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°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.

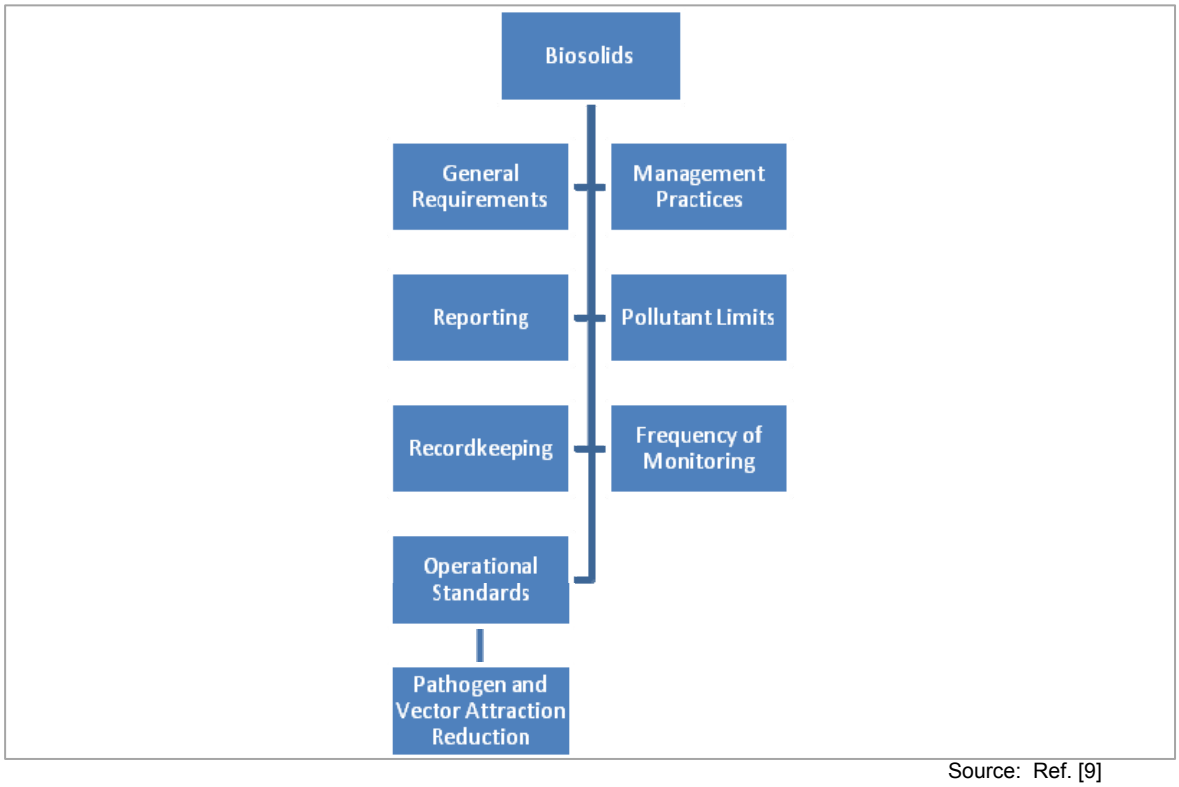
In 1998, the Ashland plant upgrade included facilities to lime stabilize waste activated sludge to meet Class B criteria and to dewater the biosolids using centrifuges prior to land application. The City of Ashland currently dewaters their waste activated sludge using centrifuges, and landfills the dewatered 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:
  - Aerobic Digestion Air Drying
  - Anaerobic Digestion
  - Composting
  - 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.

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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.

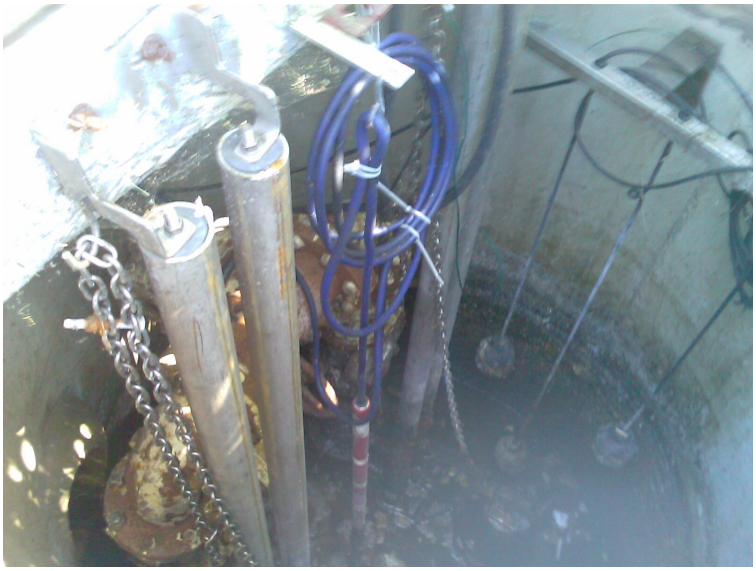
3.2.6 Shamrock Lift Station

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

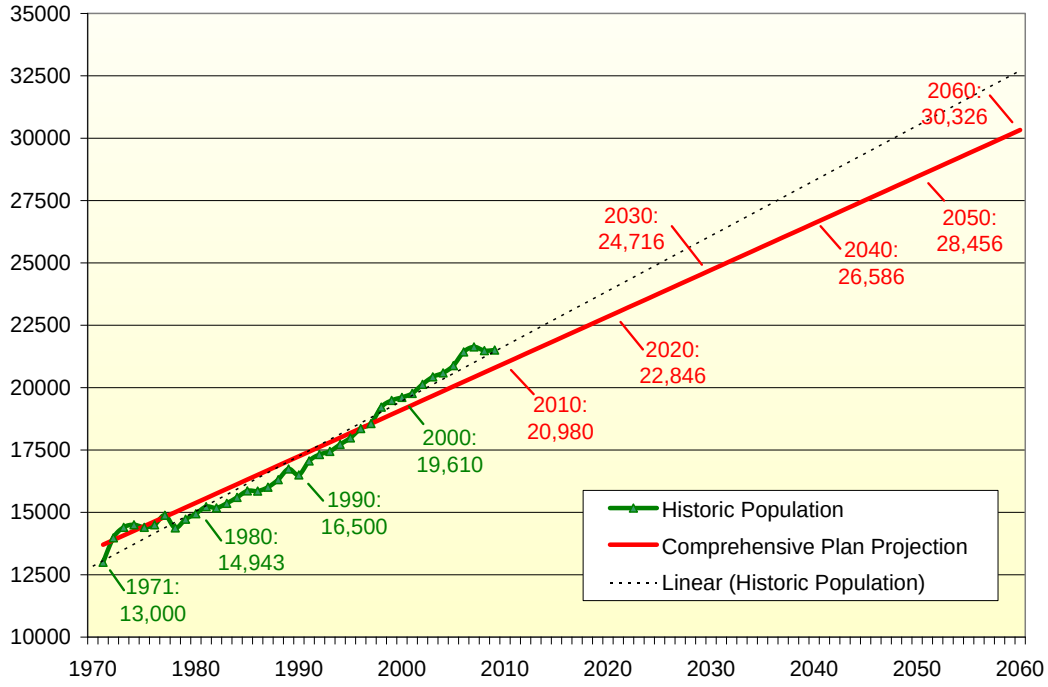
4.1 AREA DEMOGRAPHICS

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

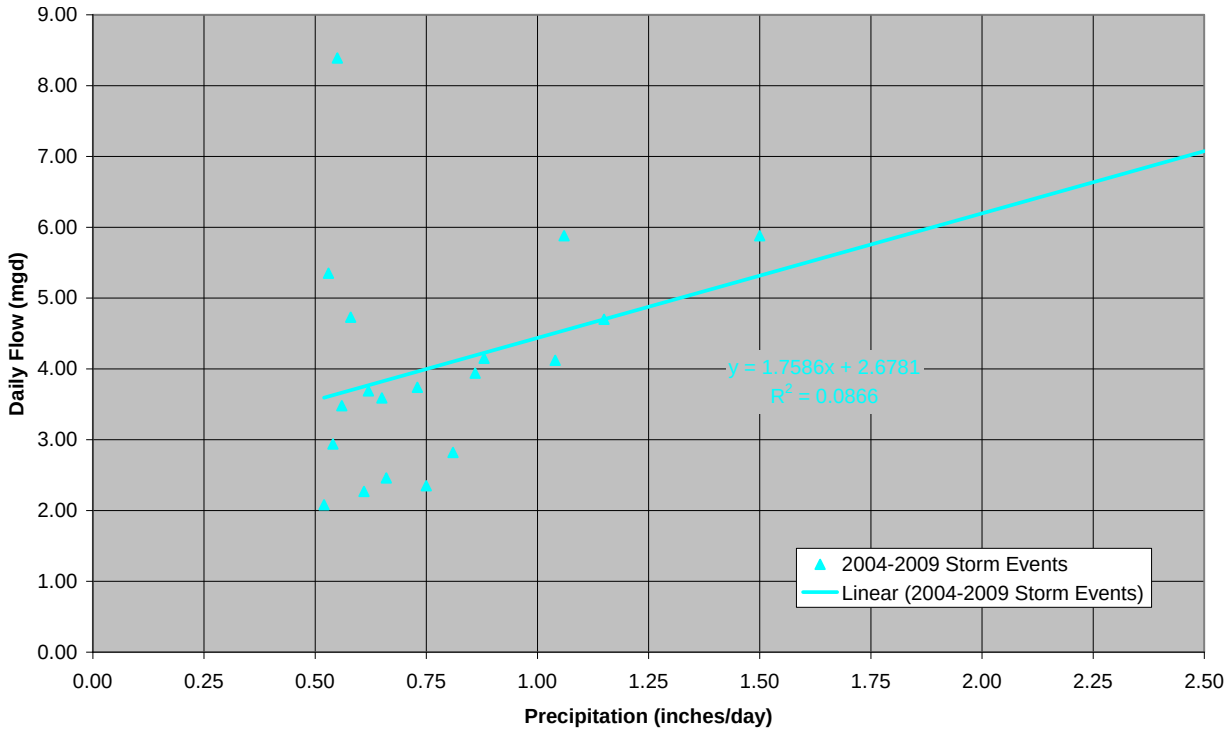
The annual average day flow represents the average flow during the entire year. The peak daily flow represents the highest average day flow during that year. The max monthly flow represents the highest average flow across an entire month for that period. **Dry-Weather** periods are **May-October** and **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

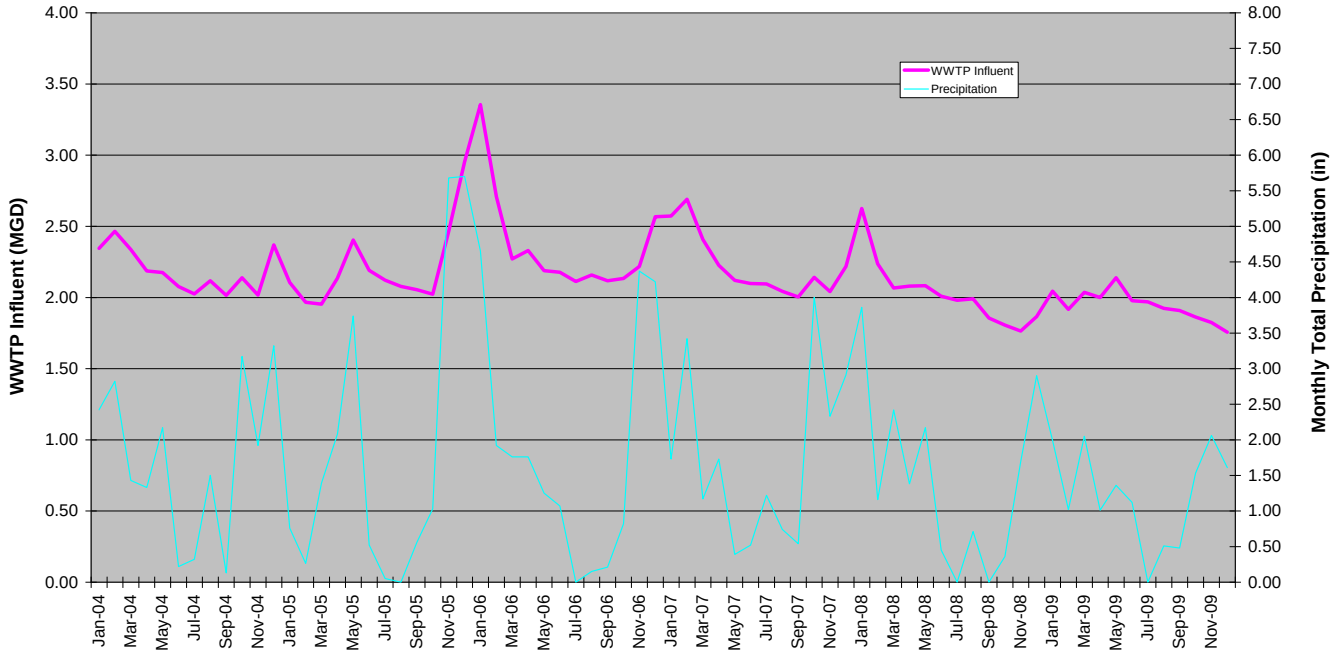


CHART 4.4: Daily WWTP Influent and Precipitation, Oct 2005 – Apr 2006

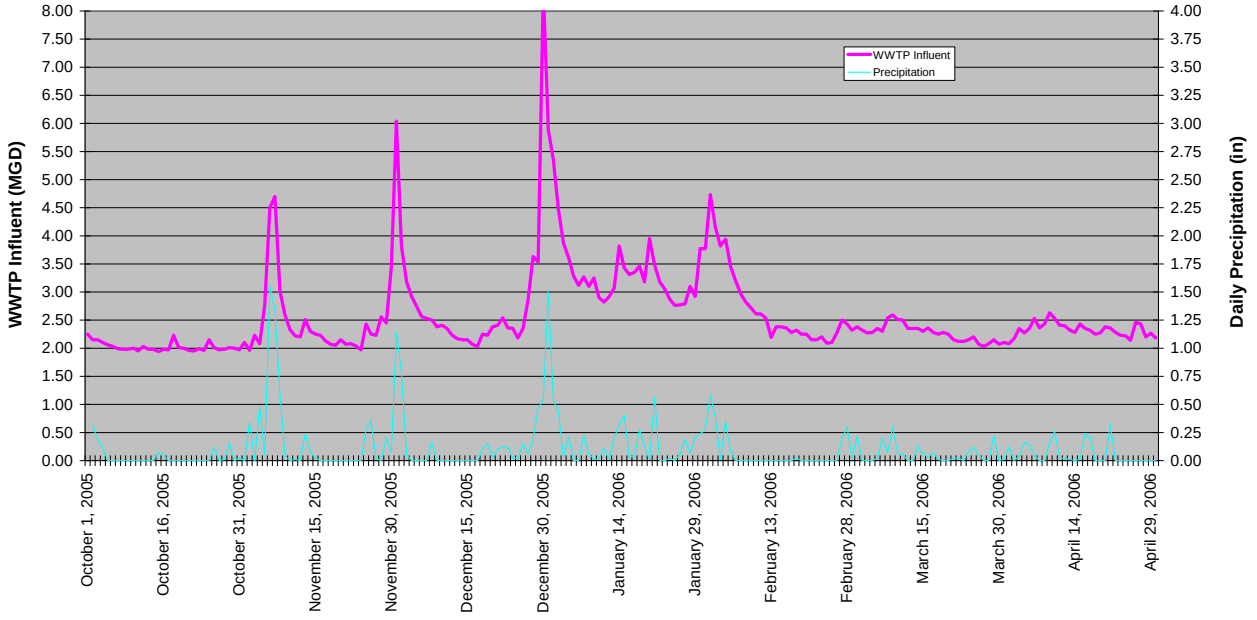


CHART 4.5: Daily WWTP Influent and Precipitation, May 2006 – Apr 2007

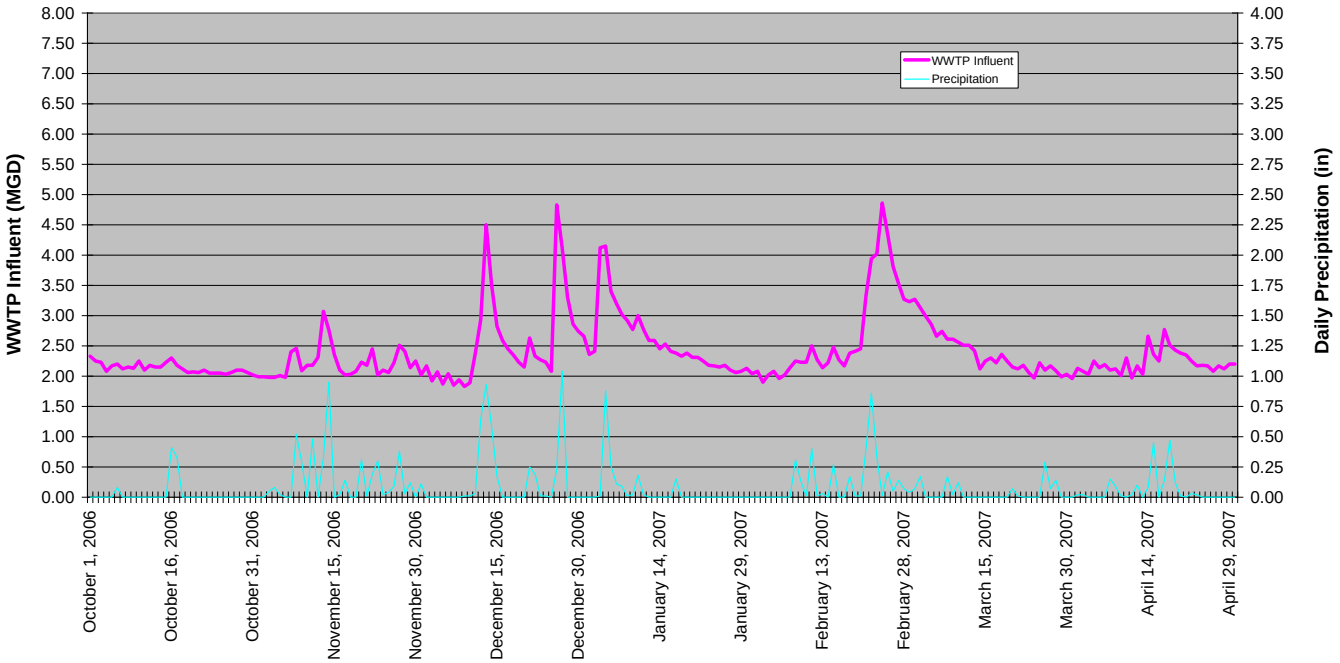


TABLE 4.6: Historical Precipitation, Total Inches (2005-2009)

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.

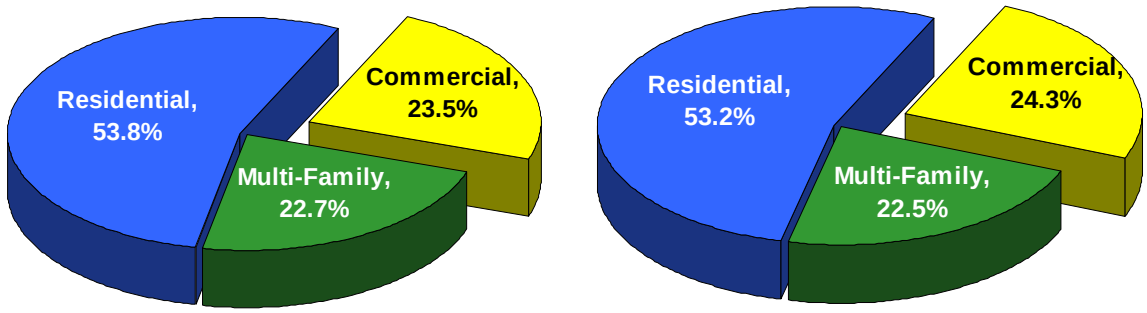
4.5.2 Projected Flow from Future Growth

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)



Winter 2008-2009 **Winter 2009-2010**

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", 4th 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

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)

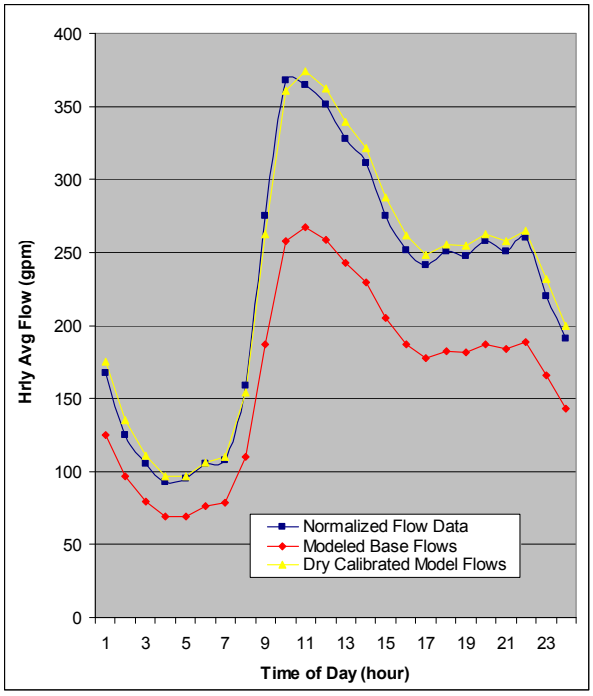
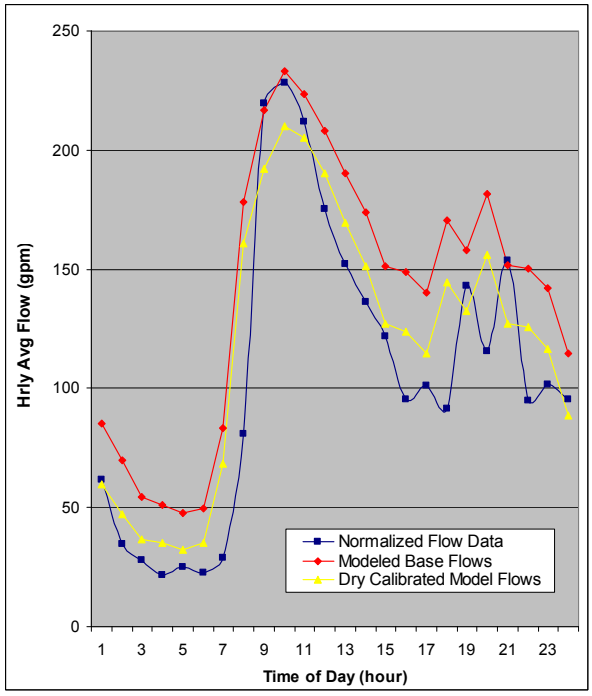


CHART 5.2: System Dry Calibration Site 7 Modeled vs. Observed Flows (MH 10BC-039)



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)

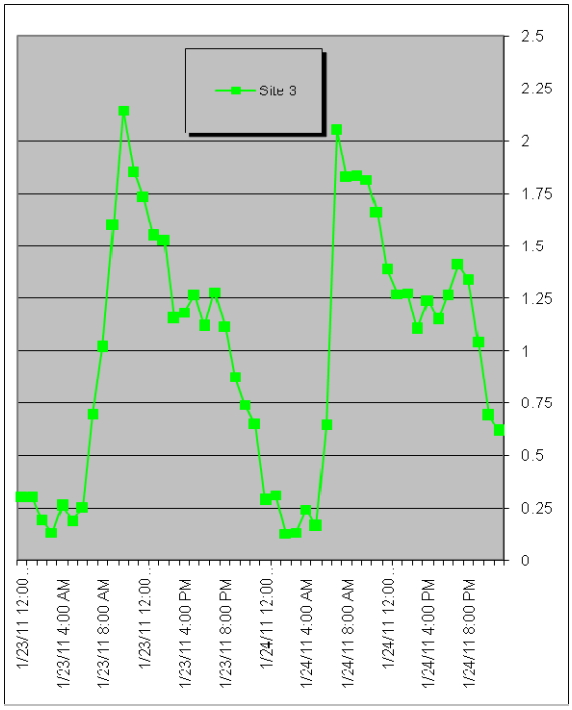
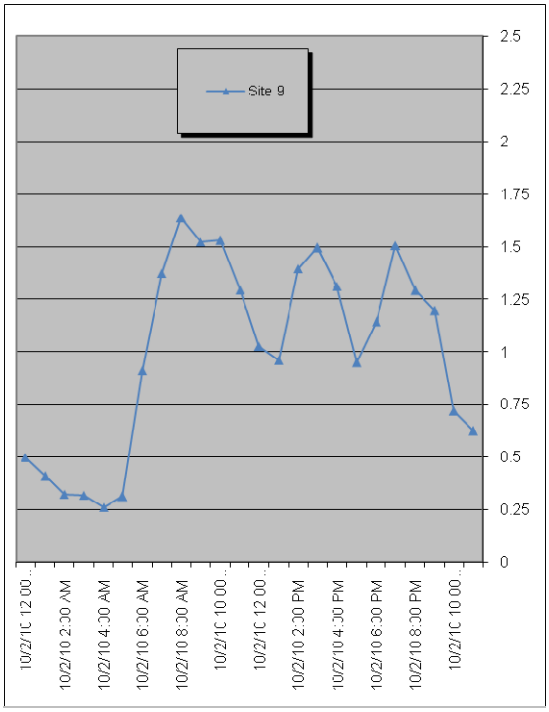


CHART 6.2: Commercial Unit Curve (Site 9 – Manhole 4CC-030)



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.

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.

Alternative 2 – Increase Pumping Capacities & Upsize Trunkline

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.

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.

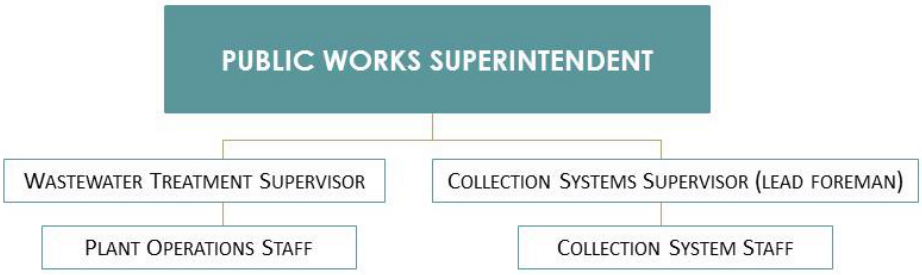
8.2.2 Collection System Staffing

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.