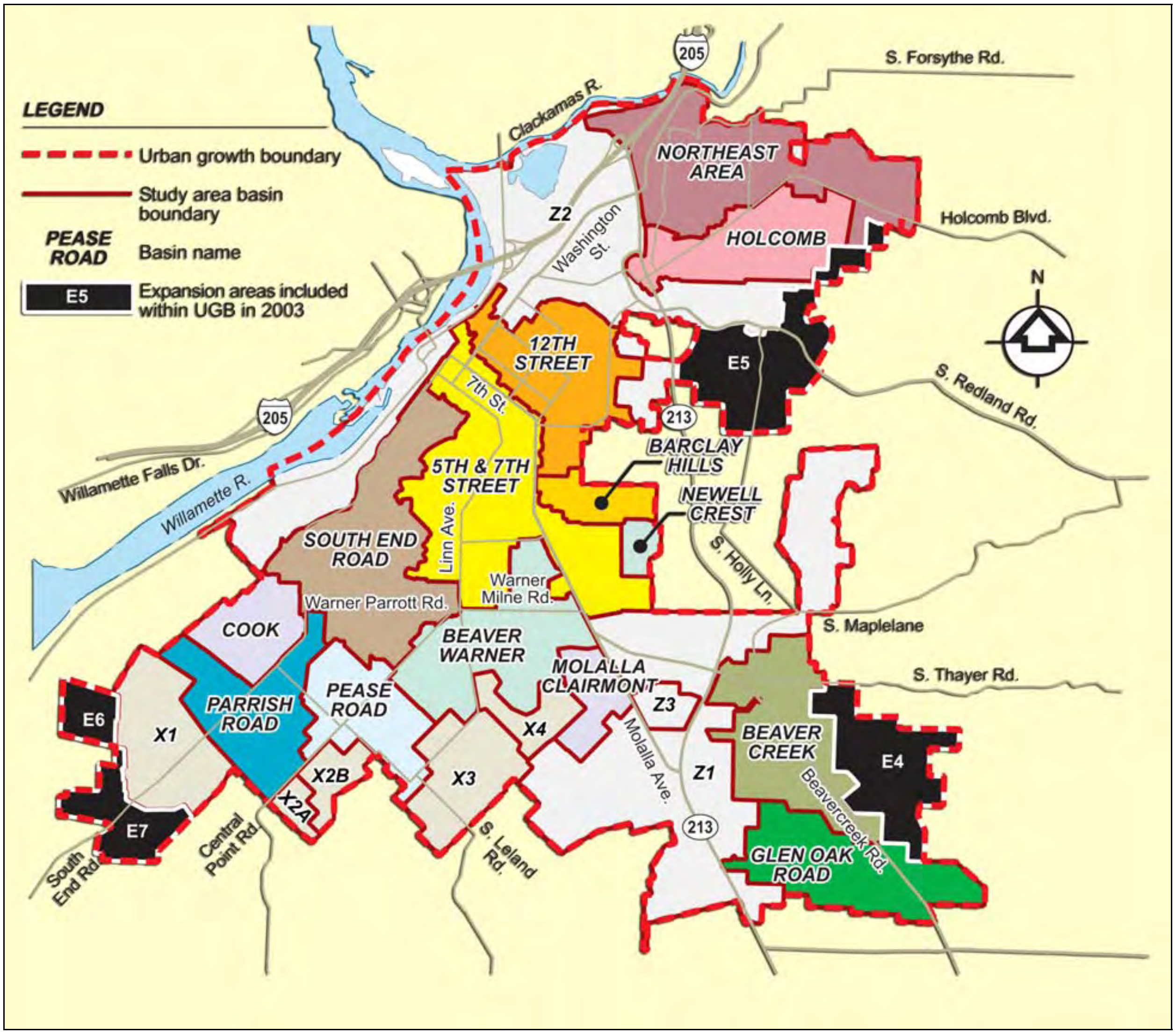
City of Oregon City
Sanitary Sewer Master Plan
Final – Volume I



December 2003

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Portland, Oregon 97223-8022

City of Oregon City
SANITARY SEWER MASTER PLAN
Final - Volume I

December 2003

Prepared for:
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ACKNOWLEDGMENTS

Tt/KCM Inc. appreciates the help and support of City Staff in identifying sanitary sewer issues, opportunities and constraints related to this Sanitary Sewer Master Plan and for their help and support through the process of public and project meetings.

City of Oregon City

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City of Oregon City
Sanitary Sewer Master Plan

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EXECUTIVE SUMMARY

BACKGROUND

The City of Oregon City's last sanitary sewer master plan was completed in 1989. Since then, the City has experienced population growth of more than 50 percent, sewer service has been extended to 34 additional subdivisions, and other capital projects have been undertaken, including projects to separate sewage flows and stormwater flows that previously were conveyed together in combined sewers. Because of the changes since the last master plan, an update is needed to fully evaluate the City's current sewer system, identify deficiencies, and outline a capital improvement program to provide adequate sewer service over a 20-year planning period. The City contracted with Tetra Tech/KCM, Inc. to update its sanitary sewer master plan. Preparation of the master plan involved the following elements:

- Document the existing sewer system.
- Assess existing and future wastewater flows.
- Evaluate sewer capacity and projected capacity needs.
- Evaluate pump station capacity and condition.
- Recommend improvements and prepare a capital improvement plan.

EXISTING CONDITIONS

Study Area

Oregon City's urban growth boundary (UGB) encompasses 5,456 acres, including 732 acres that was added in 2003 by adjusting the previous UGB to include four small expansion areas. The study area for this master plan consists of the entire area inside the expanded UGB, as shown in Figure ES-1. In addition to basins currently served by City sewers, this area includes several basins that do not currently contribute flow to the City's sewer system because they drain directly to Tri-City Service District (TCSD) sewers (Basins Z1, Z2, Z3 and E5) or are undeveloped and therefore unsewered (Basins X1, X2A, X2B, X3, X4, E4, E6, and E7).

Wastewater Collection System

The City's sanitary sewer system consists of collector sewers, trunk sewers, and pump stations within the UGB. The collection system discharges into interceptors operated by the Tri-City Service District of Water Environment Services, a department of Clackamas County. TCSD provides wastewater treatment for Oregon City, Gladstone, West Linn, and other portions of Clackamas County under a temporary diversion agreement at the Tri-City Water Pollution Control Plant in northwest Oregon City. This report focuses on the trunk sewers and pump stations in the City's collection system; smaller sewers and the facilities operated by TCSD are not addressed. Figure ES-2 shows the sewers and pump stations evaluated for the master plan.

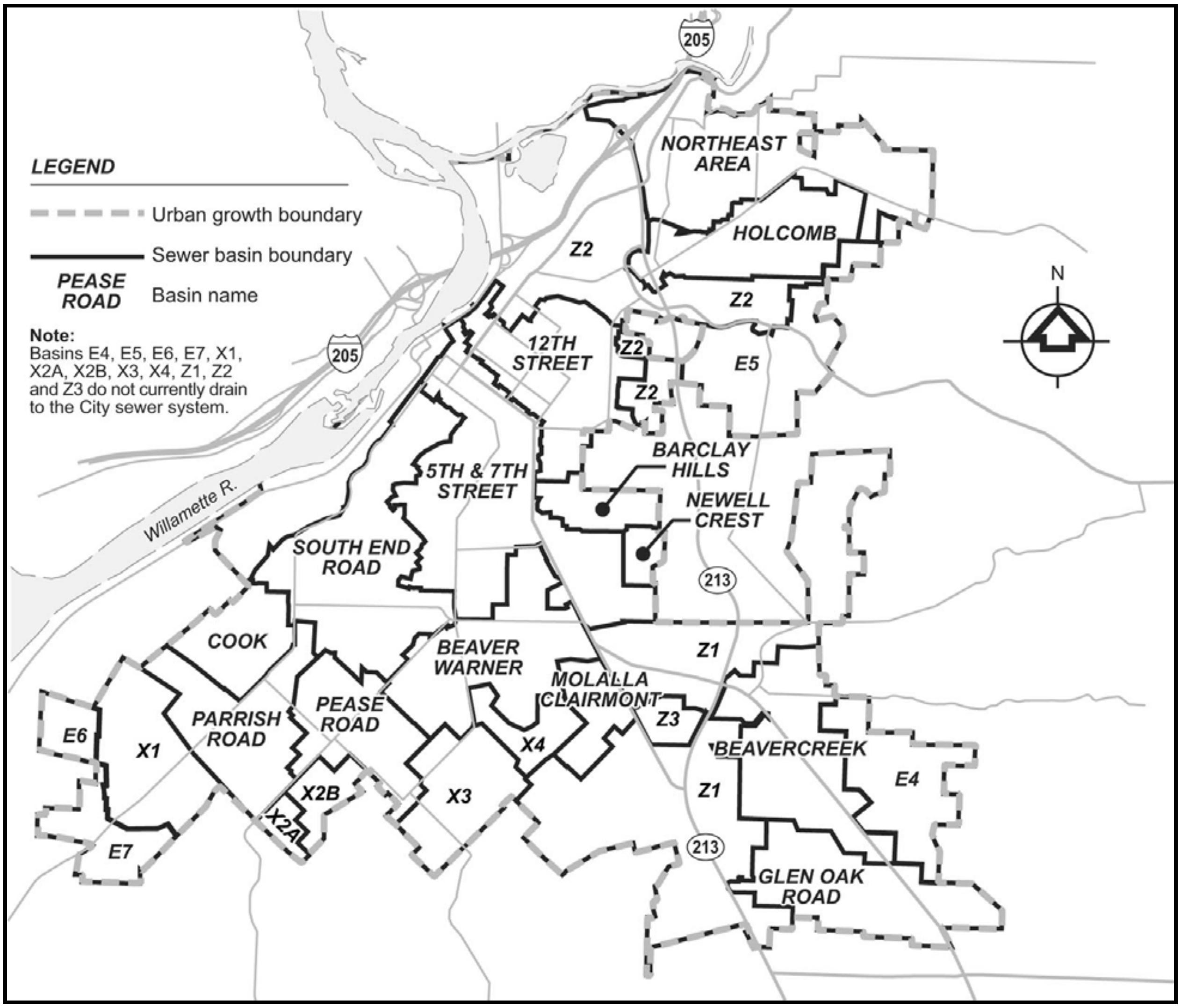


Figure ES-1. Master Plan Study Area and Sewer Basins

Evaluation of Facilities' Conditions

Previously reported problems in the collection system consist primarily of areas that experience high infiltration and inflow, grease accumulations and more frequent maintenance. These areas are shown in Figure ES-3. The pump stations are generally in good condition.

EXISTING AND FUTURE WASTEWATER FLOWS

Facility improvements recommended in this master plan are sized to accommodate the estimated peak wastewater flow at the "buildout" population, which is calculated as the expected population if all currently undeveloped areas within the expanded UGB are developed to the maximum density allowed by current zoning (no population change is assumed for currently developed areas). The buildout population is estimated to be 52,953.

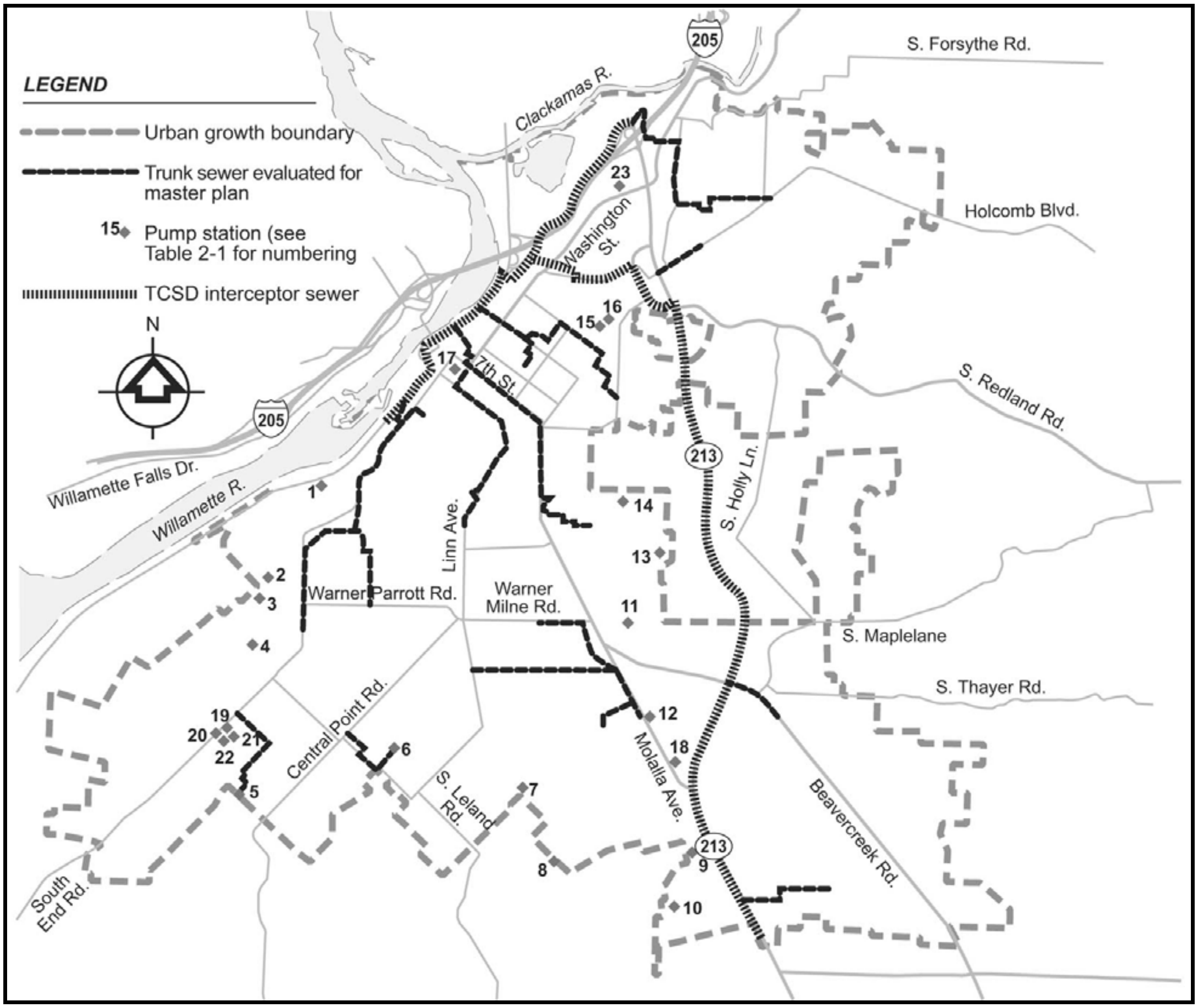


Figure ES-2. Existing Trunk Sewers and Pump Stations Evaluated for Master Plan

Flow estimates were developed by first determining the expected “unit flows” from the study area, representing the expected sewage flow per acre. Total flows for the existing conditions were estimated by applying the unit flows to areas that were developed as of June 2000; for buildout, the unit flows were applied to currently developed areas served by the City’s sewer system and to undeveloped areas within the UGB. For the analysis of flows, the basins shown in Figure ES-1 were divided into subbasins. Table ES-1 shows the existing and buildout flows in cubic feet per second (cfs) by subbasin.

EXISTING SEWER CAPACITY EVALUATION

Deficiencies in the City’s collection system were evaluated using a spreadsheet-based computer model that calculates flows for existing and buildout conditions and compares the flows to the capacity of each modeled pipe. Pipe segments whose calculated capacity is less than their predicted peak flow are identified in this report as “deficient” or “inadequate.” The findings of the capacity evaluation by basin are as follows: (See Exhibit 5 for location of described improvements)

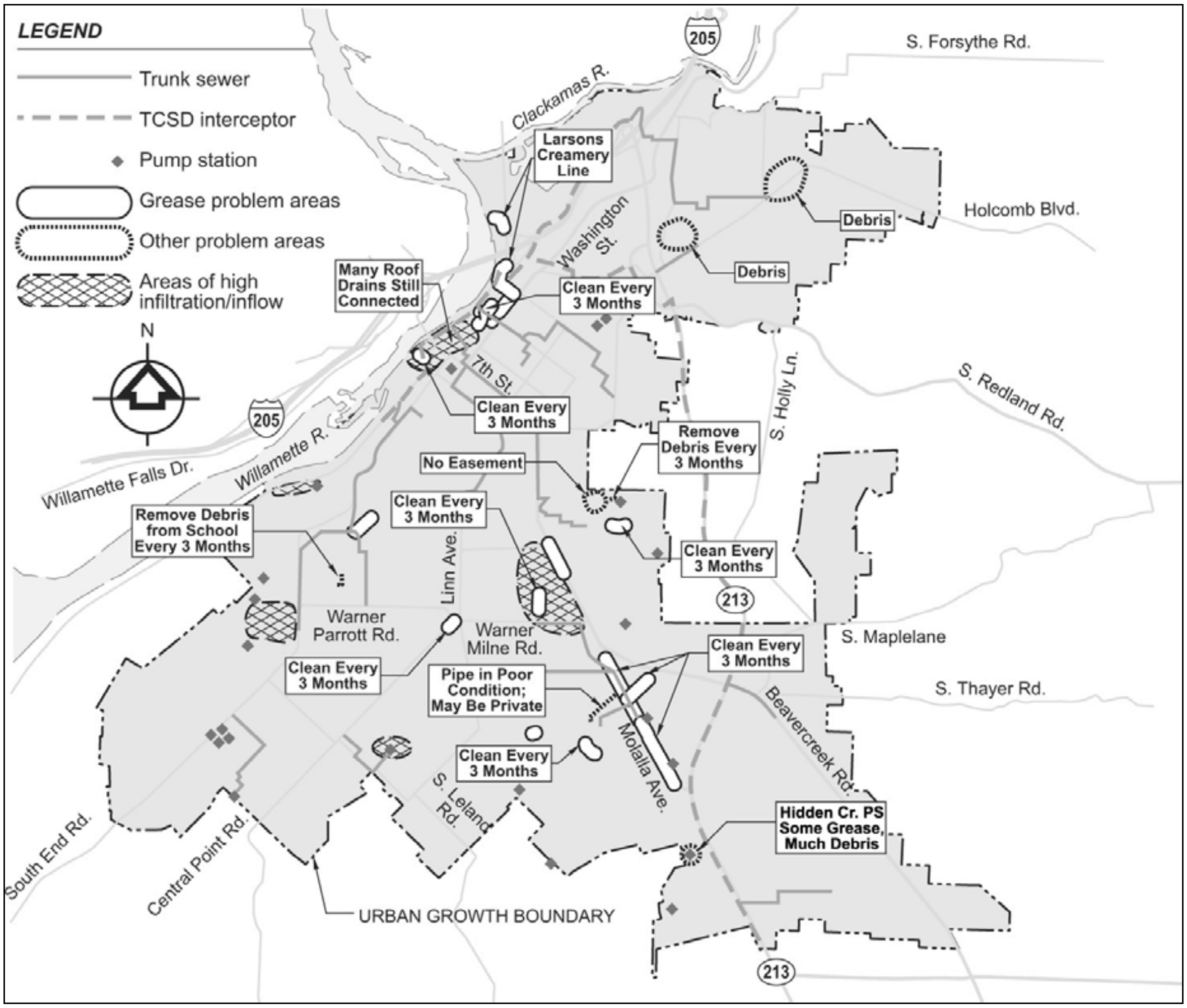


Figure ES-3. Locations of Known System Problems

- Northeast Area—No deficiencies requiring improvement were identified. The model identified some manholes that will experience slight surcharging under buildout conditions, but the surcharging will not cause overflows.
- Holcomb Basin—Under buildout conditions, the 12-inch pipe reach between this basin's Manholes 2 and 3 will be deficient and will have to be upsized to 15-inch pipe; the 10-inch pipe reach between Manholes 4 and 7 will be deficient and will have to be upsized to 12-inch pipe.
- Beavercreek Basin—Surcharging that will occur at buildout in the reach between Manholes 4 and 5 was addressed by replacing the 12-inch pipe between Manholes 1 and 5 with 15-inch PVC pipe as part of a street improvement project during the summer of 2003. Under buildout conditions, the entire modeled trunk system in this basin has deficient capacity. For this condition, the entire existing 12-inch pipe trunk system between Manhole 1 and a manhole just south of John W. Loder Road should be replaced with 15-inch pipe.

TABLE ES-1.
ESTIMATED PEAK FLOWS BY SUBBASIN

Subbasin[a]	Estimated Flow (cfs) Existing (2000)	Estimated Flow (cfs) Buildout	Subbasin[a]	Estimated Flow (cfs) Existing (2000)	Estimated Flow (cfs) Buildout
E4	0.00	2.45	MC5A6	0.48	0.52
E5	0.00	2.97	NE7	0.32	0.37
E6	0.00	0.61	NE21	1.25	1.64
E7	0.00	1.29	NE28	0.25	0.35
BC9	0.76	1.30	NE31	0.43	0.43
BC11	1.11	3.48	NE37	0.43	0.56
BW10	0.81	1.31	NE39	0.71	1.80
BW18	1.29	1.75	PA10	1.53	1.91
X2B	0.00	0.63	PA18	1.40	1.72
BW3A6	0.26	0.26	PE1	0.20	0.84
BW3A11	1.23	1.78	PE10	1.33	1.49
FS9	0.39	0.39	SE13	0.65	0.82
FS26	1.20	1.20	SE19	0.17	0.17
FS43	1.27	1.91	SE32	1.59	1.59
BH1	1.00	1.26	CO1	1.44	1.97
NC1	0.19	0.19	X1	0.00	2.46
FS8A3	0.35	0.35	X2A	0.15	0.39
FS8A6	0.33	0.33	SE5A4	0.28	0.31
FS8A16	0.46	0.46	SE5A5	0.51	0.51
FS8A21	0.77	0.77	SE19A4	0.81	0.81
FS8A28	1.42	1.64	SE19A8	0.35	0.35
FS8A36	2.56	2.56	SE19A12	2.30	2.54
GO10	0.09	0.83	TW8	0.96	0.96
GO11	0.69	1.98	TW13	0.28	0.28
HO7	0.52	1.67	TW28	1.99	2.11
HO8	0.36	0.93	TW36	1.61	1.68
MC3	1.00	1.00	TW10A3	1.14	1.27
MC7	1.06	1.16	TW10A6	0.93	0.93

a. Subbasin names indicate the basin they belong to (as shown in Figure ES-1) using the following codes: BC = Beavercreek; BH = Barclay Hills; BW = Beaver Warner; CO = Cook; FS = 5th & 7th Street; GO = Glen Oak Road; HO = Holcomb; MC = Molalla Clairmont; NC = Newell Crest; NE = Northeast Area; PA = Parrish Road; PE = Pease Road; SE = South End Road; TW = 12th Street; X = Undeveloped areas; Z = Tributary to Tri-City Interceptor

- Glen Oak Basin—Under buildout conditions, capacity is deficient in the pipe reach between Manholes 6 and 7, where the existing 10-inch PVC pipe should be increased to a 12-inch pipe.
- Molalla Clairmont Basin—Under buildout conditions, capacity is deficient in the pipe reach between Manholes 2 and 4, where the existing 10-inch pipe should be increased to a 12-inch pipe.
- Beaver Warner Basin—Under existing conditions, modeling shows some minor surcharging between Manholes 3 and 3A2, and significant deficiencies in the reach between Manholes 3A4 and 3A11 along Warner

Milne Road. The modeling predicts an overflow at Manhole 3A11. These conditions worsen under buildout conditions. All of the old 8-inch pipe between Manhole 3A4 and 3A11 should be replaced in the short term with 12-inch pipe, which will provide capacity for buildout conditions. From Manhole 3 to Manhole 3A2, the existing 10-inch pipe should be replaced with 15-inch pipe at a later date. Flow monitoring for the entire line from Manhole 3 to Manhole 11, which the modeling shows to experience slight surcharging, is warranted to better define I/I.

- Basins X3 and X4 and Portions of Basin Z1—Under buildout conditions, minor surcharging of the Settlers Point discharge trunk may occur. Modeling of these areas assumed an I/I value of 3,300 gallons per acre per day, but flow monitoring indicates that I/I may be substantially less. As such, no improvements are recommended.
- Pease Road Basin—Under existing and buildout conditions, the system is hydraulically adequate and no improvements are recommended.
- Parrish Road Basin—Modeling of this basin used a level of I/I that is the same as used for all basins contributing to the South End Trunk, because no I/I data is available for the individual contributing basins; actual I/I in this basin is likely to be less than the modeled value, given the age and material of the sewer pipes in the basin. It is recommended that further study be done in this basin to determine the extent of any upgrades that may be required. If flow monitoring of the reaches from Manhole 1 to Manhole 2 and from Manhole 7 to Manhole 10 indicate high levels of I/I, the existing 12-inch PVC pipe in these reaches should be replaced with 15-inch pipe for buildout conditions.
- South End Road Basin (evaluation includes Cook basin)—Although the modeling indicates slight surcharging in some pipe segments for existing and buildout conditions, the surcharging is not enough to warrant improvements. To verify the modeling findings, flow monitoring is needed to better define the distribution of I/I in the South End Road basin.
- 5th and 7th Street Basin (evaluation includes Barclay Hills and Newell Crest basins)—Although the modeling indicates slight surcharging in some pipe segments for existing and buildout conditions, the surcharging is not enough to warrant improvements.
- 12th Street Basin—Although the modeling indicates slight surcharging in some pipe segments for existing and buildout conditions, the surcharging is not enough to warrant improvements. Several cross connections with the storm drainage system should be eliminated, necessitating the replacement of approximately 400 feet of storm drainage pipe.

EXISTING PUMP STATION EVALUATION

There are 20 active pump stations serving Oregon City. Six of these are septic tank effluent pumping (STEP) stations, each serving only one residence. Reliable capacity of the City's 14 pump stations is defined as the capacity with the highest-capacity pump out of service. Because detailed information on pump and system curves was not available to allow a precise calculation of the total capacity when both pumps are pumping, total capacity was estimated to be 1.5 times reliable capacity. Pump stations are identified as deficient if the estimated peak flow for buildout conditions exceeds reliable capacity or if their condition warrants improvement. Table ES-2 summarizes the findings of the evaluation of active pump stations and the recommended improvements.

In addition to the recommendations for existing pump stations, a new pump station is recommended to serve a low area along Glen Oak Road. Gravity lines to the pump station site and a force main to Highway 213 were installed as part of the 2003 Glen Oak Road improvements project to avoid future trenching of the newly installed pavement section.

TABLE ES-2.
SUMMARY OF PUMP STATION EVALUATION FINDINGS AND RECOMMENDATIONS

		Estimated Peak Flow (gallons/minute)		
Pump Station[a]	Reliable Capacity (gpm)	Existing Conditions	Buildout Conditions	Comment
1. Canemah	1,200	560	700	No improvement needed
2. Amanda	100	80	80	Amanda and Riverview stations should be replaced with a single new station
3. Riverview	30	40	40	
4. Cook Street	400	720	890	Capacity upgrade required in short term
5. Parrish Road	1,100	1,320	1,630	Study required to better define peak flows
6. Pease Road	500	680	1,330	Capacity upgrade and relocation required
7. Settlers Point	830	850	1,250	Study required to better define peak flows
8. Nobel Ridge	140	160	170	No improvement needed
9. Hidden Creek	400	420	540	Capacity upgrade required in short term
10. Brendon Estates	100	40	40	No improvement needed
11. Hilltop	95	80	80	Condition warrants replacement or abandonment
13. Newell Crest	120	92	92	Permanent standby generator needed in short term
14. Barclay Hills	300	500	560	Capacity upgrade required in short term
15 - 16. 18th Street STEP Systems 1 & 2	STEP Systems, Flow Not Estimated			No improvement recommended
17. Elevator	Serves Single Building, Flow Not Estimated			No improvement recommended
19 - 22. Step Systems 1 - 4	STEP Systems, Flow Not Estimated			No improvement recommended

a. Pump Stations 12 (Fir Street) and 18 (Fire Station #2) are not listed because they have been abandoned.

CAPACITY, MANAGEMENT, OPERATION AND MAINTENANCE REGULATIONS

Impending federal Capacity, Management, Operation and Maintenance (CMOM) regulations will require public sewer collection system operators to take steps to eliminate sanitary sewer overflows. The CMOM regulations, which could be finalized and published in mid-2004, will require the City to establish performance standards for operating the sewer collection system and provide for monitoring.

The City of Oregon City has many of the elements of a CMOM program currently in place or in the process of being developed. The adoption of this master plan will meet many of the requirements of these regulations. A review and possible enhancement of these elements may be necessary. It is recommended that the City assign staff to monitor the U.S. Environmental Protection Agency's final adoption of the rule and Oregon's approach to permitting and enforcement, and ultimately to oversee the City's compliance.

CAPITAL IMPROVEMENT PLAN

A capital improvement plan (CIP) was developed incorporating recommended projects with estimated costs and proposed phasing. The projects in the CIP consist of upsizing existing sewer pipes, installing new sewer pipes, upgrading or replacing existing pump stations, performing annual operation and maintenance activities, and undertaking one-time studies and purchases. Three planning periods were identified for the purpose of project phasing: short-term (0 to 5 years); intermediate-term (6 to 10 years); long-term (11 to 20 years). Table ES-3 summarizes the proposed phasing and estimated cost for each type of project.

TABLE ES-3.
COST SUMMARY FOR ALL RECOMMENDED PROJECTS

	Total Costs			**Total for All Planning Periods**
Project type	Short-Term	Intermediate-Term	Long-Term	
Sewer Capacity Upgrade	$735,000	$1,480,000	—	**$2,215,000**
Sewer System Extensions	$1,762,500	$1,762,500		**$3,525,000**
Pump Station Improvements	$1,745,000	$1,405,000	—	**$3,150,000**
One-Time Studies and Purchases	$140,000	$0	$200,000	**$340,000**
Annual Activities	$1,730,00	$1,730,000	$3,460,00	**$6,920,000**
Total for All Projects	**$4,384,2300**	**$6,377,500**	**$203,460**	**$16,150,000**

CHAPTER 1.
INTRODUCTION

BACKGROUND

The City of Oregon City's last sanitary sewer master plan was completed in 1989. Since then, the City has experienced population growth of more than 50 percent and implemented significant sewer system improvements to address that growth. Because of the changes since the last master plan, an update is needed to fully document and evaluate the City's current sewer system, identify existing deficiencies and improvements needed for future growth, and outline a capital improvement program to provide adequate sewer service over a 20-year planning period.

AUTHORIZATION

The City of Oregon City contracted with Tetra Tech/KCM, Inc. in December 2000 to prepare an updated sanitary sewer master plan. This report represents the updated plan prepared under that agreement and documents the work performed under the agreement.

PURPOSE AND SCOPE

The purpose of this sanitary sewer master plan is to evaluate the City of Oregon City's sanitary sewer system and recommend improvements as needed to meet the City's wastewater collection needs now and through a 20-year planning period. Preparation of the master plan involved the following work elements:

- Document the existing sewer system—This work included the review of record drawings and other existing data, such as the findings of previous video inspections of the sewers and records of past reported problems, as well as field surveying to supplement existing information. Information collected from the document review and surveying was used to update the City's AutoCAD maps of the sewer system to represent the existing system.
- Assess existing and future wastewater flows—Estimates of existing flows and projections of future flows were developed based on past flow records, new flow monitoring conducted as part of the master plan effort, and projections of future population through the 20-year planning period.
- Evaluate sewer capacity and projected capacity needs—Based on the information developed on the existing sewer facilities and the wastewater flow projections, a hydraulic computer model was developed to evaluate the system's ability to convey expected flows with existing facilities and with a range of system improvement alternatives.
- Evaluate pump station capacity and condition—The pump stations that are part of the City's collection system were evaluated to identify improvement needs based on the stations' capacity or condition.

- Recommend improvements and prepare a capital improvement plan—From the identification of existing system deficiencies and evaluation of improvement alternatives, preferred improvements were recommended and incorporated into a capital improvement plan for implementation over the course of the planning period.

GENERAL POLICIES

The following City policies form the framework of the master plan:

- Oregon City will seek the most efficient and economic means available for constructing and maintaining the City's wastewater collection system and for providing wastewater treatment while protecting the environment and meeting state and federal standards for sanitary sewer systems.
- Oregon City will plan, operate, and maintain the wastewater collection system for all current and anticipated city residents within its existing urban growth boundary and adjacent urban expansion areas.
- Oregon City will work with Tri-City Service District to provide enough capacity in its collection system to meet standards established by the Oregon Department of Environmental Quality (DEQ) to avoid discharging inadequately treated sewage to surface waters.
- Oregon City will seek economical means to reduce inflow and infiltration of surface and ground water into its sanitary sewer collection system.

SIX-YEAR GOALS

The following six-year goals have been established for wastewater collection system:

- Pursue strategies with the Tri-City Service District to separate stormwater from the city's remaining cross-connections (discovered through recent smoke testing) and develop an infiltration and inflow (I/I) management program that is consistent with flow reduction guidelines established by the District.
- Develop strategies for extending sanitary sewer service to future Oregon City urban expansion areas.
- Develop and implement strategies to provide sanitary sewer service to unserviced areas within the City limits.
- Continue to inventory system integrity and model performance through ongoing inspection, flow monitoring, and video taping.
- Implement the Capital Improvement Program adopted with this master plan.
- Initiate activities in support of potential future regulations for Capacity, Management, Operations and Maintenance (CMOM), including, TV inspection, flow monitoring, and smoke testing to document existing system problems. Coordinate CMOM related activities with Tri-City Service District.

OVERVIEW

The sanitary sewer system evaluated for this master plan consists of collector sewers, trunk sewers, and pump stations within the City of Oregon City's urban growth boundary. Oregon City's collection system discharges into interceptors operated by the Tri-City Service District (TCSD) of Water Environment Services, a department of Clackamas County. Since the City's 1989 master plan was prepared, sewer service has been extended to approximately 34 additional subdivisions, and capital projects undertaken, including projects to separate sewage flows and stormwater flows that previously were conveyed together in combined sewers.

This report focuses on the trunk sewers and pump stations in the City's collection system; smaller sewers and the facilities operated by TCSD are not addressed.

CHAPTER 2.
EXISTING CONDITIONS

STUDY AREA

Oregon City is located in Clackamas County, Oregon (see Figure 2-1). Urban development beyond the current city limits will be contained within the City's urban growth boundary (UGB). The current 5,456-acre (8.5-square-mile) area within the UGB includes four small areas totaling 732 acres (1.1 square miles) that were added by expansion of the UGB in 2003. The study area for this master plan consists of the entire area inside the expanded UGB. Figure 2-2 shows the city limits, current UGB, and areas included within the UGB in 2003. A more detailed map of the area is provided in Exhibit 1 at the back of this report.

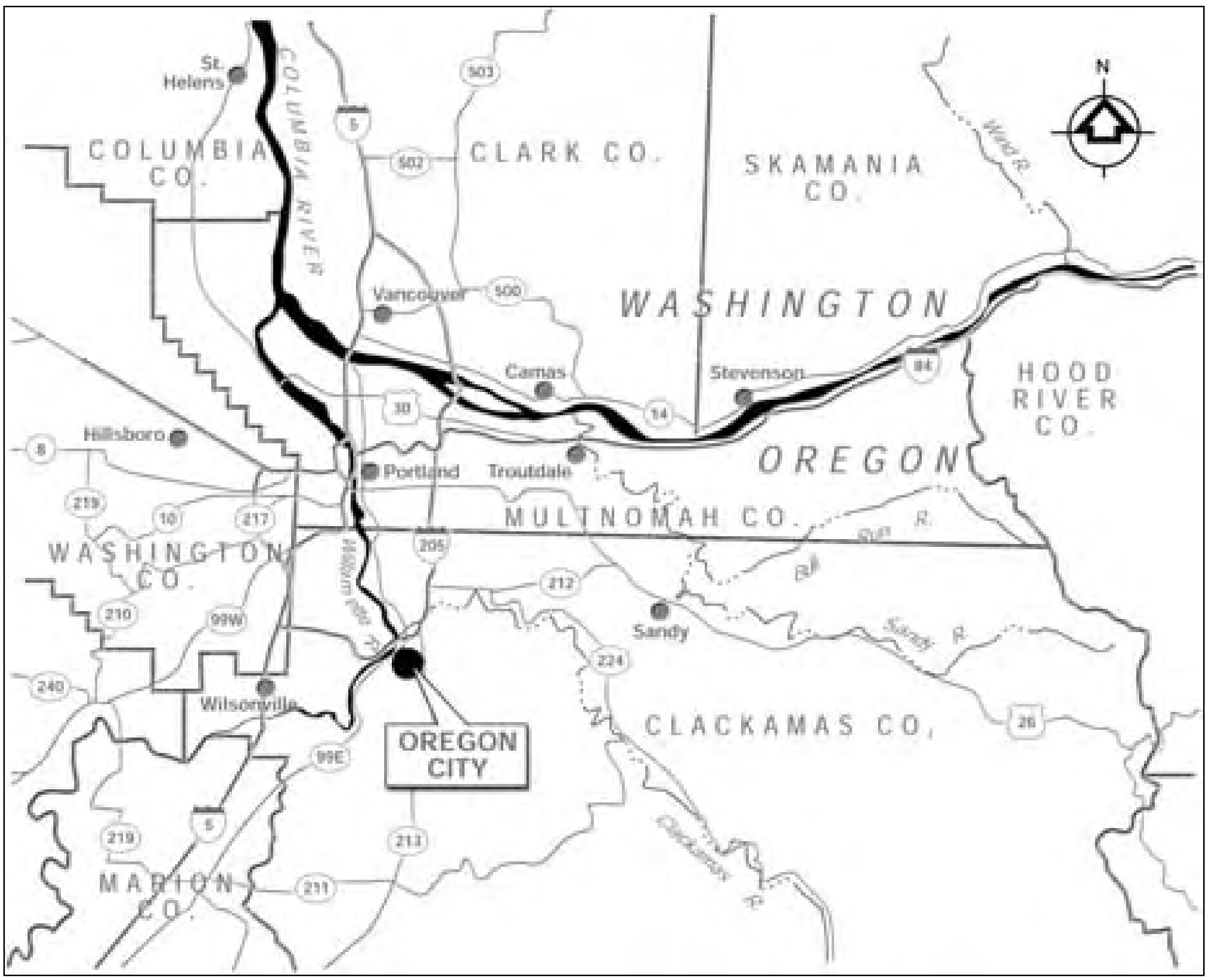


Figure 2-1. Study Area Vicinity

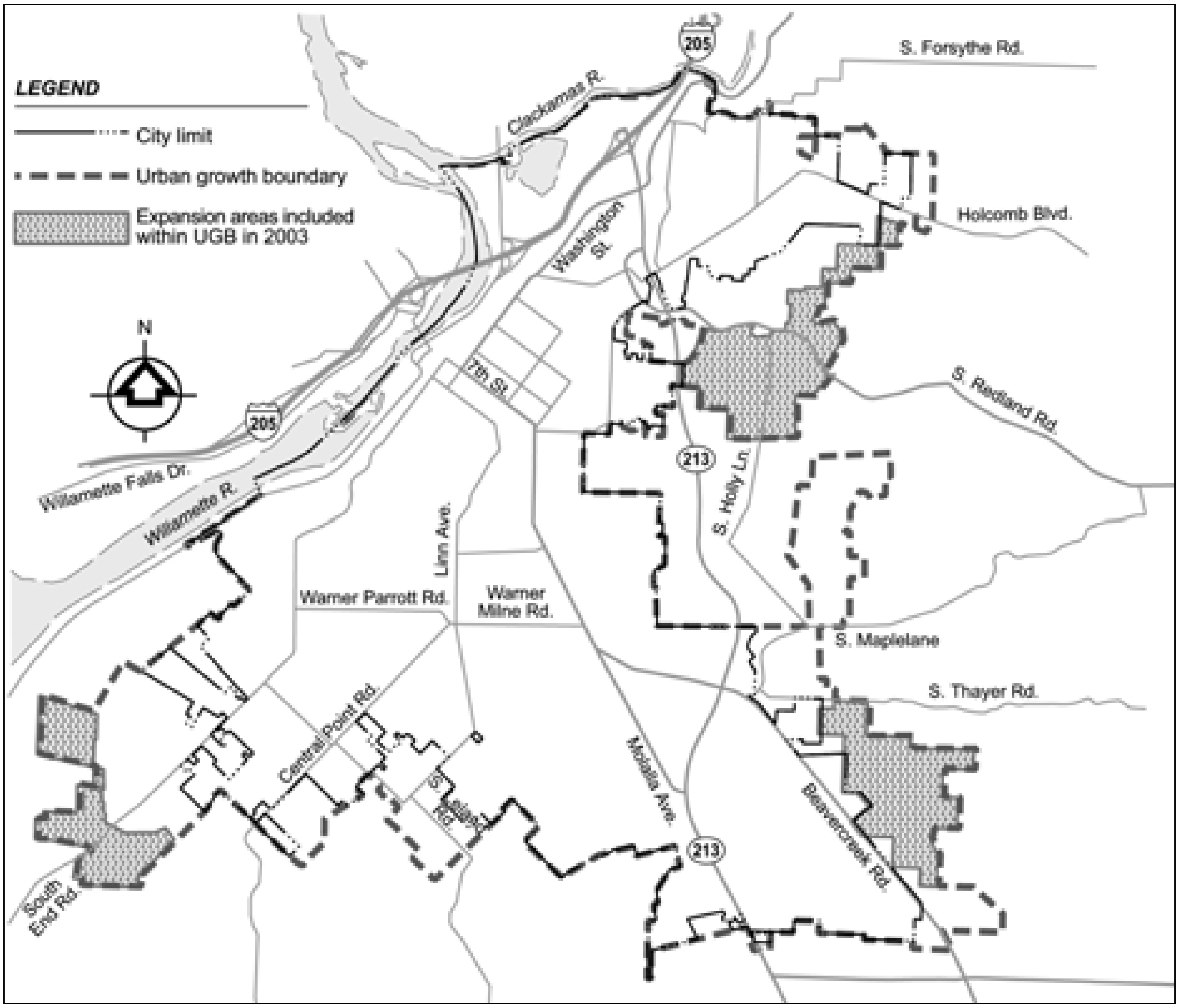


Figure 2-2. City Limits, Urban Growth Boundary and Expansion Areas Included in 2003

Topography

Oregon City lies at the confluence of the Clackamas and Willamette Rivers. The City is characterized by mild to relatively steep topography, with ground elevations ranging from approximately 20 feet near the confluence of the rivers to approximately 480 feet near the intersection of Warner Milne Road and Linn Avenue.

Climate and Rainfall

The study area's climate is moderate, primarily affected by humid maritime air masses with occasional arrival of continental air masses from the east. As a result, Oregon City has mild, wet winters and warm, dry summers. Average minimum winter temperatures are in the mid-30s with extremes seldom falling below 0ºF. Average maximum summer temperatures are in the low 80s with extremes rarely reaching above 100ºF. Average annual rainfall is about 48 inches, with most of the precipitation falling between October and March. Figures 2-3 and 2-4 show average temperatures and rainfall by month.

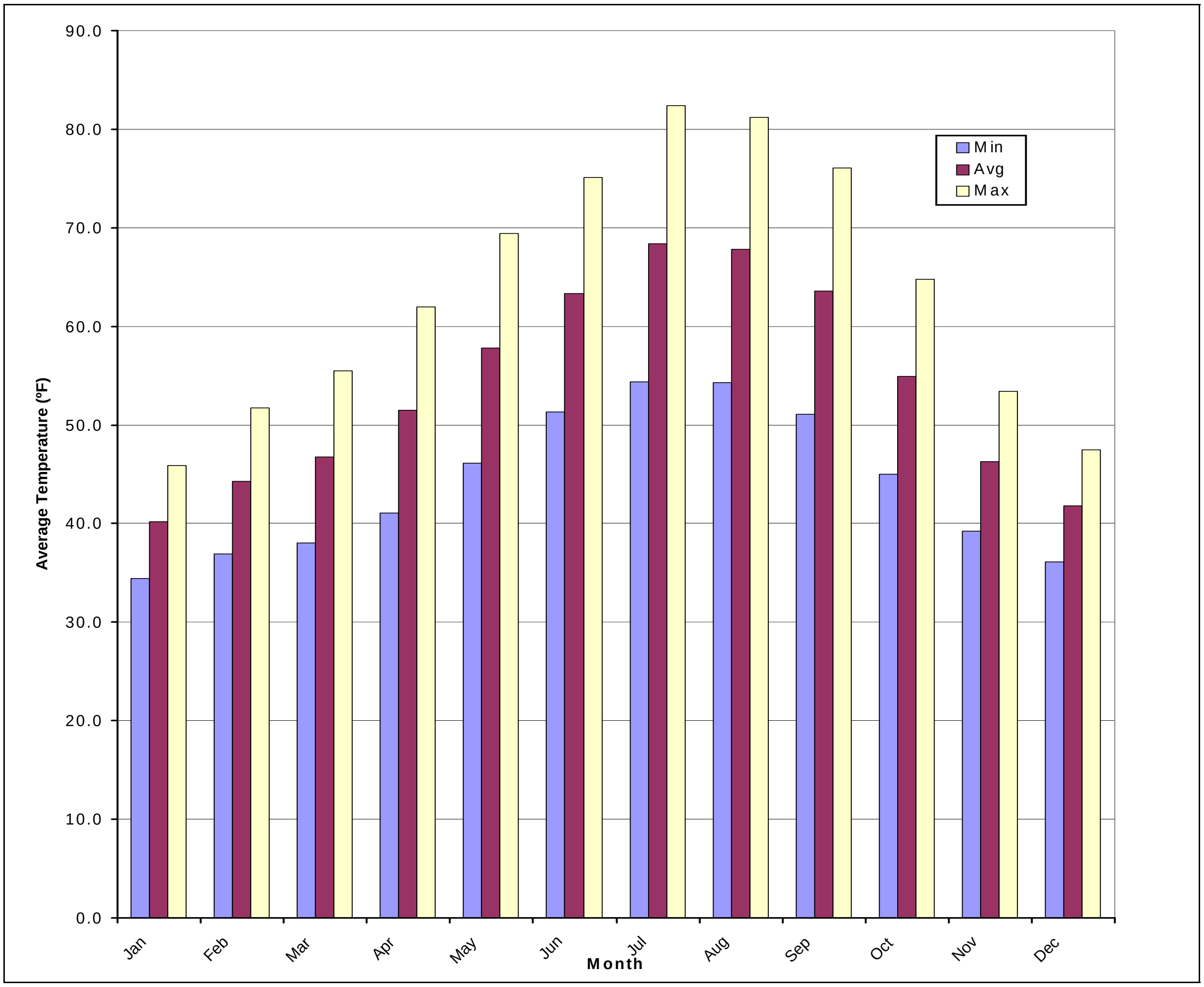


Figure 2-3. Average Oregon City Temperature by Month

Land Use

Current land use in Oregon City is primarily urban residential development with associated commercial and industrial uses. The zoning used for this master plan is shown on Exhibit 2 (inserted at the back of this report). This zoning was provided by the City of Oregon City.

WASTEWATER COLLECTION SYSTEM

Background and History

Oregon City's sewer system was originally constructed around 1900 and has been continually expanded since then. Many of the earlier sewers were combined sewers built to convey both sanitary sewage and stormwater runoff. Flows from the system were discharged directly into the Willamette River until the late 1940s when the original Willamette Interceptor and the old Oregon City sewage treatment plant were constructed.

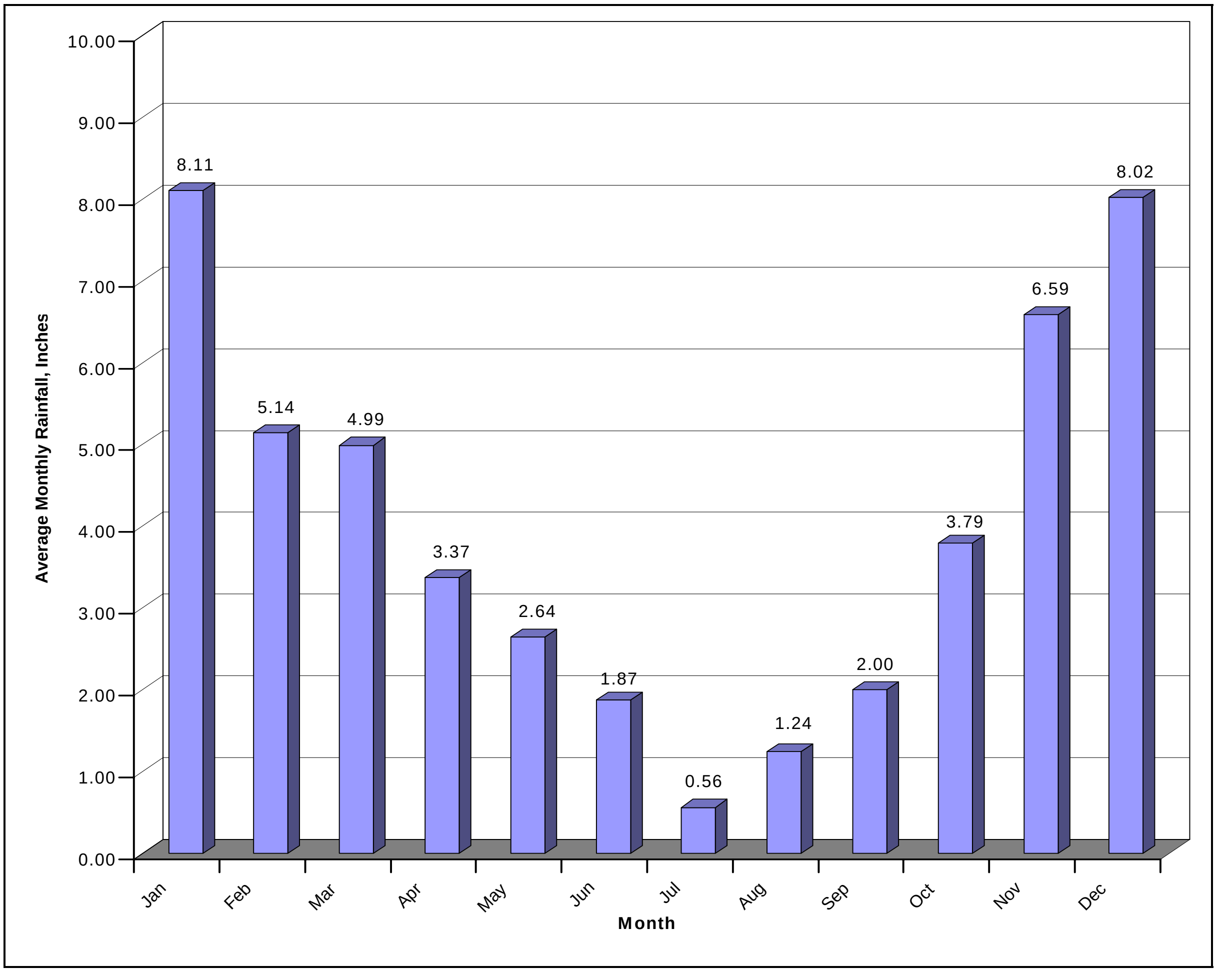


Figure 2-4. Average Oregon City Rainfall by Month

The Tri-City Water Pollution Control Plant was constructed in Oregon City in 1984-85 to provide regional treatment for Oregon City, Gladstone and West Linn. In conjunction with the new treatment plant, interceptor sewers were constructed to collect flow from the City's trunk and collector sewers and convey it to the new treatment plant. The treatment plant and interceptor sewers are operated by Clackamas County's Tri-City Service District (TCSD).

Subsequent to construction of the new treatment plant and interceptor sewers, the City implemented numerous projects to separate the old combined sewers into separate storm sewers and sanitary sewers. Expansion of the system also continued, to provide sewer service to newly developed areas.

Sewers

The sewer system consists of the following general types of sewers:

- Collectors—These pipes, the smallest units in the collection system, convey sanitary waste from residential developments, commercial or industrial areas, and point sources to the trunk sewers. A collector sewer serves smaller areas or subbasins. House services are connected to the collectors.

- Trunk sewers—These sewers make up the intermediate portion of the collection system. Trunk sewers convey wastewater flows from the collectors to the interceptor system. A trunk sewer normally serves a single basin defined by topography.
- Interceptor sewers—These sewers are the largest component of the collection system. Interceptors collect and accumulate flows from major service areas made up of many trunk sewer basins and convey the flow to the treatment plant.

Because the interceptors serving Oregon City are operated by the TCSD, they are not evaluated in this master plan. Collectors and laterals also are not addressed, as the master plan focuses on the larger, more critical trunk sewers. Figure 2-5 shows the trunk system evaluated for the master plan. The complete sewer system is shown on Exhibit 1 (inserted at the back of this report).

Pump Stations

Topography requires that pump stations be used to serve some areas of the City. There are 20 active pump stations serving Oregon City. Six pump stations are septic tank effluent pumping (STEP) stations, each serving only one residence. Table 2-1 lists the pump stations and their locations; locations are shown in Figure 2-5 and on Exhibit 1.

Basins

The existing wastewater collection system was divided into basins contributing flow to each trunk sewer. Basin boundaries were developed based on review of topographic mapping, existing collection system mapping, pump station locations, and the service areas defined in previous studies. Figure 2-6 shows the basin boundaries. Areas within the UGB that are not indicated as being in a defined basin either are undeveloped and unsewered or are served directly by the TCSD interceptor.

Evaluation of Facilities' Condition

TV Inspections of Sewers

Fourteen videotapes of TV inspections of selected City sewer pipes were reviewed for this master plan. The extent of the TV assessment is shown on Figure 2-7, on Exhibit 3 and in the figures in Appendix A. Findings of the TV inspections were entered into two databases and connected to a geographic information system (GIS). One of the databases listed the general condition of each pipe between manholes and one illustrated specific irregularities and their location along the pipe. This information can help City staff determine whether an entire pipe requires replacement due to overall poor condition or if only a specific section of pipe needs repair. Table 2-2 describes the type of information collected from the review of videotapes. Three pipe segments identified by the inspection as being in urgent need of replacement are shown on Figure 2-7 and Exhibit 3. The complete list of findings is presented in Appendix A.

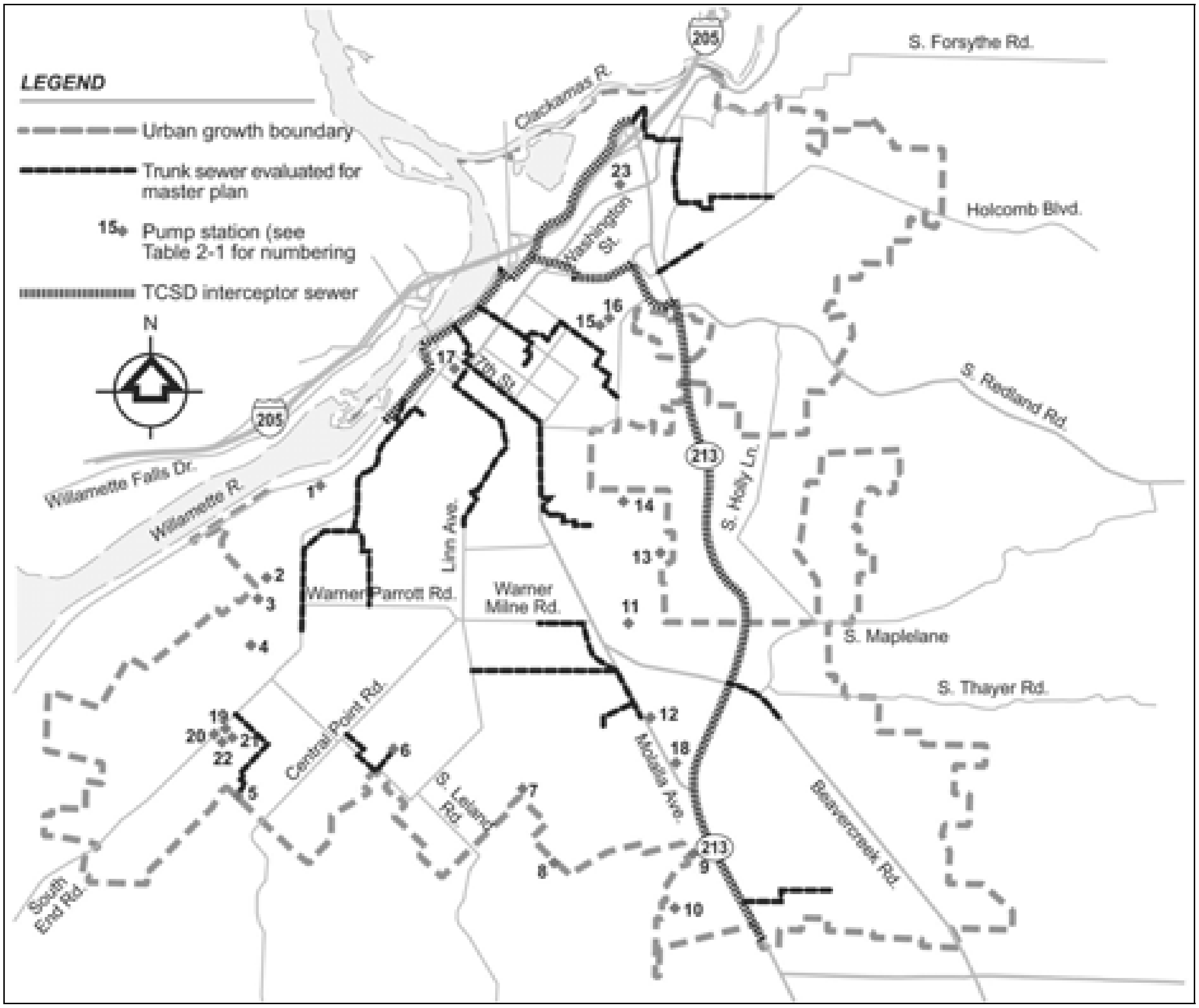


Figure 2-5. Existing Trunk Sewers and Pump Stations Evaluated for Master Plan

Reported Problems

Reported problems consist primarily of areas that experience high infiltration and inflow, grease accumulations or debris, that require more frequent cleaning. These areas are shown on Figure 2-8 and Exhibit 4 at the back of this report.

Pump Station Condition

The pump stations are generally in good condition. The City has a thorough routine maintenance and inspection regime. With the exception of the STEP stations, each pump station is inspected twice a week. Run-time readings are taken once a week.

TABLE 2-1.
PUMP STATION LOCATIONS AND DESCRIPTIONS

No.	Name	Address	Type	Number of Pumps
1	Canemah	410 S McLoughlin Blvd	Wet well/dry well	2
2	Amanda	275 Amanda Court	Suction lift Vacuum prime.	2
3	Riverview	287 Amanda Court	Grinder	2
4	Cook	18763 Cook St	Wet well/dry well	2
5	Parrish	11525 Parrish Rd	Wet well/dry well	2
6	Pease	19379 Pease Rd	Suction lift	2
7	Settlers Point	16460 S Meyers Rd	Suction lift	2
8	Nobel Ridge	13181 Gaffney Lane	Suction lift	2
9	Hidden Creek	19833 Hwy 213	Suction lift	2
10	Brendon Estates	13903 Conway Dr	Submersible on rails	2
11	Hilltop	708 Hilltop Ave	Suction lift, Vacuum prime.	2
12	Fir Street (abandoned)	19200 Molalla Ave	Suction lift	2
13	Newell Crest	18161 Newell Crest Dr	Wet well/dry well	2
14	Barclay Hills	17881 Peter Skene Way	Wet well/dry well	2
15	18th Street	1412 18th St	STEP	1
16	18th Street	1412 18th St	STEP	1
17	Elevator	610 Promenade	Grinder	1
18	Fire Station #2 (abandoned)	19340 Molalla Ave	Submersible	1
19	Step System #1	11501 Salmonberry Dr	STEP	1
20	Step System #2	11502 Salmonberry Dr	STEP	1
21	Step System #3	11520 Salmonberry Dr	STEP	1
22	Step System #4	11521 Salmonberry Dr	STEP	1

TREATMENT PLANT

The Tri-City Water Pollution Control Plant is owned and operated by the Tri-City Sewer District (TCSD) under Clackamas County jurisdiction and currently serves West Linn, Oregon City, Gladstone and part of Clackamas County. The plant is not evaluated for this master plan; TCSD's 1998 master plan covers the treatment plant.

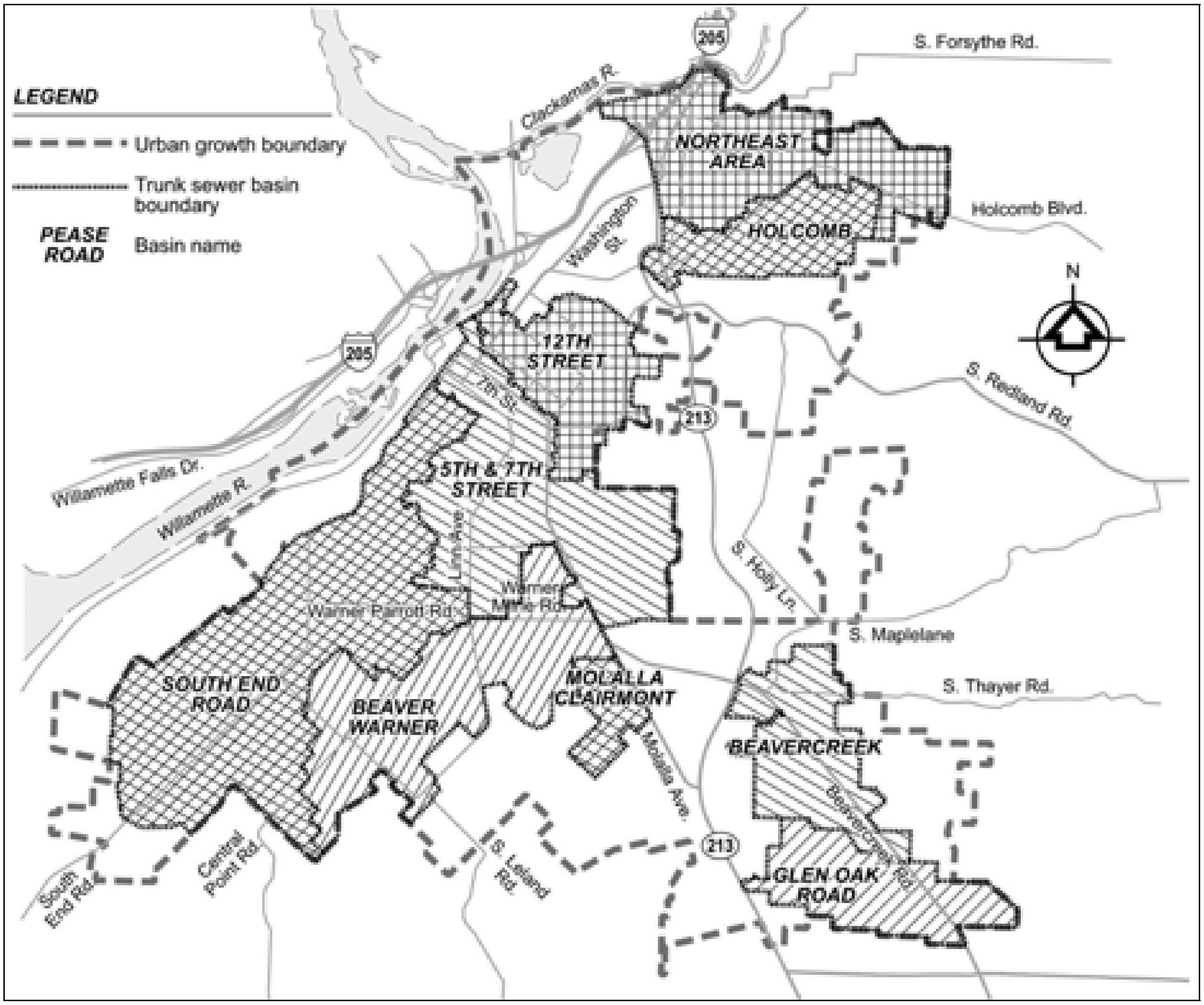


Figure 2-6. Trunk Sewer Basin Boundaries

TABLE 2-2.
TYPE OF INFORMATION COLLECTED FROM REVIEW OF TV INSPECTIONS

Condition	Assessment
Joint condition	Poor, Fair, good
Overall pipe condition	Poor, Fair, good
Pipe cracked?	Yes, No
Pipe broken?	Yes, No
Pipe material	Concrete, PVC, Clay
Pipe size	Actual diameter in inches
Tape number, Report number	Tape number and report number
Date Inspected	Date of inspection
Other comments	Flow, grease, roots, etc.

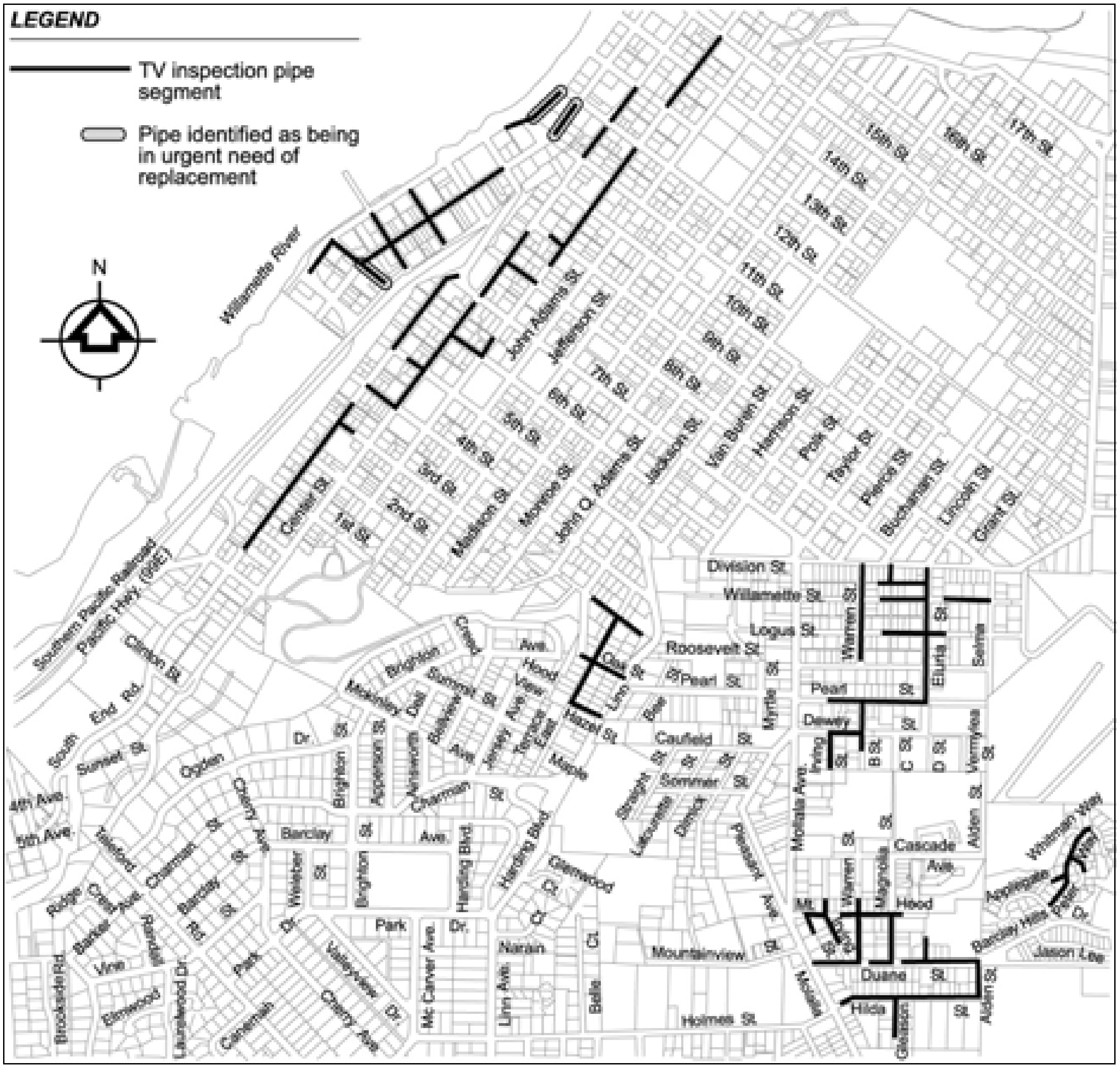


Figure 2-7. Extent of TV Sewer Inspection and Pipes in Urgent Need of Replacement

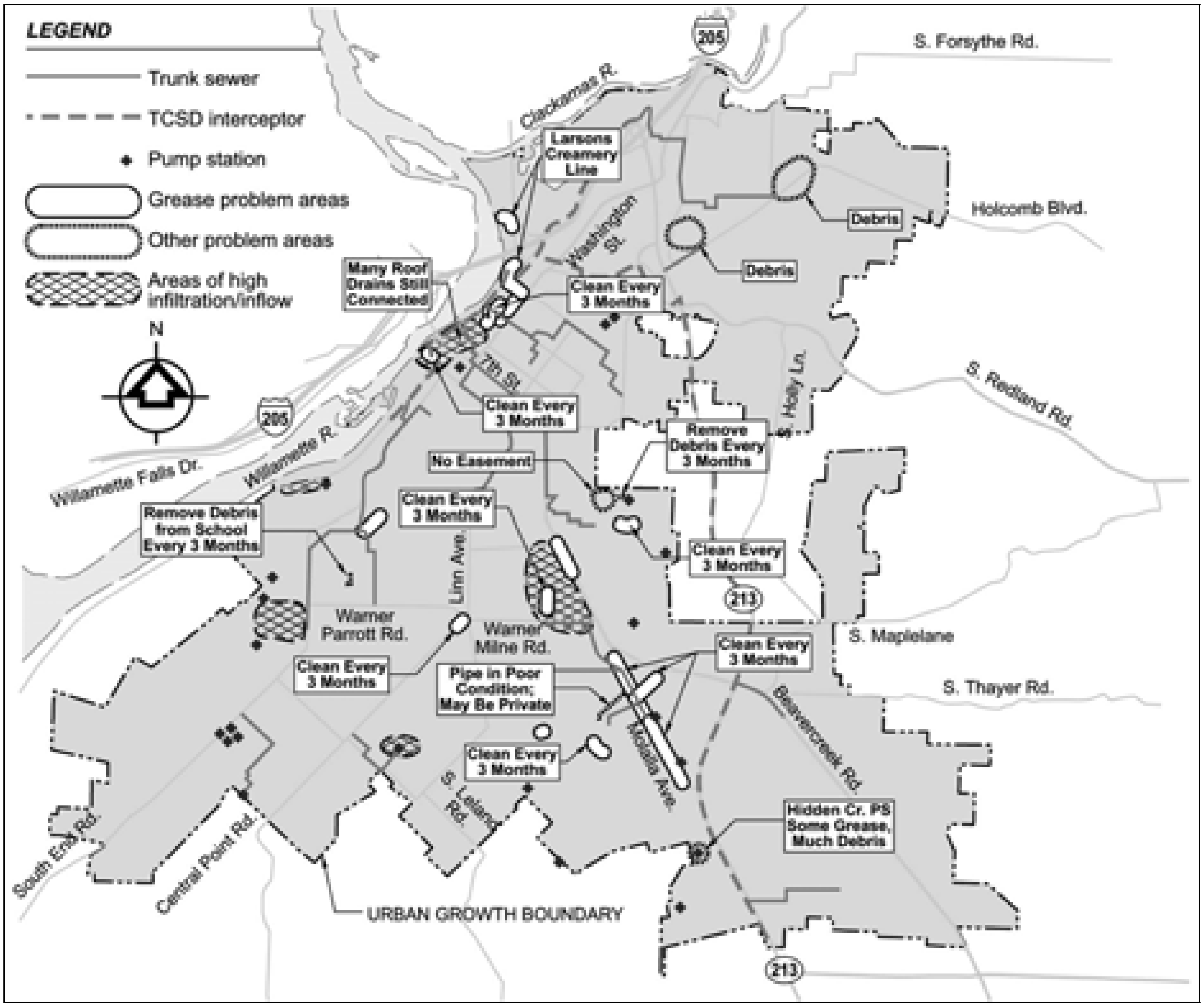


Figure 2-8. Locations of Known System Problems

CHAPTER 3.
EXISTING AND FUTURE WASTEWATER FLOWS

Estimates of wastewater flows are used to determine the required size of the facilities that must accommodate the flows. Trunk sewers, pump stations and force mains recommended in this master plan are sized to accommodate the estimated peak flow at the buildout condition (buildout is when the entire service area is developed to the maximum density allowed by zoning).

Wastewater flow consists of dry-weather flow and inflow and infiltration (I/I). Dry-weather flow is the flow directly from sewer users and is related to population density and the type of commercial and industrial activities in the service area. Inflow is surface water that enters the system through manhole covers, catchbasins, roof drains, and other stormwater sources connected to the sanitary sewer system. Infiltration is groundwater that enters the system through leaking joints, structural defects in pipes, and manholes.

To estimate existing and future flows, a detailed analysis was conducted of zoning, current development, topography, and flow monitoring records.

POPULATION

Wastewater flow is related to population and the type of commercial and industrial activities in the service area. Estimates of future population growth can be used to schedule capital improvements that will be needed to accommodate increasing flows as the population grows. Table 3-1 summarizes historical population figures for Oregon City.

TABLE 3-1.
HISTORICAL POPULATION

Year	Population	Annual Growth
2001	27,270	3.7%
2000	25,754	5.9%
1997	21,895	7.3%
1996	20,410	7.5%
1995	18,980	8.2%
1994	17,545	1.3%
1993	17,315	3.0%
1992	16,810	0.3%
1991	16,760	14.0%
1990	14,698	0.0%
1980	14,673	—

Source: U.S. Census for 2000, 1990 and 1980 data; Portland State University estimate for 2001; Web site webfoot.osl.state.or.us for all others.

Buildout Population

The buildout population is the expected population if the entire area within the UGB is developed to the maximum density allowed by current zoning. Zoning designations within the pre-2003 UGB are shown in Figure 3-1 and described in Table 3-2 (Exhibit 2, a larger more detailed presentation of zoning designations is included with the exhibits at the back of this report). Development density in recently annexed areas was assumed to be the same as the density in the RA-2 zone and density in county areas within the UGB was assumed to be the same as in the R-6 zone. Development within the 2003 UGB expansion areas was assumed to be 10 percent commercial (C), 10 percent industrial (M-1), 30 percent RA-2 and 50 percent R10.

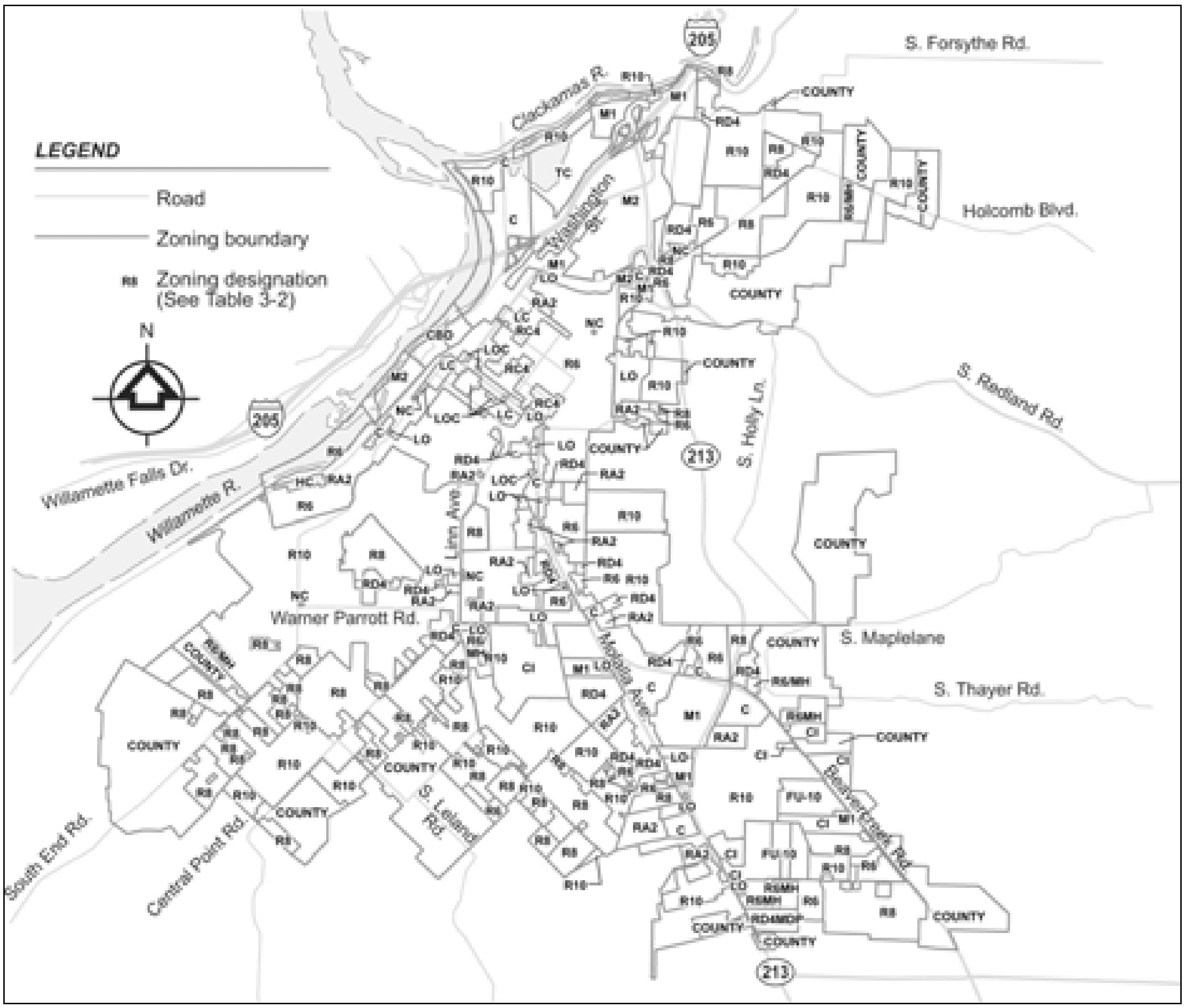


Figure 3-1. 2002 Zoning Designations Within the pre-2003 UGB.

TABLE 3-2.
ZONING DESIGNATION DESCRIPTIONS

Zone	Description	Zone	Description
C	General Commercial	R-6	Single Family Dwelling, 6,000-sq-ft min. lot
CBD	Central Business	R-6/MH	Single Family Dwelling, 6,800-sq-ft min. lot
LC	Limited Commercial	R-8	Single Family Dwelling, 8,000-sq-ft min. lot
LO	Limited Office	R-10	Single Family Dwelling, 10,000-sq-ft min. lot
LOC	Limited Office Conditional	RA-2	Multi-Family Dwelling
M-1	Light Industrial	RC-4	McLoughlin Conditional
M-2	Heavy Industrial	RD-4	Two-Family Dwelling
C-I	Campus Industrial	TC	Tourist
NC	Neighborhood Commercial	Recent Annex	Recent Annex (FU-10)
		County	County within UGB

To calculate the buildout population, the area of each residential zoning unit was determined. The total area in each zoning designation was then reduced by 25 percent to account for parking, infrastructure, etc. The developable areas were then divided by the minimum lot size allowed in each zone to determine the maximum number of dwelling units possible. An average of 2.3 residents per dwelling unit was assumed, as recommended by City staff. The buildout population was calculated to be 52,953.

Population Growth Rate

Table 3-3 summarizes the projected future population using an annual growth rate of 3 percent from the 2000 Census population of 25,754, as recommended by City staff. Growth at this rate will reach the buildout population described above in 2024, and no additional growth is projected beyond that level.

TABLE 3-3.
PROJECTED POPULATION GROWTH

Year	Population
2005	29,856
2010	34,611
2015	40,124
2020	46,515
2024 and Later	52,953 (buildout)

SEWER BASIN DEFINITION

Only the primary trunk sewers in the City's system were chosen for a capacity analysis. Smaller sewer laterals, which are generally sized to prevent blocking and are commonly oversized for expected flow, were not modeled for the collection system analysis.

For the analysis of flows, the trunk sewer basins described in Chapter 2 were divided into minor basins as shown in Figure 3-2. These basins are shown divided into subbasins in Exhibit 1 (included at the back of this report). Table 3-4 lists the trunk sewers associated with two-letter codes used in subbasin names. Several laterals were grouped together when defining the subbasins. In the modeling, flow was introduced near the upstream lateral of the group to allow for a conservative flow assessment of the trunk sewer.

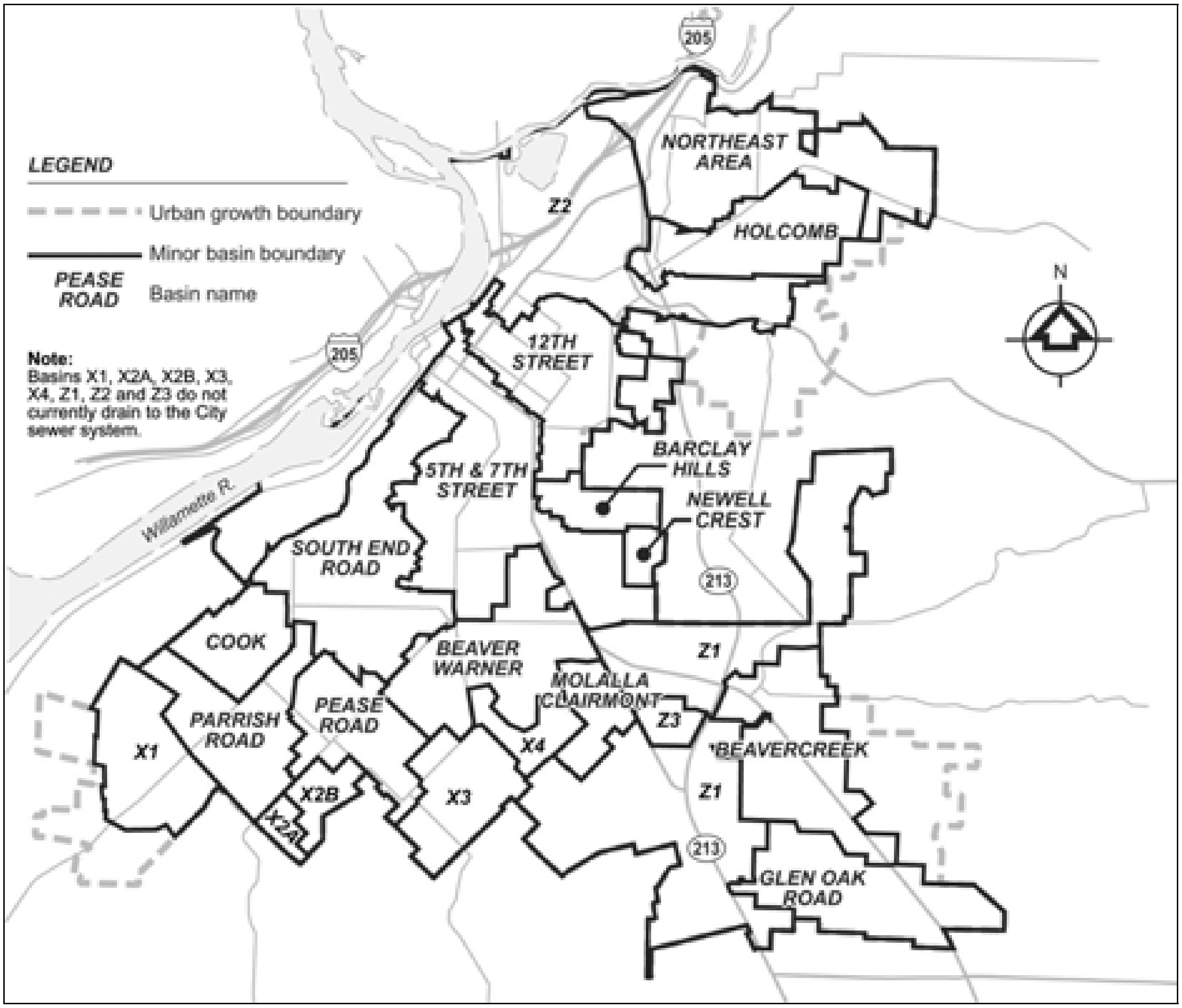


Figure 3-2. Basin Boundaries Used for Flow Estimating Analysis

Also shown on the figure are basins within the pre-2003 UGB that do not contribute flow to the City's sewer system: Basins Z1, Z2 and Z3 drain directly to the Tri-City Service District system, and Basins X1, X2A, X2B, X3 and X4 are currently undeveloped and so contribute no flow. In undeveloped areas, current basin boundaries are defined by natural drainage basin topography. The actual pattern of future development will dictate what portion of the collection system will be extended to serve these areas, and basin boundaries may shift accordingly. The analysis performed for this master plan assumed that Basins X1, X2A, and X2B will contribute flow to the City system when developed and Basins X3 and X4 will drain to the TCSD system.

TABLE 3-4.
SEWER TRUNK BASIN ABBREVIATIONS AND NAMES

Abbreviation	Trunk name	Abbreviation	Trunk name
BC	Beavercreek	NC	Newell Crest
BH	Barclay Hills	NE	Northeast Area
BW	Beaver/Warner	PA	Parrish Road
CO	Cook	PE	Pease Road
FS	5th & 7th Street	SE	South End Road
GO	Glen Oak Road	TW	12th Street
HO	Holcomb Road	X	Undeveloped areas
MC	Molalla/Clairmont	Z	Tributary to Tri-City Interceptor

UNIT FLOWS

Flow estimates were developed by first determining the expected “unit flows” from the study area. Unit flows were developed as the expected sewage flow per acre. These flows are largely defined by land use for dry-weather flows and by the condition of the sewer system for I/I.

Dry-Weather Unit Flows

Different land uses generate different dry-weather sewer flows. To estimate the dry-weather flows, each subbasin was divided into areas covered by different zoning designations, and flows were estimated for each designation.

For residential zones, lot density is the primary factor affecting population and the resulting dry-weather unit flows. City ordinances dictate minimum lot sizes. Buildout population per acre in the residential areas of each basin was determined as described in the discussion on population earlier in this chapter. From the resulting population estimates, average domestic sewage flows per acre were calculated on the basis of an average unit flow rate of 80 gallons per capita per day (gpcd). Peak flow rates were determined by multiplying the average flows by a peaking factor of 3.

For areas that are currently developed, actual density was estimated for each zone within each basin by measuring the area of a representative sample of existing lots and calculating an average lot size. Unit flows from these areas were then reduced by the ratio of actual density to maximum allowed density. Undeveloped areas were assumed to develop to the maximum allowed density.

Unit flows from commercial developments vary greatly, as they are dependent on the type of development. For example, restaurants generate much larger flows than retail stores. For the purpose of generating flows for the model, commercial zones were assigned a density equivalent to R-8 residential densities.

Unit flows from industrial areas can also vary significantly and depend on the type of industry. In some instances, industrial developments are not served by the municipal wastewater system. Industrial waste often has special treatment needs and sometimes has chemicals that are detrimental to municipal treatment plants. This study assumed industrial flows of 3,000 gallons per acre per day (gpad). As industrial areas are developed, the model should be updated to account for actual flow contributed by the development.

Table 3-5 summarizes estimated average dry-weather unit flow for each zoning designation.

TABLE 3-5.
AVERAGE DRY-WEATHER UNIT FLOWS BY ZONING DESIGNATION

	Lot Area (square feet)						
Zone[a]	Minimum Allowed by Zoning	Used for Modeling[b]	Lots per Acre	People per lot	People per acre	Unit Flow Contribution[c]	Daily Average Dry-Weather Unit Flow (gpad)[d]
Commercial[e] and Residential Zones							
C	8,000	10,667	4.08	2.3	9.4	80 gpcd	751
CBD	8,000	10,667	4.08	2.3	9.4	80 gpcd	751
LC	8,000	10,667	4.08	2.3	9.4	80 gpcd	751
LO	8,000	10,667	4.08	2.3	9.4	80 gpcd	751
LOC	8,000	10,667	4.08	2.3	9.4	80 gpcd	751
TC	8,000	10,667	4.08	2.3	9.4	80 gpcd	751
NC	6,000	8,000	5.44	2.3	12.5	80 gpcd	1,002
R-6	6,000	8,000	5.44	2.3	12.5	80 gpcd	1,002
R-6/MH	6,800	9,067	4.80	2.3	11.0	80 gpcd	884
R-8	8,000	10,667	4.08	2.3	9.4	80 gpcd	751
R-10	10,000	13,333	3.27	2.3	7.5	80 gpcd	601
RA-2	8,000	10,667	4.08	4.6	18.8	80 gpcd	1,503
RC-4	8,000	10,667	4.08	4.6	18.8	80 gpcd	1,503
RD-4	4,000	5,333	8.16	2.3	18.8	80 gpcd	1,503
County[f]	6,000	8,000	5.44	2.3	12.5	80 gpcd	1,002
Recent Annex[g]	8,000	10,667	4.08	4.6	18.8	80 gpcd	1,503
Industrial Zones[h]							
M-1	—	—	—	—	—	3,000 gpad	2,250
M-2	—	—	—	—	—	3,000 gpad	2,250
C-I	—	—	—	—	—	3,000 gpad	2,250

a. See Table 3-2 for description of zoning designations
b. The lot size used for analysis was increased from the minimum size to account for 25 percent undevelopable area.
c. gpcd = gallons per capita per day; gpad = gallons per acre per day.
d. The daily flow shown is for the maximum possible density; in areas that are currently developed, the daily flow was reduced to account for the difference between actual density and maximum allowable density. Daily average unit flow does not include a peaking factor.
e. Commercial zones were modeled as R-8 residential areas
f. County zones were modeled as R-6 residential areas.
g. Recently annexed areas were modeled as RA-2 residential areas.
h. Unit flows from industrial areas are based on an assumed 3,000 gpad standard flow with a reduction to account for 25 percent undevelopable area.

Infiltration and Inflow

Historical I/I Flows

Between 1987 and 1995 Oregon City undertook several major combined sewer separation projects to disconnect street storm water inlets from the sewer system (see Figure 3-3). These programs and similar programs in West Linn and Gladstone substantially reduced the peak flows and associated overflows to the Willamette and Clackamas Rivers that occurred during rainstorms. The volume of overflows at the Tri-City wastewater treatment plant for a 5-year storm was estimated by CH2M Hill to be reduced from 96 million gallons to 12 million gallons, an 88 percent decrease (CH2M Hill 1998).

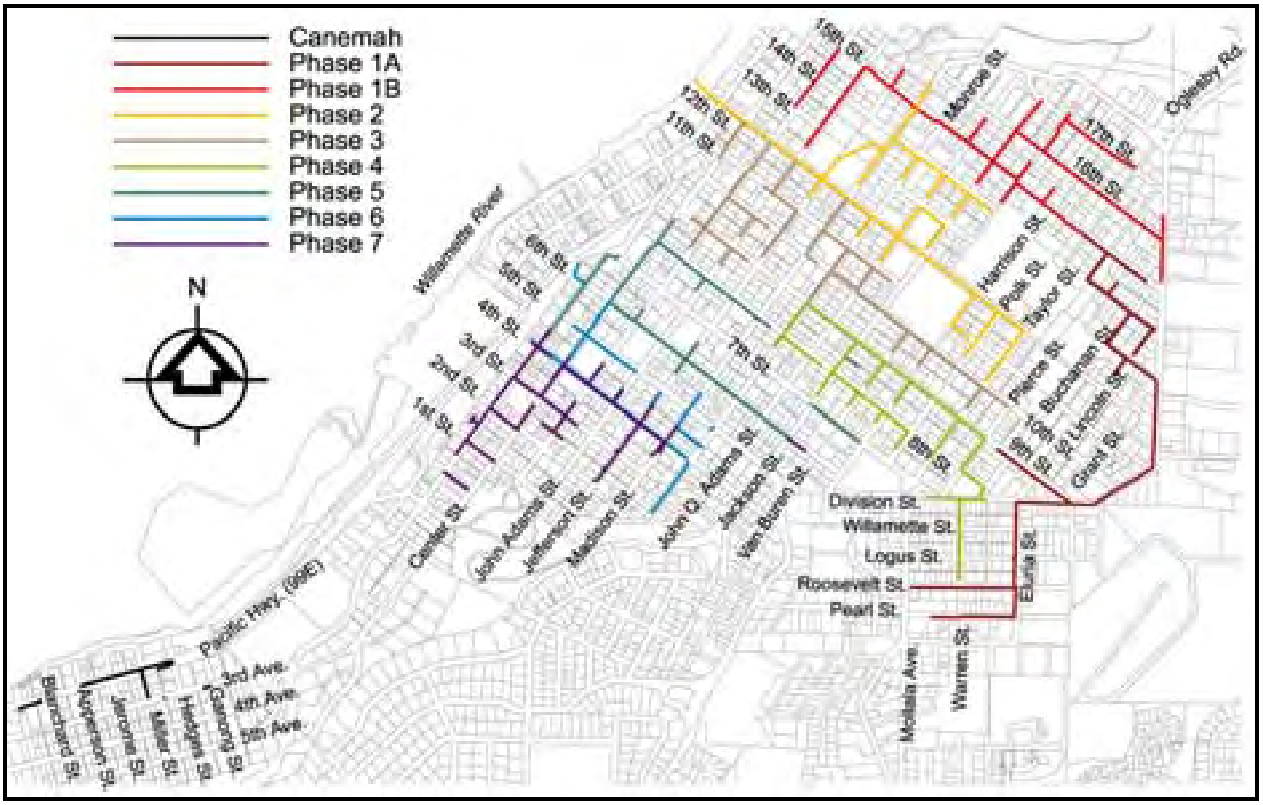


Figure 3-3. Previous Sewer Separation Projects in Oregon City

Despite the significant separation effort, I/I continues to be a problem in the City. During a severe rainstorm in 1996, the peak flow at the Tri-City treatment plant was measured at 10 times the average dry-weather flow even though street inlets had been disconnected from the sewer system. Oregon City will need to continue its current smoke testing program to determine localized problems and solutions. Although Gladstone and West Linn also contribute to these flows, both are newer cities with newer sewer systems, and it is likely that a relatively greater portion of the I/I is generated in Oregon City.

A large part of current rainfall-related inflow comes from roof downspouts, basement drains, yard drains and manhole covers. Some flow also comes from groundwater leaking into the main lines and customer service lines. This infiltration of groundwater is caused by deficiencies in the pipelines, such as misaligned pipe joints, porous walls, or damaged pipes.

Flow monitoring analysis shows that infiltration is not excessive under the definition established by the U. S. Environmental Protection Agency (EPA). The EPA defines infiltration as excessive if the average daily flow per capita (excluding major industrial and commercial flows) is 120 gallons per capita per day (gpcd) or more over a 7- to 14-day dry period during seasonal high groundwater. Wet-season flow data from the Tri-City treatment plant for 1996 were reviewed to identify dry-day flow levels. The review concluded that there was not a wet-season rise in dry-weather flows. An example of this occurred on March 29, 1996, when the Tri-City system received only 107 gallons per capita per day after storm runoff subsided. This means the increase in flows is due to direct connections (inflow) and not groundwater leaks (infiltration). Major infiltration control projects would not be cost-effective (CH2M Hill 1998).

Calculation of I/I Unit Flows

Flow monitoring data was used to determine current I/I flow rates. Limited data was available from Tri-City Service District flow monitors that recorded flows in 1997. Flows from only two Oregon City trunk basins—South End and Northeast—were recorded by the District's monitors. The City performed new flow monitoring throughout the City from January 22 to March 7 in 2001 and from January 10 to March 28 in 2002. Figure 3-4 shows the location of the Oregon City and Tri-City Service District flow monitors. Table 3-6 shows which subbasins were associated with each monitor for the flow estimating analysis.

TABLE 3-6.
SUBBASINS REPRESENTED BY FLOW MONITORS

Monitor Number	Subbasins Represented for Flow Estimating
Oregon City Flow Monitors	
SS 01	FS8A16, FS8A21, FS8A28, FS8A3, FS8A36, FS8A6
SS 02	FS26, FS43, BH1, NC1, FS9
SS 03	TW10A3, TW10A6, TW13, TW28, TW36, TW8
SS 04	HO7, HO8
SS 06	BC11, BC9
SS 07	GO10, GO11
SS 10	MC3, MC5A6, MC7
SS 11	BW10, BW18
SS 12	BW3A11, BW3A6
SS 13	PE1, PE10
Tri-City Service District Flow Monitors	
HOPS	NE21, NE28, NE31, NE37, NE39, NE7
A05674	PA10, PA18, SE13, SE19, SE19A12, SE19A4, SE19A8, SE32, SE35, X1, X2A, SE5A4, SE5A5

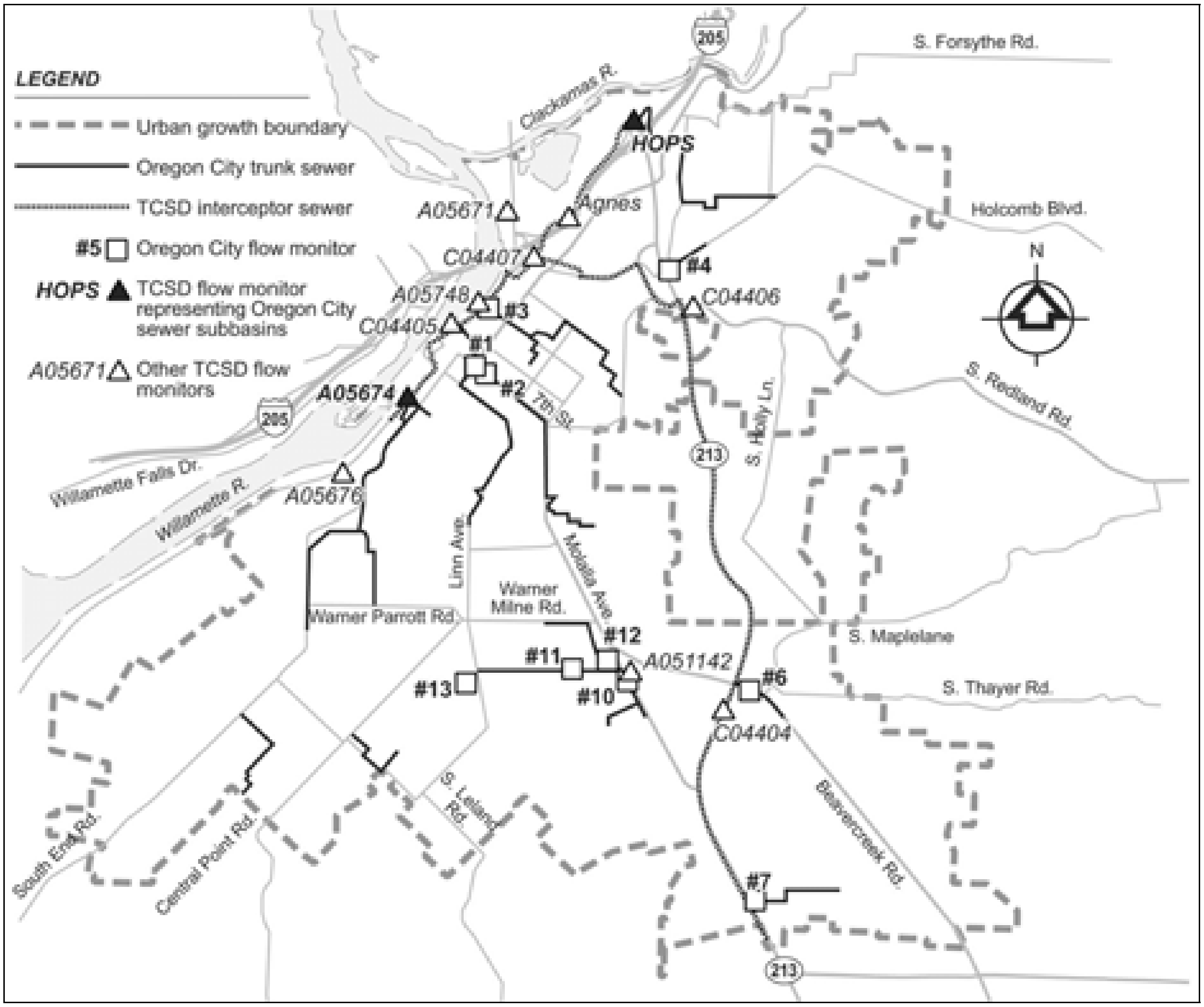


Figure 3-4. Flow Monitoring Locations

The flow monitoring data was reviewed and screened for false readings and corrected or deleted as necessary. Appendix B contains background data on rainfall and flow monitoring. The maximum flow rate was determined for each flow monitor, and the estimated dry-weather flow was subtracted from that amount. The result reflects the I/I flow. A unit value for I/I flow was calculated in gallons per acre per day. Few significant wet-weather events were experienced during the City's flow monitoring periods. The maximum 24-hour rainfall event was 2.37 inches, on March 6, 2002. This event is estimated to have a 1.6-year recurrence interval (or 62 percent probability of being exceeded in a given year). The 5-year storm flow was estimated by multiplying the 1.6-year flows by 1.31 (under Oregon Department of Environmental Quality requirements, sewer systems must have adequate capacity to convey 5-year storm flows).

The I/I rate calculated from the flow monitoring data was used to analyze areas currently served by the sewer system. For unsewered areas it was assumed that new sewer pipes would be installed to a high standard and without connections to roof drains and foundation drains. Therefore, a reduced unit I/I level of 3,300 gpad was used to model future conditions in currently undeveloped areas. Tables 3-7 and 3-8 show the I/I unit flow determined by this analysis for individual monitors and by subbasin.

TABLE 3-7.
DRY-WEATHER AND I/I FLOW ESTIMATES

	Estimated Unit Flows Based on Flow Monitoring (gpad)			Adjusted Unit Flows for 5-Year Event (gpad)		
Monitor	Dry-Weather Flow	I/I	Total	Dry-Weather Flow (Rounded)	I/I	Total
SS 01	2,813	3,852	6,665	2,800	5,100	7,900
SS 02	2,696	3,649	6,345	2,700	4,700	7,400
SS 03	3,218	6,109	9,327	3,200	8,000	11,200
SS 04	1,639	663	2,302	1,600	3,300	4,900
SS 06	1,452	7,525	8,977	1,500	9,800	11,300
SS 07	1,342	279	1,621	1,300	3,300	4,600
SS 10	3,623	8,410	12,033	3,600	11,000	14,600
SS 11	2,929	1,329	4,258	2,900	3,300	6,200
SS 12	3,763	2,577	6,340	3,800	3,400	7,200
SS 13	1,669	324	1,993	1,700	3,300	5,000

TABLE 3-8.
ESTIMATED 5-YEAR TOTAL UNIT FLOWS BY SUBBASIN

Subbasin	Estimated Unit I/I for 5-Year Event (gpad)	Subbasin	Estimated Unit I/I for 5-Year Event (gpad)	Subbasin	Estimated Unit I/I for 5-Year Event (gpad)
BC11	11,300	FS9	7,400	PE10	5,000
BC9	11,300	GO10	4,600	SE13	7,500
BW10	6,200	GO11	4,600	SE19	7,500
BW18	6,200	HO7	4,900	SE19A12	7,500
X2B	3,300	HO8	4,900	SE19A4	7,500
BW3A11	7,200	MC3	14,600	SE19A8	7,500
BW3A6	7,200	MC5A6	14,600	SE32	7,500
FS26	7,400	MC7	14,600	CO1	7,500
FS43	7,400	NE21	3,300	X1	3,300
BH1	7,400	NE28	3,300	X2A	3,300
NC1	7,400	NE31	3,300	SE5A4	7,500
FS8A16	7,900	NE37	3,300	SE5A5	7,500
FS8A21	7,900	NE39	3,300	TW10A3	11,200
FS8A28	7,900	NE7	3,300	TW10A6	11,200
FS8A3	7,900	PA10	7,500	TW13	11,200
FS8A36	7,900	PA18	7,500	TW28	11,200
FS8A6	7,900	PE1	5,000	TW36	11,200
				TW8	11,200

TOTAL FLOWS, EXISTING AND BUILDOUT

Existing and buildout total flows were modeled using the unit dry-weather and I/I flows calculated as described above. To model current flows, only areas that were developed as of June 2000, based on aerial photos, were assumed to contribute flow. Buildout flows assumed all undeveloped areas are developed to the maximum density allowed by zoning. Table 3-9 shows the existing and buildout flows in cubic feet per second (cfs) by subbasin. The areas measured and flows calculated for each subbasin are detailed in Appendix C.

TABLE 3-9.
ESTIMATED PEAK FLOWS BY SUBBASIN

	Estimated Flow (cfs)			Estimated Flow (cfs)	
Subbasin	Existing (2000)	Buildout	Subbasin	Existing (2000)	Buildout
BC9	0.76	1.30	NE21	1.25	1.64
BC11	1.11	3.48	NE28	0.25	0.35
BW10	0.81	1.31	NE31	0.43	0.43
BW18	1.29	1.75	NE37	0.43	0.56
X2B	0.00	0.63	NE39	0.71	1.80
BW3A6	0.26	0.26	PA10	1.53	1.91
BW3A11	1.23	1.78	PA18	1.40	1.72
FS9	0.39	0.39	PE1	0.20	0.84
FS26	1.20	1.20	PE10	1.33	1.49
FS43	1.27	1.91	SE13	0.65	0.82
BH1	1.00	1.26	SE19	0.17	0.17
NC1	0.19	0.19	SE32	1.59	1.59
FS8A3	0.35	0.35	CO1	1.44	1.97
FS8A6	0.33	0.33	X1	0.00	2.46
FS8A16	0.46	0.46	X2A	0.15	0.39
FS8A21	0.77	0.77	SE5A4	0.28	0.31
FS8A28	1.42	1.64	SE5A5	0.51	0.51
FS8A36	2.56	2.56	SE19A4	0.81	0.81
GO10	0.09	0.83	SE19A8	0.35	0.35
GO11	0.69	1.98	SE19A12	2.30	2.54
HO7	0.52	1.67	TW8	0.96	0.96
HO8	0.36	0.93	TW13	0.28	0.28
MC3	1.00	1.00	TW28	1.99	2.11
MC7	1.06	1.16	TW36	1.61	1.68
MC5A6	0.48	0.52	TW10A3	1.14	1.27
NE7	0.32	0.37	TW10A6	0.93	0.93

FLOWS FROM UGB EXPANSION AREAS

Much of the analysis for this master plan was performed prior to the 2003 expansion of the City's UGB. At that time, seven areas adjoining the previous UGB (E1 through E7) had been designated as potential expansion areas, as shown on Figure 3-5.

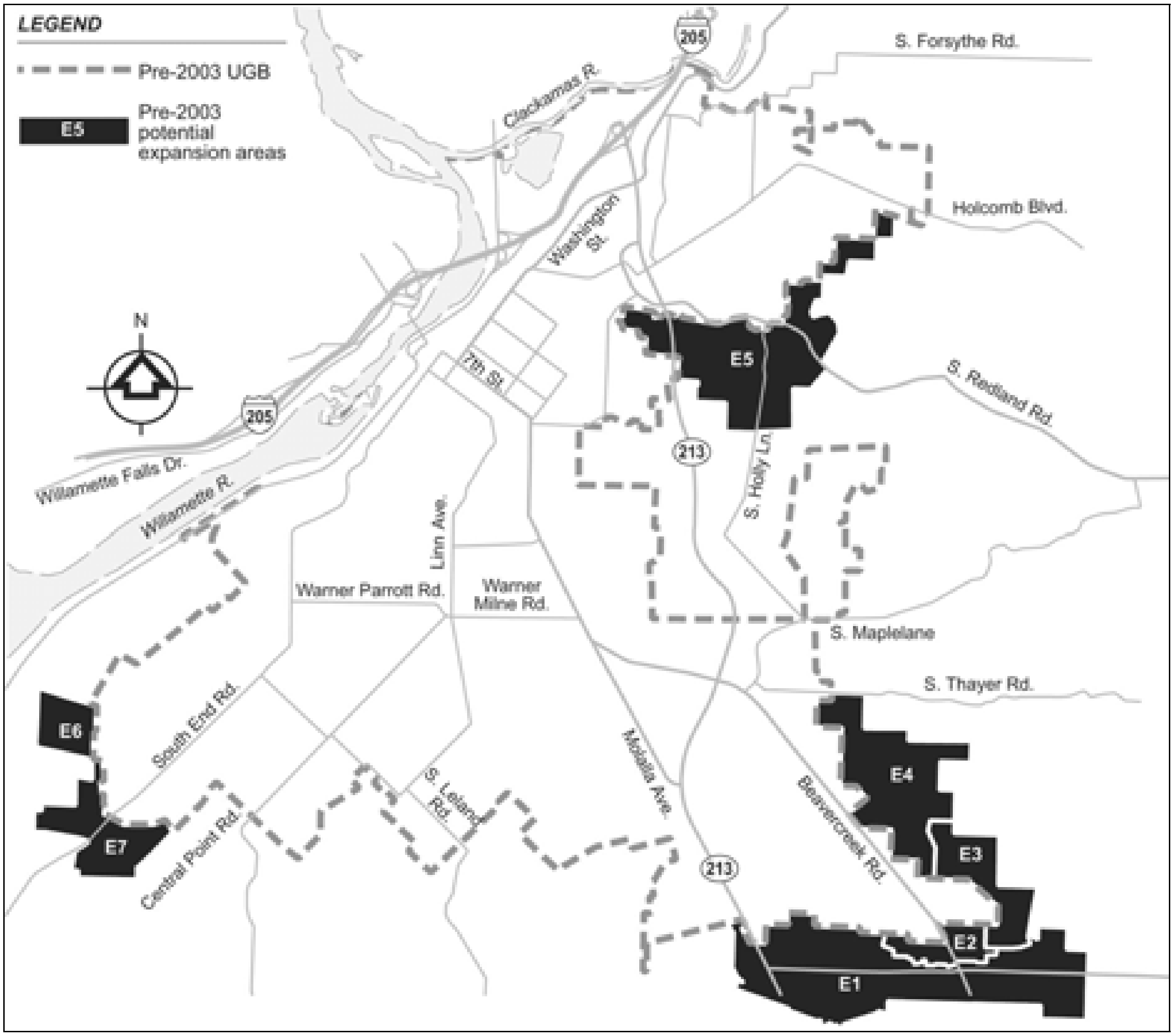


Figure 3-5. Pre-2003 Potential Oregon City UGB Expansion Areas

Analysis of the buildout condition assumed full development and sewering of these areas, with flows from them affecting the sewer system as follows:

- Expansion Areas E1 and E5 would drain directly to the TCSD system, and therefore not affect the City's sewers.
- Expansion Areas E2 and E3 would drain to the Glen Oak Road basin.
- Expansion Area E4 would drain to the Beavercreek basin.
- Expansion Areas E6 and E7 would drain to the South End basin.

Table 3-10 gives the estimated peak flows for buildout conditions in the pre-2003 potential expansion areas. The actual expansion of the UGB in 2003 included only Expansion Areas E4, E6, E7, and a portion of E5, so it affects only the Glen Oak Road and South End basins.

TABLE 3-10.
ESTIMATED PEAK FLOWS IN PRE-2003 POTENTIAL EXPANSION AREAS

Expansion Area	Estimated Flow at Buildout	
	cfs	gpm
E1	4.55	2,040
E2	0.55	246
E3	1.14	512
E4	2.45	1,101
E5	3.48	1,560
E6	0.61	273
E7	1.29	579

CHAPTER 4.
SANITARY SEWER SYSTEM EVALUATION

Deficiencies in the collection system in the Oregon City service area were evaluated using a spreadsheet-based computer model. The spreadsheet model calculates sanitary sewer system flows for existing and buildout conditions based on land use, population and I/I, and compares the flows to the capacity of each modeled pipe in the system. Pipe segments whose calculated capacity is less than their predicted peak flow are identified in this report as "deficient" or "inadequate."

ANALYSIS APPROACH

Existing conditions were based on current land uses and population distribution as estimated from year 2000 aerial photography; planning projections were used to model wastewater flows associated with buildout conditions.

Peak flow was calculated by adding sanitary flows from residential, commercial, industrial and public areas to I/I flows from a 5-year, 24-hour storm event, as described in Chapter 3. The peak flow considered by the model, therefore, is the contribution from area residents, businesses, and industry in combination with water entering the collection system during the peak storm through piping deficiencies or noncompliant direct connections.

Existing and buildout peak flows were routed through the system, and hydraulic grade lines—the surface elevations of the flow at any point along a pipeline—were calculated. As flow in a pipeline increases, the surface elevation of the flow in the pipe or, under surcharged conditions, in the manholes above the pipe, increases. To initiate hydraulic grade line computations (including at pump stations), the grade line was set at the centerline of the most downstream pipeline segment, and Manning's equation was used to determine the depth of flow in the pipe. Where the depth exceeded 90 percent of the pipe's inside diameter, the pipe was assumed to be surcharged and Manning's equation for a full pipe was used to determine the depth of the hydraulic grade line.

The model compared the hydraulic grade line with manhole rim elevations to determine the degree of surcharging and overflow in the existing collection system. A hydraulic grade line problem exists when the hydraulic grade line at a manhole exceeds the manhole's rim elevation. This analysis reveals where the City's system is inadequate.

The spreadsheet model also calculates the pipe diameter required to pass the buildout peak flow through a pipe with the same slope and interior surface roughness without the flow depth exceeding 90 percent of the pipe's inside diameter.

Pump station capacity was not modeled, so the peak flow was assumed to move continuously through the system. The pump station evaluation is described in Chapter 5.

MODELED PIPE SYSTEM

Pipe Size

To keep the amount of input data manageable and to focus on the primary wastewater transmission and interceptor lines, the model initially considered only pipelines 12 inches or more in diameter. Some 8- and 10-inch diameter pipelines critical to the evaluation of certain collection system areas were added to the model as needed. Table 4-1 summarizes the modeled collection system by basin and pipe diameter. The modeled system consists of more than 12 miles of pipeline.

TABLE 4-1.
MODELED SANITARY SEWER SYSTEM PIPE INVENTORY BY BASIN AND PIPE SIZE

Basin	No. of MHs	Length (in Feet) of Modeled Sanitary Sewer Pipe by Pipe Diameter										
		8"	10"	12"	15"	16"	18"	20"	21"	24"	26"	Total
Northeast	39			2,268	2,762	381	2,563					7,974
Holcomb	8		1,243	448								1,691
Beavercreek	11			1,800								1,800
Glen Oak	11	404	1,421	972								2,797
Molalla Clairmont	16	1,773	1,737									3,510
Beaver Warner	29	1,461	967		3,970		151					6,549
Pease Road	10		2,212									2,212
Parrish Road	18			3,407								3,407
South End[a]	54		1,114	2,359			3,473	2,609	1,620		371	11,546
5th & 7th Streets[b]	79			8,705	2,533		929	1,620		1,270		15,057
12th Street	43	2,977	1,281	3,000	447		142					7,847
Total	318	6,615	9,975	22,959	9,712	381	7,258	4,229	1,620	1,270	371	64,390
Percent of Citywide Total		10.3	15.5	35.7	15.1	0.6	11.3	6.6	2.5	2.0	0.6	100

a. Includes the Cook basin
b. Includes the Barclay Hills and Newell Crest basins

Pipe Roughness

A primary factor affecting a pipe's capacity is the roughness of its interior surface. Empirical testing over many years has established a series of roughness coefficients to characterize the relative smoothness of different types of piping. One such coefficient, called Manning's n-value, was incorporated into pipe capacity formulas for the computer model. Typical Manning's n-values for pipe materials used in wastewater collection are shown in Table 4-2. Table 4-3 summarizes the modeled collection system according to basin and pipe material.

TABLE 4-2.
PIPE ROUGHNESS COEFFICIENTS

Material Type	Age Category	Manning's n-value
Cast Iron (CI)	Old	0.010
Concrete Sewer Pipe (CSP)	Old	0.012
Ductile Iron (DI)	New	0.009
Polyethylene (PE)	New	0.009
Polyvinyl Chloride (PVC)	New Old	0.009 0.010
Reinforced Concrete Pipe (RCP)	Old	0.012
Terra Cotta (TC)		0.013

TABLE 4-3.
MODELED SANITARY SEWER SYSTEM PIPE INVENTORY BY BASIN AND PIPE MATERIAL

		Length (in Feet) of Sanitary Sewer Pipe by Pipe Material[a]								
Basin	No. of MHs	CI (old)	CSP (old)	DI (new)	PE (new)	PVC (new)	PVC (old)	RCP (old)	Terra Cotta	Total
Northeast	39			632	7,342					7,974
Holcomb	8	226				1,243		222		1,691
Beavercreek	11		997			823				1,800
Glen Oak	11					2,797				2,797
Molalla Clairmont	16		3,510							3,510
Beaver Warner	29					4,121		2,428		6,549
Pease Road	10					2,212				2,212
Parrish Road	18					3,407				3,407
South End[b]	54		2,359	163	3,890	5,134				11,546
5th & 7th Streets[c]	79		11,299				2,014		1,744	15,057
12th Street	43		142			7,705				7,847
Total	318	226	18,287	795	11,232	34,783	2,014	2,650	1,744	64,390
Percent of Citywide Total		0.4	28.4	1.2	17.4	54.0	3.1	4.1	2.7	100

a. See Table 4-2 for explanation of pipe material abbreviations.
b. Includes the Cook basin
c. Includes the Barclay Hills and Newell Crest basins

ANALYSIS RESULTS

Figures 4-1 through 4-11 (at the end of this chapter) show results for existing and buildout conditions. Spreadsheet results and hydraulic profiles are included in Appendix C. Results by basin are summarized below.

Northeast Area

The modeled trunk system in the Northeast Area consists of approximately 8,000 feet of pipe, ranging from 12 to 18 inches in diameter, and 39 manholes. The trunk system piping is primarily (92 percent) new polyethylene pipe, and the remainder is new ductile iron pipe. Under existing conditions, the sewer trunk system modeled for the Northeast Area is hydraulically adequate. Manhole 35 indicates a hydraulic grade line elevation near the manhole rim elevation, but this does not represent significant surcharging, as the crown of the pipe is also near the rim elevation.

Under buildout conditions, the analysis indicates slight surcharging of the downstream system between Manholes 1 and 7 but no risk of overflows, as the computed hydraulic grade line is more than 12 feet below the manhole rims in this reach. Surcharging will occur in the pipe reach between Manholes 30 and 31 under buildout conditions. The computed hydraulic grade line is more than 8 feet below the manhole rim and no improvement is recommended. Capacity is deficient between Manholes 33 and 34. The trunk system is on a relatively flat slope through this reach and is unable to adequately convey the projected buildout flows. Slight surcharging will be experienced in the reach between Manholes 33 and 36. As Manhole 35 is a coupling, no overflow will occur. No improvement is recommended. Peak flow increases in this basin between existing and buildout conditions are projected to be in the range of 55 to 75 percent. The majority of the Northeast Area trunk sewer system is adequate for conveying existing and projected buildout flows.

Holcomb Basin

The modeled trunk system in the Holcomb basin consists of approximately 1,700 feet of pipe, ranging from 10 to 12 inches in diameter, and eight manholes. The trunk system piping is primarily (74 percent) new PVC pipe, and the remainder is old reinforced concrete pipe and old cast iron pipe. Under existing conditions, the sewer trunk system modeled for the Holcomb basin is hydraulically adequate. Manhole 4 indicates a hydraulic grade line elevation within 5 feet of the manhole rim elevation, however this is a shallow manhole and no improvement is recommended for this condition.

Under buildout conditions, areas of deficient capacity are projected in the pipe reach between Manholes 2 and 3, where the existing 12-inch PVC pipe should be increased to a 15-inch pipe. Between Manholes 4 and 7, the existing 10-inch PVC pipe should be increased to a 12-inch pipe. Peak flow increases in this basin between existing and buildout conditions are projected to be approximately 200 percent.

Beavercreek Basin

The modeled trunk system in the Beavercreek basin consists of approximately 1,800 feet of pipe and 11 manholes. The trunk system piping consists of old 10-inch non-reinforced concrete sewer pipe from Manhole 1 to Manhole 7 and new 12-inch PVC pipe upstream of Manhole 7. Under existing conditions, the entire modeled trunk system for the Beavercreek basin is hydraulically adequate.

Under buildout conditions, the entire modeled trunk system has deficient capacity. For this condition, the entire existing 12-inch pipe trunk system between Manhole 1 and a manhole just south of John W. Loder Road should be replaced with 15-inch pipe; a portion of this work was done during the summer of 2003.

Glen Oak Basin

The modeled trunk system in the Glen Oak basin consists of approximately 2,800 feet of pipe and 11 manholes. The trunk system piping consists solely of new PVC pipe, ranging from 8 to 12 inches in diameter. Under existing conditions, the modeled trunk system for the Glen Oak basin is hydraulically adequate. Manholes 2, 3, 7 and 9 indicate hydraulic grade line elevations within 5 feet of the manhole rim elevation, but this does not represent significant surcharging as these are relatively shallow manholes, so no improvement is recommended for this condition.

Under buildout conditions, areas of deficient capacity are projected in the pipe reach between Manholes 6 and 7, where the existing 10-inch PVC pipe should be increased to a 12-inch pipe. From Manhole 9 to 10, the existing 10-inch PVC pipe indicates slight surcharging, however no improvement is deemed necessary for this condition. Peak flow increases in this basin between existing and buildout conditions are projected to be approximately 250 percent.

The pipe segment from Manhole 6 to Manhole 7 limits flow in the entire reach between Manholes 1 and 7 to 1.9 cfs. The estimated buildout peak flow in this reach with development of the urban growth expansion areas is 2.8 cfs. One option for addressing the excess flow is to divert it to a new pump station. In addition to conveying the excess flow from this reach, the new pump station could serve areas along Glen Oak Road west of Manhole 7 that are too low to be served by the Glen Oak trunk sewer (those with elevations below 400 feet). The pump station would discharge west to the sanitary sewer interceptor in Molalla Avenue (Highway 213). However, it is our recommendation that the excess flow in the Glen Oak trunk is sufficient to justify upgrading the pipe rather than diverting it to a pump station. Service to homes in the low-lying areas could be provided by STEP systems.

Molalla Clairmont Basin

The modeled trunk system in the Molalla Clairmont basin consists of approximately 3,500 feet of pipe and 16 manholes. The trunk system consists of old (non-reinforced) CSP ranging from 8 to 10 inches in diameter.

Under existing conditions, the modeled trunk system for the Molalla Clairmont basin is hydraulically adequate. Some pipe surcharging occurs between Manhole 2 and Manhole 4, however the computed hydraulic grade line is more than 12 feet below the rim elevations in this reach. No improvement is recommended for this condition. Likewise, no improvements are needed for the Clairmont Way tributary line.

Under buildout conditions some areas of deficient capacity are projected. Capacity is deficient in the pipe reach between Manholes 2 and 4, where the existing 10-inch CSP should be increased to a 12-inch pipe. No improvements are needed for the Clairmont Way tributary line. Peak flow increases in this basin between existing and buildout conditions are projected to be very small (approximately 8 percent along both the Molalla and Clairmont Way systems).

Beaver Warner Basin

The modeled trunk system in the Beaver Warner basin consists of approximately 6,500 feet of pipe, ranging from 8 to 18 inches in diameter, and 29 manholes. The trunk system piping consists primarily (63 percent) of new PVC pipe, and the remainder is old RCP. Under existing conditions, the modeled trunk system for the main trunk line along Beavercreek Road is hydraulically adequate. Manhole 11 experiences a hydraulic grade line elevation within 5 feet of the manhole rim elevation, however this is a shallow manhole and no improvement is recommended for this condition.

Under existing conditions, the modeled sewer lateral system extending toward Warner Milne Road experiences some hydraulic deficiencies. Some surcharging occurs between Manholes 3 and 3A2, but it is minor and the calculated hydraulic grade line is more than 11 feet below the manhole rims. There are significant system deficiencies in the reach between Manholes 3A4 and 3A11 along Warner Milne Road. The modeling predicts an overflow at Manhole 3A11. All of the old 8-inch RCP between Manhole 3A4 and 3A11 should be replaced with 12-inch pipe, which will provide capacity for buildout conditions.

The modeled sewer lateral system extending toward Warner Milne Road experiences further hydraulic deficiencies under buildout conditions. From Manhole 3 to Manhole 3A2, the existing 10-inch RCP should be replaced with 15-inch pipe. Modeling of the main trunk line along Beavercreek Road shows slight hydraulic deficiencies for buildout. Slight surcharging from Manhole 3 to Manhole 7 does not warrant improvement, however, the existing 15-inch PVC pipe should be replaced with 18-inch pipe if future work on this segment is needed for any reason other than capacity. Slight deficiencies are predicted between Manholes 10 and 11, which may cause an overflow at the shallow Manhole 11, depending on the I/I distribution on the basin. It is recommended that this manhole be carefully monitored during storm events to further assess capacity increases in the downstream pipe. Flow monitoring for the entire line from Manhole 3 to Manhole 11 is warranted to better define I/I.

Peak flow increases in this basin between existing and buildout conditions are projected to be approximately 65 percent along the Beavercreek main trunk and approximately 40 percent along the Warner Milne system.

Pease Road Basin

The modeled trunk system in the Pease Road basin consists of approximately 2,200 feet of pipe and 10 manholes. The trunk system consists of new 10-inch diameter PVC pipe. Under existing conditions, the modeled trunk system for the Pease Road basin is hydraulically adequate.

Under buildout conditions the system is also hydraulically adequate. Peak flow increases in this basin between existing and buildout conditions are projected to be very small (approximately 12 percent).

Parrish Road Basin

The modeled trunk system in the Parrish Road basin consists of approximately 3,400 feet of pipe and 18 manholes. The trunk system piping consists of relatively new 12-inch PVC pipe. Under existing conditions, the modeled trunk system for the Parrish Road basin indicates slight surcharging, with existing flows slightly in excess of gravity capacity (105 to 110 percent). No improvement is recommended for this condition as the depth to hydraulic grade line is in the range of 9 to 15 feet. Moreover, modeling of this basin used a level of I/I that is the same as used for all basins contributing to the South End Trunk, because no I/I data is available for the individual contributing basins; actual I/I in this basin is likely to be less than the modeled value, given the age and material of the sewer pipes in the basin. It is recommended that further study be done in this basin to determine the extent of any upgrades that may be required.

Under buildout conditions, some areas of possible deficient capacity are projected, with some pipe segments indicating flows from 130 percent to 150 percent of capacity. If flow monitoring of the reaches from Manhole 1 to Manhole 2 and from Manhole 7 to Manhole 10 indicate high levels of I/I, the existing 12-inch PVC pipe in these reaches should be replaced with 15-inch pipe for buildout conditions. Peak flow increases in this basin between existing and buildout conditions are projected to be approximately 23 percent.

South End Road Basin

The modeled trunk system in the South End Road basin consists of approximately 11,500 feet of pipe, ranging from 10 to 26 inches in diameter, and 54 manholes. The trunk system piping consists of new PVC pipe (44 percent), new PE pipe (34 percent), old CSP (20 percent) and a small amount of DI pipe (2 percent).

Under existing conditions, the modeled trunk system for the South End Road basin is adequate for conveying existing flows. The system at Manholes 3, 7, 8, 10, 13, 14 and 15 indicates hydraulic grade line elevations within 5 feet of the manhole rim, however these manholes are shallow and no improvement is recommended for this condition. The Hazelwood Drive system at Manholes 19A6, 19A10 and 19A11 experiences some slight surcharging, with existing flows slightly in excess of gravity capacity (105 to 115 percent). No improvement is recommended for this condition as the depth to hydraulic grade line is in the range of 8 to 14 feet. The High Street tributary system (line 5A) is also adequate to convey existing flows.

Under buildout conditions the sewer trunk system for the South End Road basin remains adequate for conveying the projected flows. Minor areas of concern are similar to those for existing conditions and, as such, no improvement is recommended. The Hazelwood Drive system at Manholes 19A6, 19A10 and 19A11 experiences some slight surcharging with buildout flows slightly in excess of gravity capacity (105 to 125 percent). No improvement is recommended for this condition as the depth to hydraulic grade line is in the range of 7 to 12 feet. The High Street tributary system (line 5A) is also adequate to convey buildout flows. To verify these modeling findings, flow monitoring is needed to better define the distribution of I/I in the South End Road basin. Peak flow increases in this basin between existing and buildout conditions are projected to be approximately 5 percent along the 5A line (High Street), 7 percent along the 19A line (Hazelwood Drive), and 50 to 60 percent along the main South End trunk line.

5th and 7th Street Basin

The modeled trunk system in the 5th and 7th Street basin consists of approximately 15,000 feet of pipe, ranging from 12 to 24 inches in diameter, and 79 manholes. The trunk system piping consists primarily of old CSP (75 percent), old PVC pipe (13 percent) and terra cotta pipe (12 percent).

Under existing conditions, the modeled trunk system for the 5th and 7th Street basin is hydraulically adequate. On the 7th Street trunk system, existing flows only reach a maximum of 70 percent of capacity. Manholes 30, 32 and 38 experience hydraulic grade line elevations within 5 feet of the manhole rim, however these manholes are shallow and no improvement is needed. On the 5th Street trunk system (Line 8A), existing flows reach a maximum of 85 percent capacity in most pipes. The reach between Manhole 8A23 and Manhole 8A24 has insufficient capacity because of its negative slope. This pipe causes surcharging at Manhole 8A24 to within 6 feet of the manhole rim. This is not desirable but does not pose an overflow risk. Manholes 8A30, 8A32 and 8A34 experience hydraulic grade line elevations within 5 feet of the manhole rim, however these manholes are also shallow and no improvement is recommended for this condition.

Under buildout conditions the system is also hydraulically adequate. On the upper end of the 7th Street trunk system, from Manhole 29 to Manhole 43, slight surcharging of the system is experienced and projected buildout flows reach as high as 100 to 106 percent of capacity. No improvement is recommended for this condition. On the 5th Street trunk system (Line 8A), buildout flows reach a maximum of 89 percent of capacity. Similar to the existing condition, the pipe below Manhole 8A24 surcharges and Manholes 8A30, 8A32 and 8A34 indicate hydraulic grade line elevations within 5 feet of the manhole rim; however, these manholes are shallow and no improvement is recommended for this condition.

Peak flow increases in this basin between existing and buildout conditions are projected to be approximately 30 percent for the 7th Street trunk system and 4 percent for the 5th Street (Line 8A) trunk system.

12th Street Basin

The modeled trunk system in the 12th Street basin consists of approximately 7,800 feet of pipe, ranging from 8 to 18 inches in diameter, and 43 manholes. The trunk system piping consists primarily of new PVC pipe (98 percent) and the remainder is old CSP. Under existing conditions, the modeled trunk system for the 12th Street basin is hydraulically adequate, although known inflow connections from the storm drain system may not be reflected in the model results. Corrections of the cross connections problems are included as a short-term CIP project, after which the modeling results should be more representative of actual conditions. According to the model, isolated line segments, such as the reaches from Manhole 2 to Manhole 3 and from Manhole 21 to Manhole 22, indicate existing flows of 130 percent and 155 percent of capacity. However, the depth to hydraulic grade line is from 9 to 11 feet at these locations. Also, Manholes 11 and 12 indicate hydraulic grade lines within 5 feet of the manhole rim, however these are shallow manholes. On the Line 10A lateral system, existing flows reach a maximum of 68 percent of capacity.

Under buildout conditions flows are increased slightly and the modeled trunk system for the 12th Street basin remains hydraulically adequate. Isolated line segments, such as the reaches from Manhole 2 to Manhole 3, and from Manhole 21 to Manhole 22, indicate slight increases in buildout flows to 136 percent and 164 percent of capacity. The depth to hydraulic grade line remains from 9 to 11 feet at these locations. Manholes 11 and 12 continue to experience hydraulic grade lines within 5 feet of the manhole rim, however these are shallow manholes. On the Line 10A lateral system, buildout flows reach a maximum of 72 percent of capacity. No improvements are recommended for buildout conditions.

Peak flow increases in this basin between existing and buildout conditions are projected to be very small (approximately 5 percent).


Figure 4-1A
2000 RESULTS
NE AREA
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
CLACKAMAS RIVER
HWY 213
I-205
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged
500'
0'
500'
1000'

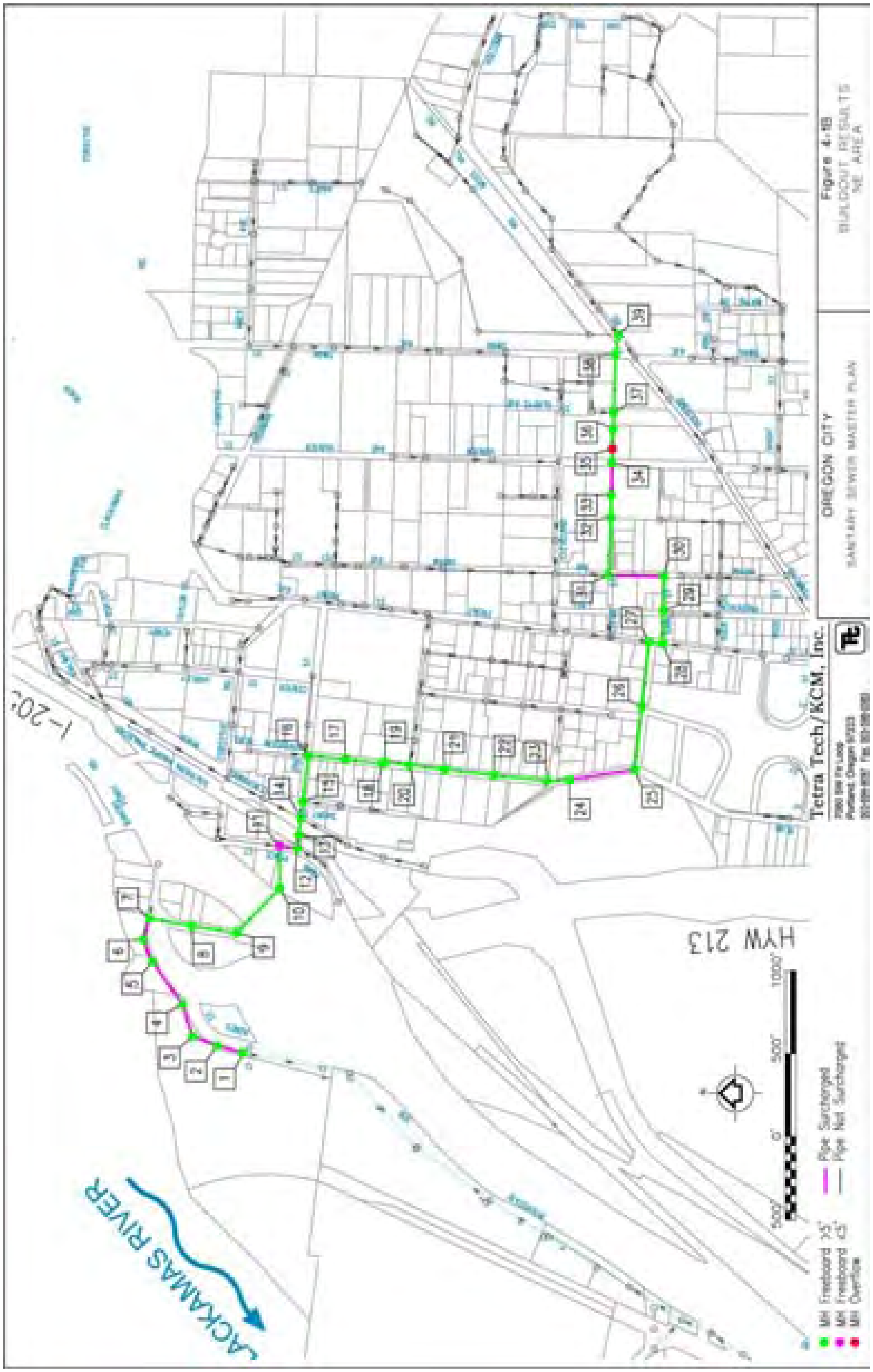
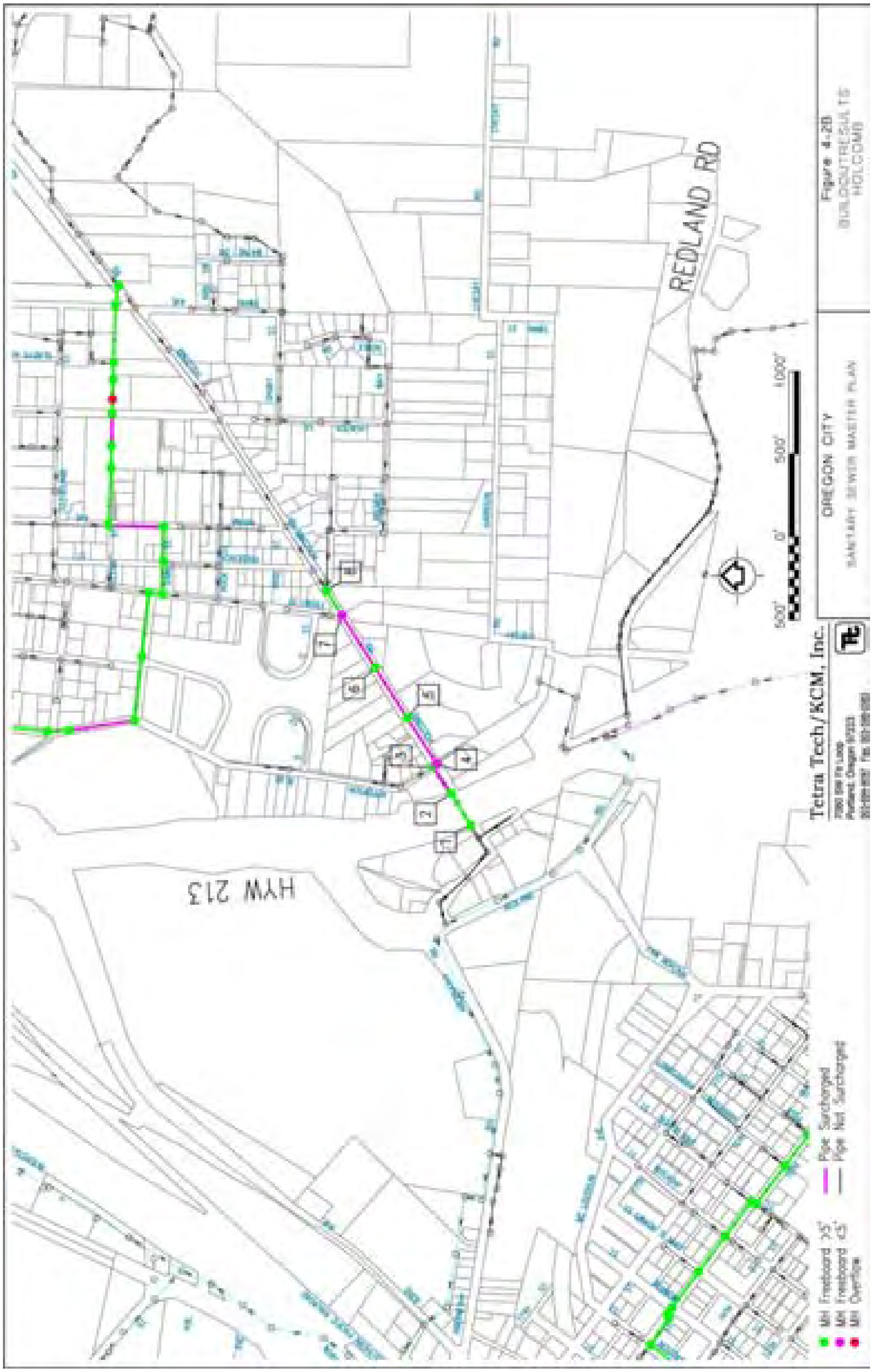


Figure 4-1B
BUILDOUT RESULTS
NE AREA
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
CLACKAMAS RIVER
HYW 213
I-205
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged
500'
0'
500'
1000'


Figure 4-2A
2000 RESULTS
HOLCOMB
OREGON CITY
SANITARY SEWER MASTER PLAN
500' 0' 500' 1000'
Tetra Tech/KCM, Inc.
REDLAND RD
HWY 213
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged


REDLAND RD
HWY 213
Figure 4-2B
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
Pipe Surcharged
Pipe Not Surcharged
0'
500'
1000'

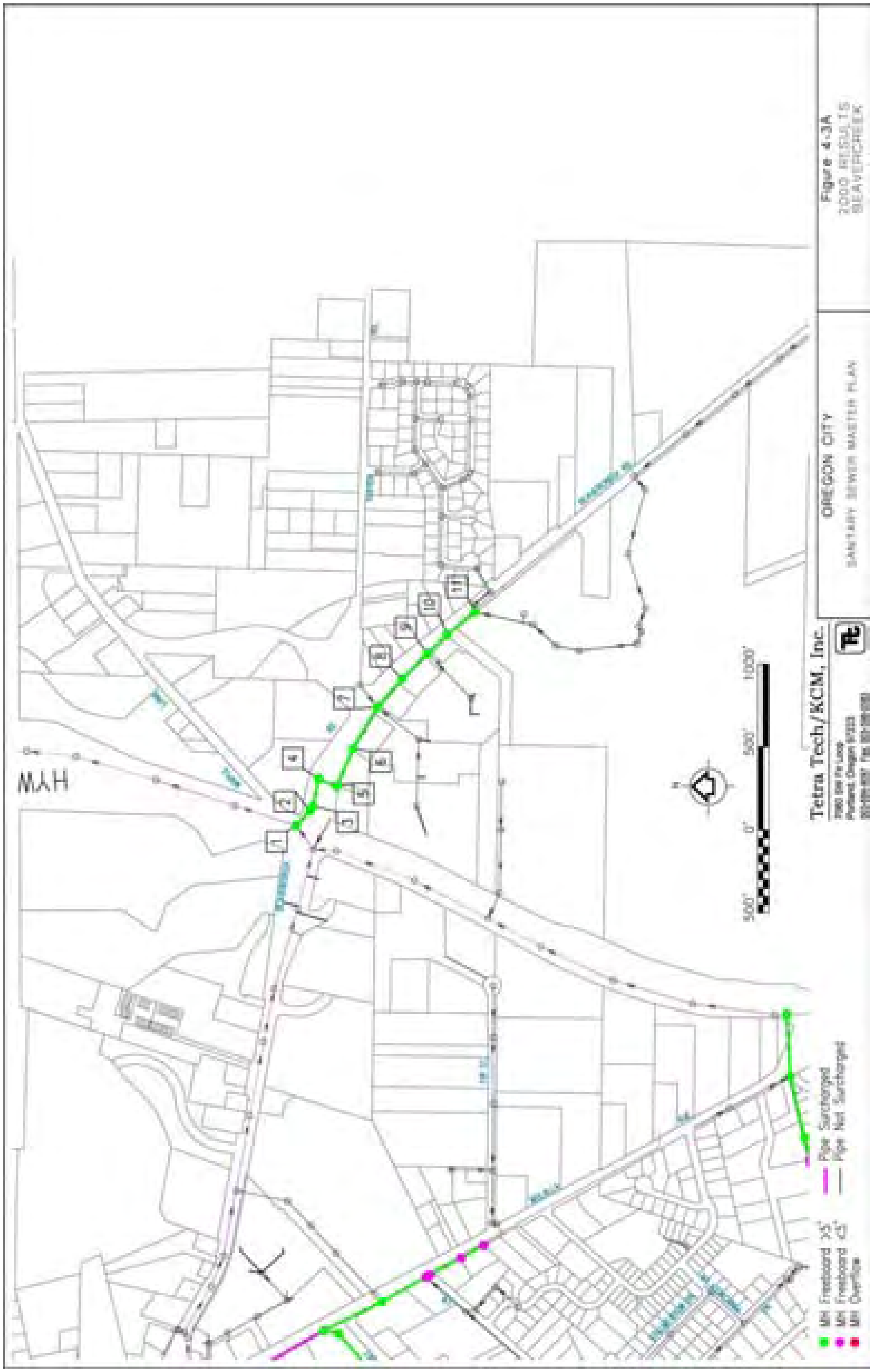


Figure 4-3A
2000 RESULTS
BEAVERCREEK
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
HWY
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged

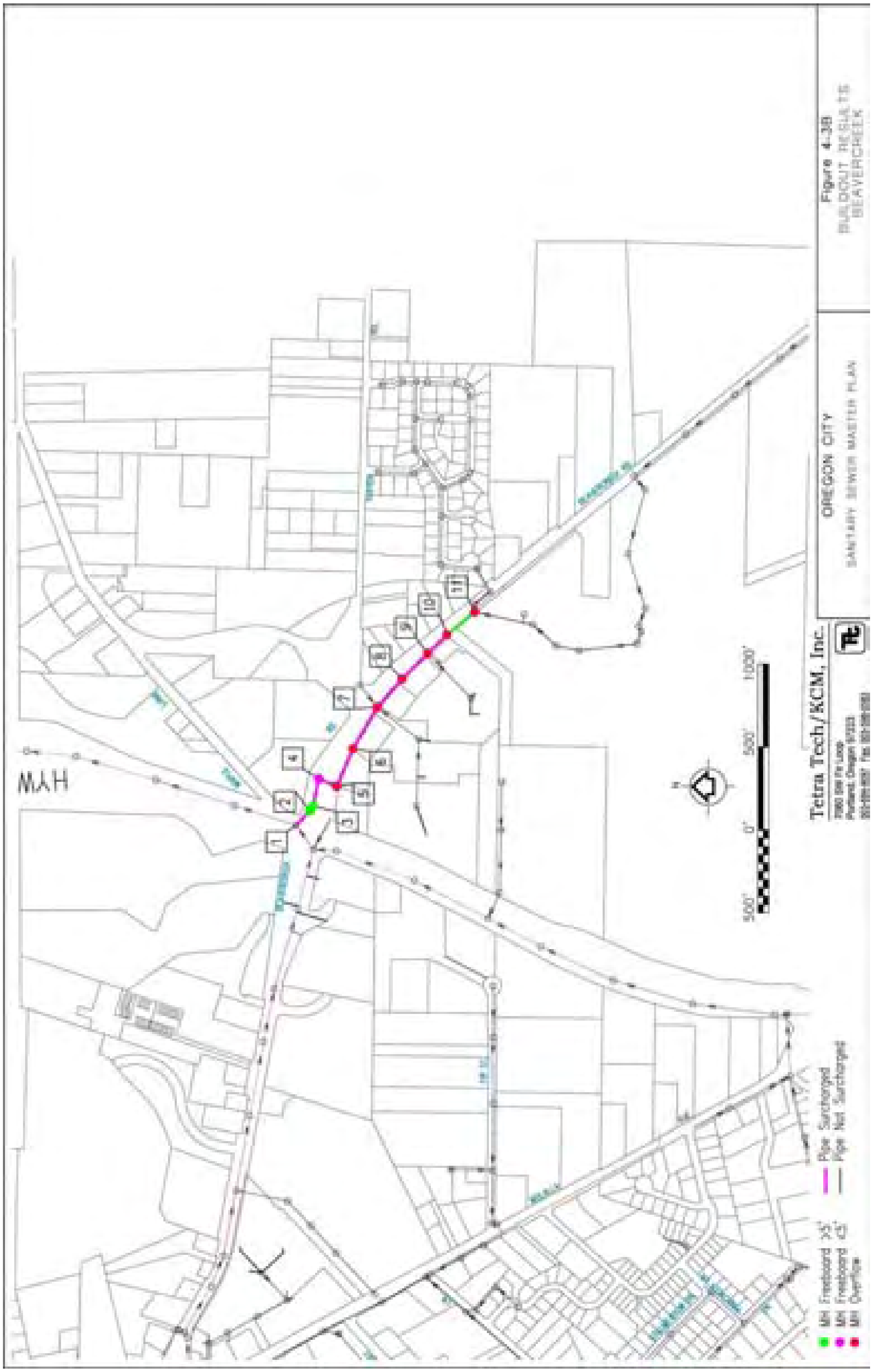
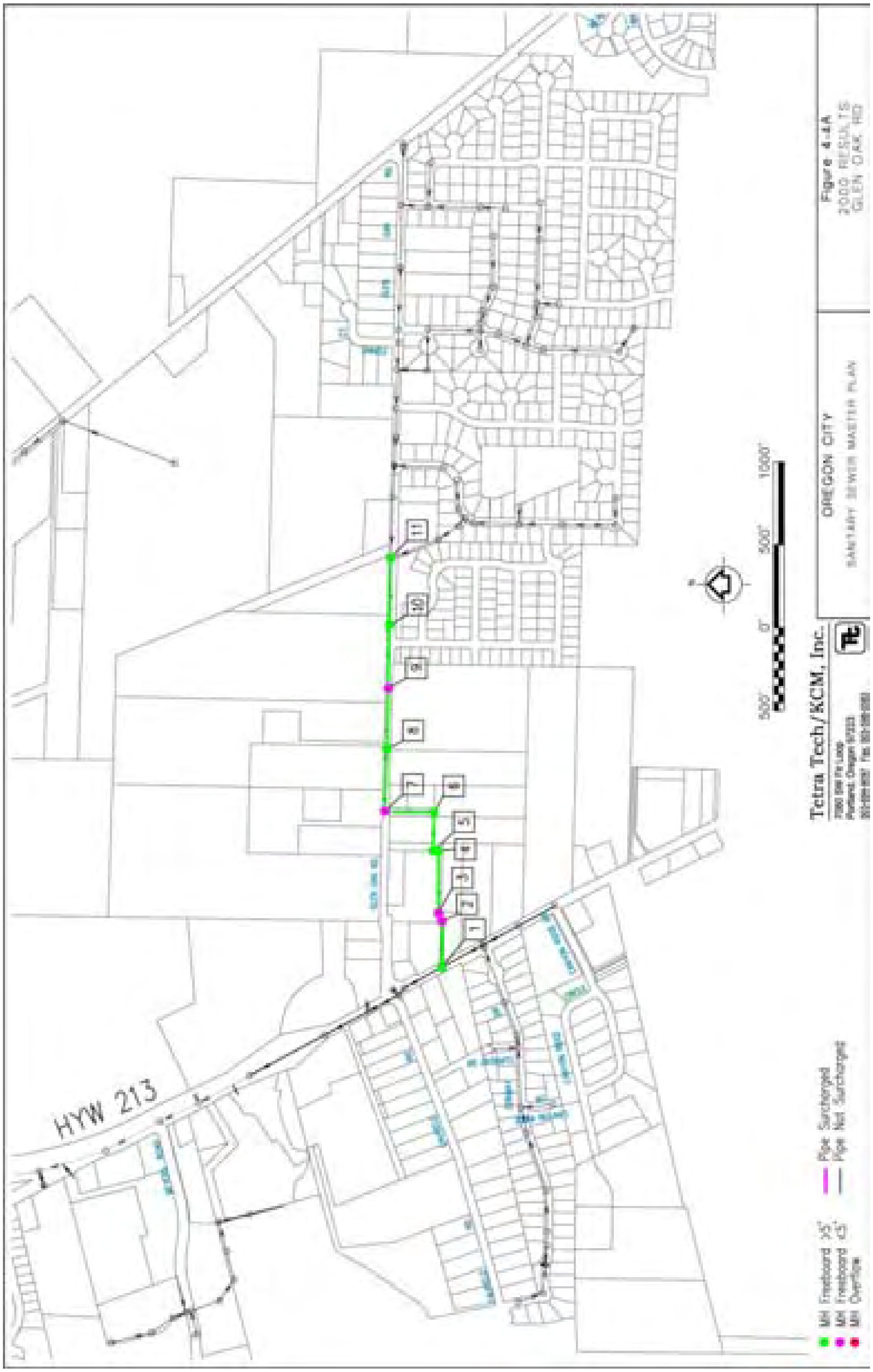
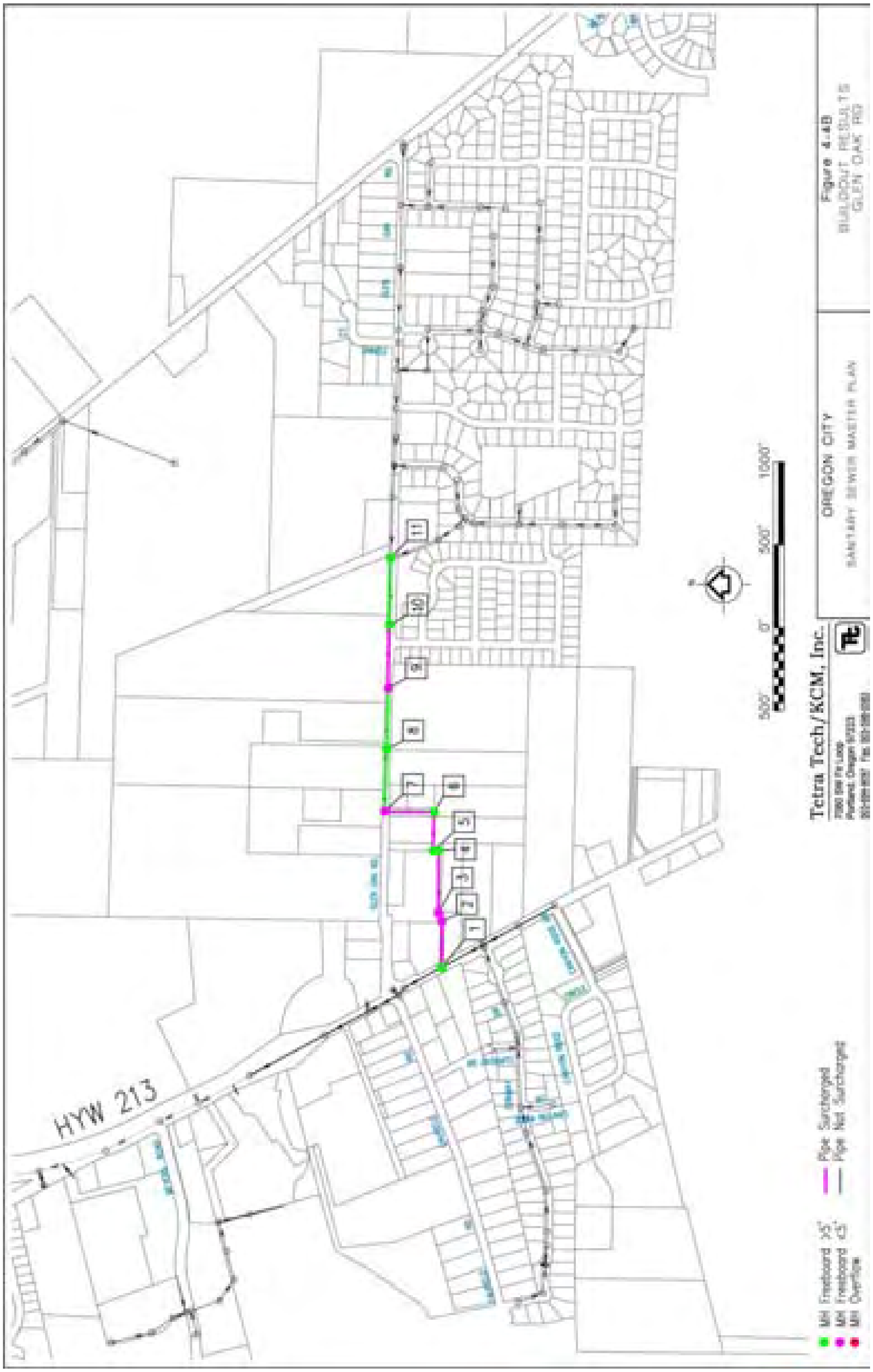
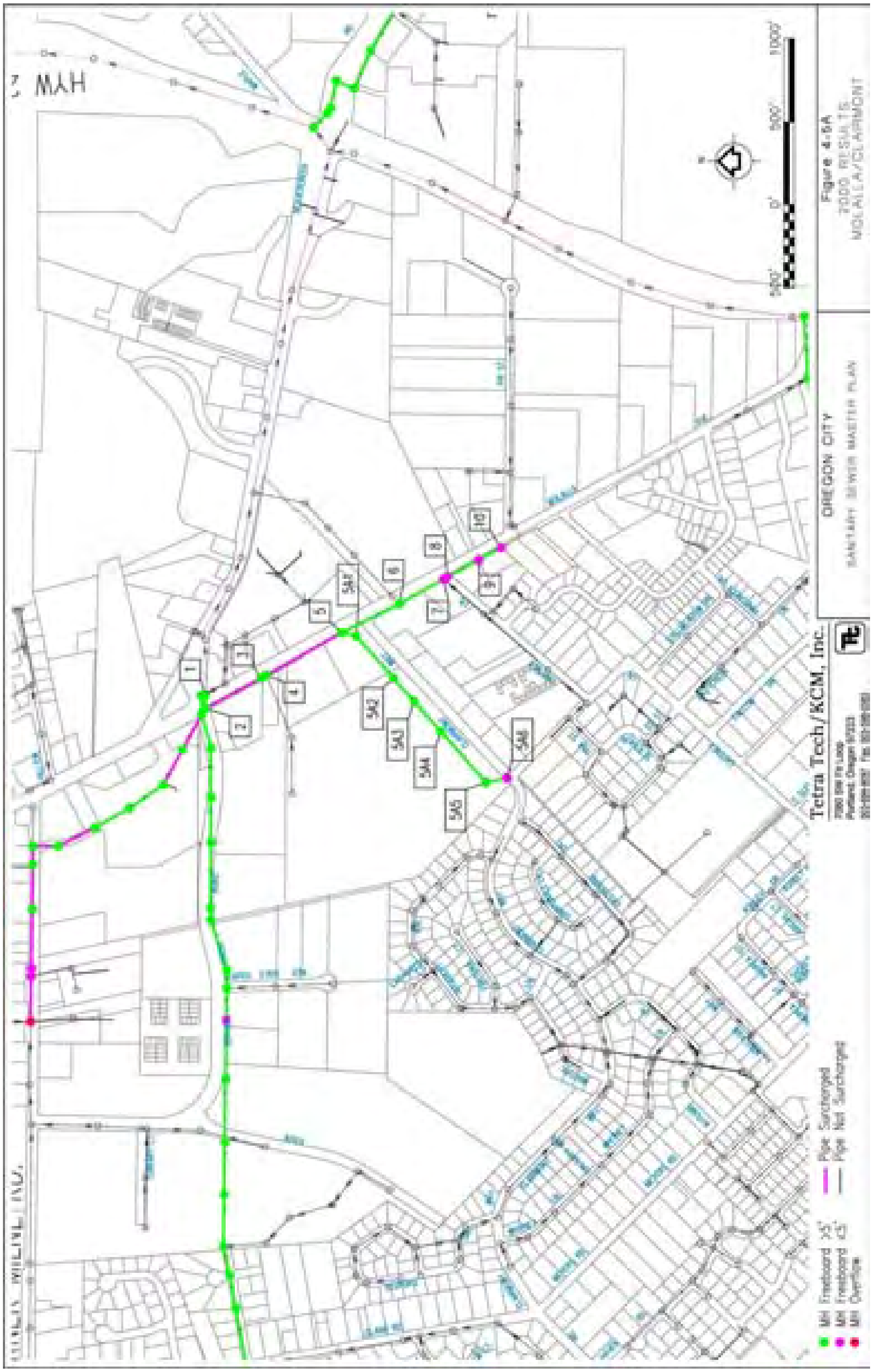


Figure 4-3B
BUILDOUT RESULTS
BEAVERCREEK
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged
500'
0'
500'
1000'
HWY
1
2
3
4
5
6
7
8
9
10
11


HYW 213
1
2
3
4
5
6
7
8
9
10
11
500'
0'
500'
1000'
Tetra Tech/KCM, Inc.
OREGON CITY
SANITARY SEWER MASTER PLAN
Figure 4-4A
2000 RESULTS
GLEN OAK RD
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged


HYW 213
1
2
3
4
5
6
7
8
9
10
11
500'
0'
500'
1000'
Tetra Tech/KCM, Inc.
OREGON CITY
SANITARY SEWER MASTER PLAN
Figure 4.4B
BUILDOUT RESULTS
GLEN OAK RD
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged


HWY
Figure 4-6A
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.

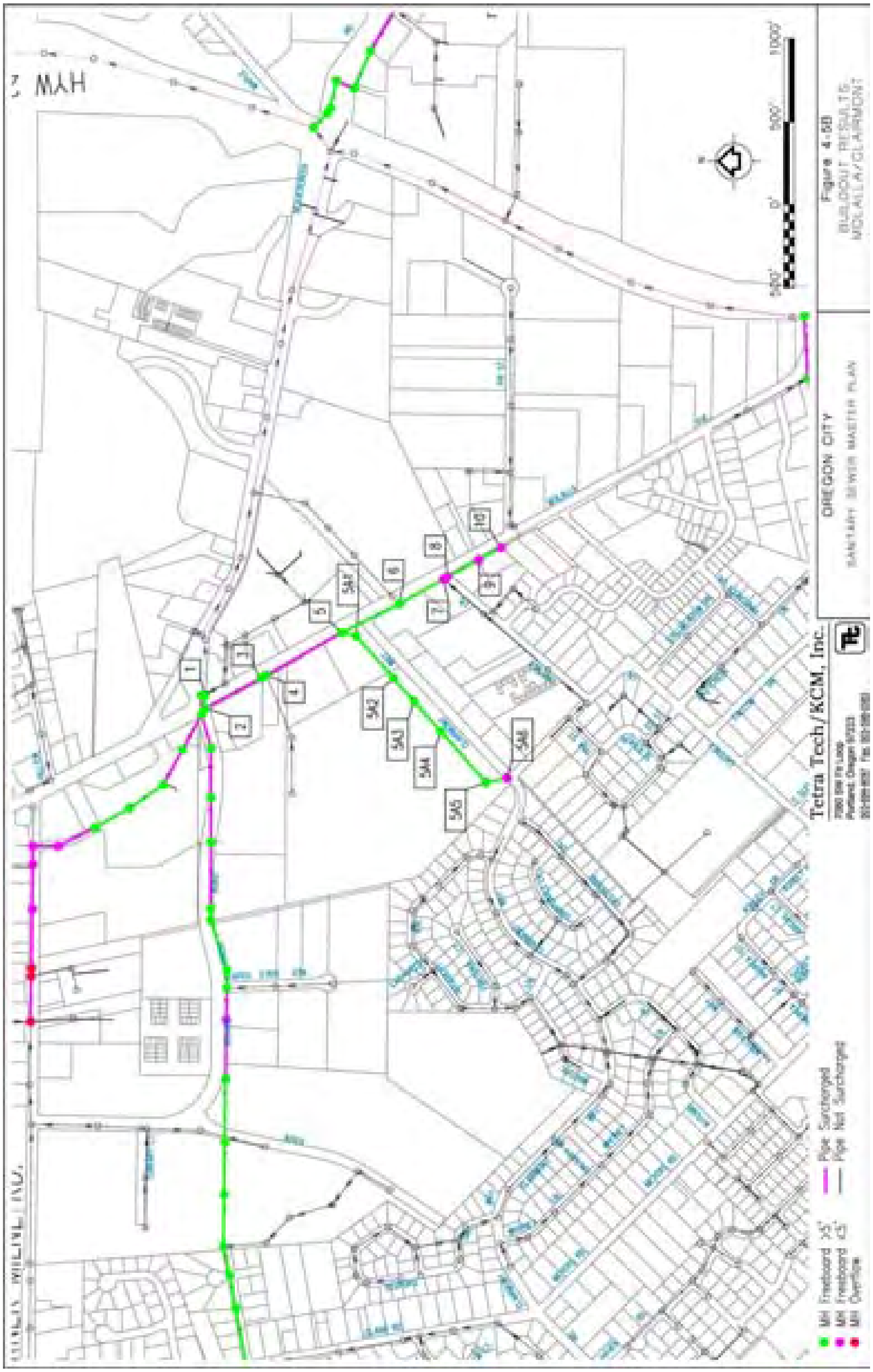


Figure 4-5B
BUILDOUT RESULTS
MOLALLA/CLAIRMONT
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged
0'
500'
1000'
1
2
3
4
5
6
7
8
5A1
5A2
5A3
5A4
5A5
5A6
HWY

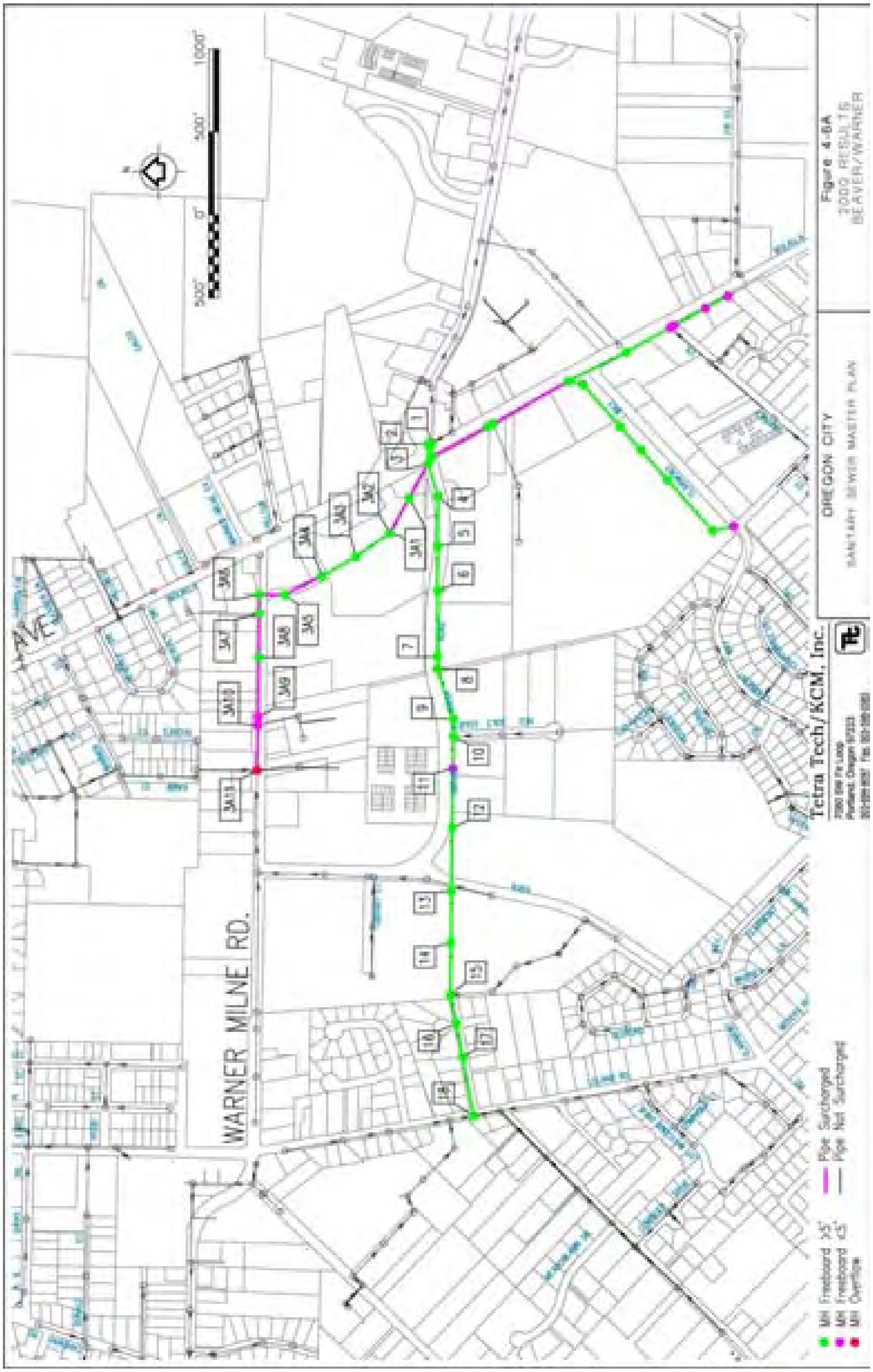


Figure 4-6A
2000 RESULTS
BEAVER/WARNER
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
WARNER MILNE RD.
AVE
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged

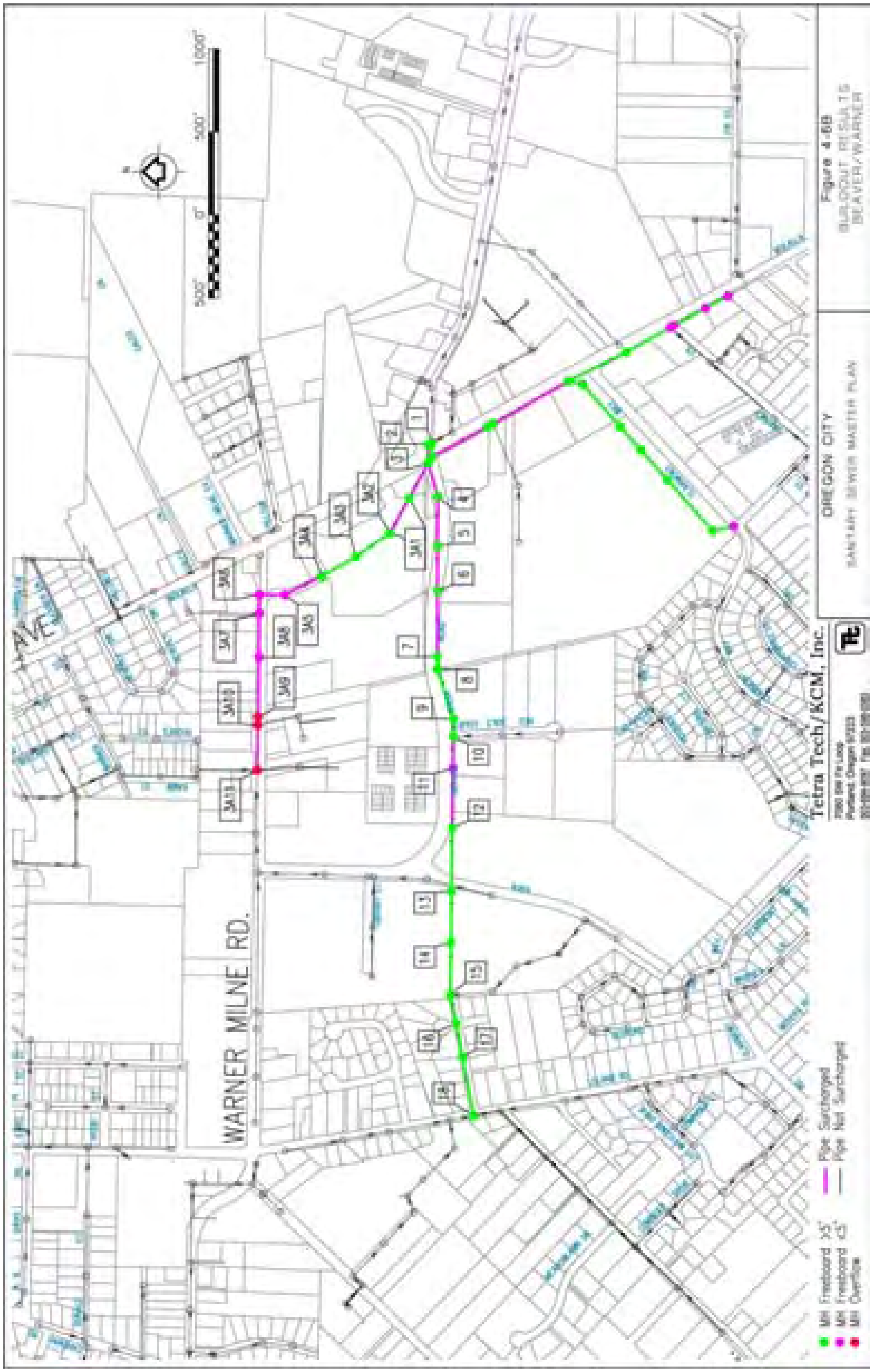
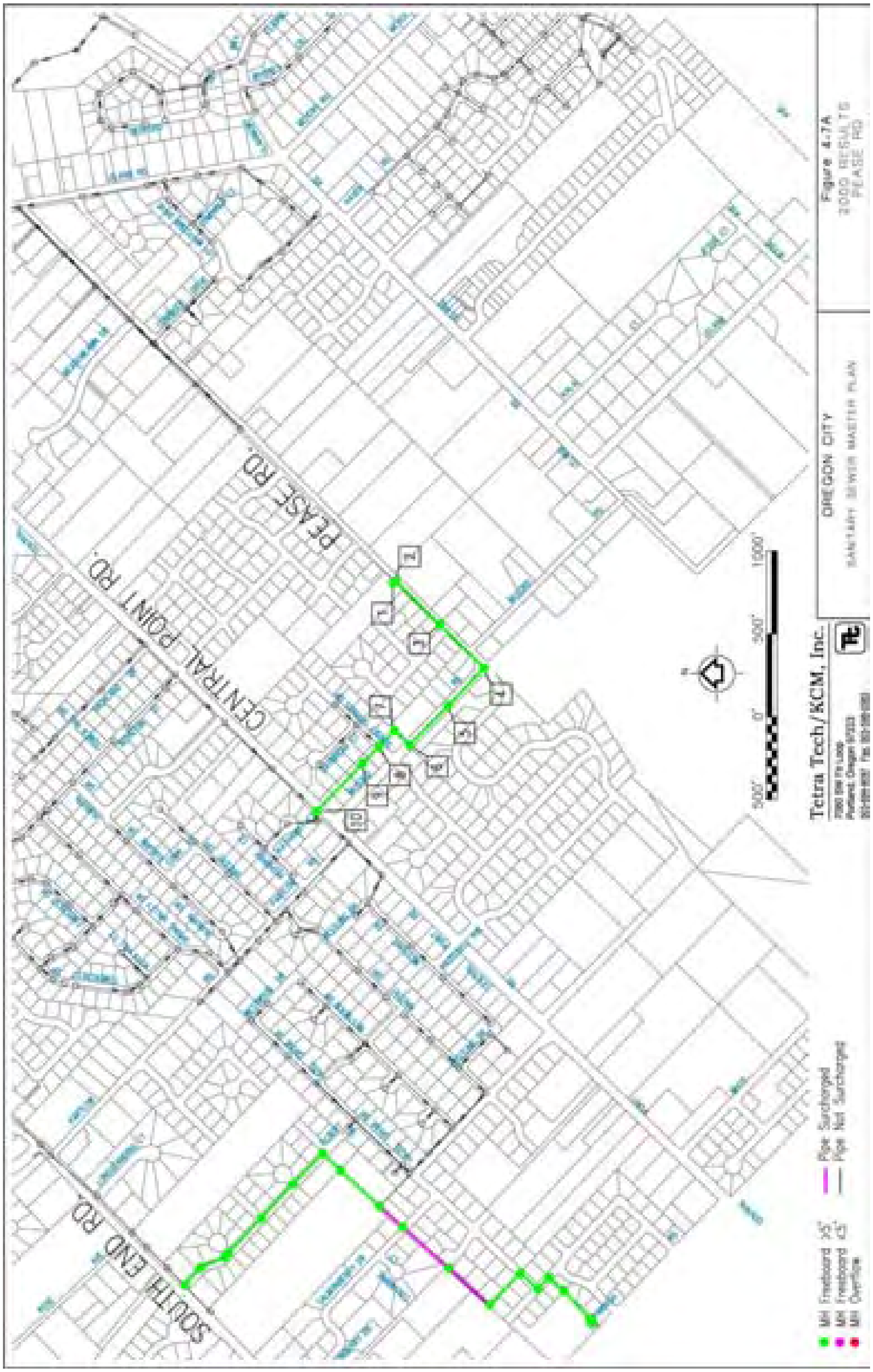
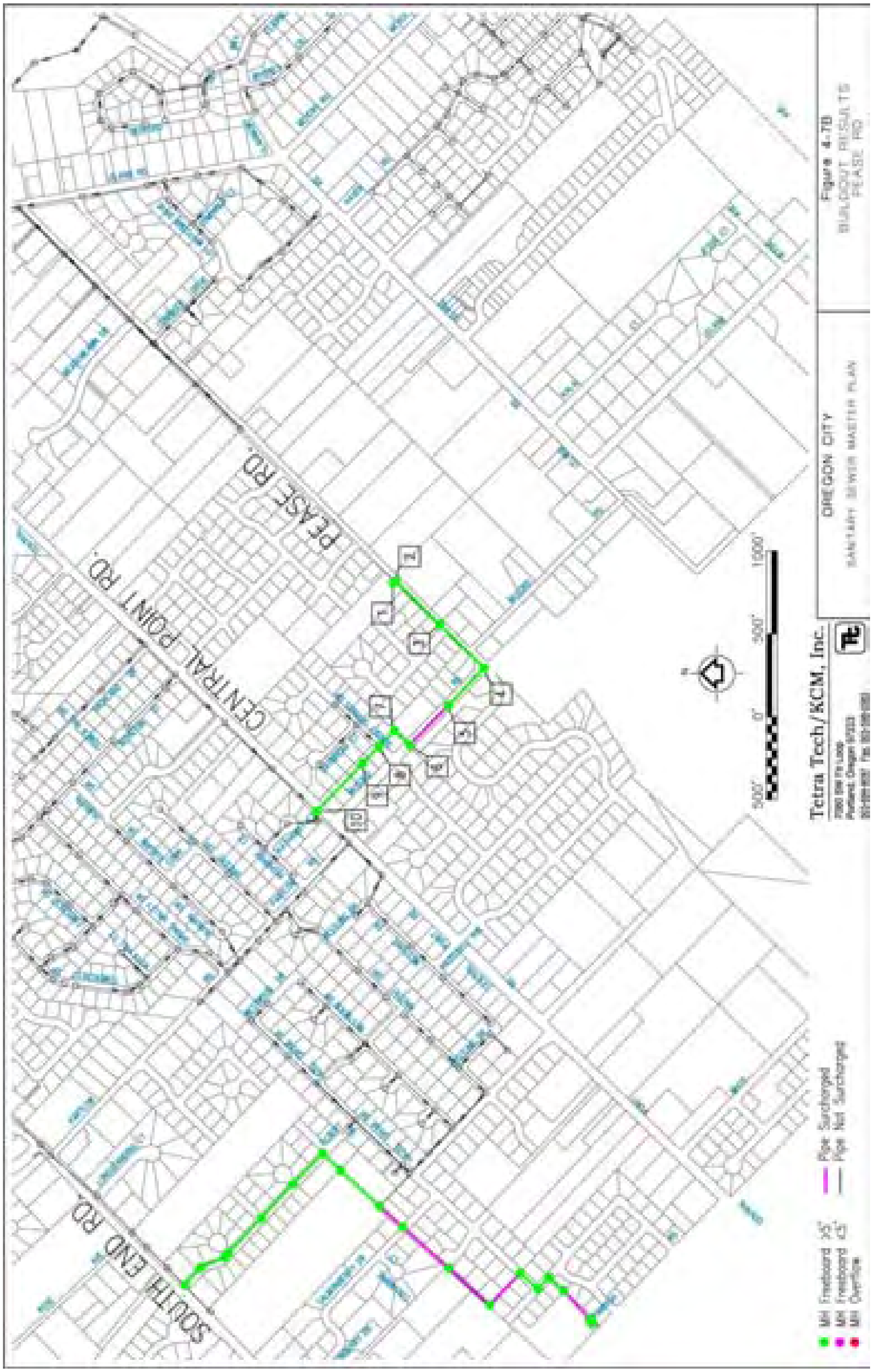
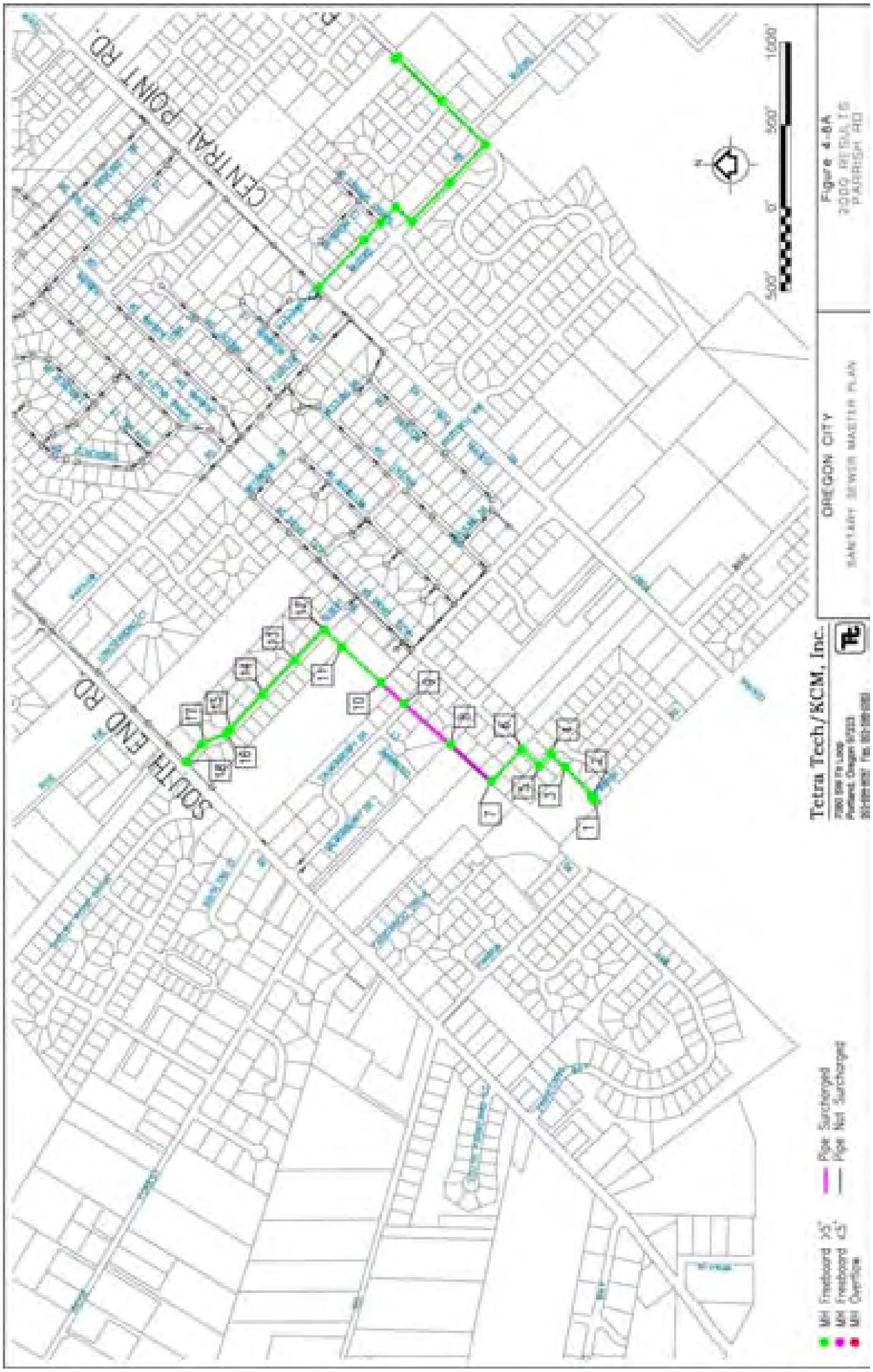


Figure 4-6B
BUILDOUT RESULTS
BEAVER/WARNER
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
WARNER MILNE RD.
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged


PEASE RD.
CENTRAL POINT RD.
SOUTH END RD.
Figure 4-7A
2000 RESULTS
PEASE RD
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
500'
0'
500'
1000'
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged


PEASE RD.
CENTRAL POINT RD.
SOUTH END RD.
Tetra Tech/KCM, Inc.
OREGON CITY


CENTRAL POINT RD
SOUTH END RD
Tetra Tech/KCM, Inc.
OREGON CITY
SANITARY SEWER MASTER PLAN
Figure 4-6A
2020 RESULTS
PARRISH RD
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged
500'
0'
500'
1000'
1
2
3
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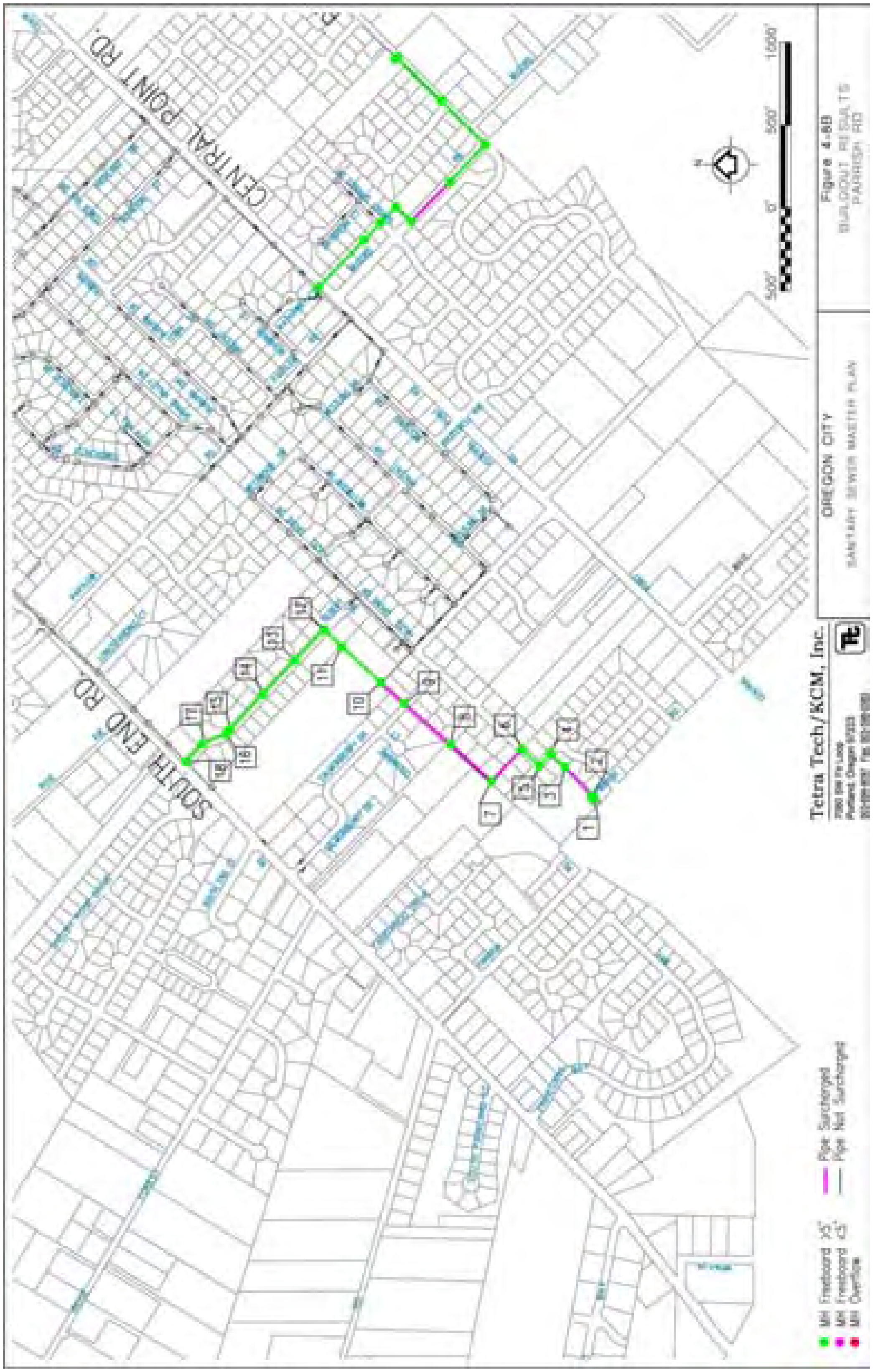


CENTRAL POINT RD.
SOUTH END RD.
MH Freeboard >5'
MH Freeboard <5'
MH Overflow
Pipe Surcharged
Pipe Not Surcharged
Tetra Tech/KCM, Inc.
OREGON CITY
SANITARY SEWER MASTER PLAN
Figure 4-8B
BUILDOUT RESULTS
PARRISH RD


OREGON CITY
Tetra Tech/KCM, Inc.
RIVER


BUILDOUT RESULTS
SOUTH END RD
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
RIVER


MOLALLA AVE
Figure 4-10A
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.

MOLALLA AVE
Figure 4-10B
BUILDOUT RESULTS
OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
Pipe Surcharged
Pipe Not Surcharged

OREGON CITY
SANITARY SEWER MASTER PLAN
Tetra Tech/KCM, Inc.
WILLAMETTE RIVER
INTERSTATE


WILLAMETTE RIVER
OREGON CITY
Tetra Tech/KCM, Inc.

CHAPTER 5.
PUMP STATIONS

The topography of Oregon City has required that pump stations be used to serve some areas of the City. There are 20 active pump stations serving Oregon City. Six of these are STEP pump stations, each serving only one residence.

The pump stations are generally in good condition. The City has a thorough routine maintenance and inspection program. With the exception of the STEP stations, each pump station is inspected twice a week. Run-time readings are taken once a week.

Table 5-1 lists the pump stations and their locations. Figure 2-5 shows the location of each pump station. Table 5-2 shows the pump operational data. Table 5-3 describes telemetry and backup power equipment at each public pump station. Table 5-4 lists information on the buildings and sites. An overall pump station inventory table is provided in Appendix D.

Pump station run times for the period from 1998 through 2001 are summarized in Table 5-5, which shows wet-weather averages, dry-weather averages, annual averages and statistics for the reporting period. Pump run times are recorded weekly; daily run times represent averages for the week. Detailed pump station run time data is provided in Appendix D. It should be noted that weekly run times are not adequate for determining when pump station capacity is insufficient. The flow monitoring performed for this master plan did not provide the level of detailed information needed to accurately assess pump station flows. As such, it is recommended that the City's proposed telemetry upgrades include flow monitoring and data logging on larger pump stations and pump on/off time data logging on all public pump stations.

PUMP STATION EVALUATION APPROACH

Pump Station Capacity

Redundant capacity of the City's pump stations is defined as the capacity with the highest-capacity pump out of service. Because detailed information on pump and system curves was not available to allow a precise calculation of the total capacity when both pumps are pumping, total capacity was estimated to be 1.5 times redundant capacity.

Peak Flow Estimates

The approach to estimating existing and buildout-condition peak flows for pump stations was the same as that used for estimating trunk sewer flows, as described in Chapter 3. Table 5-6 summarizes the results of the pump station flow evaluation. Detailed results are provided in Appendix D.

TABLE 5-1.
PUMP STATION LOCATION AND DESCRIPTION

No.	Name	Address	Type	Number of Pumps
1	Canemah	410 S McLoughlin Blvd	Wet well/dry well	2
2	Amanda	275 Amanda Court	Suction lift, vacuum prime	2
3	Riverview	287 Amanda Court	Grinder	2
4	Cook	18763 Cook Street	Wet well/dry well	2
5	Parrish	11525 Parrish Road	Wet well/dry well	2
6	Pease	19379 Pease Road	Suction lift	2
7	Settlers Point	16460 S Meyers Road	Suction lift	2
8	Nobel Ridge	13181 Gaffney Lane	Suction lift	2
9	Hidden Creek	19833 Highway 213	Suction lift	2
10	Brendon Estates	13903 Conway Drive	Submersible on rails	2
11	Hilltop	708 Hilltop Avenue	Suction lift, vacuum prime	2
12	Fir Street[a]	19200 Molalla Avenue	Suction lift	2
13	Newell Crest	18161 Newell Crest Drive	Wet well/dry well	2
14	Barclay Hills	17881 Peter Skene Way	Wet well/dry well	2
15	18th Street	1404 18th Street	STEP	1
16	18th Street	1412 18th Street	STEP	1
17	Elevator	610 Promenade	Grinder	1
18	Fire Station #2[a]	19340 Molalla Avenue	Submersible	1
19	STEP System 1	11501 Salmonberry Drive	STEP	1
20	STEP System 2	11502 Salmonberry Drive	STEP	1
21	STEP System 3	11520 Salmonberry Drive	STEP	1
22	STEP System 4	11521 Salmonberry Drive	STEP	1

a. Abandoned.

TABLE 5-2.
PUMP STATION OPERATIONAL DATA

No.	Name	Motor Speed (rpm)	Power (hp)	Capacity (gpm)	TDH (feet)	Wet Well Operating Size (gal)	Wet Well Dimensions (feet) and Features	Force Main Diameter (inches)
1	Canemah	1,400	40	1,200	102	3,792	13x13.5, 20.6 deep; lined with epoxy	8
2	Amanda	1,150	3	100	35	220	4Ø, 5 deep; lined with polyurea	4
3	Riverview	3,450	2	30	19	219	4Ø, 20 deep; lined with polyurea	6
4	Cook	1,175	15	270	54	1,122	4Ø increasing to 6-8Ø under street, 20.3 deep; white liner peeling at manhole lid; lined with polyurea; wet well under street	6
5	Parrish	1,780	40	760	106		12Ø, 26 deep; signs of I/I; lined with Strongseal	10
6	Pease	1,750	15	500	58	2,843	12Ø, 16.8 deep; unlined	6
7	Settlers Point	1,465	25	831	53.4	2,834	14Ø, 16 deep; lined; auto spray down	8
8	Nobel Ridge	1,750	5	140	30	690	9Ø, 14 deep; lined; auto spray down	4
9	Hidden Creek	1,750	25	404	73	2,019	9x14, 18.2 deep; unlined	6
10	Brendon Estates	1,780	1.74	100	18	211	7Ø, 13.8 deep; lined with Strongseal	4
11	Hilltop	1,750	7.5	95	40	282	4Ø, 9.8 deep; unlined	4
12	Fir Street (abandoned)	1,155	2	100	18	188	4Ø	4
13	Newell Crest	1,760	60	120	230	376	8Ø, 9.7 deep; No lining; three manhole accesses; no step irons	4
14	Barclay Hills	1,760	30	300	170	2,020	6x10, 14.3 deep; new concrete unlined; connected by a 12-inch pipe to 6Ø, 15.4 deep old unlined	8
15	18th Street	3,450	0.5					2
16	18th Street	3,450	0.5					2
17	Elevator	3,450	2			117	4Ø, 6.2 deep	1.5
18	Fire Station #2 (abandoned)						3x3; unlined concrete	3
19	STEP System #1	3,450	0.5					
20	STEP System #2							
21	STEP System #3							
22	STEP System #4							

TABLE 5-3.
PUMP STATION TELEMETRY AND STANDBY POWER

No.	Name	Level sensors	Telemetry	Standby Power
1	Canemah	Sonic	High water, intruder, power fail	Manual transfer switch
2	Amanda	Four mercury floats	High water	Manual transfer switch
3	Riverview	Four mercury floats	None, external light and horn	Manual transfer switch
4	Cook	Bubbler	High water, intruder	Manual transfer switch
5	Parrish	Miltronics Multi ranger	High water, low water, intruder, power fail, low air	Natural gas generator
6	Pease	Mercury floats	High water, intruder	Manual transfer switch
7	Settlers Point	Miltronics sonic	High water, intruder, power fail	Natural gas generator
8	Nobel Ridge	Miltronics sonic	High water, intruder, power fail	Natural gas generator
9	Hidden Creek	Four mercury floats	High water, intruder, power fail	Natural gas generator & manual transfer switch
10	Brendon Estates	Four mercury floats	High water, intruder, power fail	Manual transfer switch
11	Hilltop	Four mercury floats	High water, intruder, power fail	Manual transfer switch
12	Fir Street (abandoned)	Four mercury floats	—	—
13	Newell Crest	Five mercury floats	High water, intruder, power fail	None
14	Barclay Hills	Bubbler	High water, intruder, power fail	Natural gas generator
15	18th Street	Three mercury floats	None, external alarm light and horn	None
16	18th Street	Three mercury floats	None, external alarm light and horn	None
17	Elevator	Three mercury floats	None, external alarm light	None
18	Fire Station #2 (abandoned)	Mercury floats	None	None

TABLE 5-4.
PUMP STATION BUILDING AND SITE INFORMATION

No.	Name	Building	Fenced area
1	Canemah	CMU	Not fenced
2	Amanda	Fiberglass box	20’x20’
3	Riverview	None, controls in boxes above wet well	15’x15’
4	Cook	None, underground Cornell compact pump station. ~20’ deep	Irregular shape on sidewalk
5	Parrish	18’x18’ wood frame, insulated, vinyl siding, garage door, regular door	48’x51’
6	Pease	Wood frame, insulated, vinyl siding, garage door, regular door	15’x27’
7	Settlers Point	Wood frame, drywall inside, vinyl siding, garage door, regular door; restroom	
8	Nobel Ridge	15’x15’ wood frame, particle board inside, vinyl siding, garage door, regular door	45’x45’
9	Hidden Creek	15’x18’ cramped. wood frame, insulated, vinyl siding, garage door, regular door; separate room for generator	Large
10	Brendon Estates	6’x6’ wood frame, controls only	No fence, surrounded by landscaping
11	Hilltop	Canned station directly over wet well	None
12	Fir Street	Abandoned	
13	Newell Crest	Square concrete (like daylight basement); roof hatches over each pump	
14	Barclay Hills	Underground canned P/S ~15’ deep; wood frame, wood paneling, vinyl siding, garage door for generator	
15	18th Street	Controls in private garage	
16	18th Street	Controls in private garage	
17	Elevator	Out of sight of wet well; in an area with a lot of graffiti, vandalism may be a concern, none to date	
18	Fire Station #2	Abandoned	
19	STEP System #1	Controls in private garage	
20	STEP System #2	Controls in private garage	
21	STEP System #3	Controls outside of private garage	
22	STEP System #4	Controls outside of private garage	

TABLE 5-5.
SUMMARY OF PUMP STATION RUN TIME RECORDS, 1998 – 2001

		Pump Station Run Time (hours/day)[a]									
		Wet Weather Average			Dry Weather Average		Annual Average		Recording Period 1998-2001		
No.	Name	1998-1999	1999-2000	2000-2001	1999	2000	1999	2000	Average	Maximum	Minimum
1	Canemah	3.6	2.4	1.1	0.7	0.6	2.1	1.3	1.5	8.0	0.4
2	Amanda	3.4	2.8	2.3	2.6	2.2	2.9	2.4	2.6	5.4	1.2
4	Cook	9.2	7.1	4.2	2.5	2.5	5.5	4.2	4.7	15.5	1.7
5	Parrish	0.8	1.5	1.7	0.6	1.1	0.9	1.2	1.1	2.9	0.3
6	Pease	8.6	5.2	3.4	2.7	2.7	5.0	3.4	4.0	21.5	1.5
7	Settlers Point	0.0	4.3	3.0	2.2	2.6	2.8	3.3	3.1	6.0	1.9
8	Nobel Ridge	0.0	1.3	2.4	0.3	1.1	0.4	1.3	1.4	6.7	0.0
9	Hidden Creek	2.2	2.0	2.3	2.0	2.3	2.1	2.0	1.6	3.8	0.4
10	Brendon Estates	0.3	0.2	0.2	0.1	0.1	0.2	0.2	0.2	1.1	0.1
11	Hilltop	1.0	0.6	0.8	0.7	0.7	0.8	0.7	0.7	2.5	0.4
13	Newell Crest	1.0	1.2	2.4	0.9	1.3	0.9	1.9	1.4	11.9	0.7
14	Barclay Hills	5.4	4.3	3.2	3.5	3.3	4.2	3.6	3.9	7.6	2.5

a. Pump station run times are recorded only once per week. Daily run-times shown in this table are calculated as the weekly run time divided by 7.

TABLE 5-6.
ESTIMATED PUMP STATION PEAK FLOWS

Pump Station Number and Name	Estimated Peak Flow (gpm) Existing Conditions	Estimated Peak Flow (gpm) Buildout Conditions	Percent Increase
1. Canemah	560	700	25
2. Amanda	80	80	0
3. Riverview	40	40	0
4. Cook Street	720	890	24
5. Parrish Road	1,320	1,630	23
6. Pease Road	680	1,330	96
7. Settlers Point	850	1,250	47
8. Nobel Ridge	160	170	6
9. Hidden Creek	420	540	29
10. Brendon Estates	40	40	0
11. Hilltop	80	80	0
12. Fir Street	Abandoned		
13. Newell Crest	90	380	320
14. Barclay Hills	500	560	12
15. 18th Street STEP System 1	STEP System, Flow Not Estimated		
16. 18th Street STEP System 2	STEP System, Flow Not Estimated		
17. Elevator	Serves Single Building, Flow Not Estimates		
18. Fire Station #2	Abandoned		
19 - 22. Step Systems 1 - 4	STEP System, Flow Not Estimated		

Phasing of Needed Improvements

Pump stations are identified as deficient if the estimated peak flow for buildout conditions exceeds redundant capacity. For purposes of phasing improvements, the following criteria were established:

- For current operational deficiencies (i.e., known capacity problems and pump stations whose total capacity is insufficient for estimated existing flow), improvements are scheduled in the short-term planning period (0 to 5 years).
- For pump stations whose redundant capacity does not meet existing flow estimates but no problems have been observed and pump run times are not excessive, improvements are scheduled in the early portion of the long-term planning period (6 to 10 years).
- For pump stations whose redundant capacity does not meet buildout flow estimates, improvements are scheduled in the latter portion of the long-term planning period (11 to 20 years).

EVALUATION OF INDIVIDUAL PUMP STATIONS

Pump Station 1, Canemah

The operators have noted a significant increase in this station's pumping hours in winter, indicating an infiltration and inflow problem. Wet-weather average run times at this station during the last three wet-weather reporting periods (1998-1999, 1999-2000, and 2000-2001) were 25.5, 17.0 and 7.9 hours per week, respectively. The run time during the week of the late January 2003 storm (about 3 inches of rainfall over five days) was 51.2 hours. There is a 6-inch pipe with a blind flange through-wall between the wet well and the dry well. The reason for this pipe is not clear. It is probably the remnant of a prior upgrade. There is almost no room for expansion or for a generator. The site is bounded closely by houses. The Canemah pump station's capacity is 1,200 gpm; the existing peak flow is estimated to be 560 gpm and the buildout peak flow is estimated to be 700 gpm.

Pump Station 2, Amanda

Access to the pumps and valves is difficult, as they are located in the wet well. The valves are showing signs of corrosion. The vacuum system was replaced recently and works satisfactorily. This pump station is within 300 feet of the Riverview pump station. Wet-weather average run times at this station during the last three wet-weather reporting periods were 23.8, 19.6 and 15.9 hours per week, respectively. The run time during the week of the January 31, 2003 storm was 31.2 hours. The Amanda pump station's redundant capacity is 100 gpm; the existing and buildout peak flows are estimated to be 84 gpm.

Pump Station 3, Riverview

This pump station serves only a few houses, pumping flow from those houses to the Amanda pump station, which is about 300 feet away. The pumps and valves are all located inside the wet well. There is no room for expansion or for a generator. There are homes within 200 feet of this pump station. A pinch valve on a time switch is used to control hydrogen sulfide in the force main. This system allows for recirculating the flow when in manual operation. The run time during the week of the January 31, 2003 storm was 10.3 hours. The Riverview pump station's redundant capacity is 30 gpm; the existing and buildout peak flows are estimated to be 39 gpm.

Given the proximity of the Amanda and Riverview pump stations, it is recommended that both be abandoned and replaced with a new pump station serving both pump stations service areas.

Pump Station 4, Cook Street

This pump station is located under the sidewalk with its wet well under the road. Wet-weather average run times at this station during the last three wet-weather reporting periods were 64.1, 49.9 and 29.5 hours per week, respectively. The run time during the week of the January 31, 2003 storm was 124.7 hours. The Cook Street pump station's redundant capacity is 270 gpm. The total capacity with both pumps running is estimated to be in the range of 400 to 450 gpm. The existing peak flow is estimated to be 720 gpm and the buildout peak flow is estimated to be 890 gpm. It is recommended that this station be upgraded in the short-term, to increase capacity to the design flow for buildout.

Pump Station 5, Parrish Road

The air compressor used to control hydrogen sulfide at this pump station runs almost constantly. There is room on the site for future expansion. Wet-weather average run times at this station during the last three wet-weather reporting periods were 5.7, 10.5 and 11.8 hours per week, respectively. The run time during the week of the January 31, 2003 storm was 30.5 hours. The Parrish Road pump station's capacity is 760 gpm. The total capacity with both pumps running is estimated to be in the range of 1,100 to 1,150 gpm. The existing peak flow is estimated to be 1,320 gpm and the buildout peak flow is estimated to be 1,630 gpm. However, modeling of this pump station's basin used a level of I/I that is the same as used for all basins contributing to the South End Trunk, because no I/I data is available for the individual contributing basins; actual I/I in this basin is likely to be less than the modeled value, given the age and material of the sewer pipes in the basin. It is recommended that further flow study be done to determine whether this station requires an upgrade.

Pump Station 6, Pease Road

Air injection is used for hydrogen sulfide control at this pump station. The pumps have been reported to take as long as 10 minutes to prime and occasionally fail to prime. The outlet-isolating valve was not evident during an inspection. There is no room within the fenced area for expansion or for a generator. There is some undeveloped land to the south west of the pump station. Wet-weather average run times at this station during the last three wet-weather reporting periods were 60.4, 36.2 and 23.7 hours per week, respectively. The run time during the week of the January 31, 2003 storm was 73.0 hours. The Pease Road pump station's redundant capacity is 500 gpm. The existing peak flow is estimated to be 680 gpm and the buildout peak flow is estimated to be 1,330 gpm. It is recommended that this station be evaluated for additional redundant capacity and re-siting based on new development and estimated peak flows.

Pump Station 7, Settlers Point

No priming problems have been experienced at this pump station since the installation of air valves. The inlet has a velocity dissipater. There is no room inside the fenced area for expansion, however there is room inside the building for larger pumps. Wet-weather average run times at this station during the last three wet-weather reporting periods were 0, 30.3 and 21.3 hours per week, respectively. The run time during the week of the January 31, 2003 storm was 76.9 hours. The Settlers Point pump station's redundant capacity is 830 gpm. The total capacity with both pumps running is estimated to be in the range of 1,200 to 1,250 gpm. The existing peak flow is estimated to be 850 gpm and the buildout peak flow is estimated to be 1,250 gpm. However, no actual flow data was available for this pump station, so modeling of the pump station used a citywide average for flows; actual I/I in this basin is likely to be less than the modeled value, given the age and material of the sewer pipes in the basin. It is recommended that further flow study be done to determine whether this station requires an upgrade.

Pump Station 8, Nobel Ridge

There are no air valves at this pump station. There is room for expansion on the site. A pinch valve system is installed for the control of hydrogen sulfide. This system does not work well as the force main volume is almost equal to the wet well volume. A Danfoss magflow flow meter is installed on the inlet. Wet-weather average run times at this station during the last three wet-weather reporting periods were 0, 9 and 16.6 hours per week, respectively. The run time during the week of the January 31, 2003 storm was 24.2 hours. The Nobel Ridge pump station's redundant capacity is 140 gpm. The total capacity with both pumps running is estimated to be in the range of 200 to 250 gpm. The existing peak flow is estimated to be 160 gpm and the buildout peak flow is estimated to be 170 gpm. No improvements or follow-up studies are recommended for this station.

Pump Station 9, Hidden Creek

There are no air valves at this pump station. Air injection is used for hydrogen sulfide control. There is ample space for expansion. Access is through an apartment complex parking area. Wet-weather average run times at this station during the last three wet-weather reporting periods were 15.1, 14.3 and 16.3 hours per week, respectively. The run time during the week of the January 31, 2003 storm was 22.6 hours. The Meyers Road pump station's redundant capacity is 400 gpm. The total capacity with both pumps running is estimated to be 600 gpm. The existing peak flow is estimated to be 420 gpm and the buildout flow is estimated to be 540 gpm. It is recommended that this station be upgraded in the short term, to increase redundant capacity to the design flow for buildout.

Pump Station 10, Brendon Estates

There is no sign of any valves on this pump station's outlet. This pump station serves only a few homes. The step-irons in the wet well are inaccessible as the cover overhangs the wet well wall. Wet-weather average run times at this station during the last three wet-weather reporting periods were 2.4, 1.7 and 1.4 hours per week, respectively. The run time during the week of the January 31, 2003 storm was 8.0 hours. The Brendon Estates pump station's redundant capacity is 100 gpm; the existing and buildout peak flows are estimated to be 40 gpm.

Pump Station 11, Hilltop

The pump motors at this pump station have required new capacitors twice. Wet well access takes up the entire open space of the floor. Access is gained by stepping on the motors and valves. The pump station is within 40 feet of a house. Wet-weather average run times at this station during the last three wet-weather reporting periods were 7.2, 4.2 and 5.4 hours per week, respectively. The run time during the week of the January 31, 2003 storm was 6.0 hours. The Hilltop pump station's redundant capacity is 95 gpm; the existing and buildout peak flows are estimated to be 80 gpm. This small station should be abandoned if service can be converted to gravity; otherwise the station should be replaced because of its deteriorating condition.

Pump Station 12, Fir Street

This pump station was abandoned and the local sanitary system was converted to gravity service during construction work in the summer of 2002. The former Hydronix gull wing pump station was old and required frequent maintenance. There were no air valves and knife gate valves were used for isolation. There also was no room to expand. The station's proximity to a restaurant was the cause of a recurring grease problem. The pumps were stripped and cleaned annually due to grease buildup.

Pump Station 13, Newell Crest

Air injection is used for hydrogen sulfide control at this pump station. The check valves (GA Industries) leak. This has been an ongoing problem despite the valves being rebuilt. The pump station is on a steep slope and significant movement downhill was recorded when the pump station was first constructed. Wet-weather average run times at this station during the last three wet-weather reporting periods were 6.8, 8.5 and 16.8 hours per week, respectively. The run time during the week of the January 31, 2003 storm was 7.9 hours. This pump station is in good condition. The Newell Crest pump station's redundant capacity is 120 gpm. The existing and buildout peak flow are estimated to be 92 gpm . Due to difficult access conditions for a temporary generator, it is recommended that standby power be provided with a new permanent generator on-site.

Pump Station 14, Barclay Hills

This pump station has two wet wells. A new rectangular wet well drains directly into the old wet well through a 12-inch pipe. A tributary of Newell Creek is above and within 30 feet of the wet wells. Wet-weather average run times at this station during the last three wet-weather reporting periods were 38, 30.3 and 22.5 hours per week, respectively. The run time during the week of the January 31, 2003 storm was 51.6 hours. The Barclay Hills pump station's redundant capacity is 300 gpm. The total capacity with both pumps running is estimated to be in the range of 400 to 450 gpm. The existing peak flow is estimated to be 500 gpm and the buildout flow is estimated to be 560 gpm. It is recommended that this station be upgraded in the short-term, to increase redundant capacity to the design flow for buildout.

Pump Stations 15 and 16, 18th Street STEP Systems 1 and 2

These are two STEP stations, each serving one home. The homeowners are responsible for maintenance of these stations.

Pump Station 17, Elevator

This pump station's wet well is located at the top of an elevator off High Street. It serves one office building. A special Allen wrench is required to open the access lid. The flow from this pump station could be conveyed by gravity to the sewer line in High Street but frequent blockages resulted in the construction of this pump station. The discharge gate valve is located in the wet well, making access difficult.

Pump Station 18, Fire Station #2

This small pump station served only the Fire Station and was eliminated as the local sanitary system was converted to gravity service during construction work in summer 2002.

Pump Stations 19 - 22, STEP Systems 1 through 4

These four STEP systems each serve a single home. The homeowners are responsible for maintenance of these stations. The nearest sewer is 700 feet to the northeast in South End Road.

FUTURE PUMP STATION

There is an area along Glen Oak Road where the topography will not allow gravity connection to the City's trunk lines. Public sewer service for the parcels in this low area will require a new pump station. Gravity lines to the pump station site and a force main to Highway 213 were installed as part of the 2003 Glen Oak Road improvements project in preparation for the future pump station. This effort will avoid future trenching and extend the life of the newly constructed pavement section.

SUMMARY OF RECOMMENDATIONS

Based on individual pump station site reviews, interviews with City operations staff, pump station run time evaluations, and comparison of capacity estimates with projected flows for existing and buildout conditions, the following recommendations are made for pump station capital improvements:

- Short-term planning period (0 to 5 years):
  - **Amanda and Riverview Pump Stations—**Acquire property and construct a new pump station at a site near the existing pump stations; abandon the existing pump stations.
  - **Hilltop Pump Station—**Abandon the pump station if gravity service can be provided or replace the pump station.
  - **Pease Road Pump Station—**Conduct predesign analysis of alternatives for increasing pumping capacity, sanitary service routes and potential new siting for the pump station system, and implement improvements.
  - **Cook Street Pump Station—**Conduct predesign analysis of alternatives for increasing pumping capacity at the pump station, and implement improvements.
  - **Barclay Hills Pump Station—**Conduct predesign analysis of alternatives for increasing reliable pumping capacity at the pump station, and implement improvements.
  - **Parrish Road Pump Station—**Conduct flow monitoring to better establish I/I and determine the need for improvements at the pump station.

  - **Settlers Point Pump Station—**Conduct flow monitoring to better establish I/I and determine the need for improvements at the pump station.
  - **Newell Crest Pump Station—**Acquire property and construct a new building to house an emergency standby generator at the pump station site.
  - Conduct minor (non-capital) pump station repairs by adding to the maintenance and operations budget.
- Intermediate-term planning period (6 to 10 years):
  - **Glen Oak Road Basin—**Conduct predesign analysis for a new pump station to serve low areas along Glen Oak Road. Design and implement the improvements.
- Long-term planning period (11 to 20 years):
  - **Hidden Creek Pump Station—**Conduct predesign analysis of alternatives for increasing pumping capacity at the pump station, and implement improvements.

CHAPTER 6.
CAPACITY, MANAGEMENT, OPERATION AND MAINTENANCE REGULATIONS

CMOM, an acronym for Capacity, Management, Operation and Maintenance, is an integral part of the Environmental Protection Agency's (EPA) draft Sanitary Sewer Overflow (SSO) rule. The purpose of the impending CMOM regulations is to reduce the occurrence of SSOs nationwide. The regulations will require municipalities and utilities that own sewer collection systems to address all aspects of their operations that relate to eliminating sanitary sewer overflows. The goal is to encourage municipalities to work proactively and improve their practices relating to long-term planning, management, design, construction, operation and maintenance. The regulations include regular documentation including periodic audits of the CMOM program to track progress. The anticipated benefits of the program are as follows:

- To improve water quality by reducing overflows from combined sanitary/stormwater sewers
- To reduce costs by reducing the required number of emergency responses and repairs
- To reduce costs by improving management and extending system life
- To improve customer service by reducing the number of system backups and overflows.

STATUS OF REGULATIONS

The draft SSO rule was scheduled to go to the Office of Management and Budget (OMB) in November 2002 for financial analysis. Following OMB review, the SSO rule would be scheduled for comment in the Federal Register, most likely in the spring of 2003. The CMOM regulation could be finalized and published in mid-2004.

CMOM REQUIREMENTS

The CMOM regulations will require the City to develop four written components that establish performance standards for operating the sewer collection system and provide for monitoring:

- **CMOM Program Summary**—A description of the measures and activities that makeup the City CMOM program.
- **Overflow Emergency Response Plan**—CMOM will require Oregon City to develop a formal plan for securing and containing future overflows that includes scenarios, people and equipment and periodic training. The plan must identify how overflows will be secured and contained and how regulators and the public will be notified.
- **System Evaluation and Capacity Assurance Plan**—This plan will include an evaluation of the capacity of Oregon City's entire system,

identification of measures to address hydraulic deficiencies, prioritization of system upgrades, and implementation schedules. To prevent spills due to identified system deficiencies, the plan will outline proactive maintenance measures to find and correct potential problems before they happen, preventive maintenance measures, including servicing of mechanical and electrical equipment and cleaning sewers that experience recurring problems, and corrective maintenance measures for system problems. The plan also will address data management practices for a system inventory, operation and maintenance data, information on the replacement of system elements, maintenance frequency, and asset performance.

- **Program Self-Audit**—The self-audit will be a procedure for periodically assessing and documenting the City's efforts in meeting CMOM goals.

CMOM PROGRAM ELEMENTS

For municipalities to meet CMOM regulations, the following legal, administrative, and management elements will be required:

- **Legal Authority**—Adopt a sewer use ordinance that requires proper design installation, testing and inspection (including service lines) and includes pretreatment standards for fats, oils and greases.
- **Information Management**—Maintain up-to-date mapping of the collection system and establish a process to update maps with new development; maintain a database on pipes including size, material and date constructed; maintain overflow data, three years of work order history, complaint records, performance and implementation measures, and a list of system components with inadequate capacity.
- **Overflow Response Plan—**Develop and implement an SSO response plan to stop and mitigate impacts as soon as possible. The plan must outline staff training in SSO response procedures, a process for plan review and updating, a public notification program, and steps for immediate notification of health officials and the National Pollutant Discharge Elimination System (NPDES) authority.
- **Condition Assessments—**Conduct periodic video pipe inspections and smoke testing to identify structural deficiencies and illicit connections. Update information management systems as needed based on the condition assessment.
- **Capacity Assurance**—Identify deficient components of the system for both existing and future conditions through system modeling. Develop a master plan that includes a capital improvement plan to address deficiencies. Budget for capital improvements.
- **Construction Standards**—Adopt and enforce defined design criteria that include evaluation of downstream impacts for new development, capital improvements, and rehabilitation. Require proper review of construction drawings as well as acceptance tests and inspection, including laterals.

- **Staff Training**—Provide a training program for operation and administrative personnel that includes all elements of the CMOM program. Develop a mandatory certification program.
- **Compliance Audits**—Assign responsible staff to conduct an audit of the CMOM program audit report based on interviews with staff, observations of crews, SSO data records, and work order records. The audit report is to identify apparent deficiencies, steps taken to address problems and additional measures needed.

IMPLICATIONS FOR OREGON CITY

The City of Oregon City has many of the elements of a CMOM program currently in place or in the process of being developed. The adoption of this master plan will be an important element in meeting many of the requirements of these regulations. A review and possible enhancement of these elements may be necessary. It is recommended that the City assign staff to monitor the EPA's final adoption of the rule and the DEQ's approach to permitting and enforcement, and ultimately to oversee the City's compliance.

New permitting for the City as a result of the rule is likely. The City currently has no wastewater permit, but rather operates a "satellite collection system" that discharges into facilities owned by Tri-City Sewer District (TCSD). TCSD has received an NPDES permit from DEQ for the discharge of treated wastewater. It is possible that the rule will be enforced through existing NPDES permits and new permitting of satellite collection systems.

Table 6-1 provides a checklist of Oregon City action items for CMOM.

Capacity, Management, Operation and Maintenance (CMOM)
Oregon City Checklist - Action Items

Program Category, Elements and Subelements	Description/ Comments	(Rating of adequacy or completeness 1-10)	Exists	Does not exist	Not applicable	Addressed in Master Plan
Legal Authority						
Sewer Use Ordinance		5		X		
Requires proper design		7	X			
Requires proper installation, testing and inspection (including service lines)		8	X			
Complies with pretreatment standards (fats, oils and greases)		1		X		
Information Management						
Map of collection system and process to update		7	X			
Database on overflows		5	X			
Non-compliance events due to overflows		3	X			
Maintain 3 years of work order history, complaints, performance and implementation measures		3	X			
List of system components with lack of capacity		7				X
Construction requirements and standards		5	X			
For new sewers, pumps, appurtenances		5	X			
For repair and rehabilitation projects		5	X			
Overflow Response Plan						
Develop and implement an SSO response plan		8	X			
Stop and mitigate impacts as soon as possible		8	X			
Public Notification Program		3	X			
Immediation Notification of Health Officials and NPDES Authority		10	X			
Train staff on SSO response procedures		5	X			
Develop SSO Response Plan review and updating process		1		X		
Condition Assessments and Capacity Assurance						
Structural integrity and condition		5	X			
Capacity assurance (existing collection system components)		5				
Base and peak flow adequacy		7				X
Identify deficient components		7				X
Identify measures for new capacity or flow reductions		7				X
Update Information Management Systems as needed based on condition assessment		5	X			
New Construction						
Development Improvements		5	X			
Proper sizing and evaluation of downstream impacts		5	X			
Capital Projects (including rehabilitation)		5	X			
Program to identify and prioritizestructural and hydraulic deficiencies		5	X			
Short and long-term action plans		5				
New Connections		5	X			
Proper Installation, testing, inspection, including laterals		5	X			
Determine adequacy of base and peak capacity		5	X			
If inadequate reduce flows or increase capacity		5	X			
Staff Training						
Provide training for all elements of the CMOM Program		1		X		
Develop a mandatory certification program		10	X			
Compliance Audits						
Conduct an audit of the CMOM program and submit with NPDES		1		X		
Assign responsible staff and audit schedule		1		X		
Audit report based on:		1		X		
Interviews with staff		1		X		
Observations of crews		1		X		
SSO data records		1		X		
Work order records		1		X		
Audit report to include:		1		X		
Apparent deficiencies		1		X		
Steps taken to address problems		1		X		
Additional work needed		1		X		

Table 6-1. CMOM Checklist for Oregon City

CHAPTER 7.
CAPITAL IMPROVEMENT PLAN

A capital improvement plan (CIP) was developed incorporating recommended projects with estimated costs and proposed phasing. The projects in the CIP consist of upsizing existing sewer pipes, installing new sewer pipes, upgrading or replacing existing pump stations, performing annual operation and maintenance activities, and undertaking one-time studies and purchases. Exhibit 5 at the back of this report shows the locations of all the recommended projects as well as conceptual layouts for new sewer facilities in the 2003 UGB expansion areas; specific configurations for the improvements in the expansion areas will depend on actual future development plans for those areas.

COST ESTIMATING APPROACH

Capital improvement costs were estimated as the total of construction cost, contingency and allied costs. All costs were estimated for the year 2003, based on costs for similar projects. These are budget-level estimates, not engineered-design-level estimates. Actual design and construction costs may be from 20 percent less to 35 percent more than these estimates.

Construction costs were estimated as follows:

- Budget-level estimates for pipelines were derived from recently completed Tetra Tech/KCM construction projects, with allowances for special circumstances. The following variables were accounted for:
  - Pipe diameter and length
  - Depth of pipe—Two categories were defined: 0 to 10 feet, and 10 to 15 feet.
  - Surface restoration requirements—Two types of restoration were identified: roadway restoration, including crushed rock backfill, 4-inch asphalt paving and traffic control; and grassy area restoration, including crushed rock backfill, topsoil and planting grass.

  For projects to upsize existing sewers, no allowance was made for rock excavation because the new pipe would use the existing alignment and depth. Costs for rock excavation are included in the estimates for new sewer lines.
- Pump station costs were developed based on Tetra Tech/KCM project experience with similar capacity pump stations. Appendix E itemizes pump station, force main and land acquisition cost estimates for each recommended pump station project.

The contingency used in the estimates is 20 percent of construction cost. Allied costs of 30 percent of construction cost were included for engineering, administration, legal, finance and construction administration services.

PROJECT PHASING APPROACH

Three planning periods were identified for project phasing:

- Short-Term (0 to 5 years)—These are projects required to correct existing system deficiencies. These projects should be implemented as soon as possible to prevent overflows under severe wet-weather conditions; all should be completed in the next five years.
- Intermediate-Term (6 to 10 Years)—These are projects that will be required to add capacity to the system for future development and increased density in previously developed areas. These projects are anticipated to be required within 10 years.
- Long-Term (11 to 20 Years)—These are projects that will be required as future development occurs and to expand service. Project implementation will depend on factors such as the intensity of new development or the rate of density increase within the project basin or subbasin.

RECOMMENDED SEWER PIPE PROJECTS

Pipeline projects include those intended to increase system capacity and those intended to extend the sewer service area. Sheets describing each project are included at the end of this chapter. Recommended pipe sizes were calculated as those required to provide adequate capacity for projected buildout conditions for all areas within the urban growth boundary.

Capacity Upgrades

Table 7-1 summarizes the projects that are recommended to ensure that the collection system has sufficient capacity for buildout flows. All of these projects would upsize existing sewer pipes in their current alignment. No alternative routes were found to be beneficial.

Sewer System Extensions

Table 7-2 summarizes the projects to extend the sewer system to developed areas currently without service. These projects are anticipated to be completed over 10 years. The total costs are split equally between short- and intermediate-term categories. Implementation of sewer extension projects will depend on the demand for sewer service in currently unsewered areas. This in turn will depend on the usable life left in existing septic systems and on the demand for new subdivision development. Because these factors cannot be predicted with any certainty, no implementation dates have been assigned for the extension projects, but it is anticipated that these projects will be required within the next 10 years. It is recommended that the City budget 10 percent of the total cost annually and implement projects in two-year cycles. A few projects can be designed one year and constructed the next year until all the projects have been completed. Projects to be completed during each cycle can be selected by prioritizing demand at that time.

TABLE 7-1. SEWER PIPELINE CAPACITY UPGRADES		
Project Number	Category	Estimated Cost
BC-COL-1	Short-Term	$100,000
GO-COL-1	Short-Term	$205,000
BW-COL-1	Short-Term	$315,000
TW-COL-1	Short-Term	$115,000
HO-COL-1	Intermediate	$260,000
BC-COL-2	Intermediate	$300,000
BC-COL-3	Intermediate	$650,000
MC-COL-1	Intermediate	$85,000
BW-COL-2	Intermediate	$145,000
GO-COL-2	Intermediate	$40,000
Total		**$2,215,000**

TABLE 7-2. SEWER SYSTEM EXTENSION PROJECTS		
Project Number	Length (feet)	Total Cost
CA-COL-1	3,497	$735,000
SE-COL-1	1,125	$165,000
SE-COL-2	1,860	$270,000
SP-COL-1	4,850	$695,000
Z1-COL-1	2,800	$400,000
Z1-COL-2	1,965	$285,000
GO-COL-3	895	$130,000
BC-COL-4	2,020	$290,000
PE-COL-1	1,000	$145,000
Z2-COL-1	2,864	$410,000
Total CIP costs		**$3,525,000**
Cost per year over 10 years		*$352,500*

RECOMMENDED PUMP STATION IMPROVEMENTS

Table 7-3 lists recommended projects to upgrade or replace existing pump stations and to retrofit all pump stations with capacity over 1,000 gpm with permanent flow meters and recorders. In addition to these projects, it is recommended that the City's current SCADA (supervisory control and data acquisition) system project include retrofitting all existing pump stations with data loggers to log the time when each pump is switched on and off.

TABLE 7-3.
PUMP STATION IMPROVEMENT PROJECTS

Pump Station	Required Capacity (gpm)	Short-Term Cost	Intermediate-Term Cost	Improvement Description
Amanda/Riverview[a]	123	$470,000		Replace
Cook[b]	891	$480,000		Upsize for current flows
Pease Rd[c]	1,327	$25,000	$1,110,000	Relocate and upsize for current flows
Newell Crest	92	$40,000		Provide permanent standby power
Barclay Hills[b]	560	$450,000		Upsize for current flows
Hidden Creek	540	$40,000		Upside for future flows
Hilltop[a, d]	78	$210,000		Replace or abandon and use a new gravity main
Glen Oak Road	145		$295,000	New pump station to provide service for low areas
Pump Station Flow Meters	—	$30,000		Critical to determine capacity deficiencies
Pump Station Data Loggers[e]	—			
Total Pump Station Costs		**$1,745,000**	**$1,405,000**	

a. Submersible pump station with valve vault, no standby power.
b. Use existing wet well.
c. Site selection and preliminary design in the short term, final design and construction in the intermediate term.
d. Cost shown is for pump station replacement; no cost estimate was made for abandoning the station in lieu of a new gravity main.
e. No cost developed; it is recommended that this improvement be incorporated in a current City telemetry-upgrade project.

Before proceeding with design on the recommended pump station upgrades, estimated I/I flows should be verified. Some pump stations serve a large basin or subbasin where data from only one flow monitor was available to determine I/I values. It is likely that there are variations in I/I throughout these basins and this may have a substantial impact on the pump station design flow rate. Similarly, pump run times should be monitored for all pump stations to assist in better predicting when upgrades will be required.

RECOMMENDED ANNUAL ACTIVITIES

TV Inspection

TV inspections are a useful tool for inspecting the condition of a sewer pipe to locate the presence of foreign objects and local pipe defects. The City has approximately 108 miles (572,000 feet) of sewer pipe in its collection system and has TV-inspected approximately

120,000 feet of it over the last 3 years. The total cost of these inspections was approximately $88,500. This equates to approximately 40,000 feet per year (7 percent of the system) at an annual cost of $30,000.

It is recommended that the City continue TV inspections to a greater extent, approximately 60,000 feet per year at an estimated annual cost of $44,000. At this rate, the entire system will be inspected over a 10-year cycle. The older and higher maintenance areas should be the highest priority.

Defects observed from TV inspection should be entered into an asset management database to track their repair and facilitate ranking of the defects to ensure that the worst defects are repaired first. In addition, funds should be allocated under the pipe replacement budget for the repair of defects found.

Smoke Testing

Smoke testing is a useful tool for locating broken pipes, cross connections with storm drains, roof drain connections, foundation drain connections, and manhole connectivity. During the summer of 2002, Oregon City was involved in a collaborative effort with Clackamas County, Gladstone and West Linn to smoke test areas in all three cities. The cities plan to spend six to eight weeks smoke testing during the summer of 2003. They anticipate that smoke testing can be completed in all old areas if the collaborative effort is continued during the summers of 2003 and 2004.

Flow monitoring can be used to identify areas with unacceptably high I/I. These areas should be targeted for further investigation through smoke testing to identify defects and cross connections. It is estimated that the City's 2002 smoke testing cost $6,000, and that amount should be budgeted annually for future smoke tests.

Grease Control

Restaurants, hotels and some industries dispose of a significant amount of grease into the sewer system. As the wastewater cools, the grease coagulates and is deposited on the pipe walls and builds up in the sewer lines. This results in increased maintenance costs and backups leading to overflows. Grease can be effectively trapped through properly designed and maintained grease traps. The Tri-City Service District Rules and Regulations Article 6.2 "Grease, Oil, Sand and Scum Traps" requires the installation and maintenance of grease separators and establishes authority for the District or the City to determine and enforce compliance. Installation and maintenance of the grease traps are the responsibility of the business discharging the high grease loads.

It is estimated that the City's annual costs for removing grease from the system have been $28,000. This amount should be budgeted annually for future grease removal and/or the enforcement of the grease control requirements.

Flow Monitoring

Monitoring provides essential information for quantifying I/I and determining when flows to a pipe or pump station are approaching the facility's maximum capacity. Flow monitoring was done for this master plan to provide data for calculating I/I rates. At present, no other sewer flow monitoring is underway.

It is recommended that the City purchase a minimum of three data logging flow monitors and that these be installed in the sewer system at strategic locations. This will allow City staff to collect up-to-date flow data. This information can be used to refine I/I flow estimates and direct smoke testing efforts. The data also will be useful for future master planning efforts. The estimated annual cost for recommended flow monitoring, not including the cost to purchase the recommended monitors, is $18,000.

It is important that the flow monitors be installed and functioning throughout the winter to record high flows during large storm events. The flow monitors also should be used at the same locations to record dry-weather flows for four to six weeks during summer; this information is necessary to determine I/I parameters used in hydraulic modeling software. When flow data is required at a point in the middle of a trunk line, data should be recorded at two locations simultaneously to allow subtraction of the upper basin data from the lower basin data to calculate flows originating in the mid basin. When more than three monitors are required at one time, additional monitors can be rented.

Small Works

$30,000 to $50,000 is currently budgeted each year for spot repairs of the sewer system. The City executes these repairs as resources allow. This budget appears to be reasonable. It is recommended that once several defects have been identified through TV inspections, smoke testing, and other means, bids from commercial contractors be obtained to perform a number of repairs at one time.

Pipe Replacement

The City's capital facility plan provides a budget of $990,000 for the replacement of old sewer lines. An additional three projects with a combined budget of $515,000 have been approved for the replacement of old sewer lines in Canemah. These projects have not yet been executed.

We recommend that an annual budget be assigned for the replacement of old sewer lines. The annual TV inspections of sewer lines can be used to determine which sewer lines are in need of replacement. The pipes identified one year can be scheduled for replacement the following year in an ongoing cycle. An initial budget of $200,000 per year should be allowed for sewer replacement. This amount should be reviewed from time to time based on the information gathered in the TV inspections. Once several sections of pipe have been identified as requiring repair, a consultant can, if necessary, be employed to make recommendations on the method of repair, verify pipe sizes required and prepare a contract for the necessary pipe replacements.

Summary of Annual Activities

Table 7-4 lists the proposed budget costs for recommended annual activities.

TABLE 7-4.
ANNUAL ACTIVITIES

Project Number	Project Name	Budget
AN1	Smoke testing	$6,000
AN2	Grease cleaning	$28,000
AN3	Flow monitoring	$18,000
AN4	TV inspections	$44,000
AN5	Small works	$50,000
AN6	Pipe replacement	$200,000
Total		**$346,000**

RECOMMENDED ONE-TIME STUDIES AND PURCHASES

Table 7-5 presents phasing and estimated costs for the following recommended one-time studies and purchases related to the City's sewer system:

- Determine the extent and benefits of further sewer separation projects in the McLoughlin Area.
- Determine if capacity problems exist in a number of pipelines.
- Determine I/I values and capacity needs for the Parrish Road and Settlers Point pump stations.
- Purchase flow monitoring equipment.
- Implement an asset management program, including software purchase if required (this is discussed in more detail below).
- Update the sewer master plan about every 10 years.

CMOM regulations will require a system inventory with operation and maintenance data and information on the replacement of system elements, maintenance frequency, and asset performance. It is recommended that the City investigate purchasing asset management software. Several such systems are available and care should be taken to ensure that the system purchased integrates effectively with the map of the City's system and other system information. Consideration also should be given to a system that can integrate other City infrastructure, such as water, storm drainage and roads, and import that data to hydraulic modeling software for future system analysis.

OVERVIEW OF CIP PHASING AND ESTIMATED COSTS

Table 7-6 summarizes the phasing and estimated costs for the recommended CIP.

TABLE 7-5.
RECOMMENDED ONE-TIME STUDIES AND PURCHASES

Project Number	Project Name	Project Description	Term	Budget
NS1	McLoughlin Sewer Separation	Study to review smoke test information and identify cross connections and areas of excessive I/I in the area bounded by Washington Street, 16th Avenue, Division Street and 7th Street. Recommend improvements and estimate costs.	Short	$20,000
NS2	Linn Avenue I/I Study and Sewer Capacity	Additional flow monitoring in smaller subbasins draining to this sewer to determine the extent of I/I variation in this basin. Perform hydraulic analysis on this trunk line.	Short	$20,000
NS3	Parrish Road Trunk	Verify I/I values and assess pipe capacity. Recommend any improvements necessary and estimate costs.	Short	$20,000
NS4	Parrish Road Pump Station	Verify I/I values and pump run times under large rain events, assess inflow capacity. Recommend any improvements necessary and estimate costs.	Short	$20,000
NS5	Settlers Point Pump Station	Verify I/I values and pump run times under large rain events, assess inflow capacity. Recommend any improvements necessary and estimate costs.	Short	$20,000
NS6	Flow Monitors	Purchase three flow monitors	Short	$15,000
NS7	Asset Management	Asset management software.	Short	$25,000
NS8	Master Plan Update	Update the master plan.	Long	$200,000
Total				**$340,000**

TABLE 7-6.
COST SUMMARY FOR ALL RECOMMENDED PROJECTS

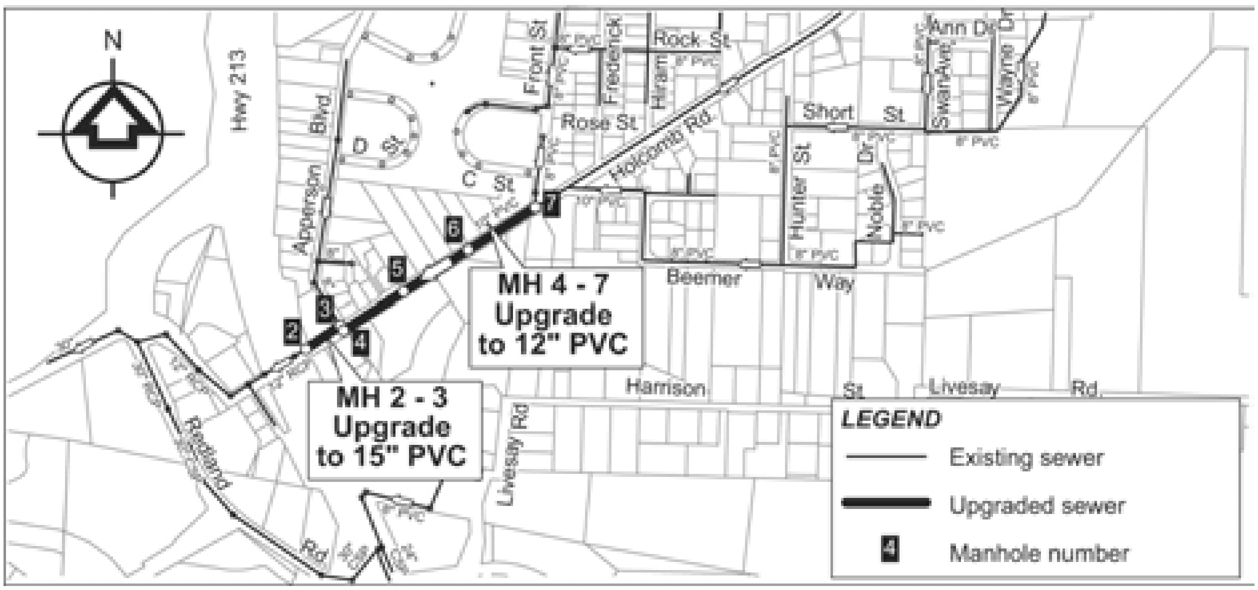
Project type	Total Costs: Short-Term	Total Costs: Intermediate-Term	Total Costs: Long-Term	**Total for All Planning Periods**
Sewer Capacity Upgrade	$735,000	$1,480,000	—	**$2,215,000**
Sewer System Extensions	$1,762,500	$1,762,500		**$3,525,000**
Pump Station Improvements	$1,745,000	$1,405,000	—	**$3,150,000**
One-Time Studies and Purchases	$140,000	$0	$200,000	**$340,000**
Annual Activities	$1,730,000	$1,730,000	$3,460,000	**$6,920,000**
Total for All Projects	**$6,112,500**	**$6,377,500**	**$3,660,000**	**$16,150,000**

SEWER PIPE IMPROVEMENT PROJECT
DATA SHEETS

PROJECT HO-COL-1 DATA SHEET

Project Name: Holcomb Road

Project Description: Replace existing 12-inch sanitary sewer line with 15-inch PVC between Manhole 2 and Manhole 3. Replace existing 10-inch sanitary sewer line with 12-inch PVC between Manhole 4 and Manhole 7.



Project Justification: Provide sufficient capacity for buildout flows. Prevent excessive surcharging.

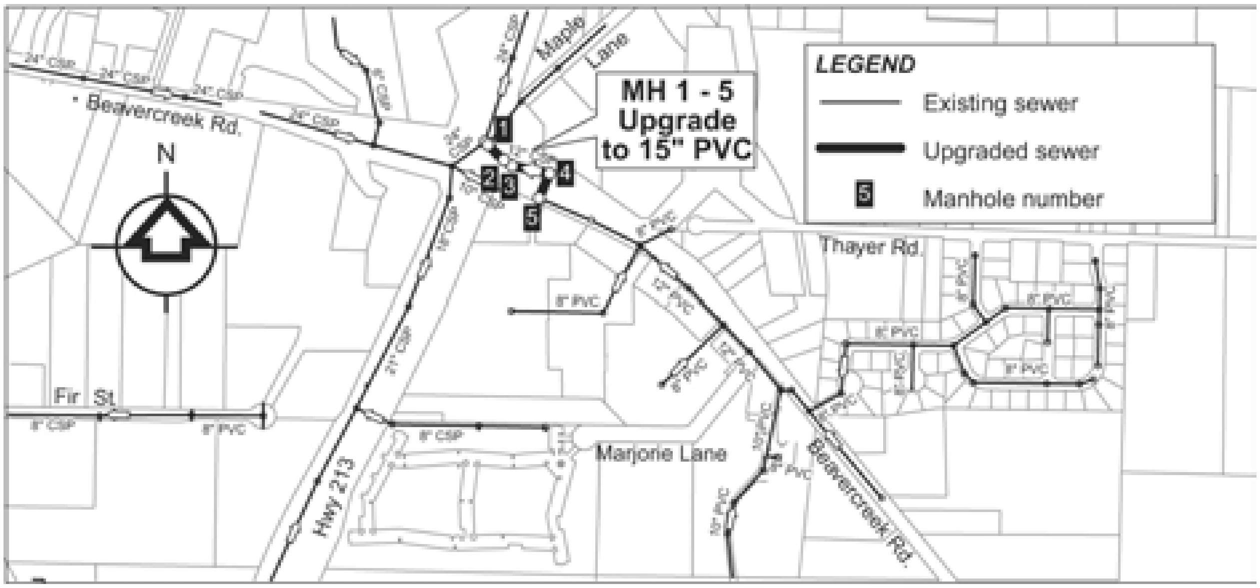
Planning Period: Intermediate-Term

Estimated Cost $260,000

PROJECT BC-COL-1 DATA SHEET

Project Name: Beavercreek Road

Project Description: Replace existing 12-inch sanitary sewer line with 15-inch PVC between Manhole 1 and Manhole 5. This project is being completed in summer 2003 as part of the Highway 213/Beavercreek Road intersection improvement project.



Project Justification: Provide sufficient capacity for buildout flows. Prevent overflow at Manhole 9, Manhole 10 and Manhole 11.

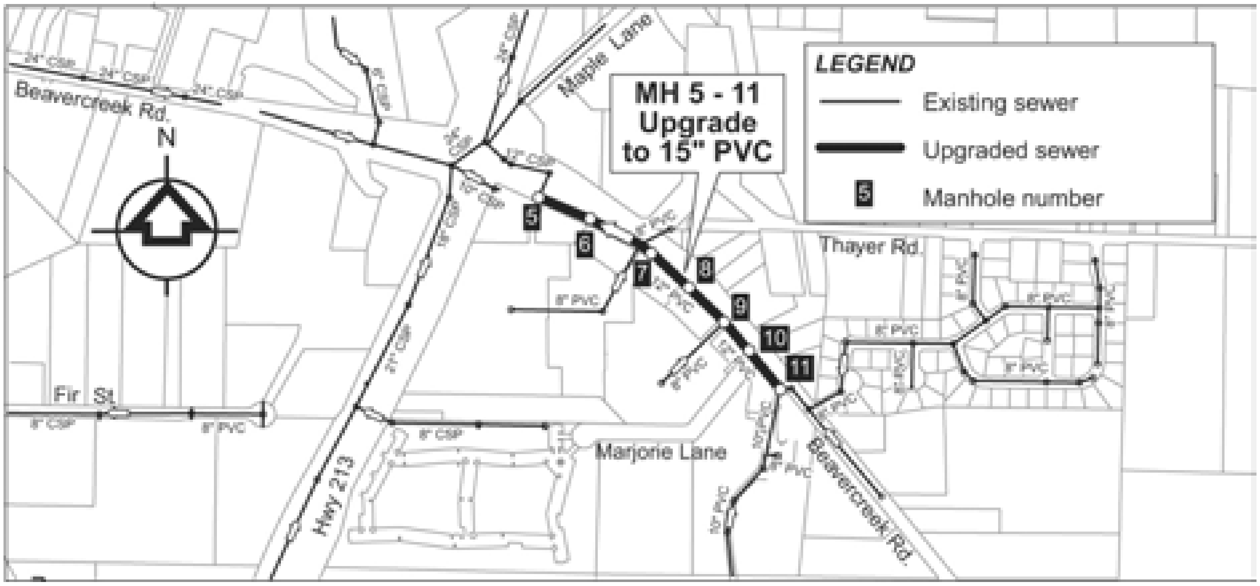
Planning Period: Short-Term

Estimated Cost $100,000

PROJECT BC-COL-2 DATA SHEET

Project Name: Beavercreek Road

Project Description: Replace existing 12-inch sanitary sewer line with 15-inch PVC between Manhole 5 and Manhole 11.



Project Justification: Provide sufficient capacity for buildout flows. Prevent overflow between Manhole 4 and Manhole 11.

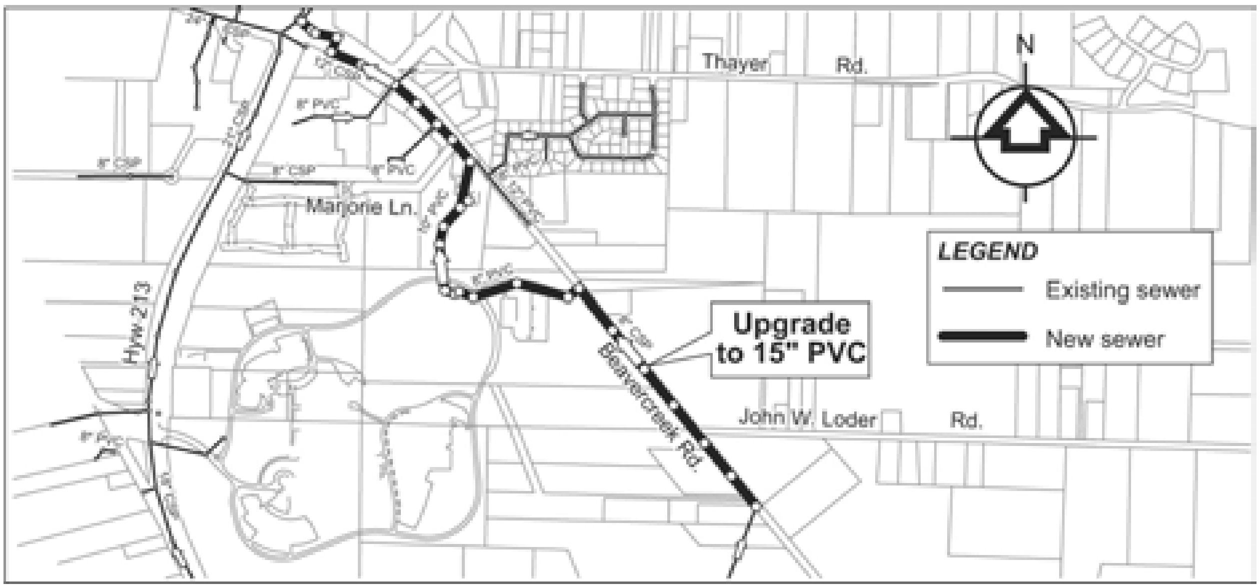
Planning Period: Intermediate-Term

Estimated Cost $300,000

PROJECT BC-COL-3 DATA SHEET

Project Name: Beavercreek Road Sewer Extension

Project Description: Replace existing 10-inch and 12-inch sewer with 12-inch and 15-inch PVC between Manhole 11 and a point just south of John Loader Road. A detailed study of the sewer system for the adjoining 2003 UGB expansion area should be done before implementing this project.



Project Justification: Provide sufficient capacity for buildout flows.

Planning Period: Intermediate-Term

Estimated Cost $650,000

PROJECT BC-COL-4 DATA SHEET

Project Name: Beavercreek Sewer Extension

Project Description: Provide sewer service to unsewered areas: Beavercreek Road from a point 620 feet north of Glen Oak Road to existing manhole 700 feet south of John W. Loder Road.

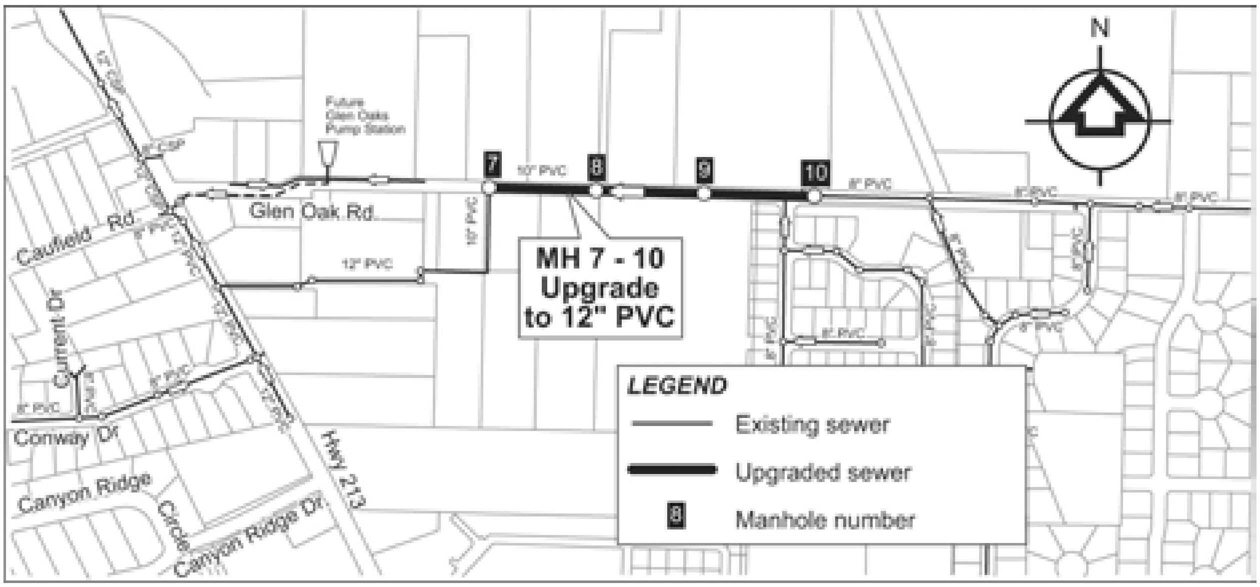
Project Justification: No existing sewer service.

Planning Period: Intermediate term

Estimated Cost $290,000

PROJECT GO-COL-1 DATA SHEET

Project Name:	Glen Oak Road
Project Description:	Replace existing 10-inch sanitary sewer line with 12-inch PVC between Manhole 7 and Manhole 10. This project was completed in summer 2003 pas part of Glen Oak Road improvements.

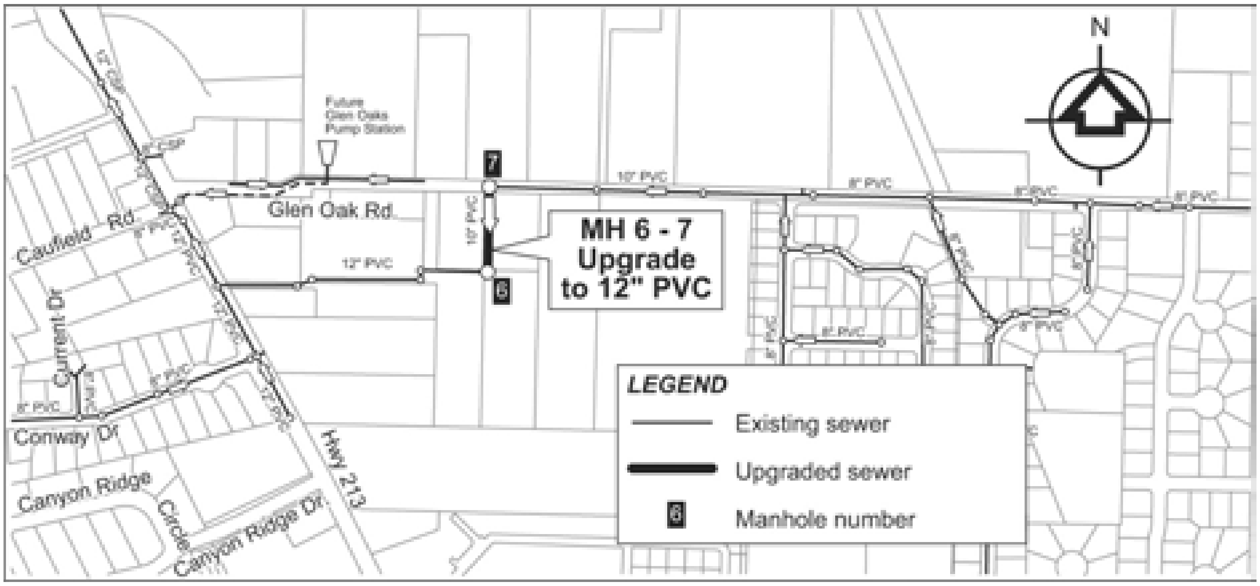


Project Justification:	Replace sewer before road improvement project; additional sewer capacity required for buildout to prevent overflow between Manhole 4 and Manhole 11.
Planning Period:	Short-Term
Estimated Cost	$205,000

PROJECT GO-COL-2 DATA SHEET

Project Name: Glen Oak Road

Project Description: Replace existing 10-inch sanitary sewer line with 12-inch PVC between Manhole 6 and Manhole 7.



Project Justification: Provide sufficient capacity at buildout; prevent excessive surcharging at Manhole 7.

Planning Period: Intermediate-Term

Estimated Cost $40,000

PROJECT GO-COL-3 DATA SHEET

Project Name: Connie Court/Beavercreek Road Sewer Extension

Project Description: Provide sewer service to unsewered areas: Connie Court to Glen Oak Road; and in Beavercreek Road from Glen Oak Road to a point 500 feet north along Beavercreek Road.

Project Justification: No existing sewer service.

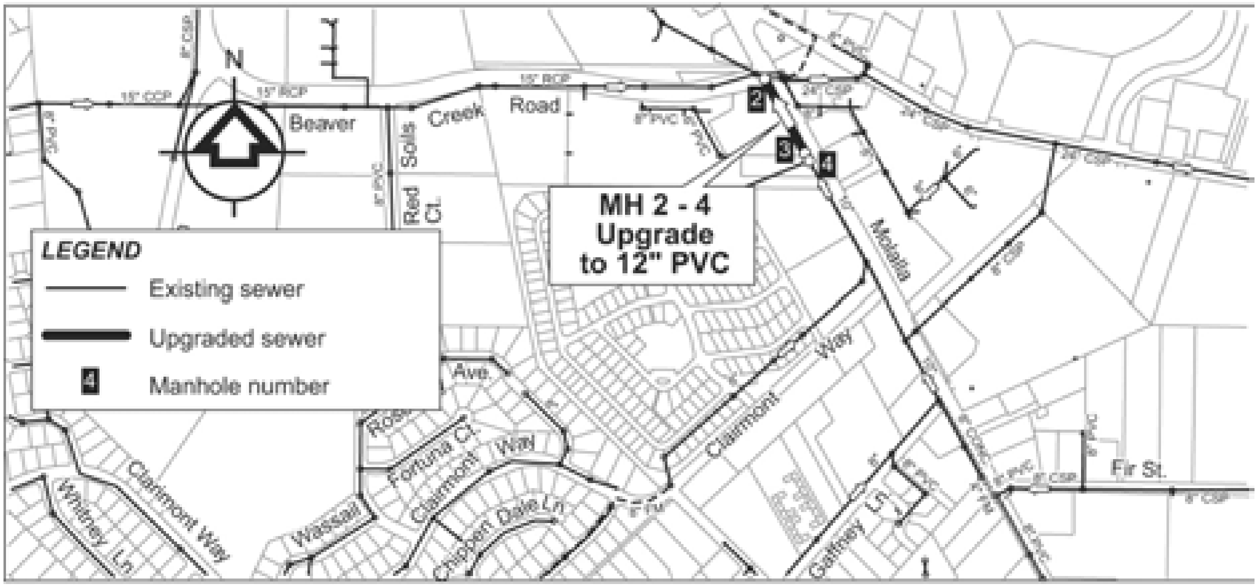
Planning Period: Intermediate term

Estimated Cost $130,000

PROJECT MC-COL-1 DATA SHEET

Project Name: Molalla Avenue

Project Description: Replace existing 10-inch sanitary sewer line with 12-inch PVC between Manhole 2 and Manhole 4.



Project Justification: Provide sufficient capacity at buildout; prevent excessive surcharging.

Planning Period: Intermediate-Term

Estimated Cost $85,000

PROJECT BW-COL-1 DATA SHEET

Project Name: Warner Milne Road

Project Description: Replace existing 8-inch sanitary sewer line with 12-inch PVC between Manhole 3A4 and Manhole 3A11.

Project Justification: Provide sufficient capacity for existing conditions. Upsizing to 12-inch PVC will provide capacity for buildout conditions.

Planning Period: Short-Term

Estimated Cost $315,000

PROJECT BW-COL-2 DATA SHEET

Project Name: Warner Milne Road to Molalla Avenue

Project Description: Replace existing 10-inch sanitary sewer line with 15-inch PVC between Manhole 3 and Manhole 3A2.

Project Justification: Provide sufficient capacity at buildout.

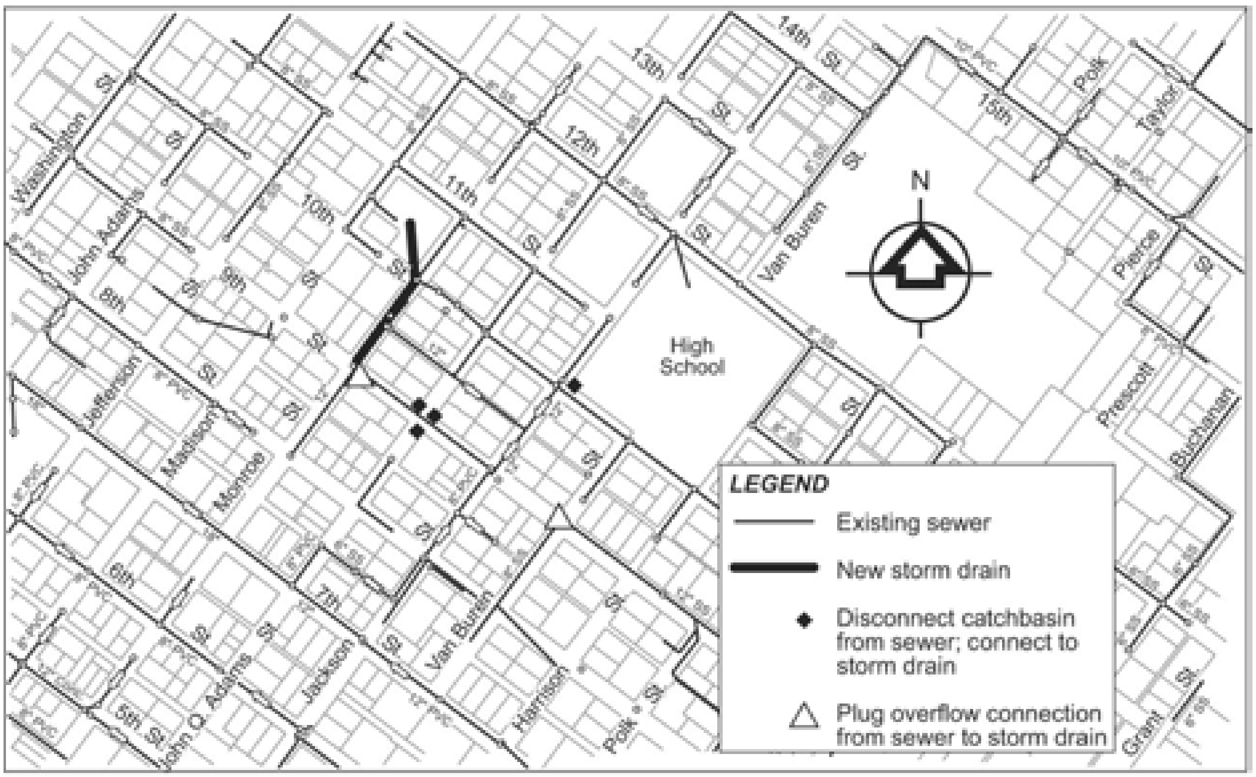
Planning Period: Intermediate-Term

Estimated Cost $145,000

PROJECT TW-COL-1 DATA SHEET

Project Name: Twelfth Street Basin Inflow Connection Corrections

Project Description: Disconnect 3 storm drain catch basins at the intersection of Adams St. and 9th Ave. and a catch basin at Jackson St. and 10th Ave. adjacent to the High School that tie into the sewer system and reconnect to the existing storm drain. Permanently plug sewer overflows from the sewer to storm drain at Jackson St. between 9th and 10th Avenue and at Adams St. and 10th Avenues. Due to additional flows directed to the storm drain in 9th Ave. and Monroe St, several sections of 8-inch and 12-inch pipe will need to be replaced with 18-inch pipe.



Project Justification: Elimination of known inflow sources.

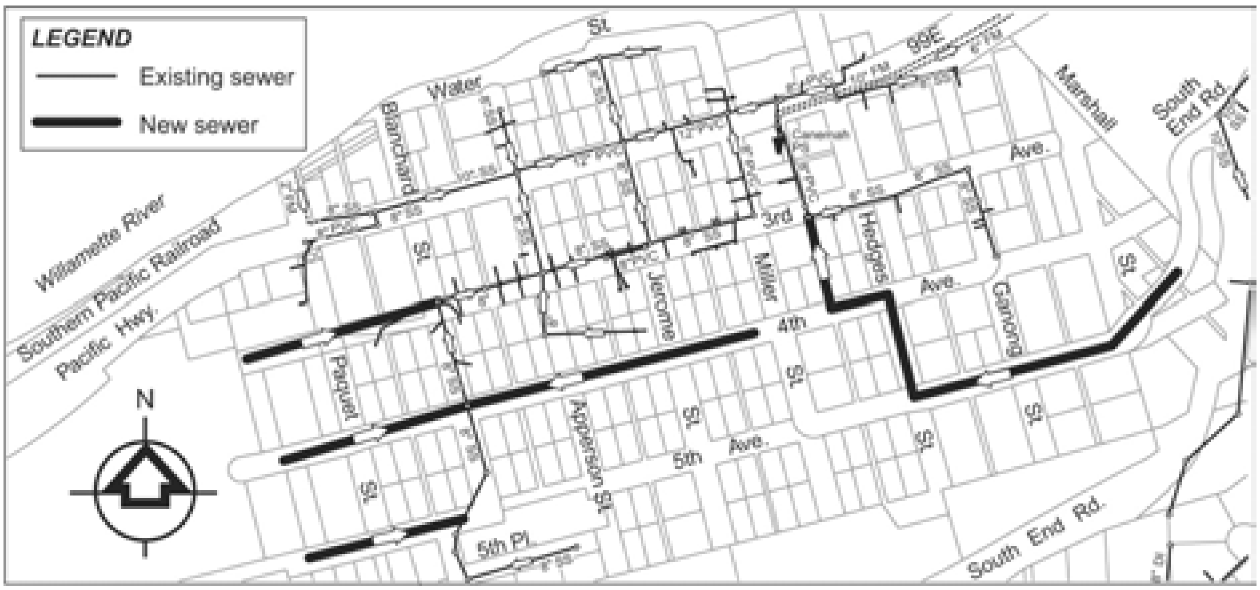
Planning Period: Short term

Estimated Cost $115,000

PROJECT CA-COL-1 DATA SHEET

Project Name: Canemah Sewer Extension

Project Description: Provide sewer service to unsewered areas: 3rd Avenue between Paquet Street and Blanchard Street; 4th Avenue between Paquet Street and Miller Street; 5th Avenue between Paquet Street and Blanchard Street; 5th Avenue from Marshall Street to Hedges Street; down Hedges Street, south along 4th Avenue and down lane to existing manhole on 3rd Avenue.



Project Justification: No existing sewer service.

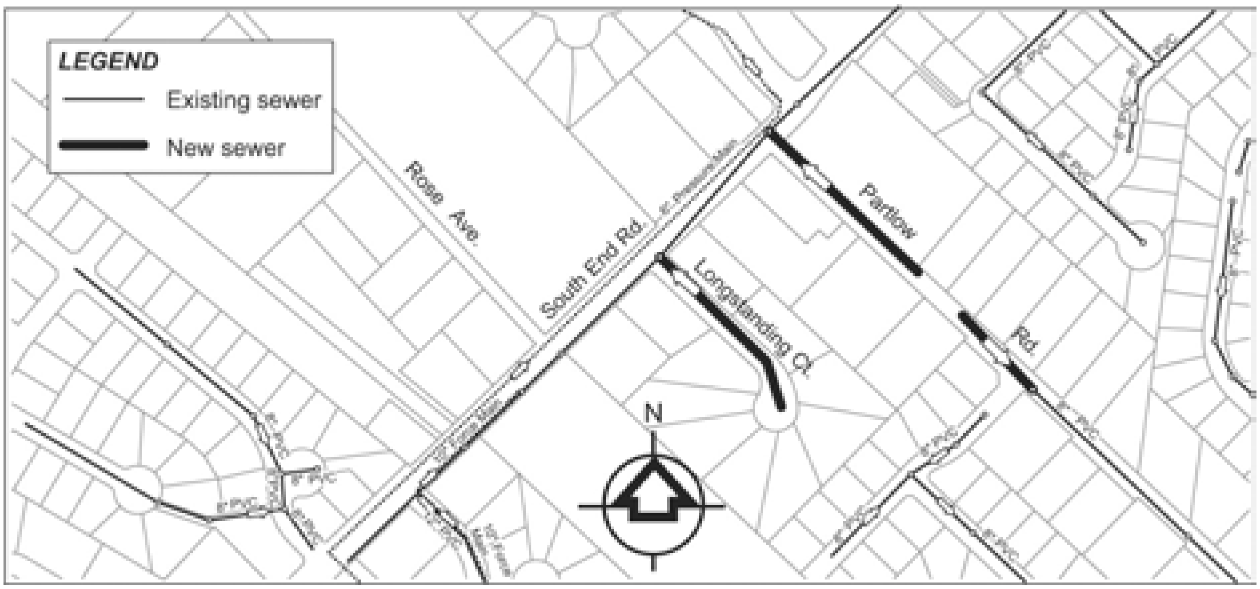
Planning Period: Short term

Estimated Cost $735,000

PROJECT SE-COL-1 DATA SHEET

Project Name: Partlow Road/Longstanding Court Sewer Extension

Project Description: Provide sewer service to unsewered areas: Partlow Road south of South End Road; and in Longstanding Court to South End Road.



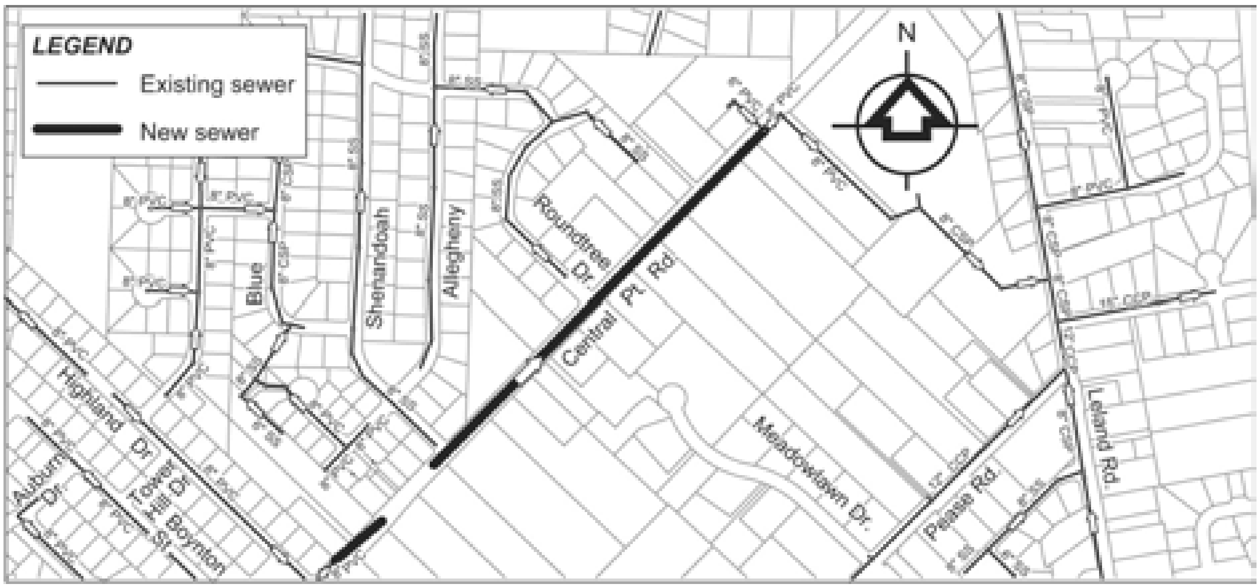
Project Justification: No existing sewer service.

Planning Period: Short term

Estimated Cost $165,000

PROJECT SE-COL-2 DATA SHEET

Project Name:	Central Point Sewer Extension
Project Description:	Provide sewer service to unsewered areas: Central Point Road from Highland Drive to existing manhole 2,000 feet to the north along Central Point Road.

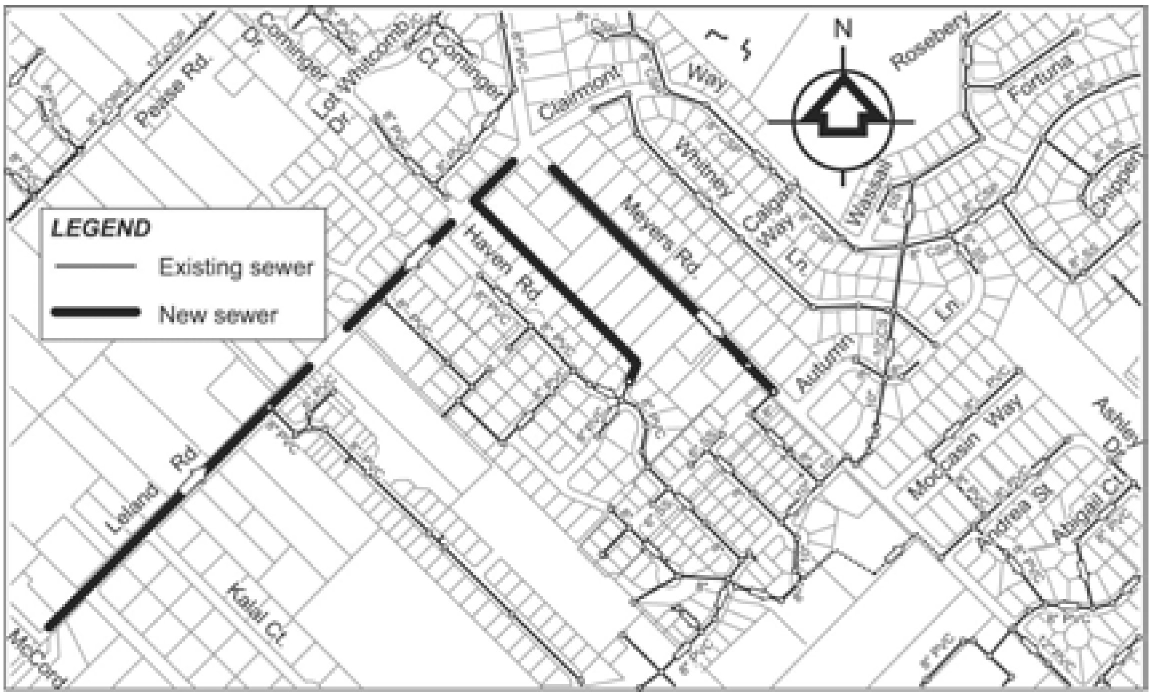


Project Justification:	No existing sewer service.
Planning Period:	Short term
Estimated Cost	$270,000

PROJECT SP-COL-1 DATA SHEET

Project Name: Leland Meyers Sewer Extension

Project Description: Provide sewer service to unsewered areas: Leland Road from McCord Road to Meyers Road; and Meyers Road from Leland Road to Autumn Lane.



Project Justification: No existing sewer service.

Planning Period: Short term

Estimated Cost $695,000

PROJECT Z1-COL-1 DATA SHEET

Project Name:	Gaffney Falcon Sewer Extension
Project Description:	Provide sewer service to unsewered areas: Gaffney Lane from Ashley Drive to Falcon Drive; up lane off Gaffney Lane opposite S. Glenview Court; and in Falcon Drive from Gaffney Lane to existing manhole 350 feet north of Castleberry Loop.

Project Justification:	No existing sewer service.
Planning Period:	Short term
Estimated Cost	$400,000

PROJECT Z1-COL-2 DATA SHEET

Project Name:	Caufield Sewer Extension
Project Description:	Provide sewer service to unsewered areas: Caufield Road from Conway Drive to Highway 213.

Project Justification:	No existing sewer service.
Planning Period:	Intermediate term
Estimated Cost	$285,000

PROJECT Z2-COL-1 DATA SHEET

Project Name: 18th Street/Anchor Way Sewer Extension

Project Description: Provide sewer service to unsewered areas: 18th Street from Jackson Street to Anchor Way, Van Buren Street from 18th Street to 17th Street, Harrison Street from 18th Street to 17th Street, Division Street from 17th Street to Anchor Way, Anchor Way from 18th Street to bridge over Abernethy creek.

Project Justification: No existing sewer service.

Planning Period: Intermediate term

Estimated Cost $410,000

PROJECT PE-COL-1 DATA SHEET

Project Name: McCord Road Sewer Extension

Project Description: Provide sewer service to unsewered areas northeast of McCord Road between Pease Road and Leland Road. Relocation of the Pease Road Pump Station should be evaluated during the predesign phase.

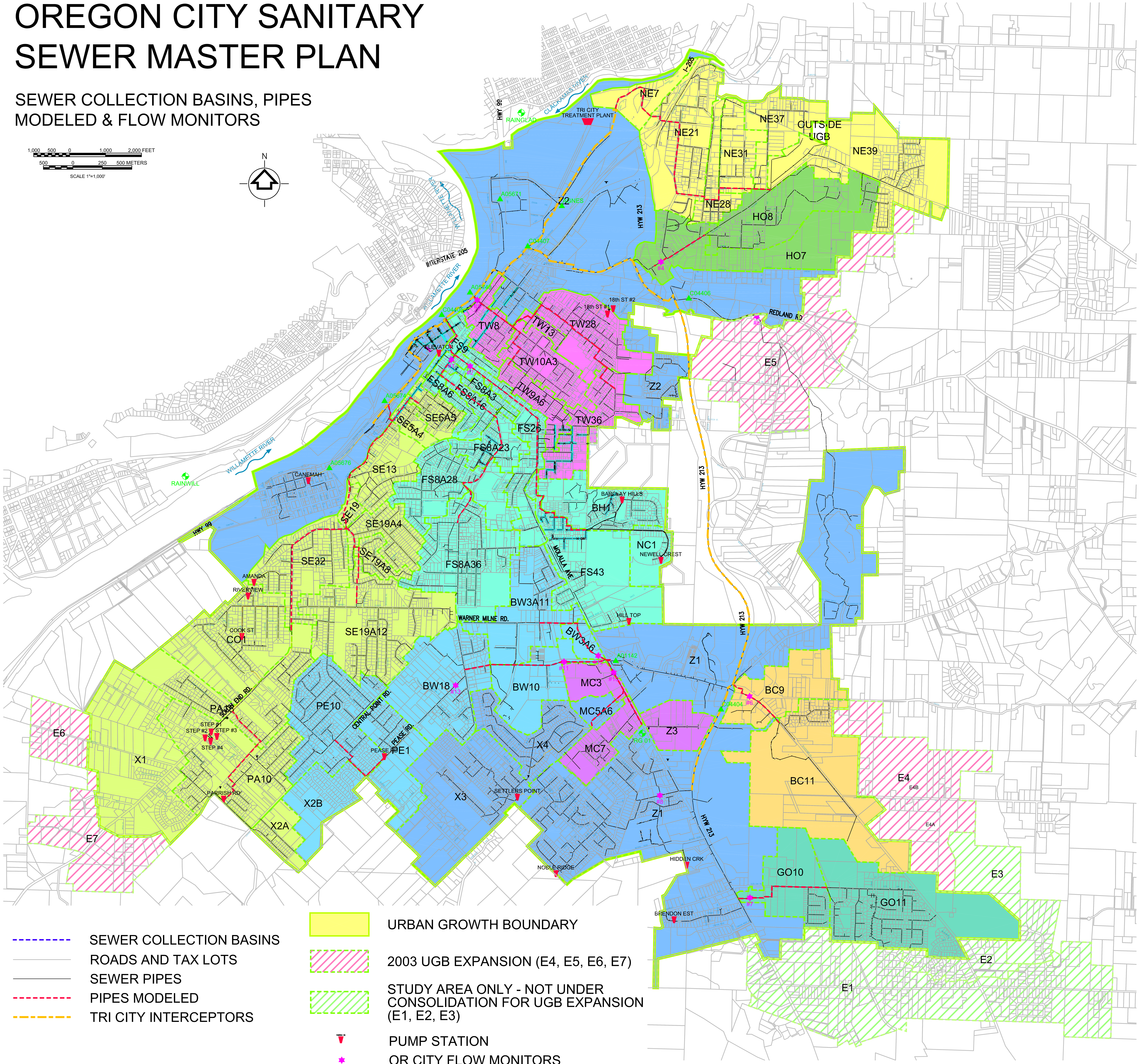
Project Justification: No existing sewer service.

Planning Period: Intermediate term

Estimated Cost $145,000

OREGON CITY SANITARY SEWER MASTER PLAN

SEWER COLLECTION BASINS, PIPES MODELED & FLOW MONITORS

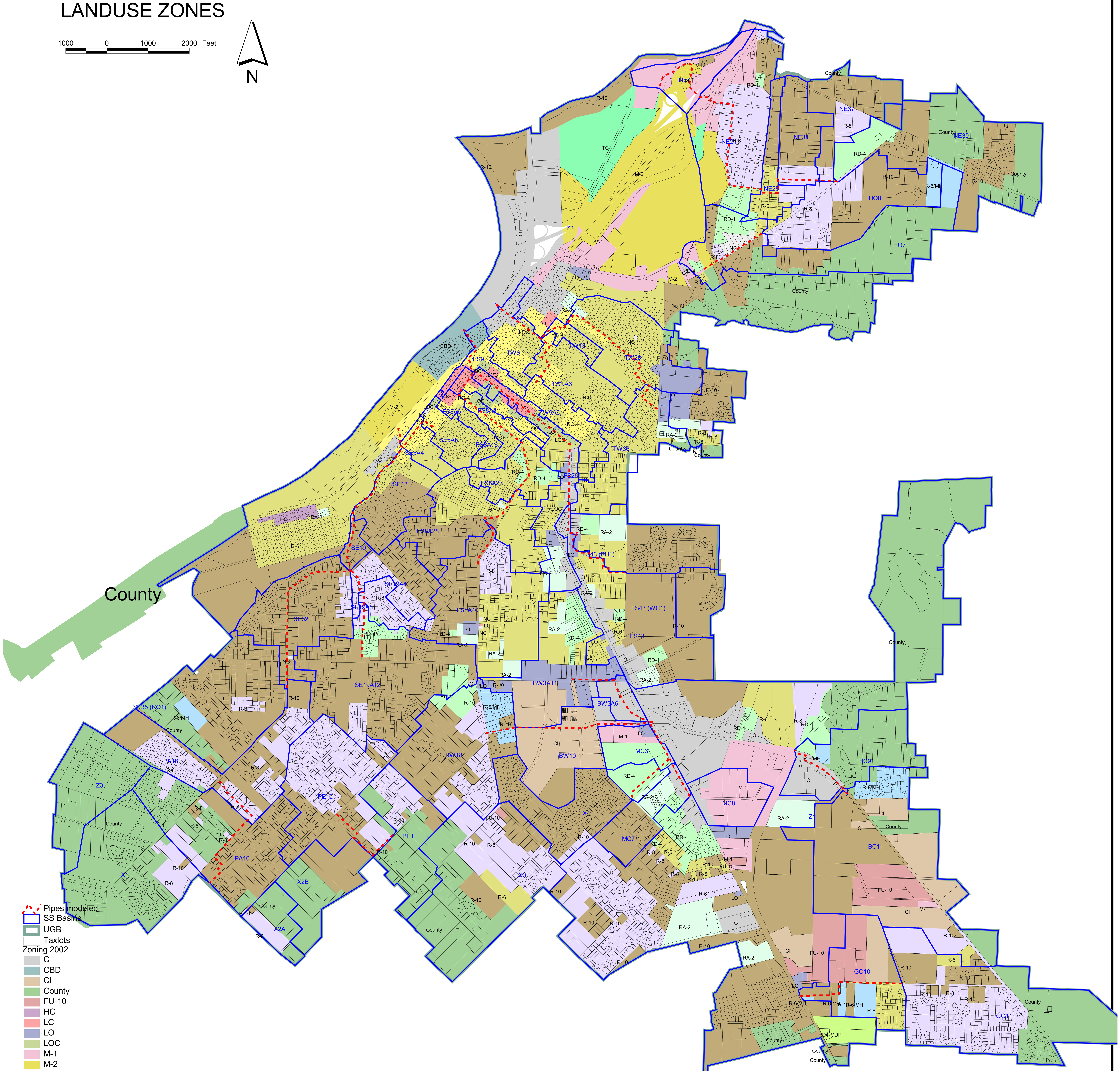


- SEWER COLLECTION BASINS
- ROADS AND TAX LOTS
- SEWER PIPES
- PIPES MODELED
- TRI CITY INTERCEPTORS
- URBAN GROWTH BOUNDARY
- 2003 UGB EXPANSION (E4, E5, E6, E7)
- STUDY AREA ONLY - NOT UNDER CONSOLIDATION FOR UGB EXPANSION (E1, E2, E3)
- PUMP STATION
- OR CITY FLOW MONITORS (#1)
- TRI CITY FLOW MONITORS (A05676)

EXHIBIT 1

OREGON CITY SANITARY SEWER MASTER PLAN

LANDUSE ZONES

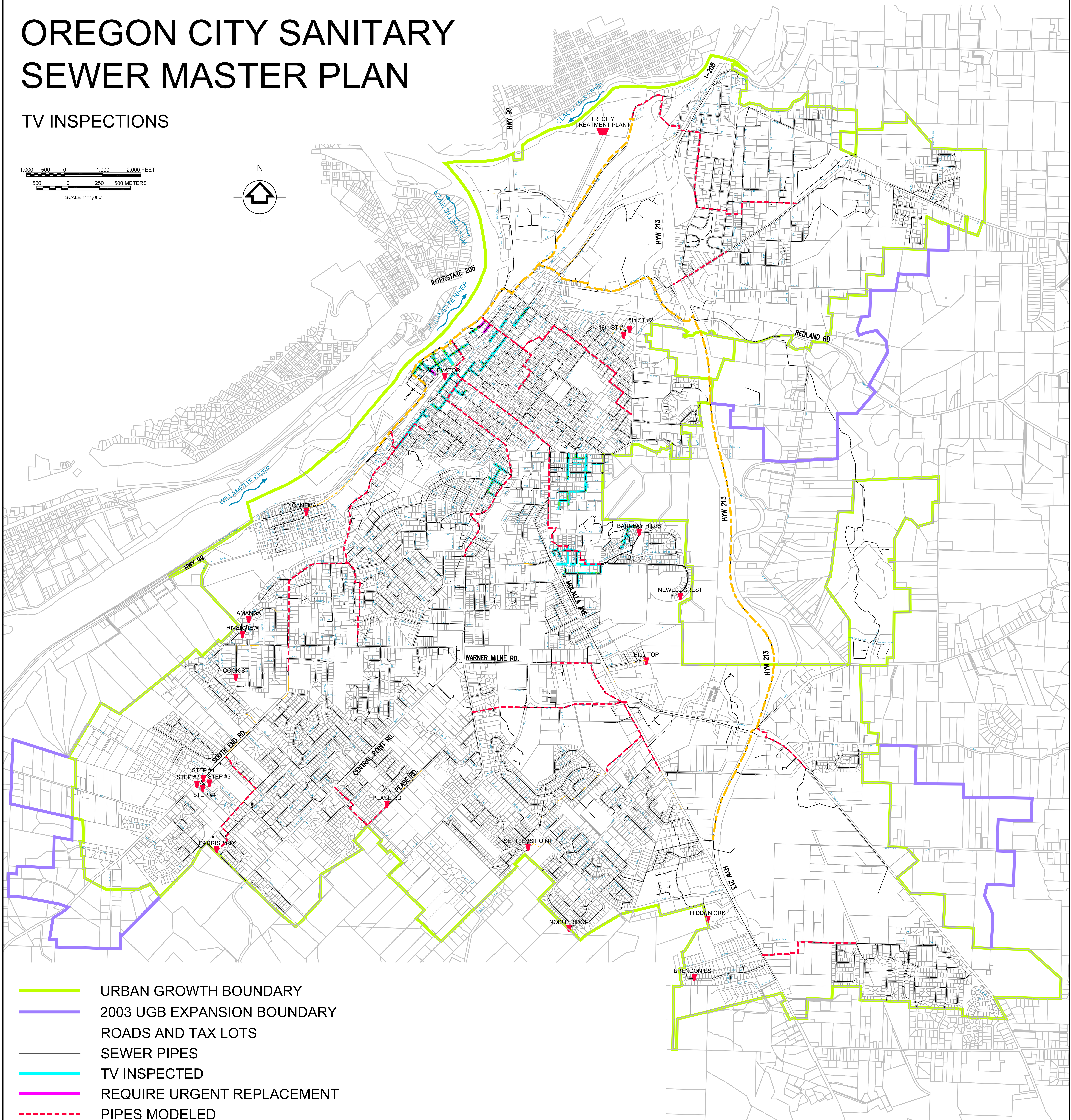


Note:
See Exhibit 1 for location of 2003 UGB expansion areas. (Expansion areas not shown). Expansion areas are county zoning.

EXHIBIT 2

OREGON CITY SANITARY SEWER MASTER PLAN

TV INSPECTIONS



- URBAN GROWTH BOUNDARY
- 2003 UGB EXPANSION BOUNDARY
- ROADS AND TAX LOTS
- SEWER PIPES
- TV INSPECTED
- REQUIRE URGENT REPLACEMENT
- PIPES MODELED
- TRI CITY INTERCEPTORS
- PROBLEMS IDENTIFIED

EXHIBIT 3

OREGON CITY SANITARY SEWER MASTER PLAN

OPERATION AND MAINTENANCE PROBLEM AREAS

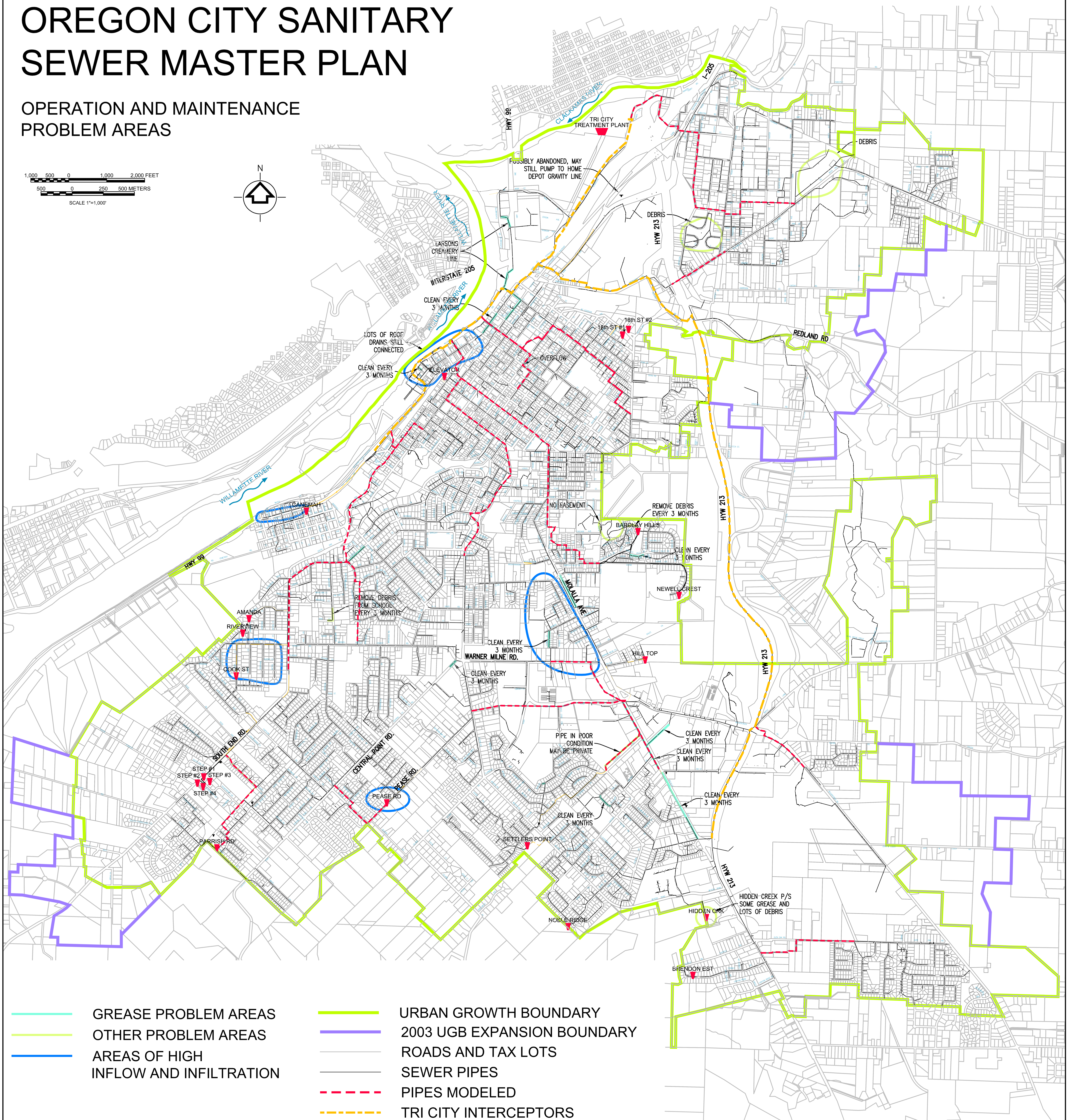
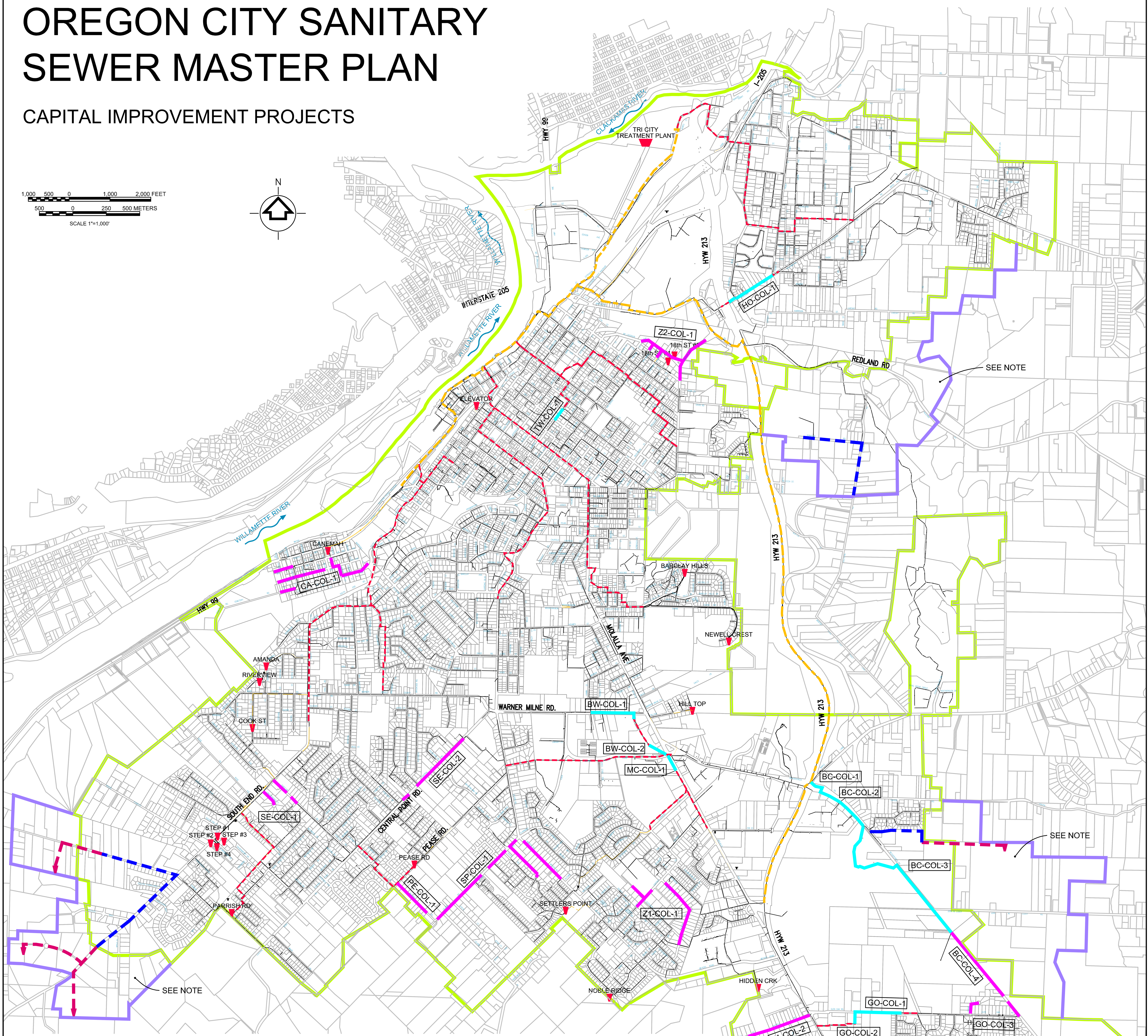


EXHIBIT 4

OREGON CITY SANITARY SEWER MASTER PLAN

CAPITAL IMPROVEMENT PROJECTS



CAPACITY INCREASE PROJECTS
SEWER EXTENSION PROJECTS
BC-COL-1 PROJECT DESIGNATION
FUTURE PUMP STATIONS AND FORCE MAIN (CONCEPTUAL)
FUTURE GRAVITY MAINS (CONCEPTUAL)
URBAN GROWTH BOUNDARY
2003 UGB EXPANSION BOUNDARY
ROADS AND TAX LOTS
SEWER PIPES
PIPES MODELED
TRI CITY INTERCEPTORS
PUMP STATIONS

* NOTE:
PUMP STATION LOCATIONS AND SEWER EXTENSION WITHIN UGB EXPANSION AREA TO BE REVIEWED FURTHER DURING DEVELOPMENT CONCEPT PLANNING

EXHIBIT 5