%	10.9%	9.4%
Additional M&S						
Tax Equivalents (<=10% Op Rcpts)						
% of Customer Receipts	7.6%	7.5%	9.5%	10.2%	8.1%	8.4%
Avg Ann % Δ	51%	28%	2%	12%	11%	9%
ENR, Annual July	4.0%	3.1%	4.2%	3.3%	3.5%	2.4%
Long Term Average						3.5%

Forecast

FY ending June 30	2011 2012	2012 2013	2013 2014	2014 2015	2015 2016	2016 2017	2017 2018	2018 2019	2019 2020
Annual Growth Rate	-3.5%	0.1%	0.3%	0.4%	0.5%	0.5%	0.5%	0.5%	0.5%
Rate Changes									
July		**10.0%**							
June			**10.0%**	**10.0%**	**10.0%**	**10.0%**	**10.0%**	**5.0%**	**5.0%**
Accumulated from 2011-12	0.00%	10.00%	21.00%	33.10%	46.41%	61.05%	77.16%	86.01%	95.31%
Base Sewer Rate	$18.70	$20.57	$22.63	$24.89	$27.38	$30.12	$33.13	$34.79	$36.53
% Δ Y-to-Y	0.0%	10.0%	10.0%	10.0%	10.0%	10.0%	10.0%	5.0%	5.0%
$/100 cf	$2.80	$3.08	$3.39	$3.73	$4.10	$4.51	$4.96	$5.21	$5.47
% Δ Y-to-Y	0.0%	10.0%	10.1%	10.0%	9.9%	10.0%	10.0%	5.0%	5.0%
Average Single-family Bill	$24.30	$26.73	$29.41	$32.35	$35.59	$39.15	$43.06	$45.22	$47.48
SDCs – Number of New EDUs	14	16	40	56	72	80	81	81	81
SDC per EDU	$4,000	$4,000	$4,000	$4,000	$4,000	$4,000	$4,000	$4,001	$4,002
Total Number of EDUs	15,856	15,872	15,912	15,968	16,040	16,120	16,201	16,282	16,363
% Δ Y-to-Y	5.9%	0.1%	0.3%	0.4%	0.5%	0.5%	0.5%	0.5%	0.5%
Food & Beverage Tax									
% Δ Y-to-Y	6.9%	1.5%	1.5%	1.5%	1.5%	1.5%	1.5%	1.5%	1.5%
Avg Ann % Δ	2.3%								
Interest Earnings									
% Δ Y-to-Y	0.6%	1.0%	1.0%	1.0%	1.0%	1.0%	1.0%	1.0%	1.0%
Personal Services									
% Δ Y-to-Y	12%	5.5%	5.5%	5.5%	5.5%	5.5%	5.5%	5.5%	5.5%
Avg Ann % Δ	5.8%								
New FTEs		39,000	78,000						
Materials & Services									
% Δ Y-to-Y	2%	6.0%	6.0%	6.0%	6.0%	6.0%	6.0%	6.0%	6.0%
Avg Ann % Δ	9.4%								
Additional M&S				55,000	13,000	13,000			
Tax Equivalents (<=10% Op Rcpts)		8.0%	8.0%	8.0%	8.0%	8.0%	8.0%	8.0%	8.0%
% of Customer Receipts	8.4%								
Avg Ann % Δ	9%								

APPENDIX B: DEBT SERVICE SCHEDULE

Existing Loans

	2009 Bank of America				2010 FFC Refunding of DEQ Loan			
FY Ending June 30	Debt Service Principal	Interest	Total Pmt	Loan Balance	Debt Service Principal	Interest	Total Pmt	Loan Balance
2011	($14,587)	($13,984)	($28,571)	$244,306	($1,120,000)	($270,888)	($1,390,888)	$14,320,000
2012	(15,318)	(13,253)	(28,571)	228,988	(1,140,000)	(501,900)	(1,641,900)	13,180,000
2013	(16,086)	(12,485)	(28,571)	212,903	(1,160,000)	(479,100)	(1,639,100)	12,020,000
2014	(16,892)	(11,679)	(28,571)	196,011	(1,185,000)	(444,300)	(1,629,300)	10,835,000
2015	(17,738)	(10,833)	(28,571)	178,273	(1,215,000)	(408,750)	(1,623,750)	9,620,000
2016	(18,627)	(9,944)	(28,571)	159,646	(1,250,000)	(372,300)	(1,622,300)	8,370,000
2017	(19,561)	(9,011)	(28,571)	140,085	(1,280,000)	(334,800)	(1,614,800)	7,090,000
2018	(20,541)	(8,030)	(28,571)	119,544	(1,325,000)	(283,600)	(1,608,600)	5,765,000
2019	(21,570)	(7,001)	(28,571)	97,974	(1,370,000)	(230,600)	(1,600,600)	4,395,000
2020	(22,651)	(5,920)	(28,571)	75,323	(1,415,000)	(175,800)	(1,590,800)	2,980,000
2021	(23,786)	(4,785)	(28,571)	51,537	(1,465,000)	(119,200)	(1,584,200)	1,515,000
2022	(24,978)	(3,593)	(28,571)	26,558	(1,515,000)	(60,600)	(1,575,600)	0
2023	(12,955)	(1,331)	(14,286)	13,604				
2024	(13,604)	(682)	(14,286)	0				
2025								
2026								
2027								
2028								
2029								
2030								
2031								
2032								
2033								
TOTALS	($258,893)	($112,531)	($371,424)		($15,440,000)	($3,681,838)	($19,121,838)	

New Loans

FY Ending June 30	2012 DEQ Loan #1 – Membrane: Principal	Interest	Fees	Total Pmt	Loan Balance	2012 DEQ Loan #2 – Effluent Cooling: Principal	Interest	Fees	Total Pmt	Loan Balance
2012		($16,973)	($3,120)	($20,093)	$ 624,000		($3,710)	($1,855)	($5,565)	$371,000
2013		(16,973)	(3,120)	(20,093)	624,000		(8,160)	(4,080)	(12,240)	816,000
2014		(33,946)	(6,240)	(40,186)	1,248,000		(14,060)	(7,030)	(21,090)	1,406,000
2015	(89,345)	(33,946)	(5,793)	(129,084)	1,158,655	(63,854)	(14,060)	(9,416)	(87,330)	1,883,146
2016	(91,775)	(31,515)	(5,334)	(128,625)	1,066,880	(64,492)	(13,421)	(10,278)	(88,192)	2,055,654
2017	(94,271)	(29,019)	(4,863)	(128,153)	972,609	(65,137)	(12,777)	(11,033)	(88,947)	2,206,516
2018	(96,835)	(26,455)	(4,379)	(127,669)	875,774	(119,721)	(22,065)	(10,434)	(152,220)	2,086,795
2019	(99,469)	(23,821)	(3,882)	(127,172)	776,305	(120,918)	(20,868)	(9,829)	(151,616)	1,965,876
2020	(102,175)	(21,115)	(3,371)	(126,661)	674,130	(122,128)	(19,659)	(9,219)	(151,005)	1,843,749
2021	(104,954)	(18,336)	(2,846)	(126,136)	569,176	(123,349)	(18,437)	(8,602)	(150,388)	1,720,400
2022	(107,809)	(15,482)	(2,307)	(125,597)	461,367	(124,582)	(17,204)	(7,979)	(149,766)	1,595,817
2023	(110,741)	(12,549)	(1,753)	(125,043)	350,626	(125,828)	(15,958)	(7,350)	(149,136)	1,469,989
2024	(113,753)	(9,537)	(1,184)	(124,475)	236,873	(127,087)	(14,700)	(6,715)	(148,501)	1,342,903
2025	(116,847)	(6,443)	(600)	(123,890)	120,026	(128,357)	(13,429)	(6,073)	(147,859)	1,214,545
2026	(120,026)	(3,265)		(123,290)	0	(129,641)	(12,145)	(5,425)	(147,211)	1,084,904
2027						(130,937)	(10,849)	(4,770)	(146,556)	953,967
2028						(132,247)	(9,540)	(4,109)	(145,895)	821,720
2029						(133,569)	(8,217)	(3,441)	(145,227)	688,151
2030						(134,905)	(6,882)	(2,766)	(144,553)	553,246
2031						(136,254)	(5,532)	(2,085)	(143,871)	416,992
2032						(137,617)	(4,170)	(1,397)	(143,183)	279,375
2033						(138,993)	(2,794)	(702)	(142,488)	140,383
TOTALS	($1,248,000)	($299,375)	($48,792)	($1,596,167)		($2,400,000)	($270,041)	($134,586)	($2,662,840)	

FY Ending June 30	2016 Revenue Bond/FF&C -- Oxidation Ditch: Debt Service Principal	Interest	Total Pmt	Loan Balance	TOTAL ALL DEBT: Debt Service Principal	Interest	Fees	Total All Pmts	Balance All Loans
2011					($13,891)	($14,680)		($28,571)	$14,564,306
2012					(1,134,587)	(284,872)	(4,975)	($1,424,434)	14,403,988
2013					(1,155,318)	(535,836)	(7,200)	($1,698,354)	13,672,903
2014					(1,176,086)	(516,718)	(13,270)	($1,706,074)	13,685,011
2015				$5,750,000	(1,201,892)	(503,985)	(15,209)	($1,721,086)	18,590,074
2016		(287,500)	(287,500)	5,750,000	(1,385,937)	(467,588)	(15,613)	($1,869,138)	17,402,180
2017	(120,477)	(287,500)	(407,977)	5,629,523	(1,424,894)	(714,681)	(15,896)	($2,155,471)	16,038,734
2018	(126,500)	(281,476)	(407,977)	5,503,023	(1,579,446)	(673,106)	(14,813)	($2,267,365)	14,350,136
2019	(132,825)	(275,151)	(407,977)	5,370,197	(1,688,598)	(621,627)	(13,711)	($2,323,935)	12,605,353
2020	(139,467)	(268,510)	(407,977)	5,230,731	(1,744,783)	(557,441)	(12,589)	($2,314,814)	10,803,932
2021	(146,440)	(261,537)	(407,977)	5,084,291	(1,801,420)	(491,004)	(11,448)	($2,303,872)	8,940,403
2022	(153,762)	(254,215)	(407,977)	4,930,528	(1,863,529)	(422,295)	(10,286)	($2,296,110)	7,014,272
2023	(161,450)	(246,526)	(407,977)	4,769,078	(1,926,131)	(351,093)	(9,103)	($2,286,328)	6,603,297
2024	(169,523)	(238,454)	(407,977)	4,599,556	(410,974)	(276,365)	(7,899)	($695,238)	6,179,331
2025	(177,999)	(229,978)	(407,977)	4,421,557	(423,966)	(263,373)	(6,673)	($694,012)	5,756,127
2026	(186,899)	(221,078)	(407,977)	4,234,658	(423,204)	(249,850)	(5,425)	($678,478)	5,319,562
2027	(196,244)	(211,733)	(407,977)	4,038,414	(436,565)	(236,488)	(4,770)	($677,823)	4,992,381
2028	(206,056)	(201,921)	(407,977)	3,832,358	(327,181)	(222,582)	(4,109)	($553,872)	4,654,078
2029	(216,359)	(191,618)	(407,977)	3,616,000	(338,303)	(211,460)	(3,441)	($553,204)	4,304,150
2030	(227,177)	(180,800)	(407,977)	3,388,823	(349,928)	(199,835)	(2,766)	($552,529)	3,942,069
2031	(238,535)	(169,441)	(407,977)	3,150,287	(362,082)	(187,681)	(2,085)	($551,848)	3,567,279
2032	(250,462)	(157,514)	(407,977)	2,899,825	(374,789)	(174,974)	(1,397)	($551,160)	3,179,200
2033	(262,985)	(144,991)	(407,977)	2,636,840	(388,079)	(161,684)	(702)	($550,465)	2,777,222
TOTALS	(3,376,146)	(4,254,934)	($7,631,079)		($22,723,039)	($8,618,719)	($183,378)	($31,525,136)	

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