*Final Report*

# Volume 2: Wastewater Collection and Transmission System

# Wastewater Facilities Plan

Prepared for

**City of Woodburn, Oregon**

May 2010

Prepared by

CH2MHILL

**Printed on Recycled and Recyclable Paper**

# Contents

**Appendixes**

## Tables

## Figures

# Acronyms and Abbreviations

| | |
|---|---|
| AACE | American Association of Cost Engineers |
| ABF | average base flow |
| AC | asbestos cement |
| CI | cast iron |
| CIP | Capital Improvement Plan |
| DEQ | Oregon Department of Environmental Quality |
| DI | ductile iron |
| EPA | U.S. Environmental Protection Agency |
| GIS | geographic information system |
| gpad | gallons per acre per day |
| gpd | gallons per day |
| gpm | gallon per minute |
| HGL | hydraulic grade line |
| I/I | infiltration and inflow |
| IRP | Inflow Reduction Program |
| m | meter |
| mgd | million gallons per day |
| mg/L | milligrams per liter |
| MWMC | Metropolitan Wastewater Management Commission |
| N/A | Not applicable |
| NOAA | National Oceanic and Atmospheric Administration |
| NPDES | National Pollutant Discharge Elimination System |
| OAR | Oregon Administrative Rule |
| O&M | operations and maintenance |
| Plan | Inflow Reduction Plan |
| POTW | publicly owned treatment works |

| | |
|---|---|
| ppd | pounds per day |
| PVC | polyvinyl chloride |
| RDII | rainfall-dependent infiltration and inflow |
| SSO | sanitary sewer overflow |
| UGB | urban growth boundary |
| URA | urban reserve area |
| WERF | Water Environment Research Foundation |

SECTION 1

# Introduction

The City of Woodburn provides sanitary sewage collection and treatment for approximately 23,350 people (July 2008 estimate, Portland State University Population Research Center) in a 7.8 square-mile area (2005 Urban Growth Area) in Marion County, Oregon. The wastewater collection and transmission system consists of approximately 87 miles of pipe and eight pump stations. Figure 1-1 shows the collection system and major sewer basins.

## 1.1 Background

The wastewater collection and transmission system has been under continual expansion since its placement in service, beginning in approximately 1910. Woodburn experiences some localized areas of concern in the existing system because of capacity and condition-related deficiencies. Strain on the system is expected to increase as growth occurs and the existing infrastructure moves toward the end of its expected useful life. To guide anticipated collection system investments, system mapping has been improved and a study prepared to evaluate the long-term condition and capacity of the collection and transmission system.

## 1.2 Purpose

This facilities plan provides an assessment of current system characteristics and data availability, documents the process and results of the collection and transmission system condition and capacity assessment, and provides recommendations for maintaining desired level of service for the collection system.

The scope of this evaluation is focused on the main trunk lines in the system, primarily pipes 10 inches or larger, and the pump stations located along the main trunk lines. In some areas, smaller pipes were analyzed where known problem areas were identified.

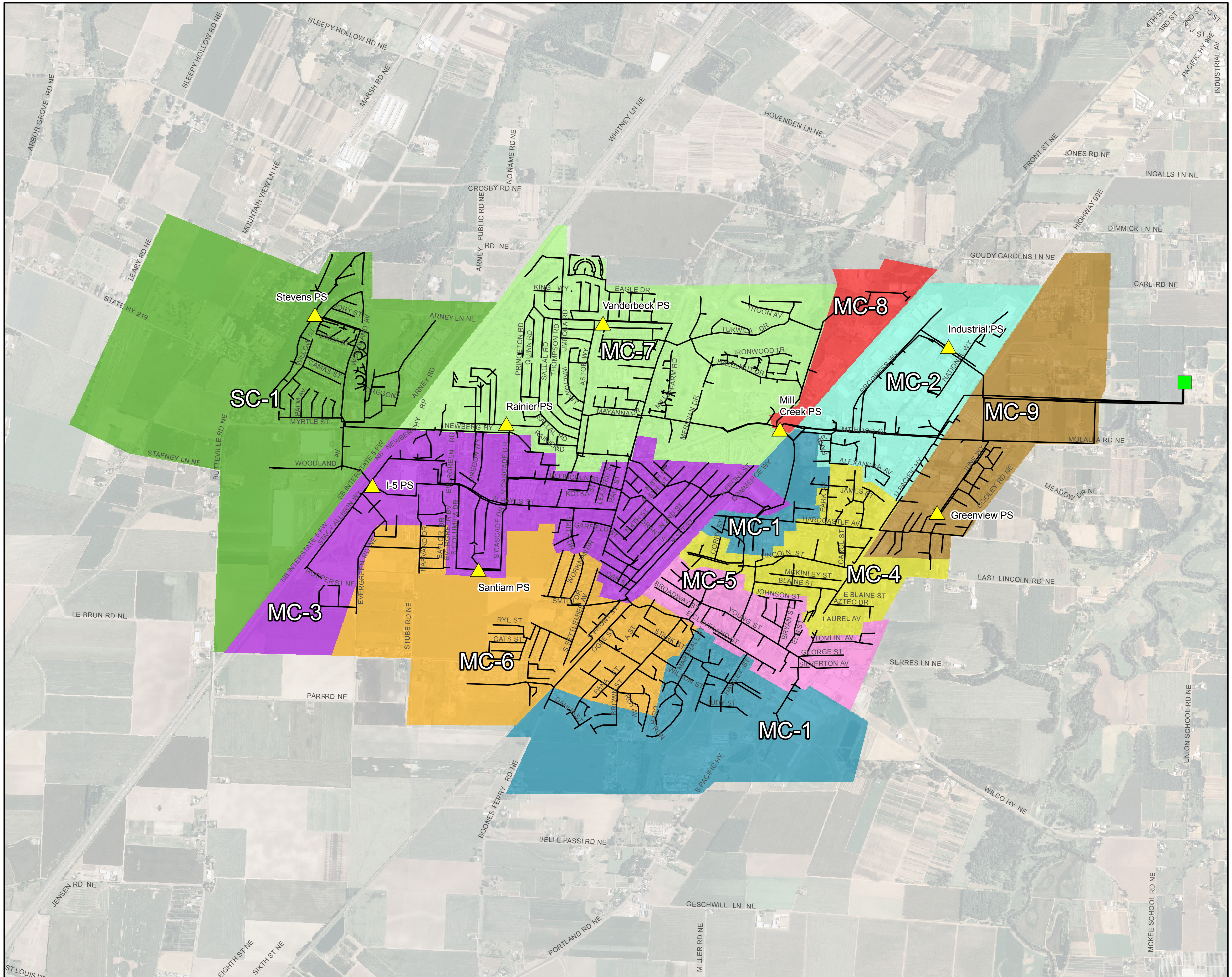
The City of Woodburn collection system consists of about 461,000 feet (87 miles) of pipe, 1,400 manholes, and eight pump stations. Pipe diameters range from 4-inch laterals to 36-inch interceptors. Over 68 percent of the system is 8 inches in diameter or less.

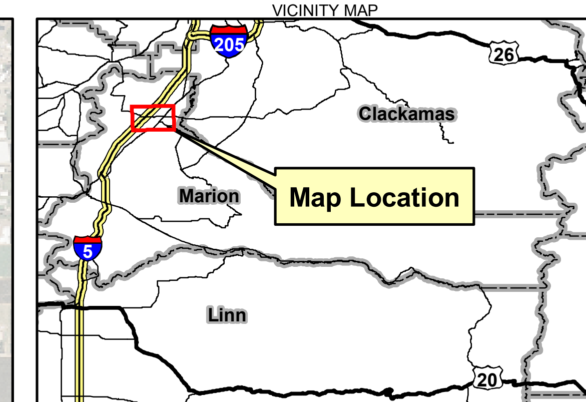
## 1.3 Organization of This Plan

This report is organized into the following sections:

- **Section 1—Introduction**.
- **Section 2—Collection System Mapping:** Summarizes the data collection process for the project geographic information system (GIS) database.

- **Section 3—Condition Assessment:** Describes available data characteristics, documents the condition assessment approach, results, and analyses, and outlines the system deficiencies and status of the conveyance system.
- **Section 4—Infiltration and Inflow Analysis:** Describes the methods used to characterize infiltration and inflow (I/I) contributions to collection system peak design flows
- **Section 5—Hydraulic Capacity Analysis:** Describes the development of a collection system model using a hydraulic modeling system, the criteria for design storms and capacity evaluation, and the collection system capacity analysis.
- **Section 6—Inflow Reduction Plan Evaluation:** Evaluates the City's Inflow Reduction Plan and recommends steps to continue implementation of the Plan.
- **Section 7—Collection System Alternatives Evaluation:** Compares three collection system alternatives: conveyance improvements, rainfall-dependent infiltration and inflow (RDII) reduction, and treatment capacity increases. The three types of improvements are analyzed in different combinations to identify a least-cost solution.
- **Section 8—Recommended Improvements and Next Steps:** Recommends improvements to the collection system and identifies steps for implementation for long-term management.
- **Appendix A:** Contains maps that characterize the collection system by age, material type, diameter, and other attributes.
- **Appendix B:** Contains maps that describe the results of field condition assessments of manholes.
- **Appendix C:** Contains graphs showing regression analyses for each monitoring station.
- **Appendix D:** Contains field notes from CH2M HILL visit to five major pump stations for a general condition assessment.
- **Appendix E:** Contains cost estimates for identified improvements.




LEGEND

- Wastewater Treatment Plant
- Pump Station
- Collection System Pipe

**Basin Name**

- MC-1
- MC-2
- MC-3
- MC-4
- MC-5
- MC-6
- MC-7
- MC-8
- MC-9
- SC-1

N

0 0.25 0.5
Miles

**FIGURE 1-1**
**Major Basins**
*City of Woodburn*



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SECTION 2

# Collection System Mapping

The purpose of this section is to summarize the approach used to map the collection system. The following steps were performed:

1. Collect physical data.
2. Collect condition data.
3. Analyze the data.
4. Establish how the analysis results have been or can be used.

## 2.1 Collect Physical Data

During development of this facilities plan, the City of Woodburn wastewater collection and transmission system mapping was improved by collecting field data and building a GIS database.

Horizontal coordinates and vertical elevation were established for each surveyed manhole. Field data were collected using survey grade accuracy (0.10th in the horizontal and 0.20th the vertical). Pump stations and some force mains were located. For each surveyed pipe, length, size, and depth were established. A description and identifying number were assigned to each pipe or manhole feature.

## 2.2 Collect Condition Data

A condition assessment of the manhole and other field data such as pipe diameter and material were collected in a database. Existing City GIS attribute data for manholes and pipes were updated in the field by survey crews.

## 2.3 Analyze Data

Additional GIS feature classes were created and delineated by CH2M HILL engineering staff. These feature classes were used to help model the sanitary system. The attributes collected for these feature classes are listed in Table 2-1.

TABLE 2-1
Attributes Collected for Geographic Information System Database

| Feature Class | Attributes Collected |
|---|---|
| Sanitary Manhole | Northing, easting, elevation, location, condition, debris, CH Unique ID, and manhole cover condition |
| Sanitary Pipe | Pipe diameter, upstream and downstream invert, pipe material type, CH Unique ID, pipe length |
| Sub-basin_EX | Basin ID, DFMH, acres |
| Major Basins | Basins and acres |

## 2.4 Establish Analysis Use

Electronic files associated with these field survey activities have been used to produce figures and maps throughout this report, and have been provided to City staff for ongoing and future use.

SECTION 3

# Condition Assessment

This section describes the condition assessment performed by CH2M HILL for the City of Woodburn. The assessment consisted of four major steps: establish an approach, collect available system data, conduct an analysis, and provide recommendations based on analysis results.

## 3.1 Establish Approach

The condition assessment approach consisted of six major tasks, as follows:

1. **Compile physical system data (age, material, size).** Use the City of Woodburn CHS database and field sources to collect system data. (CHS is a maintenance management system for the collection system.) Based on these data, create maps showing pipe construction date, material type, and age. Use the maps to identify patterns in physical characteristics; for example, most asbestos concrete pipe was installed before 1985.
2. **Compile observed data (operations and maintenance [O&M] reports, manhole observations, pump station walk-throughs)**. Conduct field surveys, examine City of Woodburn O&M staff observations and records, and perform pump station walkthroughs. Observed data include City-identified frequent maintenance areas, defined by the City as segments of the conveyance system where above-average maintenance has been required. Frequent maintenance areas were added to the GIS database and a map showing these areas was created (see Appendix B, Figure B-7).
3. **Define deficiencies.** This was the first step in the analysis portion of the project. Using regulatory requirements, industry standards, and community goals for level of service, establish a threshold for defining when facilities are deficient. This definition includes specific observation criteria and more general requirements for those parts of the system not directly evaluated as part of this plan.
4. **Examine physical and observed data**. Following the definition of deficiencies, provide closer examination of the physical and observed data compiled in tasks 1 and 2.
5. **Identify improvement needs.** Record potential project areas in need of repair, rehabilitation, or replacement.
6. **Prioritize improvements.** Develop a list of priorities based on severity and other factors (i.e., coincident capacity deficiencies).

## 3.2 Compile Available System Data

The first step in the condition assessment was to compile physical and observed data and create an inventory of collection system conditions. Physical data consisted of pipe age, material type, and size (in diameter and length). Observed data consisted of O&M reports and records, manhole surveys, and pump station walkthrough observations. The primary data source was the City of Woodburn computerized maintenance management software database (referred to as the CHS database). Survey crew observations served as a secondary source.

Data were compiled into a GIS database. Screening procedures were used to identify clearly erroneous or missing data entries. Discrepancies between CHS data and survey observations were identified and corrected through City staff input and CH2M HILL engineering staff best judgment to the extent possible.

Table 3-1 provides a summary of available data. Only limited additional data were collected as part of this study, primarily focused on field observation at manholes. The completeness of existing datasets recorded in electronic databases varies by category. Where only limited data were available, general patterns based on the most complete categories such as age and material type have been used to support recommendations.

TABLE 3-1
Data Availability Summary

| Data Source | Item | Percent of System (%) |
|---|---|---|
| CHS | Pipe Material | 81 |
| CHS | Pipe Age | 13 |
| City Staff | Pipe Age | 93 |
| City Staff | Flow Monitor Locations and Contributing Basins | 100 |
| Facilities Plan | Pipe Material | 21 |
| Facilities Plan | Pipe Diameter | 39 |
| Facilities Plan | Manhole Observations – Corrosion and Debris Levels, Invert and Barrel Conditions | 37* |
| Facilities Plan | Manhole Observations – Cover Conditions | 91 |

*Percent of data collected varies by category. Not all information could be collected at every manhole.

### 3.2.1 Computerized Maintenance Management Software (CHS)

The City's CHS database provided physical information on the pipe construction date, material type, and diameter. Map data from the CHS were placed in the GIS database.

### 3.2.2 Operations and Maintenance Areas of Interest

City operations and maintenance staff provided maps used to supplement the CHS. Among the maps provided were a map of flow monitor locations and contributing basins, a map showing areas of known capacity, design, and slope concerns, and a map showing approximate construction dates for the system (see Appendix B, Figure B-7, Operations and Maintenance Areas of Interest). City staff also provided a list of frequent maintenance areas where additional maintenance is consistently required. Typically, per industry standards, cleaning of pipe segments is only necessary once every 3 to 7 years. However, for a variety of reasons, these frequent-maintenance segments require multiple cleanings annually to remain fully functional and avoid backups. Problem areas were also indicated where grading, capacity, or design elements create additional maintenance and potential concern.

### 3.2.3 Survey Observations

The field survey team collected data on pipe invert elevations, rim elevations, and geospatial coordinates of manholes, establishing the alignment of connecting pipes. Manhole condition observations were made from the ground surface. Survey crews did not enter any manholes. Surveyors documented two types of condition observations, internal and external. External observations comprised locating the manhole with geospatial coordinates and noting the condition of the rim and cover. Internal observations were only conducted when the invert data were needed, and included observation of corrosion, invert condition, and structural condition of the interior of the manhole.

Section 3.3.2 summarizes the manhole analysis results.

## 3.3 Conduct Analysis

The collection system analysis consisted of the following steps:

1. Define system deficiencies.
2. Examine physical data.
3. Examine observed data.
4. Identify improvement needs.
5. Establish prioritization criteria.

The above steps are described in Sections 3.3.1 through 3.3.5.

### 3.3.1 Define System Deficiencies

For purposes of this study, a deficiency is defined as a threshold level for assets that do not meet the City's expectation for level of service. For purposes of this study, we have identified three criteria for defining deficient assets:

- Regulatory requirements
- Condition rating
- Maintenance reports

#### 3.3.1.1 Regulatory Requirements—Pump Station Reliability

Oregon DEQ provides guidance related to pump station reliability and redundancy in *Oregon Standards for Design of Wastewater Pump Stations* (May 2001). This guidance is summarized in Table 3-2. Access and maintenance are critical components of pump station operations that affect reliability and are thus included in Table 3-3.

TABLE 3-2
Oregon Department of Environmental Quality Requirements for Pump Station Reliability and Redundancy*

| Design Consideration | DEQ Guidance |
|---|---|
| Reliability | Design shall be consistent with EPA Class I reliability standards for mechanical and electrical components and alarms |
| Redundancy | Pump system shall consist of multiple pumps, with one spare pump sized for the largest series of same-capacity pumps to provide for system redundancy |
| Access and Maintenance | Structures of adequate size, with interior and exterior clearances to facilitate access for ease of operation and maintenance of all systems |

* Adapted from *Oregon Standards for Design of Wastewater Pump Stations,* Oregon Department of Environmental Quality, May 2001.

**Reliability Standards**. EPA Class I reliability standards were developed in 1974 for facilities discharging near drinking water reservoirs, into shellfish waters, or in proximity to areas used for water contact sports. These standards are documented in *Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability* (EPA, 1974). Criteria apply to all "works that treat[s] the wastewater, including the associated wastewater pumping or lift stations, whether or not the stations are physically a part of the works." Standards related to mechanical and electrical components are summarized in Table 3-3.

TABLE 3-3
U.S. Environmental Protection Agency Class I Reliability Standards*

| Design Consideration | U.S. Environmental Protection Agency Class I Reliability Standard |
|---|---|
| Power supply and electrical equipment location | Two separate and independent sources of electric power shall be provided to the works from either two separate utility substations or from a single substation and a works based generator. At a minimum, backup power source sufficient to operate all vital components, during peak wastewater flow conditions, together with critical lighting and ventilation. Failures resulting from plausible causes, such as fire or flooding, shall be minimized by equipment design and location. |
| Alarms and Annunciators | Alarms and annunciators shall be provided to monitor equipment whose failure could result in a controlled diversion or a violation of the effluent limitations. Treatment works not continuously manned shall have the alarms signals transmitted to a point (e.g., fire station, police station) which is continuously manned. Each alarm and annunciator shall be uniquely identifiable. Test circuits shall be provided verify working order. |
| Lubrication oil system for pumps | If a malfunction of the system can result in a controlled diversion or a violation of the effluent limitations, and the required function cannot be performed by any other means (including manual) then the system shall have backup capability in the number of vital components required to perform the system function. |
| Backup Instrumentation | Instrumentation whose failure could result in a controlled diversion or a violation of the effluent limitations shall be provided with an installed backup sensor and readout. The backup equipment may be of a different type and located at a different point, provided that the same function is performed. No single failure shall result in disabling both sets of parallel instrumentation. |

TABLE 3-3
U.S. Environmental Protection Agency Class I Reliability Standards*

| Design Consideration | U.S. Environmental Protection Agency Class I Reliability Standard |
|---|---|
| Automatic Control | Automatic control systems whose failure could result in a controlled diversion or a violation of the effluent limitations shall be provided with a manual override. Those automatic controls shall have alarms and annunciators to indicate malfunctions which require use of the manual override. The means for detecting the malfunction shall be independent of the automatic control system, such that no single failure will result in disabling both the automatic controls and the alarm and annunciator. |

* Adapted from *Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability*, U.S. Environmental Protection Agency, 1974.

#### 3.3.1.2 Regulatory Requirements—Sanitary Sewer Overflow

A substantial impact to potential future treatment technologies lies in the changing regulations for sanitary sewer overflow (SSO) restrictions. Currently, untreated emergency SSOs have specific limits on the seasonal timing and storm event conditions that create circumstances such that SSO discharges are unavoidable and allowable under Oregon state law. Oregon's current SSO rules are embedded in the bacteria water quality standard, which prohibits overflows from less than a 5-year, 24-hour winter storm, and from less than a 10-year, 24-hour summer storm. Proposed federal rule changes for SSO requirements are currently moving slowly through the review process. More restrictive future federal rules on SSOs will override the Oregon regulations. SSO requirements are a major driver for significant future wet weather improvements to the collection system as well as the treatment facility. Further, even where an SSO may be permitted during specific intensity storm events, there is potential for violation of water quality standards. DEQ is working with EPA to resolve concerns about current DEQ permit language regarding SSOs. The most recent language available continues the allowable overflows.

DEQ has a target deadline of 2010 for SSOs. Volume 1 of this Facilities Plan will document flows and Woodburn will have a plan for addressing SSO requirements.

#### 3.3.1.3 Condition Requirements

As described in the previous section, manholes were rated for several characteristics on a scale of A to D. A rating of C or D indicates that the structure does not meet expected level of service requirements now, or may no longer provide that level of service in the near future.

#### 3.3.1.4 Maintenance Reports

In general, frequent maintenance suggests that significant operating expense is being invested in an ongoing manner to maintain the needed level of service for the facility. For assets that are degraded, the frequent maintenance may be required in perpetuity unless capital improvements are made. We find that any element of the collection system that warrants frequent, repeated repairs or maintenance that significantly exceeds the industry standard is deficient.

## 3.3.2 Examine Physical Data

The City of Woodburn collection system consists of about 461,000 feet of pipe, 1,400 manholes, and eight pump stations. Pipe diameters range from 4-inch laterals to 36-inch interceptors. About 58 percent of the system was 8 inches in diameter. Examination of physical data indicating the age, material, and size of the collection system is described in the following sections.

### 3.3.2.1 Age

Pipe age was determined from date of construction. The CHS database had construction dates for approximately 13 percent of the total system. The rest of the construction dates were determined from extrapolation and maps from City personnel. Extrapolated date ranges for the entire system are shown in Table 3-4, and a system map showing pipe age (from this extrapolated data) is located in Appendix A, Figure A-5.

TABLE 3-4
Extrapolated Pipe Age for City of Woodburn Collection and Transmission System

| Construction Date Range (yrs) | Total Pipe Length (ft) | Percentage of System |
|---|---|---|
| 1995-Present | 111,273 | 24% |
| 1985-1994 | 18,792 | 4% |
| 1975-1984 | 35,697 | 8% |
| 1965-1974 | 123,390 | 27% |
| 1955-1964 | 66,827 | 15% |
| 1954 and Older | 57,005 | 12% |
| Not Assessed | 47,827 | 10% |
| Total | 460,811 | 100% |

### 3.3.2.2 Material

**Data Summary.** The majority of the collection system consists of asbestos cement (AC), concrete, clay pipe, and polyvinyl chloride (PVC).

The City sanitary system has 10 different types of pipe material in addition to pipe that had no material data listed in the GIS database. Almost the entire system downtown and south of downtown subbasins is composed of AC and concrete pipe, while the northeast subbasin is almost exclusively PVC pipe. Ninety-nine percent of all ductile iron (DI) pipe is force main pipe. Material classified as “other” could not be identified by survey efforts. Table 3-5 shows a distribution by material of pipes in the system. A map of the system by material type is located in Appendix A, Figure A-4, Pipe Material.

TABLE 3-5
Pipe Material for City of Woodburn Collection and Transmission System

| Pipe Material | Length (ft) | Percentage |
|---|---|---|
| Not Assessed | 60684 | 12% |
| Not Assessed—Force Main | 5,703 | 2% |
| Asbestos Cement (AC) | 89,678 | 19% |
| AC—Force Main | 532 | <1% |
| Polyvinyl Chloride (PVC) | 134,459 | 29% |
| PVC—Force Main | 6,360 | 1% |
| CONCRETE | 119,437 | 26% |
| CONCRETE—Force Main | 749 | <1% |
| IRON | 747 | <1% |
| Cast Iron (CI) | 1,028 | <1% |
| STEEL | 1,012 | <1% |
| RPM | 3,042 | 1% |
| OTHER | 2,747 | 1% |
| CLAY | 11,102 | 2% |
| BRICK | 3 | <1% |
| Ductile Iron (DI) | 286 | <1% |
| DI—Force Main | 23,244 | 5% |
| **Total** | 460,811 | 100% |

Life Expectancy. Factors affecting the service life of a concrete pipeline include soil, construction practices, and pipe usage. The Northwest Region of the U.S. Bureau of Reclamation states that, “Due to long-term and slow chemical reactions, the life expectancy of concrete pipe is approximately 40 or 50 years.” Typically, the design life for concrete pipe is between 40 and 100 years. Operations and maintenance staff observations confirm this approximate useful life with the observation that concrete installed before 1954 has deteriorating mortar at lateral connections. Approximately 14 percent of the existing system consists of concrete or AC pipe and was constructed before 1954.

Clay pipe is known to have brittle characteristics when life expectancy is exceeded. Nine percent of the pipe in the downtown is clay pipe and installed before 1955.

PVC, which has been installed most recently in the Woodburn system, has a longer life expectancy than concrete. PVC life is estimated to be as long as 100 years. The majority of PVC has been installed within the past 15 to 25 years. Though corrosion is not an issue with PVC pipe, durability when buried can be. In some places throughout the system, localized settling and shape distortion have developed. Reports from City O&M staff confirm that some of these areas have repeated problems and require additional maintenance.

### 3.3.2.3 Size

Table 3-6 shows the pipe length of the collection and transmission system by diameter.

TABLE 3-6
Pipe Length of Collection and Transmission System by Diameter

| | Diameter (in) | Total Pipe Length (ft) | Percentage of Total System Length |
|---|---|---|---|
| | | **Public** | |
| Force Mains | Not Assessed | 770 | <1% |
| Force Mains | 4 | 532 | <1% |
| Force Mains | 6 | 7,774 | 2% |
| Force Mains | 8 | 8,462 | 2% |
| Force Mains | 18 | 8,744 | 2% |
| Force Mains | 24 | 10,306 | 2% |
| Gravity Sewer Lines | Not Assessed | 5578 | 1% |
| Gravity Sewer Lines | 4 | 3954 | 1% |
| Gravity Sewer Lines | 6 | 14,490 | 3% |
| Gravity Sewer Lines | 8 | 264,783 | 57% |
| Gravity Sewer Lines | 10 | 42,814 | 9% |
| Gravity Sewer Lines | 12 | 23,070 | 5% |
| Gravity Sewer Lines | 14 | 1,703 | <1% |
| Gravity Sewer Lines | 15 | 16,280 | 4% |
| Gravity Sewer Lines | 16 | 3,483 | 1% |
| Gravity Sewer Lines | 18 | 9,376 | 2% |
| Gravity Sewer Lines | 24 | 7,222 | 2% |
| Gravity Sewer Lines | 27 | 1,609 | <1% |
| Gravity Sewer Lines | 30 | 52 | <1% |
| Gravity Sewer Lines | 36 | 2,336 | 1% |
| | Public Subtotal | 433,336 | 94% |
| | | **Private** | |
| | Not Assessed | 3,236 | 1% |
| | 4 | 1,499 | <1% |
| | 6 | 8,477 | 2% |
| | 6 | 13,023 | 3% |
| | 10 | 1,242 | <1% |
| | Private Subtotal | 27,477 | 6% |
| | **GRAND TOTAL** | 460,811 | 100% |

### 3.3.3 Examine Observed Data

#### 3.3.3.1 Operations and Maintenance Reports

As stated in Section 3.2.2 above, City operations and maintenance staff were interviewed to obtain information on areas of known capacity, design, and slope concerns, approximate construction dates, and frequent maintenance areas where additional maintenance is consistently required. The City provided a series of maps showing flow monitor locations and contributing basins, areas of known capacity, design, and slope concerns, and approximate construction dates for the system (see Appendix B, Figure B-7, Operations and Maintenance Areas of Interest).

#### 3.3.3.2 Manhole Observations

To determine the condition of the existing sanitary collection and transmission infrastructure, a rating system was first established for the manhole condition observations done by the survey crew. At each manhole observed by the survey crew, five measures were used: cover, barrel, invert conditions, corrosion, and debris levels to assess the internal condition. Infiltration was not used as a measure of condition for the field observation because work was performed during dry weather conditions in the spring and early summer, when infiltration and inflow would not likely be a significant occurrence. Flow monitoring data were used to relate condition assessment ratings for manholes to observed high infiltration basins. Table 3-7 gives the rating (A–D) and associated physical condition.

TABLE 3-7
Manhole Condition Rating System

| | Cover | Barrel | Invert | Corrosion | Debris |
|---|---|---|---|---|---|
| A | Good | Good | Smooth | None | None |
| B | Cracked | Fair | -- | Rough | Light |
| C | Broken | Poor | Damaged | Exposed Aggregate | Medium |
| D | Missing | Failing | -- | Exposed Rebar | Heavy |

These manhole conditions were plotted on a map and locations of interest identified.

#### 3.3.3.3 Pump Station Walkthroughs

A pump station assessment was performed by site inspection via a walkthrough. The following pump stations were inspected:

- Mill Creek
- Rainier
- Stevens
- I-5 (Wal-Mart)
- Santiam

Pump station observations were focused on developing an operational understanding, noting any known deficiencies, and recording historical usage patterns that could be beneficial either for the condition assessment or capacity evaluation for this study. Notes from the field observation are included in Appendix D.

## 3.3.4 Identify Improvement Needs

### 3.3.4.1 Regulatory Compliance

Five major pump stations were visited for a general condition assessment. However, reliability criteria summarized in Tables 3-2 and 3-3 were not explicitly investigated at each pump station. Comparison of the field visit notes to these criteria shows that each pump station is deficient with respect to at least one criterion. Full field visit notes are provided in Appendix D. Key deficiencies found during the field assessment include the following:

- Absence of backup power supply at all pump stations visited except for Mill Creek
- Inundation of the electrical vault at I-5
- Insufficient, unsafe access to the lower level of Mill Creek
- Pump removal mechanism undefined at Mill Creek
- Pump removal requires complete demolition of discharge piping and valves at Santiam

It should be noted that several of the pump stations were constructed before publication of the EPA criteria.

### 3.3.4.2 Condition Requirements

Manholes receiving a rating of "C" or "D" for any of the criteria were considered deficient. Twenty-one manholes received a rating of "C" or "D" for at least one of the criteria. Of these twenty-one, five received a rating of "C" or "D" in more than one criterion and are therefore of the highest concern. A summary of manholes receiving a "C" or "D," and the criteria for which the rating was given, are provided in Table 3-8. Maps showing the complete observations can be viewed in Appendix B, on Figures B-1 through B-6.

TABLE 3-8
Manholes Receiving Rating of C or D

| Manhole ID | Criteria | Rating |
|---|---|---|
| 05-LS10-49 | Invert | C |
| | Corrosion | C |
| | Debris | C |
| 30-WR-201 | Invert | C |
| | Debris | D |
| 28-MC30-15 | Invert | C |
| | Debris | C |
| 29-H-12 | Invert | C |
| | Debris | D |
| 29-H-13 | Invert | C |
| | Debris | C |
| 03-LS10-108 | Debris | D |
| 03-LS10-91 | Debris | D |
| 12-LS10-96 | Invert | C |
| 12-R-28 | Corrosion | D |
| 12-R-43 | Corrosion | C |
| 13-R-24 | Invert | C |
| 21-R-09 | Barrel Out | C |
| 21-R-17 | Corrosion | C |
| 28-MC32-02 | Debris | D |
| 31-WR2-71 | Debris | D |
| 37-C-01-E | Cover | C |
| 37-MC100-03 | Invert | C |
| 39-WR2-45 | Invert | C |
| 39-WR2-49 | Invert | C |
| 39-WR2-51 | Invert | C |
| 39-WR2-60 | Invert | C |

A rating of C or D was most common for the invert condition criteria, while barrel conditions received the least number of C or D ratings. Twelve manholes were rated C or D for invert condition while just one manhole was rated C or D for barrel conditions. Debris is of high concern in nine manholes, and corrosion is of high concern in three.

High-concern manholes are spread throughout the city. There is no apparent relationship between specific criteria for manhole condition and a specific installation date or connecting pipe material.

#### 3.3.4.3 Frequent Maintenance

A number of reported areas, shown on Appendix B, Figure B-7, were identified as deficient as a result of frequent maintenance activities. These areas were identified for a number of reasons, including clogging, line sags, and high infiltration rates resulting from deteriorated pipes or lateral connections.

Table 3-9 below identifies the specific pipe segments shown in Figure B-7.

TABLE 3-9
Pipe Segments Identified as Deficient

| City ID | Length (feet) | Diameter (inches) | Reason |
|---|---|---|---|
| 03-LS10-76 | 221.0 | 8 | Frequent Maintenance |
| 12-R-18 | 228.5 | 8 | Frequent Maintenance |
| 12-R-19 | 404.2 | 8 | Frequent Maintenance |
| 12-R-30 | 192.6 | 8 | Frequent Maintenance |
| 12-R-39 | 53.4 | 8 | Frequent Maintenance |
| 12-R-40 | 338.3 | 8 | Frequent Maintenance |
| 12-R-40 | 454.1 | 8 | Frequent Maintenance |
| 12-R-40 | 159.7 | 8 | Frequent Maintenance |
| 12-R-41 | 12.7 | 8 | Hole in Pipe |
| 12-R-42 | 191.5 | 8 | Frequent Maintenance |
| 12-R-43 | 108.4 | 8 | Frequent Maintenance |
| 12-R-44 | 277.1 | 8 | Frequent Maintenance |
| 12-R-45 | 271.7 | 8 | Frequent Maintenance |
| 19-MC32-05 | 296.7 | 8 | Frequent Maintenance |
| 19-MC32-06 | 311.2 | 8 | Frequent Maintenance |
| 21-L-12 | 639.4 | 8 | Deteriorating |
| 21-R-01 | 42.2 | 4 | Frequent Maintenance |
| 21-R-02 | 252.4 | 8 | Frequent Maintenance |
| 21-R-17 | 106.4 | 8 | Frequent Maintenance |
| 21-R-17 | 111.8 | 8 | Frequent Maintenance |
| 21-R-26 | 122.7 | 8 | Frequent Maintenance |
| 21-WH-27 | 472.5 | 8 | Deteriorating |
| 27-MC24-05 | 447.8 | 8 | Frequent Maintenance |
| 27-MC24-07 | 246.9 | 8 | Frequent Maintenance |

TABLE 3-9
Pipe Segments Identified as Deficient

| City ID | Length (feet) | Diameter (inches) | Reason |
|---|---|---|---|
| 27-MC24-09 | 40.7 | 8 | Frequent Maintenance |
| 28-MC21-61 | 412.8 | 8 | Frequent Maintenance |
| 28-MC21-62 | 428.4 | 8 | Frequent Maintenance |
| 28-MC25-02 | 209.9 | 8 | Frequent Maintenance |
| 28-MC25-06 | 330.0 | 8 | Frequent Maintenance |
| 28-MC25-07 | 350.1 | 8 | Frequent Maintenance |
| 28-MC25-08 | 245.1 | 6 | Frequent Maintenance |
| 29-C-13 | 217.7 | 10 | Capacity and Grade |
| 29-C-14 | 213.9 | 10 | Capacity and Grade |
| 29-C-16 | 300.2 | 8 | Frequent Maintenance |
| 29-C-18 | 73.2 | 10 | Capacity and Grade |
| 29-C-22 | 241.3 | 10 | Capacity and Grade |
| 29-C-28 | 233.7 | 10 | Capacity and Grade |
| 29-C-29 | 39.1 | 10 | Capacity and Grade |
| 29-C-32 | 29.3 | 10 | Capacity and Grade |
| 29-C-35 | 107.6 | 10 | Capacity and Grade |
| 29-C-36 | 498.6 | 10 | Capacity and Grade |
| 29-H-17 | 260.7 | 8 | Frequent Maintenance |
| 29-H-22 | 260.8 | 8 | Frequent Maintenance |
| 29-MC31-10 | 260.3 | 6 | Frequent Maintenance |
| 36-G-22 | 355.5 | 10 | Frequent Maintenance |
| 36-MC16-01 | 51.8 | 12 | Grade and Easement |
| 36-MC16-02 | 232.2 | 10 | Grade and Easement |
| 36-MC21-06 | 43.6 | 12 | Frequent Maintenance |
| 36-MC21-16 | 354.7 | 8 | Frequent Maintenance |
| 36-MC21-16 | 298.6 | 8 | Frequent Maintenance |
| 36-MC21-17 | 344.9 | 8 | Frequent Maintenance |
| 36-MC21-18 | 411.1 | 8 | Frequent Maintenance |
| 36-MC21-63 | 414.4 | 8 | Frequent Maintenance |
| 36-MC21-64 | 80.0 | 8 | Frequent Maintenance |
| 37-MC13-02 | 125.2 | 6 | Frequent Maintenance |
| 37-MC16-03 | 434.1 | 10 | Grade and Easement |

TABLE 3-9
Pipe Segments Identified as Deficient

| City ID | Length (feet) | Diameter (inches) | Reason |
|---|---|---|---|
| 37-MC16-04 | 334.2 | 10 | Grade and Easement |
| 37-MC16-06 | 239.7 | 10 | Frequent Maintenance |
| 37-MC16-26 | 299.0 | 8 | Frequent Maintenance |
| 37-MC16-35 | 351.8 | 8 | Frequent Maintenance |
| 38-MC2-49 | 153.8 | 8 | Frequent Maintenance |
| 38-MC2-55 | 58.5 | 8 | Frequent Maintenance |
| 38-MC2-56 | 336.4 | 8 | Frequent Maintenance |
| 39-WR2-04 | 264.2 | 15 | Grade |
| 39-WR2-05 | 489.6 | 15 | Grade |
| 39-WR2-07 | 1014.1 | 15 | Grade |
| 39-WR2-08 | 180.2 | 15 | Grade |
| 45-LS7-23 | 501.3 | 8 | Frequent Maintenance |
| 45-MC2-37 | 409.7 | 8 | Frequent Maintenance |
| 46-MC2-38 | 273.6 | 8 | Frequent Maintenance |
| 46-MC2-39 | 204.0 | 8 | Frequent Maintenance |
| 46-MC2-40 | 185.5 | 8 | Frequent Maintenance |
| 46-MC2-41 | 317.8 | 8 | Frequent Maintenance |
| 46-MC2-42 | 49.0 | 8 | Frequent Maintenance |
| 46-MC2-43 | 181.3 | 8 | Frequent Maintenance |
| 46-MC2-61 | 104.1 | 8 | Frequent Maintenance |
| 46-MC2-61 | 547.4 | 8 | Frequent Maintenance |
| 47-MC2-25 | 199.7 | 8 | Frequent Maintenance |

### 3.3.5 Determine Prioritization Criteria

The following criteria can be used as “yardsticks” for prioritizing improvements:

- Regulatory Requirements
- Frequent Maintenance
- Long-Term Asset Management

#### 3.3.5.1 Regulatory Requirements

Failure of facilities can potentially cause sanitary sewer overflows to occur. These overflows are regulated and can result in punitive fines if DEQ performs a review and finds deficiencies. Consequently, items that may be found deficient by a regulator form the highest priority for repair. These include pump station and pipe repairs where a failure would result in an overflow or unregulated discharge. Meeting regulatory requirements is the highest priority for condition-related improvements.

#### 3.3.5.2 Frequent Maintenance

Operating cost savings and increased level of service can be realized by reducing or eliminating high frequency maintenance needs for problem areas where a capital improvement can successfully address the problem. Areas of frequent maintenance are the next highest priority for improvements.

#### 3.3.5.3 Long-Term Asset Maintenance

As part of good stewardship of the collection system, it can be anticipated that a certain percentage of the system will require repair or rehabilitation each year. It is difficult to predict far in advance specifically which elements (pipe segments, for example) of the system will deteriorate sufficiently to require repair. Using a risk-based approach to consider the likelihood of failure and its consequences will allow the City to prioritize project improvements. For financial planning purposes, a replacement or rehabilitation allowance for those pipes that exceeded a 75-year installed use life during the planning period might be appropriate.

Effective long-term asset maintenance will result from proactive maintenance of the collection system. Many utilities report buried pipe lasting much longer than its design life. This may prove to be the case for Woodburn as well. Certainly timely repairs and rehabilitation, at lower costs than full replacement, can extend the life of buried pipe significantly. The City’s Inflow Reduction Program, Capital Improvement Program, and ongoing maintenance and repair activities have successfully kept overall infiltration rates relatively low, extending the life of the collection system. As the system continues to age, enhanced maintenance activities, along with eventual significant rehabilitation or replacement will become necessary.

## 3.4 Recommendations

This section outlines recommended improvements for condition and maintenance projects and broader asset management strategies.

### 3.4.1 Condition and Maintenance

Collection system elements deteriorate through use and aging processes. Over time, replacement or rehabilitation become an important part of a capital improvement plan. When possible, improvements resulting from condition or maintenance-related causes are coupled with capacity improvements. However, some projects are needed to maintain the current level of service, and are not directly related to any capacity deficiency. Table 3-10 identifies a number of known condition-related projects, based on the deficiencies and prioritization described in the preceding sections. Appendix B, Figure B-7, is a map showing operations and maintenance areas of interest. Although not identified as individual projects, manholes identified in Table 3-8 should also be considered for repair or replacement on a case-by-case basis. The project grouping in the table below incorporate many of the pipes listed in Table 3-9, which identified frequent maintenance deficiencies.

TABLE 3-10
Collection System Identified Condition or Maintenance Improvements

| Project | Deficiency | In Current Capital Improvement Plan? |
|---|---|---|
| ***Pump Stations and Force Mains*** | | |
| Santiam Pump Station | Reliability | Partial funding |
| Rainier Pump Station | Reliability/Repairs | Partial funding |
| I-5 Pump Station | Reliability | No |
| Stevens Pump Station | Reliability | No |
| Industrial Pump Station | Reliability | No |
| Vanderbeck Pump Station | Reliability | No |
| Greenview Pump Station | Reliability | No |
| ***Gravity Pipelines*** | | |
| Cascade Drive | Infiltration | Yes |
| West Hayes | Infiltration | Yes |
| Cleveland to Wilson Street | Frequent Maintenance | Yes |
| Rainier Road | Frequent Maintenance | Yes |
| North Trunk Rehab | N/A | Yes |
| Carol Street | Sag in line | No |
| Young Street | Clogging and slow flow | No |
| Brown Street | Clogging and slow flow | No |
| Gatch Street | Frequent Maintenance | No |
| MC-7 15-inch PVC | Sag in line | No |
| SC-1 and MC-3 | Design flaw | No |

### 3.4.2 Asset Management

As part of the implementation of best practices for collection and transmission system management and operation, a number of recommendations resulted from the condition assessment:

- An initial condition assessment was conducted, but additional, detailed evaluations are needed. A separate Pump Station Reliability Study is suggested to provide a thorough investigation of all current pump stations operated by the City. Evaluate compliance with DEQ reliability requirements including electrical and alarm systems. Perform repairs as needed to ensure continued compliance.
- Assess staffing and equipment needs for continued implementation of a rigorous maintenance program. Performing sanitary sewer maintenance activities requires highly trained staff and specialized vehicles and equipment. A new tank and vacuum-cleaning

vehicle for pipe maintenance (vactor truck) is needed to maintain existing system level of service.

- Enhance the current routine repair, rehabilitation, and replacement schedule and begin to set aside additional funds for the program. A program level budget may wish to focus on the rehabilitation or limited replacement of the 111,000 feet of sewer lines constructed in 1954 or before.
- An initial condition assessment was conducted, along with some general assessment of risk, but additional, detailed risk assessments are needed to ensure that limited maintenance funds are directed at the highest priority projects. Perform risk assessment of pipes to identify those that exhibit highest vulnerability to failure, either because of location or service area. This ensures that investment is made in the right parts of the system first.
- Perform a pilot program for spot repairs and in-situ repairs to evaluate effectiveness and costs for various repair methods. The City may determine that spot repairs may more cost effectively extend the useful life of the collection sewers than pipe segment major rehabilitation or replacement.

The recommended plan requires the City to continue its proactive maintenance of the collection system. This approach is essential for the following reasons:

- Growth includes a future allowance for RDII, but no increase is assumed.
- Existing RDII must be managed to maintain the selected improvement.

To avoid the potential cost consequences of allowing RDII to increase, a meaningful and adequately funded system maintenance program employing best practices must be an integral part of the recommended plan.

These practices are summarized as follows:

- Repair known structural problems
- Perform source identification activities
- TV inspection
- Smoke testing
- Incorporate field investigation results in capital improvement program projects
- Perform flow monitoring
- Replace or line pipe in selected areas
- Continue system data management mapping and records storage activities

SECTION 4

# Infiltration and Inflow Analysis

The objective of wastewater collection system flow monitoring is to assess total wet weather flow to the City's publicly owned treatment works plant from individual basins and to quantify rainfall-dependent infiltration and inflow (RDII). This section compares collection system flow and rainfall to assess the effectiveness of RDII reduction efforts.

## 4.1 Approach to Monitoring Flow and Rainfall

Collection systems typically show an increase in flow during periods of heavy rain and high groundwater. As part of this analysis, flow monitoring data were used to quantify RDII and to identify its general area of origin. Infiltration was distinguished from inflow by examining the response time of system flow following a rainfall event.

Collection system flow data were available for use in the hydraulic modeling task. Wet season flows were obtained for the periods shown in Table 4-1. Figure 4-1 shows the flow monitor basins and the location of the flow monitors used in this study. (Figures are provided at the end of section.)

Rainfall data from the nearby town of Aurora were obtained and used to develop RDII flows. Aurora is approximately 6 miles northeast of the City of Woodburn.

TABLE 4-1
Flow Monitor Data

| Flow Monitor ID | Monitor Location | Flow Data Collection Periods (m/d/yy) | | | |
|---|---|---|---|---|---|
| 29-H-06 | Harrison Street | | | | 10/3/07 to 10/8/07 |
| 37-MC6 | Queen City | 2/25/02 to 3/13/02 | 1/14/03 to 2/3/03 | 2/19/03 to 3/24/03 | 10/3/07 to 10/23/07 |
| 30-WR8 | Highway 214 Draw | | | | 10/3/07 to 10/23/07 |
| 37-C-01 | Burlingham Ditch | 2/25/02 to 3/14/02 | 1/14/03 to 2/3/03 | 2/19/03 to 3/25/03 | 10/3/07 to 10/23/07 |
| 38-MC2-01A | Goose Creek | | 1/14/03 to 2/3/03 | 2/19/03 to 3/25/03 | |
| 38-MC2-03 | Goose Creek | 2/25/02 to 3/14/02 | | | |
| 38-WR4-01 | Highway 214 | 2/25/02 to 3/14/02 | | | |
| 03-LS10-94 | South Woodland | | 1/14/03 to 2/3/03 | 2/19/03 to 3/24/03 | |
| 29-H-07 | 660 Harrison Street | | | 3/10/03 to 3/25/03 | |

## 4.2 RDII Analysis

Collection systems designed to convey wastewater convey a certain quantity of extraneous flow known as RDII, which enters the system through defects such as cracked or broken pipes, pipe joints, lateral service connections, and possibly through cross-connections with the stormwater system. RDII is the flow entering the sewer system as a direct result of rain. RDII increases total flow volume and peak flow, and consists of two components: infiltration, which slowly percolates into the collection system; and inflow, which reaches a peak shortly after rainfall intensity is greatest and falls off rapidly when rain subsides. Collection system RDII increases the cost of collection and treatment operations and can lead to overloaded pipes and pump stations, which in turn can lead to overflows of raw sewage into the streets or nearby bodies of water, creating a health and environmental hazard.

Because the flow monitors directly measure total flow, RDII may be estimated by subtracting the average base flow (ABF), consisting of sanitary flow and base groundwater infiltration, from the total flow.

The purpose of the RDII analysis is to identify sewer basins that are large contributors of RDII, to quantify these wet weather flows, and to rank the basins for potentially cost-effective RDII reduction. Based on the flow monitoring conducted during February through May 2002 and 2003, regression equations were developed to predict flow based on rainfall and selected dry weather flow patterns. The flow estimates were used in modeling efforts to generate design storm hydrographs in the collection system.

### 4.2.1 Develop Wet Season Average Base Flow

The wet season ABF at each flow monitoring site was developed by selecting several days of flow data from a dry period (no precipitation) during the wet season data collection period. An ABF hydrograph, composed of sanitary flow and base groundwater infiltration, was developed for each location. The composite 24-hour ABF hydrograph was created by determining the minimum flow for each hour from flow monitor data recorded over the dry days selected. The average base flow was used in the calculation of RDII.

### 4.2.2 Estimate Flows

Each flow monitor measures flow from all upstream sources and in some instances is affected by backwater conditions from downstream pipes. To isolate RDII originating from a defined contributing area between two monitors, flows from upstream basins were subtracted from the flows measured at the downstream monitor. As is typical during any flow monitoring program, some of the monitors recorded unreliable data or failed to record at all. To fill these gaps and replace unreliable data, regression-derived data (described below in Section 4.2.3) were used. This correction allowed for RDII estimates for all basins for the calibration and design storms.

### 4.2.3 Perform Regression Analysis

Estimates of RDII are based on a multiple linear regression relationship between rainfall and monitor flow. RDII is flow resulting from rainfall that has entered the collection system over the past hours, days, and weeks. The multiple linear regression analysis generates a mathematical relationship between RDII and multiple periods of past rainfall. Wet weather flow, ABF, and rainfall recorded at the nearest representative rain gage (Aurora, Oregon) were entered into a Microsoft™ Excel spreadsheet. Rainfall during the 15 days (360 hours) before each hourly flow measurement was summed for the following eight periods: 1 hour, 2 to 3 hours, 4 to 6 hours, 7 to 12 hours, 12 hours to 1 day, 1 to 2 days, 4 to 7 days, and 7 to 15 days. The spreadsheet was used to perform multiple linear regression on the correlation between rainfall and the measured flow at each monitor location to determine the regression coefficients for each rainfall summation.

The regression equation takes the following form:

$RDII = C1*Rain_{1hr} + C2*Rain_{3hr}\ldots + C9*Rain_{360hr}$

where C1, C2, … are the regression coefficients

and $Rain_{1hr,}$ $Rain_{3hr,\ldots}$ are the rainfall summations for each time interval

with:

Total Flow = ABF + RDII

Figure 4-2 provides an example of analysis results. The figure shows how flows can be estimated when flow data are not available. The regression equation of the same figure (Figure 4-2) was calibrated using the period of March 5 through March 25. The regression equation uses rainfall data to estimate Goose Creek flows during the January 31 through February 20 period, during which there is no flow monitor data. The analysis was done for each monitoring station (results are included in Appendix C). Once the regression equations are developed and visually checked, flow may be estimated for any rainfall event by applying the equation to precipitation data collected at other time periods or for specific design storm events. The regression equations can be used to generate flow estimates when monitor data are not available, provided rainfall data are available for the period of interest.

Model flows for the Harrison Basin were generated from the Burlingham Ditch regression equation, which is downstream from the Harrison monitor.

The regression equations were used in wastewater modeling efforts to generate design flow inputs to the collection system.

### 4.2.4 Determine Flow Distribution

Each flow monitor basin was divided into a smaller sub-basin to refine the distribution of model flow inputs. The two components of the flow (wet season ABF and RDII) estimated at each monitor location were distributed to selected upstream manholes (hydraulic model nodes) in proportion to the manhole's contributing area. The six flow monitor basins were subdivided into more than 130 sub-basins, with each sub-basin contributing both wet season ABF and RDII to a model manhole assigned to it. Within a given sub-basin, each parcel was assigned its own wet season ABF and RDII. The area of each parcel was classified as either developed or undeveloped, and whether its land use type was residential, commercial, or industrial. The flow contributions from all of the developed parcels within each sub-basin were then summed to give the total sub-basin flow, and subsequently the flow input at each model flow input node (manhole).

### 4.2.5 Assess Accuracy of Flow Estimates

Differences between flows computed from the regression equations and measured flows are a result of one or more of the following:

- *System Operation*. The effects of flow diversions, pump stations, and wet weather bypasses are not consistent from storm to storm and result in potentially irregular system flows under similar rainfall events.
- *Rainfall Distribution*. The regression equations were generated from the closest available rain gage that was thought to best represent the rainfall distributed over the entire monitor basin. However, variability of rainfall volume and intensity is normal across basins, resulting in differences in flow volume and timing.
- *Monitor Data*. It is common to have intermittent problems with flow measurement, particularly because of mismeasurement of velocity. The velocity probe on a flow monitor can be fouled by debris and scum, or backwater effects can change the velocity to depth relationships. In addition, some monitors were not operating during portions of the monitoring period. The regression equations were produced from storm events during periods where the monitor data appeared to be the most reliable. The majority of the monitored data are reasonable and appropriate for the uses of this study.
- *Antecedent Conditions.* RDII predicted by the regression equations will be most accurate when applied to periods when the storm intensity, duration, and antecedent conditions are similar to those used to generate the regression equations. Because each storm event is unique, and the regression equations were developed from historical storm events, there is uncertainty about how response to future storm events may differ. The regression equation approach is an appropriate and accepted tool to make predictions about future conditions.

### 4.2.6 Analyze Design Storms and Capacity Evaluation Criteria

The most critical flow condition for the collection system occurs in response to rainfall events during the wet season when soils are saturated and the collection system's response to rainfall is the most direct. This section describes the process used to develop the peak wet weather flows, including design storm characteristics, the use of the calibrated regression equations to convert design rainfall to RDII, the method employed to distribute flows to modeled pipelines upstream of the monitor locations, and the calibration process used to increase the accuracy of peak flows predicted for the selected design storms.

#### 4.2.6.1 Design Storms

Analysis of the City's collection system was conducted using RDII flows produced by a design storm developed for this project. Based on DEQ requirements, at a minimum the design storm must have a 5-year return interval with a 24-hour precipitation depth. For Woodburn the 5-year, 24-hour frequency rainfall depth is 3.0 inches. This 5-year, 24-hour storm depth was obtained from the National Oceanic and Atmospheric Administration (NOAA) intensity-duration-frequency maps for Oregon. The peak RDII flow and volume are dependent upon the distribution of the 5-year, 24-hour rainfall (3.0 inches) as well as the amount and distribution of rainfall leading up to the 5-year, 24-hour rainfall event. Design storm selection therefore dictates the level of protection against potential overflows that the associated improvements will provide. Because the distribution and antecedent rainfall are not stipulated through regulatory requirements, alternative 5-year design storm distributions were evaluated to generate RDII flows. Each design storm met the 5-year design storm criterion of 3.0 inches in 24-hours. Figure 4-3 presents the 5-year depth-duration curve for durations up to 96 hours in comparison to a 5-year synthetic design storm (using an SCS Type 1A storm distribution), as well as historical rainfall from December 2005 through January 2006.

For purposes of this analysis, 5-year storm events with durations of 24 and 96 hours were compared to assess impacts of rainfall prior to the peak 24-hour period. Each of these design storms meet the DEQ written regulatory criteria for a 5-year, 24-hour winter event. The conditions are as follows:

- *96-hour storm duration.* This condition includes 48 hours of rainfall prior to the start of the 5-year, 24-hour regulatory event and 24 hours of continued rainfall after the peak intensity occurs. The amount of rainfall during this period is consistent with a 5-year frequency 96-hour event, including the 24-hour regulatory design event, and is the most conservative of the conditions modeled.
- *24-hour storm duration.* This condition assumes no prior (antecedent) rainfall to the 24-hour regulatory event and meets the DEQ written regulatory criteria. Because this condition assumes no antecedent conditions, it is the least conservative of the conditions modeled.

The 5-year, 24-hour storm is based on the rainfall distribution of a historical rainfall event, but has been adjusted so that the total rainfall for any duration is equal to a 5-year frequency event. Figure 4-4 shows the 5-year, 24-hour rainfall used with the calibrated multiple-linear regression equation for RDII at each monitor to estimate existing 5-year RDII flows. Adding the wet season dry weather flow to the 5-year, 24-hour RDII flow gives the total 5-year unrouted flow estimate at the monitor location.

Table 4-2 summarizes the RDII for the 5-year, 24-hour, and 5-year, 96-hour storm events.

TABLE 4-2
Five-Year Rainfall-Dependent Infiltration and Inflow Comparison

| | Five-Year, 24-hour SCS Type 1A Storm | | | | |
|---|---|---|---|---|---|
| **Basin** | **Developed Area (acres)** | **Peak Hour RDII (mgd)** | **Peak Hour RDII Rate (gpad)** | **RDII Volume (MG)** | **Return (%)** |
| Burlingham Ditch | 237 | 3.7 | 15,700 | 5.4 | 28.2 |
| Goose Creek | 211 | 4.3 | 20,600 | 16.2 | 94.1 |
| Highway 214 | 655 | 0.6 | 900 | 1.5 | 2.9 |
| Harrison | 140 | 1.1 | 8,100 | 0.7 | 6.2 |
| Woodland | 134 | 0.6 | 4,600 | 1.5 | 14.0 |
| Queen City | 723 | 7.5 | 10,400 | 9.3 | 15.8 |

TABLE 4-2
Five-Year Rainfall-Dependent Infiltration and Inflow Comparison

| | Five-Year, 24-hour SCS Type 1A Storm | | | | |
|---|---|---|---|---|---|
| **Basin** | **Developed Area (acres)** | **Peak Hour RDII (mgd)** | **Peak Hour RDII Rate (gpad)** | **RDII Volume (MG)** | **Return (%)** |
| | **Five-Year, 96-hour Design Storm** | | | | |
| Burlingham Ditch | 237 | 5.5 | 23,400 | 12.7 | 28.2 |
| Goose Creek | 211 | 6.5 | 30,900 | 37.8 | 94.1 |
| Highway 214 | 655 | 0.9 | 1,300 | 3.6 | 2.9 |
| Harrison | 140 | 1.1 | 8,100 | 1.6 | 6.2 |
| Woodland | 134 | 0.9 | 6,600 | 3.6 | 14.0 |
| Queen City | 723 | 9.8 | 13,500 | 21.8 | 15.8 |

The calculated values in Table 4-2 show the impact that antecedent rainfall has on flows in the collection system. Peak RDII rates and flow volume are significantly increased. The table also shows the areas with high levels of RDII. Figure 4-5 shows the monitor basin RDII rates for the 5-year, 24-hour SCS Type 1A design storm. RDII rates greater than 5,000 gallons per acre per day (gpad) are considered high. The RDII rates for Goose Creek and Burlingham Ditch monitor basins are very high. The high return percentage for Goose Creek may indicate a stormwater/sewer cross-connection, such as foundation drains connected to the sanitary collection system.

Initial hydraulic model simulations were conducted using flow inputs derived from the 5-year, 24-hour SCS Type 1A and 5-year, 96-hour rainfall events. Pump stations were assumed to have enough capacity to convey all of the flow that they received so that pipeline capacity restrictions could be identified.

Based on the initial results of the study, the City made the decision to use a 5-year, 24-hour storm to analyze the performance of the wastewater collection system. The comparison indicated that the volume of RDII predicted in the longer event, and the subsequent expected surface overflows, were not consistent with historical observed data or recorded overflow events. For this reason, it was not considered a reasonable representation of expected conditions and eliminated from further analysis.

#### 4.2.6.2 Application of Regression Analysis

RDII is well-correlated with precipitation for the City collection system. As described previously, a mathematical relationship between collection system flows and antecedent precipitation was developed and calibrated using measured precipitation and flow monitoring data from the six flow monitor sites. Using these mathematical equations, an estimate of the RDII resulting from the 5-year, 24-hour rainfall event was made at each monitor location. The figures in Appendix C show the 5-year, 24-hour flow hydrographs generated by the regression equations. Review of the 5-year, 24-hour hydrographs generated from applying the 5-year rainfall resulted in dropping the Harrison monitor flow predictions from the analysis. Model flows for the Harrison basin were generated from the Burlingham Ditch regression equation, which is downstream from the Harrison monitor.

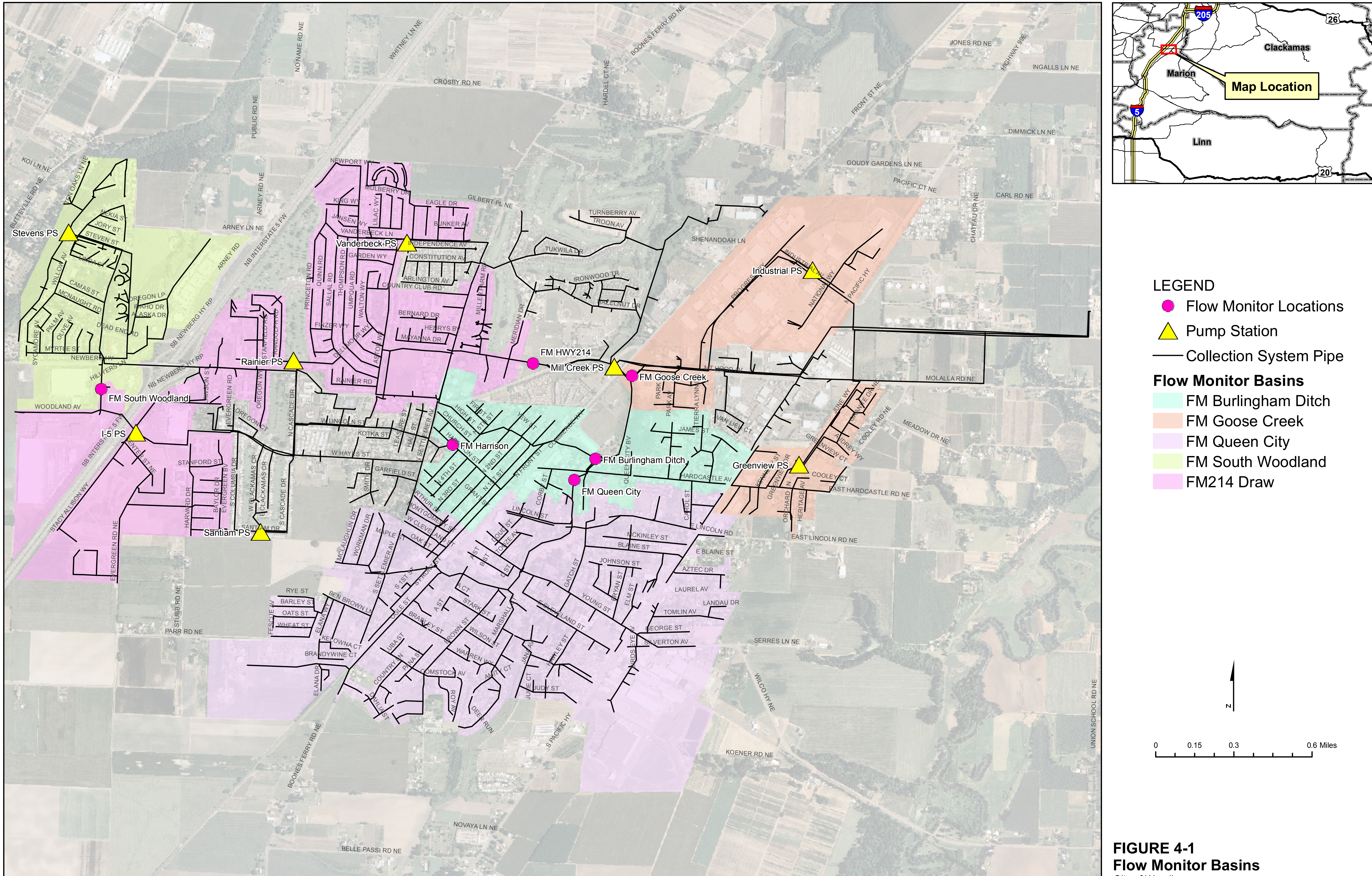



**FIGURE 4-1**
**Flow Monitor Basins**
*City of Woodburn*

**Figure 4-2: Create Regression**
**Flow Monitor: Goose Creek    Rain Gage: Aurora**

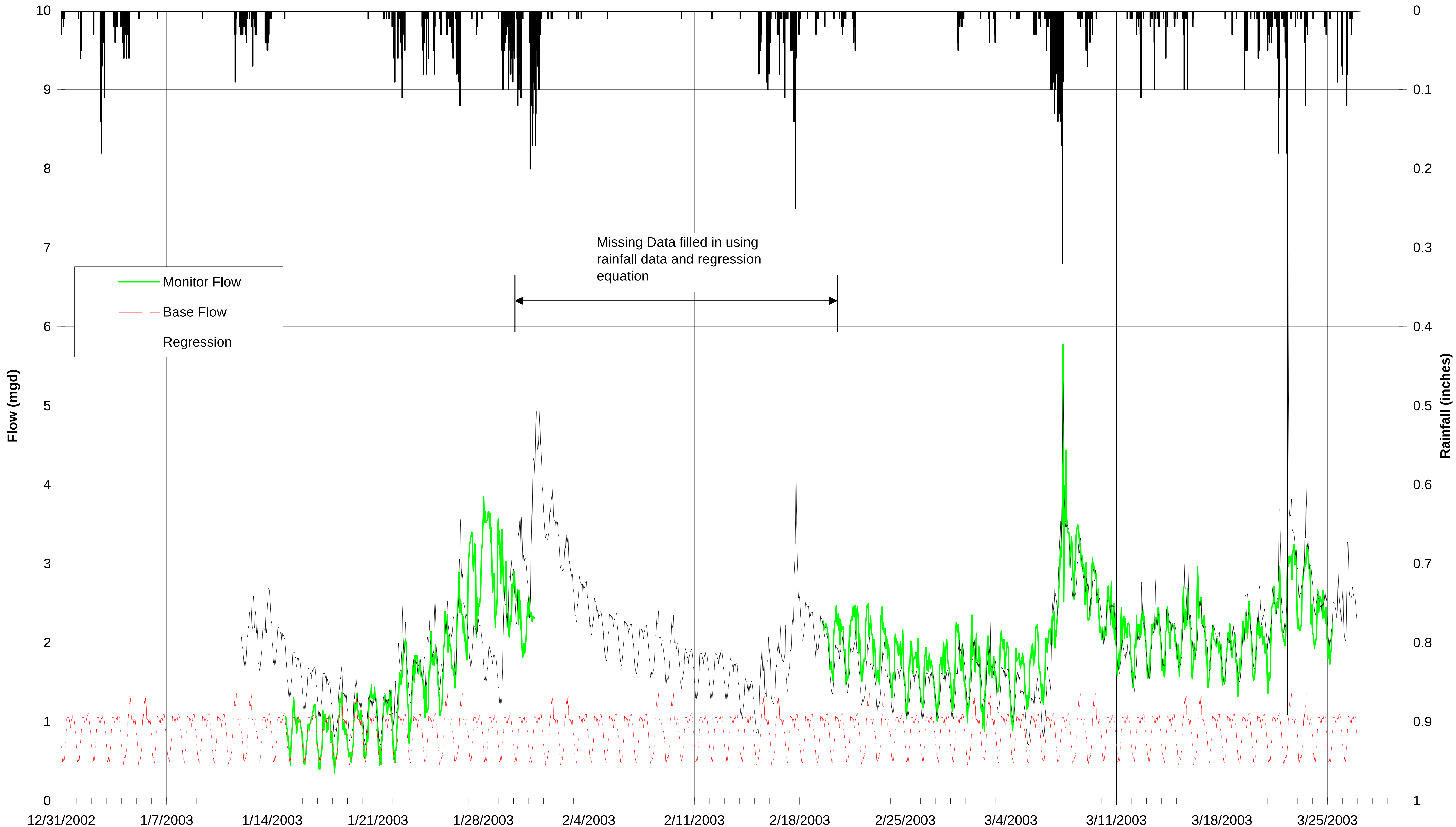
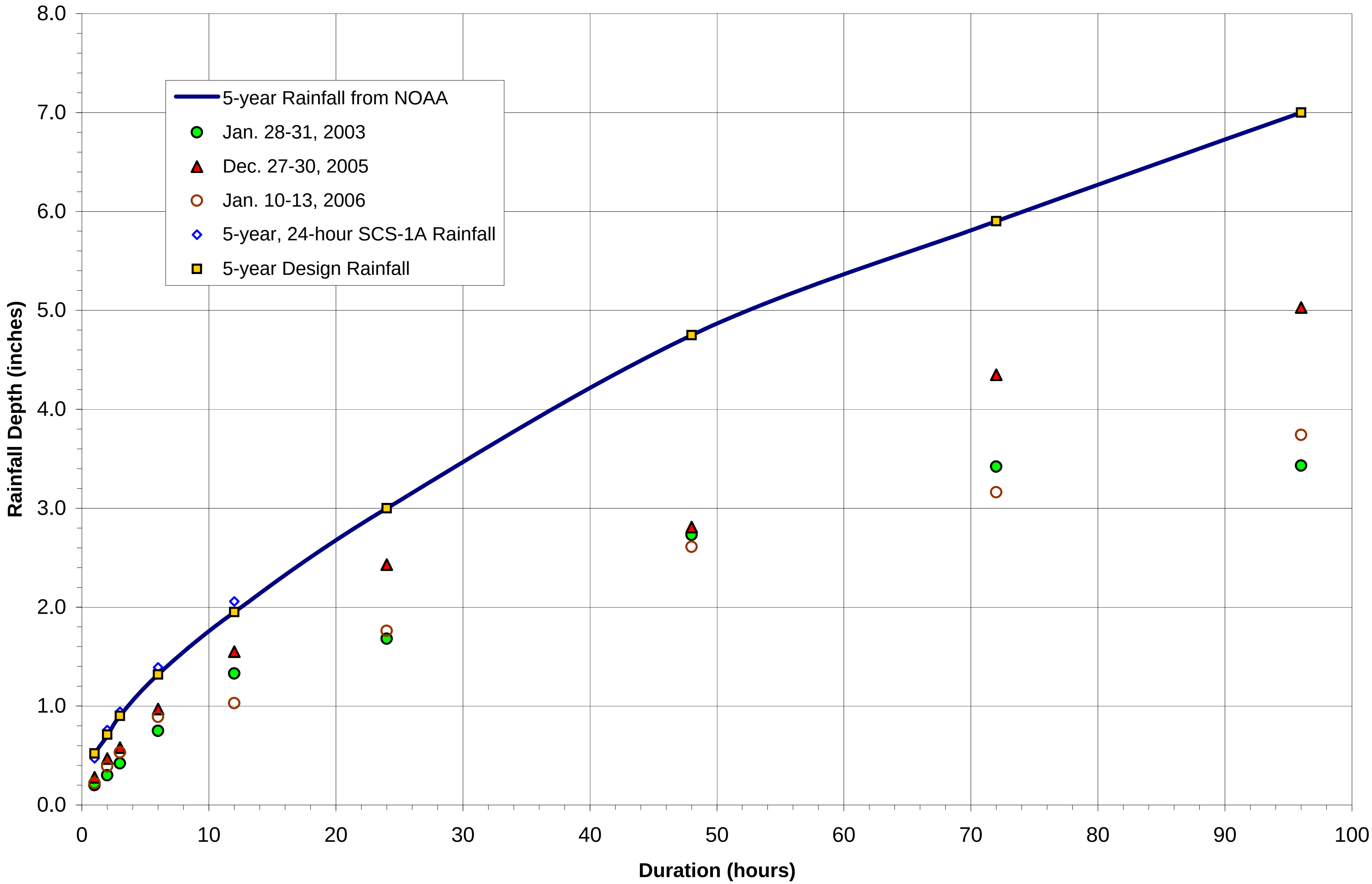
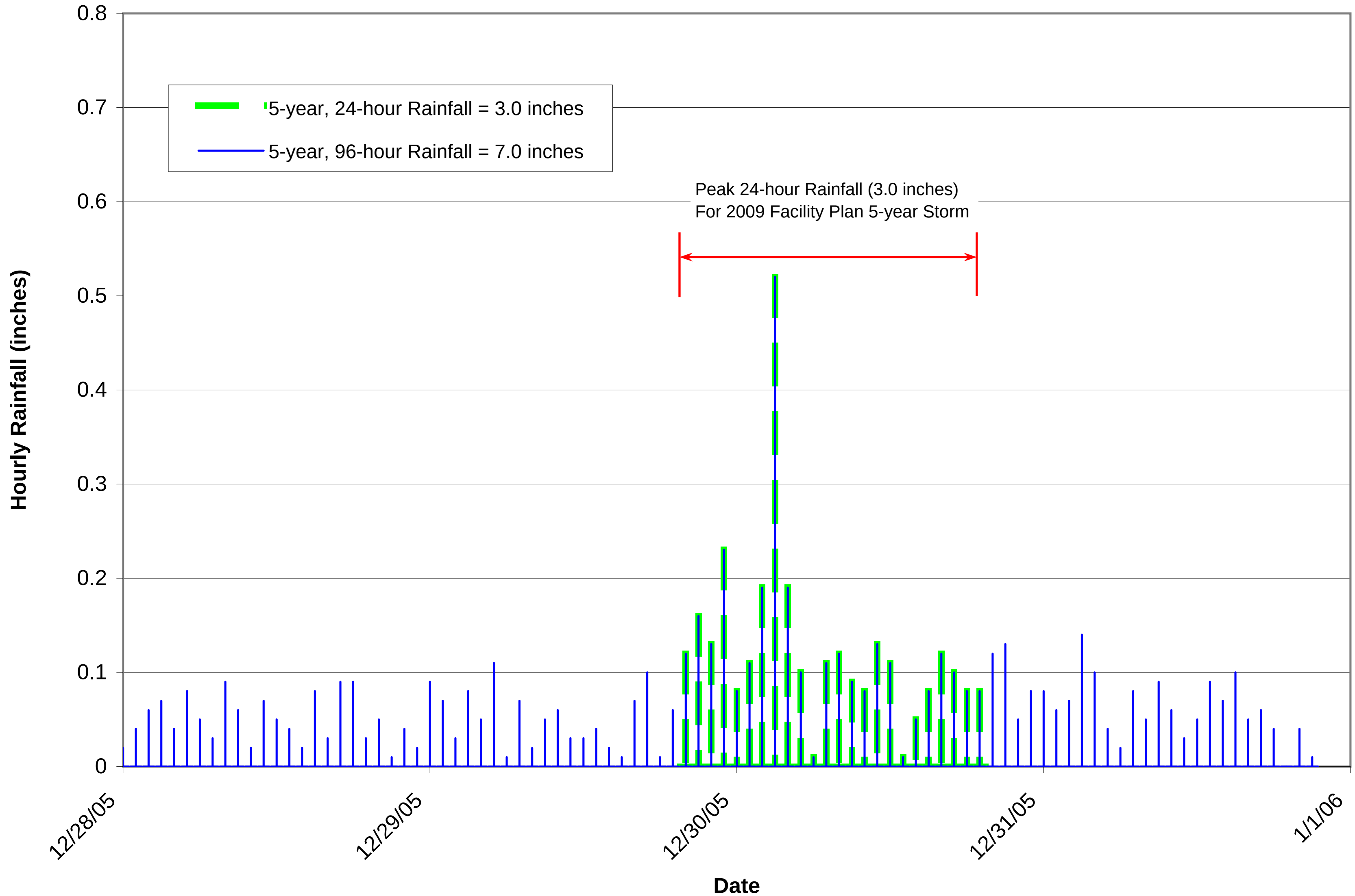


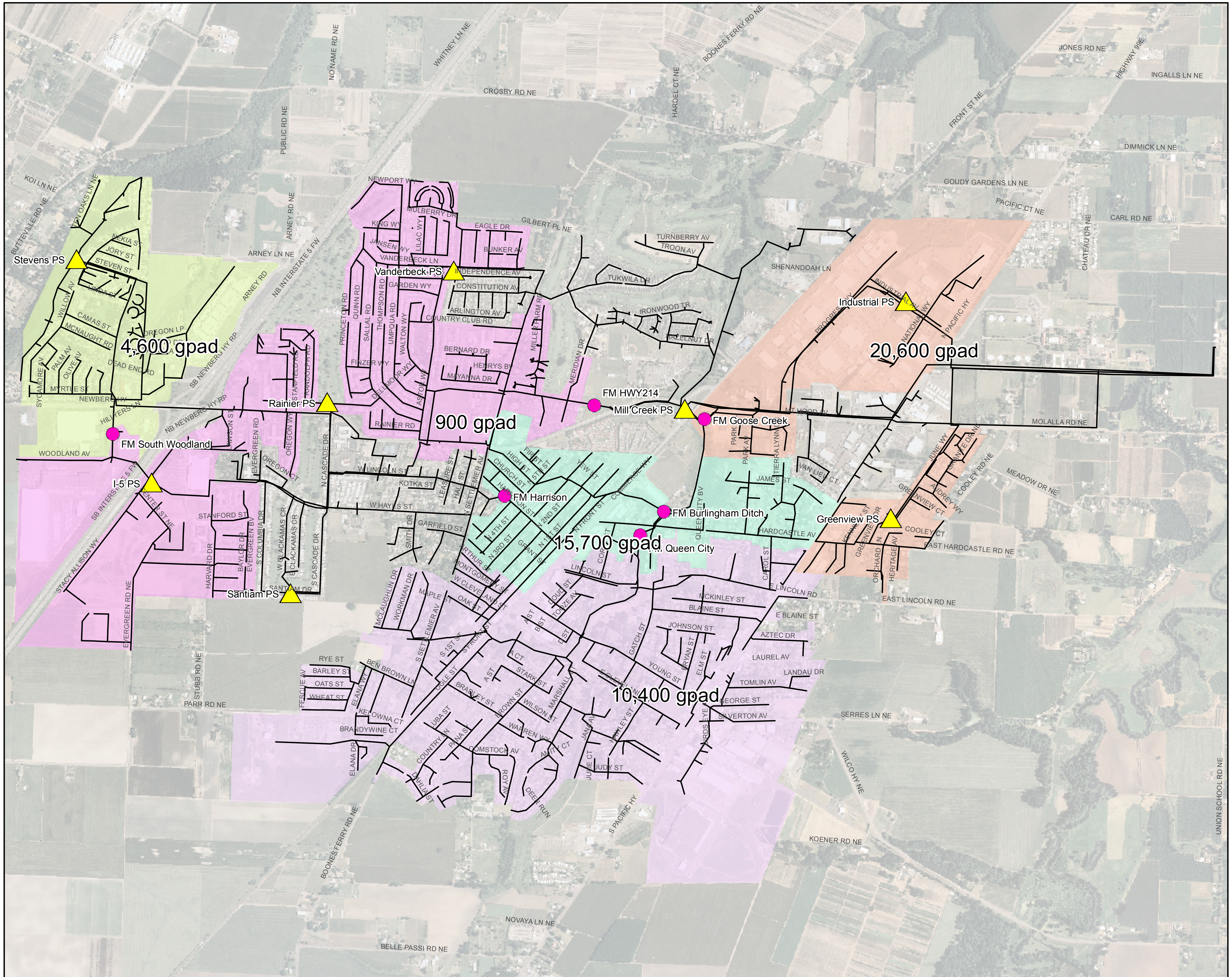

Figure4-2_Regression.xls

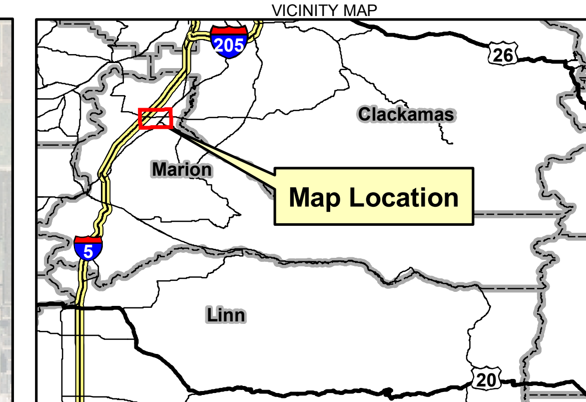
## Figure 4-3: Woodburn 5-year Frequency Rainfall Depth-Duration Curve



## Figure 4-4: Woodburn 5-year Rainfall






LEGEND

- Flow Monitor Locations
- Pump Station
- Collection System Pipe

**Flow Monitor Basins**

- FM Burlingham Ditch
- FM Goose Creek
- FM Queen City
- FM South Woodland
- FM214 Draw

0 0.15 0.3 0.6
Miles

**FIGURE 4-5**
**Monitor Basin RDII Rates**
*City of Woodburn*

CH2MHILL

\\ROSA\PROJ\WOODBURNORCITYOF\367677FP\TASK 8 - COLLECTION SYSTEM\DATA\GIS\MAPFILES\FIGURE4-5_MONITORBASINRDIIRATES.MXD MSULLIV4 3/30/2009 16:59:43

**SECTION 5**

# Hydraulic Capacity Analysis

This section describes the development of a collection system model using a hydraulic modeling system. The criteria for design storms and capacity evaluation, and the collection system capacity analysis, are described.

## 5.1 Model Development

As part of the planning process, a model of the wastewater collection system was developed using the XP-SWMM hydraulic model. The hydraulic model simulates the routing of flow through the collection system. XP-SWMM is a fully dynamic model that can simulate backwater, surcharging, split flows, and looped connections that occur in sewer systems.

The modeling task assists in the identification of areas in the collection and conveyance system where hydraulic capacity deficiencies may exist. The model was refined and calibrated to simulate the existing collection system and to reflect flow monitoring and historical pump station and treatment facility flow data. The calibrated model was used to estimate the 5-year, peak hour wet weather regulatory design flow rates for the existing condition.

### 5.1.1 Model Construction

Data required to build the hydraulic model included the physical characteristics of the system such as pipe invert and rim elevations, pipe material, spatial location of pipes and manholes, and pipe diameters and lengths.

In those areas that have adverse topographic relief, pumping is required to transport wastewater to the gravity portion of the collection system. Flow from the greater portion of the gravity system is pumped to the publicly owned treatment works (POTW) by the Mill Creek Pump Station, although a small portion of the total wastewater flow from the McLaren property goes directly to the POTW. No flows from within the current city boundary are conveyed directly to the plant by gravity. Eight pump stations operate in the collection system, as summarized in Table 5-1.

### 5.1.2 Model Components

The model includes all pipes 10 inches in diameter and greater. Sections of the system that are 8 inches in diameter are included in the model to address key areas of system operation. The following six existing pump stations and their associated force mains were incorporated into the model:

- Mill Creek
- I-5
- Stevens
- Greenview
- Industrial

- Vanderbeck

Two other stations, Santiam and Rainier, were not included in the hydraulic model. These stations discharge into 8-inch gravity lines. The Rainier force main has been identified in the current Capital Improvement Plan for a capacity-related upgrade. The expected flows to these two pump stations were calculated using the developed regression equations and compared to the firm capacity of each station, respectively, to determine whether additional improvements were required.

### 5.1.3 Model Calibration

Flow monitoring data were used not only to develop the regression equations described in Section 4, but also to calibrate the collection system model. The hydraulic model was used to characterize existing RDII volume and to evaluate the conveyance capacity of the collection system under the 5-year, 24-hour wet season storm design criterion specified by DEQ.

Calibration of the collection system model involves adjusting regression equation coefficients as well as flows and hydraulic parameters in the model such that model-predicted flows, depths, and velocities closely match observed data. This step differs from the initial development of the regression equations in that it compares peak flows from the hydraulic model to measured flows after the flow data from the regression analysis is distributed to multiple manholes upstream of the monitor location. In this way, hydraulic routing of the flow in the collection system is tracked. Calibration was performed based on model predicted versus monitored flows at the six monitor locations using different rainfall periods than were used for regression equation development.

Model calibration consisted of the following iterative procedure:

- Modify model flow input hydrographs by adjusting the basin regression model parameters,
- Distribute flows to upstream manholes in the hydraulic model,
- Route the flows through the hydraulic model,
- Compare hydrograph shapes, peaks, and volumes at flow monitor to see if they matched those measured at each location during the flow monitoring period.

The hydraulic model was run for several storms that occurred during the monitoring period to verify that the routed flows at the monitor locations were approximately the same as the sum of the dry weather and RDII flows that had been calibrated outside of the hydraulic model. Model calibration included quality control checks of the hydraulic model flow inputs, pipe and manhole connectivity, pump station configuration, model stability, and comparison of results to flow meter values. The regression model flow estimates are most accurate within the range of the rainfall and flow for which they are calibrated, and therefore it is desirable to capture a wide a range of rainfall events during the flow monitoring period. Monitoring periods selected for use in the regression analysis were chosen to capture a variety of events.

The figures in Appendix C compare modeled flows with monitored data at each flow monitor during the model calibration period.

TABLE 5-1
Pump Station Inventory

| Pump Station Name | Pump Station ID | Installation/ Construction Date | No. of Pumps | Type | Base Elevation | Rim Elevation | Pumps Off (Elev. ft) | Lead Pump On (Elev. ft) | Lag 1 Pump On (Elev. ft) | Lag 2 Pump On (Elev. ft) | Wet Well Shape | Wet Well Diameter | Capacity Pump 1 (gpm) | Capacity Pump 2 (gpm) | Firm Capacity[a] (gpm) | Wet Well Capacity (gal) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| I-5 | 11-PS10 | 1992 | 2 | Duplex Submersible | 147.5 | 180.0 | 149.0 | 152.0 | 153.0 | NA | Circular | 11-ft | 1,215 | 1,215 | 1,215 | 23,106 |
| Santiam | 19-LS6 | 1966 | 2 | Duplex Submersible | 168.0 | 180.4 | 168.3 | 170.8 | 171.3 | NA | Circular | 4-ft | 136 | 112 | 112 | 1,166 |
| Stevens | 04-LS5 | 1969 | 2 | Duplex Submersible | 148.8 | 163.5 | 148.8 | 150.3 | 150.8 | NA | Circular | 6-ft | 215 | 191 | 191 | 3,112 |
| Greenview | 45-PS7 | 2005 | 2 | Duplex Submersible | 155.9 | 183.7 | 154.4 | 158.5 | 159.0 | NA | Circular | 6-ft | 406 | ?? | 406 | 5,880 |
| Industrial | 47-LS9 | 2005 | 2 | Duplex Submersible | 150.0 | 180.8 | 152.6 | 154.4 | 154.9 | NA | Circular | 6-ft | 550 | 550 | 550 | 6,516 |
| Mill Creek | 38-PS1 | 1979/1998/ 2001/2008 | 4 | drywell/wet well | 127.3 | 148.0 | 129.0 | 131.9 | 132.3 | 132.4 | Semi-Circular | -- | 7,200 | 4,600 | 9,800[b] | 23,800 |
| Rainier | 21-LS3 | 1963/1999 | 2 | Duplex Submersible | 166.2 | 182.4 | 166.2 | 170.2 | 171.2 | NA | Circular | 8-ft | 300 | 300 | 300 | 6,092 |
| Vanderbeck | 31-LS1 | 2000 | 2 | Duplex Submersible | 159.5 | 182.5 | 164.0 | 167.4 | 168.0 | NA | Circular | 10-ft | 700 | 700 | 700 | 13,514 |

[a]Total capacity with largest pump out of service.

[b]The Mill Creek Pump Station (MCPS) has four pumps. Three pumps are electric motor driven pumps permanently installed inside the pump station. The fourth pump is an engine-driven pump temporarily installed outside the pump station building. All of the pumps originally installed when the pump station was constructed have been replaced with new pumps differing in capacity from the original installation. Woodburn has conducted pumping tests to verify the actual capacity of the pumps and the hydraulic characteristics of the force mains. Based on those tests, the firm capacity of the MCPS with the largest pump out of service in normal operations is 9,800 gpm, using only the single 24-inch force main. In an emergency, the two smaller pumps plus the engine-driven pump could be operated using both the 18-inch and 24-inch force mains in parallel. In an emergency, the MCPS could pump at a rate as a high as 11,400 gpm with the largest pump out of service.

The ability to closely predict measured flows during large storm events indicates that the model has achieved an appropriate level of calibration and can be used to predict flows for the design rainfall event. Attributes including peak flow rate, hydrograph shape, and volume are used to conclude the ability to predict performance in the system. A comparison of the regression analysis results and measured flows in Appendix C indicate a close prediction of actual events.

# 5.2 Collection System Capacity Analysis

## 5.2.1 Capacity Deficiencies

Capacity deficiencies are defined as locations where SSOs occur and flow does not reach the treatment plant, or where a pipe is surcharged and the hydraulic grade line (HGL) is within a specified distance of the ground surface for the 5-year wet season design flow. For purposes of this analysis, pipe surcharge is allowed, and when the modeled HGL reached a level less than 6 feet from the ground surface (freeboard less than 6 feet) a deficiency is identified. The 6-foot freeboard deficiency criterion was discussed with the City as an appropriate combination of allowed pipe surcharge for short-term peak flows and protection from overflows. Basement flooding was not considered to be a significant concern given their relatively limited number, and the lack of historical basement flooding complaints. For shallow pipes (pipes with less than 8 feet of available freeboard measured from ground to top of pipe) a capacity deficiency criterion that allows no more than 2 feet of surcharge was used, instead of 6 feet of surcharge allowed for deeper pipes.

The capacity deficiencies identified by the hydraulic analysis indicate where improvements may be needed to reduce the frequency of future sewer system overflows within the City and meet the DEQ criteria for control of sewer overflows. Such action may include replacing the existing pipe with a larger-diameter pipe, diversion of flows to nearby pipelines, construction of parallel pipelines, or reduction of peak flow rates through pipeline rehabilitation.

## 5.2.2 Existing Conditions

The existing condition capacity analysis ABF is based on wet season dry weather flow rates that occurred during the model calibration period. Existing RDII is calculated using the calibrated regression equations for the 5-year, 24-hour design rainfall.

The existing collection system deficiencies were identified in the following locations, as shown on Figure 5-1 (provided at the end of this section):

- West Hayes Street/North Front Street

  Surface flooding and high HGLs are predicted along West Hayes Street, from North Cascade Drive to North Settlemier Avenue and along North Settlemier Avenue. This is caused by downstream capacity limitations, with pipe flows exceeding gravity capacity and surcharging beginning at North Front Street and Yew Street and extending upstream to West Hayes Street at Cascade Drive. Figure 5-2 shows the profile and maximum HGL for this pipeline for the 5-year, 24-hour storm.

- Young Street from Bryan Street to Mill Creek

Surface flooding and high HGLs are predicted along Young Street, approximately from Gatch Street to State Highway 99E. This is caused by downstream capacity limitations, with pipe flows exceeding gravity capacity. Figure 5-3 shows the profile and maximum HGL for this pipeline for the 5-year, 24-hour storm.

- Progress Way from Highway 214 to Industrial Way

  Surface flooding and high HGLs are predicted along Progress Way, from Highway 214 to Industrial Way. This is caused by downstream capacity limitations, with pipe flows exceeding gravity capacity. Figure 5-4 shows the profile and maximum HGL for this pipeline for the 5-year, 24-hour storm.

Pipe surcharge is also observed along Brown Street as a result of capacity limitations. The magnitude of the surcharge is not enough to violate the established capacity criteria. Table 5-2 shows the Mill Creek pump station does not meet firm capacity requirements for existing conditions.

### 5.2.3 Future Conditions

This section addresses future conditions and existing system deficiencies, future conditions and new system improvements within the urban growth boundary (UGB), and future conditions and new system improvements outside of the UGB.

#### 5.2.3.1 Future Conditions—Existing System Deficiencies

Hydraulic capacity analyses were performed for three future land use scenarios: 2020, 2030, and 2060 land use and population. Future growth projections are based on *Wastewater Flow and Load* (CH2M HILL, November 2008). Using analysis of historical observed flows at the treatment plant and land use planning projections, the study assumes a 2.8 percent growth rate resulting from infill within the current service area until the year 2020, and a 1.9 percent rate of growth rate from 2020 to 2060.

It is assumed that by 2020 all commercial parcels within the 2005 UGB will be developed. Beyond 2020, the commercial growth is assumed to be consistent with residential growth. It is assumed that 75 percent of the industrial land within the 2005 UGB will be developed by 2020, and that 100 percent of the industrial land within the 2005 UGB will be developed by 2060.

Base flow rates for residential development assume 90 gallons per capita per day (gpcd), and 750 gallons per acre per day (gpad) for commercial/industrial development. To estimate flows for future conditions, the wet season average base flow (ABF) associated with future development was added to the existing wet season ABF. Therefore:

$$\text{Future Wet Season ABF} = \text{Existing Wet Season ABF} + \text{Future Wet Season ABF}$$

RDII for future development was calculated using a peak rate of 1,500 gpad applied to future developed acres. This rate is the same as was used for the 1995 Woodburn Wastewater Facilities Plan Collection System Evaluation. This rate of RDII is typical of more recently developed areas that are representative of conditions in growth basins using modern construction techniques and pipe materials. Future RDII flows were added to the existing RDII obtained from the regression equations.

### 2020 Conditions

Collection system deficiencies for year 2020 flow and land use conditions were identified in the following locations:

- Brown Street from Comstock Street to East Cleveland Street

  Surface flooding and high HGLs are predicted along Brown Street, from Comstock Street to East Cleveland Street. This is caused by downstream capacity limitations, mainly between Wilson Street and East Cleveland Street, with pipe flows exceeding gravity capacity. This analysis was performed without consideration of flow routing from the future South Pump Station. If the proposed force main discharges downstream of the constricted pipe segments, this potential deficiency may be averted.

- I-5 Pump Station

  Flow to the I-5 Pump Station will exceed its firm capacity. Surface flooding and high HGLs are predicted from development of the western portion of the 2005 UGB west of I-5, as well as the southwest industrial area.

- Stevens Pump Station

  Flow to the Stevens Pump Station will exceed its firm capacity. Surface flooding and high HGLs are predicted upstream of the pump station from development and extension of service to the northwestern portion of the 2005 UGB.

Table 5-2 shows that three pump stations do not meet firm capacity requirements for year 2020 conditions. Figure 5-5 shows the results of the hydraulic modeling for the 2020 scenario. The figure assumes that deficiencies from the previous existing condition scenario have been addressed.

### 2030 Conditions

Collection system deficiencies for year 2030 flow and land use conditions were identified in the following locations:

- Mill Creek Interceptor from Troon Avenue/Tukwila Subdivision to Highway 214

  Surface flooding and high HGLs are predicted from development of the northern portion of the 2005 UGB.

- I-5 Pump Station

  Flow to the I-5 Pump Station will exceed its firm capacity. Surface flooding and high HGLs are predicted upstream of the pump station from development of the western portion of the 2005 UGB west of I-5, as well as the extension of service beyond the 2005 UGB.

- Stevens Pump Station

  Flow to the Stevens Pump Station will exceed its firm capacity. Surface flooding and high HGLs are predicted upstream of the pump station from development and extension of service to the northwestern portion of the 2005 UGB.

  Table 5-2 shows the required pump station improvements for 2030 conditions with no rehabilitation. Figure 5-6 shows the results of the hydraulic modeling for the 2030 scenario. The figure assumes that deficiencies from all previous existing condition scenarios have been addressed.

### 2060 Conditions

Collection system deficiencies for year 2060 flow and land use conditions were identified in the following locations:

- Mill Creek Interceptor from Shenandoah Lane to Highway 214

  Surface flooding and high HGLs are predicted from further development of the northern portion of the 2005 UGB as well as extension of service beyond the 2005 UGB.

- I-5 Pump Station

  Flow to the I-5 Pump Station will exceed its firm capacity. Surface flooding and high HGLs are predicted upstream of the pump station from development of the western portion of the 2005 UGB west of I-5, as well as extension of service beyond the 2005 UGB.

- Stevens Pump Station

  Flow to the Stevens Pump Station will exceed its firm capacity. Surface flooding and high HGLs are predicted upstream of the pump station from development and extension of service to the northwestern portion of the 2005 UGB.

Table 5-2 shows the required pump station improvements for year 2060 conditions with no rehabilitation. Figure 5-7 shows the results of the hydraulic modeling for the 2060 scenario. The figure assumes that deficiencies from all previous existing condition scenarios have been addressed.

TABLE 5-2
Pump Station Deficiencies

| Pump Station | Firm Capacity (mgd) | Existing Flow (mgd) | 2020 Flow (mgd) | 2030 Flow (mgd) | 2060 Flow (mgd) |
|---|---|---|---|---|---|
| I-5 | 1.7 | -* | 2.9 | 3.7 | 6.3 |
| Stevens | 0.3 | -* | 0.5 | 0.6 | 0.6 |
| Mill Creek | 16.0 | 18.9 | 22.4 | 24.7 | 31.1 |

*Existing flows do not exceed firm capacity at specified pump station.

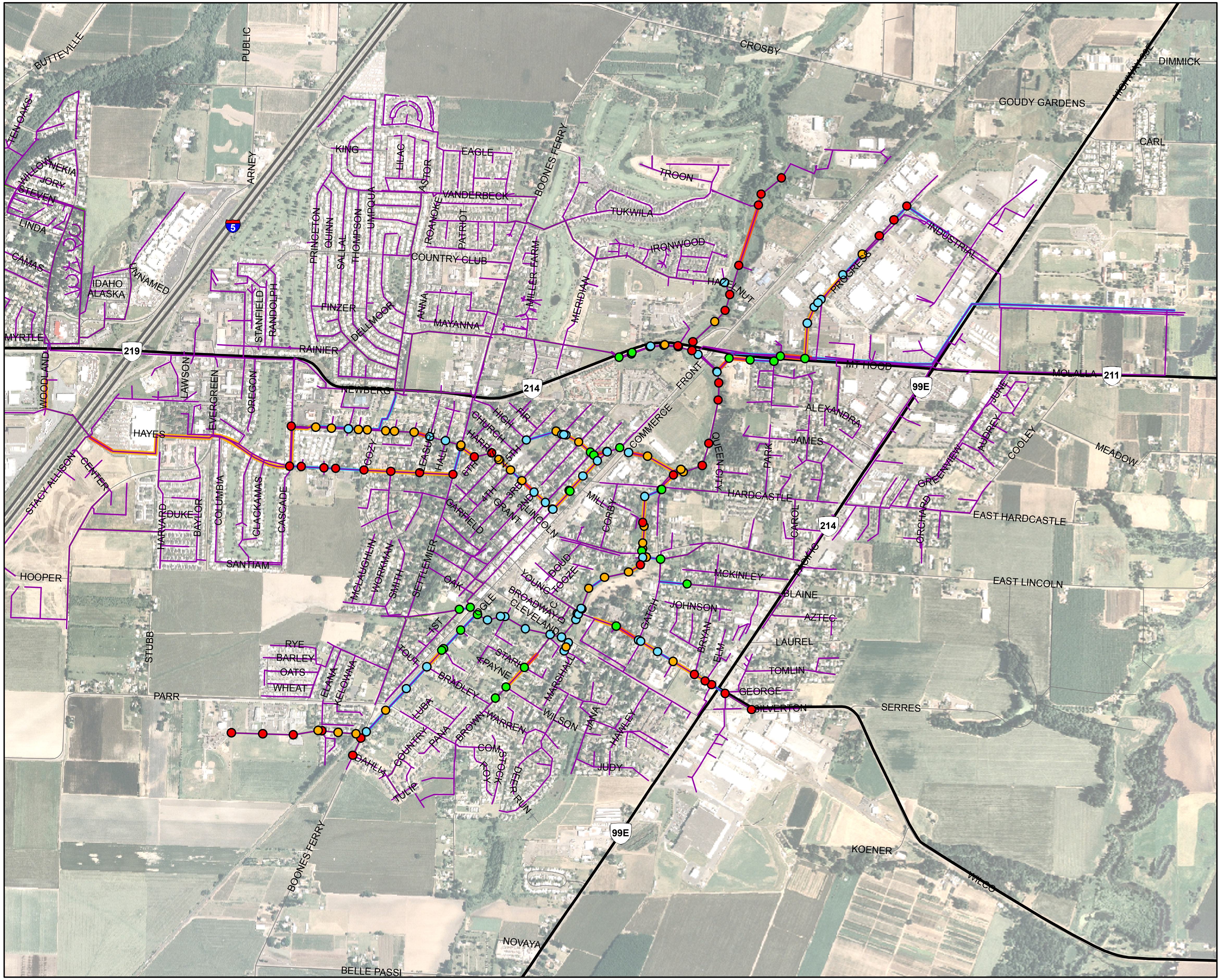
#### 5.2.3.2 Future Conditions—New System Improvements within Urban Growth Boundary

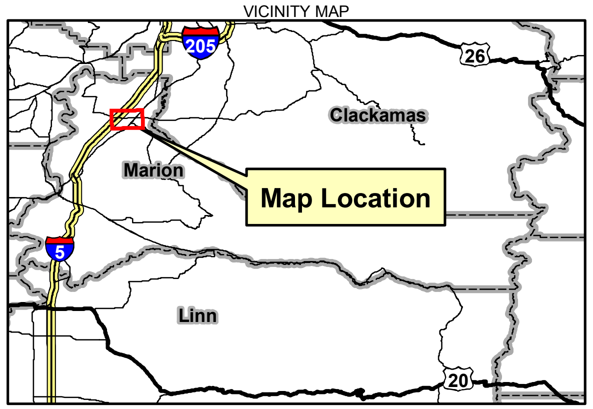
The 2005 Public Facilities Plan provided a strategy for serving areas within the UGB that do not currently have sewer service. This same strategy has been maintained and incorporated into the current Plan. Flows have been routed into the existing system in a manner consistent with the 2005 Plan.

Identified improvements adding new conveyance to the system within the UGB are shown in Figure 5-8. These improvements include over 30,000 feet of new pipe, varying in diameter from 8 to 24 inches.

#### 5.2.3.3 Future Conditions—New System Improvements Outside Urban Growth Boundary

Areas outside the current UGB, in the proposed urban reserve areas (URAs), may eventually require extension of service sewer collection. The actual pattern and timing of development is uncertain, and will affect selected routes and interim phasing strategies.




**LEGEND**

**Freeboard**

- 0 - 2 feet
- 2 - 4 feet
- 4 - 8 feet
- > 8 feet

**Capacity Ratio**

- 0.0 - 1.0
- 1.0 - 1.5
- 1.5 - 3.0
- > 3.0

Notes:
1. Pipe data are a combination of CH2M HILL survey collected data and City of Woodburn CMMS data.

N

0 0.125 0.25
Miles

## FIGURE 5-1
## System Condition
5-year, 24-hour SCS Type 1A Storm
Existing Sanitary Conveyance System
*City of Woodburn*

\\ROSA\PROJ\WOODBURNORCITYOF\367677FP\TASK 8 - COLLECTION SYSTEM\DATA\GIS\MAPFILES\FIGURE5-1SYSTEMCONDITIONS.MXD MSULLIV4 3/30/2009 15:36:29

CH2MHILL

**Figure 5-2: Hydraulic Model Pipe Profile from W. Hayes at Cascade to N. Front at Yew**

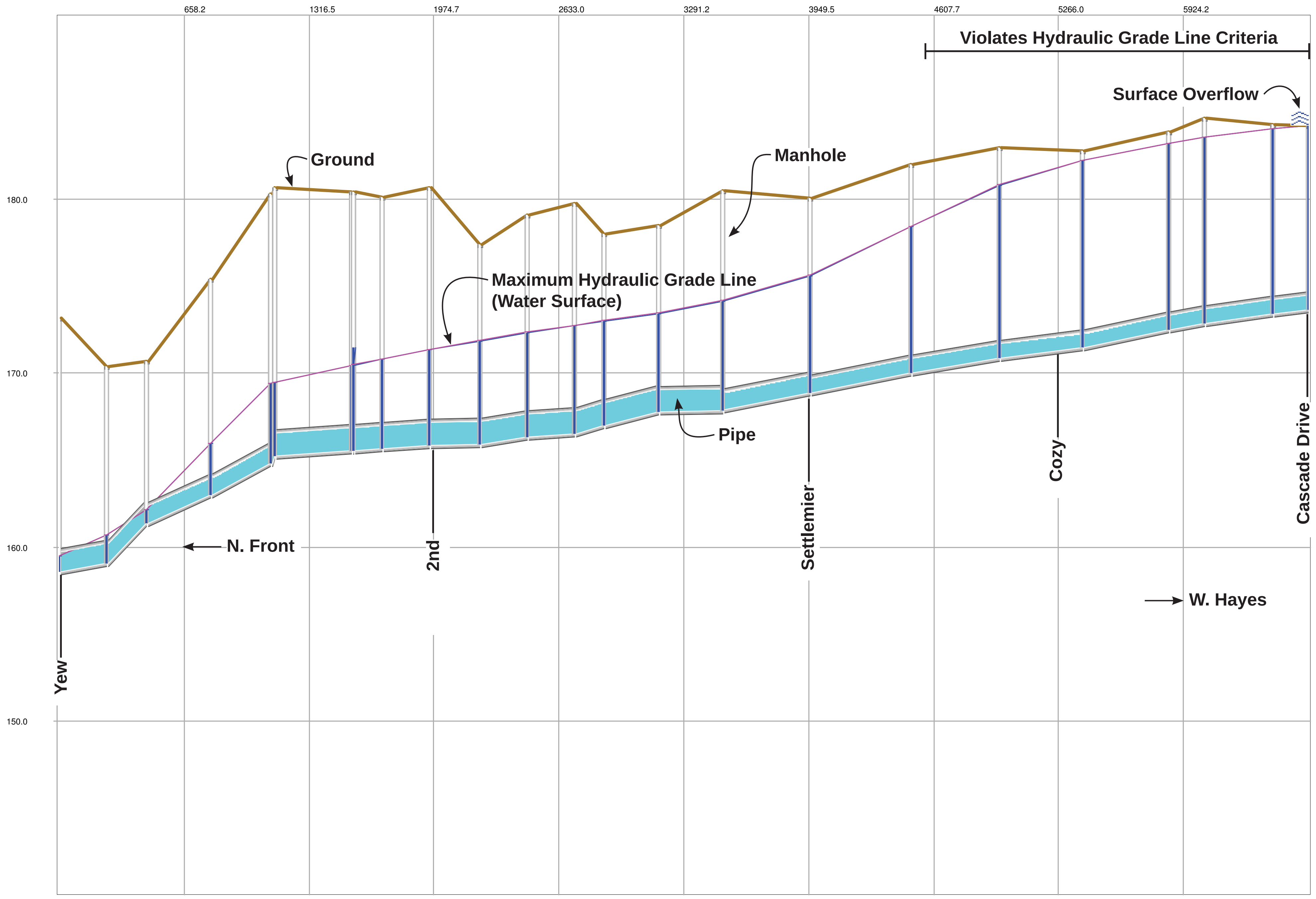


**Figure 5-3: Hydraulic Model Pipe Profile Along Young Street from Mill Creek to Silverton Road**

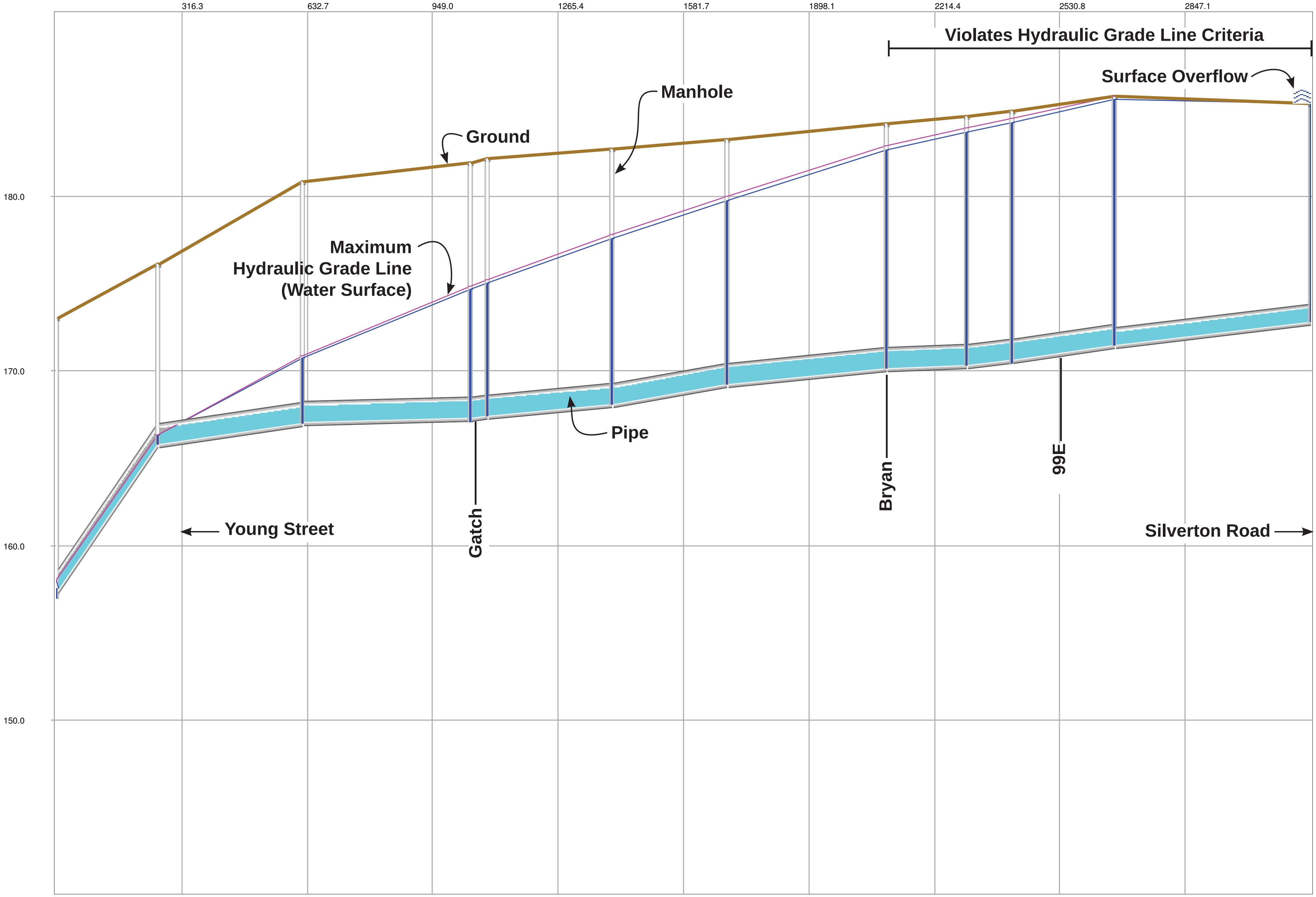
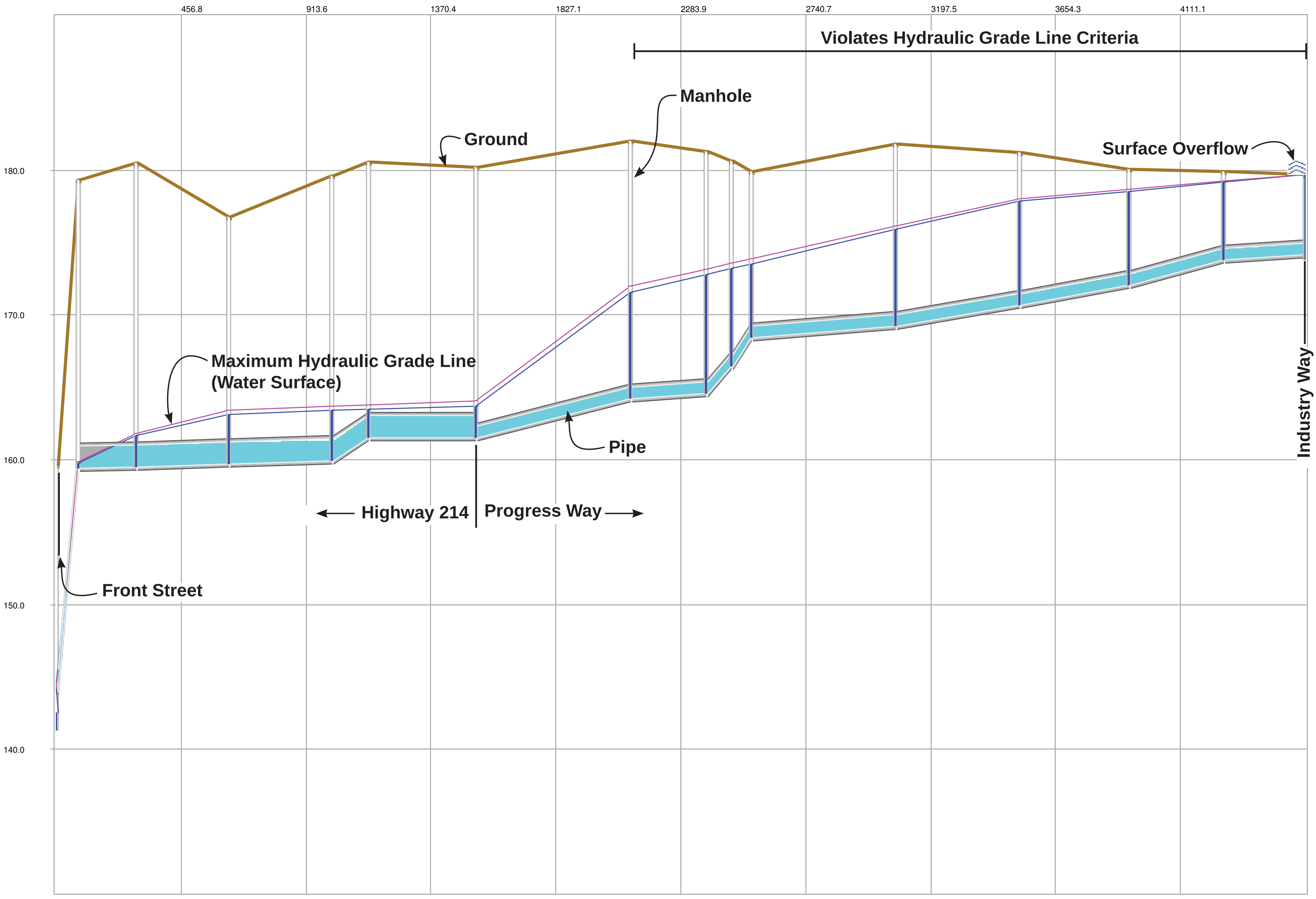


**Figure 5-4: Hydraulic Model Pipe Profile from N. Front Street at Highway 214 to Progress Way at Industrial Way**

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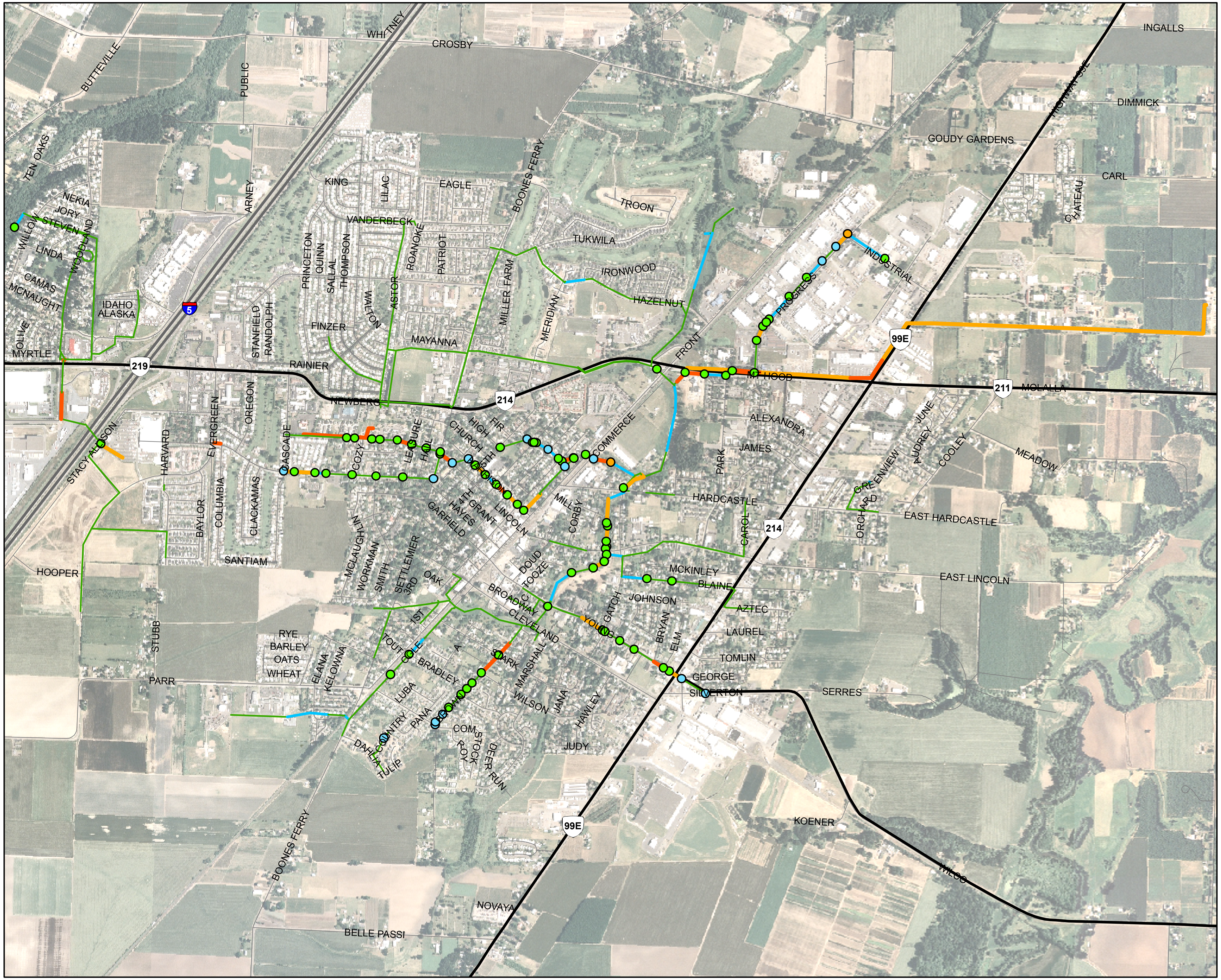
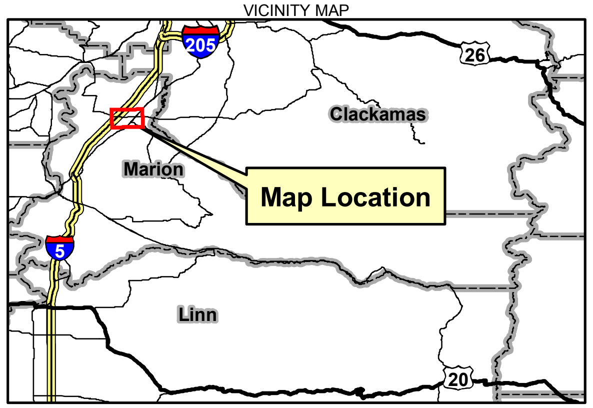



## LEGEND

**Freeboard**

- 0 - 2 feet
- 2 - 4 feet
- 4 - 8 feet
- > 8 feet

**Capacity Ratio**

- 0.0 - 1.0
- 1.0 - 1.5
- 1.5 - 3.0
- > 3.0

Note:
1. Pipe data are a combination of CH2M HILL survey collected data and City of Woodburn CHS data.

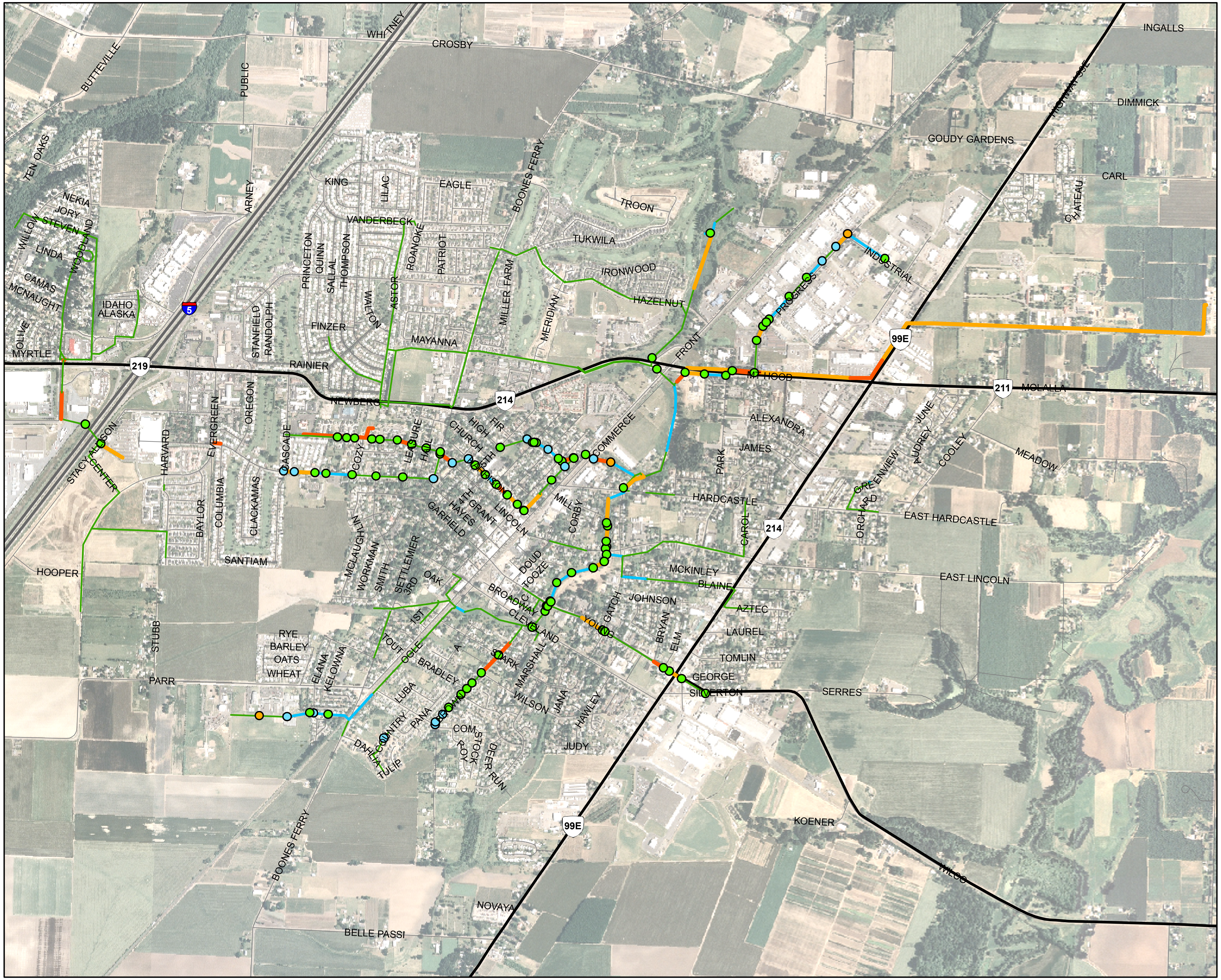
N

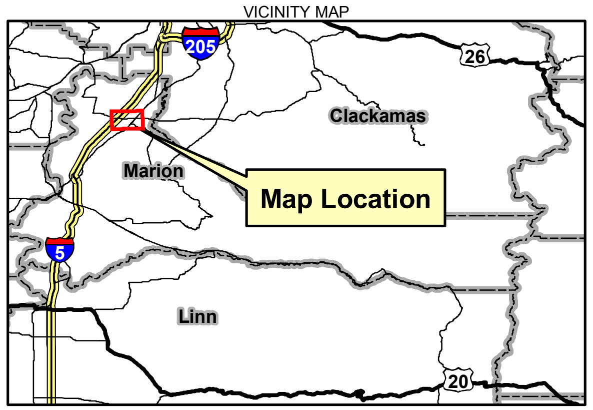
0 0.125 0.25
Miles

**FIGURE 5-5**
**2020 Scenario**
**Hydraulic Modeling Results**
5-year, 24-hour SCS Type 1A Storm
Existing Sanitary Conveyance System
*City of Woodburn*

**CH2MHILL**

\\ROSA\PROJ\WOODBURNORCITYOF\367677FP\TASK 8 - COLLECTION SYSTEM\DATA\GIS\MAPFILES\FIGURE5-5_2020SCENARIOHYDRAULICMODELINGRESULTS.MXD MSULLIV4 3/30/2009 15:29:58




## LEGEND

**Freeboard**

- 0 - 2 feet
- 2 - 4 feet
- 4 - 8 feet
- > 8 feet

**Capacity Ratio**

- 0.0 - 1.0
- 1.0 - 1.5
- 1.5 - 3.0
- > 3.0

Note:
1. Pipe data are a combination of CH2M HILL survey collected data and City of Woodburn CHS data.

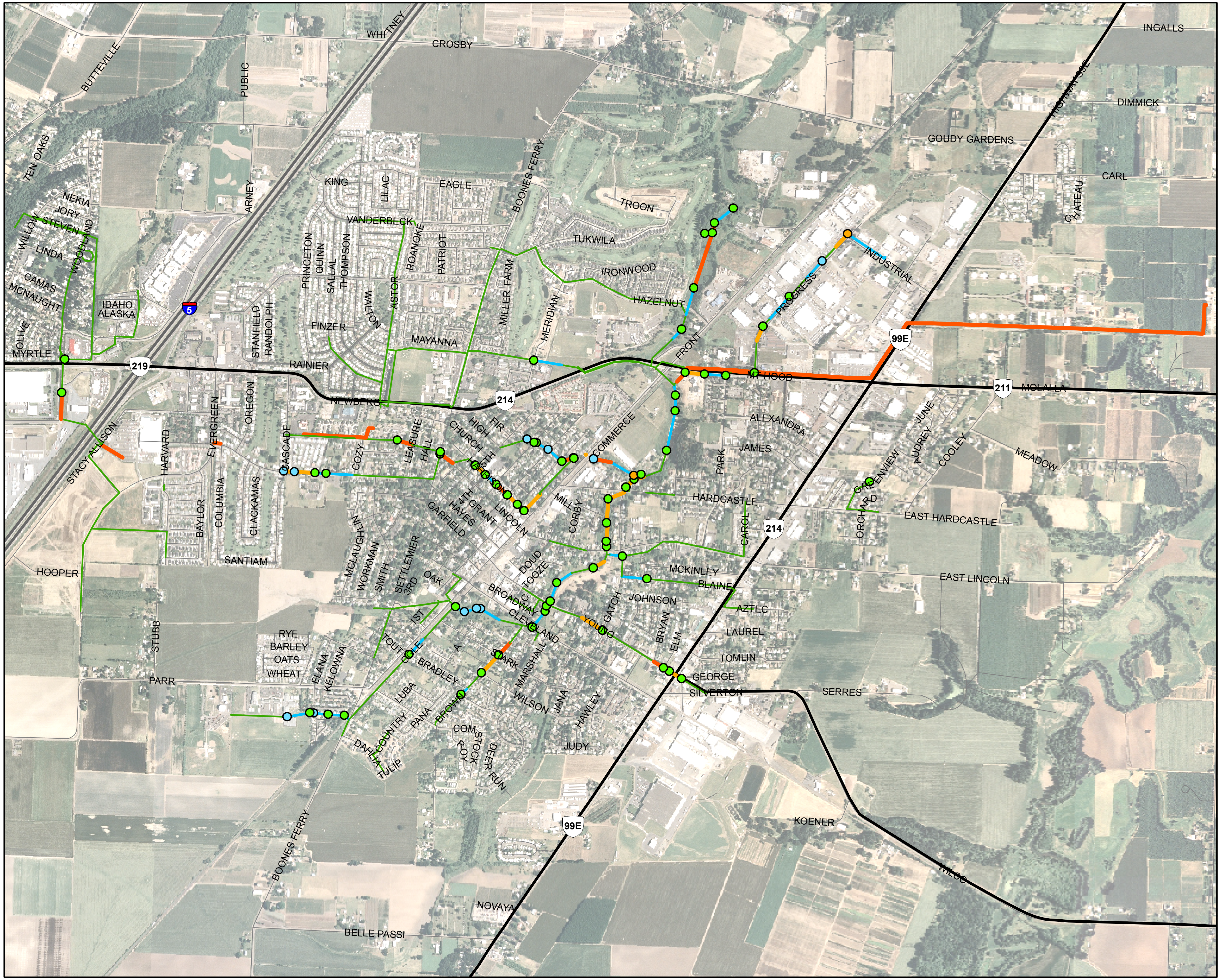
N

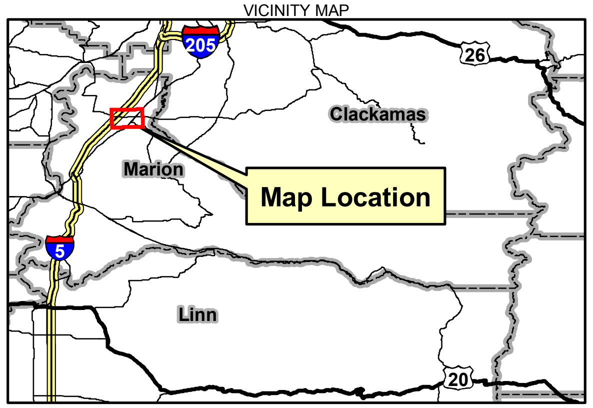
0 0.125 0.25
Miles

**FIGURE 5-6**
**2030 Scenario**
**Hydraulic Modeling Results**
5-year, 24-hour SCS Type 1A Storm
Existing Sanitary Conveyance System
*City of Woodburn*

CH2MHILL

\\ROSA\PROJ\WOODBURNORCITYOF\367677FP\TASK 8 - COLLECTION SYSTEM\DATA\GIS\MAPFILES\FIGURE5-6_2030SCENARIOHYDRAULICMODELINGRESULTS.MXD MSULLIV4 3/30/2009 13:39:55




## LEGEND

**Freeboard**

- 0 - 2 feet
- 2 - 4 feet
- 4 - 8 feet
- > 8 feet

**Capacity Ratio**

- 0.0 - 1.0
- 1.0 - 1.5
- 1.5 - 3.0
- > 3.0

Note:
1. Pipe data are a combination of CH2M HILL survey collected data and City of Woodburn CHS data.

N
0 0.125 0.25
Miles

**FIGURE 5-7**
**2060 Scenario**
**Hydraulic Modeling Results**
5-year, 24-hour SCS Type 1A Storm
Existing Sanitary Conveyance System
*City of Woodburn*



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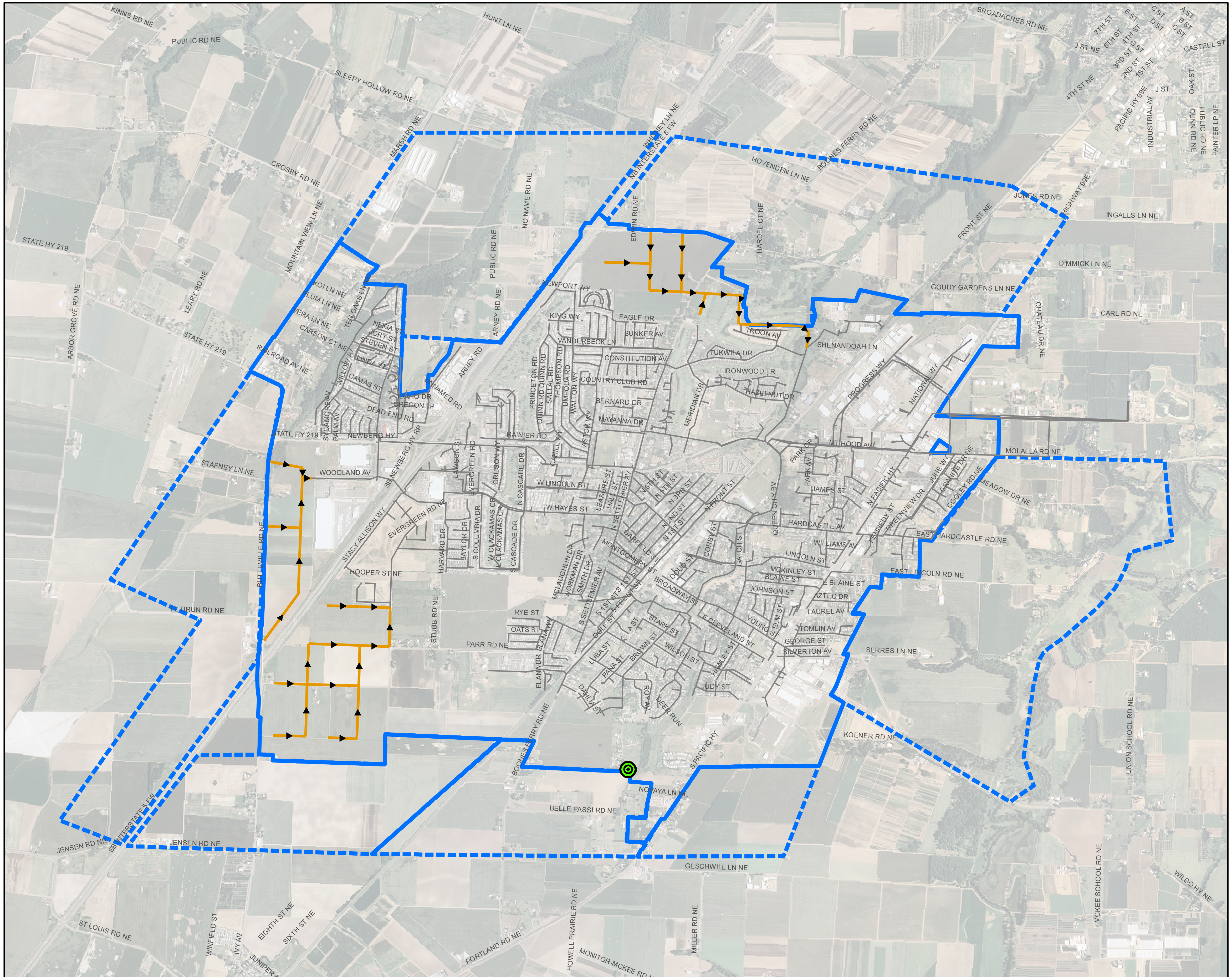
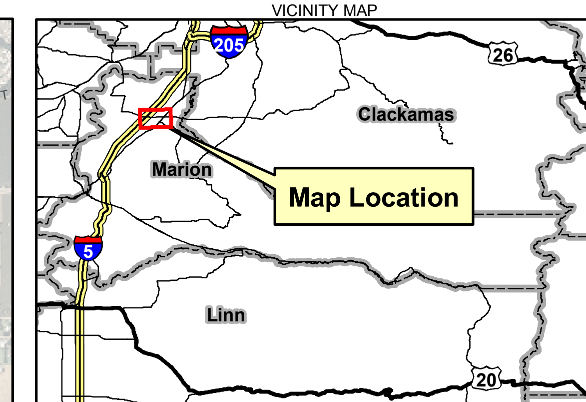





LEGEND

- Proposed Pump Station
- 2005 PFP Pipe Extensions
- Collection System Pipe
- 2005 Urban Growth Boundary (UGB)
- Potential Service Areas

0 1,500 3,000 6,000
Feet

**FIGURE 5-8**
**Future 2030-2060 Conveyance System Improvements within the Urban Growth Boundary**
*City of Woodburn*



CH2MHILL

**SECTION 6**

# Inflow Reduction Plan Evaluation

This section provides an evaluation of the City of Woodburn's Inflow Reduction Plan (Plan) and recommends steps for implementation. The Plan is part of the City's Inflow Reduction Program (IRP).

## 6.1 Overview of Inflow Reduction Program

The City's IRP was established to satisfy the National Pollutant Discharge Elimination System (NPDES) permit. The program must identify overflow points, verify that overflow does not occur up to a 24-hour, 5-year storm even or equivalent, and identify and remove all inflow sources into the sewer system over which the city has legal authority. Where the City does not have legal authority, it will gain the necessary authority to require inflow reduction, a program, and a schedule for removing inflow sources.

### 6.1.1 History

The assessment performed in 1992 to develop the IRP states that, at that time, 94 percent of the pipes in the MC-3, and 50 percent of the lines in the MC-7, were surcharging and had potential overflow.

To prevent the overflow in these areas from occurring, the City performed upgrades including, but not limited to, providing flow relief for Mill Creek Pump Station via a western reliever sewer line and extending the I-5 force main from the I-5 Pump Station, and providing flow relief by a 24-inch bypass from Astor and Rainer that relieved the Highway 214 line. Some pipes within the MC-3 basin still operate under slight surcharge during peak flow but should cause no operational problems. Other areas where slight surcharging is also apparent are in the MC-1 and MC-2 in the IRP.

CH2M HILL manhole observations, maintenance and operations data, physical characteristic analysis, and hydraulic modeling of the current system found the aforementioned sub-basins, MC-2, MC-7, MC-3, and MC-1, to be areas of interest and concern with regard to surcharging and I/I. The 24-inch bypass from Astor and Rainer was intended to relieve Highway 214 line. However, problems are still being reported along the 8-inch AC pipe from the Rainer Pump Station to the 10-inch AC pipe along Sallal Road.

### 6.1.2 Inflow Reduction Plan

The IRP outlines an Inflow Reduction Plan, which includes inflow and outflow analysis, field investigation and survey, and system corrections. In the analysis, areas will be identified based on dry versus wet weather flows, sewer mapping, interviews, flow diagrams, preliminary field studies, and the engineering action report. Given these areas, fieldwork could then be done to pinpoint potential areas of issue including groundwater analysis, rainfall simulation, selective flow monitoring, engineering analysis and report, and selective televised inspection. The corrections to implement would then include manhole repair, rerouting roof drains, replacement of defective manhole covers, raising manhole covers, plugging private storm drain inflow, and changing grading to eliminate ponding.

The City has made use of a CHS to track maintenance problem areas and to provide notes on condition-related deficiencies. Since scoring mechanisms in such programs can be complex or underutilized, some organizations find it helpful to consider a simplified rating system that helps to describe problem areas and prioritize potential projects for repair or rehabilitation. Section 6.2 outlines the general characteristics of one such rating scheme.

One Plan recommendation not yet implemented is private lateral rehabilitation. Section 6.3 discusses available options and recommendations to move toward implementation of that portion of the Plan.

## 6.2 System Rating and Rehabilitation Options

Use of a simplified rating system can help differentiate and prioritize between possible repair and rehabilitation projects. The ratings can be correlated numerically to CHS scoring or be developed based on closed circuit television inspection video, direct observation, or other methods. Table 6-1 provides a summary description of one such approach.

TABLE 6-1
Simplified System Rating by Grade

| Grade | Category | Condition Assessment |
|---|---|---|
| A | Very Good | Few minor defects. Anticipated to provide useful service life of 50 or more years. |
| B | Good | Minor and few moderate defects. May require repairs within the next 21 to 50 years. |
| C | Fair | Moderate defects that will continue to deteriorate and require repairs within next 10 to 20 years (Master planning horizon). |
| D | Poor | Severe defects that will soon deteriorate and require repairs within the next 2 to 10 years. |
| E | Very Poor | Sewer requires repairs or improvement in the next 2 years. |
| F | Emergency | Requires immediate attention – Possible health or safety hazard. |

Table 6-2 shows the types of defects expected to be found for the various assigned grades. Pipes rated C or below might typically be considered for a repair or rehabilitation project.

TABLE 6-2
Common Defects by Grade

| Grade | Most Common Defects |
|---|---|
| A | • No debris or solids deposition<br>• No misalignments |
| B | • Minor debris or solids deposition (less than 1/2 inch deep) |
| C | • Roots (frequent and infrequent)<br>• Misalignment of sewer pipe segment (vertically or horizontally)—portions of the sewer line have standing pockets of water<br>• Infrequent small (1/2 inch or less in width) cracks (radial and longitudinal)<br>• Infrequent joint problems (broken and misaligned)<br>• Moderate Debris or solids deposition (1/2 to 2 inches deep)<br>• Minor lateral problems (protrusions common)<br>• Evidence of infiltration (stains at joints or cracks) |
| D | • Major debris or solids deposition (1/4 of pipe diameter depth)<br>• Misalignment of sewer pipe segment (vertically or horizontally)—1/4 of pipeline length has standing water<br>• Medium frequency of small (1/2 inch or less in width) and large (1/2 inch or greater in width) cracks (radial and longitudinal)<br>• Medium frequency of joint problems (broken and misaligned)<br>• Visible joint gaskets<br>• Minor leaking at pipe joints<br>• Low frequency of structural problems (deterioration and ovaling of < 5%)<br>• Medium frequency of lateral problems<br>• Visible infiltration (less than 1 gallon per minute [gpm]) |
| E | • Blockages (greater than 1/4 of pipe diameter in depth)<br>• High frequency of small (1/2 inch or less in width) and large (1/2 inch or greater in width) cracks (radial and longitudinal)<br>• Misalignment of sewer pipe segment (vertically or horizontally)—over 1/4 of pipeline length has standing water<br>• High frequency of joint problems (broken and misaligned)<br>• Missing joint gaskets<br>• High frequency of structural problems (deterioration and ovaling) that are affecting the structural integrity of the pipe<br>• Minor portions of reinforcing steel exposed<br>• Concrete spalling of pipe wall<br>• Higher frequency of lateral problems<br>• Groundwater infiltrating into sewer line at flow rates less than a garden hose flow (garden hose is 2 to 5 gpm) |

TABLE 6-2
Common Defects by Grade

| Grade | Most Common Defects |
|---|---|
| F | • Full blockage of sewer line from debris or solids deposition<br>• Collapsed section of pipe or section of pipe missing<br>• Major portions of reinforcing steel exposed<br>• Full pipeline length or diameter pipeline large (1/2 inch or greater in width) cracks (radial and longitudinal)<br>• Disconnected or broken lateral<br>• Sewage exfiltrating into adjacent soil<br>• Groundwater infiltrating into sewer line at flow rates greater than 5 gpm (garden hose is 2 to 5 gpm)<br>• Contains an apparent void or opening |

Table 6-3 provides a survey of the many methods of repair and rehabilitation available to improve an identified problem area. Selection of specific methods for a project is dependent on a number of factors such as crew experience, degree of pipe degradation, and available equipment.

TABLE 6-3
Pipeline Rehabilitation Options

| Rehabilitation Option | Principal Advantages | Principal Disadvantages | Diameter Range |
|---|---|---|---|
| | **Pipeline Preparation** | | |
| Cleaning | Increases effective capacity<br>May resolve localized problems | May be costly and cause damage<br>May become a routine requirement | Up to 36” |
| Root Removal | May increase effective capacity<br>May resolve localized problems | Additional maintenance cost<br>Problem likely to recur | All |
| | **Grouting** | | |
| Internal Joint Grouting<br>Acrylamide Gel<br>Acrylate Gel<br>Urethane Gel<br>Polyurethane Foam | Seals leaking joints and minor cracks<br>Prevents soil loss<br>Low cost and causes minimal disruption<br>Can reduce infiltration<br>Can include root inhibitor | Infiltration may find other routes of entry<br>Existing sewer must be structurally sound<br>Considered short-term solution<br>Requires experienced contractors | Up to 48” |
| External Grouting<br>Cement Grout | Improves soil conditions surrounding conduit<br>Can reduce infiltration and soil loss | Difficult to assess effectiveness<br>Can be costly<br>Costly to find point of application | All |

TABLE 6-3
Pipeline Rehabilitation Options

| Rehabilitation Option | Principal Advantages | Principal Disadvantages | Diameter Range |
|---|---|---|---|
| | **Point Repairs** | | |
| Point (Spot) Repairs<br>Internal<br>External | Deals with isolated problems<br>Many internal and external solutions available | May require excavation for some defects<br>May require extensive work on brick sewers | All |
| | **Applied Linings** | | |
| Reinforced Shotcrete Placement | Variety of cross-sections possible<br>More applicable to odd shaped-sewers | Requires person entry—may be labor intensive<br>Lacks corrosion protection<br>Difficult to determine structural properties | 36-inch and larger |
| Concrete Placement | Same as above | Same as above | 36-inch and larger |
| Spray-on Coatings<br>Paint-on Coatings | No excavation<br>Variety of cross-sections possible<br>Some automated machines for small-diameter applications | Difficult to verify quality<br>Full bypass pumping required<br>May be labor intensive<br>Control of infiltration required<br>Does not correct connection problems | 36-inch and larger |
| | **Sliplining** | | |
| Segmented Linings<br>VCP (Gladding McBean, Mission Clay)<br>PVC (Weholite, Permacore, Spirolite)<br>RCP (Ameron, HydroConduit)<br>FRP (Hobas) | High strength-to-weight ratio<br>Variety of cross-sections can be manufactured<br>Minimal disruption | Some materials easily damaged during installation<br>May require temporary support during grouting<br>Labor intensive<br>Joint problems on curved pipes<br>Requires person entry<br>External lateral connection – trenching<br>Point repairs required prior to installation<br>Full bypass pumping required | 36-inch and larger |
| Continuous Pipe –<br>Fusion-welded HDPE<br>(Plexco, Driscopipe)<br>Polybutylene<br>Polypropylene | Quick insertion<br>Large-radius bends accommodated<br>Less costly in shallow trenches than other methods<br>All materials are available now | Circular cross-section only<br>Insertion/receiving trench disruptive<br>Large reduction of cross-section area in smaller sizes<br>Less cost-effective where deep<br>External lateral connection – trenching | 4-inch and larger |

TABLE 6-3
Pipeline Rehabilitation Options

| Rehabilitation Option | Principal Advantages | Principal Disadvantages | Diameter Range |
|---|---|---|---|
| | | Point repairs required prior to installation<br>Bypass pumping requirements vary for different materials | |
| Roll Down (Sewage Lining) | Same as above<br>Commonly used for water pipe rehabilitation | Same as above | 3-inch to 24-inch |

## 6.3 Private Lateral Rehabilitation

In addition to the public lateral rehabilitation program addressed in the IRP, a private rehabilitation program could help reduce the inflow in the collection system. The purpose of a private lateral rehabilitation program is to achieve more I/I reduction than with rehabilitation of the public sewer system only, and build a higher confidence for achieving reduction targets in the long term. Industry findings suggest that public system rehabilitation alone is not as effective as a public and private rehabilitation program for the following reasons: (1) a public-only program does not address the I/I from the private laterals, and (2) I/I can migrate to locations in the private system where defects allow I/I to enter the system. As a result, the most effective program combines public and private system rehabilitation.

A private lateral rehabilitation program requires new processes and associated administration, including the following elements:

- Increased public involvement
- Regulation/Ordinance
- Payment options
- Enforcement
- Inspection

As part of a 2003 Water Environment Research Foundation (WERF) study titled "*Reducing Peak Rainfall-Derived Inflow and Infiltration Flow Rates*," 44 utilities were contacted regarding their programs and a detailed analysis of 12 projects for six utilities was performed. The following conclusions were derived from the case study analyses:

- Rehabilitation of only the sewers in the public-right-of-way may provide little reduction in peak-hour RDII flows. One study found a 17 percent reduction in peak-hour flows when the portion of building laterals in the public right-of-way was replaced. The other projects of this type found 5 percent or less reduction.

- As a corollary to the above, projects that addressed private sewers from the right-of-way to the house achieved 50 to 70 percent peak-hour RDII flow reductions.

- Public sewer rehabilitation may beneficially reduce overall RDII volume. Reductions in peak 24-hour average RDII volumes ranged from 2 to 30 percent. Reductions in peak monthly average flows ranged from 2 to 65 percent. Reduction in the total volume of RDII, but not the peak, suggests that infiltration from the groundwater entering public sewers can be reduced significantly under certain conditions (depending on the overall groundwater conditions). Reductions in peak-day and peak-month RDII volumes benefit wastewater treatment facilities, but do not necessarily benefit the conveyance system.
- The exception to the rule described above was a manhole rehabilitation project in Milwaukee, Wisconsin, that apparently achieved a 45 percent reduction in peak RDII flows through manhole rehabilitation only. The circumstances suggest that attention to manholes as inflow sources in instances where ground conditions reduce the impact of groundwater may produce significant results. Manhole rehabilitation in other case studies, however, did not achieve the same results.

More information on this study (#99-WW-F8) can be found at [www.wef.org](http://www.wef.org).

If a private rehabilitation program is amended to the IRP, the following items should be considered in program development and implementation:

- Inspection of private laterals, roof drains, and foundation drains (continued field verification program to identify problem areas)
- Notice of defects and required corrections (mailers to affected property owners identifying the problem and the required action)
- Repair of defects (addresses the repair or replacement of the defective lines)
- Enforcement (policy developed to address non-compliance by property owner)
- Who Pays? (Identification of payment policy based on the alternatives stated above)
- Incentives for completion (identification of any incentives to the property owner to complete the repair work in a timely manner)

The following sections summarize program characteristics and options for implementation.

### 6.3.1 Program Participation

Private lateral replacement is a system-wide issue. However, specific drainage basins in the system have been identified through flow monitoring as contributing more I/I than other basins. The rehabilitation program could target one or both of the following groups:

- Property owners whose laterals are determined to be defective through inspection as part of a public rehabilitation project
- Anyone whose lateral fails or is determined to be defective independent of its location (relative to public rehabilitation projects)

### 6.3.2 Incentive Options for Participation

A program that includes some financial incentives would be desired given the disruption to private property caused by private lateral replacement. Several options to consider individually or in combination are as follows:

- Pay lateral replacement in part or in whole through rates (by cities).
- Reduce the property owner's sewer bill.
- Add a surcharge to the bills of property owners who do not comply with a replacement directive.
- Provide financial assistance to qualifying low-income property owners.
- Incorporate deferred payment options into the program.

### 6.3.3 Voluntary versus Mandatory

Two options to consider are as follows:

- Implement the program as a long-term, voluntary program.
- Incorporate a phased approach, where initial participation in the program developed is voluntary but would become required at some point in the future. An example would be to provide incentives for voluntary replacement during public rehabilitation projects but make inspection and potential replacement mandatory at the time of ownership transfer.

### 6.3.4 Timing of Participation

Participation could be required for one or more of the following conditions:

- When public rehabilitation is being performed in that lateral's basin
- When the lateral fails, independent of public rehabilitation activities
- When property ownership is transferred

### 6.3.5 Total or Partial Lateral Rehabilitation

- Current city policy is to publicly maintain only the mainline. Lateral maintenance to the mainline, even within public right-of-way, is the property owner's responsibility. A targeted, voluntary rehabilitation or replacement program focused on areas of high I/I contribution might operate under a different policy to provide incentive for effective participation.

Portions of basins MC-1 through MC-9, have higher I/I rates. Land use in basins MC-1, MC-3, MC-4, MC-5, MC-6, and MC-9 are highly commercial or residential, meaning the likelihood of many private laterals is high. If the City chooses to pursue a private lateral program, these target areas would provide the most positive cost-benefit response.

### 6.3.6 Recommendations for Private Lateral Rehabilitation

The condition of the private laterals and their related contribution to RDII directly affect City assets. Many RDII reduction programs and pilot testing have determined that long-term reduction effectiveness includes rehabilitation of private laterals.

Based on recent monitoring data, the benefit of significant reduction in RDII is not likely to be a cost-effective investment for the City. Details of this analysis are found in Section 7. Cost-effectiveness curves for I/I reduction do not include private laterals. This is an appropriately conservative assumption given institutional and public relations hurdles that can face communities who desire to implement a private lateral program. However, the reduction achieved may offer added benefit, possibly allowing deferral of some treatment or conveyance improvements.

The following options for private lateral rehabilitation are recommended to the City for consideration as prudent next steps:

- Develop the framework and authority for a private lateral rehabilitation program. Establish a legal right to perform inspection of private laterals. Some communities require this inspection as part of a transfer of ownership or as existing pipeline improvement projects are being performed.
- Require broken or damaged laterals to be repaired as part of ownership transfer.
- If high I/I areas show continued deterioration, consider an incentive-based or cost-sharing approach targeted at high-impact areas.

SECTION 7

# Alternatives Development and Evaluation

Cost curves were developed to compare the following collection system alternatives: conveyance and treatment improvements or RDII reduction. The two types of improvements were analyzed to identify a least-cost solution. The proposed improvements are described in Sections 7.1 and 7.2 with a brief description of how they were applied. Section 7.3 provides capital cost estimates and identifies the apparent least cost option.

## 7.1 Capital Cost Estimates

All cost estimates are order-of-magnitude estimates as defined by the American Association of Cost Engineers (AACE). An order-of-magnitude estimate is made without detailed engineering data and uses techniques such as cost curves and scaling factors applied to estimates developed for similar projects. The overall expected level of accuracy of the cost estimates presented is -30 percent to +50 percent. This means that bids can be expected to fall within a range of 30 percent under to 50 percent over the estimate for each project. These ranges are consistent with the guidelines established by the AACE for planning level studies.

The economic evaluation was based on capital cost estimates. The capital cost estimates were prepared in 2008 dollars. They do not include future escalation. Financing costs, operations and maintenance costs, and potential hazardous material mitigation costs are also not included. The cost opinion shown has been prepared for guidance in project evaluation from the information available at the time of preparation. The final costs of the project will depend on actual labor and material costs, actual site conditions, actual site productivity, competitive market conditions, final project scope, final project schedule, and other variable factors. As a result, the final project costs will vary from those presented below. Because of these factors, funding needs must be carefully reviewed prior to making specific financial decisions or establishing final budgets.

For gravity pipes, easements and right-of-way acquisitions are not included. It is assumed that groundwater dewatering during construction is not a significant issue. Pipes were assumed to require an average depth of bury of eleven feet. All new pipes were assumed to be PVC. For force mains, three feet of cover is assumed. For projects located in existing streets, imported trench backfill and a full depth street pavement overlay was assumed.

For pump and lift station cost estimates, easements and right-of-way acquisitions are not included. It is assumed that groundwater is not an issue. Emergency generators are not included.

Detailed cost estimate calculations are provided in Appendix E.

## 7.2 Conveyance and Treatment

Selected pipelines are replaced with larger diameter pipelines to convey peak flows with adequate freeboard between the hydraulic grade line and ground surface. The freeboard criterion is 6 feet, so at all manhole locations where the water surface is predicted to be less than 6 feet from the ground, or where the maximum surcharge is greater than 2 feet for shallow pipes, a pipeline replacement project was identified. This element also includes pump station improvements where the peak flow exceeds the rated firm capacity of the station (largest pump out of service as required by DEQ).

### 7.2.1 Existing Conditions Conveyance Improvements

Table 7-1 shows the required conveyance system improvements for existing conditions with no rehabilitation.

TABLE 7-1
Required Conveyance System Improvements for Existing Conditions, 5-year, 24-hour Storm Event with No Rehabilitation

| Location | City Pipe ID | Existing Diameter (inches) | 2060 Required Diameter (inches) | Length (feet) | Cost |
|---|---|---|---|---|---|
| Young Street | 36-MC21-09 | 12 | 18 | 400 | |
| | 36-MC21-08 | 12 | 18 | 292 | |
| | 36-MC21-07 | 12 | 18 | 313 | |
| | 36-MC21-06 | 12 | 18 | 44 | |
| | 36-MC21-05 | 12 | 18 | 423 | $1,773,000 |
| | 28-MC21-04 | 12 | 18 | 366 | |
| Hayes Street | 20-WH-8 | 10 | 12 | 454 | |
| | 20-WH-7 | 10 | 12 | 437 | |
| | 20-WH-3 | 10 | 15 | 465 | $2,030,000 |
| | 20-WH-2 | 10 | 15 | 534 | |
| | 29-H-09 | 12 | 15 | 457 | |
| Front Street | 29-C-09 | 12 | 18 | 338 | |
| | 29-C-12 | 16 | 18 | 404 | |
| | 29-C-11 | 16 | 18 | 20 | $1,040,000 |
| | 29-C-10 | 12 | 18 | 315 | |
| Progress Way | 47-MC2-10 | 10 | 12 | 452 | |
| | 47-MC2-09 | 10 | 12 | 528 | $1,362,000 |
| | 38-MC2-07 | 10 | 18 | 566 | |
| **Total** | | | | **6,808** | **$6,205,000** |

Table 7-2 shows the required pump station improvements for existing conditions with no rehabilitation.

TABLE 7-2
Required Pump Station Improvements for Existing Conditions, 5-year, 24-hour Storm Event with No Rehabilitation

| Pump Station | Firm Capacity (mgd) | Existing Flow (mgd) | Required Capacity Improvement (mgd) | Cost |
|---|---|---|---|---|
| Mill Creek (Stage 1) | 16.0 | 18.9 | 2.9 | $500,000 |

mgd = million gallons per day.

### 7.2.2 Future Conditions Conveyance Improvements

This section addresses future conditions and existing system deficiencies, future conditions and new system improvements within the UGB, and future conditions and new system improvements outside of the UGB.

#### 7.2.2.1 Future Conditions—Existing System

Table 7-3 shows the required conveyance system improvements for year 2020 conditions with no rehabilitation.

TABLE 7-3
Required Conveyance System Improvements for Year 2020 Conditions, 5-year, 24-hour Storm Event with No Rehabilitation

| Location | City Pipe ID | Existing Diameter (inches) | 2060 Required Diameter (inches) | Length (feet) | Cost |
|---|---|---|---|---|---|
| Brown Street* | 28-MC25-03 | 10 | 12 | 339 | |
| | 28-MC25-04 | 10 | 12 | 284 | |
| | 27-MC25-09 | 10 | 12 | 430 | |
| **Total** | | | | **1,053** | **$931,000** |

*Brown Street conveyance improvements are reported assuming an upstream connection from the South Brown Street Pump Station. This assumption should be reconsidered prior to construction of the South Brown Street Pump Station or improvements to Brown Street conveyance. Routing pump station flows downstream of the capacity deficiency may eliminate the need for the project.

Table 7-4 shows the required pump station improvements for year 2020 conditions with no rehabilitation. It is assumed that these improvements, with the exception of Mill Creek Pump Station, will be most cost-effectively constructed to meet 2060 build-out peak flows without a phased improvement schedule.

TABLE 7-4
Required Pump Station Improvements for Year 2020 Conditions, 5-year, 24-hour Storm Event with No Rehabilitation

| Pump Station | Firm Capacity (mgd) | 2060 Flow (mgd) | Required Capacity Improvement (mgd) | Cost |
|---|---|---|---|---|
| I-5 | 1.7 | 6.3 | 4.6 | $1,307,000 |
| Stevens | 0.3 | 0.6 | 0.3 | $990,000 |
| Mill Creek (Stage 2) | 16.0 | 31.1 | 15.1 | $2,605,000 |

Table 7-5 shows the required force main improvements for year 2020 conditions to be constructed in conjunction with pump station improvements.

TABLE 7-5
Required Force Main Improvements for Year 2020 Conditions, 5-year, 24-hour Storm Event with No Rehabilitation

| Pump Station | Existing Diameter (inches) | 2060 Required Diameter (inches) | Length (ft) | Cost |
|---|---|---|---|---|
| I-5 | 8 | Additional 12 | 6,319 | |
| **Total** | | | | **$3,093,000** |

Table 7-6 shows the required conveyance system improvements for year 2030 conditions with no rehabilitation.

TABLE 7-6
Required Conveyance System Improvements for 2030 Conditions, 5-year, 24-hour Storm Event with No Rehabilitation

| Location | City Pipe ID | Existing Diameter (inches) | Required Diameter (inches) | Length (feet) | Cost |
|---|---|---|---|---|---|
| Mill Creek Interceptor | 38-WR2-03 | 15 | 24 | 477 | |
| | 39-WR2-04 | 15 | 24 | 492 | |
| | 38-WR2-02 | 15 | 24 | 140 | |
| | 39-WR2-07 | 15 | 24 | 490 | |
| | 39-WR2-05 | 15 | 24 | 490 | |
| **Total** | | | | **2,676** | **$1,855,000** |

### 2060 Conditions

Although facilities have been sized to meet 2060 flow conditions, the planning horizon for the capital improvement plan is through 2030, so no additional projects were identified for this scenario.

### 7.2.2.2 Future Conditions—New System Improvements within Urban Growth Boundary

The 2005 Public Facilities Plan provided a strategy for serving areas within the UGB that do not currently have sewer service. This same strategy has been maintained and incorporated into the current Plan. Flows have been routed into the existing system in a manner consistent with the 2005 Plan.

Identified improvements adding new conveyance to the system within the UGB are shown in Figure 5-4, and summarized in Table 7-7. Cost estimates for these improvements were developed using the 2005 Plan as a basis for quantities.

TABLE 7-7
Required Conveyance System Improvements to Serve New Areas within the Urban Growth Boundary

| Location | Cost |
|---|---|
| Sanitary Sewer Service to North Area (2005 PFP Project) | **$5,219,000** |
| Sanitary Sewer Service to Southwest Industrial Area (2005 PFP Pipeline Project) | **$9,722,000** |
| **Total** | **$14,941,000** |

#### 7.2.2.3 Future Conditions—New System Improvements Outside Urban Growth Boundary

A cost estimate for these improvements was developed at a conceptual level only. It was assumed that growth would occur at a constant rate between 2020 and 2060, with costs distributed evenly as well. Consequently, 25 percent of the full cost to serve the proposed URAs would occur within the planning period ending in 2030. Therefore, one-quarter of the total cost is reflected in the Capital Plan shown in Section 10, Recommended Plan, of Volume 1: Wastewater Treatment.

### 7.2.3 Treatment Plant Improvements

Additional wet weather treatment capacity is required at the existing POTW and may include capacity increases for the headworks, primary sedimentation, secondary processes, filtration, disinfection, and site piping. Estimated peak hour flows at the POTW for the 5-year, 24-hour design storm through the year 2060 are given in Table 7-8. Figure 7-1 shows the cost relationship to increased POTW capacity.

TABLE 7-8
Publicly Owned Treatment Works Flow Components

| Flow Source | Existing Flow Estimate (mgd) | 2020 Flow Estimate (mgd) | 2030 Flow Estimate (mgd) | 2060 Flow Estimate (mgd) |
|---|---|---|---|---|
| Total Publicly Owned Treatment Works | 18.9 | 22.4 | 24.7 | 31.1 |

## 7.3 Rainfall-Dependent Infiltration and Inflow Reduction

Small-diameter, existing pipelines including private service laterals can be replaced or lined. The selected locations are within basins that exhibited the highest RDII rates (see Table 4-2 in Section 4). RDII rates are based on the 5-year, 24-hour peak flow estimate and the existing developed area for each monitor basin, rather than gross monitor basin area. The result of these improvements is the reduction of RDII and peak flows in the system.

To estimate the amount of RDII reduction that results from pipeline rehabilitation, detailed flow monitoring data representing pre- and post-rehabilitation conditions are required. Because these data are not available for Woodburn's system, a replacement versus reduction relationship was applied from work performed for the Metropolitan Wastewater Management Commission (MWMC) that serves Eugene and Springfield. The MWMC relationship was derived from the results achieved by multiple agencies in and outside of Oregon, and is shown in Figure 7-2. Estimated rehabilitation costs and RDII reduction targets for selected monitoring basins are summarized in Table 7-9. The Highway 214 and Woodland basins were not included in the rehabilitation cost calculations because of their relatively low I/I rates, based on monitoring data.

## 7.4 Cost Comparison of Alternatives

Cost curves were developed to compare the two collection system alternatives: conveyance and treatment improvements versus RDII reduction. Figures 7-3 and 7-4 show the individual and combined costs of the improvements relative to the amount of RDII reduction performed and the projected peak flow rate at the POTW for the 5-year, 24-hour storm under existing conditions and those projected for the year 2060, respectively.

Each figure provides cost on the vertical axis and the percent reduction of RDII on the horizontal axis. For the existing conditions cost curve, at zero RDII reduction, improvements to the POTW are required for flows greater than 16 mgd. In addition to the treatment improvements, conveyance improvements are also required. The existing conditions cost curve indicates that general basin-wide RDII reduction of 18 percent in basins with high rates of I/I is cost-effective. Reduction of RDII by 18 percent (the low point on the total cost curve) corresponds to rehab of 6.7 percent of the pipes in these basins.

For the year 2060 conditions cost curve, the least cost-effective solution is conveyance and treatment of all wastewater flows. General basin-wide RDII reduction is not cost-effective. This is not to say that targeted RDII reduction within a high RDII monitoring basin such as Goose Creek will not be cost-effective, only that general basin-wide rehabilitation beyond that required as a part of normal system maintenance is not cost-effective. Therefore, no rehabilitation is required for the conveyance treatment solution recommended. However, reduction would at least maintain existing levels and may be warranted if growth does not reach 2060 levels.

TABLE 7-9
Estimated Rehabilitation Costs and RDII Reduction for Targeted Monitoring Areas

| Monitor Basin | Existing RDI/I Rate from Monitor Data (gpad) | Total Length of Pipes in Basin (feet) | RDI/I Reduction Target = 27% (10% Pipe Rehabilitation) | | RDI/I Reduction Target = 40% (20% Pipe Rehabilitation) | | RDI/I Reduction Target = 47% (30% Pipe Rehabilitation) | | RDI/I Reduction Target = 53% (40% Pipe Rehabilitation) | | RDI/I Reduction Target = 57% (50% Pipe Rehabilitation) | | RDI/I Reduction Target = 61% (60% Pipe Rehabilitation) | | RDI/I Reduction Target = 63% (70% Pipe Rehabilitation) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Length (ft) | Cost ($)[a] | Length (ft) | Cost ($)[a] | Length (ft) | Cost ($)[a] | Length (ft) | Cost ($)[a] | Length (ft) | Cost ($)[a] | Length (ft) | Cost ($)[a] | Length (ft) | Cost ($)[a] |
| **Burlingham Ditch** | 15,700 | 26,624 | 2,662 | 690,000 | 5,325 | 1,379,000 | 7,987 | 2,069,000 | 10,649 | 2,758,000 | 13,312 | 3,448,000 | 15,974 | 4,137,000 | 18,637 | 4,827,000 |
| **Goose Creek** | 20,600 | 17,601 | 1,760 | 456,000 | 3,520 | 912,000 | 5,280 | 1,368,000 | 7,040 | 1,823,000 | 8,800 | 2,279,000 | 10,560 | 2,735,000 | 12,320 | 3,191,000 |
| **Harrison Basin** | - -[b] | 34,043 | 3,404 | 882,000 | 6,809 | 1,763,000 | 10,213 | 2,645,000 | 13,617 | 3,527,000 | 17,022 | 4,409,000 | 20,426 | 5,290,000 | 23,830 | 6,172,000 |
| **Queen City** | 10,400 | 119,222 | 11,922 | 3,088,000 | 23,844 | 6,176,000 | 35,766 | 9,264,000 | 47,689 | 12,351,000 | 59,611 | 15,439,000 | 71,533 | 18,527,000 | 83,455 | 21,615,000 |
| **Golf Course** | Not Monitored | 29,462 | 2,946 | 763,000 | 5,892 | 1,526,000 | 8,839 | 2,289,000 | 11,785 | 3,052,000 | 14,731 | 3,815,000 | 17,677 | 4,578,000 | 20,623 | 5,341,000 |
| **Plant** | Not Monitored | 25,436 | 2,544 | 659,000 | 5,087 | 1,318,000 | 7,631 | 1,976,000 | 10,174 | 2,635,000 | 12,718 | 3,294,000 | 15,261 | 3,953,000 | 17,805 | 4,611,000 |
| **Totals** | | **233,052** | **25,239** | **6,538,000** | **50,477** | **13,074,000** | **75,716** | **19,611,000** | **100,955** | **26,146,000** | **126,193** | **32,684,000** | **151,432** | **39,220,000** | **176,671** | **45,757,000** |

[a] Pipe rehabilitation cost is $259/foot based on historical rehabilitation project costs.

[b] Not calculated because of inadequate monitor data.

## Figure 7-1.  City of Woodburn Wastewater Treatment Facility Improvement Costs

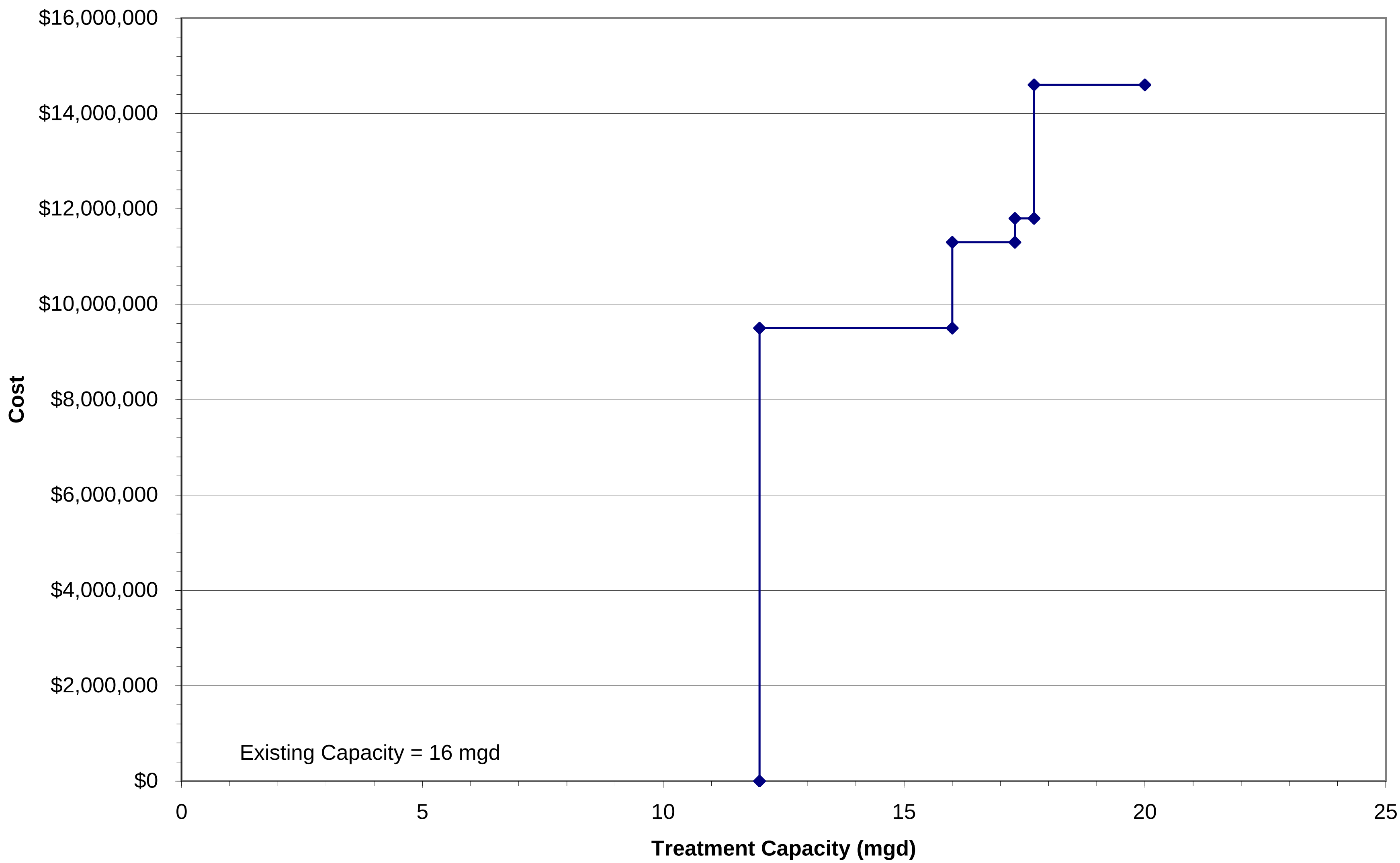


Figure 7-2. Collection System Rehabilitation Versus Estimated RDII Reduction

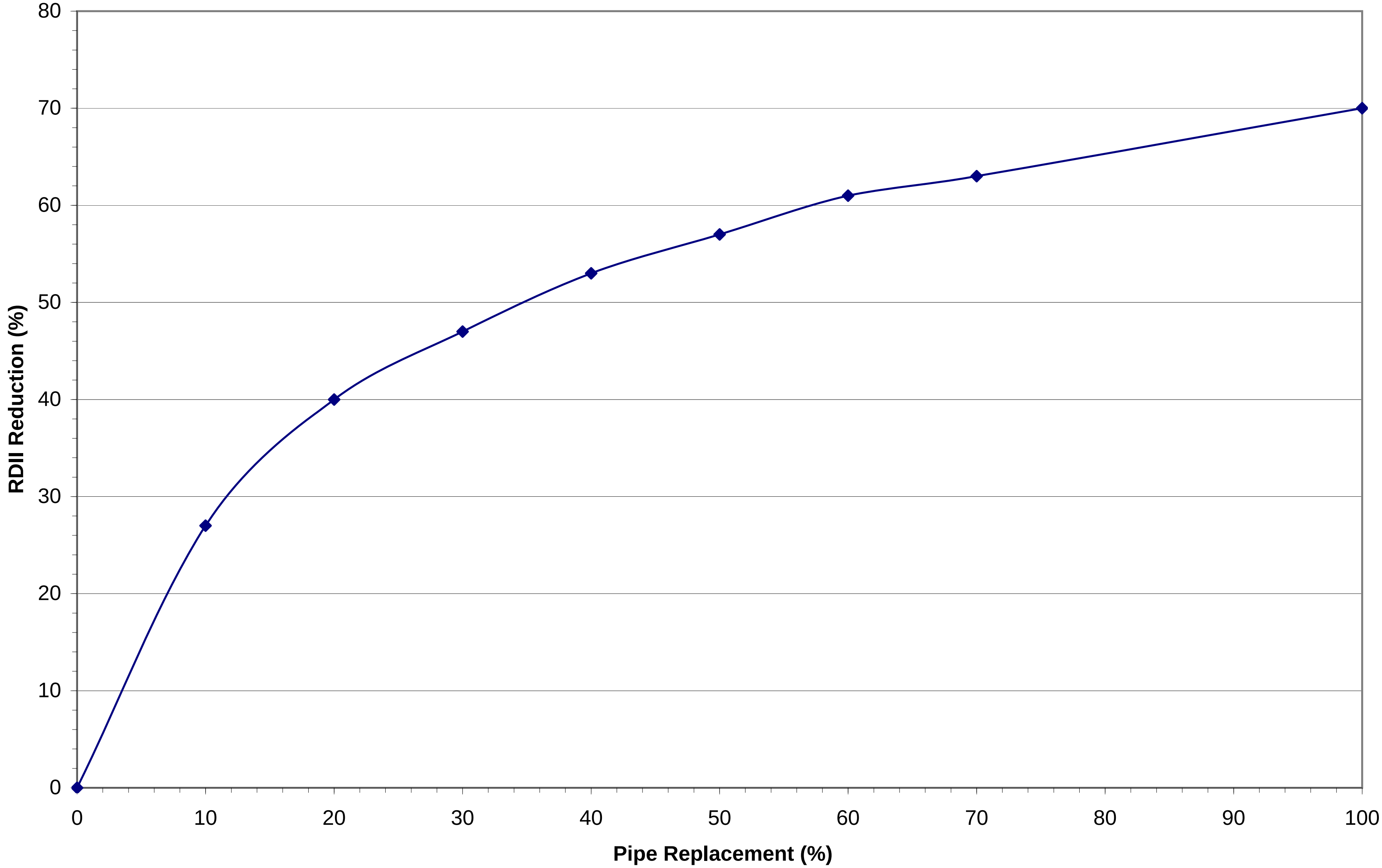


**Figure 7-3. Existing Conveyance/Treatment and I/I Reduction Costs**

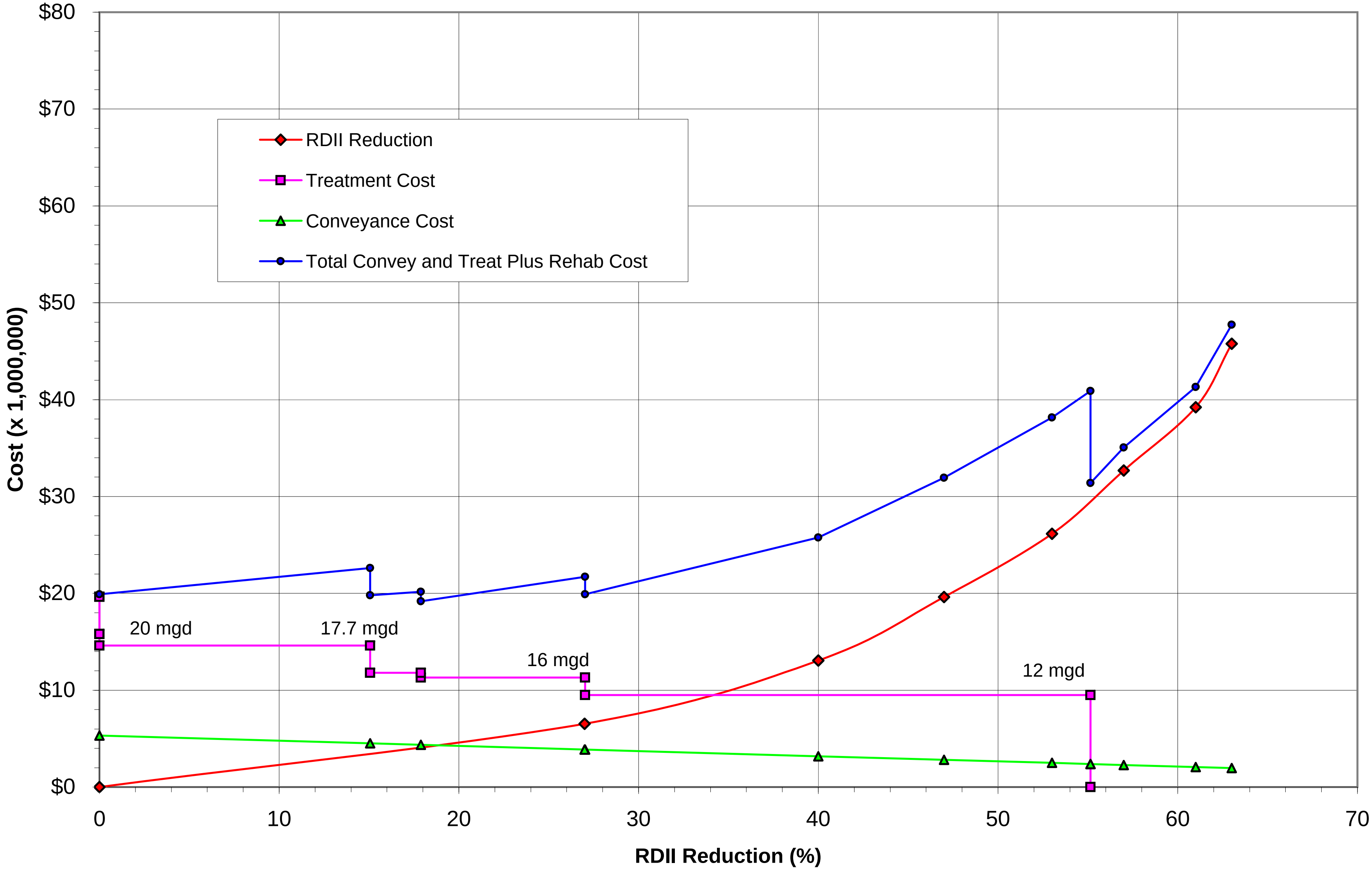
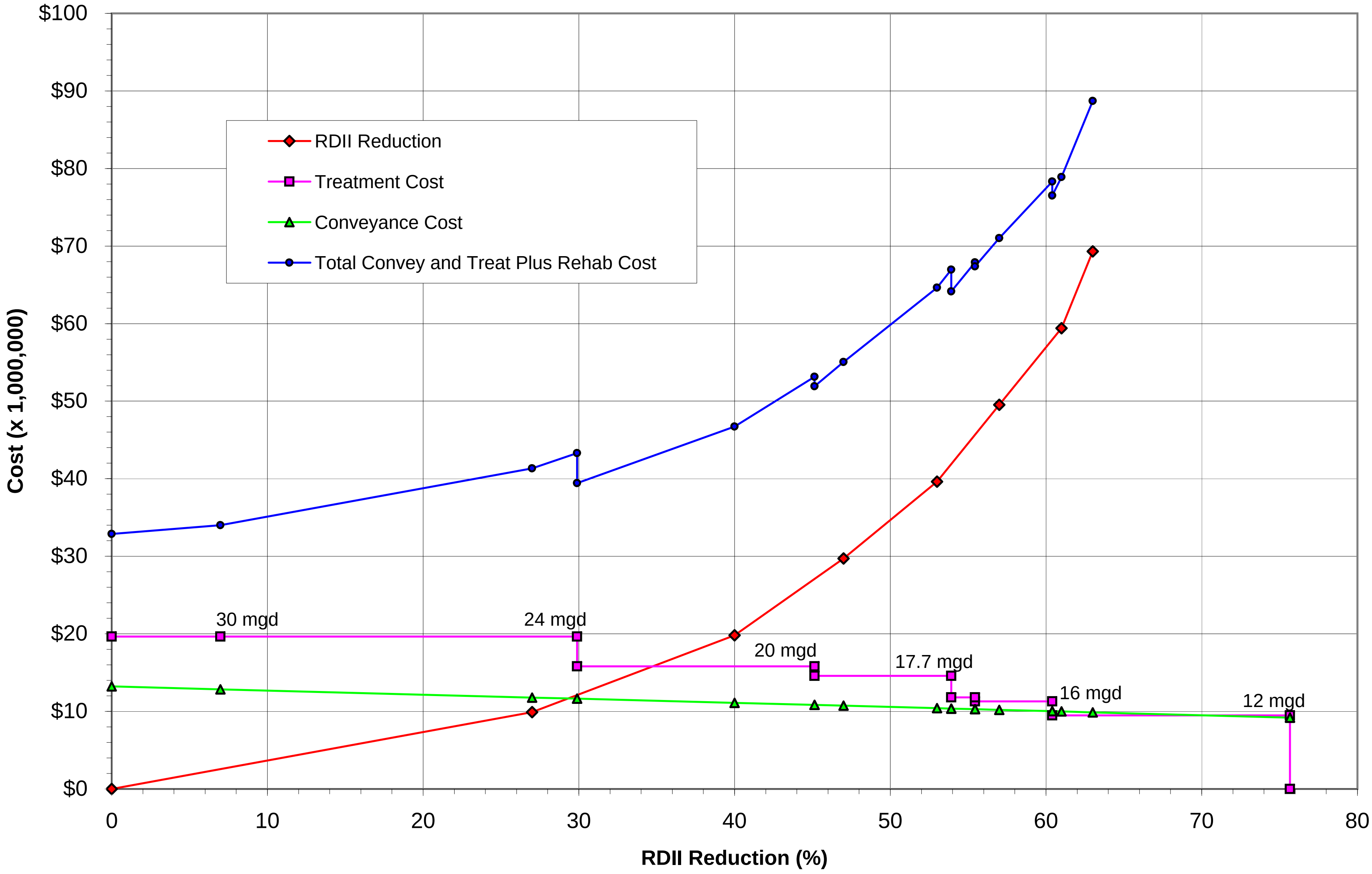


**Figure 7-4. 2060 Conveyance/Treatment and I/I Reduction Costs**



SECTION 8

# Recommended Improvements and Next Steps

This section outlines recommended improvements to the wastewater collection and transmission system. In addition, next steps consisting of implementation strategies and costs, and recommended long-term management activities, are presented.

## 8.1 Recommended Improvements

This section presents recommendations for controlling (managing) sanitary sewer overflows to waters of the State for conditions up to and including the one-in-five year, 24-hour-duration rainfall event. The two major areas of improvements are wet weather flow management and system repair, rehabilitation, and replacement.

### 8.1.1 Wet Weather Flow Management

A phased approach to wet weather flow management is recommended. The improvements associated with the 5-year, 24-hour storm event will be programmed into the Capital Improvement Plan (CIP) with consideration for expanding on that solution if observed system performance results in unacceptable overflows, or reducing the amount of system rehabilitation if effectiveness estimates are exceeded. Improvements focus on conveyance and treatment, given that rehabilitation is less cost-effective. However, rehabilitation provides multiple benefits including asset replacement as well as I/I reduction. Project prioritization should include consideration of system condition as well as capacity deficiencies. Figure 8-1 shows identified pipe improvements through 2060.

In addition to required conveyance capacity improvements, areas that will develop in the future will require service extensions to connect them to the existing collection system.

### 8.1.2 System Repair, Rehabilitation, and Replacement

Recommendations for system repair, rehabilitation, and replacement are summarized below. The recommendations consist of prudent measures for the continued good health of the collection system. Much of the system appears to be in reasonable condition. Therefore, major investment in collection system repair and rehabilitation could be deferred, within the context of ongoing inspection and maintenance, until more significant deterioration begins to show, such as through increased I/I contributions. Recommendations are as follows:

- Enhance the current routine repair, rehabilitation, and replacement schedule and begin to set aside additional funds for the program. A program level budget may wish to focus on the rehabilitation or limited replacement of the 111,000 feet of sewer lines constructed in 1954 or prior.
- Perform risk assessment of pipes to identify those that exhibit highest vulnerability to failure, either because of location or service area. This ensures that investment is made in the right parts of the system first.

- Perform a pilot program for spot repairs and in-situ repairs to evaluate effectiveness and costs for various repair methods. The City may determine that spot repairs may more cost-effectively extend the useful life of the collection sewers than pipe segment major rehabilitation or replacement.
- Improvement implementation should also include the means to define system performance for multiple rainfall events and to assess RDII reduction levels resulting from rehabilitation efforts. To achieve this result, permanent flow monitors should be placed in the system and the resulting data combined with monitored flows at the I-5 and Mill Creek pump stations. This will provide value in determining SSO control compliance and assessing accuracy of hydraulic model predictions, and subsequent refinements.

# 8.2 Implementation

## 8.2.1 Plan Summary

Table 8-1 summarizes the phasing and estimated annual cost of collection system improvement projects through fiscal year 2029-30.

## 8.2.2 Phasing Strategies

### 8.2.2.1 Pipelines

Pipeline projects are sized for 2060 build-out conditions and scheduled to be constructed just in time for projected need. Projects have been spaced to avoid volatility in cost and staff needs. A 2-year sequence has been assumed for predesign, design, permitting, and construction activities for each project.

### 8.2.2.2 Pump Stations

Pump station upgrades are generally assumed to occur in a single stage to build-out capacity requirements. Mill Creek Pump Station is the exception. A relatively small capacity increase is needed to meet current flow requirements. A staged approach to gain this small increase can defer construction of a significantly rehabilitated or all-new structure until approximately 2020.

## 8.2.3 Programmatic Rehabilitation and Replacement

Table 8-1 shows a line item for replacement costs. This cost was determined by identifying the percent of pipe expected to exceed a 75-year installation life during the planning period. That total cost was split evenly by planning year. The intent of this approach is to provide flexibility for adaptive management of the collection system, to address high priority areas as needed. Capacity deficiencies are predictable by nature of the pattern of growth and installed facilities. Maintenance and repair projects are often less predictable and more subject to shifting priorities.

By providing a replacement budget that can be used for repair, rehabilitation, or replacement, the City can prudently manage assets for efficient and sustainable delivery of service. It is assumed that early projects will include those listed in Table 3-9.

## 8.3 Long-Term Management Activities

The recommended plan requires the City to continue their proactive maintenance of the collection system. This approach is essential for the following reasons:

- Growth includes a future RDII allowance, but no increase in existing RDII is assumed. (A dedicated annual budget for an ongoing aggressive collection system maintenance program is identified in the proposed capital improvements plan.)
- Existing RDII must be managed to maintain the selected solution.

To avoid the potential cost consequences of allowing RDII to increase, a meaningful and adequately funded system maintenance program employing best practices must be an integral part of the recommended plan for wet-weather overflow management.

The City will continue to enhance RDII Best Management Practices (BMPs) to meet permit requirements and achieve the desired wet-weather overflow control frequency. These practices are summarized as follows:

- Repair known structural problems
- Perform source identification activities
- Conduct TV inspection
- Perform smoke testing
- Incorporate field investigation results in capital improvement program projects
- Perform flow monitoring
- Replace line pipe in selected areas
- Continue system data management mapping and records storage activities

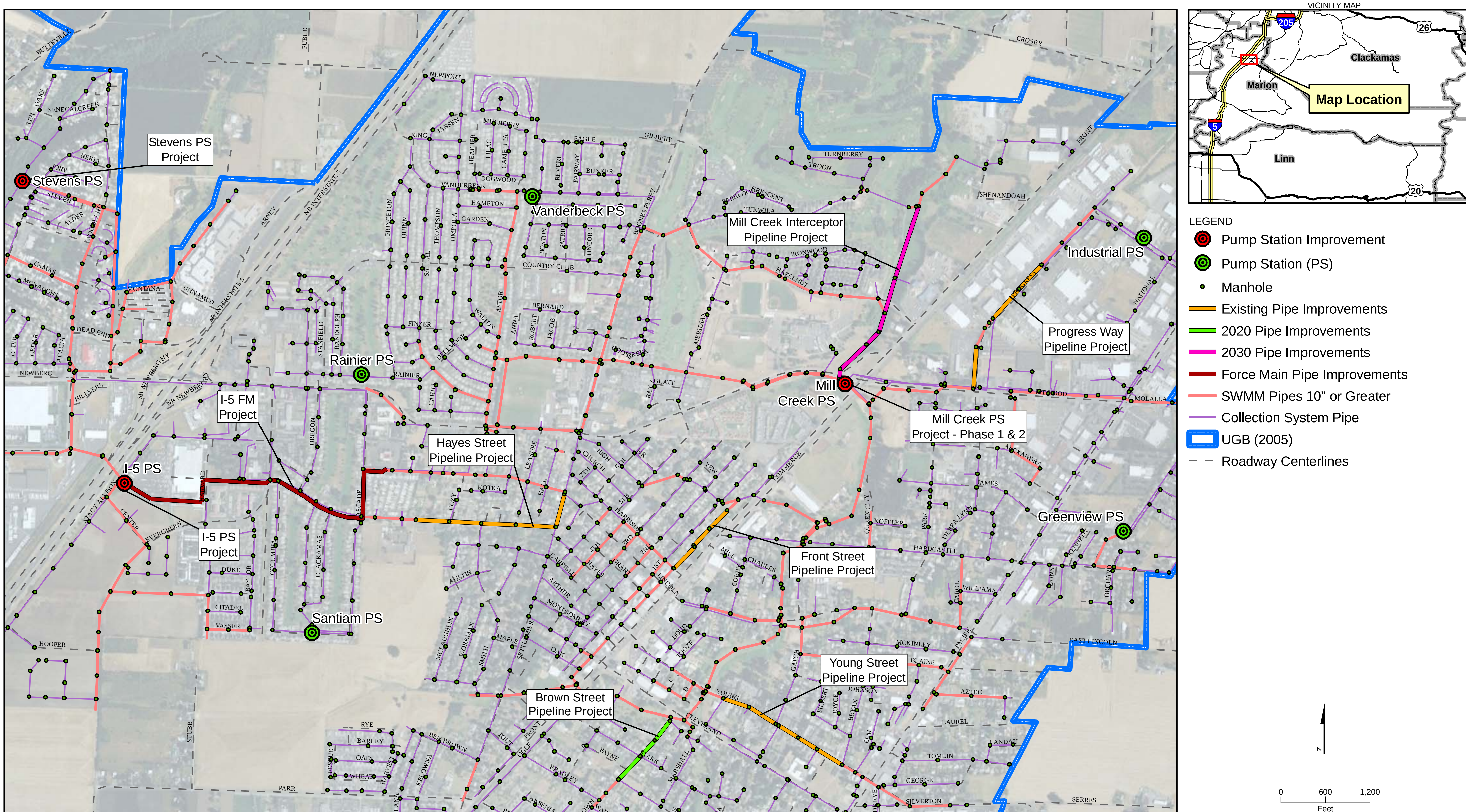



**FIGURE 8-1**
**Identified Capital Improvements**
City of Woodburn

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TABLE 8-1
Wastewater Collection System Project List, Project Cost (In Dollars), and Implementation Schedule

| | Fiscal Year | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **2009-2010** | **2010-2011** | **2011-2012** | **2012-2013** | **2013-2014** | **2014-2015** | **2015-2016** | **2016-2017** | **2017-2018** | **2018-2019** | **2019-2020** | **2020-2021** | **2021-2022** | **2022-2023** | **2023-2024** | **2024-2025** | **2025-2026** | **2026-2027** | **2027-2028** | **2028-2029** | **2029-2030** | **Total** |
| **Collection System Project** | | | | | | | | | | | | | | | | | | | | | | |
| Mill Creek PS Project - Stage 1 | 100,000 | 400,000 | | | | | | | | | | | | | | | | | | | | **500,000** |
| Mill Creek PS Project - Stage 2 | | | | | | | | | 521,000 | 1,042,000 | 1,042,000 | | | | | | | | | | | **2,605,000** |
| I-5 PS Project | | | | 261,000 | 1,046,000 | | | | | | | | | | | | | | | | | **1,307,000** |
| I-5 FM Project | | | | 619,000 | 2,747,000 | | | | | | | | | | | | | | | | | **3,093,000** |
| Stevens PS Project | | | | | | 198,000 | 792,000 | | | | | | | | | | | | | | | **990,000** |
| Young Street Pipeline Project | | | | | | 355,000 | 1,418,000 | | | | | | | | | | | | | | | **1,773,000** |
| Front Street Pipeline Project | | | | | | | 208,000 | 832,000 | | | | | | | | | | | | | | **1,040,000** |
| Mill Creek Interceptor Pipeline Project | | | | | | | | | | | | | | | | | | | | 371,000 | 1,484,000 | **1,855,000** |
| Progress Way Pipeline Project | | | | | 272,000 | 1,090,000 | | | | | | | | | | | | | | | | **1,362,000** |
| Hayes Street Pipeline Project | 406,000 | 1,624,000 | | | | | | | | | | | | | | | | | | | | **2,030,000** |
| Brown Street Pipeline Project* | | | | | | 186,000 | 745,000 | | | | | | | | | | | | | | | **931,000** |
| Sanitary Sewer Service to North Area (2005 PFP Project) | | | | | | | | | | | | | | | | 1,044,000 | 4,175,000 | | | | | **5,219,000** |
| Sanitary Sewer Service to South Area – South Brown St. PS | | | | | | 200,000 | 600,000 | | | | | | | | | | | | | | | **800,000** |
| Sanitary Sewer Service to Southwest Industrial Area (2005 PFP Pipeline Project) | | | | | | | | | | | | | | | | | | | 1,944,000 | 3,889,000 | 3,889,000 | **9,722,000** |
| Area Outside UGB | | | | | | | | | | | | 856,000 | 856,000 | 856,000 | 856,000 | 856,000 | 856,000 | 856,000 | 856,000 | 856,000 | 856,000 | **8,560,000** |
| Current CIP Projects (Funds 465, 472) | 460,000 | | | | | | | | | | | | | | | | | | | | | **460,000** |
| Replacement Costs | | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | 400,000 | **8,000,000** |
| Equipment Replacement (VAC Truck) | | | 350,000 | | | | | | | | | | | | | | | | | | | **350,000** |
| Pump Station Upgrades - Reliability | 50,000 | 75,000 | 75,000 | 75,000 | | | | | | | | | | | | | | | | | | **275,000** |
| **Total** | **1,016,000** | **2,589,000** | **885,000** | **1,687,000** | **5,270,000** | **1,601,000** | **3,029,000** | **2,084,000** | **981,000** | **1,502,000** | **1,502,000** | **1,316,000** | **1,316,000** | **1,316,000** | **1,316,000** | **2,360,000** | **5,491,000** | **1,316,000** | **3,260,000** | **5,576,000** | **6,689,000** | **$50,872,000** |

*****Brown Street Pipeline Project should be re-evaluated in conjunction with determination of alignment for the South Brown Street Pump Station. Results of pre-design efforts may indicate gravity improvements are not necessary if the force main discharges downstream of predicted capacity constraints

APPENDIX A

# Collection System Characteristics Mapping

**APPENDIX B**

# Collection System Condition Mapping

# Appendix C
# Regression Analysis

# APPENDIX D
# Pump Station Notes

## APPENDIX E

# Cost Estimate Details