GLADSTONE PLANNING COMMISSION AGENDA
GLADSTONE CITY HALL, 525 PORTLAND AVENUE

Tuesday, September 15, 2015

6:30 P.M. CALL TO ORDER
ROLL CALL
FLAG SALUTE

CONSENT AGENDA

All items listed below are considered to be routine and will be enacted by one motion. There will be no separate discussion of these items unless a commission member or person in the audience requests specific items to be removed from the Consent Agenda for discussion prior to the time the commission votes on the motion to adopt the Consent Agenda.

1. Approval of August 18, 2015 Minutes

REGULAR AGENDA

2. Public Hearing:

   - Z0347-15-CP – Comprehensive Plan Amendments, Water System Master Plan and Storm Water System Master Plan: Public Hearing to consider amending the Gladstone Comprehensive Plan, Public Facilities Section, Water System Plan to replace the existing plan developed in 1980; also under consideration is a Stormwater System Plan. Purpose of this hearing is to consider the proposal, any testimony presented, and to forward a recommendation to City Council for final decision.

BUSINESS FROM THE PLANNING COMMISSION

ADJOURN

September

CONSENT AGENDA

GLADSTONE PLANNING COMMISSION MEETING MINUTES of August 18, 2015

Meeting was called to order at 6:31 PM.

ROLL CALL:
The following City officials answered roll call: Commissioner Michele Kremers, Commissioner Malachi de AElfweald, Commissioner Linda Nease, Commissioner Kevin Johnson, Commissioner Les Poole, and Chairperson Tammy Stempel.

ABSENT:
Commissioner Kirk Stempel

STAFF:
Jolene Morishita, Assistant City Administrator; David Doughman, City Attorney; Clay Glasgow, City Planner; Melissa Jones, Prosecuting Attorney; Jeff Jolley, Interim Police Chief; Linda Beloof, Municipal Judge; and Sean Boyle, Code Enforcement.

Chairperson Tammy Stempel made a few comments regarding the duties of the Planning Commission.

CONSENT AGENDA:

1. There was no discussion. *Commissioner de AElfweald made a motion to approve the minutes from July 21, 2015. Motion was seconded by Commissioner Nease. Motion passed unanimously.*

REGULAR AGENDA:

2. <u>Public Hearing: Draft Ordinance – Amending Title 17 of the Gladstone Municipal Code to Repeal Chapter 17.61 and Adopt a new Chapter 17.61 – Wireless Telecommunication Facility.</u> City Attorney Doughman asked if any commissioners had a conflict of interest in regards to this ordinance. Chairperson Tammy Stempel said that she works for an engineering firm that does work on telecommunication towers as part of their service scope but that's not exclusively what they do. City Attorney Doughman went over the staff report and explained that most of the law governing telecommunications is dictated by Federal law. This ordinance is based on the City of Salem's code. Commissioner de AElfweald said there are more stringent requirements now.

   <u>Public Testimony:</u>
   There was no public testimony.

   Commissioner de AElfweald made a motion to approve the draft ordinance to amend Title 17 of the Gladstone Municipal Code to repeal Chapter 17.61 and adopt a new Chapter 17.61 with no additional changes. The motion was seconded by Commissioner Johnson. There was no further discussion. Motion passed unanimously.

3. <u>Work Session: Discussion of Code Revisions.</u>
Mike Kenny and Dennis McCarty wanted to address the parking situation in the area of Charolais Court and Webster Road. There has been a sudden increase in vehicles parked on Webster Road, which interferes with the line of sight of drivers. Chairperson Stempel said that this was a good time to bring up this issue because there will be professionals coming to assist with a review of the transportation plan soon.

   Municipal Judge Linda Baloot wanted to address the money amounts of forfeitures for infractions in 1.08.090 – Schedule of Forfeitures. She said there is some confusion regarding the amounts and she would like the language to be clearer. She would like to increase some fees/fines. It was agreed to mirror the ORS fines when appropriate and the City fee schedule when appropriate.

   Sean Boyle, Code Enforcement and Interim Police Chief Jeff Jolley went over issues with park regulations/hours and changes they would like to make. Commissioner Poole will be attending the next Parks meeting and will address these issues and get a decision. Mr. Boyle went over some other issues regarding the codes addressing chronic nuisance properties and vacant properties, chickens, drinking in public, trailer/RV parking, designated parking permit areas, bike lanes, sidewalk repair. Interim Police Chief Jolley went over issues with the garage sale code, business licenses, and the parks. There was a discussion regarding having a ticket kiosk for Meldrum Bar Park.

ADJOURN:
Commissioner Nease made a motion to adjourn the meeting. Motion was seconded by Commissioner Kremers. Motion passed unanimously. Meeting adjourned at 7:45 PM.

Minutes approved by the Planning Commission this ________ day of __________________, 2015.

____
Tamara Stempel, Chair

September

REGULAR AGENDA

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Gladstone, OR 97027
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18595 Portland Avenue
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STAFF REPORT

TO: Planning Commission

FROM: Clay Glasgow, Planner

DATE: September 2, 2015

RE: Z0347-15-CP, Water and Stormwater Systems master plans

PROPOSAL

This is a legislative text amendment, the purpose of which is to incorporate into the Comprehensive Plan ("Plan") new Master Plans for water and stormwater systems. The proposed amendments are to the Facilities and Services section of the Comprehensive Plan and are meant to be accomplished by reference in the Plan.

BACKGROUND

The Gladstone Stormwater Master Plan (SMP) documents the methods and results of the storm system capacity evaluation and the stormwater quality/retrofit assessment conducted for the City of Gladstone (City). The SMP identifies and prioritizes capital improvement projects to address system deficiencies and water quality improvements.

The City has historically managed its stormwater collections and conveyance system with limited mapped and survey field information. Prior to this current proposal no stormwater master plan had been developed for the City and no storm water capital improvement program is in place. Management of the system has been conducted on an as-needed basis, typically in response to failing infrastructure.
The objectives of the SMP are as detailed in the Plan – Executive Summary.

The Gladstone Water System Master Plan (WSMP) documents the methods and evaluation of the City's water system according to the master planning requirements established under Oregon Administrative Rules, Chapter 333, Division 61 for Public Water Systems. The City's current water system master plan was developed in 1980. Since then, the City has

8-1

changed water supply sources, experienced population growth and constructed water system improvements. In addition, there have been substantial changes to methodology, software, and asset management techniques.

The City owns and operates their water distribution system. Part of this project involved inventory and mapping of the existing system. The City's primary source of water supply is the North Clackamas County Water Commission (NCCWC). Water is purchased from the NCCWC to meet current water demand. As needed, and to provide emergency supply water the City supplements their primary water source with additional water purchased from the Oak Lodge Water District. Gladstone's distribution system consists of three storage tanks, two pump stations, and a network of transmission mains and distribution piping that serve the City's three pressure zones.
See WSMP for further detail on system and goals.

ANALYSIS AND FINDINGS

1. The proposed text amendments are legislative. Chapter 17.68 of the ZDO establishes procedural requirements for legislative amendments, which have been or are being followed in this case. However, the ZDO contains no specific review criteria that must be applied when considering an amendment to the text of the ZDO or the Plan. The Planning Commission's role is to arrive at a recommendation to forward to the City Council. Council will make final decision.

2. Chapter 10 of the Comprehensive Plan, Plan Evaluation and Update, contains procedural standards for plan amendments, and requires the Plan and ZDO be consistent with changing conditions. The process followed for Z0347-15-CP is compliant with these standards. Specifically, notice was mailed to the Department of Land Conservation and Development and Metro. Advertised public hearings are scheduled before the Planning Commission and the City Council to consider the proposed amendments. The Statewide Planning Goals and Guidelines and Metro's Urban Growth Management Functional Plan are addressed below.

3. Statewide Planning Goals and Guidelines

    a. Goal 1. Citizen Involvement. The text amendment does not propose to change the structure of the county's citizen involvement program. Notice of the Planning Commission and City Council hearings was published in the newspaper as required by code.

    b. Goal 2. Land Use Planning. Not applicable because the text amendment does not propose to change the City's land use planning process. The City will continue to

2-2

have a comprehensive land use plan and implementing regulations that are consistent with the plan. No exceptions from the Goals are required.

c. Goal 3. Agricultural Lands. Not applicable.

d. Goal 4. Forest Lands. Not applicable..

e. Goal 5. Open Spaces, Scenic and Historic Areas, and Natural Resources. Not applicable because the text amendment does not propose to change the City's Plan or implementing regulations regarding open space, scenic and historic areas or natural resources.

f. Goal 6. Air, Water and Land Resources Quality. This is dealt with through Title 3 – previously acknowledged for the City.

g. Goal 7. Areas Subject to Natural Disasters and Hazards. Not applicable because the text amendment does not propose to change the City's Plan or implementing regulations regarding natural disasters and hazards.

h. Goal 8. Recreational Needs. Not applicable because the text amendment does not propose to change the City's Plan or implementing regulations regarding recreational needs.

i. Goal 9. Economy of the State. Not applicable because the text amendment does not propose to change the City's Plan or implementing regulations regarding the economy of the state.

j. Goal 10. Housing. Not applicable because the text amendment does not propose to change the City's Plan or implementing regulations regarding housing.

k. Goal 11. Public Facilities and Services. This proposal deals specifically with the efficient delivery of public facilities and services, and approval will result in further compliance with Goal 11.

l. Goal 12. Transportation. Not applicable because the text amendment does not propose to change the City's Plan or implementing regulations regarding transportation.

m. Goal 13. Energy Conservation. Not applicable because the text amendment does not propose to change the City's Plan or implementing regulations regarding energy conservation.

n. Goal 14. Urbanization. Not applicable because the text amendment does not propose to change the City's Plan or implementing regulations regarding urbanization.

o. Goal 15. Willamette River Greenway. Not applicable because the text amendment does not propose to change the City's Plan or implementing regulations regarding the Willamette River Greenway.

The Department of Land Conservation and Development (DLCD) was notified of this proposal. No response has been received from that Agency.

4. Metro Urban Growth Management Functional Plan

a. Title 1. Requirements for Housing and Employment Accommodation. Not applicable because the proposed text amendment would not decrease the amount of land zoned for residential or commercial/industrial use, affect design type boundaries, alter permitted densities or prohibit accessory dwelling units.

b. Title 2. Regional Parking Policy. Not applicable because the proposed text amendment would not change the City's Plan or implementing regulations regarding parking.

c. Title 3. Water Quality and Flood Management. The proposal is consistent with Title 3 because it does not interfere with previous adoption of Metro's WQRA map and regulations substantially consistent with Metro's Title 3 model ordinance for the implementation of the water quality provisions of Title 3. Revision of the City's flood management regulations is not proposed.

d. Title 4. Industrial and Other Employment Areas. Not applicable because the proposed text amendment would not change the City's Plan or implementing regulations concerning designation of industrial and other employment areas, minimum lot sizes in these areas, or permitted uses in these areas.

e. Title 5. Neighbor Cities and Rural Reserves. Not applicable because the proposed text amendment would not change the City's Plan or implementing regulations concerning neighboring cities.

f. Title 6. Central City, Regional Centers, Town Centers and Station Communities. Not applicable.

g. Title 7. Housing Choice. Not applicable because the proposed text amendment would not change the City's Plan or implementing regulations concerning housing choice.

2-4

h. Title 8. Compliance Procedures. Not applicable. This Title is administrative and relates to Metro's process for ensuring local governments comply with the Functional Plan.

i. Title 9. Performance Measures. Not applicable. This Title is administrative and relates to requirements for measuring whether the Functional Plan is achieving the intended outcomes in the region.

j. Title 10. Functional Plan Definitions. Not applicable. This Title contains definitions only.

l. Title 11. Planning for New Urban Areas. Though this proposed text amendment does not change the City's Plan or implementing regulations concerning planning for new urban areas – it will provide information applicable to future efforts in this area.

m. Title 12. Protection of Residential Neighborhoods. Not applicable because the proposed text amendment would not change the City's Plan or implementing regulations concerning residential density, designation of neighborhood centers or access to parks and schools.

n. Title 13. Nature in Neighborhoods. Not applicable because the proposal does not alter implementation of Title 13.

Metro was notified of this proposal.

RECOMMENDATION

Staff is of the opinion the proposed Gladstone Water System Master Plan and proposed Gladstone Stormwater System Master Plan are substantially consistent with local, regional, State and Federal requirements regarding water and stormwater systems, and are necessary to effectively manage these components of municipal infrastructure. The Plans in their entirety should be included in the City's Comprehensive Plan by reference.

*plans sent separately via e-mail

2-5

FINAL

Gladstone Water System Master Plan

Prepared for

City of Gladstone, Oregon

November 2014

Brown AND Caldwell

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Portland, OR 97239
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Table of Contents

List of Appendices

List of Figures

*An * indicates figure immediately follows page listed.*

List of Tables

List of Abbreviations

AC	asbestos cement pipe
ADD	average day demand
AWWA	American Waterworks Association
BC	Brown and Caldwell
BPS	booster pump station
CMU	concrete masonry unit
EPS	extended period simulation
FCV	flow control valve
gpd	gallons per day
gpm	gallons per minute
GIS	geographic information system
HGL	hydraulic grade line
IFC	International Fire Code
LIDAR	Light Detection and Ranging
MDD	maximum day demand
MG	million gallons
MGD	million gallons per day
MMD	minimum month demand
NAD	North American Datum
NAVD	North American Vertical Datum
NCCWC	North Clackamas County Water Commission
OLWD	Oak Lodge Water District
PHD	peak hour demand
PRV	pressure reducing valve
PVC	polyvinyl chloride
psi	pounds per square inch

Executive Summary

Background/Introduction

This Water System Master Plan (WSMP) documents the methods and evaluation of the City of Gladstone's (City's) water system according to the master planning requirements established under Oregon Administrative Rules, Chapter 333, Division 61 for Public Water Systems.

The City's former water system master plan was developed in 1980. Since 1980, the City has transitioned water supply sources, experienced population growth and constructed water system improvements. There have also been substantial changes to available software for system mapping, modeling, asset management, and billing. This WSMP documents the current state of the City's water system by identifying current and future system deficiencies, identifying capital projects to address system deficiencies, prioritizing proposed capital improvement projects (CIPs) for implementation, and providing information necessary to perform a financial evaluation of the City's water utility rate in order to fund needed improvements.

Study Area Characteristics

The City owns and operates their water distribution system, which serves residential, commercial, and industrial customers within the Gladstone city limits. The current water system was inventoried and mapped by Sisul Engineering as the first phase of this project.

The City's primary source of water supply is the North Clackamas County Water Commission (NCCWC). Water is purchased from NCCWC to meet current water demand. As needed, and to provide emergency supply water, the City supplements their primary water source with additional water purchased from Oak Lodge Water District (OLWD). Gladstone's distribution system consists of three storage tanks, two pump stations, and a network of transmission mains and distribution piping that serve the City's three pressure zones.

The City's primary source of water is provided by a 27-inch main from NCCWC that enters the City's distribution system on Cason Road. Three interties with OLWD are in place to supplement the water from NCCWC. The high-pressure zone intertie is located at the intersection of Valley View Drive and Valley View Road. The intermediate-pressure zone intertie is located at the intersection of Oatfield Road and Caldwell Road. The low-pressure zone intertie is located in Rinearson Road, approximately 500 feet west of River Road. The low-pressure zone intertie in Rinearson Road is not currently used, but is in place for emergencies. The City does not own any water rights at this time, but is entitled to a minimum allocation of 2.5 million gallons per day (mgd) of treated water from the NCCWC per intergovernmental agreement and as a condition of signing over their water rights on the Clackamas River.

Regulatory Requirements

Regulation of drinking water quality by the U.S. Environmental Protection Agency (USEPA) is authorized by the Safe Drinking Water Act of 1974 and its amendments. Regulations were established to protect public health by setting national health-based standards to protect drinking water quality. The USEPA oversees the federal, state, and local water suppliers who implement the national standards.

Oregon implements drinking water protection through a partnership of the Oregon Department of Environmental Quality (DEQ) and the Oregon Health Authority (OHA). The Oregon Drinking Water Protection Program regulates drinking water quality on a statewide level by adopting the federal standards for water quality monitoring and also implementing standards for construction, cross-connection control, and system operations and maintenance (O&M).

Water supplied to the City is sourced from the Clackamas River and treated at the NCCWC treatment plant. As the City's source water provider, NCCWC provides sampling and monitoring to comply with drinking water regulations. The City is responsible for monitoring water quality parameters within its distribution system.

Study Methods

A hydraulic computer model of the City's water distribution system was developed and used as a tool for evaluating the existing water distribution system and proposed improvements to the system under estimated future water demands. The model was created using Innovyze's InfoWater v11.0 and ArcGIS v10.1.

Several water demand scenarios were created to simulate system performance (existing water use demands, future water use demands, fire flow demands) and operational settings. Evaluation criteria were then used to evaluate the water system's response to the water demands and operational settings in order to develop recommended improvements for the City. The evaluation criteria were related to system pressure for transmission and fire flow, system condition, and system capacity. The evaluation criteria were defined to provide the desired level of service to customers and to maximize the efficiency of the system.

Study Results

Results of the water system modeling effort were used to identify areas in the water system that did not meet the evaluation criteria. Such areas were identified as deficiencies to be addressed through improvements. Additionally, some deficiencies were identified by City staff during field work and based on ongoing maintenance needs. These issues included the upgrades at the Webster Pump Station and replacement of specific PRVs due to their need for repair or due to lack of access.

Collectively, the following problems/issues for correction were identified to be addressed through capital improvement projects:

- Decommissioning of the Ranney intake system.
- Fire flow deficiencies estimated at 49 locations (due to undersized pipes and lack of system looping).
- Pipe age and condition of 17 miles of asbestos concrete (AC) pipe.
- Operating pressures that exceed allowable pressures in pipes located at Meldrum Bar Park Road and at the end of Hardway Court.
- Locations and configurations of PRVs that limit the City's ability to test and maintain them.
- Unreliability of the backup propane pump at the Webster Pump Station.
- Data collection upgrades at the Webster Pump Station.
- Leaky service connections (specifically one identified between the high- and intermediate-pressure zones at Collins Crest).
- Additional storage capacity of 2.0 MG to meet selected emergency storage criteria.

These identified problems/issues resulted in the selection of 19 CIPs for implementation. Table ES-1 summarizes the selected CIPs and estimated costs. Figure ES-1 provides the general location of each of these CIPs. The last line in Table ES-1 should be included in the CIP as an annual line item with costs distributed over 30 years. An annual cost of $820,000 is recommended for AC pipe replacement in order to complete the $24.6 million worth of replacement over a 30-year implementation period.

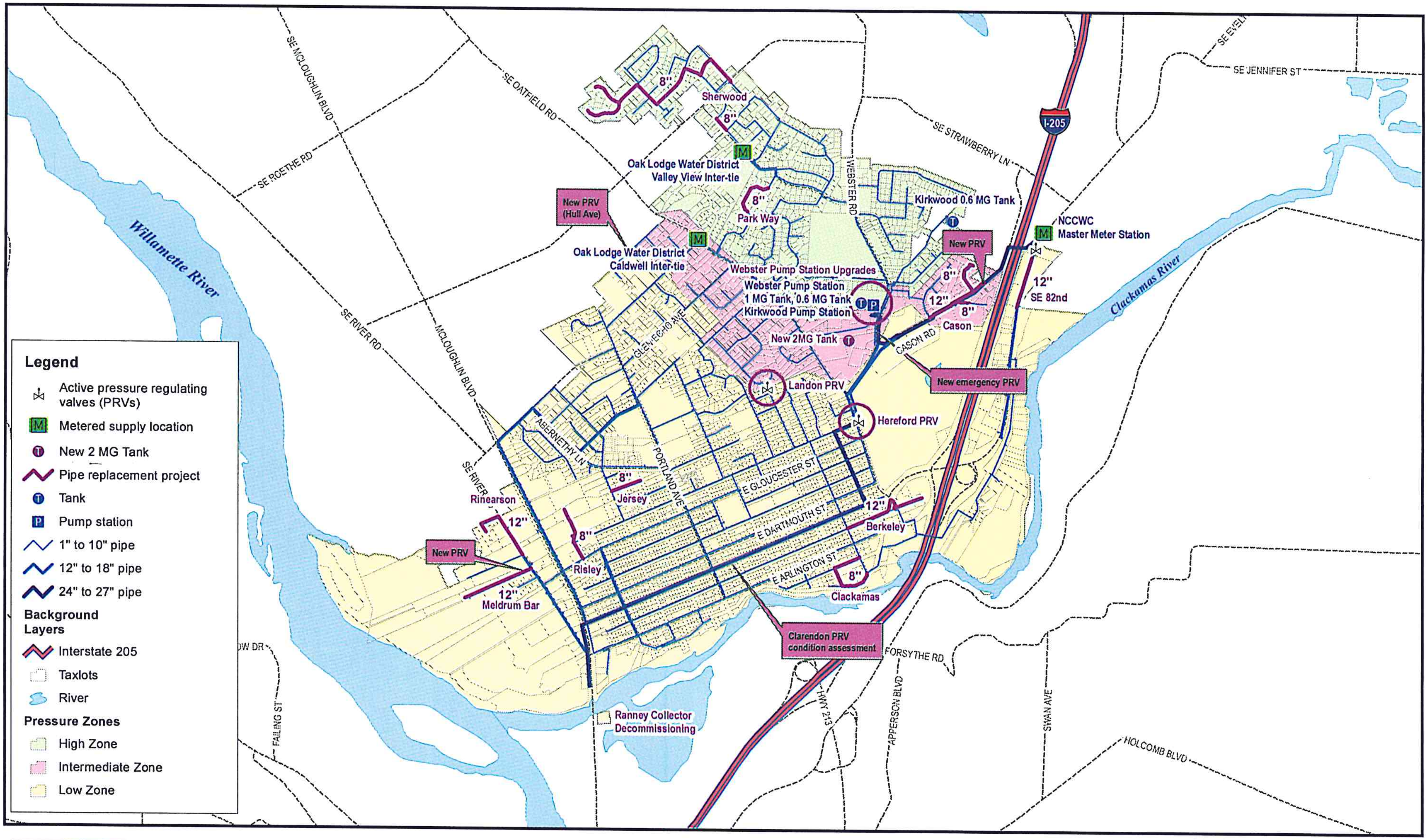
Table ES-1. CIP Estimated Cost Summary

CIP name	Total cost ($)
Supply	
Ranney Intake System Decommissioning	50,000
Piping	
Berkeley Street Pipe Replacement	960,000
Cason Road PRV and Pipe Replacement	1,260,000
Clackamas Boulevard Pipe Replacement	840,000
Clarendon PRV Condition Assessment	10,000
Hereford PRV	110,000
Hull Avenue PRV	110,000
Jersey Street Pipe Replacement	330,000
Landon PRV	110,000
Meldrum Bar Park Road PRV and Pipe Replacement	680,000
Park Way Pipe Replacement	510,000
Sherwood Neighborhood Pipe Replacement	2,170,000
Rinearson Road Pipe Replacement	590,000
Risley Avenue Pipe Replacement	460,000
SE 82nd Drive Pipe Replacement	470,000
AC Pipe Replacement[a]	$24,600,000
Pump Station	
Webster Pump Station Upgrades (Generator Set)	150,000
Webster Pump Station SCADA System	20,000
Storage	
New 2 MG Storage Tank	4,500,000
Total	$37,930,000

[a] *Recommended as an annual line item of $820,000 in the CIP. A leak detection survey is recommended prior to pipe replacement to prioritize the location of replacements.*

In addition to recommended CIPs, recommendations related to system operation and maintenance are also provided and include the following:

- **Leak Detection Survey**: While AC pipe replacement is highly recommended, prior to initiating replacement efforts, a leak detection survey is recommended to assist in prioritizing replacement. A lump sum of $75,000 has been estimated to conduct a leak detection survey prior to AC pipe replacement efforts.
- **SCADA System Maintenance**: An additional capital maintenance item includes annual maintenance of the SCADA system proposed as a CIP above. This is estimated to be approximately $2,500 per year.
- **Preventative Maintenance Program**: Preventative maintenance is essential to optimizing functionality and performance of a water system. The City currently does not have a documented O&M program, or current staffing to conduct preventative maintenance efforts at the recommended frequency. Implementation of the WSMP and CIPs is dependent upon the addition of staff to conduct/oversee preventative maintenance efforts. The addition of two full time staff is recommended in support of a preventative water system maintenance program.



Brown AND Caldwell

September 2014

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CITY OF GLADSTONE
WATER MASTER PLAN

FUTURE SYSTEM LAYOUT

FIGURE ES-1

Section 1

Introduction

This report documents the Water System Master Plan (WSMP) for the City of Gladstone, Oregon. This project was conducted in parallel with the development of a stormwater master plan and associated system-wide survey and mapping. The stormwater master plan is documented separately from this effort.

1.1 Statement of Purpose

The purpose of this WSMP is to comprehensively document and evaluate the City's water system according to the master planning requirements established under Oregon Administrative Rules, Chapter 333, Division 61 for Public Water Systems.

The City's former water system master plan was developed in 1980 by Robert C. Bitten Consulting Engineers. Since 1980, the City has transitioned water supply sources, experienced population growth and constructed water system improvements. Also, there have been substantial changes to available software for system mapping, modeling, asset management, and billing. This WSMP aims to document the current state of the City's water system by providing a comprehensive water system map, identifying current and future system deficiencies using the latest water system modeling software, identifying capital projects to address system deficiencies, prioritizing proposed capital improvement projects (CIPs) for implementation, and providing information necessary to perform a financial evaluation to assess the City's rate structure for funding needed improvements.

1.2 Study Activities

This project included creation of a system-wide map, development of a computer model of the City's water system, evaluation of the existing water system for deficiencies, development of projects for upgrading the water system, and preparation of cost estimates for improvements. City staff were consulted to gain a comprehensive understanding of the water system, ensure the accuracy of the information being analyzed, and determine practical and effective improvement alternatives. More detail regarding each of the project tasks is provided in the paragraphs below.

Data Development. Water system data were collected by Brown and Caldwell (BC) from the City to support the model and master plan development. These included historical water supply data from the North Clackamas County Water Commission (NCCWC), historical water supply data from the Oak Lodge Water District (OLWD), 2013 City billing records, and input from the fire department on required fire flows.

Facility/System Inventory. Sisul Engineering completed an electronic system-wide map in AutoCAD and geographic information system (GIS) of the City's water distribution system, which was previously recorded only on hard-copy maps. Sisul Engineering staff conducted all project surveying work and system mapping in AutoCAD, and worked with the City to horizontally locate water system valves, blow-offs, fire hydrants, water meters, master meters, tanks, pumps, and piping.

System Evaluation. BC developed a hydraulic model, calibrated the model, and evaluated the water system under existing conditions and those expected in the future.

BC prepared a technical memorandum (TM) detailing the methods and assumptions used in the development of the model (Appendix A). The TM provides a reference and background information for the City's future use of the model. The model was created in Innovyze's® InfoWater® software using the data collected earlier. The existing water demands were allocated in the model based on the billing system reference ID of each existing customer. The reference ID is unique to each customer's service address. The model was developed to include scenarios for two demand conditions: average day demand (ADD) and maximum day demand (MDD). A steady state simulation was used for modeling.

The water distribution system model was calibrated by adjusting model settings so that model results matched observed field data. BC wrote a TM outlining the calibration testing plan (Appendix B). Calibration testing of the distribution system included four hydrant tests. A BC representative assisted City staff during the testing.

An existing and future condition was evaluated in the distribution system model to identify current and future improvements that are required to meet the City's level of service goals. The future system condition represents the anticipated water demands in the year 2035, which satisfies the 20-year planning requirement for master plans. Gladstone is currently close to full buildout and future water demands are expected to be associated with infill.

CIP Development and Cost Estimation. The water distribution system model and interviews with City staff were used to develop a recommended capital improvement program. Costs were estimated for the recommended capital improvements based on the Association for the Advancement of Cost Engineering International criteria (described further in Appendix F). Proposed project improvements were developed to address the City's aging infrastructure, critical infrastructure deficiencies, and regulatory needs.

CIP Prioritization. CIPs from this WSMP were reviewed by City staff and prioritized according to predetermined criteria. A resulting CIP implementation schedule was prepared highlighting the top priority CIPs for construction in a 30-year implementation period.

Rate Evaluation and Assessment. FCS Group developed a water system financial plan with supporting rates based on the CIPs developed and prioritized for implementation. The rate evaluation is documented separately from this WSMP.

Section 2

Existing System

The City owns and operates the water distribution system, which services the residential, commercial, and industrial customers within the Gladstone city limits. The existing system described in this section was inventoried and mapped by Sisul Engineering in 2013 and 2014 as the first phase of the project. The results of the system inventory were documented on a system-wide map in AutoCAD, which was developed using multiple data collection techniques, including site survey, as-built drawing review, and interviews with City staff.

Figure 2-1 shows the water distribution system layout. A detailed system-wide map is included in Appendix C. Figure 2-2 shows a hydraulic schematic of the existing system, which illustrates the relationship between the three system delivery points, the tanks, and the pump stations.

The City's primary source of supply is purchased from NCCWC. On-demand and emergency supply water is purchased on an as-needed basis from OLWD. The distribution system consists of three storage tanks, two pump stations, and a network of transmission mains and distribution piping that serves the three pressure zones shown in Figure 2-1. This section summarizes the existing facilities that were included in the computer model and system evaluation.

2.1 Water Supply

When the 1980 master plan was developed, the City's primary water supply source was the Ranney Collector System, which included an infiltration gallery under the Clackamas River bed near the Highway 99E bridge. The City has since abandoned this source of supply due to regulatory requirements and currently purchases water wholesale from NCCWC and OLWD.

The City's primary source of water is provided by a 27-inch main from NCCWC that enters the City's system on Cason Road. The City was added to the NCCWC on July 18, 2005, with Addendum 1 to the second amended intergovernmental agreement (IGA) for the NCCWC. Other water users supplied by the NCCWC include the Sunrise Water Authority and OLWD. Under this agreement, the City is allocated a minimum of 2.5 million gallons per day (mgd) upon completion of the expansion of the NCCWC treatment plant.

Three interties with OLWD are in place to supplement the water from NCCWC. The high-pressure zone intertie is located at the intersection of Valley View Drive and Valley View Road. The intermediate-pressure zone intertie is located at the intersection of Oatfield Road and Caldwell Road. The low-pressure zone intertie is located in Rinearson Road, approximately 500 feet west of River Road. The IGA that provides the terms of this water supply agreement was last updated in 2007 and states that the purpose of the agreement is to provide emergency water between OLWD and the City and an on-demand supplemental source of water to the City. The low-pressure zone intertie in Rinearson Road is not used currently as an on-demand source of water, but is in place for emergencies.

The IGAs referred to in this section are located in Appendix D.

2.2 Water Rights

Prior to 2005, the City held water rights for 8.9 mgd of Clackamas River water. These rights were transferred to NCCWC as described in Section 3(b) of Addendum 1 to the second amended IGA for the NCCWC.

The City does not own any water rights at this time, but is entitled to a minimum allocation of 2.5 mgd of treated water from the NCCWC.

2.3 Water Quality

Regulation of drinking water quality by the U.S. Environmental Protection Agency (USEPA) is authorized by the Safe Drinking Water Act of 1974 and its amendments that were implemented to protect public health by setting national health-based standards to protect drinking water quality. The USEPA oversees the federal, state, and local water suppliers who implement the national standards.

Oregon implements drinking water protection through a partnership of the Oregon Department of Environmental Quality (DEQ) and the Oregon Health Authority (OHA). The Oregon Drinking Water Protection Program regulates drinking water quality on a statewide level by adopting the federal standards for water quality monitoring and also implementing standards for construction, cross-connection control, and system operations and maintenance (O&M). The rules were most recently adopted on May 8, 2014, and are documented in the OAR, Oregon Heath Authority Public Health Division, Chapter 333, Division 61.

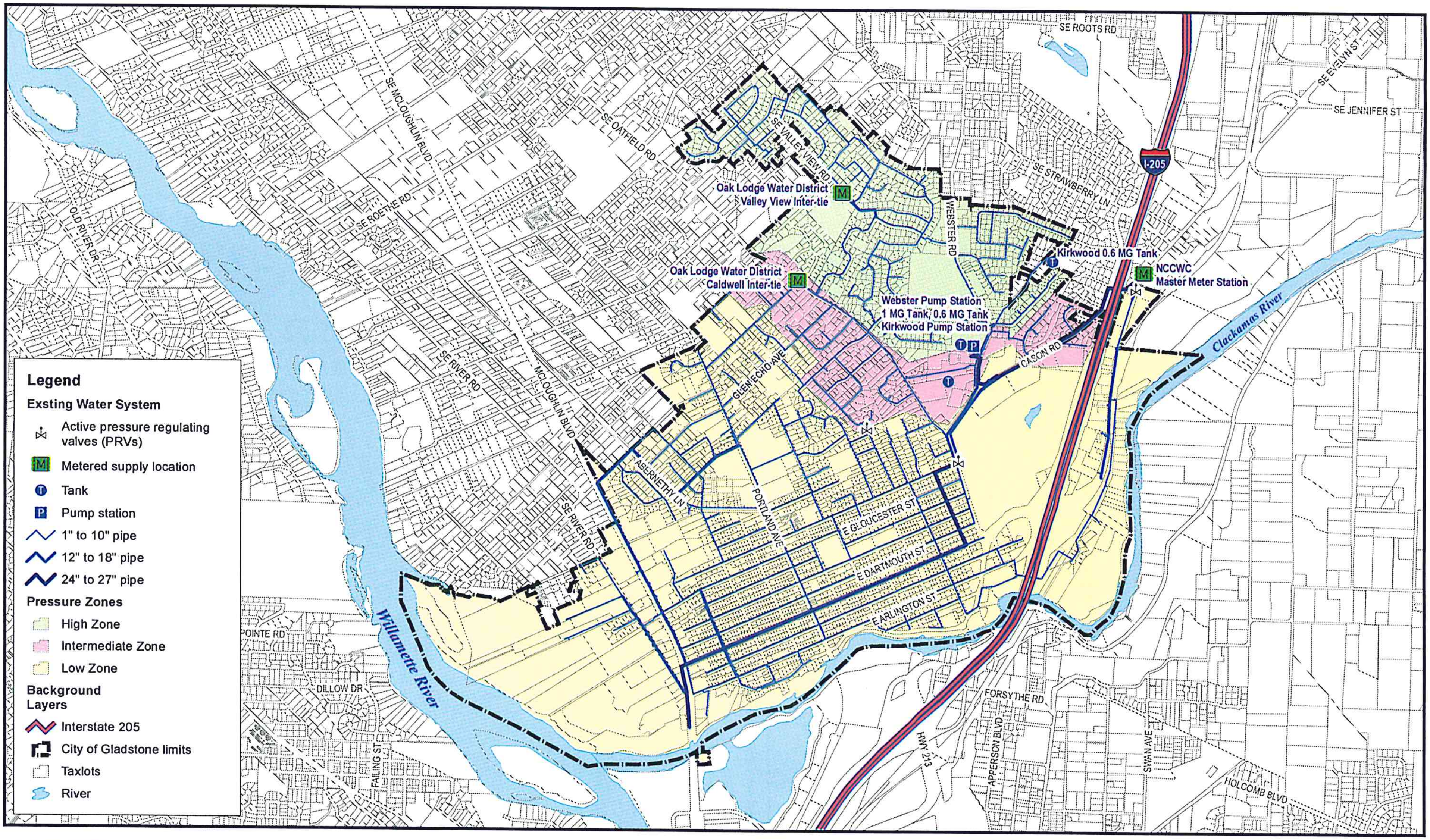
Water supplied to the City is sourced from the Clackamas River and treated at the NCCWC treatment plant. As the City's source water provider, NCCWC provides sampling and monitoring to comply with drinking water regulations. The City is responsible for monitoring water quality parameters within its distribution system that include total coliform, disinfection byproducts, copper, and lead. City staff complete weekly coliform testing and the City contracts with Backflow Management Inc. to complete the remaining testing and document results in annual consumer confidence reports.

2.4 Pressure Zones

The City's water distribution system serves a range in elevations from 35 to 330 feet and is divided into low-, intermediate- and high-pressure zones, as shown in Figure 2-2.

The low-pressure zone hydraulic grade line (HGL) is set by the pressure-reducing valve (PRV) at Oatfield Road and Hereford Road, the PRV at Southeast 82nd Drive and Hanson Court, and by the Webster Tank level. The intermediate zone HGL is set by the Kirkwood tank level and supplemented by the OLWD intertie at Oatfield Road and Caldwell Road when pressures in this zone drop below 40 pounds per square inch (psi). The high-pressure zone HGL is set by the Webster Pump Station and supplemented by the OLWD intertie at Valley View Road when pressures in this zone drop below 45 psi.

The distribution system layout shown in Figure 2-1 shows the three pressure zones described in this section.



Brown AND Caldwell

August, 2014

CITY OF GLADSTONE
WATER MASTER PLAN
EXISTING WATER SYSTEM

FIGURE 2-1

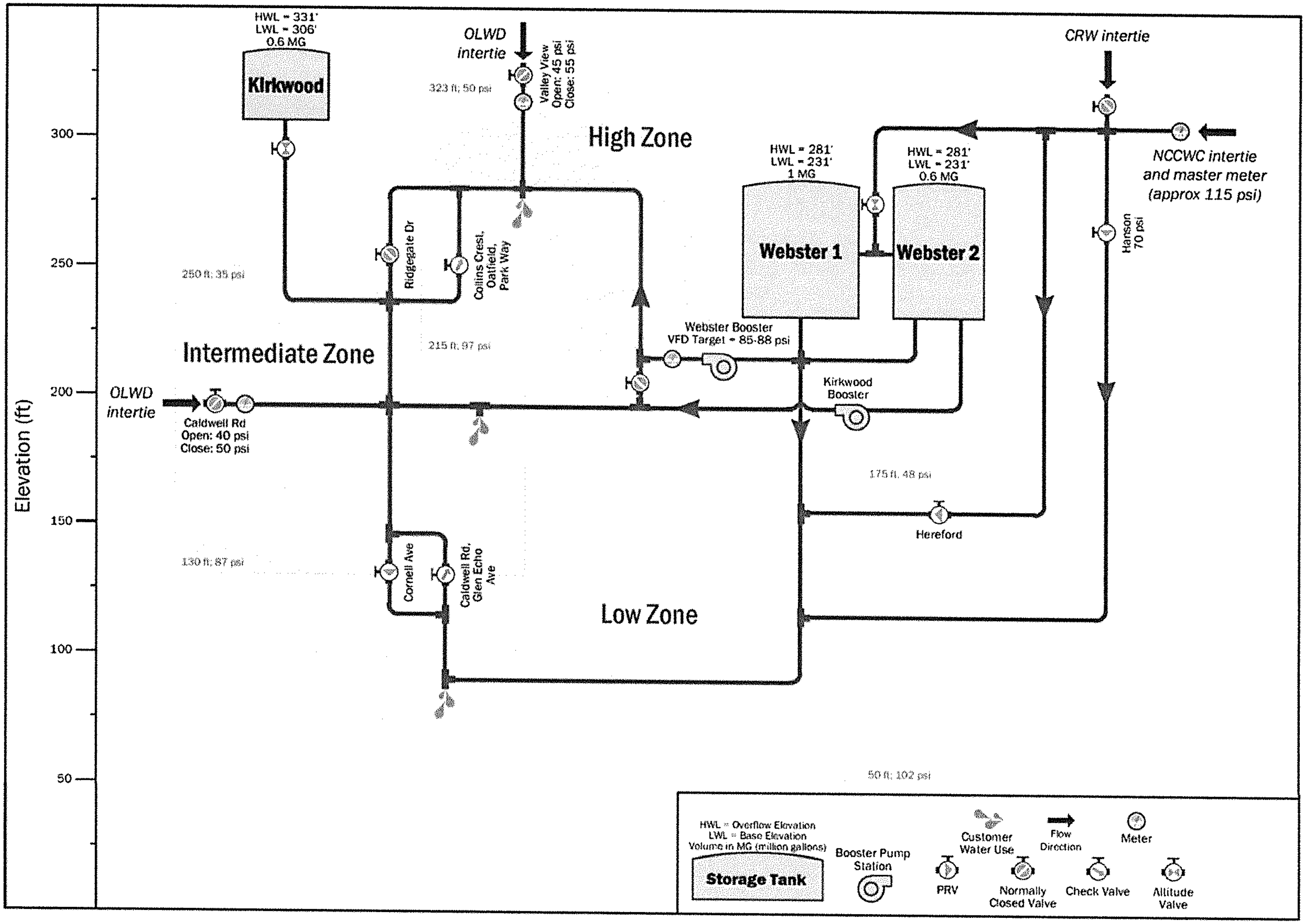


Figure 2-2. Existing system hydraulic schematic

Brown AND Caldwell

2.5 Storage Tanks

The City owns and operates three water storage tanks, which have a total volume of 2.2 million gallons (MG). The Kirkwood tank is a 60-foot-diameter steel cylinder that was rebuilt in 2001 and is located at the end of Kirkwood Street. The Kirkwood tank has a volume of 0.6 MG and supplies the intermediate-pressure zone with water at maximum elevation of 332 feet. There are also two steel cylinder tanks located at the Webster site on Webster Road south of Ridgewood Drive. The 0.6 MG Webster tank was constructed in 1928 and the 1-MG tank was constructed in the 1960s. The tanks at the Webster site serve the low-pressure zone and act as a reservoir for the Webster Pump Station, which serves the high-pressure zone. The tanks are both 50 feet high and are equalized to supply water at the same maximum HGL of 284.5 feet. A summary of the existing water storage is provided in Table 2-1.

Table 2-1. Existing Storage Tank Summary

Tank	Type	Installation date	Base elevation, feet	Overflow height, feet	Diameter, feet	Capacity, gallons
Kirkwood	Welded steel	2001	308	332	60	600,000
Webster 1 MG	Welded steel	1960	234.5	284.5	60	1,000,000
Webster 0.6 MG	Welded steel	1928	234.5	284.5	45	600,000

2.6 Pump Stations

There are two pump stations in the City's distribution system that are located at the Webster site. The Kirkwood pumps were installed in 2003 and are positioned under a small stainless steel cover, which houses the two pumps. The pumps are identical, type C, end suction close-coupled general purpose pumps (Peerless Model C1025A). The two pumps are constant-speed and follow a lead/lag operation that is alternated on a weekly basis.

The Webster Pump Station was originally constructed in the 1960s. It is of concrete masonry unit construction and houses the two Webster pumps, a backup propane pump, the flow meter interface for the meter downstream of the pumps, and circle charts that provide data regarding the Kirkwood tank level, Webster pump discharge pressure, Webster pump flow, and Webster Tank level. The pump motors were rebuilt in 2007 and variable frequency drives (VFDs) were added to the pumps. Following work on the pump motors, one was damaged by falling equipment and is now considerably louder during operation than the other. The pumps follow a lead/lag operation with the first pump matching demand until the downstream system pressure drops below 75 psi, at which time the second pump is turned on.

The City owns a backup generator that is located at the Webster Pump Station, but according to City maintenance staff, the generator is unreliable and in need of replacement.

Existing pump stations are summarized in Table 2-2.

Table 2-2. Existing Pump Station Summary

Pump station	Service level	Installation date	Number of pumps	Manufacturer/ model	Single pump operating point, flow/head	Horsepower	VFD or constant speed
Kirkwood	Intermediate	2003	2 (identical)	Peerless/C1025A	350 gallons per minute (gpm)/65 feet	10	Constant speed
Webster	High	1960s	2 (identical)	Peerless/3AE9	500 gpm/185 feet	40	VFD

2.7 Flow Control Valves

There are eight flow control valves (FCVs) in the City's water system. Details for the FCVs are listed in Table 2-3.

Table 2-3. FCV Summary

Valve	Control type	Setting
OLWD Valley-View meter station	Pressure regulating	Opens at 40 psi Closes at 50 psi
OLWD Caldwell meter station	Pressure regulating	Opens at 40 psi Closes at 50 psi
12-inch Webster Tank inflow	Altitude valve	51 feet
14-inch Webster Tank inflow	Altitude valve	Off
Kirkwood Tank inflow	Altitude valve	22 feet
Hereford pressure regulating valve (PRV)	Pressure regulating	60 psi
Cornell PRV	Pressure regulating	50 psi
Hanson PRV	Pressure regulating	70 psi

There are also four PRVs in the low-pressure zone along Clarendon Street, which were previously used to regulate pressures from the Ranney Collector system to the downtown area. The 24-inch-diameter main that previously supplied water from the Ranney Collector system is still in use, but since the City switched to the NCCWC supply, this main serves the low-pressure zone and does not require PRVs. The physical condition of the four PRVs is unknown.

2.8 Pipe Network

The City's existing distribution system consists of pipes that range in diameter from 1 to 27 inches. The 27-inch-diameter piping serves as the transmission main that conveys flow from the NCCWC master meter to the low-pressure zone and Webster site. The City also has an existing 24-inch transmission main on Clarendon Street that was used to draw water from the Clackamas River via the Ranney Collector Pump Station. The Ranney Collector Pump Station was removed from service in the mid 1980s, when the City began to receive wholesale water. As mentioned above, the 24-inch-diameter main is still in use to serve the low-pressure zone.

The total length of piping in the system is approximately 207,000 linear feet (LF). Table 2-4 lists the length of piping in the water system by pipe material and diameter. These totals do not include service lines or private and OLWD piping within Gladstone.

Table 2-4. Existing Pipe Network Summary												
Pipe material	Length in LF by pipe diameter in inches											
	1	2	4	6	8	10	12	14	18	24	27	Total
Asbestos cement (AC)			5,410	71,120	11,688							88,218
Cast iron (CI)		1,197	4,261	31,600	3,657	261	2,989					43,965
Concrete cylinder pipe (CCP)										1,812	3,898	5,709
C900[a]				755								755
Ductile iron (DI)			1,797	23,875	18,025	677	12,782	21	1,871	6,175		65,224
Poly-vinyl chloride (PVC)	302	2,672										2,975
Total	302	3,869	11,468	127,350	33,370	938	15,771	21	1,871	7,987	3,898	206,846

[a] *Conforms to American Water Works Association (AWWA) Standard C900 for PVC pipe.*

The distribution of the six pipe materials within the system is depicted as percentages in Figure 2-3.

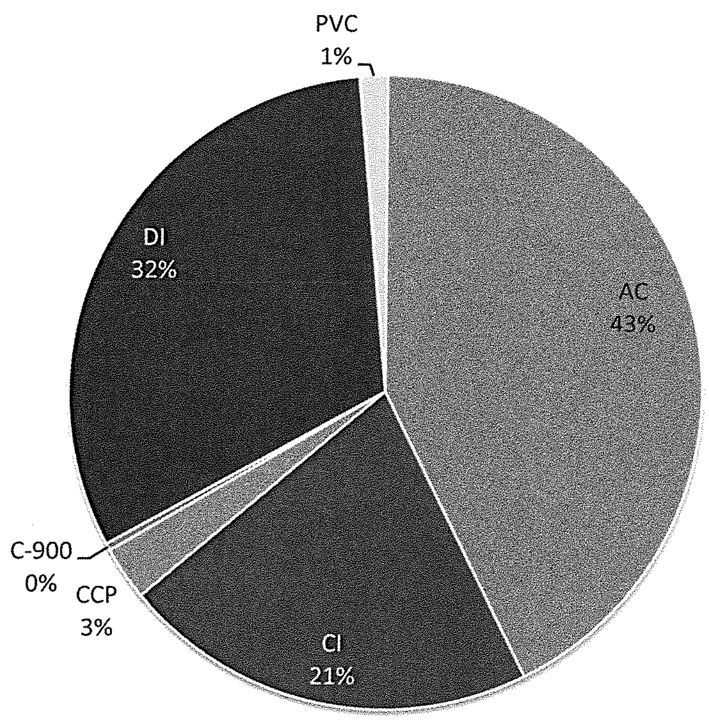


Figure 2-3. Pipe material by percent distribution

As shown in Figure 2-3, AC pipe makes up 43 percent of the City's distribution system. The high- and intermediate-pressure zones contain most of the City's AC pipe, but there are segments of it in the low-pressure zone as well.

AC pipe was a common construction material used for potable water systems beginning in the 1920s and continuing through the 1970s. It was made from a slurry mixture of concrete and chrysotile asbestos fibers and its popularity was due to its low cost, light weight, and ease of installation. Use of AC pipe was widely discontinued in the 1980s due to the asbestos-related health concerns for the workers cutting and installing the pipe.

As the design life of the segments of AC pipe nears, the City is experiencing some main breaks and leakage in areas where AC pipe is installed. A replacement program for AC pipe is discussed in Section 7.

Section 3

Water Demands

This section includes a description of how the water demands were developed for use in evaluating the City's water system and includes a description for existing and future demands. A description of how these demands were allocated in the computer model used for master planning is provided in Section 4.

3.1 Existing System Demands

Existing water system demand scenarios were developed for MDD and ADD. NCCWC provided daily production records that were used in combination with monthly purchase records from OLWD to determine the ADD. Data from 2013 included the most recent full year of data available for this plan. Table 3-1 lists total system demands and resulting scaling factors for the existing water system.

Table 3-1. Total Existing System Demand

Demand condition	Daily demand, mgd	Demand, gpm	Scaling factor from ADD[a]
ADD[b]	1.31	908	1.00
MDD	2.12	1,472	1.62

[a] *Scaling factors are the ratio of the stated demand to the ADD.*

[b] *From 2013 NCCWC daily production records and monthly OLWD purchase records. Daily demands from OLWD were averaged from the monthly records. Outlying data points on January 2 and July 16 and July 17 were removed from the data set.*

The City had a 2010 census population of 11,497 and approximately 3,395 water system billing accounts. The billing accounts were identified using the City's reference ID number, which is unique to each property served and previously corresponded to meter reading routes. Account numbers are unique to each customer and property served, which can result in multiple account numbers for one property; therefore, they were not used. Bi-monthly water demands were calculated for each reference ID for 2013 from the City's water billing database.

Table 3-2 provides a summary of the billed water use, total existing system demand and the difference between the two, which is the percentage of non-revenue water.

Table 3-2. Unaccounted-for Water

Water use	ADD, mgd	MDD, gpm
Billed water use	1.12	Not available
Total existing system demand	1.31	2.12
Percent non-revenue water	16	Not available

To capture the total amount of water distributed throughout the system, individual customer demands for each reference ID were scaled up proportionally to distribute the non-revenue water throughout the system. The customer demands were assigned as model demands to the model junction located nearest to the X and Y coordinates listed for each water meter mapped in the system inventory.

As listed in Table 3-2, the non-revenue water is approximately 16 percent on average. A more typical number for non-revenue water is in the range of 10 percent. While 16 percent is somewhat high, it is not surprising considering the age of the pipes. Per the 2012 AWWA "Water Loss Control: Water Loss Control Terms Defined" publication, non-revenue water is defined to reflect the total distributed volume of water; this is not reflected in customer billings. Non-revenue water is the sum of the following three types of non-revenue water:

- Unbilled Authorized Consumption – water for firefighting, hydrant flushing, etc.
- Apparent Losses – customer meter inaccuracies, unauthorized consumption and systematic data handling errors
- Real Losses – system leakage and storage tank overflows

The percentage listed in Table 3-2 is provided as a snapshot of non-revenue water at this time. See the International Water Association/AWWA Water Audit Method for water audit methods and performance indicators recommended to determine the efficacy of a program to reduce non-revenue water over time.

3.2 Future System Demands

System demands typically increase over time as a city expands its service area or redevelops land at an increased density within the existing service boundary. This increased demand due to population growth can be offset to some extent through the implementation of water conservation measures.

A population growth rate of 0.3 percent annually over the planning period of 20 years is projected by the Portland State University Population Research Center for Gladstone. The projected 2035 population of Gladstone is approximately 12,308. The modest growth rate is due to the limited amount of privately-held vacant lands and expansion area available for development. The current population is estimated to be 11,636. Most future growth is expected in the form of infill.

Demands were calculated for the future 2035 planning horizon assuming a one-to-one relationship between increased population and increased demand because there is no known estimate for a reduction in demand due to water conservation measures. The ratio between the current population and projected population for 2035 is 1.06. This ratio was applied to the existing demands in gpm and converted to mgd. Results are listed in Table 3-3.

Table 3-3. Future System Demands (2035)

Water use	ADD, mgd	MDD, mgd
Total system demand	1.38	2.24

3.3 Fire Flow Demands

Fire flow demands were used to evaluate the system's capacity to supply adequate water for fire suppression. The Oregon Fire Code (OFC) and Insurance Services Office standards were referenced to assign a planning level fire flow demand to each major land-use category within the city, as documented in Section 5. Table 3-4 lists the assigned fire flow rates that were used for existing and future system evaluations.

Table 3-4. Fire Flow Demand by Land Use/Customer Type

Customer class/land use	Required fire demand[a]
1 - Single family residential	1,000 gpm, 1-hour duration
2 - Multi-family residential[b]	2,500 gpm, 2-hour duration
3 - Commercial/industrial/institutional	3,500 gpm, 3-hour duration

[a]Required fire demand while maintaining 20 psi residual pressure.

[b]The Riverview Place Apartments, The Brookside Village, the Fairview Village, the Rivergreens Apartments, and schools will be assigned a fire flow of 3,500 gpm due to their size.

Section 4

Computer Model Development

A hydraulic computer model of the City's water distribution system was developed to be used as a tool for evaluating the existing system and any proposed improvements to the system under estimated future demands. This section provides a basic description of the model, including the process of developing model scenarios, allocating system demands, and calibrating the model. For a detailed description of the model attributes and methodology used to create the model, refer to the Model Creation Memorandum in Appendix A.

4.1 General Model Description

The data collected during the system inventory phase of this project were converted from AutoCAD to GIS and imported into InfoWater modeling software to create a hydraulic computer model of the water distribution system. The system inventory was supplemented by interviews with City staff and site visits to clarify network connectivity and system operations.

The model was created using Innovyze's InfoWater v11.0 and ArcGIS v10.1. A copy of the model was provided to the City on a DVD. The model consists of an ArcGIS .mxd file (Gladstone_W_Model_Build_v6.mxd) and an .IWDB folder, which contains the model attribute data (GLADSTONE_W_MODEL_BUILD_V6.IWDB). A map of the model is viewable in ArcGIS by opening the .mxd file; however, viewing model attributes and operating the model requires an InfoWater software license.

4.2 Model Scenarios

Several scenarios were created for this project to simulate system performance with different system demands and operational settings. Scenarios were also added to the model to include different facility improvements for future planning purposes. All of the scenarios included in the model were categorized as follows:

- Base: this was not used for evaluation purposes, only to store existing model facility data for the other scenarios.
- Calibration: this was used to simulate the system at the time of each hydrant test and used to calibrate the model to observed field data.
- Existing system (2015): this was a post-calibration scenario used to evaluate the existing system.
- Future system: this was a post-calibration scenario used to include future demands and to evaluate the proposed future improvements for the 20-year planning horizon (2035).

4.3 Model Demand Allocation

The existing and future demands described in Section 3 were allocated in the model as described below.

4.3.1 Existing System Demand Allocation

The existing system demand allocation consisted of distributing the total system demand appropriately in the computer model. The following steps describe how the existing system demands were assigned to the model:

1. Obtained billing data, including the water billing reference ID, for each property and calculated the MDD for each property (described in Section 3).
2. Geocoded (located geographically) each of the customers by matching the property reference ID in the billing data with the reference ID included in the water meter inventory (described in Section 3).
3. Calculated the total demand at each demand junction as the sum of the demand for the customers closest to the pipes connecting to the junction.
4. Scaled up the system demand at each junction to distribute the non-revenue water (described in Section 3) throughout the water system demand junctions.

4.3.2 Future System Demand Allocation

Future system demand allocation involved city-wide application of the projected increase (0.3 percent annually for 20 years) in population over the planning period to the existing system demands in the model. This method was used because the City does not have large growth areas planned that would require expansion of the existing network.

4.3.3 Fire Flow Demand Allocation

Fire flow sets were created for the fire flow evaluation of the existing and future system. Fire flow requirements by land use type are listed in Table 3-4. Each hydrant mapped during the system inventory was assigned a fire flow demand based on the surrounding land use types.

All large multi-family developments were reviewed individually to determine if the building footprint was too large for a fire flow demand of 2,500 gpm per the OFC. The Riverview Place Apartments, the Brookside Village, the Fairview Village and the Rivergreens Apartments were assigned a fire flow of 3,500 gpm.

4.4 Steady-State Model Calibration

The purpose of steady-state calibration is to verify pipe connectivity (how pipes connect to other pipes), pipe roughness factors, and the elevation of facilities (i.e., tanks, pumps, and valves) in the model. To obtain field data for calibration, four hydrant tests were performed on the system and used for the steady-state calibration. A dynamic calibration was not conducted because supervisory control and data acquisition (SCADA) system information was not available to determine the system diurnal curve and continuous system operations. The calibration test plan followed during the hydrant testing is located in Appendix B.

The steady-state calibration scenarios in the model were set up to represent the system on the day of testing. City operations and maintenance staff were staged at the NCCWC master meter, OLWD interties and the Webster Tank and Pump Station during the test to record flows on 1-minute intervals. Flow at OLWD was read by observing the position of the hand sweep valve which indicates a flow of 100 cubic feet for every revolution. Tank levels and pump discharge pressure were obtained from circle charts at the Webster Tank, Kirkwood Tank, and Webster Pump Station.

The NCCWC master meter reads data in increments of 10,000 gallons, which did not provide enough resolution for the amount of flow coming into the system during each test. Demands for each scenario were scaled to match system demands at the time of the test based on the Gladstone daily average production value provided by the NCCWC and readings taken at the OLWD interties during the tests.

Adjustments were made to the model until pressures in the model matched the recorded field data from before and during the hydrant tests. The following sections describe the specific results from the four hydrant tests.

4.4.1 High-Pressure Zone Test – Crownview Drive

The high-pressure zone is supplied by the OLWD intertie at Valley View Road and the Webster Pump Station. Visual readings were taken at the OLWD intertie, the Webster Pump Station, and the hydrants where flow and pressure were recorded on Crownview Drive. A pressure logger was installed at the intersection of Lancaster Drive and Buckingham Drive to record pressure throughout all four tests.

The pressures at the pressure hydrant and pressure logger matched well with the data collected during the hydrant testing (see Table 4-1). Therefore, no adjustments were made to Hazen-Williams roughness coefficient values. The flow balance between OLWD and the Webster Pump Station was adjusted in the model to match observed values.

Table 4-1. Model Calibration Results

Test no.	Pressure zone	Static pressure (before flowing hydrant), psi		Test pressure (while flowing hydrant), psi		Comment
		Field	Difference in model[a]	Field	Difference in model[a]	
1	High	59.5	1.5	53.5	-1.1	
2	Intermediate	58.5	-6.5	35.5	1.5	
3	Low	84.0	0.6	79.0	4.0	Hydrant was not fully open during test
4	High	86.5	1.5	61.5	-4.0	Previously thought to be in the intermediate-pressure zone. Calibration efforts identified an unmapped connection to the high-pressure zone.

[a] *A negative value indicates that model results were less than field measurements.*

4.4.2 Intermediate-Pressure Zone Test – Collins Crest

The intermediate-pressure zone is supplied by the 0.6-MG Kirkwood Tank located at the end of Kirkwood Drive via the Kirkwood Booster Pumps located at the Webster site. The OLWD Caldwell Road intertie supplies an on-demand backup and emergency source for this zone, but was not operating at the time of this test.

The initial results for this test did not provide a good match between modeled and observed pressures. Further investigation of the model connectivity resulted in the following corrections:

- Check valves between the low- and intermediate-pressure zones at Caldwell Road, Glen Echo Road, and Collins Crest Road were labeled as closed valves in the system inventory. A meeting with the City on June 18, 2014 confirmed that these are actually operating single-swing check valves. Flow through the Collins Crest check valve occurred during this test upon revision of the model.
- Following correction of the check valves, the model static pressure prior to flowing the hydrant was lower than the observed value at the pressure hydrant by approximately 15 psi. Due to the hydraulic grade measured at the pressure hydrant prior to the test and the response at the Web-

ster Pump Station during the test, it was determined that there is an unintentional connection that exists to the high-pressure zone. The two locations where this may be occurring are at the singles-swing check valve at Park Way and the closed gate valve at Ridgegate Drive.

Following corrections and verification of the model connectivity, no adjustments to C-factors were made. The leaky connection between the high- and intermediate-pressure zones that was identified in the intermediate zone test at Collins Crest did not result in corrections to the model that will be carried forward to the system evaluation. A proposed project to identify and repair the leaky connection is described in Section 6.

4.4.3 Low-Pressure Zone Test – Gloucester Street

The low-pressure zone is primarily supplied by the NCCWC master meter station via a PRV at Hanson Court and 82nd Drive near the master meter with a setting of 85 psi and a PRV at Hereford Road and Oatfield Road with a setting of 60 psi. There is also a PRV between the intermediate and low zones on Cornell Avenue, north of Landon Street. This PRV is on private property and was not found during the system inventory. A setting of 55 psi was assumed for this PRV.

No adjustments were made to the hydraulic connectivity or Hazen-Williams roughness coefficients as a part of the model calibration. Model results matched observed values within 5 psi.

4.4.4 Intermediate-Pressure Zone Test - Ridgewood

This test was originally intended to be an additional test for the intermediate zone; however initial model results indicated a poor match to observed pressures. Other observations included an observed spike in the Webster Pump Station and a dip in the high-pressure zone pressure logger value at the time of this test. The City confirmed that there could be a connection between the high- and intermediate-pressure zones near Ridgewood Drive and Webster Road that was not found or mapped during the system inventory. However, further investigation into the system's HGL indicated that the homes served along Ridgewood Drive are connected directly to the high-pressure zone, with no open connection to the intermediate zone.

No adjustments to Hazen-Williams roughness coefficients were made. Following the correction of model connectivity, the modeled pressures matched observed pressures within 5 psi.

4.4.5 Steady-State Model Calibration Summary

The steady-state model was calibrated to observed readings during each hydrant test by correcting system connectivity issues described above. The field test data and the steady-state calibration results are summarized in Table 4-1 and Appendix E.

Section 5

Evaluation Criteria

This section describes the criteria that were used for evaluating the existing water system and developing the future improvements for the City. The criteria were developed to provide the desired level of service to each customer and to maximize the efficiency of the future system.

5.1 Reference Documents

The documents listed below were reviewed for the development of these criteria. The criteria listed meet state regulations and are in keeping with industry standards.

1. OAR 333-061-0050 [OAR, 2014] – This document contains the state regulations for transmission, supply, pumping, and storage facilities.
2. Recommended Standards for Water Works [WSC, 2012] – This document, frequently referred to as the Ten State Standards, is produced by the Water Supply Committee of the Great Lakes–Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers (WSC) and is widely accepted in the industry. This document was referenced where criteria were not provided by the OAR.
3. *Water Supply: Determining Distribution System Storage Needs* [AWWA, 2005] – This document was referenced where tank storage criteria were not provided by the documents listed above.
4. OFC 2014 [OFC 2014] – All fire flow criteria were based on this document and should be accepted by the fire department.

The criteria, described in more detail below, include the specific capacity, operations, and reliability requirements for supply, piping, pumping, and storage facilities in the water system.

5.2 Supply Criteria

The City obtains its water through wholesale agreements with NCCWC and OLWD, regional water providers that supply water to several other neighboring communities. Water is delivered to the City's system through master meters stations. The overall supply reliability from the providers was not evaluated for this plan. Table 5-1 lists the criteria that were used to evaluate City supply sources.

Table 5-1. Supply Criteria

Criterion		Value/description	Reference, if applicable
Capacity	Flow rate	Equal to average of MDD	[WSC, 2012]
	Head	Maintain a hydraulic grade sufficient to refill the Webster Tanks	
Reliability	Redundant capacity	Meet capacity requirements with the largest producing pump out of service	[WSC, 2012]
	Power supply	At least two independent power sources or a standby/auxiliary source should be provided (e.g., generator)	

5.3 Pipe Criteria

The piping criteria were used to do the following:

- Identify existing pipes that are inadequately sized
- Determine the appropriate size for future piping improvements
- Identify pipes that should be relocated or extended for reliability purposes

Table 5-2 lists the capacity and reliability criteria for evaluating and designing the water system piping.

Table 5-2. Pipe Criteria

Criterion		Value/description	Reference, if applicable
Diameter	Required size (for mains)	As calculated based on flow demand to satisfy pressure, velocity, and head loss requirements listed below. Should not be smaller than 6 inches.	[WSC, 2012]
System pressures	• Maximum • Minimum working pressure • Minimum under any demand condition (including fire) • Normal working pressure	• 100 psi • 35 psi • 20 psi • 60 to 80 psi	[OAR, 2014] [WSC, 2012]
Velocity (transmission and distribution)[a]	• Maximum for MDD • Maximum (peak hour demand (PHD) or fire flow with MDD for small diameter mains)	• 5 feet per second (fps) • 10 fps	
Maximum headloss for MDD[b]	• Transmission pipe (≥12 inches in diameter) • Distribution pipe (<12 inches in diameter)	• 2 feet/1,000 feet • 6 feet/1,000 feet	
Reliability	Distribution system pipe	Dead ends should be minimized by looping	[OAR, 2012] [WSC, 2012]
Location	Transmission and distribution piping	Water mains should be installed in public streets or other public access ways wherever possible. Existing water lines that are in easements and/or right-of-ways in alley ways or behind houses/buildings will be relocated wherever feasible.	City[c]

[a] *A maximum velocity of 10 fps for fire flow with MDD is specified due to the potential to damage pipes through water hammer and cavitation at velocities of greater than 10 fps. Note that design velocities for new mains will be lower. This criterion will be used to design proposed improvements. It is not intended to serve as an independent justification to replace existing facilities.*

[b] *Used in the design of new mains. AWWA recommends this criterion to avoid high operating costs. The cost of adding piping to meet it may exceed the benefit; therefore, it is provided by way of recommendation rather than requirement.*

[c] *Based on interviews with City staff.*

5.4 Fire Flow Criteria

Fire flow is the water available for fire suppression at fire hydrants within the water distribution system. The fire flow criteria were used to determine the fire demand required at each hydrant during the MDD scenario. These criteria were used to identify hydraulic constraints in the system that result in inadequate fire flows.

The OFC [OFC 2014] was used to identify fire flow criteria for the City's three customer types, which include single-family residential (class code 1), multi-family residential (class code 2) and commercial, industrial or institutional (class code 3).

Single-family residential

The fire flow criteria for single-family residential land use are based on building square footage. Appendix B, B105.2 of the OFC specifies a minimum fire flow of 1,000 gpm for 1 hour for one- and two-family dwellings with a fire-flow calculation area that does not exceed 3,600 square feet. The minimum fire flow for dwellings larger than 3,600 square feet is 1,500 gpm and is specified in OFC Appendix B, Table B 105.2, as modified by applicable occupancy hazards listed in B105.4. The average square footage of buildings in class code 1 within the City is approximately 1,800 square feet. A fire flow capacity of 1,000 gpm was evaluated in the hydraulic model for single-family accounts.

Multi-family residential

The fire flow criteria for multi-family residential land use are based on building square footage and type of construction per OFC Appendix B, Table B 105.2, as modified by applicable occupancy hazards listed in B105.4. Buildings within this category within Gladstone typically have a footprint of less than 8,000 square feet. However, there are four large apartment complexes in Gladstone that exceed this size. A fire flow of 2,500 gpm was assigned to buildings with a footprint of less than 8,000 square feet. The Riverview Place Apartments, The Brookside Village, the Fairview Village, the Rivergreens Apartments, and schools were assigned a higher fire flow of 3,500 gpm due to their size.

Industrial, Commercial and Institutional

A maximum fire demand of 3,500 gpm with a 3-hour duration was used to evaluate the water distribution system within industrial, commercial, and institutional land uses that are associated with class code 3. A maximum fire flow of 3,500 gpm is used by the Insurance Services Office to calculate a community's Public Protection Classification. It is assumed that any development with a fire demand of greater than 3,500 gpm is equipped with onsite fire suppression facilities.

A summary of these demands is listed in Table 5-3.

Table 5-3. Fire Demand by Land Use/Customer Type

Customer class/land use	Required fire demand[a]
1 - Single-family residential	1,000 gpm, 1-hour duration
2 - Multi-family residential[b]	2,500 gpm, 2-hour duration
3 – Commercial/industrial/institutional	3,500 gpm, 3-hour duration

[a] *Required fire demand while maintaining 20 psi residual pressure.*

[b] *The Riverview Place Apartments, the Brookside Village, the Fairview Village and the Rivergreens Apartments were assigned a higher fire flow of 3,500 gpm due to their size.*

5.5 Pump Station Criteria

Two types of pump stations were considered in this study: booster and closed-loop pump stations. Booster pump stations add energy, or head, to maintain a flow rate and/or a hydraulic grade from one pressure zone or water system to another that is served by one or more storage tanks. Closed-loop pump stations pump from one pressure zone to a higher pressure zone that is not served by a storage tank. Closed-loop systems are often less reliable than systems served by a storage tank and should be avoided where possible. Table 5-4 summarizes the evaluation and design criteria for the existing and future pump stations.

Table 5-4. Pump Station Criteria			
Criteria		Value/description	Reference
Minimum capacity	Booster	Average of MDD	[WSC, 2012]
	Closed-loop	MDD plus fire flow demand	
Reliability	Redundancy	Areas served by pumps should have a minimum of two supply pumps	[OAR, 2014] [WSC, 2012]
	Redundant pump sizing	Pumps should be sized to meet the minimum capacity requirement with the largest pump out of service (redundant fire pumps are not necessary)	
	Power supply	At least two independent power sources or a standby/auxiliary source (e.g., generator) should be provided	
	Suction tanks	Wherever possible, booster pumps shall take suction from tanks and reservoirs to avoid the potential for negative pressures on the suction line which can result when the pump suction is directly connected to a distribution main	
Operations	Minimum suction pressure	Pumps that take suction from distribution mains for the purpose of serving areas of higher elevation shall be provided with a low-pressure cutoff switch on the suction side set at no less than 20 psi	[OAR, 2014]
	Control settings	Adequate range shall be provided between high-/low-pressure or tank level settings to prevent excessive cycling of the pump	

5.6 Storage Criteria

A variety of methods are used to calculate the volume of storage required for a service area. A commonly used method states that the volume of required storage consists of three components: equalization, fire, and emergency storage. The following describes each storage component and its respective storage criteria.

5.6.1 Equalization Storage

Equalization storage capacity is used to meet peak demands when the available water supply to the system is exceeded. This criterion is often calculated as the difference between the PHD and the MDD. When actual PHD data are unavailable, an equalization storage criterion of 25 percent of MDD is a generally accepted industry standard. The following storage volumes were used as criteria for the existing and future system scenarios:

- Required equalization criteria for existing demands = 0.53 MG (Based on 25 percent of 2.12 MG)
- Required equalization criteria for future demands = 0.56 MG (Based on 25 percent of 2.24 MG)

5.6.2 Fire Storage

Fire storage capacity is reserved to supply the highest fire demand for the duration of a fire event. The required fire storage volume under these criteria is equal to the 3,500 gpm demand over a 3-hour duration, which equals 630,000 gallons (0.63 MG)

5.6.3 Emergency Storage

Emergency storage capacity is reserved to provide water during events such as power outages, standard maintenance procedures, natural disasters, facility failures, etc. Emergency storage criteria are highly subjective and dependent upon local conditions and possible emergency scenarios. Oregon does not have a standard for determining the volume of emergency storage required, which leaves cities to set the level of service for emergency storage based on the desired level of risk and reliability of system infrastructure. Examples of emergencies that could warrant use of emergency storage in this area include power outages, earthquakes, equipment failures, and pipeline failures.

Two days of ADD is a typical value that is used with some consideration given to the reliability of supply sources. Two average days are equal to 2.62 MG for the existing ADD and 2.76 MG for the future ADD.

5.6.4 Storage Criteria Summary

Table 5-5 summarizes the total volume needed to meet the three required components of storage capacity. In addition to the storage volume criteria, operations criteria include avoiding excessive storage capacity to prevent water quality issues and maintaining adequate control to maintain levels in storage tanks.

Table 5-5. Storage Criteria Summary

Criterion		Existing storage needs, MG	Future storage needs, MG
Capacity	Equalization	0.53	0.56
	Fire	0.63	0.63
	Emergency	2.62	2.76
Total		3.78	3.95

5.7 Operation and Maintenance

The purpose of an O&M program is to ensure satisfactory management of a water system's operation in accordance with the pertinent requirements of OAR 333-061-0065. A comprehensive O&M program includes guidance for water system staff to identify the necessary tasks required to ensure that the system and utility asset life are maximized, utility costs are appropriately managed, and safe and reliable public water supply is maintained. The following sections include a summary of existing staffing and guidelines for O&M activities to include in a documented O&M plan.

5.7.1 Management and Staffing

City staff manages, operates, and maintain the water system as a local government function under City governance. City governance is organized under a City Council, composed of six elected members and an elected Mayor, charged with executing policies set forth by the City Council. The day-to-day management is the responsibility of the Public Works Supervisor, supported by staff members in maintenance, clerical services, and billing.

City Council. The City Council sets policy and water system rate schedules, approves ordinances, and serves as a sounding board for public response, feedback, and guidance. The City Council also approves the water system budgets, sets City-wide priorities, and provides funding and support for water system projects.

City Administrator. The City Administrator works with the Public Works Supervisor and City Council regarding policy development, project issues, and annual water system budget preparation.

Public Works Supervisor. The Public Works Supervisor works directly with City staff to ensure system operations. The Public Works Supervisor oversees the day-to-day work necessary to maintain, operate, test, and analyze the water system to ensure proper function. Additional responsibilities include staff oversight and responding to customer complaints.

Maintenance Staff. There are currently five staff supervised by the Public Works Supervisor. These individuals perform monthly meter reading for use in preparing the monthly water bills. They also work with the Public Works Supervisor to coordinate and schedule necessary repair and maintenance tasks.

The Public Works Supervisor and Maintenance Staff have multiple areas of responsibility and are not dedicated solely to O&M of the water system. Other responsibilities include O&M of the storm system, sanitary system, roads, and parks. Due to these multiple responsibilities, water system maintenance is typically conducted on a reactionary basis, as sufficient staff are not available for preventive maintenance activities.

Operator Certification is prescribed by OAR 333-061-0210 through 333-061-0272, which mandate that public water systems retain in their employment individuals who are certified by examination as competent in water supply operation and management as determined by the OHA. The OHA determines the required level and number of certified positions based on the population and complexity of the water system. For the City, this is a Distribution Class 2 Certification level for the person in Direct Responsible Charge. The Public Works Supervisor maintains this certification.

To promote and maintain expertise as required for operator certification, the Public Works Supervisor attends short schools and workshops to achieve required continuing education units (CEUs).

Besides annual training for the Public Works Supervisor to maintain required CEUs, all water system staff should have training in the following safety-related areas:

- Chlorination O&M
- Trench safety/cave-in protection
- Confined space entry
- Asbestos pipe handling
- Backhoe safety
- Safety awareness
- Cardiopulmonary resuscitation/first aid
- Defensive driving/vehicle inspections
- Flagging/work zone safety

5.7.2 O&M Guidelines

The City does not currently have a documented O&M plan/manual for the water system. Development of a manual is recommended to document expectations regarding activities and frequencies. In the development of an O&M plan, the following activities should be considered and/or included:

Pipe Mains and Valves

- Flush water mains approximately every 3 years with dead-end mains and those with periodic water quality issues as customer complaints arise.
- Exercise critical valves approximately every 6 months.
- Exercise non-critical valves approximately every 3 years or in conjunction with the main flushing program.

Hydrants

Flush hydrants flushing on a routine cycle (e.g., every 5 years).

Tanks

- Operate the water system to achieve complete turnover in each tank every 3 to 5 days, to keep water fresh and maintain disinfection residual.

- Inspect reservoir exteriors routinely.
- Maintain coating as needed (approximately every 20 to 30 years depending on exposure).
- Conduct an interior diving inspection (as recommended by USEPA) at least every 5 years.

Pumps

Record and track pump data such as voltage, pump speed, and pump output to identify any performance changes that would require maintenance.

System Performance Evaluation

Conduct routine/daily system performance evaluation for parameters including the following:

- Normal pump station operations and performance
- Storage tank levels within expected norms
- Customer complaints, if applicable
- General system performance
- Main breaks and other repairs

Comprehensive Monitoring Plan

Water quality sampling requirements are established primarily by federal rule, adopted by the state, and enforced by OHA. The City's sample collection and analysis procedures should be conducted according to department-approved methods as detailed in OAR 333-061-0036 at required frequencies.

Cross-Connection Control Program

Document the City's cross-connection control and backflow prevention program.

Emergency Response Program

Document emergency response planning to identify and prioritize procedures to be used in the event of an emergency. Emergency situations may include major water supply line breaks, fire, weather events, earthquakes, droughts, or damage to key facilities.

Water Use Records

Collect and maintain data to record equipment operational status and key process variables such as tank water levels, pipeline pressures, and pump running speeds.

Additional staff resources will be needed to implement O&M guidelines as recommended above because the City does not have sufficient staff to support a preventive maintenance program. Implementation of a preventive maintenance program will limit future infrastructure and equipment failures and reduce the need for reactive repairs of system components.

In conjunction with the preventive O&M activities, there are additional O&M activities that result from the Capital Maintenance Plan (see Section 7.2) and should be documented in the O&M manual (e.g., AC pipe replacement).

OAR 333-061-0060(5)(a) requires the City to maintain a current Water System Master Plan that is prepared by a professional engineer and approved by the Oregon Health Authority's Drinking Water Program. It is recommended that the CIP be updated annually as projects are completed and that a comprehensive review of the master plan is completed every 10 years or as needed based on changes to land use, system improvements, or system supply.

Section 6

System Evaluation Results

This section summarizes the evaluation of the City's water distribution and storage system based on the criteria provided in Section 5. It includes findings for both the existing and future system evaluation.

6.1 Existing System Evaluation

The existing water system evaluation included an analysis of the City's supply, transmission piping, pumping, and storage facilities. The water system model was used to simulate the demand conditions that represent the greatest strain on the system: a steady-state MDD simulation and a steady-state MDD plus fire flow simulation. Model results were compared to the criteria listed in Section 5. Areas in the existing system that did not meet the criteria were identified as deficiencies that should be addressed.

6.1.1 Supply

The system supply was evaluated based on capacity, quality, and reliability. NCCWC and OLWD are responsible for the quality and reliability of the water supply at the master meter and intertie points. Water quality testing performed by the City, as described in Section 2, indicates that the water supply meets state and federal regulations for quality.

The IGA with NCCWC allocates a minimum of 2.5 mgd to the City, which is greater than the City's existing and future MDD and meets the flow rate supply criteria.

The supplied hydraulic grade should be sufficient to fill the Webster Tanks. Based on circle chart data obtained during hydrant testing, the 50-foot Webster Tanks have a water level of approximately 43.9 feet. Future storage improvements should account for the supplied hydraulic grade from NCCWC and set the maximum tank level(s) accordingly. It should be noted that diurnal readings were not available so fluctuations in the supply system and varying periods to fill the tank relative to peak demand were not evaluated.

6.1.2 Piping

Evaluation of the existing system piping included analysis of standard operating pressures, velocity, head loss and fire flow capacity as described in the following subsections.

6.1.2.1 Operating Pressures

Operating pressures were evaluated during the MDD scenario and found generally to meet the evaluation criteria. There were no areas in the system with customer demands where water pressure was estimated by the model to drop below the minimum allowable pressure of 35 psi.

There are two areas in the system where the water pressure was estimated to exceed the maximum allowable pressure of 100 psi. The first area is located at the end of Meldrum Bar Park Road, where pressures were model-estimated to range between 100 and 105 psi. The second area is located at the end of Hardway Court, where pressures were model-estimated to reach 102 psi.

City staff were interviewed to validate these model results. The City did not report any pressure complaints within the system during normal operations. However, the City does receive low-pressure complaints in some areas during fire flow testing. Projects to address this were not proposed because the pressures are still estimated to meet criteria.

6.1.2.2 Velocity and Head Loss

Model results showed that the existing system is expected to meet the velocity criterion/requirement of less than 5 fps for the MDD and the headloss criterion listed in Table 5-2.

6.1.2.3 Fire Flow Deficiencies

Fire flow was evaluated for each hydrant in the model based on the criteria provided in Section 5.4. Hydrants with residual pressures of lower than 20 psi at the required fire flow are shown as fire flow deficiencies in Figure 6-1. The majority of the deficiencies are due to undersized distribution piping and a lack of looping. Recommendations to eliminate fire flow deficiencies can be found in Section 7.

6.1.2.4 Reliability

The degree of distribution system reliability varies by area within the city. The piping in the low-pressure zone south of Hereford Street has very good system looping that follows the gridded layout of the streets. The street layout in the northern portion of the low-pressure zone and the intermediate- and high-pressure zones provides fewer opportunities for system looping with many dead-end streets and steeper terrain. Specific locations that lack system looping include the following:

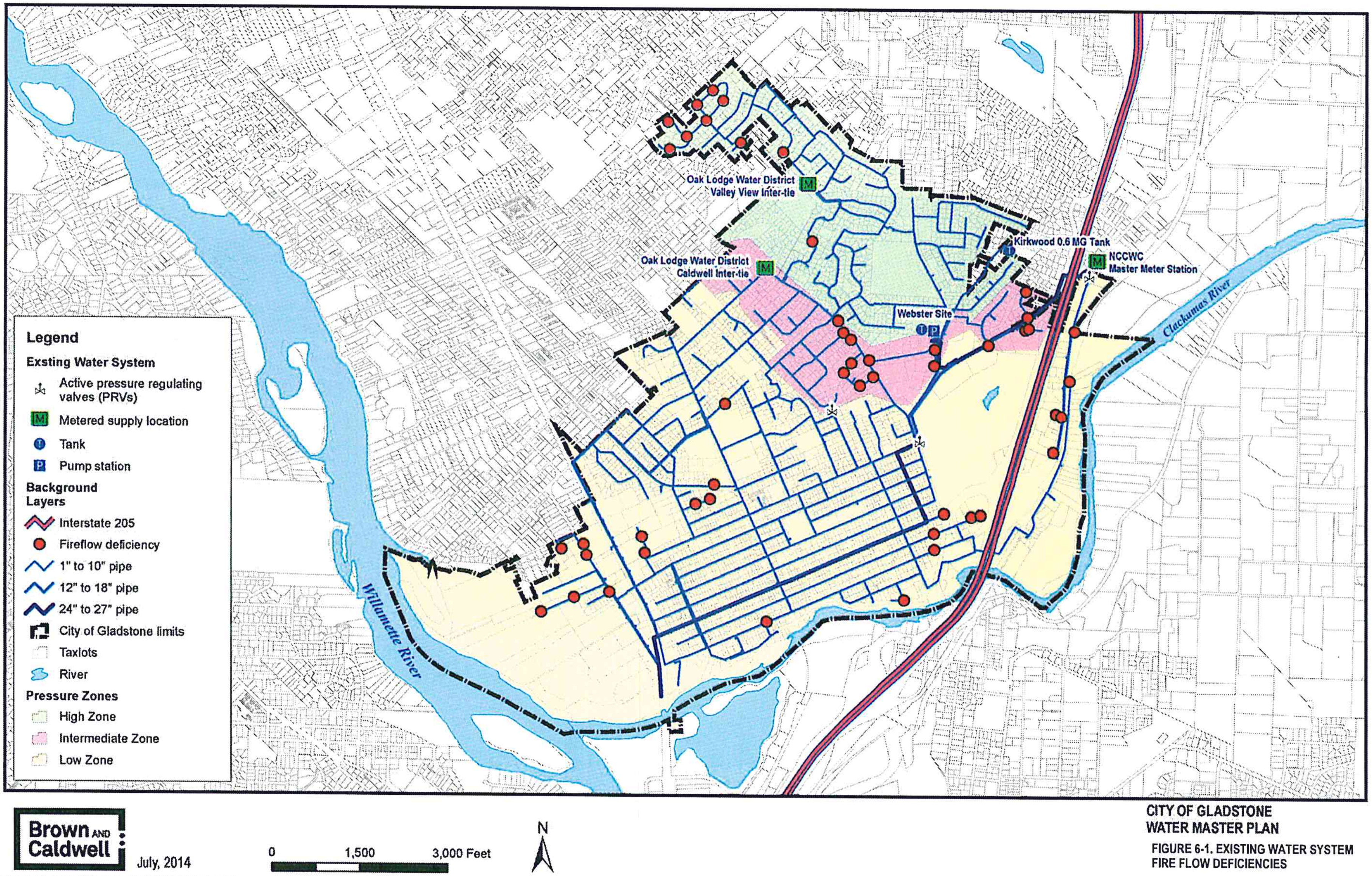
- Customers on Ridge Drive, Cason Circle, and Rivergate School Drive are served by a single 6-inch AC pipeline that is aligned between lots from Ridgewood Drive to Ridge Drive. The access to this pipeline is limited and there are concerns with its condition due to its material type. A project to improve reliability in this area is discussed in Section 7 (see Cason Rd. PRV and Pipe Replacement CIP).
- Customers north of Buckingham Drive in the high-pressure zone are served by a single 8-inch AC pipeline on Lancaster Drive. A project to improve reliability is discussed in Section 7 (see Sherwood Neighborhood Pipe Replacement CIP).

6.1.3 Pump Stations

Two pump stations within the existing system are discussed in this section. The Webster Pump Station boosts pressures from the Webster Tanks to supply the high-pressure zone and the Kirkwood Pump Station boosts pressures from the Webster Tanks to fill the Kirkwood Tank, that serves the intermediate-pressure zone.

There are two identical 500-gpm pumps at the Webster Pump Station. These pumps would be able to serve the existing MDD of 267 gpm in the high-pressure zone with one pump out of service, but do not have the ability to meet the existing MDD with fire flow demand. However, during fire flow events or testing, the OLWD intertie at Valley View opens and supplies the remaining demand. A project would be needed to address pump capacity only if the City does not remain reliant on the intertie. The largest fire flow demand within the high-pressure zone is 3,500 gpm, which is assigned to the fire hydrant serving Kraxburger Middle School. All other fire flow demands within the high-pressure zone were set to 1,000 gpm. There is also a propane pump within the Webster Pump Station that is intended to serve as a backup pump for the high-pressure zone in the case of a power outage. According to City staff, the pump has not been exercised regularly and is not reliable.

The Kirkwood Pump Station has two 350-gpm pumps that are housed in a stainless steel enclosure at the Webster site. These pumps would be able to serve the existing MDD of 125 gpm in the intermediate-pressure zone with one pump out of service. Because the intermediate zone is served



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July, 2014

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FIGURE 6-1. EXISTING WATER SYSTEM FIRE FLOW DEFICIENCIES

by the Kirkwood Tank, the Kirkwood Pump Station does not need to be sized to provide fire demand. If pressures in the intermediate-pressure zone drop to below 40 psi at the OLWD intertie, the backup source would be anticipated to supply the zone until pressures reach 50 psi.

6.1.4 Storage

Available storage capacity was compared to the criteria for equalization, fire, and emergency storage for the system. As indicated in Table 6-1, an additional 1.58 MG is needed to meet storage criteria for the existing system. A project to increase system storage is discussed in Section 7.

Table 6-1. Existing Storage System Analysis

Available storage (Kirkwood and Webster tanks)	Required storage				Storage deficiency
	Equalization	Fire flow (3,500 gpm for 3 hours)	Emergency (2 days of ADD)	Total	
2.2	0.53	0.63	2.62	3.78	1.58

6.2 Future System Analysis

This section presents a summary of the future 2035 system evaluation scenario. This scenario was developed to identify the improvements needed to meet the evaluation criteria presented in Section 5, given a future 2035 population of 12,308. Figure 6-2 shows the layout of the future system at year 2035 and Figure 6-3 shows the hydraulic schematic of the future system.

6.2.1 Supply

The IGA with NCCWC allocates a minimum of 2.5 mgd to Gladstone, which is greater than the City's future MDD and meets the flow rate supply criteria. The supplied hydraulic grade should be sufficient to fill the Webster Tanks.

No improvements to the existing system supply were identified based on demand. Reliability of supply was not evaluated for this plan.

The City's previous Ranney Collector supply system has not been in operation since the mid 1980s, but was never formally decommissioned. Permitting and construction needs associated with decommissioning are unknown at this time, but the City is in the process of coordinating with contractors to aid in this effort. A project is identified in Section 7 to pursue decommissioning of the Ranney Collector system with a $50,000 placeholder given the unknown scope of work.

6.2.2 Piping

The primary drivers for improvements to distribution system piping are undersized pipes, aging AC pipe and a lack of system looping. Undersized pipes and lack of system looping resulted in the fire flow deficiencies described in Section 6.1.2.3. Proposed pipe replacement projects (see Figure 6-2) were developed to provide adequate capacity to meet fire flow requirements. These piping improvements are needed as soon as possible given that the current system does not meet criteria specified in the OFC.

The City owns approximately 17 miles of AC pipe, which make up approximately 43 percent of the distribution system. Main breaks have occurred in areas of AC pipe in recent years and the City is concerned about the condition of the aging pipes. In response to concerns about pipe condition, fire flow testing has been severely limited in the high-pressure zone, which is comprised almost entirely of AC pipe. Much of this pipe was installed behind the curb in common trenches with other utilities, making inspection and maintenance difficult. Figure 6-4 highlights areas of AC pipe within the system. A replacement program is needed and is presented in Section 7. In addition, a condition assessment is recommended to prioritize pipe replacements. The condition assessment would include all pipe types because there may be pipes constructed of other materials that warrant near-term replacement. A condition assessment would ensure replacements are conducted in order of greatest need. A leak detection program could also be conducted to prioritize pipe replacements where leaks currently exist.

6.2.3 Pump Stations

The Webster and Kirkwood Pump Stations are powered from the 480-volt, 200-amp, three-phase electrical service panel in the Webster Pump Station Building. This electrical service does not have a backup emergency power source large enough to operate both the Webster and Kirkwood pumps in the event of a utility power outage. The Webster Pump Station currently has backup power supply from a dated propane engine pump. There is no backup power supply for the Kirkwood pumps.

For reliability, it is recommended that a new diesel electric standby emergency generator be installed at the site to accommodate all normal duty pumps (Webster pumps and Kirkwood pumps) and ancillary electric loads currently supplied from the existing electrical service panel. It is further recommended that the dated propane pump and small ancillary generator be retired from service and removed from the building. Removing the obsolete equipment from the building would create much needed work space and room for new electrical equipment necessary for the operation of the backup emergency generator system. A cost estimate for procurement and installation is presented in Section 7.

Additionally, the Webster and Kirkwood Pump Stations' instrumentation data logging and alarm annunciation are currently conducted with existing chart recorders and an analog alarm dialer connected to the land line phone system. There is no method to retrieve and store the data remotely, and the existing equipment is prone to failure. The alarm autodialer is also prone to sending false alarms and must be reset locally, requiring staff to drive to the site. It is recommended that data logging and alarming functions be upgraded by using a SCADA monitoring service. Due to the size of the pump station and the number of data and alarm points, purchase, installation, and maintenance of a City-owned and operated SCADA system is not warranted. An outside SCADA monitoring and alarm handling service is economical and would improve the reliability of the system operation. A typical system is detailed along with a budget estimate in Section 7.

6.2.4 Storage

Available storage capacity was compared to the criteria for equalization, fire, and emergency storage for the future demand scenario. As indicated in Table 6-2, an additional 1.75 MG of storage is needed to meet criteria for the existing system.

Table 6-2. Future Storage System Analysis

Available storage (Kirkwood and Webster tanks)	Required storage, MG				Storage deficiency, MG
	Equalization	Fire flow (3,500 gpm for 3 hours)	Emergency (2 days of ADD)	Total	
2.2	0.56	0.63	2.76	3.95	1.75

To address the storage deficiency, a new 2.0-MG tank south of the existing Webster site is proposed. The 2.0-MG volume was determined by using the storage deficiency plus an additional 0.25 MG to account for unused storage space in the Webster Tanks, as they are not being operated full. The City owns 11.7 acres of vacant land between Oatfield Road and Webster Road, which was identified previously as a potential storage location. The proposed new tank would provide equalization, fire flow, and emergency storage for the City's system. The tank would supply the low-pressure zone by gravity and would maintain the same hydraulic grade as the Webster Tanks to supply the intermediate- and high-pressure zones by the Kirkwood and Webster Pump Stations, respectively. This additional storage is shown to be needed for emergency storage. As discussed in Section 5, emergency storage criteria are highly subjective and dependent upon local conditions and possible emergency scenarios. Two days of ADD was the value chosen for emergency storage in this evaluation.

6.3 Field Identified Operational Problems

Some water system problems were identified by staff during completion of field work and ongoing maintenance. These issues included necessary upgrades at the Webster Pump Station and replacement of specific PRVs. The Webster Pump Station upgrades are addressed in section 6.2.3. The PRVs that require replacement include the following:

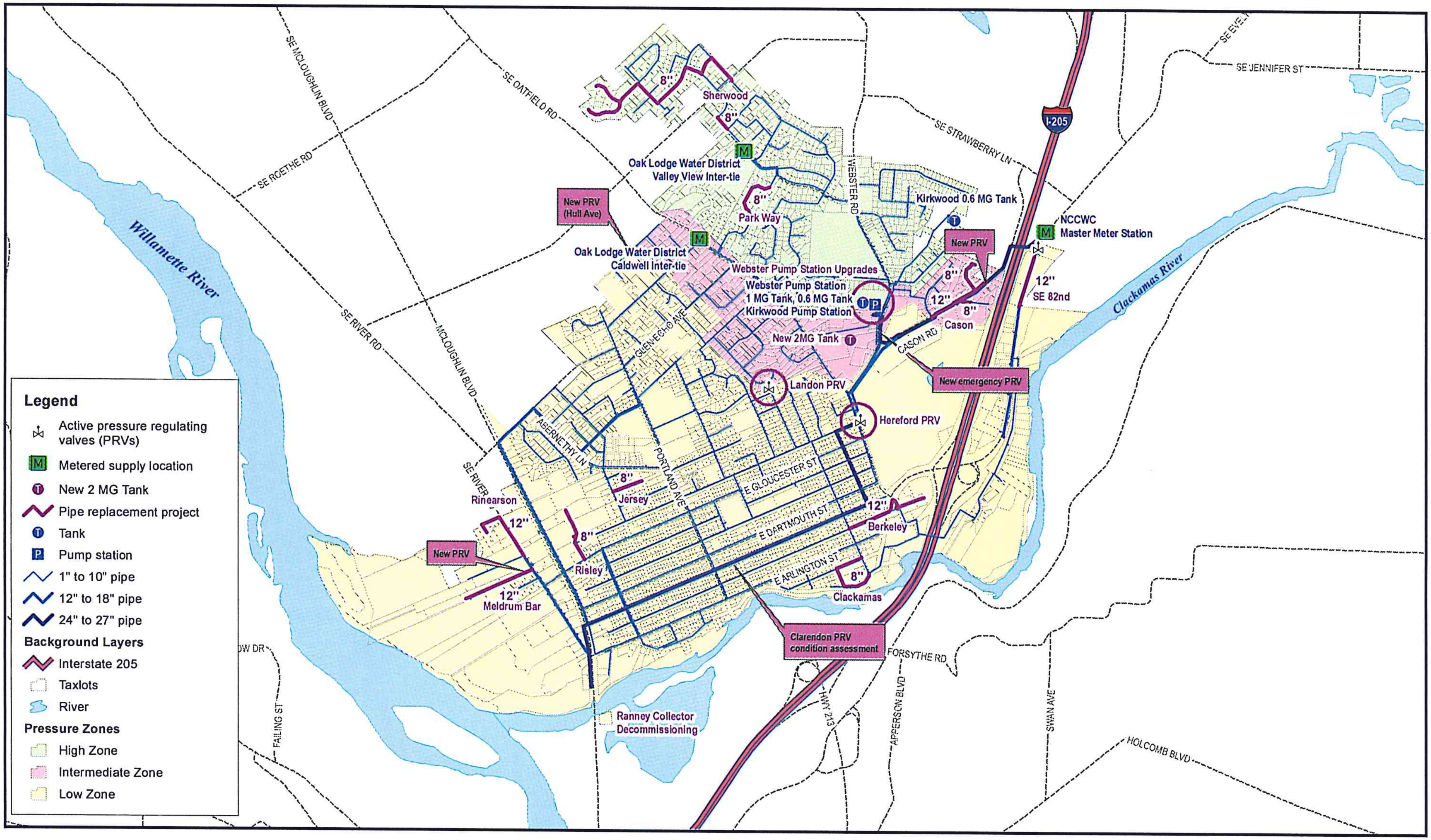
- Hereford PRV because it is submerged in the existing vault and known to be in poor condition.
- Landon PRV due to lack of access. This PRV appears to be buried on private property.
- A leaky connection was identified between the high- and intermediate-pressure zones at Collins Crest. This connection is in need of repair.
- Clarendon PRVs. There are also four PRVs in the low-pressure zone along Clarendon Street that were used previously to regulate pressures from the Ranney Collector system to the downtown area. The physical condition of the four PRVs is unknown. This plan includes a recommendation (Section 7.2) to investigate the condition of these PRVs to ensure that they are not hindering pressures and/or do not require decommissioning.

6.4 Summary of Identified Problems/Issues

The system evaluation identified the following problems/issues for correction with proposed capital improvements as summarized in Section 7:

1. The Ranney intake system could require decommissioning prior to sale of property.
2. Fire flow deficiencies are estimated at 49 locations (due to undersized pipes and lack of looping). Specific locations of dead-end systems include Ridge Drive, Cason Circle, Rivergate School Drive, and Lancaster Drive, which also have limited access.
3. There is concern regarding the age and condition of the 17 miles of AC piping in the city.
4. Operating pressures exceed allowable pressures in pipes located at Meldrum Bar Park Road and at the end of Hardway Court.
5. The current location and configuration of a number of PRVs limits the City's ability to test and maintain them.
6. The backup propane pump at the Webster Pump Station is not reliable.
7. An updated data collection system is needed at the Webster Pump Station.
8. A leaky connection was identified between the high- and intermediate-pressure zones at Collins Crest.
9. An additional storage capacity of 2.0 MG is needed to meet emergency storage criteria.

Proposed improvements to address these issues are described in Section 7.



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September, 2014

0 1,500 3,000 Feet

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FUTURE SYSTEM LAYOUT

FIGURE 6-2

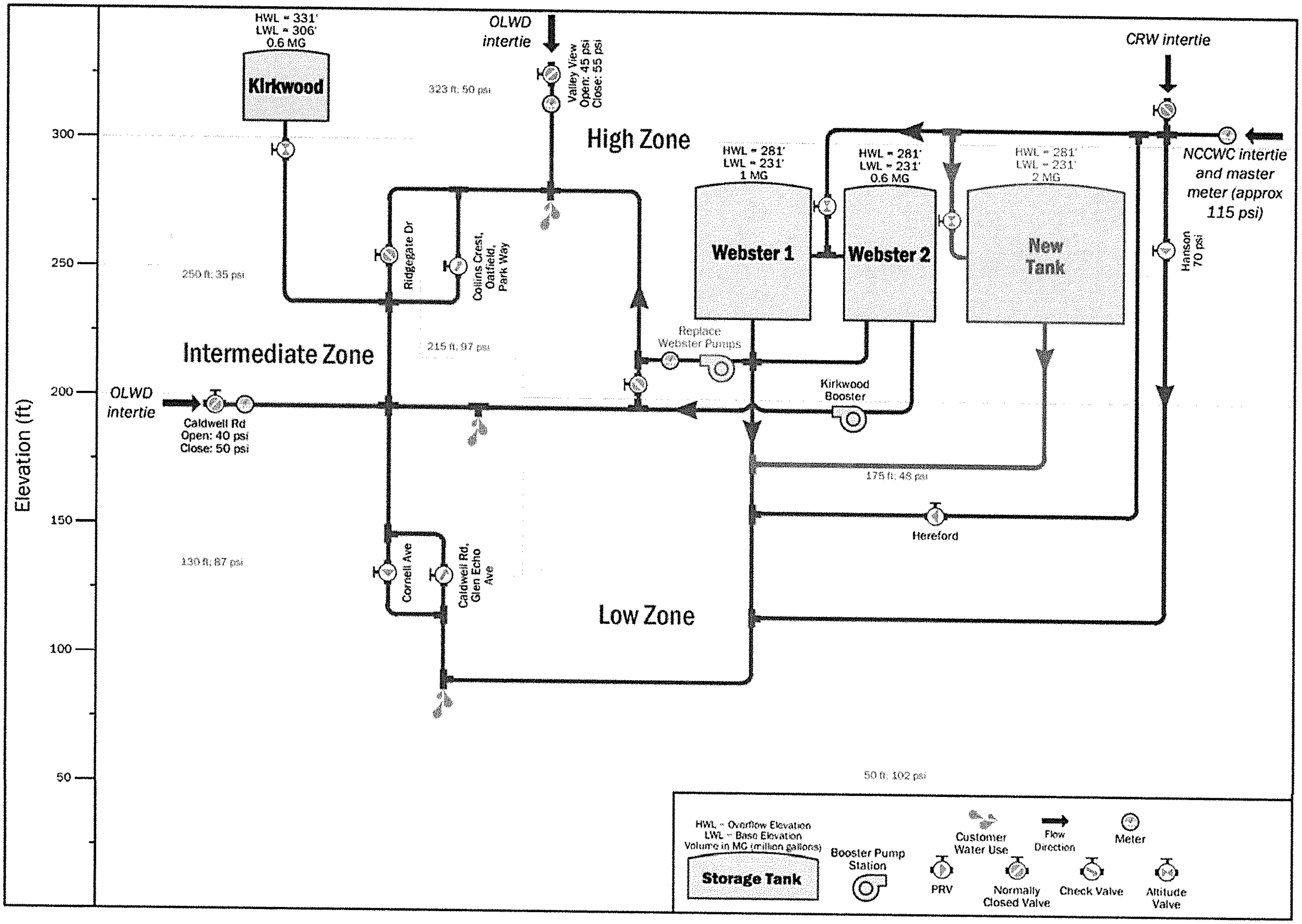

Figure 6-3. Future system hydraulic schematic

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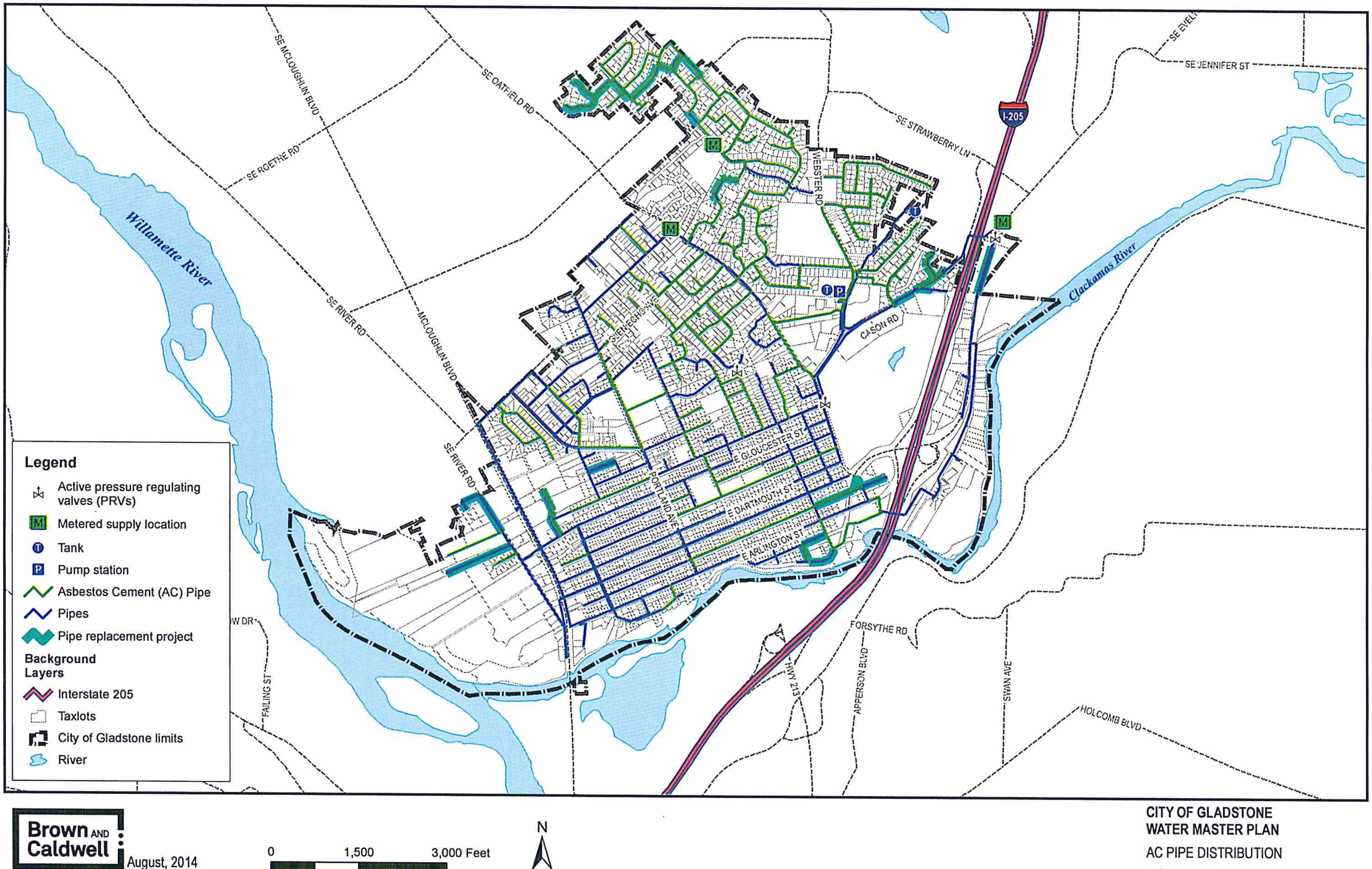

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August, 2014

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AC PIPE DISTRIBUTION

FIGURE 6-4

Section 7

Recommended Improvements

This section presents CIPs designed to address existing system deficiencies and future system demands through 2035 per the evaluation criteria described in Section 5. A summary of the proposed CIPs, the basis for cost estimates, and project prioritization are included in this section.

7.1 CIP Descriptions

A summary of each proposed CIP is provided below and includes identification of the project objective, statement of need, project description, estimated project cost, and associated design assumptions. Projects are listed alphabetically for each facility type.

Proposed projects are shown in Figure 6-2.

7.1.1 Supply

CIP name	Ranney Intake System Decommissioning
No. of problem addressed from Section 6.4	1
Objective addressed	Supply
Pressure zone	N/A
Statement of need	The Ranney Collector system intake has not been used since the mid 1980s, but was never formally decommissioned.
Project description	In conjunction with current City efforts, formally decommission the Ranney Collector system intake to allow for potential future sale of the property.
Estimated total project cost	$50,000
Design assumptions	• Scope and level of effort are currently unknown. The City is currently coordinating with a contractor to develop a scope of work. A placeholder of $50,000 was included for purposes of CIP development and documentation. • Coordination with DEQ will be required to confirm requirements related to access and in-water work. • Decommissioning efforts should include the 16-inch intake waterline along Highway 99E.

7.1.2 Piping

CIP name	Berkeley Street Pipe Replacement
No. of problem addressed from Section 6.4	2 and 3
Objective addressed	Existing fire flow deficiency
Pressure zone	Low
Statement of need	The fire flow deficiencies along Berkeley Avenue are due to undersized piping. These hydrants serve light industrial and commercial properties near Southeast 82nd Drive and Berkeley Avenue.
Project description	Replace 1,604 LF of existing 6-inch asbestos cement (AC) and ductile iron (DI) pipe with new fully restrained, 12-inch DI pipe to be located on Berkeley Avenue between Columbia Avenue and Interstate 205.
Estimated total project cost	$960,000

CIP name	Berkeley Street Pipe Replacement (continued)
Design assumptions	• Acquisition of easements may be required due to the existing pipe alignment on private property. • Approximately 225 feet of this pipeline is currently aligned on private property at 250 Princeton Avenue. This CIP includes re-routing the pipeline alignment to the north to eliminate crossing private property. • Approximately 625 feet of this CIP is aligned on SE 82nd Drive and Princeton Avenue, which are arterial roadways requiring traffic control during construction. Traffic control was assumed as part of the CIP cost estimate. • Hydrant replacement is proposed in the same location as existing. • The CIP cost estimate does not reflect decommissioning and/or removal of existing AC pipe. Such activity is likely to impact the CIP cost if required. Coordination with DEQ may be needed to ensure appropriate environmental precautions are taken.

CIP name	Cason Road PRV and Pipe Replacement
No. of Problems Addressed from Section 6.4	2 and 3
Objective addressed	Existing fire flow deficiency, reliability
Pressure zone	Low
Statement of need	The intermediate zone east of the Webster site and south of Ridgewood Drive is served via a 640 foot, 6-inch AC pipeline that is aligned between tax lots from 17928 Webster Road to 7495 Ridge Drive. This pipeline is undersized to meet fire flow requirements and is difficult to access due to its alignment on private property. As a result of the undersized 6-inch piping and a lack of system looping, fire flow deficiencies occur on Cason Circle Cason Drive, and Ohlson Road. Limited fire flow capacity is currently available to serve the Rivergate Adventist Elementary School, Gladstone Park Seventh Day Adventist Church, Somerset Retirement Living and other residences in this area.
Project description	This project includes a new PRV and vault, which would be used to regulate pressures so that the 27-inch CCP main could be used to serve properties off of Ridge Drive, Cason Circle and Ohlson Road. This option also includes a new 12-inch pipeline from Ridge Drive to 8396 Cason Road and 8-inch pipelines on Cason Circle and Ohlson Road. The alternative to supplying this area with the intermediate zone would require a new pipeline along Webster Road and Cason Road, which was determined to be a more expensive alternative and was not reflected in the cost estimate. To provide emergency service to these properties off of Ridge Drive, Cason Circle and Ohlson Road in the event that the NCCWC connection is disabled, an emergency connection PRV will be located between the intermediate zone and the 27-inch supply pipe from NCCWC. This will allow service and fire flow from the Kirkwood tank in the event that the NCCWC connection is disabled.
Estimated total project cost	$1,260,000
Design assumptions	• Acquisition of easements may be required due to the existing pipe alignment on private property. • This CIP is located within the ROW, thus traffic control may be required during construction. Traffic control was assumed as part of the CIP cost estimate. • The emergency service PRV is proposed to ensure this area has a backup source of water service and fireflow to the main system in the event the NCCWC master meter is disabled. Alternatively, the existing 6-inch AC pipeline (proposed for decommissioning) may be preserved to provide domestic service from the Kirkwood tanks in the event the NCCWC master meter is disabled; however it would not provide adequate fireflow. • The PRV for emergency service should be set to 45 psi, and for purposes of this CIP has been located south of the Webster pump station in the ROW. It may be located at the Webster pump station dependent on available space . Sixty feet of 8-inch DI pipe was estimated for connection. • The PRV cost estimate is based on quote by Carol Wells at GC Systems Inc. and includes a packaged 4-inch PRV with 1 ½-inch bypass and a pressure relief valve. Installation to be completed by general contractor. Vault size is 10 feet x 5 feet x 7.5 feet. • Hydrant replacement is proposed in the same location as existing. • The CIP cost estimate does not reflect decommissioning and/or removal of existing AC pipe. Such activity is likely to impact the CIP cost if required. Coordination with DEQ may be needed to ensure appropriate environmental precautions are taken.

CIP name	Clackamas Boulevard Pipe Replacement
No. of problem addressed from Section 6.3	2 and 3
Objective addressed	Existing fire flow deficiency, reliability
Pressure zone	Low
Statement of need	The fire flow deficiency at the hydrant on Cornell Avenue and Clackamas Boulevard is due to undersized piping and a lack of system looping.
Project description	• Replace 1,309 LF of existing 4-inch DI pipe and 6-inch AC pipe with new fully restrained, 8-inch DI pipe to be located within the roadway on First Street between Columbia Avenue and Cornell Avenue and also on Cornell Avenue between First Street and Clackamas Boulevard. • Improve system looping by installing 527 LF of new fully restrained, 8-inch DI pipe between Columbia Avenue and Clackamas Boulevard.
Estimated total project cost	$840,000
Design assumptions	• Property acquisition is not included in the cost estimate. The City may need to obtain easements. • Hydrant replacement is proposed in the same location as existing. • This CIP is located within the ROW, thus traffic control may be required during construction. Traffic control was assumed as part of the CIP cost estimate. • The CIP cost estimate does not reflect decommissioning and/or removal of existing AC pipe. Such activity is likely to impact the CIP cost if required. Coordination with DEQ may be needed to ensure appropriate environmental precautions are taken.

CIP Name	Clarendon PRV Condition Assessment
No. of problem addressed from Section 6.3	5
Objective addressed	Reliability
Pressure zone	Low
Statement of need	There are four PRVs in the low zone along Clarendon Street, which were previously used to regulate pressures from the Ranney Collector system to the downtown area. The physical condition of the four PRVs is unknown.
Project description	Conduct a condition assessment of the four PRVs to ensure they are not hindering pressures and/or require decommissioning
Estimated total project cost	$10,000
Design assumptions	None.

CIP name	Hereford PRV
No. of problem addressed from Section 6.3	5
Objective addressed	Reliability
Pressure zone	Low
Statement of need	The existing Hereford PRV reduces pressures from the incoming 27-inch main to supply the low pressure zone. This PRV is reported to be submerged in the existing vault and in poor condition according to City staff.
Project description	Replace the existing Hereford PRV and vault.
Estimated total project cost	$110,000
Design assumptions	• This CIP was identified from City staff input. • The PRV cost estimate is based on quote by Carol Wells at GC Systems Inc. and includes a packaged 4-inch PRV with 1 ½-inch bypass and a pressure relief valve. Installation to be completed by general contractor. Vault size is 10 feet x 5 feet x 7.5 feet. • The CIP cost estimate assumes 100 feet of 12-inch DI piping for hook up.

CIP name	Hull Avenue PRV
No. of problem addressed from Section 6.3	4
Objective addressed	Operating pressures exceed allowable pressures
Pressure zone	Low
Statement of need	Operating pressures along Hardway Court exceed allowable pressures. A PRV is needed to reduce pressures below 100 psi.
Project description	Install 1 PRV on Hull Ave between Hardway Court and Scutton Lane.
Estimated total project cost	$110,000
Design assumptions	• The PRV cost estimate is based on quote by Carol Wells at GC Systems Inc. and includes a packaged 4-inch PRV with 1 ½-inch bypass and a pressure relief valve. Installation to be completed by general contractor. Vault size is 10 feet x 5 feet x 7.5 feet. • The CIP cost estimate assumes 100 feet of 12-inch DI piping for hook up.

CIP name	Jersey Street Pipe Replacement
No. of problem addressed from Section 6.3	2
Objective addressed	Existing fire flow deficiency
Pressure zone	Low
Statement of need	Fire flow deficiencies along Jersey Street, east of Beatrice Avenue are due to undersized distribution piping and a lack of looping.
Project description	Replace 510 feet of existing 4-inch CI pipe with new fully restrained, 8-inch DI pipe to be located within the roadway on Jersey Street, east of Beatrice Avenue.
Estimated total project cost	$330,000
Design assumptions	• System looping was not included in this CIP due to the location of the existing 48-inch storm drain which conveys flows to Rinearson Creek and crosses Jersey Street and Bellevue Avenue to the east and south of the project pipe. • This CIP is located within the ROW, thus traffic control may be required during construction. Traffic control was assumed as part of the CIP cost estimate. • Hydrant replacement is proposed in the same location as existing.

CIP name	Landon PRV
No. of problem addressed from Section 6.3	5
Objective addressed	Reliability
Pressure zone	Low
Statement of need	The existing PRV north of Landon Street and Cornell Avenue provides a connection between the intermediate and low pressure zones. The PRV was not found during the system inventory phase of this project because it is buried on private property. The City is unable to maintain this PRV in its current location.
Project description	Replace the existing Landon PRV with a new valve and vault located within the ROW of Cornell Avenue.
Estimated total project cost	$110,000
Design assumptions	• This CIP was identified from City staff input. • The PRV cost estimate is based on quote by Carol Wells at GC Systems Inc. and includes a packaged 4-inch PRV with 1 ½-inch bypass and a pressure relief valve. Installation to be completed by general contractor. Vault size is 10 feet x 5' x 7.5 feet. • The CIP cost estimate assumes 100 feet of 12-inch DI piping for hook up. • Property acquisition is not included in the cost estimate. The City may need to obtain easements.

CIP name	Meldrum Bar Park Road PRV and Pipe Replacement
No. of problem addressed from Section 6.3	2 and 4
Objective addressed	Existing fire flow deficiency and operating pressures exceed allowable pressures
Pressure zone	Low
Statement of need	Fire flow deficiencies along Meldrum Bar Park Road are due to undersized piping and a lack of looping. In addition to serving fire flow demands in Meldrum Bar Park, the hydrants along Meldrum Bar Park Road also have the potential to provide backup fire flow to the mobile home park to the north and subdivision to the south. Additionally, the operating pressures along Meldrum Bar Park Road exceed allowable pressures. A PRV is needed to reduce pressures below 100 psi.
Project description	• Replace 1,194 feet of existing 6-inch DI pipe with new fully restrained, 12-inch DI pipe to be located within the roadway on Meldrum Bar Park Road. • Install 1 PRV near the intersection of Meldrum Bar Park Road and River Road.
Estimated total project cost	$680,000
Design assumptions	• System looping was not included in this CIP because the system terminates at the end of the project pipe with no available looping opportunities within the right of way nearby. • The PRV cost estimate is based on quote by Carol Wells at GC Systems Inc. and includes a packaged 4-inch PRV with 1 ½-inch bypass and a pressure relief valve. Installation to be completed by general contractor. Vault size is 10 feet x 5 feet x 7.5 feet. • This CIP is located within the ROW, thus traffic control may be required during construction. Traffic control was assumed as part of the CIP cost estimate. • Hydrant replacement is proposed in the same location as existing. • No service line connections were included in this CIP. The service line for the mobile home park is a separate 620 foot, 2-inch steel line not assumed to be replaced as part of this CIP.

CIP name	Park Way Pipe Replacement
No. of problem addressed from Section 6.3	2 and 3
Objective addressed	Existing fire flow deficiency, reliability
Pressure zone	High
Statement of need	Fire flow deficiencies along Park Way between Los Verdes Drive and Oatfield Road are due to undersized distribution system piping. This neighborhood is also served by AC pipe which is nearing the end of its useful life and has prevented the City from completing fire flow testing due to concerns of main failures.
Project description	• Replace 155 feet of existing 6-inch AC pipe with new fully restrained, 8-inch DI pipe to be located within the roadway on Park Way between Oatfield Road and the first hydrant on the Park Way line. • Replace 750 feet of existing 6-inch AC pipe with new fully restrained, 8-inch DI pipe to be located within the roadway on Park Way between house numbers 6820 and 6703.
Estimated total project cost	$510,000
Design assumptions	• This CIP is located within the ROW, thus traffic control may be required during construction. Traffic control was assumed as part of the CIP cost estimate. • The cost estimate assumes replacement of hydrants in the same location as existing, plus one additional hydrant. • The CIP cost estimate does not reflect decommissioning and/or removal of existing AC pipe. Such activity is likely to impact the CIP cost if required. Coordination with DEQ may be needed to ensure appropriate environmental precautions are taken.

CIP name	Sherwood Neighborhood Pipe Replacement
No. of problem addressed from Section 6.3	2 and 3
Objective addressed	Existing fire flow deficiency, reliability
Pressure zone	High
Statement of need	Fire flow deficiencies in the Sherwood Forest neighborhood north of Jennings Avenue are due to undersized distribution system piping and a lack of looping. This neighborhood is also served by AC pipe which is nearing the end of its useful life and has prevented the City from completing fire flow testing due to concerns of main failures. Much of the existing piping in this neighborhood was installed behind the curb in common trenches with other utilities, which has made maintenance and repair difficult.
Project description	• Improve system looping by installing 260 feet of new fully restrained, 8-inch DI pipe on Valley View Drive between Churchill Drive and Buckingham Drive. • Replace 3,930 feet of existing 6-inch AC pipe with new fully restrained, 8-inch DI pipe to be located within the roadway.
Estimated total project cost	$2,170,000
Design assumptions	• This CIP is located within the ROW, thus traffic control may be required during construction. Traffic control was assumed as part of the CIP cost estimate. • Hydrant replacement is proposed in the same location as existing. • The CIP cost estimate does not reflect decommissioning and/or removal of existing AC pipe. Such activity is likely to impact the CIP cost if required. Coordination with DEQ may be needed to ensure appropriate environmental precautions are taken.

CIP name	Rinearson Road Pipe Replacement
No. of problem addressed from Section 6.3	2
Objective addressed	Existing fire flow deficiency
Pressure zone	Low
Statement of need	Fire flow deficiencies along River Road and Rinearson Road, north of Meldrum Bar Park Road are due to undersized distribution piping and a lack of looping. The hydrants along this line serve two apartment complexes west of River Road.
Project description	Replace 1,207 feet of existing 6-inch DI pipe with new fully restrained, 8-inch DI pipe to be located within the roadway on River Road and Rinearson Road.
Estimated total project cost	$590,000
Design assumptions	• System looping was not included in this CIP because the system terminates at the end of the project pipe with no available looping opportunities nearby. • This CIP is located within the ROW, thus traffic control may be required during construction. Traffic control was assumed as part of the CIP cost estimate. • Hydrant replacement is proposed in the same location as existing.

CIP name	Risley Avenue Pipe Replacement
No. of problem addressed from Section 6.3	2 and 3
Objective addressed	Existing fire flow deficiency
Pressure zone	Low
Statement of need	Fire flow deficiencies along Risley Avenue, north of Gloucester Street are due to undersized distribution piping and a lack of looping. The hydrants at the end of Risley Avenue serve an apartment building complex. This neighborhood is also served by AC pipe.
Project description	Replace 893 feet of existing 6-inch AC pipe with new fully restrained, 8-inch DI pipe to be located within the roadway on Risley Avenue.
Estimated total project cost	$460,000
Design assumptions	• System looping was not included in this CIP because Risley Avenue is a dead end street, which terminates at Rinearson Creek. • This CIP is located within the ROW, thus traffic control may be required during construction. Traffic control was assumed as part of the CIP cost estimate. • Hydrant replacement is proposed in the same location as existing. • The CIP cost estimate does not reflect decommissioning and/or removal of existing AC pipe. Such activity is likely to impact the CIP cost if required. Coordination with DEQ may be needed to ensure appropriate environmental precautions are taken.

CIP name	SE 82nd Drive Pipe Replacement
No. of problem addressed from Section 6.3	2
Objective addressed	Existing fire flow deficiency
Pressure zone	Low
Statement of need	The fire flow deficiencies along SE 82nd Drive are due to a section of undersized 8-inch piping.
Project description	Replace 860 feet of existing 8-inch DI pipe with new fully restrained, 12-inch DI pipe to be located between 17765 82nd Drive and 1250 SE 82nd Drive.
Estimated total project cost	$470,000
Design assumptions	• This CIP is located on 82nd Drive, an arterial roadways requiring traffic control during construction. Traffic control was assumed as part of the CIP cost estimate. • Hydrant replacement is proposed in the same location as existing.

7.1.3 Pump Station

CIP name	Webster Pump Station Upgrades (Generator Set)
No. of problem addressed from Section 6.3	6
Objective addressed	Provide a backup emergency power source large enough to operate both the Webster and Kirkwood pumps in the event of a utility power outage.
Pressure zone	N/A
Statement of need	The current electrical service for the Webster and Kirkwood Pumps does not have a sufficient backup emergency power source.
Project description	• Install a new Diesel Electric standby emergency generator at the Webster pump station building to include the following: • 125 KW, 480 volt, 3 phase outdoor standby emergency generator set with an integral 250 gallon sub-base mounted fuel storage tank. • 480 volt, 200 amp, 3 pole, 100 percent load rated service entrance rated circuit breaker cabinet installed inside the pump room. • 480 volt, 200 amp, 4 pole automatic transfer switch cabinet installed inside the pump room. • New power conduit and cabling reconnecting the Utility Electrical Service from the meter box to the new main circuit breaker, new power conduit and cabling connecting the new main circuit breaker and generator set to the automatic transfer switch and new power conduit and cabling connecting the automatic transfer switch to the existing pump station power panel. • New control conduit and cabling connecting the generator set, automatic transfer switch and SCADA monitoring system.
Estimated total project cost	$150,000
Design assumptions	• This CIP does not include replacement of the existing Webster or Kirkwood pumps. Analysis of the system found the pumps to be insufficient for addressing fire flow demand in the high zones. However, this demand is met with a reliance on the OLWD inter-ties. The City will continue to rely on the inter-ties to meet this demand. • The generator set should be installed in a weatherproof residential area-rated sound insulated enclosure. • A 250 gallon fuel reserve provides approximately 25 hours continuous run time at full load. Average loading for this station under heavy use is assumed to be approximately 75 percent which would allow approximately 33 hours continuous runtime before requiring refueling. • Prices for the generator, fuel tank and quit zone enclosure were provided by Pacific Power Products in Kent, WA, a local distributor for MTU Onsite Energy products. • Price for the power service was provided by Eaton Power Products (local contact in Wilsonville, Ore.). • Installation to be provided by contractor. A general installation cost of $25,600 was reflected in the CIP cost estimate. • AC pipe replacement in the vicinity of the pump station was not reflected in the cost estimate. AC pipe replacement may be prioritized in this location per the annual replacement program at the time of construction.

CIP name	Webster Pump Station SCADA System
No. of problem addressed from Section 6.3	7
Objective addressed	Provide updated system to collect and store data from the Webster and Kirkwood pump stations.
Pressure zone	N/A
Statement of need	The current system to log data and trigger alarms is outdated, it does not allow remote access, and it is prone to failure.
Project description	Update the data logging and alarming functions using a SCADA monitoring service. Use an outside SCADA monitoring and alarm handling service to improve the reliability of the system operation. This would include installing SCADA systems meters at the Webster Road pump station building, the NCCWC master meter main station, the OLWD Valley View intertie meter station and the OLWD Caldwell intertie meter station.
Estimated total project cost	$20,000
Design assumptions	• SCADA system components including the RTU System, analog expansion module, and antenna cable were provided by Mission Communications (local distributor is Correct Equipment in Canby, Ore.). • SCADA system freight charges, set up charges, and onsite training charges were provided by Mission Communications (local distributor is Correct Equipment in Canby, Ore.). • Installation to be provided by contractor. A general installation cost of $2,000 was reflected in the CIP cost estimate for the pump station and $750 for each meter location. • Annual maintenance is required when using an outside SCADA monitoring and alarm service. A total annual maintenance cost estimate is approximately $2,500 as provided by Mission Communications (local distributor is Correct Equipment in Canby, Ore.).

7.1.4 Storage

CIP name	New 2MG Storage Tank
No. of problem addressed from Section 6.3	9
Objective addressed	Provide additional storage capacity to meet future emergency storage demands.
Pressure zone	N/A
Statement of need	Available storage capacity in the Webster and Kirkwood tanks does not exist to meet an emergency storage demand (defined as two days of average daily water demand). Such emergency storage demand is a subjective criteria as stated in Section 5.6.3.
Project description	Install a 2 MG steel or reinforced concrete tank at the City-owned Oatfield Road and Webster Road location to provide equalization, fire flow and emergency storage for the City's system. The tank would supply the low pressure zone by gravity.
Estimated total project cost	• $4,500,000 (steel)
Design assumptions	• The proposed location of the facility is on the 11.7 acre vacant, City-owned lot at Oatfield Road and Webster Road. A 250' x 250' tank placement area was assumed for purposes of cost estimating (site preparation and clearing). • Placement of the tank is assumed in the center of the tax lot, at the same elevation and height as the Webster tanks. • An alternative cost estimate was prepared for a reinforced concrete tank (see Appendix F). • Inlet pipe connection (to the existing 27" main) is not included in the cost estimate. 1000' of 12" DI pipe was included in the cost estimate for site piping. • An altitude valve and vault and mixer for water quality are not specifically included in the cost estimate. • An access road was included in the cost estimate. The proposed facility access road is estimated as 600' x 24' paved, with a 16' shoulder. • Unit tank construction costs include continuous footings, grade, and tie beams, foundation slabs, painting and surface finishes.

7.2 Capital Maintenance Program

7.2.1 AC Pipe Replacement and Pipe Condition Assessment

The City's water distribution system includes approximately 17 miles of AC pipe as shown on the AC pipe distribution (Figure 6-4). The size distribution of the AC pipe includes 5,410 LF of 4-inch, 71,120 LF of 6-inch and 11,690 LF of 8-inch-diameter pipes. AC pipes have been prone to failure in recent years and are difficult to work with due to special precautions that must be taken when working with the material. It is recommended that the City begin an AC replacement program, with the goal of replacing all existing AC pipe. At the time of replacement, 8-inch-diameter should be used as a minimum pipe size to meet fire flow requirements. Connections to existing AC pipe are difficult to make and have the tendency to leak, so it is recommended that the City strategize the replacement of their AC pipe to minimize the need for connections to existing AC pipe.

The total cost in 2014 dollars to replace all existing AC pipe, excluding the pipe replacement already reflected in existing CIP projects to address fire flow deficiencies is anticipated to be approximately $24,600,000. This cost includes the net construction cost and associated gross markups (see Appendix F). An annual cost of $820,000 is recommended for AC pipe replacement, in order to complete replacement over a 30-year implementation period.

While AC pipe replacement is recommended, prior to initiating replacement efforts, a leak detection survey and/or pipe conditions assessment is highly recommended to assist in prioritizing replacement. It is possible that higher priority maintenance problems may also exist in other areas of the system. Please note that a leak has already been detected at Collins Crest. A lump sum of $75,000 has been incorporated into the 2014 water utility funding analysis/rate evaluation to conduct a leak detection survey prior to AC pipe replacement efforts. At this time, cost for a condition assessment of the entire water conveyance system has not been included.

7.2.2 Preventative Maintenance Program

Preventative maintenance is essential to optimizing functionality and performance of a water system. As described in Section 5.7.2, the City currently does not have a documented O&M program, or current staffing to conduct preventative maintenance efforts at the recommended frequency. Implementation of this Master Plan and CIP projects is dependent upon the addition of staff to conduct/oversee preventative maintenance efforts. The addition of two full time staff has been incorporated into the 2014 water utility funding analysis/rate evaluation to supplement existing staff in support of a preventative water system maintenance program.

7.2.3 Third-Party SCADA System Maintenance

An additional capital maintenance item includes annual maintenance of the SCADA system proposed as a CIP above. This is estimated to be approximately $2,500 per year.

7.3 Cost Estimates for CIP Development

Cost estimates for CIP design and construction were based on the total capital investment necessary to complete a project (i.e., engineering through construction). Expenditures were calculated for construction or capital elements, based on the CIP design and representing material costs, labor costs, other services (traffic control, erosion control), and contingency. Expenditures were calculated separately for administrative and design services, including engineering and permitting. It should be noted that construction contingencies in this plan of 40 percent are higher than these used for cost estimating CIPs in the stormwater master plan. This is due to added complexities or constructing pressurized water pipe (as opposed to gravity-fed storm pipes) and the unknown issues associated with proper decommissioning and disposal of asbestos concrete pipe.

Unit cost information for construction or capital elements of the CIP facilities was compiled from the Association for the Advancement of Cost Engineering International Criteria (see Appendix F). Land acquisition and easement costs are not included in the cost estimates, as most projects proposed are located on City property or within the City ROW. It is assumed that the City will obtain necessary easements for work conducted on private property.

Unit cost information and individual cost estimates for CIPs are included in Appendix F. CIPs in Appendix F follow the same order as CIP descriptions listed in Section 7.1. For planning purposes in Section 7.1, the cost for CIPs under $100,000 were rounded to the nearest $1,000; CIPs over $100,000 were rounded to the nearest $10,000.

A summary of CIP costs is provided in Table 7-1.

Table 7-1. CIP Estimated Cost Summary

CIP name	Total cost ($)
Supply	
Ranney Intake System Decommissioning	50,000
Piping	
Berkeley Street Pipe Replacement	960,000
Cason Road PRV and Pipe Replacement	1,260,000
Clackamas Boulevard Pipe Replacement	840,000
Clarendon PRV Condition Assessment	10,000
Hereford PRV	110,000
Hull Avenue PRV	110,000
Jersey Street Pipe Replacement	330,000
Landon PRV	110,000
Meldrum Bar Park Road PRV and Pipe Replacement	680,000
Park Way Pipe Replacement	510,000
Sherwood Neighborhood Pipe Replacement	2,170,000
Rinearson Road Pipe Replacement	590,000
Risley Avenue Pipe Replacement	460,000
SE 82nd Drive Pipe Replacement	470,000
AC Pipe Replacement[a]	24,600,000
Pump Station	
Webster Pump Station Upgrades (Generator Set)	150,000
Webster Pump Station SCADA System	20,000
Storage	
New 2 MG Storage Tank	4,500,000
Total	$37,930,000

[a] *Recommended as an annual line item in the CIP of $820, 000. A leak detection survey is recommended prior to pipe replacement to prioritize the location of replacements.*

7.4 CIP Prioritization and Implementation

This section summarizes the general process the City used to prioritize identified CIPs. The City conducted its CIP prioritization in conjunction with its water utility rate evaluation (separate deliverable).

7.4.1 CIP Prioritization Criteria and Process

As described in Section 7.1, a total of 19 CIPs were developed to address water system supply, piping, pump stations, and storage deficiencies. Due to the significant cost of the CIPs proposed, an extended implementation period was used for the water system rate evaluation. Therefore, the 30-year implementation period as opposed to the traditional 20-year planning horizon was used for CIP scheduling.

Per discussion with the City on September 18, 2014, all CIPs are considered viable and necessary projects, but some CIPs were identified as lower priority that could be constructed later in the 30-year implementation timeframe. Lower priority CIPs included those where modeling alone indicated deficiencies but there were no reported complaints. Lower priority CIPs also included those that did not address established evaluation criteria.

In conjunction with identification of lower priority CIPs, City staff identified general guidelines to be used to identify higher priority CIPs. Guidelines included whether the CIP addresses ongoing maintenance issues/concerns, whether the CIP addresses modeled fire flow deficiencies, whether the CIP is located in the high pressure zone (with an ongoing history of citizen complaints), and whether the CIP includes replacement of AC pipe. Identified higher priority CIPs are recommended for scheduling earlier in the 30-year CIP implementation process.

7.4.2 CIP Scheduling

Results of the CIP prioritization efforts are documented in Table 7-2. CIPs were not specifically ranked but rather grouped according to whether they were identified as a lower priority project or higher priority project. Again, lower priority projects would be targeted for construction toward the end of the 30-year CIP implementation period, and higher priority projects would be targeted for construction toward the beginning of the 30-year CIP implementation period. CIPs not indicated as lower or higher priority would be constructed within the 30-year CIP implementation period as funding is available and at the discretion of City staff.

Table 7-2. CIP Implementation Schedule

CIP name	Priority	Rationale for schedule
Supply		
Ranney Intake System Decommissioning	L	Does not address established evaluation criteria
Piping		
Berkeley Street Pipe Replacement	H	Fire flow deficiency, AC pipe
Cason Road PRV and Pipe Replacement	H	Fire flow deficiency, AC pipe
Clackamas Boulevard Pipe Replacement	H	Fire flow deficiency, AC pipe
Clarendon PRV Condition Assessment	L	Does not address established evaluation criteria
Hereford PRV	↔	
Hull Avenue PRV	L	Service may transfer to OLWD
Jersey Street Pipe Replacement	L	Recently replaced (still undersized), no reported complaints
Landon PRV	↔	
Meldrum Bar Park Road PRV and Pipe Replacement	L	No residential or commercial services affected
Park Way Pipe Replacement	H	Fire flow deficiency, AC pipe, high pressure zone
Sherwood Neighborhood Pipe Replacement	H	Fire flow deficiency, AC pipe, high pressure zone
Rinearson Road Pipe Replacement	↔	
Risley Avenue Pipe Replacement	↔	
SE 82nd Drive Pipe Replacement	↔	
Pump Station		
Webster Pump Station Upgrades (Generator Set)	H	Ongoing maintenance concern
Webster Pump Station SCADA System	H	Ongoing maintenance concern
Storage		
New 2 MG Storage Tank	↔	

H = Higher priority projects targeted for construction toward the beginning of the 30-year CIP implementation period.

L = Lower priority projects targeted for construction toward the end of the 30-year CIP implementation period.

↔ = Projects would be constructed within the 30-year CIP implementation period as funding is available and at the discretion of City staff.

7.4.3 CIP Implementation

As stated above, CIP implementation is projected over a 30-year period. The financial analysis and water utility rate evaluation effort considers the CIP project costs and anticipated project scheduling in development of recommended water utility rates.

In addition, the financial analysis considers capital maintenance costs and expenditures in the calculation of rates (Section 7.2). An annual cost of $820,000 is included for implementation of the AC Pipe Replacement effort. An annual cost of $2,500 is dedicated for maintenance of the proposed SCADA system. A lump sum of $75,000 is proposed for the beginning of the CIP implementation period to conduct a leak detection investigation in order to prioritize pipes (including AC pipe) for replacement.

Historically, due to limited staff availability, preventative maintenance of the water system has not been performed routinely and proactively. The City's existing public works department consists of six full time staff that are shared amongst stormwater, sanitary, water, parks, and streets. There is no dedicated water department staff. Preventative maintenance is essential to optimizing functionality and performance of a water system. The financial analysis includes the addition of two full-time employees (FTE) to supplement existing staff in support of a preventative water system maintenance program. With the addition of staff, and as preventative maintenance activities are conducted and tracked at specified intervals, the staffing allocation should be revisited amongst all utilities to ensure that adequate levels of service are achieved.

Section 8

Limitations

This document was prepared solely for the City of Gladstone (City) in accordance with professional standards at the time the services were performed and in accordance with the contract between the City and Brown and Caldwell dated October 1, 2012. This document is governed by the specific scope of work authorized by the City; it is not intended to be relied upon by any other party except for regulatory authorities contemplated by the scope of work. We have relied on information or instructions provided by the City and other parties and, unless otherwise expressly indicated, have made no independent investigation as to the validity, completeness, or accuracy of such information.

Section 9

References

American Water Works Association (AWWA). (2012). AWWA Manual of Water Supply Practices, M32 Computer Modeling of Water Distribution Systems, Third Edition.

Oregon Administrative Rules (OAR). (2014). Chapter 333, Division 061-0050. Public Water Systems, Construction Standards.

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Appendix A: Model Creation TM

Technical Memorandum

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Prepared for: City of Gladstone

Project Title: Stormwater and Water Master Plan

Project No: 142799

Draft Technical Memorandum

Subject: Water Distribution System Model Development, Task 4

Date: September 5, 2014

To: Scott Tabor, City of Gladstone

From: Krista Reininga, Brown and Caldwell

Prepared by: Janice Keeley, Brown and Caldwell

Reviewed by: Colin Ricks, Brown and Caldwell

Table of Contents

List of Figures

List of Tables

Section 1: Introduction

Prior to this Water Master Plan project, the City of Gladstone (City) maintained a paper copy map of its water system. The first phase of this project involved taking an inventory of the City's water system. The inventory was developed by Sisul Engineering using site survey, interviews with City staff and as-built drawing review. The inventory was documented in AutoCAD. To add accuracy and detail to the master planning effort and future modeling work done by the City, Brown and Caldwell (BC) created a computer model of the water system. The AutoCAD-based inventory was converted to ArcGIS and used as the basis for the computer model, which includes all City-owned distribution mains.

This technical memorandum (TM) describes the methods and data that were used to create the model including the modeling software and workflow, model element information, demand allocation, and quality assurance protocols. Model calibration is documented in the Model Calibration Plan TM and in the final report.

Section 2: Computer Modeling Software and Workflow

The hydraulic model of the City's water system was created using Innovyze's InfoWater. InfoWater is an ArcGIS-based water distribution system modeling software and is well suited for modeling the City's water system. InfoWater is based on the U.S. Environmental Protection Agency's EPANET modeling engine. The final model will be provided to the City in both InfoWater and EPANET formats.

Figure 1 outlines the workflow that was followed to develop the model.

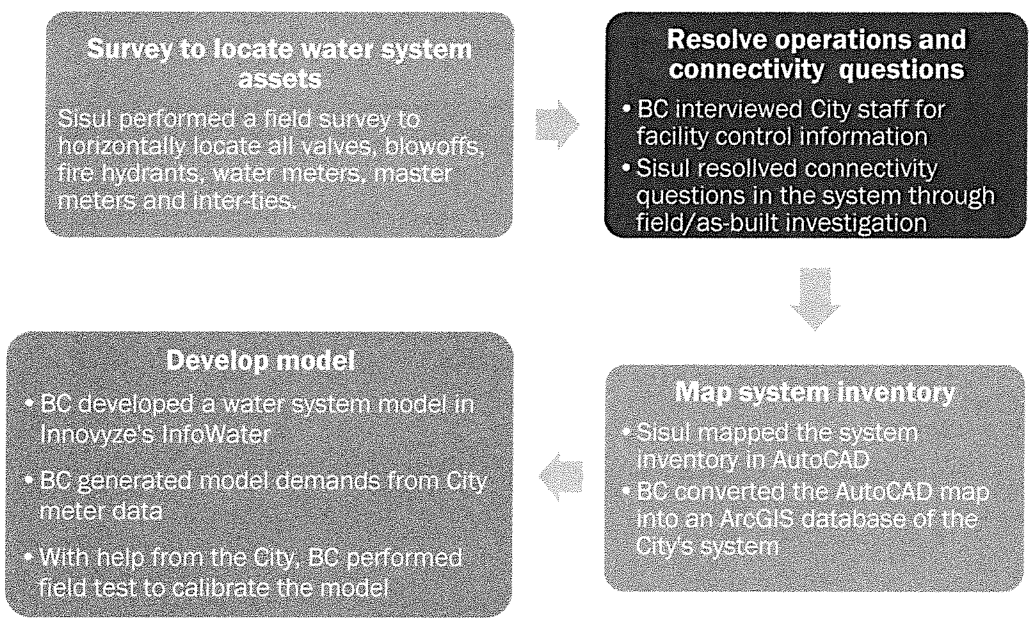

Figure 1. Model development workflow

Section 3: Model Facilities

This section describes how the model facilities were developed from the CAD data. Facility information that was used in the computer calculations, or that can be useful to the model user is stored as attributes in the database files of the model. Table 1 describes the model attributes that apply to all facilities. The data processing and model attributes specific to each element type (e.g., junctions, pipes, etc.) are described in the following sub-sections.

Table 1. Common Attributes

Attribute	Value
ID	ID numbering is alphanumeric, with a prefix and a unique identifier. The prefix indicates element type and the unique identifier includes text describing the facility or a unique number.
Year of installation	This value is used to specify that a facility will be active in a scenario. For example, a facility with an installation year of 2035 or before will be active in a 2035 scenario. This value will be set to 0 if the installation year is unknown.
Year of retirement	This value is used to specify that a facility will be retired (not active) in a scenario. For example, a facility with a retirement year of 2035 or before will not be active in a 2035 scenario.

3.1 Junctions

Junction nodes were created in the model at all changes in pipe diameter, pipe connections, intersections, dead ends, water system valves, water control valves, and hydrant locations, where provided in CAD. New junctions were added at pipe endpoints in the model where existing CAD data did not have a feature already. Demands were applied to junctions in the model (see Section 4 for more detail on how the demands were allocated to each of the junctions). Table 2 lists the attributes applied to the junctions.

Table 2. Junction Attributes

Attribute	Value	
ID	The model junctions were imported from the CAD data and given a descriptive ID representing the junction type from CAD. A description of the various prefixes used to create the junctions is provided below.	
	Prefix	*Sample ID*
	BV (blow-off valve)	BV1023
	HYD (hydrant)	HYD040
	J (model only junction)	J1354
	V (valve)	V1517
Demand 1	The model demand at a junction (see Section 4).	
Elevation	Digital elevation model (DEM) data were used to set the junction elevation. The DEM was based on Light Detection and Ranging data from the City.	

3.2 Pipes

Model pipes were created from CAD data provided by Sisul. In addition to the transmission and distribution system piping, the CAD data include small-diameter service connections. These service connections were not included in the model. Some cleanup of the pipe network was required after the CAD data were imported into the model. A description of the cleanup work performed is described below.

3.2.1 Pipe Network Cleanup

The following steps were taken to clean up the pipe network in the model.

Deleted Very Short Pipes. The imported model pipes included a number of pipes with a length of 1 foot or less. For the most part, the short pipes resulted from pipes in the CAD data that were not snapped directly to a water fitting feature. These short pipes interfere with the other cleanup procedures and add unnecessary calculation nodes to the model. All short pipes that did not connect two water fitting features were deleted. Pipes formerly connected by the small pipes were connected to each other at a common junction.

Deleted Duplicate Pipes. The imported CAD pipes included a number of duplicate overlapping pipes. BC used tools provided in InfoWater to review and delete unnecessary pipes.

Corrected Connectivity. The model software requires that pipes be broken at connections between water mains. Many locations were identified in the CAD data where pipes were not broken at connections. Many locations were also identified where pipe endpoints were drawn very close to each other but not snapped together. BC used tools provided in InfoWater to review all dead-end pipes that did not end at a hydrant. Pipes were split and endpoints were connected where appropriate.

3.2.2 Pipe Attributes

Pipe attributes were used for hydraulic calculations and/or management of model data. InfoWater uses the Hazen-Williams equation to determine friction-related headloss. The roughness factor (C-Factor) used in the equation is assumed for each pipe based on pipe material, lining, and age (if known). Lower factors equate to higher headloss. Pipes in the model were assigned C-Factors taken from industry standards. Table 3 summarizes how the attributes are used in the model.

<table>
<tr><th colspan="5">Table 3. Pipe Attributes</th></tr>
<tr><th>Attribute</th><th colspan="4">Value</th></tr>
<tr><td>ID</td><td colspan="4"><table><tr><td>Prefix
P
CV (check valve)
GV (gate valve)</td><td>Unique suffix
A unique number
A unique description</td><td>Sample ID
P6609
CV_PARKWAY
GV_RIDGEGATE</td></tr></table></td></tr>
<tr><td>Length</td><td colspan="4">Calculated in the model based on the actual GIS length of the pipe</td></tr>
<tr><td>Diameter</td><td colspan="4">Inside diameter from the CAD</td></tr>
<tr><td>Material</td><td colspan="4">Pipe material from the CAD</td></tr>
<tr><td rowspan="7">Roughness</td><td>Material</td><td>C Factor</td><td>Source</td><td>Notes</td></tr>
<tr><td>Default/blank/other</td><td>130</td><td>Assumed</td><td>Pipes with unspecified pipe material were assigned a roughness factor that is typical for ductile iron (DI) pipe. The CAD data indicate that asbestos cement and DI are the most common materials in the system.</td></tr>
<tr><td>Asbestos cement</td><td>140</td><td>Linsley, Lindeburg</td><td>Values are only given for clean pipe in the M32 manual</td></tr>
<tr><td>Cast iron</td><td>130 (New)
120 (5 years old)
100 (20 years old)</td><td>Linsley, Lindeburg</td><td>Interviews with the City indicated most of the cast iron pipe was installed 20 or more years ago. In the absence of more detailed information, a value of 100 was used for all cast iron pipe.</td></tr>
<tr><td>Concrete cylinder</td><td>130</td><td>Linsley, Lindeburg</td><td></td></tr>
<tr><td>DI</td><td>130</td><td>Linsley</td><td>Not listed in American Water Works Association's (AWWA) M32 manual</td></tr>
<tr><td>Poly-vinyl chloride/C900</td><td>140</td><td>AWWA, InfoWater</td><td>Using lower end of range of values to be conservative</td></tr>
<tr><td>Minor loss</td><td colspan="4">Set to 0 unless a valve or other facility causes a known headloss at a specific location. The C Factor is more appropriate to account for losses due to bends and fittings because it accounts for losses based on the length of a pipe. If a minor loss is used, it causes the same headloss for short and long pipes.</td></tr>
<tr><td>Check valve</td><td colspan="4">Set to Yes if there is a check valve on a pipe.</td></tr>
<tr><td>Zone</td><td colspan="4">Set to the pressure zone the pipe is a part of.</td></tr>
</table>

3.3 Tanks

Tank information from the City's as-built drawings was used for tank attributes. Table 4 lists the model's tank attributes.

Table 4. Tank Attributes			
Attribute	Value		
ID	*Prefix* T-	*Unique suffix* Tank description	*Sample ID* T-WEBSTER_1MG
Type	Set to Cylindrical for all tanks		
Elevation	The elevation of the bottom of the tank		
Minimum level	The minimum depth of water in the tank to which the tank can physically drain; set to 0 if unknown. Minimum water levels controlled by a pump or valve were set by adding controls to the pump or valve.		
Maximum level	The maximum possible depth of water in the tank, set as the depth from the bottom of the tank to the tank overflow or the tank roof (if overflow depth was not available).		
Initial level	Set to an average depth of water in the tank at the start of a day. This value was based on staff interviews.		
Diameter	The tank diameter		

3.4 Supply Points

The supply connection from North Clackamas County Water Commission (NCCWC) and Oak Lodge Water District (OLWD) were modeled as fixed-head reservoirs with valves to control the flow. The NCCWC connection is the primary source of supply to the City and is delivered through a dedicated pipe from the NCCWC treatment plant. The conditions upstream of the connection were represented with a general purpose valve with a headloss-flow curve developed from field investigation and production records. Table 5 lists the model's supply point attributes.

Table 5. Supply Point Attributes			
Attribute	Value		
ID	*Prefix* RES-	*Unique suffix* Supply description	*Sample ID* RES-NCCWC
Type	Set to Fixed Head		
Head	The hydraulic grade line of a supply point		

3.5 Pumps

All pumps were included in the model. The pump curves were developed from City records and entered into the model using the multipoint curve option. Table 6 lists the model's pump attributes.

Table 6. Pumps				
Attribute	Value			
ID	*Prefix* BP- (booster pump)	*Unique suffix* Description of the facility	*Suffix* Pump number	*Sample ID* BP-WEBSTER_1
Type	Multiple point curve - the most accurate representation of a pump, used when a pump curve is available			
Elevation	Pump elevation from DEM			

3.6 Valves

The City's water distribution system includes isolation and tank altitude valves, and pressure-reducing valves (PRVs). Isolation valves were modeled by adding controls to pipes in the model (i.e., by opening or closing a pipe). The tank altitude valves were represented as PRVs in the model. A PRV also was used to simulate the variable-speed Webster pumps. Table 7 lists the model's valve attributes.

Table 7. Valves			
Attribute	Value		
ID	*Prefix* AV- (altitude valve) MMS- (master meter at NCCWC) PRV	*Unique suffix* A description of the facility or a unique number	*Sample ID* AV-KIRKWOOD MMS-NCCWC PRV-HEREFORD
Type	PRV General purpose valve (GPV)		
Elevation	Valve elevation supplied by the City, otherwise set the elevation from the DEM		
Diameter	The diameter of the valve		
Setting	Settings were based on the information from the City and field tests. For PRVs this is the downstream pressure setting.		
Minor loss	Minor loss coefficient, K. InfoWater calculates the minor loss as k(V*2)/2g. This field is optional.		
Curve	Only used for MMS-NCCWC. A curve defining the headloss in feet as a function of the flow in gallons per minute.		

Section 4: Section 4: Model Demands

Accuracy of a model is highly dependent on the accuracy of the distribution of demands in the model. Two demand sets, maximum day demand (MDD) and average day demand (ADD) were developed for both the existing and future systems. The different methods used for allocating those demands to the model nodes are described below followed by a discussion on the fire flow demand allocation method.

4.1 Existing System Demand Allocation

Existing system demand allocation consists of appropriately distributing the total system demand in the computer model of the water system. Total existing system demand was calculated using the City's water billing data and daily water meter data from OLWD and NCCWC. Demands were assigned to nodes referred to as demand junctions in the computer model. Demand junctions were designated as all nodes not located on dedicated transmission piping or near pump stations and storage tanks.

InfoWater tools were used to assign the geocoded customer demands to the closest demand junction. The following steps summarize the demand allocation process that was followed for this project:

1. Obtain billing data (including location) for each customer and calculate the MDD and ADD for each customer.
2. Geocode (locate geographically) each customer by matching the customer to a parcel, street address, or global positioning system point.
3. Flag each junction in the model as a demand or non-demand junction. Non-demand junctions include transmission pipelines or pump stations.
4. Calculate the total demand at each demand junction as the sum of the demand for the customers closest to each junction.

4.2 Future System Demand Allocation

The total future demand was calculated using the projected population growth. The existing demands were scaled by the population growth rate.

4.3 Fire Flow Demand Allocation

Fire flow demands were calculated and assigned to the closest model hydrant nodes.

Section 5: Quality Assurance Protocols

In addition to daily input on the modeling work, senior level engineering staff provided detailed quality control reviews at four pre-established milestones in the computer modeling process. The review performed at each of the milestones is listed below.

1. Model Build - This review was performed upon completion of building the model and loading the demands into the model. It included a review of the input data (e.g., facility information, elevations, controls) and demand allocation.
2. Calibration - This review was performed upon completion of the model calibration. It included a review of the calibration results and the modifications made to the model to achieve those results.
3. System Evaluation - This review was performed upon completion of the existing and future system evaluation. It included a review of the MDD, ADD, and fire flow evaluations to verify that criteria established to evaluate the system were used appropriately.
4. Capital Improvement Plan (CIP) Development - This review was performed after modeling work to develop capital improvement projects was completed and prior to completion of the CIP. During this review, each project was scrutinized to verify that the evaluation criteria were satisfied and that there were no undesirable ancillary outcomes from the projects (e.g., unmanageable water age).

References

American Water Works Association, *Computer Modeling of Water Distribution Systems, M32*, Third Edition, AWWA, Denver, 2012, pp. 33.

Innovyze, *InfoWater Help*, 2012.

Lindeburg, *Civil Engineering Reference Manual for the PE Exam*, Eighth Edition, Professional Publications, Inc., Belmont, CA, 2001, pp. A-25.

Linsley, R. K. and Franzini, J. B., *Water Resources and Environmental Engineering*, Third Edition, McGraw-Hill Book Company, 1979, pp. 281.

Ray, R., Moore, P.B., Harrington, D.A, and Hauffen, P. M. 2008. The Achilles' Heel of GIS-built Hydraulic Models: Maintaining/Updating a Model from GIS Data. In *AWWA Conference Proceedings*. June 2008

Appendix B: Calibration Test Plan

Brown AND Caldwell

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Technical Memorandum

Prepared for: City of Gladstone

Project Title: Gladstone Stormwater and Water Master Plan

Project No: 143454

Technical Memorandum

Subject: Water Model Calibration Test Plan, Subtask 4.4

Date: April 14, 2014

To: Scott Tabor, City of Gladstone

From: Jim Harper, Brown and Caldwell

cc: Janice Keeley, Brown and Caldwell

Prepared by: Colin Ricks, Brown and Caldwell

Reviewed by: Shem Liechty, Brown and Caldwell

Section 1: Introduction

This technical memorandum (TM) describes the methods for gathering the information required to calibrate the City of Gladstone's (City) water distribution system model that Brown and Caldwell (BC) is creating. The data gathered will be compared to model results to verify that the model is well-calibrated. The effort will include operational data gathering and hydrant flow tests performed over 1 to 2 days.

Section 2: Operations Data Gathering

Operations data are used in the calibration to verify that facility controls and settings have been represented appropriately in the model. These data will need to be recorded during the field tests. The operational data needed to verify model calibration include the following:

- *Pump Discharge Flow Rate, Suction Pressure, and Discharge Pressure*. Pumps that are off before the test should remain off during the test. Pumps that are on before the test should remain on during the test. The pump discharge rates before and during the tests should be recorded at 1- to 2-minute intervals. Suction and discharge pressures at the pump also should be recorded.
- *Tank water levels*. Tank water levels should be obtained from the tank circle chart recorders. Copies of the circle charts for all days of field testing should be provided to BC.
- *Pressure Relief Valve (PRV) Settings*. PRV settings should be obtained from City staff or measured in the field.
- *Pressure Logger Data*. Pressure loggers should be installed at the locations described in Section 4 of this TM for the selected data gathering period. The pressure logger ID used at each location should be noted so that BC can calibrate the pressure loggers after field testing is complete. The pressure loggers should be set to record pressure at a 1- to 2-minute time step and should be set to record for the duration of the hydrant flow tests.
- *Master Meter Flow Rates*. BC understands that the City is supplied by several interconnects with neighboring water utilities. Any flow through interconnects must be accounted for in the calibration process. It is very important to manage and/or monitor the flow through interconnects during the calibration data gathering period following the approaches listed below.
  - Close all interconnects that do not have a contractual or hydraulic requirement to leave them open. These interconnects should be closed to prevent flow transfers during the hydrant flow tests.
  - Interconnects that must be left open should be monitored during the testing by sending a City operations and maintenance staff member to measure and record the flow rate. The flow rate through the master meter should be recorded every 1 minute during the hydrant tests. The master meter can be recorded every 10 minutes between hydrant tests.

Section 3: Field Tests

Field test data are used to verify that system hydraulics have been represented correctly in the model. The required personnel, equipment, and operations data and the test procedures for the field tests are described below. It is expected that all field testing will be complete within a 2-day time period, as described in Section 5 of this TM.

Personnel

One representative from BC will be present to coordinate the calibration testing and to help collect and record test data. At least two City staff members are needed to escort BC staff, assist with data collection, and operate hydrants, pumps, etc.

Preparation and Necessary Equipment

Some preparation of equipment for hydrant testing will be required of both BC and the City. Table 3-1 lists the equipment needed for the calibration testing. Equipment will be checked prior to the day of testing to verify that it is functional and/or accurate. Watches used to record the time of each test should be synchronized to ensure that the test data can be correlated accurately. The City will be responsible for providing transportation of City staff and equipment to each test location.

Table 3-1. Required Equipment for Calibration Testing

Item	Quantity	Provided by
Hydrant key	2	City
Valve wrench	2	City
Flow-metering hydrant flow diffuser	2	City
Radios	3	City
Hydrant cap with 1/4-inch threaded tap (for pressure gauges/logger)	5	BC
Crescent wrench sets	2	BC
Digital camera	1	BC
Watch	2	BC
Calibrated 200 pounds per square inch (psi) pressure gauge	4	BC
Hose bib connection for pressure gauge/logger	4	BC

Collection of Operations Data

The operations data described in Section 2 of this TM should be gathered for the days of field testing. This will make it possible to match the operational conditions in the model to the system operations at the time of each test.

Test Procedures

Hydrant flow tests and site inspections of the storage and pump facilities will be performed during the calibration testing visit. Each test and inspection should follow the procedures described below. The testing is expected to take approximately 1 to 2 days. All data and comments should be recorded on the forms provided by BC. During the testing period, any valves in the system that are known or suspected to be closed should be reported to the BC representative as along with any pipe breaks or other water system emergency.

Important note: *The pressure loggers installed for the operations data gathering should not be removed until after all hydrant flow tests are completed. The pressure loggers will record valuable information during the tests.*

Hydrant Flow Tests

The objective of hydrant flow tests is to obtain instantaneous flow and pressure data at various locations throughout the distribution system. Up to four flow tests will be performed: a minimum of three tests with one additional optional test. The flow tests must stress the distribution system so that the calibration data will reflect the system's reactions to a range of operating conditions.

To accomplish this, water is released during each test from one or more hydrants until a minimum pressure drop of 5 psi (10 psi desired) is experienced at the test location. (Note: these tests are not the same as hydrant tests performed by the fire department to determine available flow from a hydrant.) Step-by-step instructions for setting up the hydrant flow tests are listed in the attached field forms.

The test coordinator will instruct the person monitoring the master meter to begin recording the flow rate every 1 minute for the duration of the flow test.

Storage/Pump Facility Inspections

Each storage/pump facility will be inspected to test pump performance and review equipment condition and strategy for pumping and tank fill controls. Site conditions will be documented with photos and the pump and tank name plate information and controls will be recorded on the appropriate forms. Pump performance will be tested at each pump at each facility using the following procedure:

Step 1. Verify that the pump is on.

Step 2. Attach a pressure gauge on the discharge side (and suction side if possible) and record the pressure.

Step 3. Record the time.

Step 4. Collect flow from the supervisory control and data acquisition software system or a flow meter for the pump at the time the pressure is recorded.

Step 5. If the pump has a variable-frequency drive, record the pump speed.

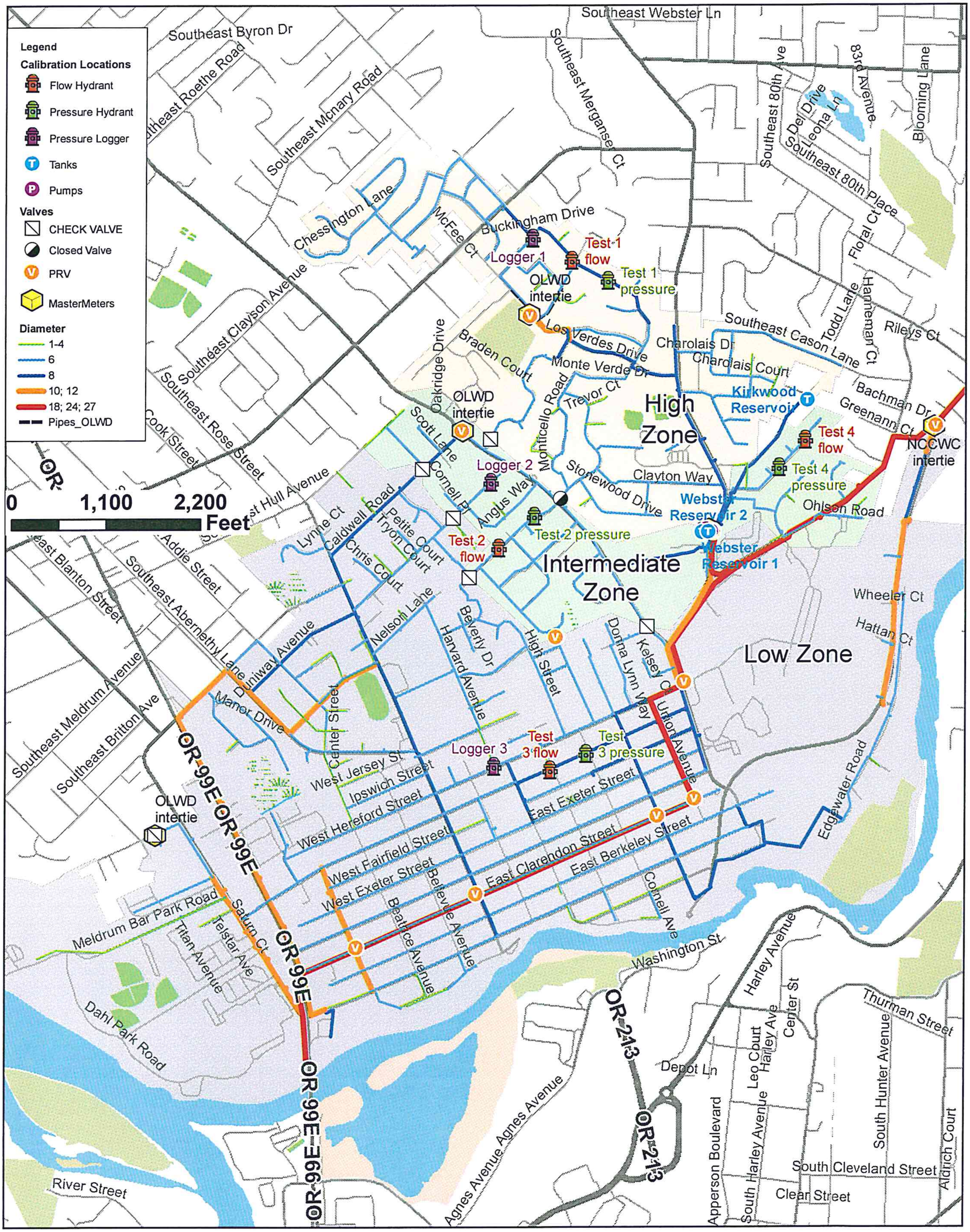
Section 4: Test Locations

This section specifies the flow test locations and the locations where the pressure loggers should be installed and the hydrant tests performed. Up to four flow tests will be performed throughout the system. Tests 1 through 3 must be completed. If time permits, test 4 will be performed also.

Figure 4-1 shows an overall view of flow tests and the pressure logger locations. Figures 4-2, 4-3, 4-4, and 4-5 show detailed views of each site. Tables 4-1 and 4-2 list the approximate addresses for the loggers and tests.

Table 4-1. Pressure Logger Approximate Locations

Logger	Address
Logger 1	Buckingham Drive/Lancaster Drive
Logger 2	6830 Glen Echo Avenue
Logger 3	270 East Hereford Street

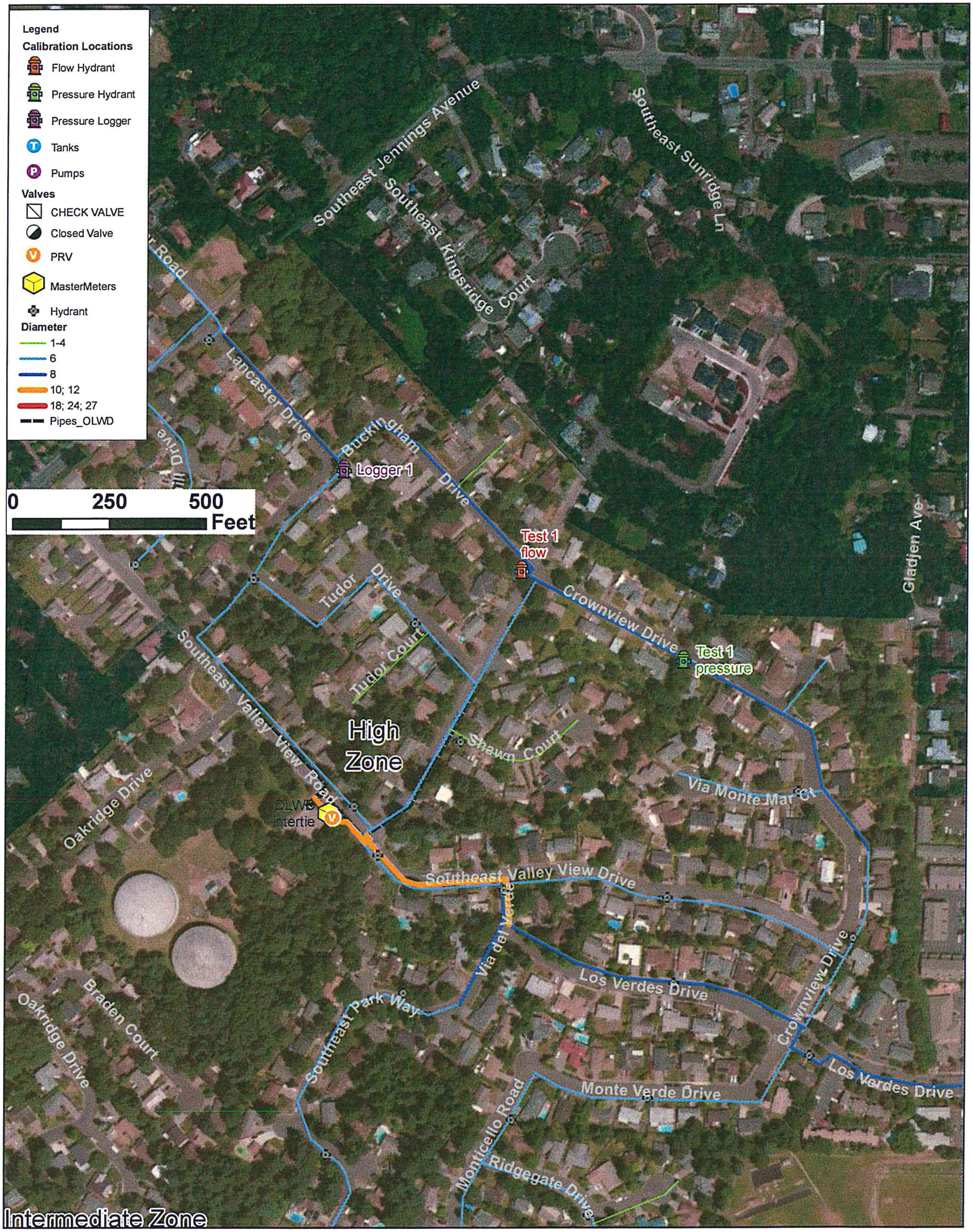


Brown AND Caldwell

City of Gladstone Water System Master Plan

Calibration Locations

Figure 4-1. Hydrant test and logger locations



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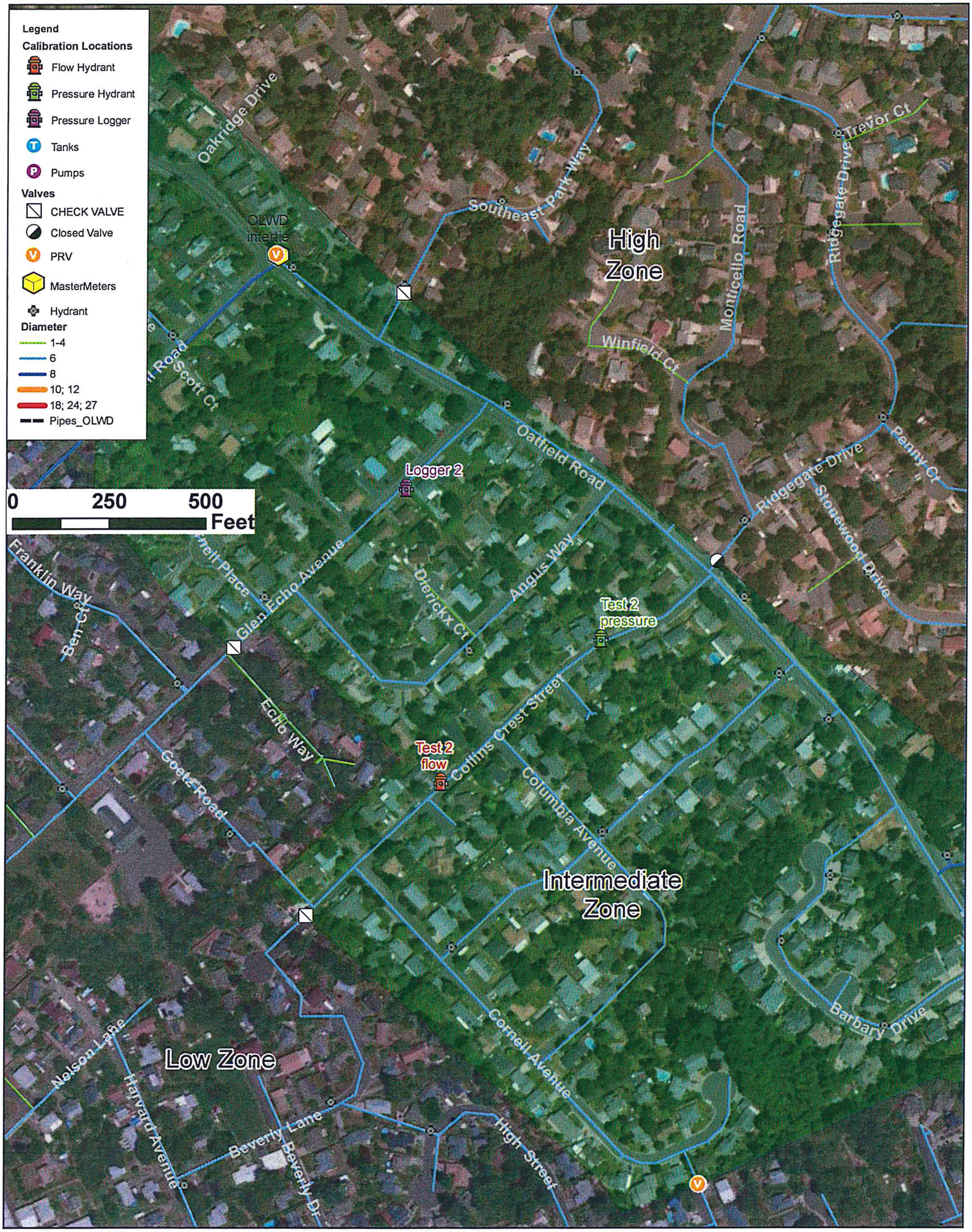
City of Gladstone Water System Master Plan

Calibration Locations

Figure 4-2. Hydrant test 1 and Logger 1



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Calibration Locations

Figure 4-3. Hydrant test 2 and Logger 2

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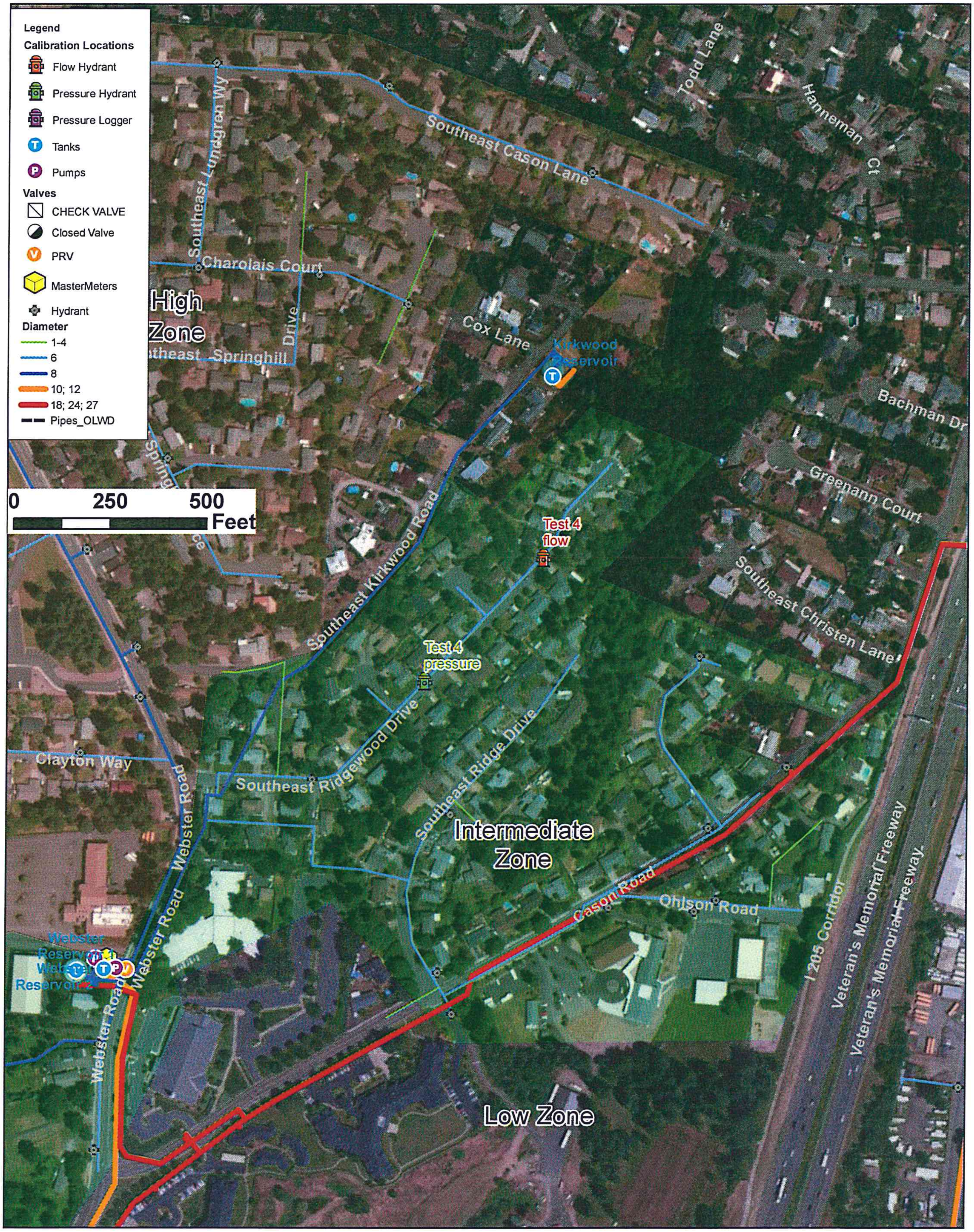
City of Gladstone Water System Master Plan

Calibration Locations

Figure 4-4. Hydrant test 3 and Logger 3



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City of Gladstone Water System Master Plan

Calibration Locations

Figure 4-5. Hydrant test 4 (optional)

Table 4-2. Test Hydrant Approximate Locations		
Test	Flow hydrant address	Pressure hydrant address
Flow Test 1	16790 Buckingham Drive	17299 Crownview Drive
Flow Test 2	585 Collins Crest Street	663 Collins Crest Street
Flow Test 3	377 E Gloucester Street	482 E Gloucester Street
Flow Test 4	7599 Ridgewood Drive	7615 Ridgewood Drive

Hydrant test flows may cause flooding or erosion damage. City staff should check the hydrant flow test locations prior to the day of testing to verify that there is little potential for flooding or erosion damage at each site. If any of the locations are found to be unsuitable or inoperable during field inspection or calibration testing, an alternate site will be selected and documented with approval of a BC representative.

Section 5: Testing Schedule

Pressure logger data should be gathered for the duration of all hydrant tests. The field tests will be performed in April 2014 according to the following schedule:

Day 1 – April 21

8:30 a.m. – Meet with City staff to coordinate testing and document control strategy.

10:00 a.m. – Inspect Webster tanks, Kirkwood tank, and pump stations.

11:30 a.m. – Install pressure loggers 1 through 3.

12:30 p.m. – Lunch

1:00 p.m. – Conduct hydrant flow tests.

Day 2 – April 22

8:30 a.m. – Meet with city staff to coordinate testing.

9:30 a.m. – Complete remaining hydrant flow tests and remove pressure loggers when finished.

FORMS

Gladstone Water System Testing

Hydrant Test

Brown AND Caldwell

Collected By ______________________________

Zone / Test No.	Pressure Readings				Flow Readings			Notes
	Gauge ID(s)	Reading	Date / Time	Pressure (psi)	Diff-user	Gauge ID	Pressure (psi)	
		Before:			1			
		During:			2			
		After:			3			
		Before:			1			
		During:			2			
		After:			3			
		Before:			1			
		During:			2			
		After:			3			
		Before:			1			
		During:			2			
		After:			3			
		Before:			1			
		During:			2			
		After:			3			

Step 1. Confirm that SCADA is operating and recording data at each of the required sites.

Step 2. Attach a pressure gauge to the residual hydrant. Use the bleeder valve on the hydrant cap assembly to release air and protect the gauge while opening the hydrant.

Step 3. Attach the hydrant diffuser to the flow hydrant.

Step 4. Record the static pressure at the residual hydrant and the time of test.

Step 5. Instruct to start opening the flow hydrant **SLOWLY** until get minimum 5 psi (10 psi if possible) drop at the residual hydrant. If cannot get sufficient pressure drop, turn the flow hydrant off **SLOWLY**, add another diffuser to the other hydrant nozzle or a nearby hydrant (record which hydrant is used), and re-start the test at step 4.

Step 6. When the pressure at the residual hydrant stabilizes (usually 3-5 minutes), record the time and residual pressure and signal the flow hydrant operator to record the flow.

Step 7. Instruct the flow hydrant to be closed **SLOWLY**.

Step 8. Record the static pressure again at the residual hydrant.

Step 9. Remove the equipment. **Be sure to open the bleeder valve on the pressure gauge hydrant cap assembly while closing the hydrant to avoid drawing a negative pressure on the gauge.**

Gladstone Water System Testing

Tanks

Brown AND Caldwell

Collected By ______________________ Date ______________

Item	Tank 1	Tank 2
Tank Name		
Location		
Type and Shape (Elevated, Ground)		
Material		
Volume		
Diameter / Dimensions		
Floor Elevation		
Height		
Overflow Height		
Fill (e.g. from PS)		
Feeds (e.g. to System)		
Diagram		

Gladstone Water System Testing

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Pump Test

Collected By ____

Pump Name	Date/Time	Flow (gpm)	Upstream Pressure (psi) /Tank Elev (ft)	Downstream Pressure (psi)	Elev Diff from Up/Down Pressures (ft)	Notes

Step 1. Attach pressure gauge(s) on the discharge side and the suction side (if a booster pump) of each pump. Note: If there are gauges on the pump, remove them and use the testing gauges if possible. The vibration of the pumps can cause the gauges on the pumps to be inaccurate.

Step 2. Record the difference in elevation between the two gauges.

Step 3. Record the time, flow rate, and pressures before, during and after operating the pump. The pump should be operated long enough that readings stabilize (at least 5 minutes).

Gladstone Water System Testing

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Pump Station Information

Pump Station Name / Location ______________________________

Collected By ______________________________ Date ______________________________

Item	Pump 1	Pump 2	Pump 3	Pump 4	Pump 5
Pump Name / ID					
Type					
Speed (Constant / VFD)					
Design Head (ft)					
Design Flow (gpm)					
Horsepower					
Number of Stages					
Impeller Diameter (in)					
Manufacturer					
Serial Number					
Model Number / Type / Size					
Pump Operation (Lead / Lag / Standby)					
How Pump Controlled (Tank Level, etc.)					
On / Off Settings					

Notes / Diagrams (Use back of sheet for additional notes):

Gladstone Water System Testing

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PRVs

PRV Name / Location ______________________________

Collected By ______________________________ Date ______________

Category	Item	Value	Notes
Traffic control	Requires traffic control (Y/N)		
Vault cover	Requires crane (Y/N)		
	Hatch is accessible (Y/N)		
Vault Interior	PRV is accessible (Y/N)		
	Vault is Flooded (Y/N)		
	Vault filled with debris (Y/N)		
PRV	PRV is Operational (Y/N)		
	Flow is going through PRV (Y/N)		
	Taps are Accessible		
	Describe needed fittings for pressure gauge		
Pressure at PRV	Upstream Pressure (psi)		
	Downstream Pressure (psi)		
Exterior	Locate nearby hydrants and valves for testing PRV, mark on map		
Photos	Area around vault		
	Vault cover		
	Vault Interior		
	PRV		

Step 1. Do a condition assessment on the PRV. See if the PRV is accessible, pressures can be read at the PRV, etc.

Step 2. If the PRV is active but pressures cannot be read (e.g. vault needs cleaned out), clean vault and/or obtain with equipment needed to read pressures. Make a return trip and take pressures.

Notes:

A. To verify if flow is going through the PRV (note that a pressure differential across the PRV does not mean that flow is going through the PRV):

Option 1 – Flow already going through PRV. At times a humming noise through the PRV signifies flow through the PRV.

Option 2 – Attach flow diffuser. If there is no flow through the PRV or if you are unsure, place a flow diffuser at a downstream hydrant (mark hydrant on map) and turn on the hydrant **SLOWLY**.

B. If a PRV vault has 2 PRVs, the 2 PRVs may have different settings for low and high flows. If possible, obtain both settings.
For example, if flow is going through a smaller PRV (for lower flows), record that pressure and then add a diffuser to get flow through the larger PRV.

Gladstone Water System Testing

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Choked Valves

Collected By ____

Date / Time	Location / Description	Valve Type (butteryfly, gate, etc. if known)	Amount Open (Number Turns, %, etc.)	Notes

Step 1. Locate the choked valve and mark the location on a map.

Step 2. If possible, find out how many turns the valve is closed out of the total number of turns.

Gladstone Water System Testing

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Pressure Loggers

Collected By ____

Logger ID	Location / Description	Date / Time Installed	Date / Time Removed	Notes

Step 1. Locate the hydrant identified for pressure logger installation from maps.

Step 2. Flush the hydrant, and then install a pressure **gauge** mounted on a hydrant cap.

Step 3. Open the hydrant nut **completely**, then read and record the pressure. Confirm that the recorded pressure is not close to or over the pressure rating of the pressure logger.

Step 4. After opening the bleeder valve on the pressure gauge/hydrant cap assembly (this prevents drawing a negative pressure which will ruin the gauge), close the hydrant nut and replace the pressure gauge/hydrant cap assembly with a pressure **logger** mounted on a hydrant cap.

Step 5. Open the hydrant nut **completely** with the bleeder valve on the pressure logger/hydrant cap assembly open to allow the release of air from the hydrant. Check to see that there are no leaks from the hydrant or the ground around the hydrant.

Step 6. Connect computer to the logger to confirm that the logger is sensing and logging data.

Step 7. Record the address, hydrant ID, and mark on the map the location of the logger.

Step 8. Before removing the logger, connect the computer to the logger to confirm that data has been recording. Data download can be performed at this point or postponed until later.

Step 9. While closing the hydrant nut to remove the logger, leave the bleeder valve on the pressure logger/hydrant cap assembly open to avoid ruining the loggers by drawing a negative pressure.

Step 10. Check to see that all hydrant caps are replaced and the hydrant nut is fully closed prior to leaving the test site.

Gladstone Water System Testing

Brown AND Caldwell

Notes

Collected By ______________________________

Appendix C: Detailed System Map

Appendix D: IGAs

**INTERGOVENMENTAL
COOPERATIVE AGREEMENT**

This agreement is made and entered into by and between the City of Gladstone, Oregon, an Oregon municipal corporation, hereinafter referred to as "City"; and the Oak Lodge Water District, a domestic water supply district created pursuant to ORS Chapter 264, hereinafter referred to as "District".

WITNESSETH:

RECITALS

1. In 1990, the parties hereto entered into an intergovernmental cooperative agreement to provide for the construction and maintenance of a water system interties between the City's water supply system and the water supply system of the District. The purposes of that intertie in Valley View Road are to provide emergency water supply between the two systems of the District and City and an on-demand supplemental source of water to the City's high level service zone from the District as hereinafter described in this agreement.

2. In 1998 the parties subsequently amended that agreement to provide for construction and maintenance of a second water system intertie in Oatfield Road to provide supplemental water service to the City's intermediate level service zone.

3. Purposes of this current agreement are to incorporate the intent of earlier agreements, to provide for construction and maintenance of a third water system intertie in Rinearson Road and to provide supplemental water service for fire suppression to a portion of the City's lower level service zone.

The parties acknowledge that they have authority to execute this cooperative intergovernmental agreement pursuant to the terms of ORS 190.010.

NOW, THEREFORE, the premises being in general as stated in the foregoing recital, it is agreed by and between the parties hereto as follows:

1. Emergency Condition Defined. An emergency condition is considered to be an occurrence created by a physical failure of facilities, fire suppression activities or premeditated shutdown of water supply facilities whereby insufficient supply to water customers of either party would threaten the health or safety of those customers. Such emergency condition includes failure of water supply transmission pipelines.

2. On-Demand Condition Defined. An on-demand condition is considered to be an occurrence which results in a decrease in the water pressure normally present. Such decrease in pressure below a predetermined level will result in the utilization of a pressure regulating facility, through which District water will flow, augmenting the City's water supply.

3. Location. The location of the water system interties between City and District are-in the vicinity of the intersections of: Valley View Drive and Valley View Road; Oatfield Road and Caldwell Road; and in Rineason Road about 500 feet west of River Road.

4. Cost, Construction Maintenance and Ownership. Interties in Valley View Road and Oatfield Road exist at the time of this agreement and therefore no further installation cost is anticipated. For the intertie in Rinearson Road, the District agrees to extend, maintain and own a 6" diameter water main from the northerly to the southerly side of River Road terminating in a vault recently installed by the City. The City agrees to install, maintain and own water appurtenances in the vault including a meter to measure water consumption, a pressure valve and 6" diameter water piping extending from the vault to a proposed 12" diameter water main that will extend in River Road from Rinearson Road to River Road's intersection with McLoughlin Blvd.

5. Quantity of Water Supplied in Emergency Conditions. The party supplying water during an emergency condition as defined herein shall endeavor to supply the maximum quantity of water to the other and take all reasonable actions necessary to accomplish the same so long as such actions are consistent with minimum standards for the operation of its own internal water system.

6. Quantity of Water Supplied in On-Demand Conditions. The party supplying water during an on-demand condition as defined herein shall endeavor to supply the maximum quantity of water to the other and take all reasonable actions necessary to accomplish the same so long as such actions are consistent with minimum standards for the operation of its own internal water system.

7. Cost of Water Provided. District agrees to pay monthly to City for all water provided to District through the interties at the same wholesale rate charged by the North Clackamas County Water Commission (NCCWC) per 100 cubic feet plus an additional charge of 5¢ per 100 cubic feet to cover the cost of pumping. City agrees to pay monthly to the District for water provided to City at the same wholesale rate charged by NCCWC per 100 cubic feet plus an additional charge of 5¢ for inteties in Oatfield Road and Rineason Road, and an additional charge of 15¢ per 100 cubic feet for the intertie in Valley View Road to cover the cost of the existing in-place facilities including reservoirs owned and operated by District which allow the District to provide surplus water to the City high level system at a volume and pressure greater than can be provided by existing City facilities. The volume of water delivered to City from District shall be calculated by District through the metered interconnection between the parties.

8. Amendment Provisions. The terms of this agreement may be amended by mutual agreement of the parties. Any amendments shall be in writing and shall refer specifically to this agreement, and shall be executed by the parties.

9. Prior Agreements. This agreement shall replace and supersede the previous agreements between the parties referred to in the recitals.

10. Termination of Agreement. This agreement shall continue in effect until terminated by City or District with written notice of such intent to terminate provided to the other party. Notice to terminate must be provided by July 1 of any year, with termination effective January 1 of the succeeding year.

11. Written Notice Addresses. All written notices required under this agreement shall be sent to:

Oak Lodge Water District	Manager Oak Lodge Water District 14496 SE River Road Milwaukie, OR 97222
Gladstone:	City Administrator City of Gladstone 525 Portland Avenue Gladstone, OR 97027

IN WITNESS WHEREOF, the parties have set their hands and affixed their seals as of the date and year hereinabove written.

Oak Lodge Water District has acted in this matter pursuant to Resolution No. _____, adopted by its Board of Commissioners on the ________ day of __________________, 2007.

Adopted by the Gladstone City Council on the 8th day of May 2007.

Oak Lodge Water District
by and through its District Board

By: ______________________________
Chairman

By: ______________________________
Secretary

Approved as to form: By: ______________________________
District Counsel

City of Gladstone,
by and through its City Officials

By: ______________________________
Mayor

By: ______________________________
City Recorder

WATER SERVICE AGREEMENT

THIS AGREEMENT, made and entered into this 14th day of November, 1994, by and between OAK LODGE WATER DISTRICT, a domestic water supply district created pursuant to ORS Chapter 264, hereinafter referred to as Oak Lodge, and CITY OF GLADSTONE, an Oregon municipal corporation, hereinafter referred to as Gladstone.

W I T N E S S E T H:

WHEREAS, the parties to this agreement, Oak Lodge and Gladstone, were each formed pursuant to Oregon Revised Statutes for the purpose, among others, of providing domestic water service to water users within their respective boundaries; and

WHEREAS, presently there are few properties located within the boundaries of Oak Lodge and Gladstone for which the other party currently provides domestic water and these said properties receive the benefit of the "inside user" rate; and

WHEREAS, currently the majority of said properties are within the boundaries of Gladstone; and

WHEREAS, in accordance with the present rate charges of Oak Lodge and Gladstone, unless specific agreement is entered into between said parties, it may be necessary for said parties to incur unnecessary construction expense or for owners of said properties to be charged an "outside" user rate, and it is the desire of said parties to continue existing cooperation so an "inside" water rate applies to said properties and as a result thereof; and

WHEREAS, Oak Lodge and Gladstone have authority to execute this intergovernmental agreement pursuant to the terms of ORS 190.010 to provide for appropriate understanding, resolution and agreement of matters of mutual interest;

NOW, THEREFORE, said parties mutually agree as follows:

1. Oak Lodge and Gladstone will continue to provide domestic water service to properties specifically designated in Exhibit A attached hereto and by this reference made a part hereof, even though said properties are located within the boundaries of the other, to-wit: Oak Lodge and Gladstone, respectively.

2. In recognition of the community, cooperation and other practical considerations, each party hereto agrees that they will charge said properties for providing domestic water service at an inside user rate the same as those charges against other like properties within their own boundaries.

3. Gladstone will continue to read water meters to determine the amount of water provided to said properties as shown by Exhibit A and bill customers of said properties. Oak Lodge will continue to read meters for those said properties in Gladstone where water is provided by Oak Lodge and bill Gladstone; otherwise Gladstone will send meter data to Oak Lodge and Oak Lodge will bill Gladstone.

4. This agreement repeals a memorandum of understanding entered into on the 1st day on May, 1978 between Oak Lodge and Gladstone; this agreement shall have no affect upon an intergovernmental cooperative agreement for distribution system interties presently in effect between the parties hereto.

5. No additional properties may be made subject to nor receive the benefits of this agreement without mutual written consent by the General Manager of Oak Lodge and City Administrator of Gladstone.

6. This agreement may be cancelled or terminated without cause by either party hereto upon the giving of a notice in writing to the other 180 days in advance of the desired termination date.

7. This contract shall be in full force and effect from and after the 14th day of November, 1994.

IN WITNESS WHEREOF, the parties hereto do each authorize their elected and appointed officials to execute this Agreement in duplicate as of the 14th day of November, 1994.

OAK LODGE WATER DISTRICT

By: Allen F. Hart
President of Board

By: Lester T. Larson
Secretary of Board

Oak Lodge Water District
14496 S. E. River Road
Milwaukie, OR 97222

CITY OF GLADSTONE

By: Wade Byers
Mayor

By: Verna Howell, CMC
City Recorder

City of Gladstone
525 Portland Avenue
Gladstone, OR 97027

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Exhibit A

Properties in Gladstone and where water is provided by Oak Lodge

Address	Oak Lodge Acct. #	Gladstone Acct. #
18105 Hardway Court	560072	1606340
18115 Hardway Court	560073	1606350
18125 Hardway Court	560074	1606360
18120 Hardway Court	560075	1606370
18110 Hardway Court	560076	1606380
18100 Hardway Court	560077	1606390
17651 Oatfield Road	560840	1702450
17707 Oatfield Road	560850	1702500
17711 Oatfield Road	560860	1702550
17717 Oatfield Road	560870	1702600
5306 Rinearson Road	400990	1802950
5202 Rinearson Road	400985	1803000
16455 Ormae Road*		
16465 Ormae Road*		

*Assigned addresses for future lots approved by Gladstone Planning Commission, File No. PART-94-4.

Properties in Oak Lodge where water is provided by Gladstone

18221 S. E. Portland Avenue	N/A	1607410

ADDENDUM No. 1 TO THE SECOND AMENDED INTERGOVERNMENTAL AGREEMENT FOR THE NORTH CLACKAMAS COUNTY WATER COMMISSION TO ADD THE CITY OF GLADSTONE AS A MEMBER OF THE COMMISSION

Dated ~~June~~ July 18, 2005

Agreement Between

The Sunrise Water Authority,

The Oak Lodge Water District,

And

The City of Gladstone

ADDENDUM No. 1
TO THE SECOND AMENDED INTERGOVERNMENTAL AGREEMENT FOR THE NORTH CLACKAMAS COUNTY WATER COMMISSION TO ADD THE CITY OF GLADSTONE AS A MEMBER OF THE COMMISSION

This INTERGOVERNMENTAL AGREEMENT is made this ___ day of June, 2005, by and between the Sunrise Water Authority ("Sunrise"), the Oak Lodge Water District ("Oak Lodge") and the City of Gladstone ("Gladstone"), hereinafter collectively referred to as the Parties.

RECITALS:

WHEREAS, the North Clackamas County Water Commission ("NCCWC" or the "Commission") is an intergovernmental agency organized under ORS Chapter 190 by Oak Lodge and Sunrise to process water from the Clackamas River into safe, clean drinking water and to supply the treated water to members of the Commission; and

WHEREAS, Gladstone is an Oregon home rule city authorized by law to operate a municipal water supply system and to enter into intergovernmental agreements under ORS Chapter 190; and

WHEREAS, the Parties desire that Gladstone become a member of the NCCWC;

NOW THEREFORE, in consideration of the mutual covenants and agreements contained herein, the Sunrise Water Authority, the Oak Lodge Water District and the City of Gladstone agree as follows:

AGREEMENT

1. Addendum.

This Addendum No. 1 is to the Second Amended Intergovernmental Agreement between Oak Lodge and Sunrise, dated September 3, 2004, which is incorporated and referred to herein as the "Second Amended IGA". The City of Gladstone adopts and agrees to be bound by the terms of the Second Amended IGA as if they were fully set out in this Addendum. Unless modified in this Addendum No. 1, all terms and conditions in the Second Amended IGA shall remain in effect. Conflicts shall be resolved in favor of Addendum No. 1. This Addendum confers no rights or power to enforce its terms upon any other person or Party not specifically mentioned in either document except for the Sunrise Water Authority, the Oak Lodge Water District, the City of Gladstone or the North Clackamas County Water Commission.

2. Reconstituted Commission.

(a) The reconstituted Commission shall be composed of seven (7) members. Three (3) members shall be selected by the Board of Commissioners of Oak Lodge, three (3) members shall be selected by the Board of Commissioners of Sunrise and one (1) member shall be selected by the City Council of Gladstone.

(b) A Commissioner shall be a voting member of the governing body, council or board of commissioners of the Party making the selection.

(c) The Commission shall select a Chair from among its members to serve a term of one year beginning July 1st of each year. The position of Chair shall rotate each year to represent each Party beginning with Sunrise, then Oak Lodge and then Gladstone. The Commissioners shall also select a Vice Chair to serve in the absence of the Chair.

(d) Each Commissioner shall have one vote on any matter coming before the Commission. Five (5) Commissioners shall be present to meet the requirement for quorum of the Commission. Five (5) affirmative votes shall be needed to adopt any measure, ordinance or resolution.

3. Commission Assets.

(a) Each Party to this Addendum No. 1 shall have an undivided interest in the assets of the Commission in the following percentages: Sunrise – forty eight percent (48%), OLWD --forty two percent (42%), and, Gladstone – ten percent (10%).

(b) Gladstone shall pay $2.5 million in cash for its ten percent interest in the assets and liabilities of the Commission and the entitlement to an Allocation of 2.5 million gallons per day (MGD) of treated water from the Commission. $2.0 million shall be paid to Oak Lodge and $0.5 million shall be paid to Sunrise. In addition, Gladstone shall assign or transfer a total of 8.9 MGD of Clackamas River water rights. 5 MGD which includes water rights on the Clackamas River certificated at the time of this Addendum to the Commission and 3.9 MGD to Sunrise. Payment of money due from Gladstone under the terms of this Addendum shall be made within thirty (30) days of the receipt of proceeds from bonds sold to finance the purchase or by December 31, 2005, whichever first occurs.

(c) Gladstone shall support substitution of NCCWC for itself as a member of the Willamette Water Resources Commission (WWRC) and shall support NCCWC efforts to secure the largest possible access to Willamette River water rights.

4. Water Allocations.

(a) Each Party shall have a right to call upon the Commission to supply the Party with treated water up to the amount of its Allocation as provided in this paragraph. The Allocations of the Parties upon completion of the expansion of the NCCWC treatment plant referred to in section 4 of the Second Amended IGA are, at a minimum: Sunrise 12 MGD, Oak Lodge Water District 10.5 MGD and Gladstone 2.5 MGD. If the operating capacity of the expanded plant exceeds the designed capacity of 25 MGD, then Sunrise shall be entitled to call upon eighty percent (80%) and Oak Lodge twenty percent (20%) of the exceedance.

Oak Lodge and Gladstone agree to grant to Sunrise a first right to purchase any part of their Allocation not needed to serve their customers and ratepayers, at a price to be based upon cost of service at the time of purchase and without a premium. To facilitate planning, Oak Lodge and Gladstone will each provide Sunrise with a rolling five (5) year forecast of projected water use by January 1 of each year. By March 1 of each year, Sunrise will provide Oak Lodge and Gladstone with a firm commitment of its need for water from the Oak Lodge Allocation, for the five (5) year period. Sunrise may, but is not obligated to, purchase such excess Allocation from Oak Lodge or Gladstone before using (a) groundwater from wells in use listed in paragraph 6.02 B. of the Second Amended IGA; (b) water from Clackamas River Water contract dated March 8, 2001; and (c) water as may be authorized for use under permit application No. S-74056 now pending before the Water Resources Department.

5. Use of the Facilities and Improvements of Other Commission Members.

(a) Each Party shall make its facilities available for the use of every other Party to this Addendum and to the Commission to the extent such facilities are not presently required to serve the customers and ratepayers of the party that owns them. The owner of the facility shall be entitled to a fee for its use. Fees charged by a Party to this Addendum to any other Party or to the Commission shall be established on a cost of service basis.

(b) Facilities owned by Parties to be made available under this paragraph include but are not limited to Oak Lodge's 24" transmission line and the Valley View Reservoirs, the Oak Lodge pump station from Clackamas River Water and the City of Gladstone 27" transmission line. These facilities may be used by any Party, as needed, on a cost of service basis, provided as above that ownership of the facility remains with the Party providing it and that Party shall have first priority in its use. The right to use a facility under this Addendum shall cease if the facility is sold or transferred to another by the owner or retired from service by the owner.

(c) In case a facility or improvement that is owned by a Party and which is used by the Commission or by a Party to the Agreement or this Addendum is to be retired, sold or transferred, the owner of the facility or improvement shall notify the other Parties and the Commission in writing at least sixty days prior being irrevocably committed to the sale, transfer or retirement. In no case shall use by a Party to this Addendum other than the owner be used as grounds in any proceeding or litigation of any kind the intent or effect of which is to interfere with the facility owner's right to sell, transfer or retire the facility. Should a Party to the Agreement or this Addendum elect to sell, transfer or retire an improvement or facility used by another Party to the Agreement or Addendum the Commission or a Party to the Agreement or to this Addendum shall have a "right of first refusal" to purchase the facility which shall be exercised within sixty (60) days of receipt of the notice above provided for.

6. Termination.

(a) The Parties have transferred a total of 48.9 MGD of water rights permits and applications to the Commission. Upon termination of this Addendum No. 1 or dissolution of the NCCWC with the first phase of the facility expansion completed, but the second phase not completed, the Parties shall be deemed to be the owners of 25 MGD of Clackamas River water rights held by the Commission for use at the site. The share of ownership for each party shall be in the same proportions as its undivided ownership in the Commission. In addition to its partial ownership of the 25 MGD, Sunrise shall also be deemed to be the owner of 10 MGD of the remaining 23.9 MGD and Oak Lodge shall be deemed the owner of 10 MGD of the remaining 23.9 MGD and Gladstone shall be deemed the owner of 3.9 MGD. Gladstone's share of water rights shall include those Clackamas River water rights certificated at the time of this Addendum given to the Commission by Gladstone (2.6 MGD).

(b) Upon termination of this Addendum No. 1 or dissolution of the NCCWC after both phases of construction of the membrane filter, the Parties shall be deemed to be joint owners of Clackamas River water rights, permits and certificates held by the Commission, up to the maximum beneficial use Commission facilities are capable of delivering at that time. Water rights certificates and permits given to the Commission by the Parties to the Second Amended IGA that cannot be used at the Commission site shall be distributed to Sunrise up to the amount in section 6.03 C of the Second Amended IGA, provided that, Gladstone shall be entitled to 3.9 MGD of Clackamas River water rights contributed to the Commission including the certificated water rights held by the Commission received from Gladstone. Water rights held separately by the Parties shall be unaffected.

(c) Unless otherwise agreed, upon dissolution of the NCCWC the disposition of assets shall be in accordance with section 8.02 of the Second Amended IGA, provided that Gladstone shall have the same rights as Oak Lodge. In the event neither Sunrise nor Oak Lodge purchases the assets of the Commission upon termination, Gladstone may do so. If a Party withdraws from the NCCWC or if termination occurs under section 8.03 of the Second Amended IGA, and if the remaining Parties wish to continue the Commission, the remaining Parties shall purchase the interest of the terminating Party.

Effective Date. This Addendum No. 1 shall be effective on the 17th day of August, 2005.

Address of the Parties. The physical addresses of the Parties are as follows:

Sunrise Water Authority:	Sunrise Water Authority 10602 SE 129th Ave. Portland, OR 97236
Oak Lodge Water District:	Oak Lodge Water District 14496 SE River Rd. Oak Grove, OR 97267
City of Gladstone:	City of Gladstone City Hall 525 Portland Avenue Gladstone, OR 97027

7. Execution of Counterparts and Duplicate Originals.

This Agreement may be executed in counterparts and by the Parties on separate counterparts. The Agreement shall be made when each Party has executed a counterpart. There shall be at least three (3) duplicate originals containing the signatures of the representatives of all Parties. One original shall be retained by each Party and one original shall be kept in the files of the Commission.

IN WITNESS WHEREOF, the Parties have, pursuant to the official action of their governing bodies, duly authorizing the same, caused their respective officers to execute this instrument.

SUNRISE WATER AUTHORITY

By Robert L. Fientress

Attest ________________

OAK LODGE WATER DISTRICT

By Jim Knapp Jr

Attest ________________

CITY OF GLADSTONE

By ________________

Attest ________________

Sunrise Water Authority:	Sunrise Water Authority 10602 SE 129th Ave. Portland, OR 97236
Oak Lodge Water District:	Oak Lodge Water District 14496 SE River Rd. Oak Grove, OR 97267
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IN WITNESS WHEREOF, the Parties have, pursuant to the official action of their governing bodies, duly authorizing the same, caused their respective officers to execute this instrument.

SUNRISE WATER AUTHORITY

By______________________________

Attest______________________________

OAK LODGE WATER DISTRICT

By______________________________

Attest______________________________

CITY OF GLADSTONE

By Wade Byers

Attest Jonathan Block

Appendix E: Field Data and Calibration Results

Table E-1. Steady-State Calibration Results

Test No.	Zone	Model pressure junction	Model flow junction	Date, 2014	Time	Pressure	Time	Pressure	Diffuser flow	Time	Pressure	Before pressure	During pressure	Before	During	Notes
CALIB_1	High	HYD401	HYD400	4/23	11:50 a.m.	59.5	11:58 a.m.	53.5	1260	12:05 p.m.	59.5	61	52	1.5	-1.1	Pressure at logger during test = 53.3, P in model = 49 psi.
CALIB_2	Intermediate Collins Crest	HYD166	HYD177	4/23	9:51 a.m.	58.5	10:00 a.m.	35.5	875	10:06 a.m.	57	52	37	-6.5	1.5	Not a good match on static pressure prior to the test. With a 1-inch connection at the Park Way check valve, the static pressure prior to the test is 52 psi and 37 psi during the test. The Webster station circle chart read approximately 340 gpm at the time of the test. The modeled pump station supplies 360 gpm at the time of the test given this connection. This connection should be closed for the evaluation.
CALIB_3	Low	HYD283	HYD284	4/22	1:45 p.m.	84	1:58 p.m.	79	619	2:04 p.m.	84	85	83	0.57	4	Hydrant was not fully open during test
CALIB_4	High Ridgewood	HYD071	HYD070	4/23	9:08 a.m.	86.5	9:18 a.m.	61.5	1240	9:22 a.m.	82.5	88	58	1.48	-3.97	The Ridgewood neighborhood was previously thought to be in the intermediate zone, however, there is an unmapped connection to the high zone. The OLWD - high zone inter-tie was not monitored during the intermediate zone tests, but is now set to the remaining demand in the high zones minus the reading at Webster

Appendix F: Basis of Estimate Report

Appendix F

Basis of Estimate Report

Introduction

Brown and Caldwell's opinion of the probable construction cost (estimate) for the Gladstone Water Master Plan is presented below.

Summary

This Basis of Estimate contains the following information:

- Scope of work
- Background of this estimate
- Class of estimate
- Estimating methodology
- Direct cost development
- Indirect cost development
- Bidding assumptions
- Estimating assumptions
- Estimating exclusions
- Allowances for known but undefined work
- Contractor and other estimate markups

Scope of Work

This cost estimate includes preliminary pricing for the following water system features:

1. Unit price for installation of a 6-, 8- and 12-inch DI water mains which includes patching of asphalt.
2. Unit Price for fire hydrant installation with 40 LF of 6" ductile iron pipe.
3. Unit price for 1.5 inches of milling and over-lay of asphalt based on 1 foot L x 12 foot W area.
4. Unit price for a typical copper domestic water service (40 LF of 1-inch copper pipe).
5. Installation of a 600 LF x 24-foot-wide asphalt access road to the 2 MG water tank.
6. Installation of a 2MG concrete water tank.
7. Alternate price as a separate estimate to install a 2MG welded steel water tank.

Background of this Estimate of Probable Construction Cost

The attached estimate of probable construction cost is based on documents dated July 21, 2014, received by the ESG. These documents are described as 0 – 2 percent complete based on the current project progression, additional or updated scope and/or quantities, and ongoing discussions with the project team. Further information can be found in the detailed estimate reports.

AACEI Estimate Classification

In accordance with the Association for the Advancement of Cost Engineering International (AACE) criteria, this is a Class 5 estimate. A Class 5 estimate is defined as a Conceptual Level or Project Viability Estimate. Typically, engineering is from 0 to 2 percent complete. Class 5 estimates are used to prepare planning level cost scopes or evaluation of alternative schemes, long range capital outlay planning and can also form the base work for the Class 4 Planning Level or Design Technical Feasibility Estimate.

Expected accuracy for Class 5 estimates typically ranges from -50 to +100 percent, depending on the technological complexity of the project, appropriate reference information and the inclusion of an appropriate contingency determination. In unusual circumstances, ranges could exceed those shown.

Estimating Methodology

This estimate was prepared using quantity take-offs, vendor quotes and equipment pricing furnished either by the project team or by the estimator. The estimate includes direct labor costs and anticipated productivity adjustments to labor, and equipment. Where possible, estimates for work anticipated to be performed by specialty subcontractors have been identified.

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

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

Direct Cost Development

Costs associated with the General Provisions and the Special Provisions of the construction documents, which are collectively referred to as Contractor General Conditions (CGC), were based on the estimator's interpretation of the contract documents. The estimates for CGCs are divided into two groups: a time-related group (e.g., field personnel), and non-time-related group (e.g., bonds and insurance).Labor burdens such as health and welfare, vacation, union benefits, payroll taxes, and workers compensation insurance are included in the labor rates. No trade discounts were considered.

Indirect Cost Development

A percentage allowance for contractor's home office expense has been included in the overall rate markups. The rate is standard for this type of heavy construction and is based on typical percentages outlined in Means Heavy Construction Cost Data.

The contractor's cost for builder's risk, general liability and vehicle insurance has been included in this estimate. Based on historical data, this is typically two to four percent of the overall construction contract amount. These indirect costs have been included in this estimate as a percentage of the gross cost, and are added after the net markups have been applied to the appropriate items.

Bidding Assumptions

The following bidding assumptions were considered in the development of this estimate.

1. Bidders must hold a valid, current Contractor's credentials, applicable to the type of project.
2. Bidders will develop estimates with a competitive approach to material pricing and labor productivity, and will not include allowances for changes, extra work, unforeseen conditions or any other unplanned costs.
3. Estimated costs are based on a minimum of four bidders. Actual bid prices may increase for fewer bidders or decrease for a greater number of bidders.
4. Bidders will account for General Provisions and Special Provisions of the contract documents and will perform all work except that which will be performed by traditional specialty subcontractors as identified here:
   - Electrical and Instrumentation
   - HVAC systems
   - Paintings and Coatings

Estimating Assumptions

As the design progresses through different completion stages, it is customary for the estimator to make assumptions to account for details that may not be evident from the documents. The following assumptions were used in the development of this estimate.

1. Contractor performs the work during normal daylight hours, nominally 7 a.m. to 5 p.m., Monday through Friday, in an 8-hour shift. No allowance has been made for additional shift work or weekend work.
2. Contractor has complete access for lay-down areas and mobile equipment.
3. Equipment rental rates are based on verifiable pricing from the local project area rental yards, Blue Book rates and/or rates contained in the estimating database.
4. Contractor markup is based on conventionally accepted values that have been adjusted for project-area economic factors.
5. Major equipment costs are based on both vendor supplied price quotes obtained by the project design team and/or estimators, and on historical pricing of like equipment.
6. Process equipment vendor training using vendors' standard Operations and Maintenance (O&M) material, is included in the purchase price of major equipment items where so stated in that quotation.
7. Bulk material quantities are based on manual quantity take-offs.
8. There is sufficient electrical power to feed the specified equipment. The local power company will supply power and transformers suitable for this facility.
9. Soils are of adequate nature to support the structures. No piles have been included in this estimate.
10. The asphalt access road is estimated as 600 LF long by 24 wide asphalt.

    A 16' shoulder/ ditch will be located on one side of the access road for a total width 40 feet per Angela Wieland's request. The estimate assumes grading 4' of dirt over half the area for the access road (12,000 SF), resulting in 1,778 CY of earth movement.
11. The tank clearing area is 250' x 250'. The earth movement quantity is based on moving 4' of dirt over half the area (31,250 SF), resulting in 4,629 CY of earth movement.

Estimating Exclusions

The following estimating exclusions were assumed in the development of this estimate.

1. Hazardous materials remediation and/or disposal.
2. O&M costs for the project with the exception of the vendor supplied O&M manuals.
3. Utility agency costs for incoming power modifications.
4. Permits beyond those normally needed for the type of project and project conditions.
5. SCADA for water tank operation

Allowances for Known but Undefined Work

The following allowances were made in the development of this estimate.

1. Chain Link Fence - 1,000 LF
2. Gravel at perimeter of tank – 416 CY
3. Excavate & fill for access road – 1,778 CY
4. Excavate & fill for tank 4,629 CY

Contractor and Other Estimate Markups

Contractor markup is based on conventionally accepted values which have been adjusted for project-area economic factors. Estimate markups are listed in Table 1.

Table 1 Estimate Markups

Item	Rate (%)
Net cost markups	
Labor (employer payroll burden)	10
Materials and process equipment	10
Equipment (construction-related)	10
Subcontractor	5
Material shipping and handling	2
Gross cost markups	
Contractors general conditions	10
Traffic control (in lieu of 2% for contractor start-up, training and O&M)	2
Undesigned/undeveloped detail construction contingency	40
Builders risk, liability and auto insurance	2
Performance and payment bonds	1.5
Escalation to midpoint of construction	0

Net Cost Markups

Net cost markups are applied to specific components of the net construction cost. Net costs plus net cost markups are reflected in the unit pricing for system components as shown in the individual capital improvement project cost estimates.

Labor Markup

The labor rates used in the estimate were derived chiefly from the latest published State Prevailing Wage Rates. These include base rate paid to the laborer plus fringes. A labor burden factor is applied to these such that the final rates include all employer paid taxes. These taxes are FICA (which covers social security plus Medicare), Workers Comp (which varies based on state, employer experience and history) and unemployment insurance. The result is fully loaded labor rates. In addition to the fully loaded labor rate, an overhead and profit markup is applied at the back end of the estimate. This covers payroll and accounting, estimator's wages, home office rent, advertising and owner profit.

Materials and Process Equipment Markup

This markup consists of the additional cost to the contractor beyond the raw dollar amount for material and process equipment. This includes shop drawing preparation, submittal and/or re-submittal cost, purchasing and scheduling materials and equipment, accounting charges including invoicing and payment, inspection of received goods, receiving, storage, overhead and profit.

Equipment (Construction) Markup

This markup consists of the costs associated with operating the construction equipment used in the project. Most GCs will rent rather than own the equipment and then charge each project for its equipment cost. The equipment rental cost does not include fuel, delivery and pick-up charges, additional insurance requirements on rental equipment, accounting costs related to home office receiving invoices and payment. However, the crew rates used in the estimate do account for the equipment rental cost. Occasionally, larger contractors will have some or all of the equipment needed for the job, but in order to recoup their initial purchasing cost they will charge the project an internal rate for equipment use which is similar to the rental cost of equipment. The GC will apply an overhead and profit percentage to each individual piece of equipment whether rented or owned.

Subcontractor Markup

This markup consists of the GC's costs for subcontractors who perform work on the site. This includes costs associated with shop drawings, review of subcontractor's submittals, scheduling of subcontractor work, inspections, processing of payment requests, home office accounting, and overhead and profit on subcontracts.

Material Shipping and Handling

This can range from 2 to 6 percent, and is based on the type of project, material makeup of the project, and the region and location of the project. Material shipping and handling covers delivery costs from vendors, unloading costs (and in some instances loading and shipment back to vendors for rebuilt equipment), site paper work, and inspection of materials prior to unloading at the project site. BC typically adjusts this percentage by the amount of materials and whether vendors have included shipping costs in the quotes that were used to prepare the estimate. This cost also includes the GC's cost to obtain local supplies; e.g., oil, gaskets and bolts that may be missing from the equipment or materials shipped.

Gross Cost Markups

Gross cost markups are applied to the net construction cost plus net cost markups. Gross cost markups are applied to the accumulative cost in the order of markups reflected in Table 1.

General Conditions

General conditions are associated with contractor start-up costs and reflect scheduling, mobilization, and demobilization.

Traffic Control

A 2% markup was assigned for traffic control, given construction primarily in the public right-of-way (i.e., pipe replacement).

Undesigned/Undeveloped Detail Construction Contingency

The contingency factor covers unforeseen conditions, area economic factors, and general project complexity. This contingency is used to account for those factors that cannot be addressed in each of the labor and/or material installation costs. Based on industry standards, completeness of the project documents, project complexity, the current design stage and area factors, construction contingency can range from 10 to 50 percent. Contingency is applied at the estimators discretion based on the amount of Undesigned/undeveloped detail for the particular project. Specific for this master plan-level assessment, contingency was more conservatively estimated.

Builders Risk, Liability, and Vehicle Insurance

This percentage comprises all three items. There are many factors which make up this percentage, including the contractor's track record for claims in each of the categories. Another factor affecting insurance rates has been a dramatic price increase across the country over the past several years due to domestic and foreign influences. Consequently, in the construction industry we have observed a range of 0.5 to 1 percent for Builders Risk Insurance, 1 to 1.25 percent for General Liability Insurance, and 0.85 to 1 percent for Vehicle Insurance. Many factors affect each area of insurance, including project complexity and contractor's requirements and history. Instead of using numbers from a select few contractors, we believe it is more prudent to use a combined 2 percent to better reflect the general costs across the country. Consequently, the actual cost could be higher or lower based on the bidder, region, insurance climate, and on the contractor's insurability at the time the project is bid.

Performance and Payment Bonds

Based on historical and industry data, this can range from 0.75 to 3 percent of the project total. There are several contributing factors including such items as size of the project, regional costs, and contractor's historical record on similar projects, complexity and current bonding limits. BC uses 1.5 percent for bonds, which we have determined to be reasonable for most heavy construction projects.

Escalation to Midpoint of Construction for All Project Cost

Typically for design estimates, in addition to contingency, it is customary for projects that will be built over several years to include an escalation to midpoint of anticipated construction to account for the future escalation of labor, material and equipment costs beyond values at the time the estimate is prepared. For this project, given the unknown nature of construction schedule, costs are reflected in 2014 dollars, and no escalation to midpoint of construction has been estimated.

Attachment A
Estimate of Probable Construction Cost

Unit Costs		
Item	Unit	Cost
Water Facility Installation		
PRV Station	EA	$30,000
6-inch Ductile-Iron Pipe with asphalt patch	LF	$154
8-inch Ductile-Iron Pipe with asphalt patch	LF	$179
12-inch Ductile-Iron Pipe with asphalt patch	LF	$212
Service Line Connection (40' of 1" Copper Domestic Service)	EA	$2,473
Hydrant Replacement	EA	$9,615
Hydrant Removal	EA	$884
Site Clearing (access road)	AC	$26,417
Cut and Fill (access road)	CY	$18.84
Site Clearing (storage tank)	AC	$29,844
Cut and Fill (storage tank/ vault)	CY	$11.56
Scaffolding (tank construction)	LS	$43,200.00
Concrete Tank Construction	GAL	$0.85
Steel Tank Construction	GAL	$0.84
SCADA - Model M-800 SCADA RTU System, NEMA 1	EA	$1,995.00
SCADA - Model M-800 SCADA RTU System, NEMA 4X	EA	$2,095.00
SCADA - Analog Input Expansion Module	EA	$495.00
SCADA - 50' Antenna Cable	EA	$75.00
MTU Onsite Power, 125 kW diesel Generator, Sub Base Fuel Tank and Quiet Zone Enclosure	EA	$51,569.00
Eaton Power Products, 480V, 200A, 3 Pole Service Entrance ATS	EA	$6,500.00
Misc. Electrical Materials (conduit, cable, hardware) for gen set installation	LS	$6,200.00
Restoration/ Resurfacing		
Milling, Asphalt and Overlay (one lane width for pipe install)	LF	$15
Asphaltic Paving	SF	$5.06
Fencing/ Exterior Stonework	LF	$58.54
Gross Markups (applied to project subtotals)		
Contractors General Conditions (%)	LS	10%
Traffic Control (%)	LS	2%
Construction Contingency (%)	LS	40%
Builders Risk, Liability and Auto Insurance (%)	LS	2%
Performance and Payment Bonds (%)	LS	1.50%
Design/ Administrative		
Engineering and Permitting (%)	LS	Varies (20-40%)
Construction Administration (%)	LS	5%
SCADA System - Freight Charges	LS	$35.00
SCADA System - Set Up Fee and Onsite Training	LS	$250.00
SCADA System - Install (Pump Station)	LS	$2,000.00
SCADA System - Install (Meter)	LS	$750.00
Gen Set - Install	LS	$25,600.00
Maintenance		
SCADA - Service Plan Fee	Annual	$570.00
SCADA - Analog Expansion Service Fee	Annual	$60.00

Berkeley Street Pipe Replacement				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
12-inch Ductile-Iron Pipe with asphalt patch	1,604	LF	$212	$340,048
Service Line Connection (40' of 1" Copper Domestic Service)	26	EA	$2,473	$64,298
Hydrant Removal	4	EA	$884	$3,536
Hydrant Replacement	4	EA	$9,615	$38,460
Milling, Asphalt and Overlay (one lane width for pipe install)	1,604	LF	$15	$24,060
Capital Expense Subtotal				$470,402
Gross Markups				
Contractors General Conditions (%)		LS	10%	$47,040
Traffic Control (%)		LS	2%	$10,349
Construction Contingency (%)		LS	40%	$211,116
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$14,778
Performance and Payment Bonds (%)		LS	1.5%	$11,305
Project Subtotal				$764,991
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$152,998
Construction Administration (%)		LS	5%	$38,250
Administrative Expense Total				$191,248
Capital Implementation Cost Total				$956,239

Cason Road PRV and Pipe Replacement				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
8-inch Ductile-Iron Pipe with asphalt patch	817	LF	$179	$146,243
8-inch Ductile-Iron Pipe with asphalt patch	60	LF	$179	$10,740
12-inch Ductile-Iron Pipe with asphalt patch	1,088	LF	$212	$230,656
Service Line Connection (40' of 1" Copper Domestic Service)	32	EA	$2,473	$79,136
Hydrant Removal	6	EA	$884	$5,304
Hydrant Replacement	6	EA	$9,615	$57,690
Milling, Asphalt and Overlay (one lane width for pipe install)	1,905	LF	$15	$28,575
PRV Station	2	EA	$30,000	$60,000
Cut and Fill (storage tank/ vault)	56	CY	$12	$647
Capital Expense Subtotal				$618,991
Gross Markups				
Contractors General Conditions (%)		LS	10%	$61,899
Traffic Control (%)		LS	2%	$13,618
Construction Contingency (%)		LS	40%	$277,803
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$19,446
Performance and Payment Bonds (%)		LS	1.5%	$14,876
Project Subtotal				$1,006,634
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$201,327
Construction Administration (%)		LS	5%	$50,332
Administrative Expense Total				$251,659
Capital Implementation Cost Total				$1,258,293

Clackamas Blvd Pipe Replacement				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
8-inch Ductile-Iron Pipe with asphalt patch	1,836	LF	$179	$328,644
Service Line Connection (40' of 1" Copper Domestic Service)	14	EA	$2,473	$34,622
Hydrant Removal	2	EA	$884	$1,768
Hydrant Replacement	2	EA	$9,615	$19,230
Milling, Asphalt and Overlay (one lane width for pipe install)	1,836	LF	$15	$27,540
Capital Expense Subtotal				$411,804
Gross Markups				
Contractors General Conditions (%)		LS	10%	$41,180
Traffic Control (%)		LS	2%	$9,060
Construction Contingency (%)		LS	40%	$184,818
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$12,937
Performance and Payment Bonds (%)		LS	1.5%	$9,897
Project Subtotal				$669,696
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$133,939
Construction Administration (%)		LS	5%	$33,485
Administrative Expense Total				$167,424
Capital Implementation Cost Total				$837,120

Hereford PRV				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
12-inch Ductile-Iron Pipe with asphalt patch	100	LF	$212	$21,200
Milling, Asphalt and Overlay (one lane width for pipe install)	100	LF	$15	$1,500
PRV Station	1	EA	$30,000	$30,000
Cut and Fill (storage tank/ vault)	28	CY	$12	$324
Capital Expense Subtotal				$53,024
Gross Markups				
Contractors General Conditions (%)		LS	10%	$5,302
Traffic Control (%)		LS	2%	$1,167
Construction Contingency (%)		LS	40%	$23,797
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$1,666
Performance and Payment Bonds (%)		LS	1.5%	$1,274
Project Subtotal				$86,230
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$17,246
Construction Administration (%)		LS	5%	$4,311
Administrative Expense Total				$21,557
Capital Implementation Cost Total				$107,787

Hull PRV				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
12-inch Ductile-Iron Pipe with asphalt patch	100	LF	$212	$21,200
Milling, Asphalt and Overlay (one lane width for pipe install)	100	LF	$15	$1,500
PRV Station	1	EA	$30,000	$30,000
Cut and Fill (storage tank/ vault)	28	CY	$12	$324
Capital Expense Subtotal				$53,024
Gross Markups				
Contractors General Conditions (%)		LS	10%	$5,302
Traffic Control (%)		LS	2%	$1,167
Construction Contingency (%)		LS	40%	$23,797
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$1,666
Performance and Payment Bonds (%)		LS	1.5%	$1,274
Project Subtotal				$86,230
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$17,246
Construction Administration (%)		LS	5%	$4,311
Administrative Expense Total				$21,557
Capital Implementation Cost Total				$107,787

Jersey Street Pipe Replacement				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
8-inch Ductile-Iron Pipe with asphalt patch	510	LF	$179	$91,290
Service Line Connection (40' of 1" Copper Domestic Service)	17	EA	$2,473	$42,041
Hydrant Removal	2	EA	$884	$1,768
Hydrant Replacement	2	EA	$9,615	$19,230
Milling, Asphalt and Overlay (one lane width for pipe install)	503	LF	$15	$7,545
Capital Expense Subtotal				$161,874
Gross Markups				
Contractors General Conditions (%)		LS	10%	$16,187
Traffic Control (%)		LS	2%	$3,561
Construction Contingency (%)		LS	40%	$72,649
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$5,085
Performance and Payment Bonds (%)		LS	1.5%	$3,890
Project Subtotal				$263,247
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$52,649
Construction Administration (%)		LS	5%	$13,162
Administrative Expense Total				$65,812
Capital Implementation Cost Total				$329,059

Landon PRV				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
12-inch Ductile-Iron Pipe with asphalt patch	100	LF	$212	$21,200
Milling, Asphalt and Overlay (one lane width for pipe install)	100	LF	$15	$1,500
PRV Station	1	EA	$30,000	$30,000
Cut and Fill (storage tank/ vault)	28	CY	$12	$324
Capital Expense Subtotal				$53,024
Gross Markups				
Contractors General Conditions (%)		LS	10%	$5,302
Traffic Control (%)		LS	2%	$1,167
Construction Contingency (%)		LS	40%	$23,797
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$1,666
Performance and Payment Bonds (%)		LS	1.5%	$1,274
Project Subtotal				$86,230
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$17,246
Construction Administration (%)		LS	5%	$4,311
Administrative Expense Total				$21,557
Capital Implementation Cost Total				$107,787

Meldrum Bar Park Road PRV and Pipe Replacement				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
12-inch Ductile-Iron Pipe with asphalt patch	1,194	LF	$212	$253,128
Service Line Connection (40' of 1" Copper Domestic Service)	-	EA	$2,473	$-
Hydrant Removal	3	EA	$884	$2,652
Hydrant Replacement	3	EA	$9,615	$28,845
Milling, Asphalt and Overlay (one lane width for pipe install)	1,194	LF	$15	$17,910
PRV Station	1	EA	$30,000	$30,000
Cut and Fill (storage tank/ vault)	28	CY	$12	$324
Capital Expense Subtotal				$332,859
Gross Markups				
Contractors General Conditions (%)		LS	10%	$33,286
Traffic Control (%)		LS	2%	$7,323
Construction Contingency (%)		LS	40%	$149,387
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$10,457
Performance and Payment Bonds (%)		LS	1.5%	$8,000
Project Subtotal				$541,311
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$108,262
Construction Administration (%)		LS	5%	$27,066
Administrative Expense Total				$135,328
Capital Implementation Cost Total				$676,639

Park Way Pipe Replacement				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
8-inch Ductile-Iron Pipe with asphalt patch	905	LF	$179	$161,995
Service Line Connection (40' of 1" Copper Domestic Service)	18	EA	$2,473	$44,514
Hydrant Removal	2	EA	$884	$1,768
Hydrant Replacement	3	EA	$9,615	$28,845
Milling, Asphalt and Overlay (one lane width for pipe install)	905	LF	$15	$13,575
Capital Expense Subtotal				$250,697
Gross Markups				
Contractors General Conditions (%)		LS	10%	$25,070
Traffic Control (%)		LS	2%	$5,515
Construction Contingency (%)		LS	40%	$112,513
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$7,876
Performance and Payment Bonds (%)		LS	1.5%	$6,025
Project Subtotal				$407,696
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$81,539
Construction Administration (%)		LS	5%	$20,385
Administrative Expense Total				$101,924
Capital Implementation Cost Total				$509,620

Sherwood Neighborhood Pipe Replacement				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
8-inch Ductile-Iron Pipe with asphalt patch	4,190	LF	$179	$750,010
Service Line Connection (40' of 1" Copper Domestic Service)	65	EA	$2,473	$160,745
Hydrant Removal	9	EA	$884	$7,956
Hydrant Replacement	9	EA	$9,615	$86,535
Milling, Asphalt and Overlay (one lane width for pipe install)	4,190	LF	$15	$62,850
Capital Expense Subtotal				$1,068,096
Gross Markups				
Contractors General Conditions (%)		LS	10%	$106,810
Traffic Control (%)		LS	2%	$23,498
Construction Contingency (%)		LS	40%	$479,361
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$33,555
Performance and Payment Bonds (%)		LS	1.5%	$25,670
Project Subtotal				$1,736,990
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$347,398
Construction Administration (%)		LS	5%	$86,850
Administrative Expense Total				$434,248
Capital Implementation Cost Total				$2,171,238

Rinearson Road Pipe Replacement				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
8-inch Ductile-Iron Pipe with asphalt patch	1,207	LF	$179	$216,053
Service Line Connection (40' of 1" Copper Domestic Service)	10	EA	$2,473	$24,730
Hydrant Removal	3	EA	$884	$2,652
Hydrant Replacement	3	EA	$9,615	$28,845
Milling, Asphalt and Overlay (one lane width for pipe install)	1,207	LF	$15	$18,105
Capital Expense Subtotal				$290,385
Gross Markups				
Contractors General Conditions (%)		LS	10%	$29,039
Traffic Control (%)		LS	2%	$6,388
Construction Contingency (%)		LS	40%	$130,325
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$9,123
Performance and Payment Bonds (%)		LS	1.5%	$6,979
Project Subtotal				$472,238
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$94,448
Construction Administration (%)		LS	5%	$23,612
Administrative Expense Total				$118,060
Capital Implementation Cost Total				$590,298

Risley Avenue Pipe Replacement				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
8-inch Ductile-Iron Pipe with asphalt patch	893	LF	$179	$159,847
Service Line Connection (40' of 1" Copper Domestic Service)	12	EA	$2,473	$29,676
Hydrant Removal	2	EA	$884	$1,768
Hydrant Replacement	2	EA	$9,615	$19,230
Milling, Asphalt and Overlay (one lane width for pipe install)	893	LF	$15	$13,395
Capital Expense Subtotal				$223,916
Gross Markups				
Contractors General Conditions (%)		LS	10%	$22,392
Traffic Control (%)		LS	2%	$4,926
Construction Contingency (%)		LS	40%	$100,494
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$7,035
Performance and Payment Bonds (%)		LS	1.5%	$5,381
Project Subtotal				$364,143
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$72,829
Construction Administration (%)		LS	5%	$18,207
Administrative Expense Total				$91,036
Capital Implementation Cost Total				$455,179

SE 82nd Drive Pipe Replacement				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
12-inch Ductile-Iron Pipe with asphalt patch	860	LF	$212	$182,320
Service Line Connection (40' of 1" Copper Domestic Service)	2	EA	$2,473	$4,946
Hydrant Removal	3	EA	$884	$2,652
Hydrant Replacement	3	EA	$9,615	$28,845
Milling, Asphalt and Overlay (one lane width for pipe install)	860	LF	$15	$12,900
Capital Expense Subtotal				$231,663
Gross Markups				
Contractors General Conditions (%)		LS	10%	$23,166
Traffic Control (%)		LS	2%	$5,097
Construction Contingency (%)		LS	40%	$103,970
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$7,278
Performance and Payment Bonds (%)		LS	1.5%	$5,568
Project Subtotal				$376,742
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$75,348
Construction Administration (%)		LS	5%	$18,837
Administrative Expense Total				$94,185
Capital Implementation Cost Total				$470,927

Webster Pump Station Upgrades (Generator Set)				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
MTU Onsite Power, 125 kW diesel Generator, Sub Base Fuel Tank and Quiet Zone Enclosure	1	EA	$51,569	$51,569
Eaton Power Products, 480V, 200A, 3 Pole Service Entrance ATS	1	EA	$6,500	$6,500
Misc. Electrical Materials (conduit, cable, hardware) for gen set installation	1	LS	$6,200	$6,200
Capital Expense Subtotal				$64,269
Gross Markups				
Contractors General Conditions (%)		LS	10%	$6,427
Traffic Control (%)		LS	0%	$-
Construction Contingency (%)		LS	40%	$28,278
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$1,979
Performance and Payment Bonds (%)		LS	1.5%	$1,514
Project Subtotal				$102,468
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$20,494
Construction Administration (%)		LS	5%	$5,123
Gen Set - Install	1	LS	$25,600	$25,600
Administrative Expense Total				$51,217
Capital Implementation Cost Total				$153,685

Webster Pump Station SCADA System Upgrades				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
Webster Pump Station Building (includes Kirkwood Pumps)				
SCADA - Model M-800 SCADA RTU System, NEMA 1	1	EA	$1,995	$1,995
SCADA - Analog Input Expansion Module	1	EA	$495	$495
SCADA - 50' Antenna Cable	1	EA	$75	$75
Remote Locations (3)				
SCADA - Model M-800 SCADA RTU System, NEMA 4X	3	EA	$2,095	$6,285
Capital Expense Subtotal				$8,850
Gross Markups				
Contractors General Conditions (%)		LS	10%	$885
Traffic Control (%)		LS	0%	-
Construction Contingency (%)		LS	40%	$3,894
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$273
Performance and Payment Bonds (%)		LS	1.5%	$209
Project Subtotal				$14,110
Administrative Expenses				
Engineering and Permitting (%)		LS	0%	-
Construction Administration (%)		LS	5%	$706
SCADA System - Freight Charges	4	LS	$35	$140
SCADA System - Set Up Fee and Onsite Training	4	LS	$250	$1,000
SCADA System - Install (Pump Station)	1	LS	$2,000	$2,000
SCADA System - Install (Meter)	3	LS	$750	$2,250
Administrative Expense Total				$6,096
Capital Implementation Cost Total				$20,206
Annual Maintenance Expenses				
SCADA - Service Plan Fee	4	Annual	$570	$2,280
SCADA - Analog Expansion Service Fee	1	Annual	$60	$60
Annual Maintenance Cost Total				$2,340

Storage Tank-Alternative A (Steel)				
Description	Quantity	Unit	Unit Cost (2014)	2014 Cost
Capital Expenses				
Access Road				
Site Clearing (access road)	0.6	AC	$26,417	$15,850
Cut and Fill (access road)	1,778	CY	$19	$33,498
Asphaltic Paving	15,000	SF	$5	$75,900
Site Preparation				
Site Clearing (storage tank)	1.4	AC	$29,844	$41,782
Cut and Fill (storage tank/ vault)	4,629	CY	$12	$53,511
Tank Construction				
Scaffolding (tank construction)	1	LS	$43,200	$43,200
Steel Tank Construction	2,000,000	GAL	$0.84	$1,680,000
Fencing/ Exterior Stonework	1,000	LF	$59	$58,540
12-inch Ductile-Iron Pipe with asphalt patch	1,000	LF	$212	$212,000
Capital Expense Subtotal				$2,214,281
Gross Markups				
Contractors General Conditions (%)		LS	10%	$221,428
Traffic Control (%)		LS	2%	$48,714
Construction Contingency (%)		LS	40%	$993,769
Builders Risk, Liability and Auto Insurance (%)		LS	2%	$69,564
Performance and Payment Bonds (%)		LS	1.5%	$53,216
Project Subtotal				$3,600,972
Administrative Expenses				
Engineering and Permitting (%)		LS	20%	$720,194
Construction Administration (%)		LS	5%	$180,049
Administrative Expense Total				$900,243
Capital Implementation Cost Total				$4,501,215

FINAL

Gladstone Stormwater Master Plan

Prepared for

City of Gladstone, Oregon

November 2014

Brown AND Caldwell

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Table of Contents

List of Appendices

List of Figures

*An * indicates figure immediately follows page listed.*

List of Tables

List of Abbreviations

AMC	antecedent moisture conditions
BMP	best management practice
CAD	computer-aided design
CCI	Construction Cost Index
CCSD 1	Clackamas County Service District 1
CCTV	closed-circuit television
cfs	cubic foot/feet per second
CIP	capital improvement project
City	City of Gladstone
CMP	corrugated metal pipe
CN	curve number
County	Clackamas County
CSP	corrugated steel pipe
CWA	Clean Water Act
DDE	dichlorodiphenyldichloroethylene
DDT	dichlorodiphenyltrichloroethane
DEQ	Oregon Department of Environmental Quality
ENR	*Engineering News-Record*
EPA	U.S. Environmental Protection Agency
F	Fahrenheit
FC	flood control
ft^2	square foot/feet
ft^3	cubic foot/feet
GIS	geographic information system
Gladstone Standards	Stormwater Treatment and Detention Standards for the City of Gladstone
GPS	global positioning system
HDPE	high-density polyethylene
H/H	hydrologic and hydraulic
HSG	hydrologic soil group
H:V	horizontal:vertical
ID	identifier
IDDE	Illicit Discharge Detection and Elimination
LF	linear foot/feet
LiDAR	Light Detection and Ranging
LOS	level of service
MS4	municipal separate storm sewer system
NAD83	North American Datum of 1983
NAVD88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
ODOT	Oregon Department of Transportation
PAH	polycyclic aromatic hydrocarbon
PCB	polychlorinated biphenyl
PVC	polyvinyl chloride
RCP	reinforced concrete pipe
ROW	right-of-way
SCS	Soil and Conservation Service
SMM	City of Portland *Stormwater Management Manual* (2008)
SMP	Stormwater Master Plan
SWMM	Surface Water Management Model
SWMP	stormwater management plan
TMDL	total maximum daily load
UIC	underground injection control
WQ	water quality

Executive Summary

Background/Introduction

This 2014 Gladstone Stormwater Master Plan (SMP) documents the methods and results of the storm system capacity evaluation and the stormwater quality/retrofit assessment conducted for the City of Gladstone, Oregon (City). The SMP identifies and prioritizes capital improvement projects (CIPs) to address identified system capacity deficiencies and water quality improvements. The SMP also identifies additional stormwater program implementation needs in the form of equipment and staffing.

The City of Gladstone has historically managed its stormwater collection and conveyance system with limited mapped and surveyed field information. To date, no stormwater master plan has been developed for the City and no stormwater capital improvement program is in place. As a result, management of the stormwater collection and conveyance system is conducted on an as-needed basis, primarily in response to failing/failed infrastructure. Additionally, without an identified stormwater capital improvement program, the City currently funds stormwater program and infrastructure improvements without a dedicated stormwater utility fee.

The objectives of this plan include the following:

- Conduct a survey of the stormwater collection and conveyance system infrastructure within the city limits. Map the stormwater system in computer-aided design (CAD) and geographic information system (GIS) format.
- Compile surveyed system information into a comprehensive hydrologic and hydraulic (H/H) model for use in evaluating the capacity of the stormwater system and identifying capacity deficiencies.
- Interview City staff to develop an understanding of current system performance, function, and areas of concern.
- Identify water quality improvement projects and stormwater retrofits to address National Pollutant Discharge Elimination System (NPDES) municipal separate storm sewer system (MS4) permit requirements.
- Develop CIPs and associated cost estimates to address water quality and identified system capacity deficiencies under existing and future development scenarios. Where feasible, integrate flood control CIPs with water quality CIPs to address multiple objectives.
- Review staffing needs in consideration of updated regulatory requirements and proposed CIP implementation.
- Provide information needed to develop a dedicated stormwater funding source.

Study Area Characteristics

The city of Gladstone is located in Clackamas County, Oregon, approximately 12 miles south of Portland, Oregon. The city is bordered by the Clackamas River to the south, the Willamette River to the west, unincorporated Clackamas County (Clackamas County Service District 1) to the north and east, and Oak Grove/Oak Lodge Service District to the north. Surrounding cities include West Linn (to the west), Oregon City (to the south), and Milwaukie (to the north).

The city is approximately 2.5 square miles in area with elevations ranging from approximately 10 to 330 feet. The Clackamas River, a major tributary to the Willamette River, flows along the southern border of the city. The Willamette River flows along the western border of the city. Rinearson Creek, a piped and open-channel tributary to the Willamette River, flows from east to west and divides the city in half, approximately.

The city of Gladstone is primarily developed, with only about 10 percent of the city area identified as vacant. Vacant lands are scattered throughout the city, and a majority are publicly held. Single-family residential development is the primary land use within the city. Most commercial development is located along Oregon Highway 99E, along Portland Avenue, and at the intersection of 82nd Avenue and Interstate 205. Other land use categories include multifamily residential, industrial, and parks and open space.

Based on survey information obtained for this project, the City's drainage system is composed of approximately 30 miles of City-owned pipe and major open-channel conveyance system, 299 manholes, and more than 1,000 catch basins and cleanouts. Approximately 21 miles of pipe and open channel were modeled as part of this SMP, composed primarily of 12-inch-diameter pipe and greater.

Survey of the City's drainage system defined 26 major basins, reflecting 32 modeled pipe system outfalls: 13 to the Clackamas County stormwater system (piped or open channel); 11 to natural areas within Meldrum Bar Park, the Olson Wetlands, Glen Echo Wetlands, or Boardman Creek (all within the Willamette River drainage area), 5 to the Clackamas River, and 3 to natural areas adjacent to the Clackamas River.

Regulatory Requirements

The City operates under a Phase I NPDES MS4 permit, which requires implementation of stormwater management strategies to reduce pollutants discharged from the City's stormwater system. The City implements a Stormwater Management Plan (SWMP), which includes a variety of programmatic, non-structural, and source control activities to improve stormwater quality and reduce pollutant discharges in stormwater.

In addition to the implementation of the SWMP for water quality improvement, DEQ included a specific provision in the NPDES MS4 permit for Gladstone to complete and submit a SMP by January 1, 2014. DEQ is also requiring a stormwater retrofit assessment by July 1, 2015 to identify areas in the city underserved or lacking structural stormwater treatment facilities. Both the SMP and stormwater retrofit assessment are intended to identify stormwater quality controls to reduce the discharge of pollutants from the MS4. This SMP was developed to address DEQ's requirements related to development of a SMP and stormwater retrofit assessment. The draft of this plan was submitted to DEQ by January 1, 2014 to fulfill this regulatory obligation.

Study Methods

Development of this SMP involved an evaluation of the capacity of Gladstone's stormwater drainage system and an evaluation of opportunities to implement stormwater water quality facilities within the study area.

To evaluate the capacity of the Gladstone stormwater drainage system, a computer model was developed to simulate the hydrologic and hydraulic conditions of the public drainage system for pipes 12 inches in diameter and greater. The storm system was evaluated under both existing and anticipated future development conditions. Computational Hydraulics International's 2012 PC SWMM model software was selected to conduct this analysis. The system evaluation included the 2, 10, 25, and 100-year design storms. Extensive flooding was indicated by the hydraulic model for the 10-year and greater design events, consistent with flooding conditions reported by City staff.

In conjunction with the hydraulic evaluation of the City's stormwater system, water quality CIP opportunity areas were identified by reviewing system information including locations of existing vacant areas, publically-owned lands, existing and future land use conditions, storm system layout, topography, soils and drainage areas. Water quality CIPs focused on the use of infiltration-based facilities (e.g., vegetated infiltration basins, rain gardens, planters) on public property to provide runoff volume reduction in addition to conventional water quality treatment.

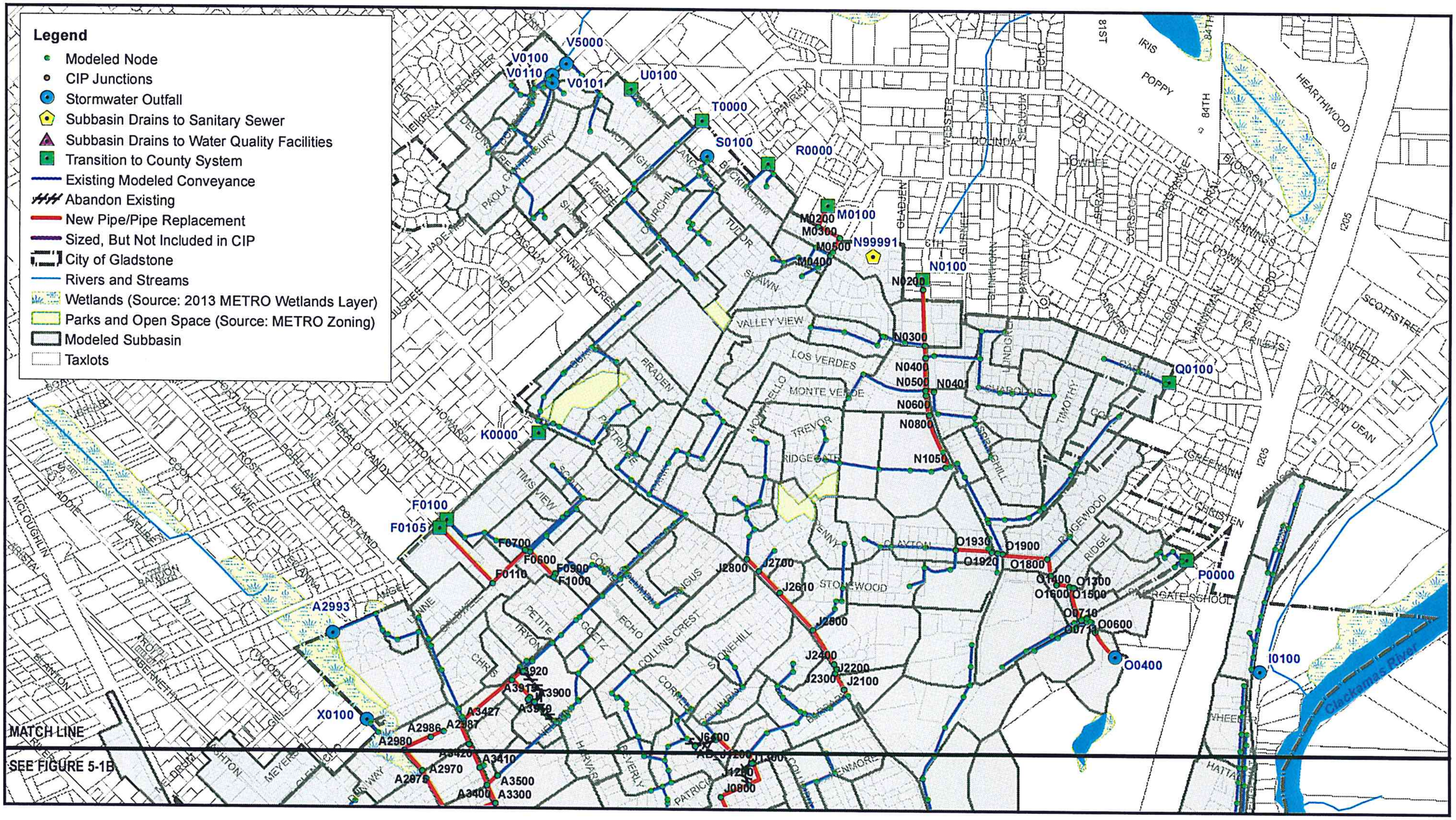
In order to integrate development of the flood control and water quality CIPs, the flood control and water quality opportunities were reviewed together to determine whether a water quality facility or CIP (to address a specific water quality opportunity area) could be sized, designed, and/or located in such a way that it would also address an identified system capacity deficiency.

Study Results

Analysis of the stormwater drainage system in Gladstone resulted in the selection of 18 CIPs. Of the 18 CIPs, five are integrated flood control and water quality CIPs. Eight of the CIPs address flood control only and five of the CIPs address water quality only. Table ES-1 summarizes the selected CIPs and Figures ES-1a and ES-1b provide the general location of each of these CIPs.

Table ES-1. CIP Estimated Cost Summary

CIP number	CIP name	Total cost ($)
A-1	Rinearson Creek Stream Enhancement	410,000
A-2	Portland Avenue Bypass and Upstream Improvements	5,790,000
A-3	High School Storm Drain Improvements and Detention	1,840,000
A-4	High School Rain Garden	12,000
A-5	Tryon Rain Garden	220,000
A-6	Glen Echo Pipeline Realignment	280,000
A-7	Meldrum Bar Bioswale	230,000
A-8	Riverdale Drainage Improvements	280,000
B-1	Basin B Drainage Improvements	270,000
F-1	Caldwell to Hull Pipe Replacement/Realignment	570,000
H-1	System H Channel Improvement	36,000
J-1	Cornell at Landon Pipe Replacement/Realignment	640,000
J-2	Oatfield Pipe Replacement	480,000
M-1	Crownview Drive Pipe Replacement	160,000
N-1	Kraxberger Middle School Bioswale and Pipe Replacement	940,000
N-2	System N Inlet Replacement	140,000
0-1	Ridgewood and Oatfield to Pond Pipe Replacement	650,000
0-2	Church Pond Retrofit	15,000
	Total	12,963,000
	Annual Green Streets Pilot Project	110,000/year



Brown AND Caldwell

September 2014

0 750 1,500 Feet

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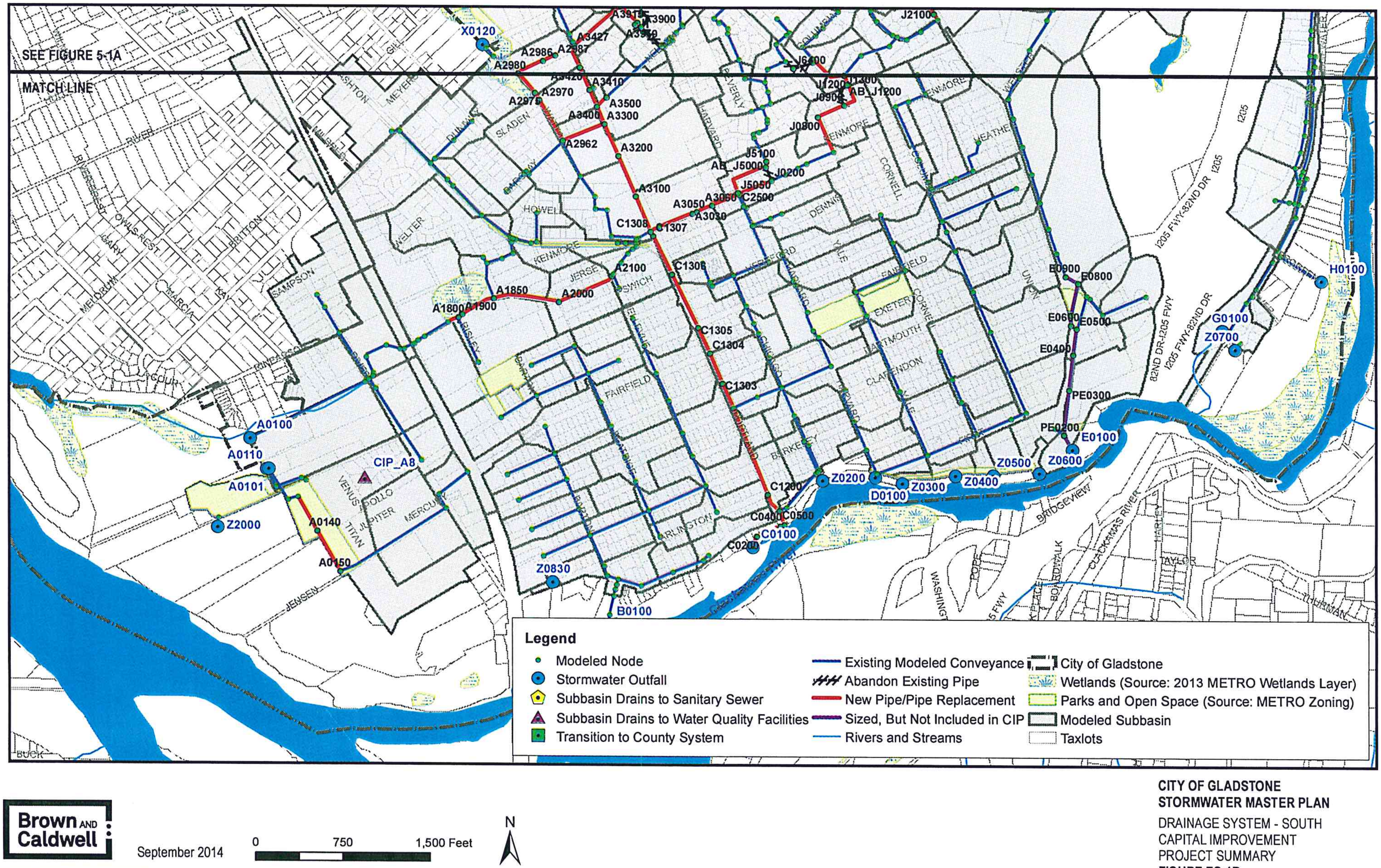
CITY OF GLADSTONE
STORMWATER MASTER PLAN

DRAINAGE SYSTEM - NORTH
CAPITAL IMPROVEMENT
PROJECT SUMMARY
FIGURE ES-1A



CITY OF GLADSTONE
STORMWATER MASTER PLAN

DRAINAGE SYSTEM - SOUTH
CAPITAL IMPROVEMENT
PROJECT SUMMARY
FIGURE ES-1B

Section 1

Introduction

This 2014 Gladstone Stormwater Master Plan (SMP) documents the methods and results of the storm system capacity evaluation and the stormwater quality/retrofit assessment conducted for the City of Gladstone, Oregon (City). The SMP identifies and prioritizes capital improvement projects (CIPs) to address identified system capacity deficiencies and water quality improvements. The SMP also identifies stormwater program implementation needs in the form of staffing and funding recommendations.

The study area includes land within the Gladstone city limits that drains to the Clackamas and Willamette rivers. A large amount of the City's stormwater conveyance system is also interconnected with the stormwater drainage system regulated by Clackamas County (County), the Oregon Department of Transportation (ODOT), and the Oak Lodge Sanitary District. As such, implementation of some identified improvements may depend on coordination with other agencies to ensure that backwater conditions from other regulated drainage systems do not exist.

This section provides a summary of the project need, the project objectives and approach, and a summary of how the SMP is organized.

1.1 Need for the Master Plan

The City of Gladstone has historically managed its stormwater collection and conveyance system with limited mapped and surveyed field information. To date, no stormwater master plan has been developed for the City and no stormwater capital improvement program is in place. As a result, management of the stormwater collection and conveyance system is conducted on an as-needed basis, primarily in response to failing/failed infrastructure. Additionally, without an identified stormwater capital improvement program, the City currently funds stormwater program and infrastructure improvements without a dedicated stormwater utility fee.

Since 1994, the City has operated under a Phase I National Pollutant Discharge Elimination System (NPDES) municipal separate storm sewer system (MS4) permit. In March 2012 the City was reissued its NPDES MS4 permit, which requires completion of a water quality retrofit assessment and identification of a water quality improvement project to be initiated during the permit term. The City's reissued NPDES MS4 permit also contains a specific requirement to develop an SMP by January 1, 2014, that identifies stormwater quality controls to reduce the discharge of pollutants from the MS4.

In 2012, the City began efforts to develop its SMP. As the first phase of the project, the City initiated full survey and mapping of its public stormwater collection and conveyance system. Results of the survey and mapping efforts have been used to analyze the capacity of the City's existing system and identify CIPs for water quality and water quantity control under this Plan.

The City's overarching goal for this master planning effort is to comprehensively evaluate the existing stormwater system, identify opportunities to improve water quality and system performance, and prioritize CIPs to be constructed on an implementation schedule.

1.2 Master Plan Objectives

This SMP is intended to help the City in the development, prioritization, and scheduling of a 30-year stormwater CIP. The objectives include the following:

- Conduct survey of the stormwater collection and conveyance system infrastructure within the city limits. Map the stormwater system in computer-aided design (CAD) and geographic information system (GIS) format.
- Compile surveyed system information into a comprehensive hydrologic and hydraulic (H/H) model for use in evaluating the capacity of the stormwater system and identifying capacity deficiencies.
- Interview City staff to develop an understanding of current system performance, function, and areas of concern.
- Identify water quality improvement projects and stormwater retrofits to address NPDES MS4 requirements.
- Develop CIPs and associated cost estimates to address water quality and identified system capacity deficiencies under existing and future development scenarios. Where feasible, flood control CIPs and water quality CIPs will be integrated into a single CIP to address multiple objectives.
- Review staffing needs in consideration of updated regulatory requirements and proposed CIP implementation.
- Provide information needed to develop a dedicated stormwater funding source.

1.3 Approach

The approach for developing the City's SMP is summarized in Figure 1-1. This approach was developed to meet the City's objectives, described above.

As shown in Figure 1-1, tasks associated with the SMP development were intended to be conducted in parallel with development of the water master planning effort, in order to optimize potential schedule efficiencies related to data collection and CIP development. However, stormwater system data collection efforts (survey and mapping) required additional time and resources due to site access limitations. As a result, to meet the Oregon Department of Environmental Quality (DEQ)'s January 1, 2014, compliance date for development of the SMP, stormwater master planning efforts were conducted independent of the water master planning.

At the time of the interim SMP submittal to DEQ, the staffing analysis and utility rate evaluation were not complete. This was communicated and confirmed with DEQ on July 16, 2013.

Specific to the stormwater master planning approach detailed in Figure 1-1, highlights of the approach are as follows:

1. The facility/system inventory effort (survey and mapping) was initiated at the beginning of the project (in 2012) but continued during the system evaluation and CIP development tasks in order to refine the H/H model.
2. CIP locations are identified to collectively address flood control and water quality/stormwater retrofit needs. Development of the comprehensive CIP includes water quality projects to meet NPDES MS4 permit requirements.
3. The staffing analysis is based on typical staffing levels in surrounding areas and reflects staff time needed to implement proposed projects.
4. The funding evaluation was initiated after CIP development, to ensure that the financial levels of service (LOS) analyzed correspond to specific program and project objectives.

Coordination with City staff is ongoing throughout the project duration in order to validate and verify assumptions related to the system configuration, reported maintenance issues and concerns, and CIP concepts and feasibility.

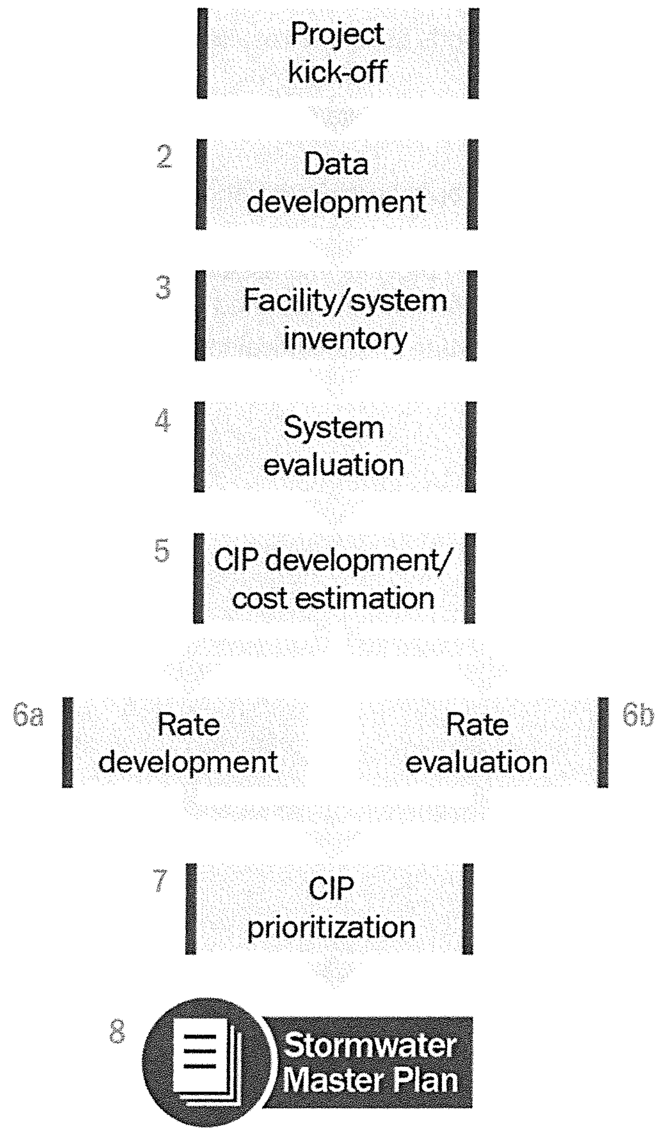


Figure 1-1. SMP

1.4 Master Plan Organization

This SMP is organized as follows:

- Section 2 includes a description of study area characteristics.
- Section 3 describes the modeling methods used and results of the storm system capacity evaluation.
- Section 4 describes the methods used and results of the storm system water quality evaluation/ stormwater retrofit assessment.
- Section 5 describes the integration of capital projects to address the City's storm system capacity and water quality needs.
- Section 6 describes the CIP prioritization process and assumptions related to CIP implementation to ensure that resources are available to implement identified CIPs over the next 30 years.
- Appendices A through D provide supporting and technical information used in the development of the SMP.

Section 2

Study Area Characteristics

This section includes an overview of study area characteristics including location, topography, soils, land use, rainfall, the drainage system, and current water quality conditions and regulations.

2.1 Location

The city of Gladstone is located in Clackamas County, Oregon (Figure 2-1), approximately 12 miles south of Portland, Oregon. The city is bordered by the Clackamas River to the south, the Willamette River to the west, unincorporated Clackamas County (Clackamas County Service District 1 or CCSD 1) to the north and east, and Oak Grove/Oak Lodge Service District to the north. Surrounding cities include West Linn (to the west), Oregon City (to the south), and Milwaukie (to the north).

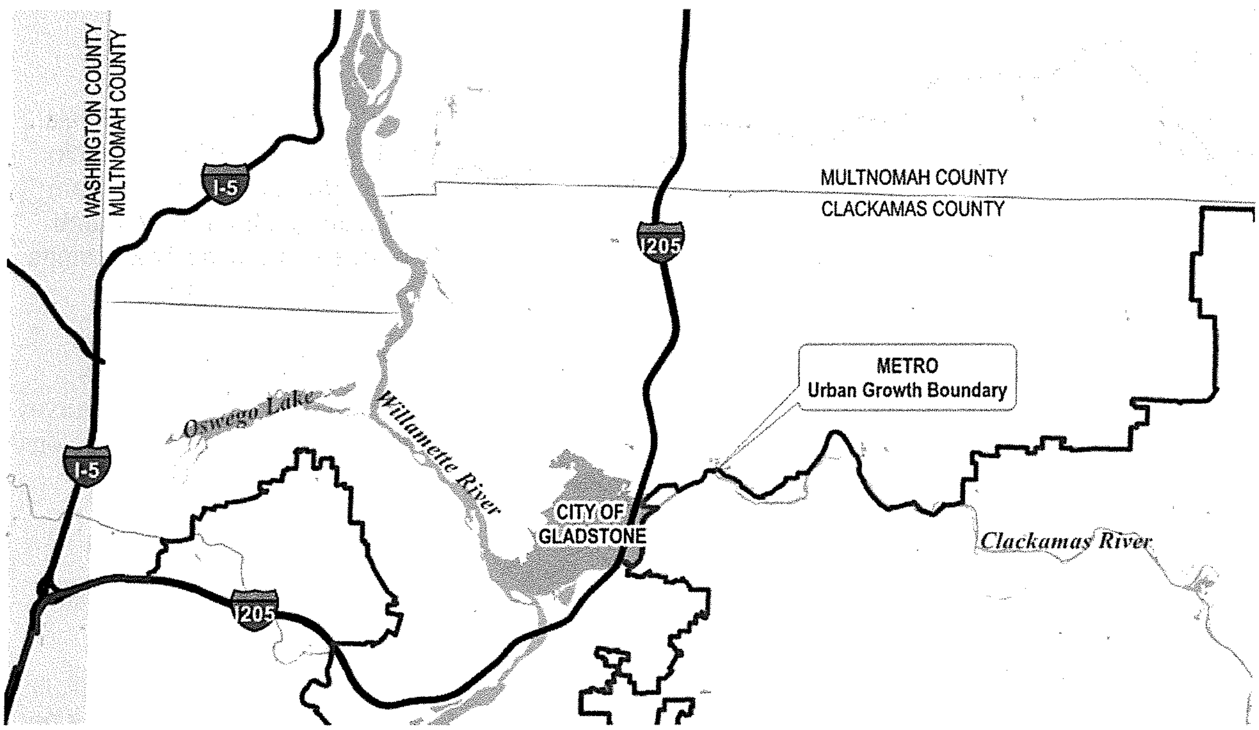


Figure 2-1. Vicinity map

The city is approximately 2.5 square miles in area. The Clackamas River, a major tributary to the Willamette River, flows along the southern border of the city. The Willamette River flows along the western border of the city. Rinearson Creek, a piped and open-channel tributary to the Willamette River, flows from east to west and divides the city in half, approximately. A majority of reported flooding problems are affiliated with the Rinearson Creek drainage system.

2.2 Topography

The topography in the city of Gladstone is influenced by the Clackamas and Willamette river drainage systems. As the Clackamas River runs west along the city's southern boundary to its confluence with the Willamette River, area from the southern portion of the city (approximately one third of the total city area) discharges to the Clackamas River, with elevations ranging from 10 to 300 feet.

Rinearson Creek, a tributary to the Willamette River, runs west in the middle of the city and is a combined piped and open-channel system. Drainage to Rinearson Creek is composed of a stormwater runoff and spring flow/overland flow. Rinearson Creek also runs through the Olson wetland system, located approximately 600 feet east of Oregon Highway 99E. Rinearson Creek discharges to the Willamette River at Meldrum Bar Park, a wetland/open recreation area in the city limits. Area from the central and western portions of the city (approximately half of the total city area) discharges to the Willamette River drainage system, with elevations ranging from 10 to 330 feet.

The northern and eastern portions of the city (approximately 20 percent of the total city area), discharge to the Clackamas County stormwater conveyance system. These portions of the city are elevated (average elevation range 120 to 330 feet).

Figure 2-2, located at the end of this section, illustrates the topography in the city of Gladstone.

2.3 Soils

The predominant soil types in the city of Gladstone are Xerochrepts-Rock outcrop complex (hydrologic soil group [HSG] C), Woodburn silt loam (HSG C), and Salem silt loam (HSG B). The Xerochrepts-Rock outcrop complex and Woodburn silt loam have slow soil permeability (HSG C) and the Salem silt loam has moderate soil permeability (HSG B). A majority of the HSG C soils are located in the northern and eastern portions of the city. A significant amount of bedrock outcrop, also located in these areas, limits infiltration. A majority of HSG B soils are located in the southern and western portions of the city.

Soil classification is an important characteristic to consider when determining runoff flow rates and volumes. Soil type within the study area was identified using data from version 7 of the Clackamas County Soil Survey, updated in 2012 by the Natural Resources Conservation Service (NRCS) Soil Survey. The NRCS HSG classification was used to assign pervious area runoff curve numbers (CN) for hydrologic calculations. Figure 2-3 shows the distribution of HSG within the city.

2.4 Land Use

The city of Gladstone is primarily developed, with only about 10 percent of the city area identified as vacant. Vacant lands are scattered throughout the city, and a majority are publicly held.

Single-family residential development is the primary land use within the city, with more dense (eight dwelling units per acre) located in the southern portion of the city and less dense (six dwelling units per acre) located in the northern portion of the city. A significant amount of commercial development is located along Oregon Highway 99E, along Portland Avenue, and at the intersection of 82nd Avenue and Interstate 205. Other land use categories include multifamily residential, industrial, and parks and open space.

For development of this SMP, zoning coverage and vacant lands coverage provided by Metro were used to assign the impervious area percentages applicable to future development (buildout) conditions for hydrologic modeling. All vacant lands are assumed to be developed according to zoning for future conditions.

Figure 2-4, at the end of this section, shows the zoning coverage within the city of Gladstone.

2.5 Climate and Rainfall

The city of Gladstone experiences a similar temperate climate to the surrounding Portland metropolitan area, with relatively warm, dry summers and mild, wet winters. Winter temperatures average approximately 40 degrees Fahrenheit (F) and summer temperatures average approximately 70 degrees F.

The average annual precipitation for the Portland metropolitan area ranges from 37 to 43 inches, with most of the rainfall occurring between November and April.

2.6 Drainage System

As part of this SMP development, the City's storm drainage system was surveyed and mapped. Based on the obtained survey information, the City's drainage system is composed of approximately 30 miles of City-owned pipe and major open-channel conveyance system, 299 manholes (nodes), and more than 1,000 catch basins and cleanouts. Approximately 21 miles of pipe and open channel were modeled as part of this SMP, composed primarily of 12-inch-diameter pipe and greater.

The City does not currently own and operate any regional detention facilities within the city limits. Therefore, no detention pipe or ponds were modeled as part of the evaluation. Private detention facilities are currently located within the city limits, but no as-built information was available at this time so such facilities were not included in the evaluation.

Survey of the City's drainage system defined 26 major basins, reflecting 32 modeled pipe system outfalls: 13 to the Clackamas County stormwater system (piped or open channel); 11 to natural areas within Meldrum Bar Park, the Olson Wetlands, Glen Echo Wetlands, or Boardman Creek (all within the Willamette River drainage area), 5 to the Clackamas River, and 3 to natural areas adjacent to the Clackamas River. Major basin Z has additional outfalls that do not contain a significant piped conveyance system and therefore were not included in the modeling effort.

Subbasins were delineated based on obtained survey information and aerial imagery. Several subbasins were delineated to be included in the hydrologic modeling effort only, as they have limited piped infrastructure. Several subbasins, where stormwater runoff enters the receiving water directly and does not enter a conveyance system at all, were also not reflected in the hydrologic or hydraulic modeling efforts.

The drainage system was originally surveyed and maps were developed in AutoCAD. Information from AutoCAD was then imported into GIS for purposes of ongoing mapping support and development of the hydraulic model for evaluation of system capacity.

Figure 2-5 (a and b), located at the end of this section, shows the modeled stormwater drainage system including pipes and open channel. Figure 2-5 (a and b) also shows the subbasin delineation. The AutoCAD map book containing the major basin delineation is provided as Appendix A.

2.7 Stormwater Quality and Regulatory Drivers

DEQ is responsible for implementing provisions of the federal Clean Water Act (CWA) pertaining to stormwater discharge and surface water quality. DEQ also conducts permitting for activities that discharge to surface waters, establishes water quality criteria for water bodies based on designated beneficial use, and conducts water quality assessments and evaluations to determine whether a water body adheres to water quality standards.

2.7.1 NPDES MS4 Permit

The City was reissued its Phase I NPDES MS4 permit on March 16, 2012. The City's reissued NPDES MS4 permit contains a variety of requirements to address the following categories/activities and improve the quality of stormwater discharge to receiving waters:

- Illicit Discharge Detection and Elimination (IDDE)
- Industrial and Commercial Facilities
- Construction Site Runoff Control
- Public Education and Outreach
- Public Involvement
- Post-Construction Site Runoff Control
- Pollution Prevention for Municipal Operations
- Stormwater Management Facility Operations and Maintenance

Implementation of its NPDES MS4 permit is described in the City's SWMP (effective date May 2012). The SWMP includes measurable goals, responsible parties, and tracking measures to assess progress of implementing the activities (best management practices [BMPs]) to address requirements. The NPDES MS4 permit and the City's SWMP require the City to select, design, install, and maintain structural stormwater facilities for water quality improvement.

In addition to the implementation of the SWMP for water quality improvement, DEQ included a specific provision in the NPDES MS4 permit for Gladstone to complete and submit a SMP by January 1, 2014. DEQ is also requiring all co-permittees to conduct a stormwater retrofit assessment by July 1, 2015, to identify areas in the city underserved or lacking structural stormwater facilities. Both the SMP and stormwater retrofit assessment are intended to identify stormwater quality controls to reduce the discharge of pollutants from the MS4. This SMP was developed to address DEQ's requirements related to development of a SMP and stormwater retrofit assessment.

2.7.2 TMDL and 303(d) Requirements

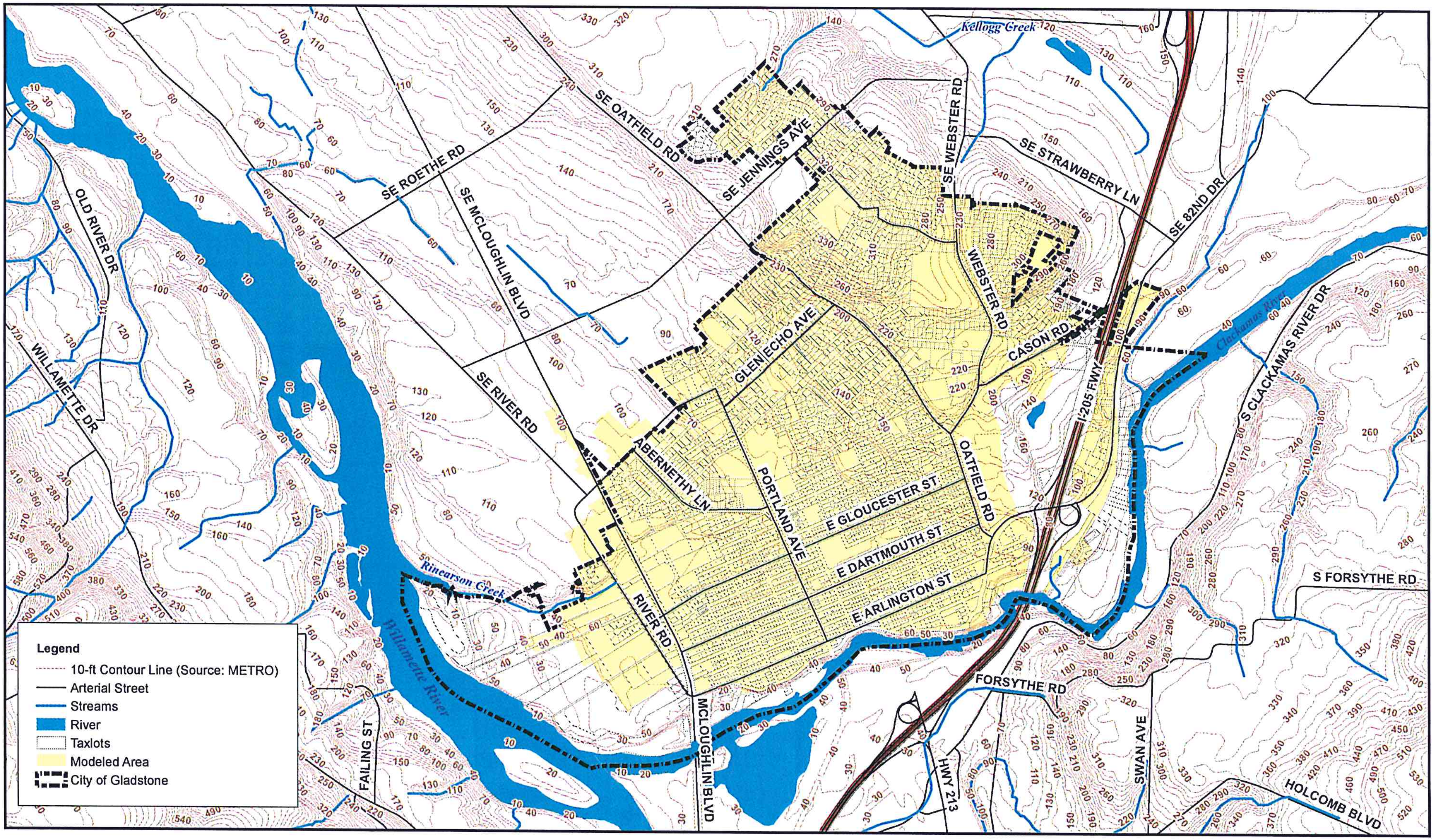
Section 303(d) of the CWA requires states to develop a list of water bodies that do not meet water quality standards. DEQ develops such a list for Oregon, which is used to identify and prioritize water bodies for development of a pollution reduction plan or total maximum daily load (TMDL). TMDLs identify the assimilation capacity of a water body for a particular pollutant and establish pollutant load allocations for sources of discharge to such water body.

Table 2-1 identifies the 303(d) parameters and TMDLs that are applicable to the City. The Willamette River TMDL includes Rinearson Creek as a tributary.

Table 2-1. Summary of TMDL and 303(d) Listed Streams for Gladstone									
Monitored water body	Bacteria	Temperature	Mercury	PCBs	PAHs	DDE/DDT	Dieldrin	Iron	Manganese
TMDLs									
Willamette River (and tributaries) (2006)	✓	✓	✓						
Clackamas River (2006)	✓	✓	✓						
Additional 303(d) listed streams/parameters									
Willamette River (lower) and tributaries (2010)				✓	✓	✓	✓	✓	✓
Clackamas River (2010)				✓	✓	✓	✓	✓	✓

TMDL and 303(d) requirements are integrated into the City's implementation of its NPDES MS4 permit. At the end of the permit term, the City's NPDES MS4 permit requires calculation of TMDL pollutant load reduction benchmarks, to show progress toward meeting applicable TMDL requirements. Such progress is observed through implementation of structural stormwater facilities and pollutant source control measures (e.g., public education, street sweeping, etc.) that are targeted at addressing TMDL pollutants.

Given the limited development activities within the city, and because a majority of the city is built out with limited available area for large stormwater treatment or detention facilities, few structural stormwater facilities are located within the Gladstone city limits. This SMP provides a list of water quality improvement projects to promote water quality improvement specific to TMDL parameters.



Brown AND Caldwell

January 1, 2014

0 1,500 3,000 Feet

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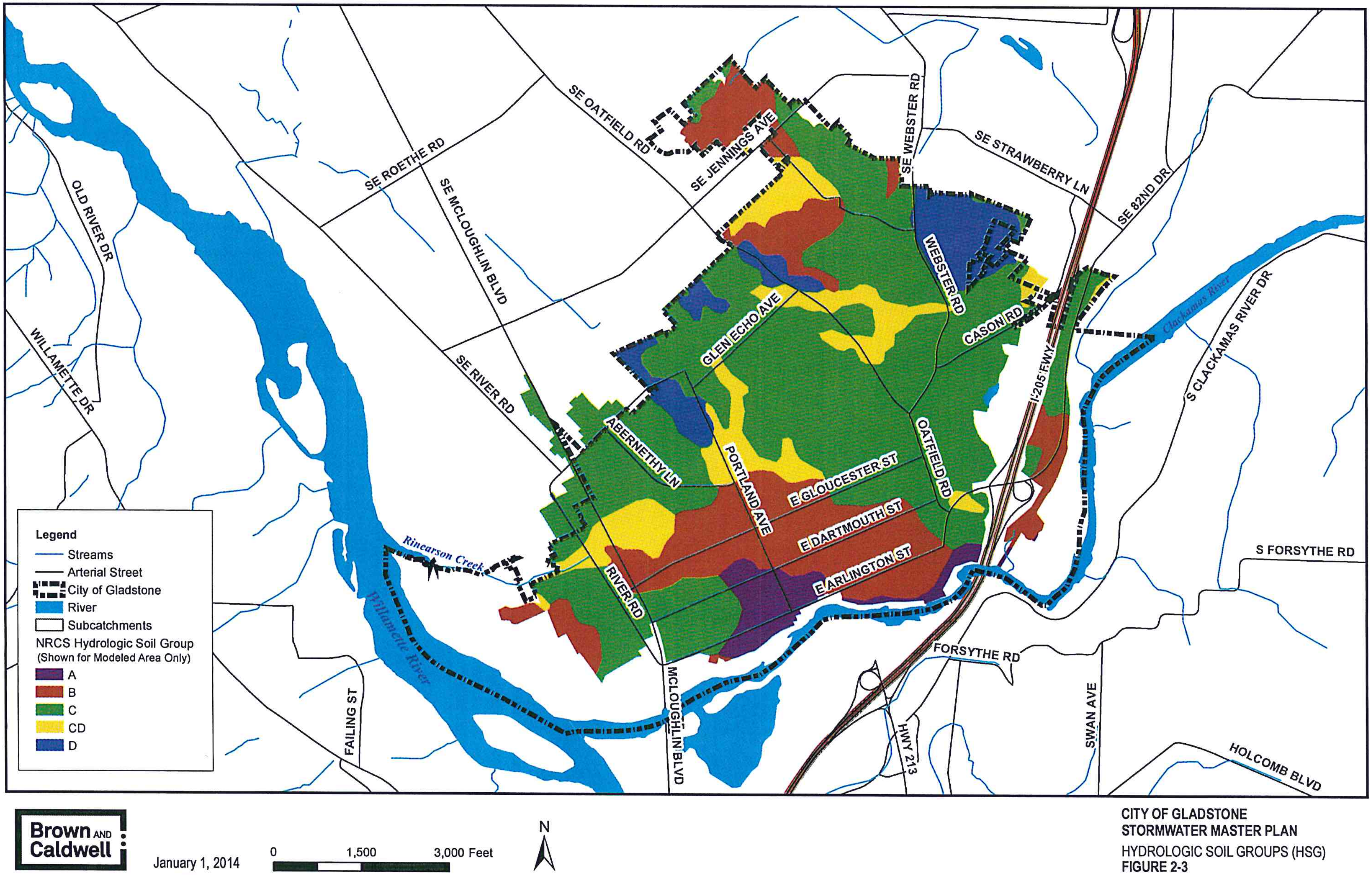
CITY OF GLADSTONE
STORMWATER MASTER PLAN

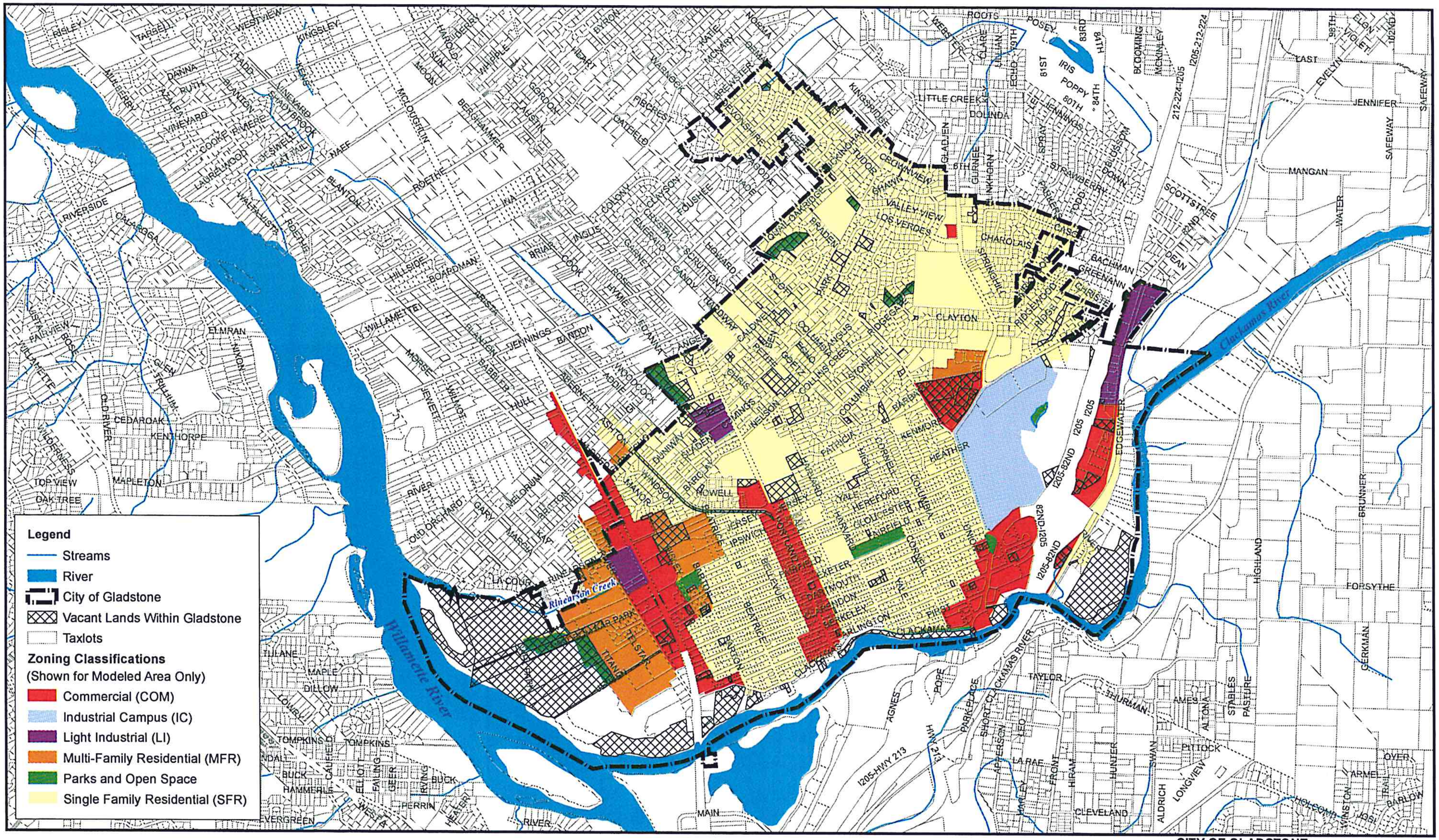
TOPOGRAPHY
FIGURE 2-2



Brown AND Caldwell

January 1, 2014

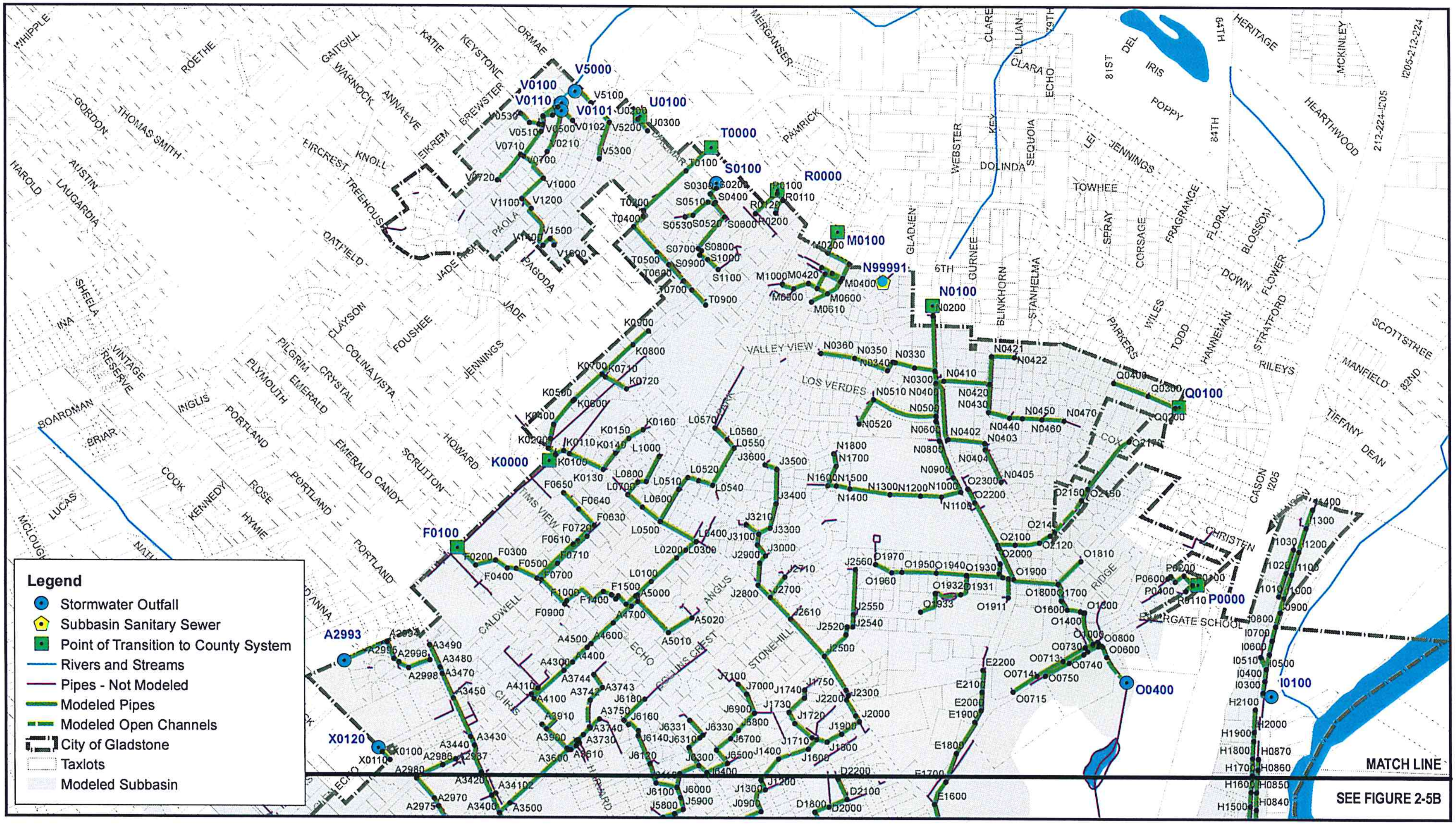
CITY OF GLADSTONE
STORMWATER MASTER PLAN
HYDROLOGIC SOIL GROUPS (HSG)
FIGURE 2-3



Brown AND Caldwell

January 1, 2013

CITY OF GLADSTONE
STORMWATER MASTER PLAN
METRO ZONING AND VACANT
LANDS INVENTORY
FIGURE 2-4



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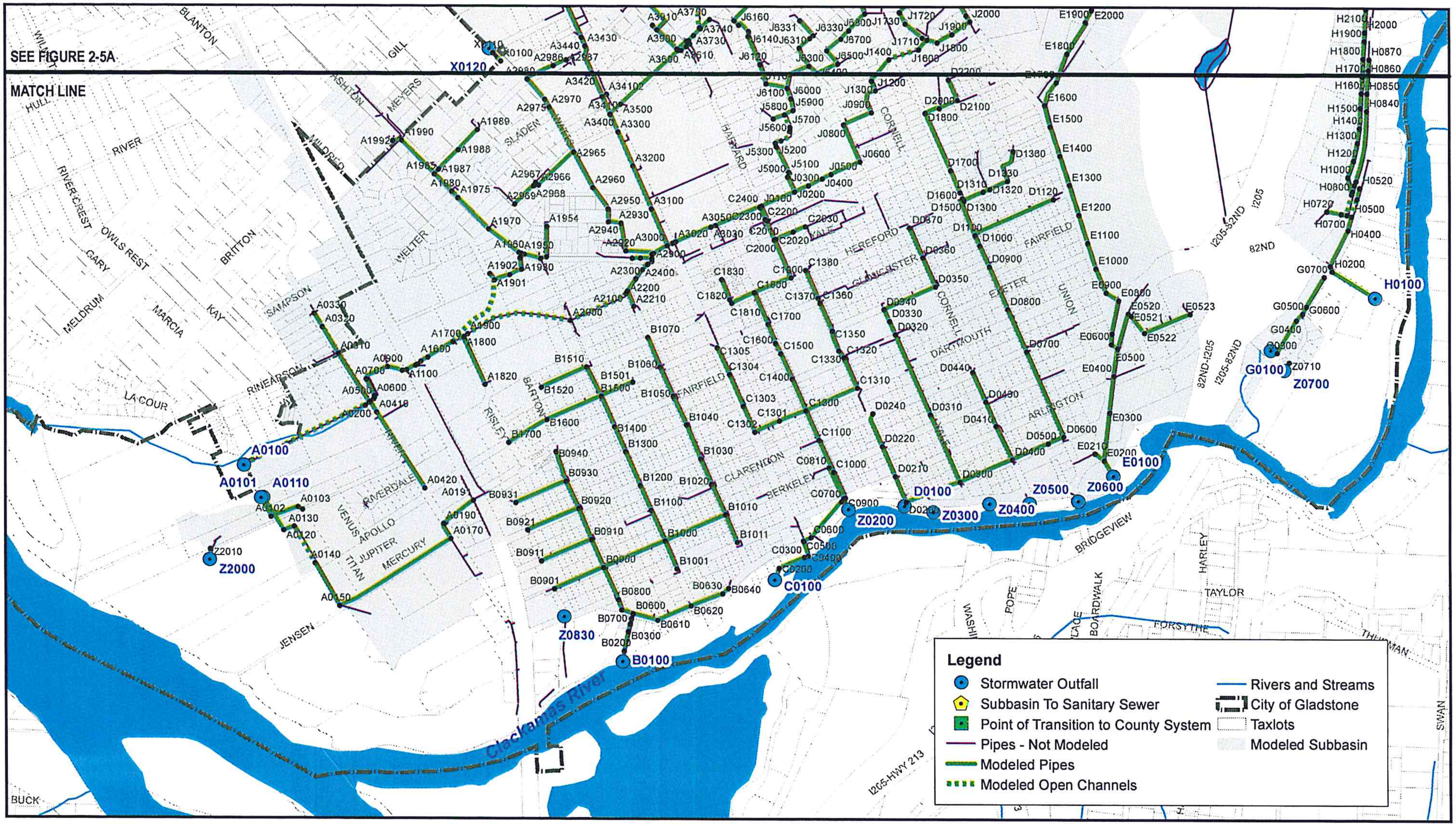
January 1, 2013

CITY OF GLADSTONE
STORMWATER MASTER PLAN

DRAINAGE SYSTEM - NORTH
FIGURE 2-5A



Brown AND Caldwell

January 1, 2013

CITY OF GLADSTONE
STORMWATER MASTER PLAN

DRAINAGE SYSTEM - SOUTH
FIGURE 2-5B

Section 3

Storm System Capacity Evaluation

To identify flooding problems and opportunities for CIPs, the City's public stormwater drainage system was evaluated using an H/H model. The stormwater drainage system was evaluated under existing and future development scenarios. This section provides a description of H/H modeling methods used for the system capacity evaluation and provides a summary of results.

3.1 City of Gladstone Study Area

The total study area evaluated for the Gladstone SMP is approximately 1,227 acres and includes a majority of area within the city limits. The study area does exclude area along the southern city boundary that directly discharges to natural areas and/or receiving waters with very little to no public infrastructure or stormwater conveyance system. Some areas outside of the city limits, specifically along Highway 99E/SE McLoughlin Boulevard, were included in the model because stormwater runoff from these areas discharges to the City's stormwater conveyance system.

The majority of the study area (approximately 80 percent) is collected and conveyed in a pipe or open-channel system to the Willamette River in the west and Clackamas River in the south. Areas along the north and east city boundary primarily discharge to stormwater infrastructure owned and operated by other agencies.

3.2 Model Development

Computational Hydraulics International's PC SWMM 2012, v. 5.0.022, was used to develop the H/H model for the City's storm system. PC SWMM 2012 provides a graphical interface for the U.S. Environmental Protection Agency (EPA) SWMM5 engine. The PC SWMM interface is integrated with ESRI ArcGIS. Model files provided to the City will be in EPA SWMM5 format, which can be used by the City for internal modeling after completion of this SMP.

The model of the City's storm drain system includes City-owned storm drainage pipes 12 inches and larger in diameter and major open-channel conveyance. Inlet leads, pipes smaller than 12 inches in diameter, and pipes not owned by the City were not included in this effort. System-wide mapping was completed following survey of the City's stormwater drainage system and was used for model development.

Model development requires input of meteorological (precipitation) data, subbasin hydrologic data, and surface water system hydraulic data. Precipitation data were input as design storms to evaluate capacity of the conveyance system. Input parameters associated with subbasin hydrology include soil infiltration and land use/zoning characteristics and were confirmed through review of aerial photographs. Input parameters associated with conveyance system hydraulics were developed based on information collected during the field survey effort.

Details related to model input parameters and modeling methods are described in the following sections:

Section 3.2.1: Stormwater System Survey

Section 3.2.2: Meteorological Data (e.g., rainfall)

Section 3.2.3: Hydrologic Data (e.g., area, impervious area [as a percent], soil infiltration parameters)

Section 3.2.4: Hydraulic Data (e.g., pipe size, material, length and invert elevations)

3.2.1 Stormwater System Survey

In September 2012, Sisul Engineering began conducting a system-wide survey of the public stormwater collection and conveyance system within Gladstone. The survey effort included horizontal location of all storm drain manholes, cleanouts, catch basins, culverts, and outfalls. Measure-downs were conducted for all manholes and in-line catch basins and included flow direction, diameter, and depth to the invert of all pipes.

The survey was conducted using a combination of global positioning system (GPS) survey, topographic survey, Light Detection and Ranging (LiDAR) data, field measurements, a review of construction and record drawings, and meetings with Public Works personnel. A combination of GPS survey and topographic survey data was used to locate the horizontal and vertical positions of manhole and inlet rims, culverts, and pipe outfalls. LIDAR data were used to verify structure elevations in some instances. Storm drain manholes and flow-through catch basins with significant upstream drainage basins were each inspected and a record was made of the number of pipes connected to each structure. The depth and diameter of each pipe was measured and the type of pipe material was recorded.

The stormwater survey effort for purposes of the SMP was completed in June 2013. The focus was on the system in the public right of way. Although the majority of the system has been inventoried, several features were found to be located on private property and were not able to be inspected or surveyed due to the lack of access. In the future, the City may wish to pursue access to these areas in order to complete the system inventory. Appendix A mapping reflects the survey work that was completed for this plan.

3.2.1.1 Horizontal and Vertical Datum

All reported elevations and coordinates in the survey effort are measured in feet and use the North American Vertical Datum of 1988 (NAVD88) and the North American Datum of 1983 (NAD83) state plane coordinate systems, respectively.

3.2.1.2 System Nomenclature

The city was divided into 26 major drainage basins (A through Z) to categorize data from the stormwater system survey. The majority of the drainage basins discharge to a single outfall; however, there are basins with multiple outfalls due to diverging pipe and split flow. Major drainage basin Z was reserved for drainage basins that discharge directly to a natural area or receiving water with little or no upstream public infrastructure.

The system nomenclature provides a unique identifier (ID) for each system node (manhole, catch basin, cleanout, outfall). The first character of the ID is the major drainage basin letter associated with the node, and the remainder of the ID is a 4- or 5-digit number. The numbers increase from downstream in the system (i.e., the outfall) to upstream.

The major drainage basins are subdivided into subbasins in accordance with location-specific drainage patterns. Each subbasin is named per the system node where runoff from the subbasin is directed in the model, also known as an inlet node.

3.2.1.3 Stormwater System Survey Challenges

The survey effort lasted 9 months, which was considerably longer than anticipated. The extended duration of the system survey effort was partially due to the alignment of many City-owned pipes on private property, which required review of as-built information and access to private property. In locations listed below, as-built information was unavailable and access to the private property for purposes of obtaining survey information was not possible. As a result, invert data were interpolated based on known upstream and downstream inverts at these locations in order to fill in data gaps:

- System D: Kelsey Court to Hereford Street (node D1360 to node D1330)
- System F: Cornell Place to Franklin Way (node F1000 to node F1400)
- System J: High Street to Patricia Drive (node J6000 to node J5500)
- System N: Trevor Court to Ridgegate Court (node N1800 to node N1600)
- System O: Ridgewood Drive to Ridge Drive (node 01800 to node 01500)
- System V: Canterbury Drive to Doncaster Drive (node V1000 to node V0700)

In addition, the stormwater system survey extended only to the city limits. Gladstone-owned stormwater pipes discharge to the County-owned and -operated stormwater system at the city limits in the following locations:

- System F: Hull Avenue and Hardway Court
- System K: Hull Avenue and Oatfield Road
- System M: Between Strawberry Lane and Valley View Drive, north of Crownview
- System N: Webster Road between 5th Avenue and 6th Avenue
- System P: Cason Road, west of Cason Circle
- System Q: Cason Lane, west of Cason Court
- System R: Between Kingsridge Court and Buckingham Drive
- System T: Jennings Avenue and Catlyn Woods Drive
- System U: Dagmar Road and Londonderry Lane

For locations where the City's stormwater system discharges to the County system, there is limited information available from the County. Downstream flow conditions may impact the City's system. As a result, CIP development within these major basins needs to include coordination with the County to ensure that backwater effects are not present, which would limit the functionality of the CIP.

3.2.2 Meteorological Data

Traditional design storms are precipitation patterns typically used to evaluate the capacity of storm drainage systems and design capital improvements for the desired level of flood protection.

Design storms evaluated for this study included the 2-year, 10-year, 25-year, and 100-year 24-hour-duration design storms.

The rainfall depths for these design storms were based on isopluvial maps published in the National Oceanic and Atmospheric Administration (NOAA) Atlas 2, Volume X. The rainfall distribution for these design storms is based on the Soil and Conservation Service (SCS) 24-hour, Type 1A distribution, which is applicable to western Oregon, Washington, and northwestern California. Table 3-1 lists the precipitation depths for each design storm used in the model.

Table 3-1. Design Storm Depths	
Design storm event	Rainfall depth, inches
Water quality storm	1.0
2-year, 24-hour	2.4
10-year, 24-hour	3.4
25-year, 24-hour	3.9
100-year, 24-hour	4.7

3.2.3 Hydrologic Data

This section includes a summary of the input parameters used to define hydrologic characteristics of the subbasins. Table 3-2 identifies model input parameters identified for each subbasin. A description of each parameter is provided below. Appendix B, Table B-1 provides hydrologic input parameters and peak flows calculated for each subbasin and modeled design storm.

Table 3-2. Hydrologic Input Parameters	
Attribute	Value
Name/outlet	Identified by the subbasin inlet node
Area	Area of the subbasin (acres)
Width	Width of the overland flow path for sheet flow (feet)
Slope	Average slope of the subbasin (as a percent)
Imperv	Average percent of land area that is directly connected impervious area (as a percent)
Nimperv	Manning's roughness coefficient for overland flow on impervious area (default value = 0.012)
Nperv	Manning's roughness coefficient for overland flow on pervious area (default value = 0.24)
CurveNo	Assigned by subbasin, based on an area-weighted average of imperviousness and the HSG

3.2.3.1 Subbasin Delineation and Area

As described in Section 3.2.1.2, major drainage basins are subdivided into subbasins in accordance with location-specific drainage patterns. The subbasin areas were calculated using GIS.

3.2.3.2 Subbasin Width and Slope

Both the subbasin width and slope are calculated in GIS based on the subbasin delineation and digital topographic information. The subbasin width is the maximum width of the overland flow path within each subbasin (prior to flow entering a channelized conveyance or pipe). The subbasin slope is the average slope along the pathway of overland flow to the inlet of the drainage system.

3.2.3.3 Subbasin Impervious Percentage

Effective impervious percentage is the portion of impervious area that is directly connected to the drainage collection system. For example, curb-and-gutter streets are directly connected to the drainage collection system and represent "effective impervious area." However, a sidewalk that is separated from the street by vegetation is not considered to be directly connected because runoff has the opportunity to infiltrate. This area would be considered "non-effective impervious surface." The City does not have citywide specific information for impervious surface (effective or non-effective) so for purposes of this

SMP, impervious estimates are based on aerial imagery and zoning coverage, which assumes that the impervious area in a subbasin would vary depending on land use.

To estimate the subbasin impervious percentage based on current development conditions, aerial imagery was reviewed and impervious area was estimated based on the presence of street and paved surfaces and rooftop areas.

Future development conditions assume the city is fully built out. As described previously, there is currently limited vacant area in the city that would develop in the future. However, redevelopment activities such as street improvements and infill development are assumed to occur during full buildout. Street improvements typically increase the "effective impervious area" to the storm drainage system. Currently, many areas of the city lack curb-and-gutter streets, but street improvements would add curb and gutter. Infill redevelopment activities typically include construction of larger, newer houses on the same size lot as the original, smaller house. Collectively, these redevelopment activities increase the amount of impervious surface and the connectivity of the impervious surface.

To estimate the subbasin impervious percentage based on future development conditions and address the potential for fully connected, effective impervious surface throughout the city, an area-weighted impervious percentage was calculated for each subbasin using the zoning-based impervious percentages (Table 3-3).

Table 3-3. Future Condition Impervious Percentage by Zoning Classification

Land use	Abbreviation	Average impervious percentage
Single-family residential	SFR	55
Multifamily residential	MFR	75
Commercial	COM	90
Industrial campus	IC	50
Light industrial	LI	90
Parks and open space	POS	7

3.2.3.4 Time of Concentration

The time of concentration is the time for runoff to travel from the most distant point of the watershed to the point in question. The time of concentration is computed by summing all the travel times for consecutive components of the drainage system (i.e., sheet flow, shallow concentrated flow, open-channel flow, and pipe flow). The time of concentration for each subbasin is calculated using the digital topographic information contained in the GIS.

3.2.3.5 Curve Number

The curve number (CN) is a dimensionless number that depends on HSG, cover type (zoning or land use), and antecedent moisture conditions (AMC). The CN method is the hydrologic method used to model runoff characteristics. Because each subbasin contains multiple land uses and soil types, an area-weighted CN was calculated for each subbasin in the current and future development scenarios. This method is documented in EPA *Technical Release 55.*

3.2.4 Hydraulic Data

This section describes the model input parameters used to characterize the hydraulic characteristics of the system.

System hydraulic components including modeled conduits (pipes and open channels) and nodes (manholes, catch basins) are based on survey data described in Section 3.2.1. Where needed, survey data were supplemented with LiDAR information for ground surface elevations, site visits, and aerial imagery. A description of the hydraulic components is provided below. Appendix B, Table B-2, provides peak flow and maximum water surface elevation calculated for each modeled node and modeled design storm.

3.2.4.1 Nodes

Model nodes include manholes, catch basins, and other relevant connection points or locations where a conduit direction, slope, material, or size changes. Model nodes have the attributes (input parameters) as listed in Table 3-4 and described below.

Table 3-4. Model Node Input Parameters

Attribute	Value
ID	Unique identifier or naming convention as described in Section 3.2.1.2, System Nomenclature.
Invert elevation	Lowest invert elevation of conduits entering or exiting the node (feet). Typically reflects the outlet conduit.
Depth	Depth (feet) = rim elevation – invert elevation.
Ponded area	Area occupied by ponded water atop the node after flooding occurs in square feet (ft^2). The model allows ponded water to be stored and subsequently returned to the drainage system when capacity exists.

For each node, ground elevation information was obtained during the system survey described in Section 3.2.1 and confirmed through comparison with contour information developed from LIDAR. Invert elevations were established based on measure-down information collected during the survey effort. The depth of each node is based on the difference between calculated rim elevation and invert elevation of the outlet pipe from the node.

3.2.4.2 Modeled Conduits

Modeled conduits include pipes, culverts, and channels. Model conduits have the attributes (input parameters) as listed in Table 3-5 and described below.

Table 3-5. Conduit Input Parameters

Attribute	Value
ID	Upstream Node ID_Downstream Node ID
Length	Length between upstream and downstream nodes (feet).
Roughness	Varies based on material. See Table 3-6. Manning's roughness coefficient.
Cross-section	Varies. Either circular, arch, or trapezoidal.
Size	Varies. Pipe diameters (feet). Open channels in depth and width measurements.
Inlet elevation	Elevation of conduit inlet (feet).
Outlet elevation	Elevation of conduit outlet (feet).

Pipe shape (cross-section), size, and material assumptions are based on survey information collected. Pipes of 12-inch diameter and greater were included in the model. Table 3-6 summarizes the Manning's roughness coefficient "n" assumed for each conduit material.

Table 3-6. Manning Roughness Coefficients

Material	Manning's n
Concrete pipe	0.013
Corrugated metal pipe	0.024
Plastic (HDPE)	0.0125
Plastic (PVC)	0.0125
Open channels	0.04–0.06

Open-channel dimensions not collected as part of the survey effort were developed using Gladstone 2004 LiDAR data, developed by the Oregon Department of Geology and Mineral Industries. LiDAR was also used to refine the longitudinal slope of the open-channel system, and field visits were conducted to confirm the side slopes and bottom widths of the open-channel segments. Custom cross-sections were developed using LIDAR data for Rinearson Creek because it is an irregular channel with a non-uniform shape. Cross-sections for irregular channels are called "transects" in PC SWMM and can be found in the model files.

3.2.4.3 Outfalls

The study area includes 32 modeled outfalls to represent discharge from each major basin and modeled conveyance system. As described in Section 3.2.1.3, nine of the modeled outfalls are actually points of transition to the piped stormwater conveyance system owned and operated by Clackamas County. Modeled outfalls have the attributes (input parameters) described in Table 3-7. The modeling effort did not include evaluation of backwater conditions from areas or conveyance systems downstream of the outfalls.

Table 3-7. Outfall Input Parameters

Attribute	Value
ID	Unique identifier as described in Section 3.2.1.2, System Nomenclature
Invert elevation	Invert elevation of the outfall (feet)
Rim elevation	Ground surface elevation at the outfall (feet)
Type	Varies based on the outfall configuration and flow condition; options used include: • FREE: outfall stage is determined by minimum of critical flow depth and normal flow depth in the upstream conduit • FIXED: outfall stage is set to a fixed water surface elevation equal to the top of the outfall pipe

3.3 Drainage Standards

The *Stormwater Treatment and Detention Standards for the City of Gladstone* (Gladstone Standards) are referenced under Chapter 17.56 of the Gladstone Municipal Code. The Gladstone Standards were referenced for general design criteria related to stormwater conveyance system and stormwater treatment sizing for this SMP. The Gladstone Standards reference Section 5.2.1 of the CCSD 1 *Surface Water Management Rules and Regulations* for collection system sizing and Section 5.3 of the CCSD 1 *Surface Water Management Rules and Regulations* for water quality treatment.

As documented under Section 5.2.1 of the CCSD 1 *Surface Water Management Rules and Regulations* (2005), stormwater collection system sizing should be based on the following:

- storm sewer outfalls draining less than 640 acres: 25-year, 24-hour design storm
- storm sewer outfalls draining greater than 640 acres: 50-year, 24-hour design storm
- open channels draining less than 250 acres: 25-year, 24-hour design storm
- open channels draining greater than 250 acres: 50-year, 24-hour design storm
- open channels draining greater than 640 acres: 100-year, 24-hour design storm

CCSD 1 developed draft updates to its collection system design criteria, as documented in the draft Clackamas County Water Environment Services *Stormwater Management Design Standards* (2010). Such updates are expected to be adopted by Gladstone and are consistent with the bulleted criteria above. For purposes of CIP sizing and design, stormwater collection and conveyance piping was sized for a 25-year, 24-hour storm event. As required in the Gladstone Standards, CIP design reflects use of a minimum pipe diameter of 12 inches within the public right-of-way (ROW).

As documented under Section 5.3 of the CCSD 1 *Surface Water Management Rules and Regulations* (2005), stormwater treatment facilities (i.e., rain gardens, planters, etc.) need to be sized for two-thirds of the 2-year, 24-hour storm event. The City of Gladstone, along with other Clackamas County jurisdictions, participated in a collective effort to define the water quality design storm in accordance with conditions of its NPDES MS4 permit, which requires treatment of 80 percent of the average annual runoff. Such a design storm was identified as 1 inch over 24-hour storm. The 1 inch over 24-hour design storm was used in the sizing of select water quality treatment facilities.

3.4 Model Results

PC SWMM v. 2012 was used to simulate the 2-year, 10-year, 25-year, and 100-year design storms for the current and future development conditions.

Results of the H/H simulations are tabulated in Appendix B (Table B-1 for hydrologic results and Table B-2 for hydraulic results). For reporting purposes, the hydrologic results reflect peak flow calculations for all design storms. The hydraulic results tables reflect just the 10-year flows used to prioritize capacity deficiencies and the 25-year flows used to size CIPs.

The hydrologic results table (Table B-1) is sorted by major basin and modeled inlet node and includes the corresponding subbasin name, subbasin area, CN, impervious percentage, and associated design flow. The hydraulic results table (Table B-2) is also sorted by major basin and outfall and includes the conduit name, upstream and downstream node ID, conduit length, conduit size, invert and rim elevations, and the 10-year peak flow and 25-year peak flow and water surface elevations.

3.4.1 Initial Identification of Flooding Problems

Based on the hydraulic model results summarized in Table B-2, conduits experiencing backwater conditions that result in flooding of the upstream node were identified. Flooding of the upstream node is indicated by the loss of runoff volume in the closed-conduit system. For open-channel segments, flooding was identified by water overtopping the banks. Again, the 25-year, 24-hour design storm was initially used to identify flooding in the storm drain system. Evaluation of the 10-year, 24-hour design storm was used to prioritize the flooding conduits. Table B-2 also indicates during which design storm and development scenario flooding occurs.

Due to the extensive flooding indicated by the hydraulic model, and the number of conduits experiencing flooding during the 10-year, 24-hour storm event, additional conduit prioritization was conducted. As shown in Figure 3-1 (a and b), the duration of flooding during the 10-year, 24-hour storm event (for piped conduits) and 25-year, 24-hour (for Rinearson Creek) was calculated. Conduits experiencing flooding for greater than 2 hours for the identified design storm were prioritized for CIP development

For purposes of reporting results and facilitating discussion with City staff, conduits were geographically grouped into "flooding problem areas." Figure 3-1a shows the modeled flooding locations and duration under the existing development condition for the southern half of the city, and Figure 3-1b shows the modeled flooding locations and duration under the existing development condition for the northern half of the city. Both figures are located at the end of this section. Per Figures 3-1a and 3-1b, modeled pipes (conduits) are labeled according to the upstream node number.

The model results were reviewed with City staff during a meeting on September 16, 2013. City staff provided comment and discussion about each identified modeled flooding area. Additional flooding areas associated with the operation and maintenance of the storm system that are not reflected in modeled results were also identified by City staff. Based on model results, City feedback, and field reconnaissance, a second meeting was held on October 29, 2013, with City staff to select recommended CIPs for each flooding area.

3.4.2 Summary of Flooding Problems

Table 3-8 summarizes the initially identified flooding problem areas by major basin. The flooding frequencies and scenarios are identified, and the source of the capacity deficiency is provided. The CIP recommendation is also provided.

Table 3-8. Initial Flood Control (FC) Capital Improvement Projects

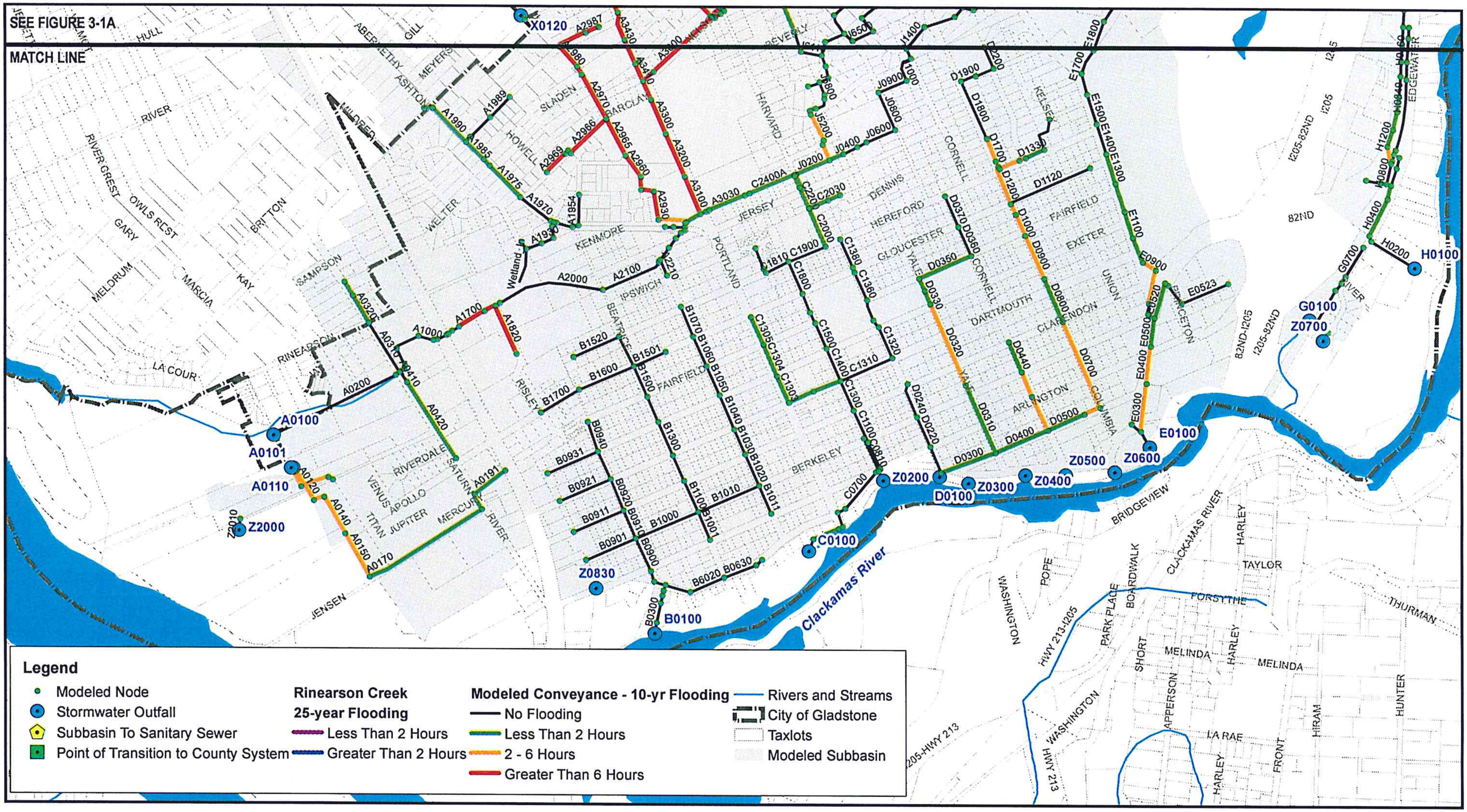
Major basin	Outfall	Conduits (DSNode_USNode)	Flooding frequency, scenario, and duration	Source of capacity deficiency	City feedback	CIP recommended? (Y/N)	FC CIP description and associated CIP number
A	A0101	A0101_A0104	Existing 10-year 2+ hour duration	Existing 12" pipe is undersized for contributing drainage area.	• No upstream collection system present. Current drainage area is not curbed or paved. • Private development and no reported flooding.	N	N/A
A	A0110	A0110_A0191	Existing 10-year 2+ hour duration	• Existing 18" pipe and open channel is undersized for contributing drainage area. • Pipe slope is limited.	• Observed flooding along Jensen Road. • Area is adjacent to Meldrum Bar Park with access and room for construction. • Proposed alignment for Lake Oswego-Tigard Raw Water Pipeline is in close proximity.	Y	Pipe upsize (CIP A-7)
A	A0100	A1100_A3000	Existing 10-year 2+ hour duration	Modeled flooding along Rinearson Creek due to under-capacity culverts, limited floodplain area, and significant vegetation in the channel.	• Rinearson Creek has berm that separates the channel from Olson Wetland (and associated floodplain). • Coordination and access with ODOT regarding replacement of culverts across 99E will be difficult. • Coordination with businesses (private property) along 99E corridor regarding infrastructure installation will be difficult.	Y	Maintain Rinearson Channel (CIP A-1) Install bypass pipe down Portland Avenue (CIP A-2)
A	A0100	A3000_A3050	Existing 10-year < 2 hour duration	Existing 48" pipe is undersized for contributing drainage area and has back slope.	• Existing alignment is on private property. • Pipe condition is failing and needs replacement.	Y	Pipe replacement and realignment (CIP A-3)
A	A0100	A2900_A2987	Existing 10-year 2+ hour duration	• Significant contributing flow from re-routed Glen Echo wetland. • Minimal slope for a majority of the pipe segments.	• Re-routed Glen Echo wetland is a significant source of flooding. Glen Echo Road is often closed due to flooding. • Preliminary design indicates potential to tie in with Portland Avenue system.	Y	Pipe re-route and pipe upsize (CIP A-2)
A	A0100	A3000_A3490	Existing 10-year 6+ hour duration	• Significant contributing flow from re-routed Glen Echo wetland. • Existing 18" pipe is undersized for contributing drainage area and has back slope or minimal slope. • Existing pipe alignment has limited pipe cover.	• Significant flooding problem. • Improvements should be coordinated with existing transportation plan for Portland Avenue.	Y	Pipe replacement and upsize (CIP A-2)

Table 3-8. Initial Flood Control (FC) Capital Improvement Projects

Major basin	Outfall	Conduits (DSNode_USNode)	Flooding frequency, scenario, and duration	Source of capacity deficiency	City feedback	CIP recommended? (Y/N)	FC CIP description and associated CIP number
A	A0100	A3400_A4300	Existing 10-year 6+ hour duration	• Existing 15" and 18" pipe is undersized for contributing drainage area and has back slope or minimal slope.	• Existing alignment is partially on private property. • Pipe condition is failing and needs replacement.	Y	Pipe re-route and pipe upsize (CIP A-6)
B	B0100	Varies	Reported by City staff (not modeled)	Observed flooding on Arlington Street, between Barton and Bellevue, and on Gloucester Street, between Beatrice and Bellevue due to a lack of collection system infrastructure.	• Two catch basins located on Arlington discharge to sanitary system. • Streets were recently paved; installation of piped collection system not recommended until repaving needed.	Y	Sanitary sewer separation and installation of rain gardens (CIP B-1)
C	C0100	C2400_C1900	Existing 10-year < 2 hour duration	Existing diversion pipe from System J causes backwater conditions.	• Reported system flooding is due to pipe condition and capacity issue on System J. • CIP A-3 and J-1 should alleviate flooding issue.	N	N/A
C	C0100	C1300_C1304	Existing 10-year < 2 hour duration	Existing 12" pipe is under-capacity and has minimal slope.	• Reported flooding along Portland Avenue is due to a lack of storm drainage infrastructure. • CIP A-1 and A-2 should alleviate capacity and slope issue.	N	N/A
C	C0100	C0200_C0300 and C0500	Existing 10-year < 2 hour duration	Existing 24" pipe is under-capacity and has minimal slope.	• Maintenance staff reports condition issue with the outfall. • MH C0500 is a combined (sanitary and storm) manhole with high flow bypass to Outfall C0100.	Y	Sanitary sewer separation and outfall replacement (CIP A-2)
D	D0100	D0310_D0340 D0400_D0430 D1700_D0800 D0500_D0700	Existing 10-year 2+ hour duration	Existing pipe (12", 15") is under-capacity and has minimal slope.	• Maintenance staff reports limited flooding in the area. Area has curbless streets and wide ROW. • May be opportunity for green street installation, but no CIP required to address capacity deficiencies.	N	N/A
E	E0100	E0200_E0900	Existing 10-year 2+ hour duration	Existing pipe is under-capacity and has minimal slope.	• Maintenance staff reports limited flooding in the area. • Existing flooding in area generally due to maintenance issues (blocked catch basins).	N	N/A
F	F0100	F0100_F0700	Existing 10-year < 2 hour duration	Existing modeled pipe is under-capacity and has minimal slope.	• Existing alignment is on private property. • Maintenance staff reports flooding on neighboring Durie Court and Hardway.	Y	Pipe re-route and upsize (CIP F-1)

Table 3-8. Initial Flood Control (FC) Capital Improvement Projects

Major basin	Outfall	Conduits (DSNode_USNode)	Flooding frequency, scenario, and duration	Source of capacity deficiency	City feedback	CIP recommended? (Y/N)	FC CIP description and associated CIP number
H	H0100	H0400_H1400	Existing 10-year 2+ hour duration	Existing modeled open channel and culvert under the railroad is under-capacity and has minimal slope.	• No reported flooding in this area. • Open channel is located adjacent to railroad with gravel ballast that promotes infiltration.	N	N/A
J	A0100	J2400_J2800	Existing 10-year 2+ hour duration	Existing 12" pipe is under-capacity for contributing drainage area and has minimal slope/pipe cover.	• Poor infiltration and limited available area for regional detention. • Bedrock may be present in area, which may limit excavation to improve slope of alignment.	Y	Pipe upsize (CIP J-2)
J	A0100	J0200_J5300	Existing 10-year 2+ hour duration	Existing pipe (12", 15") is under-capacity and has minimal slope.	• Existing alignment is on private property. • Maintenance staff reports flooding in vicinity.	Y	Pipe re-route and upsize (CIP J-1)
M	M0100	M0200_M0800	Existing 10-year < 2 hour duration	• Existing pipe (12", 15") is under-capacity and has minimal slope. • Lack of collection system infrastructure in vicinity.	• Existing alignment is on private property. • Maintenance staff reports flooding in vicinity (possibly due to ineffective routing and lack of manholes at pipe bends). • Maintenance staff indicates potential pipe condition issue.	Y	Pipe upsize (CIP M-1)
N	N0100	N0400_N1400 N0400_N0800	Existing 10-year 2+ hour duration	Existing 12" is under-capacity and has minimal slope.	• Maintenance staff reports that Webster Road floods frequently and requires closure. • Downstream Clackamas County pipe capacity deficiencies need to be addressed.	Y	Pipe re-route and upsize (CIP N-1)
N	N0100	Not modeled	N/A	• Maintenance staff reports that a lack of catch basins in area promotes ponding and flooding. • Reported flooding in vicinity of Los Verdes Drive, Crownview Drive, and Charolais Way.	Curb inlets and additional catch basins are recommended to reduce catch basin clogging and roadway flooding.	Y	Add catch basins (CIP N-2)
O	O0400	O0700_O1933 O1930_O1970	Existing 10-year 6+ hour duration	Existing pipe (12", 15") is under-capacity and has minimal slope.	• Existing alignment is on private property. • Bedrock may be present in area, which may limit excavation to improve slope of alignment.	Y	Pipe upsize (CIP O-1)
S	S0100	Not modeled	N/A	Maintenance staff reports that a lack of catch basins in area promotes ponding and flooding.	Currently, the City is adding catch basins to see if problem is alleviated. No CIP required at this time.	N	N/A



Brown AND Caldwell

January 1, 2013

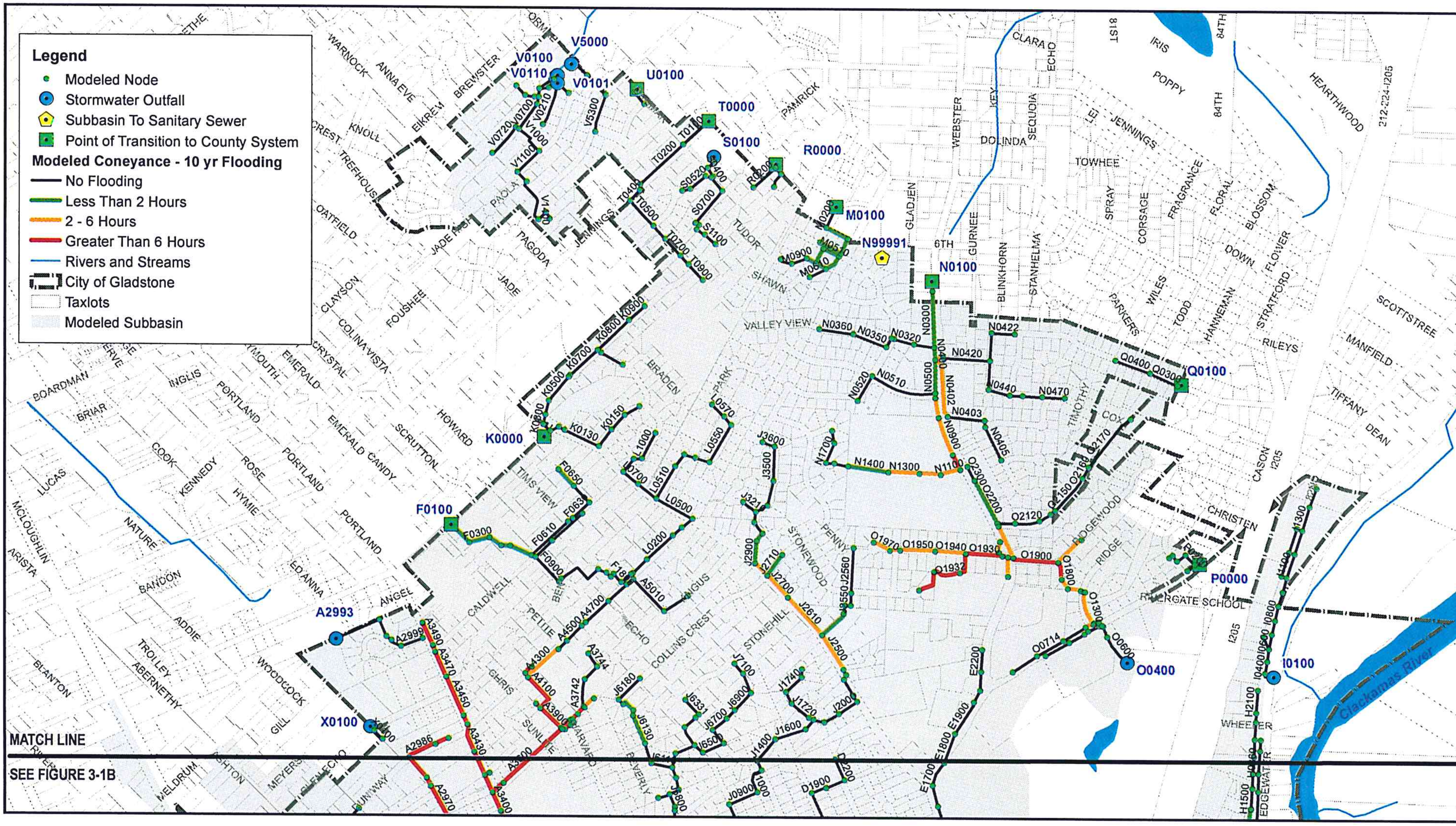
0 750 1,500 Feet

N

CITY OF GLADSTONE
STORMWATER MASTER PLAN

DRAINAGE SYSTEM - SOUTH
EXISTING LAND USE
PREDICTED FLOODING

FIGURE 3-1B



Brown AND Caldwell

January 1, 2013

0 750 1,500 Feet

N

CITY OF GLADSTONE
STORMWATER MASTER PLAN

DRAINAGE SYSTEM - NORTH
EXISTING LAND USE
PREDICTED FLOODING
FIGURE 3-1A

Section 4

Water Quality Retrofit Assessment

As part of this SMP and CIP development, water quality improvement projects were identified for inclusion in the CIP. Review and identification of water quality improvement projects to reduce the discharge of pollutants from the MS4 is a specific objective of the SMP in accordance with requirements of the City's NPDES MS4 permit (Schedule D.6.a).

In addition, the City's NPDES MS4 permit requires development of a stormwater retrofit strategy and identification of stormwater retrofit projects to aid in water quality improvement by July 1, 2015. Specific NPDES MS4 permit requirements (Schedule A.6.b) of the water quality retrofit strategy are listed below:

1. Stormwater retrofit strategy statement and summary, including objectives and rationale
2. Summary of current stormwater retrofit control measures being implemented, and current estimate of annual program resources directed to stormwater retrofits
3. Identification of developed areas or land uses impacting water quality that are high-priority retrofit areas
4. Consideration of new stormwater control measures
5. Preferred retrofit structural control measures, including rationale
6. A retrofit control measure project or approach priority list, including rationale, identification, and map of potential stormwater retrofit locations where appropriate, and an estimated timeline and cost for implementation of each project and approach

As the methodology for identifying water quality improvement projects or CIPs and stormwater retrofits is the same, the City opted to conduct both efforts as part of this SMP. This collective effort, referred to herein as the water quality retrofit assessment, is described in this section. This section includes descriptions of the objectives, methodology, project identification (i.e., project list), and applicability to the City's NPDES MS4 permit requirement(s).

Identified water quality CIPs have been coordinated with flood control CIPs (identified in Section 3.4) to develop a comprehensive project list to address stormwater quality and quantity management and NPDES MS4 permit compliance in the city (Section 5).

4.1 Objectives

The City's strategy for conducting the stormwater retrofit assessment is to target high pollutant generating areas where existing stormwater treatment is currently limited, in order to make progress toward achieving TMDL pollutant load reduction and improve overall surface water quality conditions.

The City currently has minimal public or private stormwater treatment facilities within the city limits. Existing facilities are limited primarily to detention ponds or underground separation vaults on private property, and the retrofit potential for these types of facilities is limited. The stormwater retrofit assessment focuses on the use of infiltration-based facilities (e.g., vegetated infiltration basins, rain gardens, planters) on public property to provide runoff volume reduction in addition to conventional water quality treatment.

To the extent possible, water quality improvement projects are identified in conjunction with locations of existing system capacity deficiencies (Section 3) to allow for the CIPs to address multiple objectives.

4.2 Methodology

Water quality improvement projects were initially identified through a review of system mapping and GIS information including aerial photos, existing (public and private) vacant areas, publicly owned lands, existing and future condition land uses, topography, and locations where flood control CIPs are needed.

The City's stormwater collection and conveyance system discharges through 32 modeled stormwater outfalls, representing 26 major drainage basins. Stormwater is discharged to the Willamette River (directly or via Rinearson Creek), the Clackamas River, or the Clackamas County-owned stormwater collection and conveyance system. Each major basin was individually reviewed in accordance with the following steps in order to identify initial water quality opportunity areas for water quality improvement projects and stormwater retrofits:

Step 1 **Identify vacant lands.** Review of vacant lands was conducted to identify parcels where space may be available for siting of a new regional or local water quality facility. Publicly owned vacant lands were prioritized. Vacant lands observed (based on aerial photographs) to be forested or riparian area were not considered to be priority areas, as such areas should be preserved.

Step 2 **Review land use.** High pollutant generating land uses (e.g., industrial, commercial) with high imperviousness were prioritized for installation of a stormwater treatment facility.

Step 3 **Review soils.** A significant proportion of the city contains Type C/D soils and underlying bedrock, which prevents use of traditional infiltration-based facilities for water quality treatment. However, the southern portions of the city contain Type A/B native soils that are ideal for installation of rain gardens, planters, and vegetated infiltration facilities.

Step 4 **Identify placement/location within the overall conveyance system.** Water quality opportunity areas were reviewed with respect to the contributing drainage area and location within the major basin. This review was especially applicable in the identification of potential regional stormwater facilities. Facility placement at the upstream end of the stormwater conveyance system would allow for only a minimal drainage area to be treated, whereas placement toward the downstream end of the stormwater conveyance system has potential to treat a larger area.

Step 5 **Identify available right-of-way.** Although existing water quality facilities within the city limits are limited, there is significant potential to install water quality facilities within the public ROW. The city features wide streets, and a significant proportion of the local and arterial streets are not curbed. Green street facilities (stormwater planters, rain gardens) were prioritized for water quality retrofit due to their placement in public property and ability to collectively address water quality and stormwater conveyance.

Step 6 **Review proposed flood control project needs.** The City is coordinating its water quality retrofit assessment and the identification of water quality CIPs with its overall SWMP development. To the extent that a CIP can address multiple objectives, such CIPs would be prioritized (see Section 6). Coordination is particularly beneficial for those flood control/pipe replacement projects isolated to the ROW, as new green street facilities may be installed at the same time, resulting in schedule and cost efficiencies.

4.3 Water Quality Retrofit Assessment Results

This section presents the results of the water quality retrofit assessment, including the identification of water quality improvement projects.

In conjunction with the methodology described in Section 4.2, initial water quality improvement projects were reviewed with City staff at a workshop on September 16, 2013. During the workshop, project feasibility and practicability was discussed. Additional water quality opportunity areas/projects identified by City staff were also discussed. Based on City feedback and, in some cases, field reconnaissance, a recommendation to include the project as a CIP in the SMP was made.

Table 4-1 summarizes the initially identified water quality improvement project (by major basin), the associated project description, and feedback from City staff regarding feasibility. The CIP recommendation is also provided.

Table 4-1. Initial Water Quality (WQ) Capital Improvement Projects

Major basin	Project name	Initial project description and location	Project rationale	Opportunity to coordinate with an identified flood control projects?	City feedback	CIP recommended? (Y/N)	WQ CIP description and associated CIP number
A	Gladstone High School Rain Garden	Install rain garden in existing public vacant parcel NE of the high school baseball field along Harvard	• Limited water quality treatment in vicinity • Project may be coordinated with high school as an educational tool	No	• Rain garden to collect runoff from shop building rooftop. • High school students may provide design and planting services.	Y	Rain Garden (CIP A-4)
A	Gladstone High School Regional Detention Facility	Install detention/ retention facility on existing public vacant parcel N of Jersey along Portland Avenue	• Limited water quality treatment in vicinity • Downstream system location allows for treatment of large contributing drainage area • Combined treatment and detention facility addresses downstream flooding	Yes; facility location allows for collection and detention of runoff to impact downstream flood control CIP.	• Location was previously considered for a detention/retention facility. • Requires coordination with the high school for implementation. • Coordinate installation with pipe replacement/upsizing along System J.	Y	Regional Detention Facility (CIP A-3)
A	Portland Avenue Green Street Installation (Phase 2)	Install green streets along Portland Avenue north of Jersey	• Limited water quality treatment in vicinity • Pipe repair/replacement required on Portland Avenue for capacity deficiencies • High pollutant generating area (commercial)	Yes, indirectly. Flood control CIP sizing does not account for peak flow reduction due to use of green streets, but there may be sizing impacts.	• Green streets (planters) recently installed to alleviate flooding on Portland Avenue have been effective. • City wants to implement a citywide green streets program.	N	See Green Streets Pilot project
A	Tryon Regional Detention	Install detention/ retention pond at private vacant property currently owned by church (N of Nelson by Tryon)	• Limited water quality treatment in vicinity • Pipe repair/replacement required on Nelson for capacity deficiencies	No; downstream pipe condition is poor and replacement is needed.	Facility may be considered an asset to church property. Proposed project location is unmaintained.	Y	Rain Garden (CIP A-5)
A	Rinearson Creek/ Olson Wetland Enhancement	Reconnect Rinearson Creek to adjacent floodplain (Olson wetland)	• Limited water quality treatment in vicinity • Location targeted for stream restoration by volunteer groups and DEQ • Flooding occurs on Rinearson Creek, and adjacent private property is located with limited setback	Yes	• Large berms separate Olson wetland from Rinearson Creek. • Maintenance activities along Rinearson Creek should include sediment removal, vegetation management, and possible tree removal.	Y	Stream Maintenance and Restoration (CIP A-1)

Table 4-1. Initial Water Quality (WQ) Capital Improvement Projects

Major basin	Project name	Initial project description and location	Project rationale	Opportunity to coordinate with an identified flood control projects?	City feedback	CIP recommended? (Y/N)	WQ CIP description and associated CIP number
A	Manor Water Quality Facility	Install detention/ retention pond at existing public vacant parcel north of the Olson wetland	• Limited water quality treatment in vicinity • Downstream system location allows for treatment of large contributing drainage area	No	• Vacant parcel is currently zoned multifamily development. • Topographic constraints prevent use of parcel for detention/retention to impact Rinearson Creek or Portland Avenue.	N	N/A
A	Meldrum Bar Regional Detention Facility	Install detention/ retention pond at Meldrum Bar Park	• Limited water quality treatment in vicinity • Downstream system location allows for treatment of large contributing drainage area • Existing flooding along Jensen Road and west of Titan Avenue • Large amount of public vacant property • High pollutant generating area (Jensen Road)	Yes	Topographic and infrastructure constraints (steep slope west of Titan and adjacent baseball field at Meldrum Bar) prevent installation of detention facility to mitigate flooding.	Y	Vegetated swale and open channel improvements (CIP A-7)
A	Riverdale Drainage Improvements	City maintenance reports connection of three catch basins to sanitary. Sanitary backups occur on street.	• Limited water quality treatment in vicinity • Storm system cross-connection to sanitary results in citizen complaints	N	No existing storm drainage infrastructure. Storm system connection would require boring to Meldrum Bar Road via private property.	Y	Sanitary system disconnection, green street installation, and installation of UICs (CIP A-8)
B	System B Green Streets	City maintenance reports connection of two catch basins to sanitary on Arlington.	• Limited water quality treatment in vicinity • Storm system cross-connection to sanitary results in citizen complaints	Yes	• No existing storm drainage infrastructure. • Localized flooding present.	Y	Green Street installation and sanitary system disconnection (CIP B-1)
C	Portland Avenue Green Street Installation (Phase 1)	Install green streets along Portland Avenue south of Jersey	• Limited water quality treatment in vicinity • Pipe repair/replacement required on Portland Avenue for capacity deficiencies • High pollutant generating area (commercial)	Yes, indirectly. Flood control CIP sizing does not account for peak flow reduction due to use of green streets, but there may be sizing impacts.	• Green streets (planters) recently installed to alleviate flooding on Portland Avenue have been effective. • Want to implement a citywide green streets program.	N	See Green Streets Pilot project

Table 4-1. Initial Water Quality (WQ) Capital Improvement Projects

Major basin	Project name	Initial project description and location	Project rationale	Opportunity to coordinate with an identified flood control projects?	City feedback	CIP recommended? (Y/N)	WQ CIP description and associated CIP number
D	Yale, Cornell, and Columbia Green Street Installation	Install green streets along Yale, Cornell, and Columbia south of Gloucester	• Limited water quality treatment in vicinity • Wide ROW and limited existing curb and gutter • Modeled pipe capacity deficiency	Yes, indirectly. No flood control CIPs were identified in the area, but peak flow reduction due to use of green streets may occur.	• Green streets (planters) recently installed to alleviate flooding on Portland Avenue have been effective. • Want to implement a citywide green streets program.	N	See Green Streets Pilot project
D	Clackamas Avenue Rain Garden	Install rain garden in existing public, vacant parcel along Clackamas Avenue	• Limited water quality treatment in vicinity • Highly infiltrating soils in vicinity • Construction of the Lake Oswego-Tigard water intake is in progress at the proposed project location	No	• Proposed location is in close proximity to banks of Clackamas River, and may pose a safety concern. • The City is currently coordinating with the Lake Oswego-Tigard water program to install additional water quality facilities.	N	N/A
E	Oatfield Regional Detention Facility (South)	Utilize existing public vacant parcel at intersection of Oatfield Road and Webster Road.	• Limited water quality treatment in vicinity • Pipe repair/replacement required on Oatfield Road for capacity deficiencies • Combined treatment and detention facility addresses downstream flooding	Yes	• Proposed location is the future site of the Gladstone Library. • Underlining bedrock makes infiltration of runoff infeasible and construction (excavation) difficult.	N	N/A
H	Wetland improvement/ enhancement at Edgewater	Install regional water quality facility in existing public vacant parcel east of I-205 and south of Evergreen	• Limited water quality treatment in vicinity • Large existing vacant area • Downstream system location allows for treatment of large contributing drainage area	No	• Ownership of parcel is in question. • Existing area surrounding Edgewater wetland is forested and in good natural condition. Excavation and construction may impede water quality improvement.	N	N/A
H	Railroad channel improvement	Conduct maintenance on existing open channel conveyance adjacent to railroad along 82nd Avenue.	• Limited water quality treatment in vicinity • Modeled capacity deficiency in vicinity • Field reconnaissance indicates presence of overgrown vegetation and garbage	Yes, indirectly. No flood control CIPs were identified, but maintenance may improve channel capacity.	No reported flooding complaints in the area.	Y	Channel maintenance (CIP H-1)

Table 4-1. Initial Water Quality (WQ) Capital Improvement Projects

Major basin	Project name	Initial project description and location	Project rationale	Opportunity to coordinate with an identified flood control projects?	City feedback	CIP recommended? (Y/N)	WQ CIP description and associated CIP number
J	Oatfield Regional Detention Facility (West)	Utilize existing public vacant parcel at intersection of Oatfield Road and Webster Road.	• Limited water quality treatment in vicinity • Pipe repair/replacement required on System J for capacity deficiencies • Combined treatment and detention facility addresses downstream flooding	Yes	• Proposed location is the future site of the Gladstone Library. • Underlining bedrock makes infiltration of runoff infeasible and construction (excavation) difficult.	N	N/A
J	Salty Acres Regional Facility	Install regional water quality facility in existing public vacant parcel east of the Salty Acres subdivision	• Limited water quality treatment in vicinity • Large existing vacant area • Downstream system location allows for treatment of large contributing drainage area	No	• The vacant parcel is actually a delineated wetland (per Salty Acres development application from 1991). • Existing wetland is in good condition.	N	N/A
O	Kraxberger Middle School Regional Detention	Install regional water quality facility in existing open area southwest of the school	• Limited water quality treatment in vicinity • Large existing vacant area • Combined treatment and detention facility addresses downstream flooding on Webster	Yes	• Limited available area on school grounds due to topography. • Native soils have limited infiltration capacity. • Relocate facility to reduce flooding along System N (Webster) instead of System O.	Y	Bioswale for conveyance and treatment (CIP N-1)
O	Church Detention Pond Retrofit	Retrofit existing detention pond on church property for water quality improvement	• Limited water quality treatment in vicinity • Existing facility with potential to retrofit • Downstream system location allows for treatment of large contributing drainage area	No	• Limited as-built or ownership information available for facility. • Need to obtain survey information and determine pond functionality and downstream flow conditions. • Existing pond has standing water at all times.	Y	Survey and Functional Evaluation (CIP O-2)

Section 5

Integrated Stormwater Management Strategy

This section identifies the flood control and water quality CIPs designed to address flooding (Section 3) and water quality improvement (Section 4). To the extent possible, CIPs were developed as integrated solutions to address multiple objectives (e.g., flood control, water quality, etc.).

5.1 Integrated CIP Development

Integrated CIP development refers to the selection and design of CIPs to address multiple objectives including flood control, regulatory requirements, and water quality improvements.

An integrated CIP development approach was used during the identification of the water quality improvement projects and water quality retrofit assessment (as described in Section 4). Areas with flood control needs were prioritized for purposes of targeting a water quality improvement project.

As described in Section 3.4, a total of 22 flood control projects were initially identified. After meeting with City staff, 15 of the flood control projects were further developed as CIPs. As described in Section 4.3, a total of 19 water quality projects were initially identified. After meeting with City staff, 10 of the water quality projects were further developed as CIPs. Another three of the water quality projects were classified as covered under a green streets pilot project, a separate program being developed within the City to install planters and rain gardens within the public ROW in conjunction with street improvement projects.

Together, the flood control and water quality projects that were selected for further development as CIPs were consolidated and integrated to reflect consistent contributing drainage areas. CIP design concepts and approaches described in Sections 3 and 4 were revisited during the CIP development to formalize the overall CIP design.

A comprehensive summary of flood control and water quality CIPs is provided in Section 5.4. A total of 18 CIPs are identified. Of the 18 CIPs, five are integrated flood control and water quality CIPs. Eight of the CIPs address flood control only and five of the CIPs address water quality only. Section 5.4 also includes a problem description and project description for each CIP, and indicates whether the CIP would qualify as a water quality retrofit project for NPDES MS4 permit compliance. CIPs are sorted and named by major basin. Figure 5-1 (a and b) at the end of this section shows the location of each CIP. Detailed CIP cost summaries are located in Appendix C.

5.2 CIP Sizing and Design Assumptions

This section includes a summary of the CIP sizing and design criteria based on the type of system improvement proposed. System improvements include pipe upsizing and pipe replacement, open-channel conveyance improvements, installation of rain gardens or stormwater planters, installation of underground injection controls (UICs), and a detention pond installation. Proposed CIPs may reflect a combination of system improvements.

A revised hydraulic results table reflecting inclusion of system improvements for flow control (e.g., pipe replacement and detention facility installation) is included as Appendix D (Table D-1). In Table D-1, CIPs are described by system conduit. The conduit and node numbering is consistent with the existing system configuration. As not all flooding is addressed with identified CIPs, the duration of future condition flooding both with and without CIP implementation is reflected in Table to identify the impact the CIP has on system-wide flooding.

As described in Section 3.3, the Gladstone Standards reference Section 5.2.1 of CCSD 1 *Surface Water Management Rules and Regulations* for collection system sizing and Section 5.3 of the CCSD 1 *Surface Water Management Rules and Regulations* for water quality treatment. For purposes of CIP design, applicable design criteria are listed below:

- Design storm(s):
  - Conveyance sizing: 25-year, 24-hour design storm. Due to the extensive reported flooding, CIP identification used the 10-year, 24-hour design storm peak flow and duration of flooding to prioritize flooding areas.
  - Stormwater treatment sizing: 1-inch/24-hour design storm (water quality design storm).
- Minimum pipe diameter = 12 inches in the public ROW

Additional detail regarding CIP sizing and design assumptions is provided below.

5.2.1 Conveyance System Sizing and Design

Although not specifically referenced in the Gladstone Standards, the following design criteria were referenced from Clackamas County Water Environment Services *Stormwater Management Design Standards* (2010) and used in CIP design for stormwater conveyance piping:

- minimum pipe slope = 0.5 percent (with possible exception to reflect a minimum flow velocity of approximately 3 feet/second when flowing half full)
- maximum pipe slope = 20 percent
- minimum pipe cover = 3 feet (used for HDPE pipe, but with exception for RCP pipe where the existing cover is less than 3 feet)
- maximum manhole spacing = 250 feet

Open-channel conveyance design criteria were also not available in the Gladstone Standards. The following open-channel conveyance design criteria were used in CIP development:

- maximum side slope for bioswales and stream restoration = 3:1 (H:V)
- Manning's *n* = 0.040
- channel type = trapezoidal with minimum 1 foot bottom width
- freeboard depth at 10-year design storm = 1 foot
- maximum velocity = 3 feet/second

Pipe improvements were evaluated using XP-SWMM to ensure that installation of the CIP (i.e., relief of the constriction) did not result in downstream flooding.

5.2.2 Infiltration Planter Boxes and Rain Gardens

Rain gardens and planters were sized to infiltrate the stormwater runoff volume associated with the water quality design storm (1 inch/24 hours) from contributing impervious surface. The maximum ponding depth in the facility is 6 inches. Depending on proximity to a public stormwater collection and conveyance system, the facilities may be designed with an overflow and connection piping. If a stormwater collection and conveyance system is not located in close proximity, no overflow is specified.

Unit costs associated with rain gardens and planter boxes assume 18 inches of amended soil (growing media). Water quality facility plantings are costed separately. Excavation (beyond the 24 inches of total facility design depth, assuming a 6 inch ponding depth) and inclusion of additional amended soils is costed separately and applied when the facility is installed in Type C/D soils and/or without an overflow.

5.2.3 Detention Pond

One new detention pond is proposed as a combined flood control and water quality CIP at Gladstone High School. This detention pond sizing is opportunistic based on available space and ability to detain peak flow impacting the downstream conveyance system sizing. Therefore, the pond is not designed to accommodate (store) a specific runoff volume from the system.

The detention pond design includes installation of 18 inches of amended soil, 18 inches of drain rock, and water quality facility plantings along the pond bottom and side slope surface area.

5.2.4 Underground Injection Controls

Estimated UIC sizing was based on the 2008 *Stormwater Management Manual* (SMM) for the City of Portland, Exhibit 2-31.

5.3 Cost Estimates for CIP Development

Cost estimates for CIP design and construction are based on the total capital investment necessary to complete a project (i.e., engineering through construction). Expenditures are calculated for construction or capital elements, based on the CIP design and representing material costs, labor costs, other services (traffic control, erosion control), and contingency. Expenditures are calculated separately for administrative and design services, including engineering and permitting.

Unit cost information for construction or capital elements of the CIP facilities was compiled from recent, local planning and design projects for the City of Milwaukie (2012), City of Central Point (2013), City of Portland (2010), and Clean Water Services (2012). The 2012 RS Means *Book for Site Work and Landscaping* was referenced for additional work not covered by recent bid tabs. A construction contingency of 30 percent (on average) is added to the construction element cost. It is appropriate to allow for this degree of uncertainty due to limited information available during the SMP-level development of projects. Factors unknown at the time of this SMP development may include geotechnical and groundwater conditions and utility relocation and realignment needs.

Administrative and design service elements include engineering, permitting, and construction administration costs, and such costs are based on a general percentage of the total construction or capital element cost. Land acquisition and easement costs are not included in the cost estimates, as most projects proposed are located on City property or within the City ROW. It is assumed that the City will obtain necessary easements for work conducted on private property.

A good indicator of changes over time in construction costs is the *Engineering News-Record* (ENR) 20-city Construction Cost Index (CCI), which is computed from prices of construction materials and labor. Cost adjustments may be found in the ENR CCI to adjust costs provided in this SMP to the time the project is being bid.

Unit cost information and individual cost estimates for CIPs are included in Appendix C. CIPs in Appendix C follow the same order as CIP descriptions listed in Section 5.4. Where possible, large CIPs (i.e., CIP A-2) are separated by construction phase, and a detailed cost estimate is provided for each construction phase. For planning purposes in Section 5.4, the cost for CIPs under $100,00 are rounded to the nearest $1,000; CIPs over $100,000 are rounded to the nearest $10,000.

5.4 CIP Descriptions

A summary of each CIP is provided below and includes identification of the project objective, statement of need, project description, estimated project cost, and associated design assumptions. CIP summaries are organized by major basin.

During workshops with City staff related to CIP development, the City opted to include a programmatic CIP for green street facility installation in conjunction with road improvements. This programmatic CIP was developed as a lump-sum amount to be spent annually. For purposes of future planning, sizing and cost of green streets were estimated on a per block basis, depending on native soil infiltration rate. A description of this programmatic CIP is provided in Section 5.4.9.

5.4.1 Basin A

CIP name	A-1. Rinearson Creek Stream Enhancement
Objective addressed	Flood control and water quality
Contributing drainage area	341 acres
Statement of need	• The open-channel portion of Rinearson Creek, west of Beatrice St., experiences flooding during high flows and storm events. • Heavy vegetation and debris obstructs flow through the creek between Beatrice St. and the Olson Wetlands. The Olson Wetlands historically served as the creek's floodplain. The Olson Wetlands are currently disconnected from the creek along the right bank by a man-made berm. • Contributing drainage area has limited water quality treatment. Development encroaches on the creek, which also contributes to pollutant discharge and results in safety issues and property damage.
Project description	• Conduct approximately 500' of channel maintenance to clear vegetation obstructing the flow path from Beatrice St. to the Olson Wetlands (A2100 to A1850). • Remove the existing 500' berm along the Olson Wetlands to restore connectivity between the creek and wetlands.
Estimated total project cost	$410,000
Design assumptions	• Flow control improvements affecting Rinearson Creek are addressed with CIP A-2. • An engineering evaluation to determine the flow regime in the creek is costed under CIP A-2. • Engineering and permitting costs were assigned at 25% of the construction costs (instead of 20% typical) to account for unknown related to permitting for in-water work and the need for a flow bypass during construction activities. Construction will occur during summer low flows. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City. • A lump sum of $20,000 is assumed for flow bypass for the total 1,000 foot project length.

CIP name	A-2. Portland Avenue Bypass and Upstream Improvements
Objective addressed	Flood control
Contributing drainage area	357 acres
Statement of need	• Modeling predicts flooding in the existing stormwater collection and conveyance system along Portland Ave., Watts St., Risley Rd., and Nelson Rd during the existing and future 10-year design storm. Modeling also predicts flooding during the existing and future 25-year design storm along Rinearson Creek. • Areas of historical flooding reported by City maintenance staff include: – Risley Rd. and along Rinearson Creek. Property adjacent to the creek has limited setback distance and building flooding is reported. – Rinearson Creek between Bellevue Ave. and Beatrice Ave. – Portland Ave. (north of Jersey St.). – Glen Echo Ave. through private property to SE Watts St. Clackamas County installed an 18" culvert across SE Watts St. that serves as a diversion pipe routing flow south from the Glen Echo Wetlands during high flow events. Flow crosses Glen Echo Ave., flooding the street. – Duniway Ave. Negative pipe slope along Duniway limits system capacity. The pipe segment from MH A2980 to MH A2975, south of Duniway is reported to be in poor condition. • There is an existing sanitary sewer overflow on Portland Ave. south of Dartmouth and Clackamas Ave. (MH C0500). Portland Ave., south of Dartmouth, does not have a dedicated stormwater collection and conveyance system.

CIP name	A-2. Portland Avenue Bypass and Upstream Improvements (continued)
Project description	This project extends from Glen Echo Ave. to the Clackamas River along Portland Ave. and includes a new 48" bypass pipeline to divert high flows to the Clackamas River instead of down Rinearson Creek. The project includes pipe replacement/realignment north of Jersey St. Due to size and cost, this CIP has been phased into four smaller projects, which are described in the recommended order of completion. CIP A-2.1 Portland Avenue High Flow Bypass • Install 2,650 LF of 48" HDPE from the intersection of Jersey St. and Portland Ave. to the Clackamas River (new MH C1308 to C0500). This bypass pipe routes all drainage north of Jersey St. along Portland Ave. (see CIP A-2.3) to the Clackamas River and diverts high flow from the 48" pipe system east of Portland Ave. at Jersey St. (see CIP C-1). Low flow from the east pipe system continues to discharge to Rinearson Creek. • Detailed design of the high flow bypass should include a downstream assessment of Rinearson Creek to set the appropriate flow regime to maintain aquatic habitat. • To accommodate the increase in conveyance pipe size, replace the existing 500' of 24" storm drain outfall pipeline from MH C0500 to the Clackamas River with 500' of 48" RCP. A lump-sum capital expense of $50,000 was also included in the project cost for outfall improvements. • Installation of this portion of the CIP, at the specified elevations, is necessary to ensure that the bypass will operate in conjunction with improvements outlined as part of CIP A-2.3. • Capital implementation cost subtotal: $3,773,000 CIP A-2.2 Sanitary Sewer Disconnection • Disconnect existing catch basins and inlet leads, which currently drain to the sanitary sewer along Portland Ave. between Clarendon St. and Arlington St. Install new catch basins and inlet leads to the Portland Ave. High Flow Bypass (CIP A-2.1). • Capital implementation cost subtotal: $78,000 CIP A-2.3 Portland Avenue Pipe Replacement/Realignment North of Jersey • Replace and realign the existing storm drain on Portland Ave. from Glen Echo Ave. to Jersey St. The realignment is intended to lower the elevation of the pipeline on Portland Ave. and eliminate negative slopes. Connection at Jersey St. is required at new MH C1308 (CIP A-2.1). Pipe replacement and realignment from Glen Echo Ave. to Nelson Ln is required to accommodate additional flow associated with CIP A-6. Details related to the pipe replacement are listed below: • Replace existing 12" and 18" CSP with 416 LF of 24" RCP from Glen Echo Ave. south along Portland Ave. (MH A3427 to A3410). • Replace existing 18" CSP with 153 LF of 36" RCP from A3410 to the intersection of Portland Ave. and Nelson Ln (MH A3410 to A3400) • Replace existing 18" CSP with 1,168 LF of 42" HDPE from Nelson Ln to Jersey St. along Portland Ave. (MH A3400 to new MH C1308). • Capital implementation cost subtotal: $1,336,000 CIP A-2.4 Duniway to Barclay Pipe Replacement/Realignment • Replace and realign the existing 12" and 18" PVC on Duniway with 116 LF of 12" RCP (MH A2987 to A2986) and 252 LF of 18" HDPE (MH A2986 to A2980), respectively. • Replace and realign the existing 18" CSP with 692 LF of 24" HDPE from Duniway to the intersection of Barclay St. and Watts St. (MH A2980 to A2962). Install a ditch inlet near A2980 to accommodate flow from the Glen Echo Wetlands, which have historically flooded this area during storm events. • Install 385 LF of new 30" HDPE from Watts St. to Portland Ave. along Barclay (MH A2962 to A3300). This CIP routes flow that previously drained south to Rinearson Creek via Watts St. to the new Portland Ave. storm system. CIPs A-2.1 and A-2.3 must be installed prior to this project. • Capital implementation cost subtotal: $607,000
Estimated total project cost	A-2.1 $3,773,000 A-2.2 $78,000 A-2.3 $1,336,000 A-2.4 $607,000 Total $5,790,000

CIP name	A-2. Portland Avenue Bypass and Upstream Improvements (continued)
Design assumptions	• CIPs A-2.1 and A-2.3 must be completed prior to construction of CIPs A-2.4 and A-6. CIP A-2.2 may be completed in conjunction with CIP A-2.1. • Per CIP A-2.2, replacement of sanitary sewer infrastructure is not reflected in the cost estimate at this time. • A lump sum construction cost of $50,000 was included in CIP A-2.1 to account for design and construction considerations associated with steep slopes and outfall piping to the Clackamas River. • 15 cfs of baseflow was input at model node A2980 to simulate flow entering the City's storm drain from the Glen Echo Wetlands. • Property and easement acquisition is not included in the cost estimate. • The engineering and permitting percentage for CIP A-2.1 was increased from 20% to 30% to reflect the need to assess Rinearson Creek and determine the low-flow regime required to maintain aquatic habitat.

CIP name	A-3. High School Storm Drain Improvements and Detention
Objective addressed	Flood control and water quality
Contributing drainage area	116 acres
Statement of need	• Modeling predicts existing-condition flooding from MH J5500 on Patricia Dr. to MH J0200 on private property between Kenmore St. and Jersey St. Maintenance staff report flooding in this segment is increasing in severity, possibly due to debris accumulation in the pipe. • The model also predicts existing-condition flooding along the 48" CMP from Harvard Ave. to Portland Ave. along the south property line of the high school. This alignment is not within the ROW. • City maintenance staff report that the CMP pipe from MH J0200 to A2000 is in poor condition. • Water quality treatment is limited throughout this contributing drainage area. A water quality facility located at the high school would provide water quality benefit and educational opportunities.
Project description	• Abandon the existing 18" HDPE from MH J5100 to MH J0200 (located on private property). Install 468 LF of 30" RCP from MH J5100 at the end of Kenmore St. to new MH J5050 at Harvard Ave. to direct flow in the public ROW and toward the new high school detention pond. • Install 250 LF of 30" RCP from MH J5050 to a new 105,000 ft^3 detention pond in the vacant grassy area east of the high school ball field. The detention pond has a maximum depth of 4.6' from the top of the amended growing media to existing grade, 3:1 side slopes, a 24,000 ft^2 top area, and a bottom outlet diameter of 8". The detention pond is installed with 18" rock underdrain and 18" of amended growing media on the pond bottom and side slope and water quality facility plantings along the facility side slopes to promote treatment and infiltration. • Replace the existing (27" to 42" diameter) CMP from Harvard Ave., north of Jersey St., to the new detention pond outlet (MH J0200 to A3030) with 740LF of 36" RCP. • Replace the existing 48" CMP from the detention pond outlet to Portland Ave. (MH A3030 to A3000) with 389 LF of 48" RCP. This CIP maintains existing grade at A3000 so low flows/baseflow will continue to be routed to Rinearson Creek. A weir is included at A3000 to direct high flows to the Portland Ave. Bypass at the new MH C1308.
Estimated total project cost	$1,840,000
Design assumptions	• The rock underdrain layer associated with the detention pond was included to increase storage capacity in the facility. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City. • Design of the overflow weir at A3000 should be evaluated during the design of CIPs A-1 and A-2. • If the detention pond and associated 24" inlet pipe is not constructed, the pipe replacement from C2400 to A3030 would increase in diameter from 36" to 48".

CIP name	A-4. High School Rain Garden
Objective addressed	Water quality
Contributing drainage area	2,500 ft^2 (estimated as the rooftop area of the high school shop building)
Statement of need	Water quality treatment is limited throughout this contributing drainage area and existing public vacant property northeast of the high school ball field. This project would provide water quality benefit and educational opportunities for the high school.
Project description	• Install a 200 ft^2 infiltration rain garden to treat runoff from the shop building rooftop area. Route runoff from the rooftop to the facility. Facility sizing is based on infiltration of the water quality storm runoff volume. • Additional excavation and soil amendment are included in the cost estimate due to the native soil infiltration characteristics.
Estimated total project cost	$12,000
Design assumptions	• It is anticipated that project design and installation would be conducted in-house by the City or in collaboration with the high school. • Infiltration testing required prior to design to confirm sizing of the rain garden. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City.

CIP name	A-5. Tryon Rain Garden
Objective addressed	Water quality
Contributing drainage area	1 acre (estimated as the rooftop and parking area of the existing church)
Statement of need	• Water quality treatment is limited throughout this contributing drainage area. • The vacant portion of the site for this CIP is overgrown with non-native vegetation. Enhancement of this area for water quality may be considered an asset for the church property.
Project description	• Install a 3,330 ft^2 rain garden on the vacant portion of the church property (N of Nelson Ave. by Tryon). The proposed vacant portion of the property is existing open space not currently landscaped. • Additional excavation and soil amendment are included in the cost estimate due to the native soil infiltration characteristics.
Estimated total project cost	$220,000
Design assumptions	• Construction of this facility may be conducted in conjunction with CIP A-6, as both projects are located in the same vicinity. However, due to the condition of the existing downstream pipe on Nelson, design of this facility for flood control in conjunction with CIP A-6 was not deemed cost-effective. • Infiltration testing required prior to design to confirm sizing of the rain garden. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City.

CIP name	A-6. Glen Echo Pipeline Realignment
Objective addressed	Flood control
Contributing drainage area	47 acres
Statement of need	Modeling predicts existing-condition flooding from Nelson Lane to Glen Echo Ave. near Tryon Ct. Flooding in this location was not confirmed by maintenance crews; however, the condition of this pipeline is reportedly poor. The City recently attempted to inspect the pipe using CCTV, but was not able to complete the inspection due to debris blocking the pipeline and blind bends that would not pass the CCTV camera. Access to this segment is also challenging because the alignment is on private property.
Project description	• Abandon the existing 18" storm drain running on private property from MH A4100 at Dickerson Ct. and Glen Echo Ave. to MH A3800 on Nelson Lane. • Install 625 LF of new 24" HDPE from Glen Echo Ave. and Dickerson Lane to Glen Echo Ave. and Portland Ave. (MH A4100 to A3427). • Install 203 LF of new 12" RCP on McCall and connect to the new 24" HDPE pipeline at MH A3920.
Estimated total project cost	$280,000
Design assumptions	• Installation of CIP A-2.1 and A-2.3 is required prior to the installation of this project. If that is not feasible, an alternative realignment could be made with a new 24" pipe down McCall Ct. prior to connecting with the existing storm drain alignment at A3910. The pipe replacement would end at A3800 on Nelson Lane. This alternative includes a portion of the pipe realignment on private property. • Any easements required shall be obtained by the City.

CIP name	A-7. Meldrum Bar Bioswale
Objective addressed	Flood control and water quality
Contributing drainage area	37 acres
Statement of need	• There is no water quality treatment in the contributing drainage area. This project is located on public property within an existing park area. This project is also located at the downstream end of an existing collection system such that water quality treatment can affect a large contributing drainage area. • Modeling predicts existing-condition flooding from the outfall at MH A0110 upstream to Jensen Rd. and Highway 99E. Maintenance staff report flooding along Jensen Rd., west of River Rd.
Project description	• Install 340' of open-channel improvements and 402' of new channel (bioswale) between model nodes A0150 and A0130. • Combined treatment and flow control in a single facility allows the project to treat a relatively high pollutant generating area while increasing capacity to convey flow from the Jensen Rd. system. This CIP alleviates 10-year flooding along Jensen Rd. and Highway 99E. 25-year flooding reported in the open-channel segment is reduced to less than 2 hours duration.
Estimated total project cost	$230,000
Design assumptions	• Relocation or adjustment of the existing ball fields and walking path at Meldrum Bar Park were not included as part of the cost estimate. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City.

CIP name	A-8. Riverdale Drainage Improvements
Objective addressed	Water Quality
Contributing drainage area	4.5 acres total drainage area (stormwater planter sizing based on approximately 0.5 acre of contributing impervious area).
Statement of need	• There is no water quality treatment in the contributing drainage area. • The Riverdale subdivision is located on the west of River Rd. on Riverdale Dr. Stormwater runoff is currently routed to three catch basins at the west end of Riverdale Dr., which drain to the sanitary sewer. The sanitary sewer in this area is under-capacity and the combined system has historically flooded the cul-de-sac.
Project description	• Install 1,765 ft^2 of stormwater planters to infiltrate stormwater runoff from the public ROW and adjacent private property within the subdivision. Facility sizing is based on infiltration of the water quality design storm. Planters are designed to bypass to the road. • Disconnect three existing catch basins from the sanitary sewer system. Install three new catch basins and a sediment manhole in the cul-de-sac to collect runoff from the total contributing drainage area. • Install three UICs to dispose treated stormwater runoff.
Estimated total project cost	$280,000
Design assumptions	• The capacity and condition of the sanitary sewer was not evaluated as a part of this study and should be evaluated at this location in future sanitary sewer master planning efforts. • Installation of a dedicated storm pipe to Meldrum Bar Rd. was considered but not pursued due to required access, easements, and construction on private property. Such an alternative would also require extension of the existing storm line on Meldrum Bar Rd. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City. • Infiltration testing required prior to design to confirm sizing of stormwater planters and feasibility of using UICs for stormwater disposal. • Registration costs for UICs not included in the cost estimate.

5.4.2 Basin B

CIP name	B-1. Basin B Drainage Improvements
Objective addressed	Flood control and water quality
Contributing drainage area	• Roadway surface on Arlington St. from Barton Ave. to Bellevue Ave.: 18,750 ft2 (contributing impervious area based on 750' length and 25' half street width) • Roadway surface on Gloucester St. from Beatrice Ave. to Bellevue Ave.: 12,500 ft2 (contributing impervious area based on 500' length and 25' half street width)
Statement of need	• There is no water quality treatment in the contributing drainage area. • City maintenance staff report surface water flooding due to poor grade and lack of roadway crown along (1) Arlington St. from Barton Ave. to Bellevue Ave. and (2) Gloucester St. from Beatrice Ave. to Bellevue Ave. • On Arlington St., two existing catch basins currently drain to the sanitary sewer.
Project description	• Install 1,335 ft2 of stormwater planters on Arlington St. and 745 ft^2 of stormwater planters on Gloucester St. (in the locations identified above) to improve drainage and water quality treatment. Stormwater planter sizing is based on infiltration of the water quality design storm and contributing impervious area associated with the contributing ROW and half of the street width. • On Arlington, disconnect two existing catch basins from sanitary system. Planters are designed to bypass to the road. • On Gloucester, install beehive overflows and connect to existing stormwater conveyance system.
Estimated total project cost	$270,000
Design assumptions	• Planter facility installation would be opportunistic as available ROW exists. Therefore, facility sizes (footprint area), the number of facilities, and associated costs may vary. Treatment of the contributing area is assumed with installation of the specified area of stormwater planter. • Infiltration testing required prior to design to confirm sizing of the planters. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City. • A dedicated stormwater collection and conveyance system does not currently existing along Arlington St. Installation of a storm system may be conducted in conjunction with future roadway improvements. Overflows and connection to the piped conveyance system may be considered for the stormwater planters at that time.

5.4.3 Basin F

CIP name	F-1. Caldwell to Hull Pipe Replacement/Realignment
Objective addressed	Flood control
Contributing drainage area	59 acres
Statement of need	• Modeling predicts existing-condition flooding along the 15" storm drain in the Tall Oaks apartment complex, located between Caldwell Ave. and Hull Ave. Maintenance staff have not reported flooding at this location, but do receive complaints about flooding in Hardway Ct. and Durie Ct. (located directly west of the apartment complex). • The storm drain currently serving Hardway Ct. and Durie Ct. is a 6" corrugated plastic line that is reported to have an undulating slope due to poor installation.
Project description	• Install a total of 1,347 LF of 24" RCP from MH F1000 at Franklin Ave., south of Cardwell Ave., to Durie Ct. at new MH F0110, turning north along Durie Ct. and Hardway Ct. to a new connection to the County-owned pipe on Hull Ave. at MH F0105. This realignment minimizes conveyance on private property. • The cost of this CIP accounts for replacement of six catch basins with inlet leads and installation of 8-48" diameter manholes.
Estimated total project cost	$570,000
Design assumptions	• The existing 15" storm drain alignment through the Tall Oaks apartment complex is currently used to convey private drainage to Hull Ave. Subbasins F0400 and F0200 remain connected to the existing 15" storm drain in the CIP model. Therefore, abandoning the existing 15" storm drain is not included in the CIP. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City. • This CIP requires connection to a County-owned pipe on Hull Ave. The condition and attributes of this pipe are unknown. Evaluation of the County-owned pipe should be conducted during detailed design.

5.4.4 Basin H

CIP name	H-1. System H Channel Improvement
Objective addressed	Water quality
Contributing drainage area	22 acres
Statement of need	• There is no water quality treatment in the contributing drainage area. • Modeling predicts minor existing-condition flooding along the open-channel system between nodes H0700 and H0400. Maintenance staff do not report system flooding. Field reconnaissance indicates that maintenance of the open channel (which runs adjacent to railroad tracks and ballast) may improve system capacity and enhance water quality treatment.
Project description	Conduct targeted maintenance activities including hand removal of non-native vegetation, sediment and trash removal, and replanting activities on approximately 1,000 LF of open channel between nodes H1200 and H0300.
Estimated total project cost	$36,000
Design assumptions	Cost estimate assumes 10' wide channel improvement and an average of 6" of sediment removal over the channel improvement area.

5.4.5 Basin J

CIP name	J-1. Cornell at Landon Pipe Replacement/Realignment
Objective addressed	Flood control
Contributing drainage area	74 acres
Statement of need	• Modeling predicts existing-condition flooding in System J between Kenmore St. and High St. and along Oatfield Rd. between Ridgegate Dr. and Stone Oaks Ct. • Maintenance staff report the manhole cover at the upstream intersection of Oatfield Rd. and Ridgegate Dr. (MH J2700) blowing off during storm events. Maintenance staff also report flooding and debris accumulation in the piped system from Patricia Dr. and Kenmore St. • Much of the storm drain alignment in System J is on private property.
Project description	• Due to the extensive nature of flooding in System J, CIPs J-1 and J-2 should be conducted in tandem. CIP J-1 addresses the downstream portion of the system and CIP J-2 addresses the upstream portion of the system. • Abandon 397 LF of existing 15" and 18" diameter storm drain that is routed through private property between Cornell Ave. and High St. (J6500 to J6400, J1200 to J0900). • Install 334 LF of 18" HDPE from MH J6500 on Cornell Ave. south to MH J1300 on Cornell Ave. This new pipe is located in the public ROW and bypasses flow around the existing 18" storm drain on private property between Cornell St. and High St. (to be abandoned). • Replace 906 LF of 18" storm pipe with 30" HDPE from MH J1300 on Cornell Ave. to MH J0600 on High St., south of Kenmore St.
Estimated total project cost	$640,000
Design assumptions	• Existing flooding from Patricia Dr. to Kenmore St. is partially addressed by this CIP due to the flow diversion to Cornell St. and High St. CIP A-3 provides additional capacity downstream of High St. to address additional flow associated with this diversion. Installation of CIP A-3 should occur prior to installation of CIP J-1. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City. • It is recommended that CIP J-1 be installed prior to CIP J-2.

CIP name	J-2. Oatfield Road Pipe Replacement
Objective addressed	Flood control
Contributing drainage area	38 acres
Statement of need	• Modeling predicts existing-condition flooding in System J between Ridgegate Dr. and Stone Oaks Ct. • Maintenance staff report the manhole cover at the upstream intersection of Oatfield Rd. and Ridgegate Dr. (MH J2700) blowing off during storm events.
Project description	• Due to the extensive nature of flooding in System J, CIPs J-1 and J-2 should be conducted in tandem. CIP J-1 addresses the downstream portion of the system and CIP J-2 addresses the upstream portion of the system. • Replace 790 LF of 12" CSP with 18" RCP from MH J2800 at Oatfield Rd., north of Ridgegate Dr., to MH J2500 on Oatfield Rd. • Replace 623 LF of 12" and 18" CSP with 24" RCP from MH J2500 on Oatfield Rd. to MH J2000 at the intersection of Oatfield Rd. and Barbary Dr.
Estimated total project cost	$480,000
Design assumptions	• City maintenance reports bedrock in the project vicinity. The CIP cost estimate does not include rock blasting, which may be required for construction. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City. • It is recommended that CIP J-1 be installed prior to CIP J-2.

5.4.6 Basin M

CIP name	M-1. Crownview Drive Pipe Replacement
Objective addressed	Flood control
Contributing drainage area	13 acres
Statement of need	• Modeling predicts flooding along the existing 15" storm drain from the connection to the county pipe system north of Crownview Dr. to MH M0800, which is between Crownview Dr. and Valley View Dr. on private property. City maintenance staff also reports flooding, possibly due to routing (lack of manholes and catch basins) in this area. • Much of the storm drain alignment in System M is on private property.
Project description	• Install 542 LF of 18" HDPE from MH M0500 on Crownview Dr. to the County-owned MH M0100. • The cost of this CIP accounts for replacement of four catch basins with inlet leads and installation of three new manholes.
Estimated total project cost	$160,000
Design assumptions	• The existing 15" storm drain transitions from City to County ownership at MH M0100. The condition and attributes of this County pipe are unknown. Evaluation of the County-owned pipe should be conducted during detailed design. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City.

5.4.7 Basin N

CIP name	N-1. Kraxberger Bioswale and Pipe Replacement
Objective addressed	Flood control and water quality
Contributing drainage area	93 acres
Statement of need	• Modeling predicts existing-condition flooding along Webster Rd. and upstream through the Kraxberger Middle School grounds to Ridgegate Ct. Flooding along Webster Rd. was confirmed by City maintenance staff and Clackamas County staff. The flooding along Webster reportedly requires road closures during large storm events. • Water quality treatment is limited throughout Basin N.
Project description	• Install a parallel bioswale for 500 LF from MH N1050 at the school driveway entrance on Webster Rd. to MH N0800 near Springhill Place and Webster Rd. The bioswale utilizes a grassy strip along the west side of Webster Rd. • Replace 202 LF of 12" CSP with 24" RCP from MH N0800 to MH N0500 near Los Verdes Dr. and Webster Rd. • Install 67 LF of 24" RCP from MH N0401 to MH N0500 to divert piped flow along the east side of Webster Rd. to the new storm system along the west side of Webster Rd. • Replace 905 LF of 21" and 24" CSP with 36" RCP from N0500 to the point of County ownership at MH N0100, north of SE 5th Ave. and Webster Rd.
Estimated total project cost	$940,000
Design assumptions	• The existing 24" storm drain transitions from City to County ownership at MH N0100. The condition and attributes of this County pipe are unknown, but both City and County maintenance staff report Webster Rd. as a location of severe flooding. Evaluation of the County-owned pipe should be conducted during detailed design. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City.

CIP name	N-2. System N Inlet Replacement
Objective addressed	Flood control
Contributing drainage area	Varies
Statement of need	Localized flooding in System N is reported by City maintenance staff and is attributed to a lack of catch basins, use of single catch basins (as opposed to double catch basins or curb inlets), and clogging of catch basins with leaf debris.
Project description	Install new inlets in the following locations: • Los Verdes Dr. between Crownview Dr. and Via Del Verde: – Install two new inlets near 7145 Los Verdes Dr. with a new 450 LF 12" HDPE storm drain to Crownview Dr. – Install two new inlets at the west side of the intersection of Los Verdes Dr. and Crownview Dr. – Install a new inlet on the southeast side of the intersection of Monte Verde Dr. and Los Verdes Dr., and connect the new inlet to the existing storm drain. • The intersection of Crownview Dr. and Valley View Dr.: – Install two new inlets at the west side of the intersection of Valley View Dr. and Crownview Dr. Connect new inlets to the existing storm drain. • Lundgren Dr. and Charolais Way: – Install a new inlet at the northwestern corner of Charolais Way and Lungren Way. Connect the new inlet to the existing storm drain.
Estimated total project cost	$140,000
Design assumptions	Inlet replacement locations were identified based on a windshield survey of Monte Verde Dr., Los Verdes Dr., Valley View Dr., Crownview Dr., Charolais Dr., and Lundgren Dr.

5.4.8 Basin O

CIP name	O-1. Ridgewood and Oatfield Pipe Replacement
Objective addressed	Flood control
Contributing drainage area	67 acres
Statement of need	• Modeling predicts existing-condition flooding along Clayton Way, Stonewood Dr., Ridgewood Dr., and Cason Cir. to the outfall at the pond west of Poplar Ln. City maintenance staff reports that flooding from the intersection of Ridge Dr. and Cason Rd. impacts Cason Cir. • Additional flooding is reported on private property between Ridgewood Dr. and Ridge Dr. The characteristics of the storm line on private property are unknown.
Project description	• Replace 384 LF of 12" and 15" CSP with 18" RCP along Clayton Way to Webster Rd. (MH 01930 to 01900). • Replace 1,105 LF of 12" and 15" CSP with 24" RCP from Webster Rd. to Ridgewood Dr. (MH 01900 to MH 00700). Approximately 336 LF of this alignment is on private property. • The existing 36" HDPE from MH 00700 to MH 00600 has sufficient capacity and is not replaced. • Increase the capacity of the existing 257 LF of open channel on private property from node 00600 to the pond. Increase the channel width from 1' to 2', depth from 1' to 2', and side slope from 2:1 (H:V) to 3:1 (H:V). Channel improvements are costed to include installation of stream bed gravel and water quality facility plantings on the channel side slopes.
Estimated total project cost	$650,000
Design assumptions	• There are known water system deficiencies on private property between Ridgewood Dr. and Ridge Dr. that should be addressed concurrently with this CIP. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City. • An alternative alignment from Webster Rd. and Ridgewood Dr., down System E on Webster Rd., was evaluated and deemed cost-prohibitive due to shallow bedrock in Webster Rd. and the length of required pipe replacement along this alignment, which was more than 1 mile.

CIP name	O-2. Church Pond Retrofit Evaluation
Objective addressed	Water quality
Contributing drainage area	73 acres
Statement of need	• Water quality treatment is limited throughout Basin O. The downstream location of this existing retention pond facility would address water quality for a large contributing drainage area, comprising higher pollutant generating land use including commercial, industrial campus, and residential properties. • The functionality and inlet/outlet controls of the pond are currently unknown. Standing water is reported in the pond at all times.
Project description	Conduct survey and evaluation of existing pond to determine functionality and ability to utilize the pond as a regional water quality facility.
Estimated planning cost	$15,000
Design assumptions	• Detailed design and constructed facility improvements are not reflected in the cost estimate at this time. The cost estimate reflects a lump sum for a planning-level evaluation of the pond. • City staff to identify ownership of the pond and obtain necessary easements prior to survey and evaluation.

5.4.9 Green Streets Pilot Project

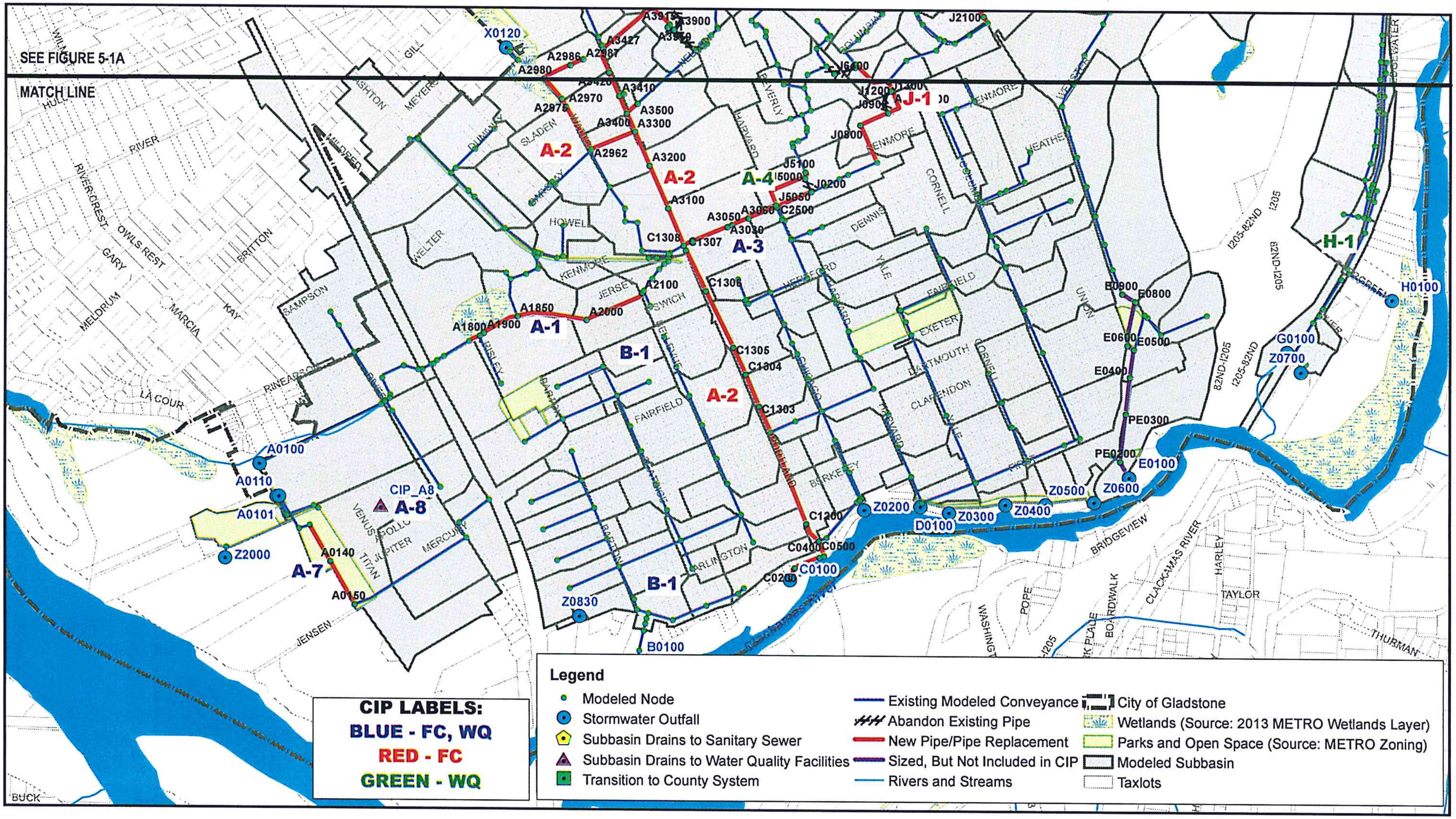
CIP name	Green Streets Pilot Project
Objective addressed	Flood control and water quality
Contributing drainage area	Varies (cost estimate by typical block area = 500' length by 50' street width)
Statement of need	• Water quality treatment is limited throughout the city. Green street applications in conjunction with infrastructure and roadway improvements are one option for stormwater retrofit. The City installed planter boxes along Portland Ave., which have proved successful at mitigating ponding and street flooding. • The City opted not to include green street applications as a standalone CIP or in conjunction with proposed flood control CIPs because applications are often opportunistic and based on available ROW. The City chose to include a green streets pilot project as part of its CIP, which would provide dedicated funding for green street applications as opportunities arise.
Project description	Depending on measured infiltration rates, preliminary green street sizing by assumed block area is as follows: • Type A/B soils (infiltration rate = 0.25"/hr or greater) = 1,490 ft^2 of planter area/block • Type C/D soils (infiltration rate = 0.10"/hr to 0.25"/hr) = 1,965 ft^2 of planter area/ block Targeted applications at this time include: • System A: Portland Ave. (from Glen Echo to Abernathy) • System C: Portland Ave. (from Jersey St. to Arlington St.) • System D: Yale Ave. (from Exeter St. to Arlington St.) • System D: Cornell St. (from Clarendon St. to Arlington St.) • System D: Columbia St. (from Gloucester St. to Arlington St.)
Estimated annual project allocation	$110,000
Design assumptions	• Flood control CIP sizing does not account for flow reduction that would occur with installation of green street applications. This is a conservative design assumption. Detailed design of flood control CIPs, if combined with a green street application, may be sized to account for flow reduction achieved with use of the planters in pipe sizing. • Infiltration testing is required prior to installation of any green street facility. • Property acquisition is not included in the cost estimate. Any easements required shall be obtained by the City.

5.4.10 CIP Summary

Estimated costs associated with each CIP are summarized in Table 5-1. Appendix C contains a detailed cost breakdown for each CIP.

Table 5-1. CIP Estimated Cost Summary

CIP number	CIP name	Total cost ($)
A-1	Rinearson Creek Stream Enhancement	410,000
A-2	Portland Avenue Bypass and Upstream Improvements	5,790,000
A-3	High School Storm Drain Improvements and Detention	1,840,000
A-4	High School Rain Garden	12,000
A-5	Tryon Rain Garden	220,000
A-6	Glen Echo Pipeline Realignment	280,000
A-7	Meldrum Bar Bioswale	230,000
A-8	Riverdale Drainage Improvements	280,000
B-1	Basin B Drainage Improvements	270,000
F-1	Caldwell to Hull Pipe Replacement/Realignment	570,000
H-1	System H Channel Improvement	36,000
J-1	Cornell at Landon Pipe Replacement/Realignment	640,000
J-2	Oatfield Pipe Replacement	480,000
M-1	Crownview Drive Pipe Replacement	160,000
N-1	Kraxberger Middle School Bioswale and Pipe Replacement	940,000
N-2	System N Inlet Replacement	140,000
O-1	Ridgewood and Oatfield to Pond Pipe Replacement	650,000
O-2	Church Pond Retrofit	15,000
	Total	12,963,000
	Annual Green Streets Pilot Project	110,000/year



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January 1, 2014

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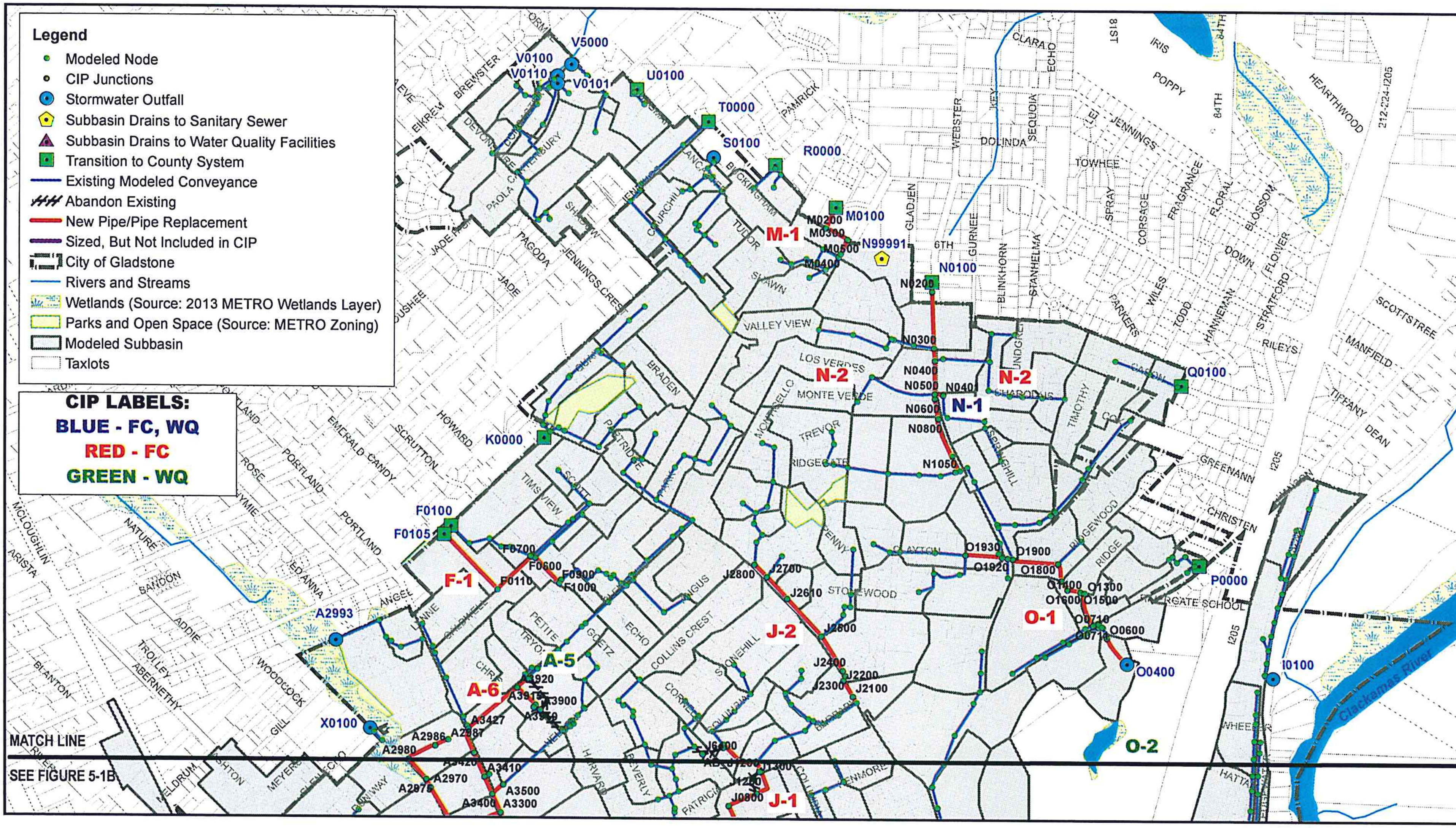
N

CITY OF GLADSTONE
STORMWATER MASTER PLAN

DRAINAGE SYSTEM - NORTH
CAPITAL IMPROVEMENT
PROJECT SUMMARY
FIGURE 5-1B



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January 1, 2014

CITY OF GLADSTONE
STORMWATER MASTER PLAN

DRAINAGE SYSTEM - NORTH
CAPITAL IMPROVEMENT
PROJECT SUMMARY
FIGURE 5-1A

Section 6

CIP Prioritization and Implementation

This section summarizes the general process the City used to prioritize identified CIPs and schedule project funding. The City conducted its CIP prioritization in conjunction with its stormwater financial evaluation (separate deliverable).

6.1 CIP Prioritization Criteria and Process

As described in Section 5, a total of 18 CIPs were developed to address flood control and water quality improvement within the city of Gladstone. To the extent possible, individual CIPs were developed to address multiple objectives (e.g., addressing flood control, regulatory compliance, water quality improvement, etc.). Please note that the Green Streets Pilot Project is not considered a CIP for inclusion in the overall prioritization, as it is a programmatic activity proposed for annual funding.

All CIPs identified in Section 5 are considered priority and recommended for implementation. Due to the significant cost of the CIPs proposed, the City's limited existing stormwater fund balance, and the fact a stormwater utility has not yet been formed, an extended implementation period of 30 years was used for the financial evaluation. Therefore, the 30-year implementation period as opposed to the traditional 20-year planning horizon was used for CIP scheduling.

During the CIP development workshop on October 29, 2013, City staff identified criteria to be used to schedule and prioritize CIP implementation (see Table 6-1). Criteria include historical/persistent problems, flooding/safety issues, regulatory compliance, ongoing maintenance, water quality improvement, project concurrence, and system sustainability. Identified criteria can overlap (e.g., water quality improvements would also address regulatory compliance). Such overlap creates an indirect weighting of projects for implementation based on the City's deemed importance of the overlapping issue.

Table 6-1. Multi-Objective CIP Prioritization Criteria

Criterion	Higher Priority	Lower Priority
Historical problem/ persistent problem	City staff considers area or system to be of ongoing concern	New CIP identified as part of this evaluation
Flooding issue/safety concern	• Significant hazard or threat to public safety or property • System experiences flooding for longer than a 2-hour duration during a modeled 10-year design event • Flooding currently observed.	No safety hazard addressed with CIP
NPDES permit requirements	Addresses NPDES permit requirement related to stormwater retrofits	Does not directly address NPDES permit requirements
Ongoing maintenance need	• City staff frequently responds to citizen complaints in the area • Frequent onsite response/maintenance required	City staff does not maintain facility outside of a typical maintenance cycle
Water quality improvement	Facility installation will directly reduce TMDL/303(d) pollutants to receiving water bodies.	CIP does not address water quality control
Concurrence	Required prerequisite or preliminary project for other CIPs.	CIP construction scheduling would not impact or be impacted by other stormwater or infrastructure projects
Sustainability	CIP would provide long-term benefits (aesthetics, livability, etc.)	CIP would address immediate need but may not enhance or improve benefits over the long term

6.2 CIP Scheduling

As each identified CIP is recommended for implementation, the prioritization criteria identified in Table 6-1 was used to identify those highest priority CIPs that should be scheduled and completed first within the implementation period. CIPs that address the most criteria were considered highest priority. Therefore, multi-objective CIPs (that address flood control and water quality) were prioritized over CIPs that just address one objective.

After evaluating the CIPs and criteria, the highest priority projects in terms of scheduling are as follows:

1. Portland Avenue Bypass (CIP A-2.1, CIP A-2.2, and CIP A-2.3)
2. Riverdale Drainage Improvements (A-8)
3. Basin B Drainage Improvements (B-1)
4. Kraxberger Middle School Bioswale and Pipe Replacement (N-1)

Each of these projects addresses historic and reported flooding issues, water quality improvement and NPDES compliance, and maintenance needs. Construction of CIP A-2.1, A-2.2, and A-2.3 are specifically required prior to construction of numerous other upstream system improvements, thereby addressing the project concurrence criteria. Two of the projects (A-8 and B-1) are proposed to eliminate known cross connections between the stormwater and sanitary conveyance systems and may reduce intermittent flooding.

It should be noted that water quality may also be considered with CIPs proposed solely to address flooding issues, even though the CIP was not developed specifically with water quality in mind. Incorporation of sedimentation or pollution control manholes or sumps could be installed in conjunction with pipe and inlet replacement projects (see CIP F-1, M-1 and N-1). Implementation of the annual green street pilot projects should be scheduled and located when other construction activities including stormwater pipe replacement projects are being installed to provide efficiencies.

6.3 CIP Implementation

As stated above, CIP implementation is projected over a 30-year period.

The initial financial analysis and stormwater rate evaluation effort includes the CIP project costs and anticipated project scheduling in development of recommended stormwater utility rates and system development charges (SDCs) (see Section 6.1 and 6.2). In addition, the financial analysis includes annual project costs (see the annual green street pilot project and expenditures (vehicle and equipment replacement) in the calculation of rates. An annual cost of $110,000 is assumed for implementation of the annual green street pilot project (see Section 5.4.9). An annual cost of $75,000 is dedicated for replacement of vehicles and equipment in support of the stormwater system (street sweeper, vactor, emergency response jetter, etc.).

Historically, due to limited staff availability, preventative maintenance of the stormwater system has not been performed routinely and proactively. The City's existing public works department consists of six full time staff that are shared amongst stormwater, sanitary, water, parks, and streets. There is no dedicated stormwater staff. Preventative maintenance is essential to optimizing functionality and performance of a stormwater system. Each identified CIP will require routine maintenance to ensure ongoing operation. Such maintenance efforts include regular vactoring and debris removal of catchbasins, manholes, and ditches; TV-inspection of pipes; planting and grading of vegetated stormwater facilities; trash, debris, and invasive vegetation removal along creeks and streams; and inspection and repair of hard infrastructure (culverts, manholes, outfalls, outlet control structures).

The financial analysis includes the addition of 2.5 full time employees (FTE) to supplement existing staff in support of a preventative stormwater maintenance program. With the addition of staff, and as preventative maintenance activities are conducted and tracked at specified intervals, the staffing allocation should be revisited amongst all utilities to ensure that adequate levels of service are achieved amongst all utilities.

Appendix A: Stormwater Conveyance System Map

Appendix B: Hydrologic and Hydraulic Results Tables

Table B-1: Hydrologic Results

Table B-2: Hydraulic Results

Table B-1. Hydrologic Input Data and Peak Flow Results

				Curve Number		Impervious Area (%)			Existing Land Use Scenario Subbasin Peak Flow (cubic feet per second)				Future Land Use Scenario Subbasin Peak Flow (cubic feet per second)			
Subbasin	Inlet Node	Area (acre)	Average Slope (%)	Existing Land Use	Future Land Use	Existing Land Use	Future Land Use	Percent Increase	2yr 24hr Design Storm	10yr 24hr Design Storm	25yr 24hr Design Storm	100yr 24hr Design Storm	2yr 24hr Design Storm	10yr 24hr Design Storm	25yr 24hr Design Storm	100yr 24hr Design Storm
BASIN A																
A0102	BA0102	13.7	0.0	89	92	60	74	23%	3.9	6.1	7.2	8.9	4.6	6.8	8.0	9.7
A0104	BA0104	14.6	0.6	82	82	50	54	8%	3.1	5.1	6.2	7.9	3.4	5.4	6.5	8.2
A0150	BA0150	8.1	1.3	92	92	80	80	0%	2.9	4.2	4.8	5.8	2.9	4.2	4.8	5.8
A0170	BA0170	12.0	0.3	93	93	80	80	0%	4.2	6.1	7.1	8.6	4.2	6.1	7.1	8.6
A0190	BA0190	5.1	1.2	97	97	95	95	0%	2.0	2.8	3.2	3.8	2.0	2.8	3.2	3.8
A0191	BA0191	5.1	2.5	96	96	95	95	0%	1.9	2.8	3.2	3.8	1.9	2.8	3.2	3.8
A0200	BA0200	6.1	10.3	88	92	55	73	33%	1.9	2.9	3.4	4.2	2.2	3.2	3.7	4.4
A0330	BA0330	10.7	0.9	90	91	65	70	8%	3.5	5.2	6.1	7.5	3.7	5.4	6.2	7.6
A0410	BA0410	4.6	0.5	97	97	95	95	0%	1.7	2.5	2.8	3.4	1.7	2.5	2.8	3.4
A0420	BA0420	3.2	0.1	97	97	95	95	0%	1.2	1.7	2.0	2.4	1.2	1.7	2.0	2.4
A0500	BA0500	6.3	9.4	97	97	95	95	0%	2.4	3.4	3.9	4.7	2.4	3.4	3.9	4.7
A1000	BA1000	9.9	2.4	97	97	95	95	0%	3.8	5.3	6.1	7.3	3.8	5.3	6.1	7.3
A1300	BA1300	6.3	0.3	97	97	95	95	0%	2.4	3.4	3.9	4.7	2.4	3.4	3.9	4.7
A1600	BA1600	13.0	10.1	88	94	55	82	49%	4.1	6.1	7.2	8.8	4.8	6.8	7.9	9.5
A1820	BA1820	10.4	1.3	86	89	50	64	28%	2.8	4.5	5.3	6.7	3.4	5.0	5.9	7.2
A1902	BA1902	10.1	5.4	86	87	50	55	10%	3.0	4.6	5.4	6.7	3.2	4.7	5.5	6.8
A1930	BA1930	5.7	7.4	78	92	10	73	630%	0.7	1.5	2.0	2.6	2.0	3.0	3.4	4.2
A1952	BA1952	2.3	1.0	86	88	50	58	16%	0.7	1.0	1.2	1.5	0.7	1.1	1.3	1.6
A1954	BA1954	3.8	1.4	86	87	50	55	10%	1.1	1.7	2.0	2.5	1.2	1.8	2.1	2.6
A1975	BA1975	3.6	6.3	86	87	50	55	10%	1.1	1.6	1.9	2.4	1.1	1.7	2.0	2.4
A1985	BA1985	8.3	4.8	87	89	55	61	11%	2.6	3.9	4.6	5.6	2.8	4.1	4.7	5.8
A1989	BA1989	6.8	5.3	82	87	35	55	57%	1.6	2.6	3.2	4.0	2.1	3.2	3.7	4.6
A1992	BA1992	10.1	0.7	84	88	40	60	50%	2.2	3.8	4.6	5.9	3.1	4.7	5.5	6.8
A2000	BA2000	6.6	2.6	82	84	50	58	16%	1.9	2.9	3.4	4.2	2.0	3.0	3.6	4.4
A2210	BA2210	5.3	0.9	81	87	50	68	36%	1.4	2.2	2.6	3.3	1.8	2.6	3.0	3.7
A2720	BA2720	2.7	5.0	85	87	45	56	24%	0.8	1.2	1.4	1.8	0.9	1.3	1.5	1.9
A2920	BA2920	8.1	8.0	87	92	50	72	44%	2.5	3.7	4.4	5.4	2.9	4.2	4.8	5.8
A2965	BA2965	4.4	2.9	91	91	65	65	0%	1.5	2.2	2.6	3.1	1.5	2.2	2.6	3.1
A2967	BA2967	8.1	5.1	86	87	50	55	10%	2.4	3.6	4.3	5.3	2.5	3.8	4.4	5.4
A2969	BA2969	4.9	3.8	86	87	50	55	10%	1.5	2.2	2.6	3.2	1.5	2.3	2.7	3.3
A2987	BA2987	4.7	5.3	93	96	75	88	17%	1.7	2.4	2.8	3.4	1.8	2.5	2.9	3.5
A2993	BA2993	14.7	2.2	83	86	30	42	40%	2.9	5.2	6.4	8.2	3.7	6.1	7.3	9.2
A2999	BA2999	2.3	9.1	86	87	50	55	10%	0.7	1.1	1.2	1.5	0.7	1.1	1.3	1.6
A3020	BA3020	4.4	0.4	83	88	60	73	22%	1.3	2.0	2.4	2.9	1.5	2.2	2.6	3.1
A3100	BA3100	6.5	0.2	81	88	20	55	175%	0.7	1.3	1.7	2.4	1.7	2.7	3.2	4.0
A3200	BA3200	9.2	8.3	87	88	50	55	10%	2.8	4.2	5.0	6.1	2.9	4.4	5.1	6.3
A3410	BA3410	3.7	0.6	93	93	70	74	6%	1.3	1.9	2.2	2.7	1.3	1.9	2.2	2.7
A34102	BA34102	3.3	2.1	87	88	50	55	10%	1.0	1.5	1.8	2.2	1.0	1.6	1.8	2.3
A3430	BA3430	1.7	3.8	86	87	50	55	10%	0.5	0.8	0.9	1.1	0.5	0.8	1.0	1.2
A3450	BA3450	4.9	3.9	85	87	45	55	22%	1.3	2.1	2.5	3.1	1.5	2.3	2.7	3.3
A3470	BA3470	3.1	0.3	84	87	40	55	38%	0.7	1.2	1.5	1.9	0.9	1.4	1.7	2.1
A3490	BA3490	4.3	2.7	85	87	45	55	22%	1.2	1.9	2.2	2.8	1.3	2.0	2.4	2.9
A3500	BA3500	7.2	4.7	85	88	40	55	38%	1.9	3.0	3.6	4.5	2.3	3.4	4.0	4.9
A3640	BA3640	6.2	8.4	85	87	45	55	22%	1.8	2.7	3.2	4.0	1.9	2.9	3.4	4.2
A3744	BA3744	3.6	6.4	86	87	50	55	10%	1.1	1.6	1.9	2.4	1.1	1.7	2.0	2.4
A3800	BA3800	2.0	1.5	87	88	50	55	10%	0.6	0.9	1.1	1.4	0.6	1.0	1.1	1.4
A3910	BA3910	5.8	7.6	83	88	35	55	57%	1.4	2.3	2.7	3.5	1.8	2.7	3.2	3.9
A4110	BA4110	2.0	0.2	87	88	50	55	10%	0.6	0.9	1.1	1.3	0.6	0.9	1.1	1.4
A4120	BA4120	2.9	6.4	84	87	40	55	38%	0.8	1.2	1.4	1.8	0.9	1.3	1.6	1.9
A4400	BA4400	6.8	2.1	84	87	40	55	38%	1.7	2.8	3.3	4.2	2.1	3.1	3.7	4.5
A4700	BA4700	3.4	8.2	85	87	45	55	22%	1.0	1.5	1.8	2.2	1.1	1.6	1.9	2.3
A5000	BA5000	4.2	2.4	86	87	50	55	10%	1.2	1.9	2.2	2.8	1.3	2.0	2.3	2.8
A5020	BA5020	5.9	5.7	87	88	50	55	10%	1.8	2.7	3.1	3.9	1.8	2.8	3.2	4.0
BASIN B																
B0600	BB0600	3.4	0.6	77.0	79.0	50.0	55.0	10%	0.8	1.3	1.6	2.0	0.9	1.4	1.7	2.1
B0620	BB0620	2.3	1.0	69.0	71.0	50.0	55.0	10%	0.5	0.8	1.0	1.3	0.5	0.9	1.1	1.3
B0640	BB0640	3.1	0.7	69.0	78.0	50.0	67.0	34%	0.6	1.1	1.4	1.7	0.9	1.4	1.7	2.0
B0800	BB0800	1.9	1.5	86.0	96.0	50.0	90.0	80%	0.6	0.9	1.0	1.2	0.7	1.0	1.2	1.4
B0900	BB0900	4.5	0.1	86.0	87.0	50.0	55.0	10%	1.2	1.9	2.3	2.9	1.3	2.0	2.4	3.0
B0901	BB0901	4.9	2.5	91.0	91.0	70.0	72.0	3%	1.7	2.5	2.9	3.5	1.7	2.5	2.9	3.5
B0911	BB0911	4.9	2.1	87.0	89.0	55.0	64.0	16%	1.5	2.3	2.7	3.3	1.6	2.4	2.8	3.4
B0920	BB0920	1.6	1.1	86.0	87.0	50.0	55.0	10%	0.5	0.7	0.9	1.1	0.5	0.8	0.9	1.1
B0921	BB0921	3.6	2.6	87.0	89.0	55.0	64.0	16%	1.1	1.7	2.0	2.4	1.2	1.8	2.1	2.5
B0930	BB0930	1.5	0.9	86.0	87.0	50.0	55.0	10%	0.4	0.7	0.8	1.0	0.5	0.7	0.8	1.0
B0931	BB0931	3.6	4.7	87.0	89.0	55.0	62.0	13%	1.1	1.7	2.0	2.4	1.2	1.7	2.0	2.5
B0940	BB0940	5.6	6.1	82.0	84.0	50.0	58.0	16%	1.6	2.4	2.9	3.6	1.7	2.6	3.0	3.7
B1000	BB1000	3.3	1.0	77.0	79.0	50.0	55.0	10%	0.8	1.3	1.6	2.0	0.9	1.4	1.7	2.1
B1001	BB1001	1.6	0.2	70.0	73.0	50.0	55.0	10%	0.3	0.5	0.7	0.8	0.4	0.6	0.7	0.9
B1010	BB1010	5.2	1.8	70.0	81.0	50.0	69.0	38%	1.2	2.0	2.4	2.9	1.7	2.5	2.9	3.5
B1011	BB1011	4.2	3.2	69.0	83.0	50.0	75.0	50%	0.9	1.6	1.9	2.4	1.4	2.1	2.4	2.9
B1020	BB1020	5.7	1.6	70.0	82.0	50.0	72.0	44%	1.1	2.0	2.4	3.1	1.8	2.7	3.2	3.9
B1040	BB1040	5.9	1.6	77.0	85.0	50.0	71.0	42%	1.5	2.4	2.8	3.5	2.0	2.9	3.3	4.1
B1070	BB1070	5.4	1.5	80.0	87.0	50.0	69.0	38%	1.3	2.2	2.6	3.3	1.8	2.6	3.1	3.7
B1100	BB1100	2.1	0.3	86.0	87.0	50.0	55.0	10%	0.6	0.9	1.1	1.4	0.6	1.0	1.1	1.4
B1200	BB1200	5.9	0.4	80.0	82.0	50.0	55.0	10%	1.5	2.4	2.9	3.6	1.6	2.6	3.1	3.8
B1300	BB1300	6.3	6.1	78.0	80.0	50.0	55.0	10%	1.6	2.6	3.1	3.8	1.7	2.7	3.2	4.0
B1501	BB1501	4.1	1.0	80.0	81.0	50.0	55.0	10%	1.1	1.7	2.0	2.5	1.2	1.8	2.1	2.6
B1510	BB1510	2.7	1.2	80.0	81.0	50.0	55.0	10%	0.7	1.1	1.4	1.7	0.8	1.2	1.4	1.7
B1520	BB1520	2.4	0.3	80.0	81.0	50.0	55.0	10%	0.6	1.0	1.2	1.5	0.7	1.1	1.2	1.5
B1600	BB1600	2.3	0.5	80.0	81.0	50.0	55.0	10%	0.6	1.0	1.1	1.4	0.7	1.0	1.2	1.5
B1700	BB1700	6.3	0.5	89.0	90.0	75.0	79.0	5%	2.1	3.1	3.6	4.5	2.2	3.2	3.7	4.5

Table B-1. Hydrologic Input Data and Peak Flow Results

Subbasin	Inlet Node	Area (acre)	Average Slope (%)	Curve Number		Impervious Area (%)			Existing Land Use Scenario Subbasin Peak Flow (cubic feet per second)				Future Land Use Scenario Subbasin Peak Flow (cubic feet per second)			
				Existing Land Use	Future Land Use	Existing Land Use	Future Land Use	Percent Increase	2yr 24hr Design Storm	10yr 24hr Design Storm	25yr 24hr Design Storm	100yr 24hr Design Storm	2yr 24hr Design Storm	10yr 24hr Design Storm	25yr 24hr Design Storm	100yr 24hr Design Storm
BASIN C																
C0500	BC0500	2.8	0.1	69.0	92.0	50.0	90.0	80%	0.6	0.9	1.1	1.4	1.1	1.5	1.7	2.1
C0810	BC0810	2.3	2.5	69.0	72.0	50.0	56.0	12%	0.5	0.9	1.0	1.3	0.6	0.9	1.1	1.4
C1000	BC1000	3.4	1.2	72.0	75.0	50.0	55.0	10%	0.7	1.3	1.5	1.9	0.9	1.4	1.6	2.0
C1100	BC1100	4.1	0.5	77.0	79.0	50.0	56.0	12%	0.8	1.4	1.7	2.1	1.0	1.5	1.8	2.3
C1301	BC1301	2.6	4.3	78.0	88.0	50.0	73.0	46%	0.7	1.1	1.3	1.6	0.9	1.3	1.5	1.8
C1302	BC1302	1.6	0.1	69.0	92.0	50.0	90.0	80%	0.3	0.5	0.7	0.8	0.6	0.9	1.0	1.2
C1303	BC1303	3.9	1.1	80.0	89.0	50.0	75.0	50%	1.1	1.6	1.9	2.4	1.4	2.0	2.3	2.8
C1305	BC1305	5.5	0.2	81.0	90.0	55.0	80.0	45%	1.4	2.3	2.7	3.4	1.9	2.8	3.3	3.9
C1310	BC1310	6.0	0.6	76.0	81.0	40.0	55.0	38%	1.0	1.8	2.2	2.9	1.5	2.4	2.8	3.6
C1320	BC1320	8.0	3.4	74.0	77.0	35.0	42.0	20%	1.3	2.5	3.1	4.0	1.6	2.9	3.5	4.4
C1370	BC1370	5.6	4.9	82.0	84.0	50.0	55.0	10%	1.6	2.4	2.8	3.5	1.7	2.5	3.0	3.7
C1380	BC1380	3.7	0.5	82.0	83.0	50.0	55.0	10%	0.9	1.5	1.8	2.2	1.0	1.6	1.8	2.3
C1400	BC1400	2.3	0.2	92.0	92.0	85.0	85.0	0%	0.9	1.2	1.4	1.7	0.9	1.2	1.4	1.7
C1700	BC1700	3.7	1.1	80.0	81.0	50.0	55.0	10%	0.9	1.5	1.8	2.2	1.0	1.6	1.8	2.3
C1830	BC1830	2.7	0.3	83.0	87.0	60.0	70.0	17%	0.8	1.2	1.4	1.8	0.9	1.3	1.5	1.9
C1900	BC1900	4.7	0.8	80.0	82.0	50.0	55.0	10%	1.2	2.0	2.3	2.9	1.3	2.1	2.5	3.0
C2030	BC2030	4.1	15.2	90.0	90.0	65.0	65.0	0%	1.4	2.0	2.4	2.9	1.4	2.0	2.4	2.9
C2510	BC2510	3.0	19.8	85.0	88.0	40.0	55.0	38%	0.9	1.3	1.5	1.9	1.0	1.4	1.7	2.1
BASIN D																
D0200	BD0200	2.0	0.5	77.0	79.0	50.0	55.0	10%	0.5	0.8	1.0	1.2	0.5	0.9	1.0	1.3
D0210	BD0210	3.3	0.1	80.0	81.0	50.0	55.0	10%	0.7	1.2	1.4	1.8	0.8	1.3	1.5	1.9
D0240	BD0240	5.9	0.9	80.0	81.0	50.0	55.0	10%	1.5	2.4	2.9	3.6	1.6	2.5	3.0	3.7
D0300	BD0300	6.8	0.2	78.0	81.0	45.0	55.0	22%	1.4	2.4	2.9	3.8	1.7	2.8	3.3	4.2
D0310	BD0310	11.9	6.3	80.0	82.0	50.0	55.0	10%	2.9	4.7	5.7	7.1	3.2	5.1	6.0	7.5
D0320	BD0320	3.2	7.1	71.0	72.0	25.0	29.0	16%	0.5	1.0	1.2	1.5	0.6	1.0	1.3	1.6
D0340	BD0340	2.0	5.0	82.0	82.0	45.0	46.0	2%	0.6	0.8	1.0	1.2	0.6	0.8	1.0	1.2
D0350	BD0350	2.3	11.0	86.0	87.0	50.0	55.0	10%	0.7	1.1	1.3	1.6	0.7	1.1	1.3	1.6
D0360	BD0360	2.9	3.9	86.0	87.0	50.0	55.0	10%	0.9	1.3	1.6	1.9	0.9	1.4	1.6	2.0
D0370	BD0370	9.2	5.1	86.0	87.0	50.0	55.0	10%	2.5	4.0	4.7	5.9	2.7	4.2	4.9	6.1
D0400	BD0400	1.9	0.2	80.0	83.0	50.0	60.0	20%	0.5	0.8	1.0	1.2	0.6	0.9	1.0	1.3
D0410	BD0410	3.9	0.8	80.0	81.0	50.0	55.0	10%	1.0	1.6	1.9	2.4	1.1	1.7	2.0	2.5
D0440	BD0440	5.3	0.9	80.0	81.0	50.0	55.0	10%	1.5	2.2	2.6	3.3	1.5	2.3	2.7	3.4
D0600	BD0600	4.4	0.7	92.0	92.0	85.0	85.0	0%	1.6	2.3	2.7	3.2	1.6	2.3	2.7	3.2
D0700	BD0700	8.8	3.9	80.0	85.0	45.0	63.0	40%	2.0	3.4	4.1	5.2	2.7	4.1	4.8	5.9
D0800	BD0800	7.3	7.3	84.0	86.0	50.0	55.0	10%	1.8	3.0	3.6	4.5	2.0	3.2	3.8	4.7
D0900	BD0900	3.4	6.3	84.0	86.0	50.0	55.0	10%	1.0	1.5	1.8	2.2	1.0	1.6	1.8	2.3
D1000	BD1000	4.4	3.6	86.0	87.0	50.0	55.0	10%	1.3	2.0	2.3	2.9	1.4	2.1	2.4	3.0
D1100	BD1100	2.8	5.4	86.0	87.0	50.0	55.0	10%	0.9	1.3	1.5	1.9	0.9	1.3	1.6	1.9
D1120	BD1120	2.0	5.4	86.0	87.0	50.0	55.0	10%	0.6	0.9	1.1	1.3	0.6	1.0	1.1	1.4
D1300	BD1300	2.1	4.9	84.0	87.0	40.0	55.0	38%	0.5	0.9	1.0	1.3	0.6	1.0	1.1	1.4
D1310	BD1310	4.8	13.4	86.0	87.0	50.0	55.0	10%	1.4	2.2	2.6	3.2	1.5	2.2	2.6	3.2
D1360	BD1360	7.7	4.8	86.0	87.0	50.0	55.0	10%	2.1	3.3	3.9	4.9	2.2	3.5	4.1	5.1
D1700	BD1700	4.3	8.6	86.0	87.0	50.0	55.0	10%	1.3	2.0	2.3	2.9	1.4	2.0	2.4	2.9
D1800	BD1800	3.3	10.0	82.0	87.0	35.0	55.0	57%	0.8	1.3	1.5	1.9	1.0	1.5	1.8	2.2
D2200	BD2200	4.3	7.7	86.0	87.0	50.0	55.0	10%	1.3	2.0	2.3	2.9	1.4	2.0	2.4	2.9
BASIN E																
E0100	BE0100	1.6	6.0	80.0	92.0	70.0	90.0	29%	0.5	0.8	0.9	1.1	0.6	0.8	1.0	1.2
E0200	BE0200	8.9	6.7	91.0	93.0	85.0	89.0	5%	3.3	4.7	5.4	6.5	3.3	4.7	5.4	6.6
E0210	BE0210	3.4	0.0	94.0	94.0	90.0	90.0	0%	1.2	1.8	2.0	2.5	1.2	1.8	2.0	2.5
E0400	BE0400	4.3	6.1	93.0	93.0	80.0	80.0	0%	1.6	2.3	2.6	3.2	1.6	2.3	2.6	3.2
E0520	BE0520	2.6	4.4	95.0	96.0	85.0	90.0	6%	1.0	1.4	1.6	1.9	1.0	1.4	1.6	1.9
E0523	BE0523	4.6	2.0	96.0	96.0	90.0	90.0	0%	1.7	2.4	2.8	3.4	1.7	2.4	2.8	3.4
E0900	BE0900	20.6	5.5	79.0	86.0	20.0	50.0	150%	2.3	4.9	6.4	8.7	5.3	8.6	10.2	12.8
E1300	BE1300	8.2	6.2	75.0	86.0	5.0	50.0	900%	0.8	1.9	2.5	3.4	2.4	3.7	4.4	5.4
E1600	BE1600	14.0	1.5	80.0	92.0	25.0	76.0	204%	1.6	3.1	3.9	5.4	4.8	7.0	8.1	9.9
E2200	BE2200	7.5	4.1	79.0	93.0	20.0	80.0	300%	1.1	2.2	2.8	3.7	2.7	3.9	4.5	5.5
BASIN F																
F0100	BF0100	6.0	3.9	85.0	88.0	40.0	53.0	33%	1.3	2.3	2.8	3.5	1.7	2.7	3.2	3.9
F0200	BF0200	7.0	19.1	85.0	88.0	40.0	55.0	38%	1.9	3.0	3.6	4.4	2.2	3.3	3.9	4.8
F0400	BF0400	3.3	16.7	85.0	88.0	40.0	55.0	38%	0.9	1.4	1.7	2.1	1.1	1.6	1.9	2.3
F0610	BF0610	1.9	16.5	84.0	87.0	40.0	55.0	38%	0.5	0.8	1.0	1.2	0.6	0.9	1.0	1.3
F0630	BF0630	2.4	11.7	86.0	88.0	45.0	55.0	22%	0.7	1.0	1.2	1.5	0.8	1.1	1.3	1.6
F0650	BF0650	3.0	14.1	85.0	88.0	40.0	55.0	38%	0.8	1.3	1.5	1.9	0.9	1.4	1.6	2.0
F0710	BF0710	0.7	9.7	86.0	87.0	50.0	55.0	10%	0.2	0.3	0.4	0.5	0.2	0.4	0.4	0.5
F0720	BF0720	1.9	12.9	83.0	89.0	30.0	55.0	83%	0.4	0.7	0.9	1.1	0.6	0.9	1.1	1.3
F0900	BF0900	4.5	4.5	83.0	88.0	35.0	55.0	57%	1.1	1.8	2.2	2.7	1.4	2.1	2.5	3.1
F1400	BF1400	6.3	11.6	84.0	88.0	40.0	55.0	38%	1.7	2.6	3.1	3.9	2.0	3.0	3.5	4.3
BASIN G																
G0400	BG0400	4.4	2.0	66.0	86.0	20.0	71.0	255%	0.4	1.0	1.3	1.8	1.5	2.2	2.5	3.1
G0700	BG0700	1.2	1.2	85.0	85.0	65.0	65.0	0%	0.4	0.6	0.7	0.8	0.4	0.6	0.7	0.8
BASIN H																
H0300	BH0300	0.9	1.1	87.0	87.0	70.0	70.0	0%	0.3	0.4	0.5	0.6	0.3	0.4	0.5	0.6
H0520	BH0520	1.8	1.1	87.0	87.0	70.0	70.0	0%	0.6	0.9	1.0	1.2	0.6	0.9	1.0	1.2
H0710	BH0710	2.4	12.8	68.0	88.0	20.0	73.0	265%	0.3	0.7	0.8	1.1	0.8	1.2	1.4	1.7
H0720	BH0720	5.7	24.6	81.0	95.0	45.0	90.0	100%	1.5	2.4	2.8	3.5	2.1	3.0	3.5	4.2
H0820	BH0820	0.8	14.8	76.0	82.0	40.0	56.0	40%	0.2	0.3	0.4	0.5	0.3	0.4	0.4	0.5
H0870	BH0870	1.0	1.7	83.0	89.0	40.0	64.0	60%	0.3	0.4	0.5	0.6	0.4	0.5	0.6	0.7
H1200	BH1200	7.4	6.4	77.0	94.0	40.0	89.0	123%	1.5	2.6	3.2	4.1	2.8	3.9	4.5	5.4
H2100	BH2100	4.6	11.7	84.0	96.0	40.0	90.0	125%	1.3	2.0	2.3	2.9	1.7	2.5	2.8	3.4

Table B-1. Hydrologic Input Data and Peak Flow Results																
				Curve Number		Impervious Area (%)			Existing Land Use Scenario Subbasin Peak Flow (cubic feet per second)				Future Land Use Scenario Subbasin Peak Flow (cubic feet per second)			
Subbasin	Inlet Node	Area (acre)	Average Slope (%)	Existing Land Use	Future Land Use	Existing Land Use	Future Land Use	Percent Increase	2yr 24hr Design Storm	10yr 24hr Design Storm	25yr 24hr Design Storm	100yr 24hr Design Storm	2yr 24hr Design Storm	10yr 24hr Design Storm	25yr 24hr Design Storm	100yr 24hr Design Storm
BASIN I																
I0510	BI0510	5.4	9.7	96.0	96.0	90.0	90.0	0%	2.0	2.9	3.3	4.0	2.0	2.9	3.3	4.0
I1030	BI1030	4.0	6.2	96.0	96.0	90.0	90.0	0%	1.5	2.1	2.4	2.9	1.5	2.1	2.4	2.9
I1400	BI1400	3.7	7.9	96.0	#REF!	35.0	90.0	157%	0.9	1.5	1.8	2.2	1.4	2.0	2.3	2.7
BASIN J																
J0200	BJ0200	9.3	16.4	83.0	88.0	35.0	55.0	57%	2.4	3.8	4.5	5.7	3.0	4.4	5.2	6.3
J0800	BJ0800	6.3	4.5	86.0	87.0	50.0	55.0	10%	1.9	2.9	3.4	4.2	2.0	3.0	3.5	4.3
J1300	BJ1300	3.2	4.3	84.0	87.0	40.0	55.0	38%	0.9	1.4	1.6	2.0	1.0	1.5	1.8	2.2
J1600	BJ1600	4.4	14.0	84.0	87.0	40.0	55.0	38%	1.2	1.9	2.2	2.8	1.4	2.1	2.4	3.0
J1710	BJ1710	3.1	6.0	84.0	87.0	40.0	55.0	38%	0.8	1.3	1.5	1.9	1.0	1.5	1.7	2.1
J1750	BJ1750	3.7	6.3	84.0	88.0	40.0	55.0	38%	1.0	1.6	1.9	2.3	1.2	1.8	2.1	2.6
J2000	BJ2000	2.6	6.2	75.0	96.0	5.0	90.0	1700%	0.2	0.5	0.7	1.0	1.0	1.4	1.6	1.9
J2300	BJ2300	8.6	8.4	81.0	89.0	25.0	59.0	136%	1.6	3.0	3.6	4.7	2.8	4.2	4.9	6.0
J2520	BJ2520	4.4	6.6	85.0	87.0	45.0	55.0	22%	1.2	1.9	2.3	2.8	1.4	2.1	2.4	3.0
j2560	Bj2560	3.4	7.0	85.0	88.0	40.0	54.0	35%	0.9	1.5	1.7	2.2	1.1	1.6	1.9	2.3
J2610	BJ2610	1.8	7.0	85.0	87.0	45.0	55.0	22%	0.5	0.8	0.9	1.2	0.6	0.8	1.0	1.2
J2710	BJ2710	6.4	11.7	85.0	86.0	40.0	46.0	15%	1.7	2.7	3.2	4.0	1.8	2.8	3.3	4.1
J3000	BJ3000	2.8	7.3	85.0	87.0	45.0	55.0	22%	0.8	1.2	1.4	1.8	0.9	1.3	1.5	1.9
J3210	BJ3210	3.8	21.7	83.0	85.0	50.0	55.0	10%	1.1	1.7	2.0	2.5	1.2	1.8	2.1	2.5
J3600	BJ3600	4.3	3.4	85.0	87.0	45.0	55.0	22%	1.2	1.9	2.2	2.8	1.4	2.0	2.4	2.9
J5300	BJ5300	7.1	1.2	86.0	87.0	50.0	55.0	10%	1.9	3.1	3.6	4.5	2.1	3.2	3.8	4.7
J5820	BJ5820	2.7	2.6	86.0	87.0	50.0	55.0	10%	0.8	1.2	1.5	1.8	0.9	1.3	1.5	1.8
J6100	BJ6100	4.8	5.3	86.0	87.0	50.0	55.0	10%	1.4	2.2	2.6	3.2	1.5	2.3	2.6	3.3
J6130	BJ6130	5.2	14.2	87.0	87.0	55.0	55.0	0%	1.7	2.5	2.9	3.5	1.7	2.5	2.9	3.5
J6180	BJ6180	7.8	0.4	86.0	87.0	50.0	55.0	10%	2.0	3.3	3.9	4.9	2.2	3.4	4.1	5.1
J6331	BJ6331	5.1	4.4	86.0	87.0	50.0	55.0	10%	1.5	2.3	2.7	3.3	1.6	2.4	2.8	3.4
J6500	BJ6500	2.8	2.9	84.0	87.0	40.0	55.0	38%	0.7	1.1	1.3	1.7	0.8	1.3	1.5	1.8
J6600	BJ6600	3.3	4.8	84.0	87.0	40.0	55.0	38%	0.9	1.4	1.6	2.1	1.0	1.5	1.8	2.2
J7100	BJ7100	8.6	7.0	86.0	88.0	50.0	55.0	10%	2.6	3.9	4.6	5.7	2.7	4.1	4.8	5.9
BASIN K																
K0120	BK0120	4.3	15.7	75.0	75.0	25.0	25.0	0%	0.8	1.4	1.7	2.2	0.8	1.4	1.7	2.2
K0130	BK0130	2.9	16.0	84.0	84.0	55.0	55.0	0%	0.9	1.3	1.5	1.9	0.9	1.3	1.5	1.9
K0160	BK0160	1.7	23.6	79.0	81.0	45.0	51.0	13%	0.5	0.7	0.8	1.0	0.5	0.7	0.9	1.1
K0200	BK0200	8.0	0.6	77.0	86.0	20.0	55.0	175%	0.8	1.6	2.1	2.9	2.1	3.3	4.0	5.0
K0500	BK0500	4.0	1.2	83.0	86.0	40.0	53.0	33%	1.0	1.6	1.9	2.4	1.2	1.8	2.1	2.6
K0720	BK0720	3.4	1.9	83.0	84.0	50.0	55.0	10%	1.0	1.5	1.7	2.1	1.0	1.5	1.8	2.2
K0900	BK0900	4.2	1.5	88.0	89.0	50.0	55.0	10%	1.3	1.9	2.3	2.8	1.3	2.0	2.3	2.9
BASIN L																
L0100	BL0100	1.9	5.0	85.0	89.0	40.0	55.0	38%	0.5	0.8	1.0	1.2	0.6	0.9	1.1	1.3
L0400	BL0400	3.9	17.2	77.0	87.0	15.0	55.0	267%	0.5	1.1	1.4	1.9	1.2	1.8	2.1	2.6
L0500	BL0500	4.2	15.9	78.0	84.0	35.0	55.0	57%	0.9	1.6	1.9	2.4	1.3	1.9	2.2	2.8
L0540	BL0540	2.2	11.5	83.0	86.0	45.0	55.0	22%	0.6	1.0	1.1	1.4	0.7	1.0	1.2	1.5
L0570	BL0570	4.1	3.3	80.0	82.0	50.0	55.0	10%	1.1	1.7	2.0	2.5	1.2	1.8	2.1	2.6
L0700	BL0700	2.7	22.6	87.0	88.0	50.0	55.0	10%	0.8	1.3	1.5	1.8	0.9	1.3	1.5	1.8
L1000	BL1000	2.8	25.4	79.0	83.0	45.0	55.0	22%	0.7	1.1	1.3	1.7	0.8	1.2	1.5	1.8
BASIN M																
M0300	BM0300	1.7	3.7	84.0	87.0	40.0	55.0	38%	0.5	0.7	0.9	1.1	0.5	0.8	0.9	1.1
M0610	BM0610	4.1	13.2	86.0	87.0	50.0	55.0	10%	1.2	1.9	2.2	2.7	1.3	1.9	2.3	2.8
M0800	BM0800	2.2	4.0	82.0	87.0	35.0	55.0	57%	0.5	0.8	1.0	1.3	0.7	1.0	1.2	1.5
M1000	BM1000	5.3	8.0	87.0	87.0	50.0	50.0	0%	1.6	2.4	2.9	3.5	1.6	2.4	2.9	3.5
BASIN N																
N0300	BN0300	4.7	12.7	91.0	91.0	65.0	65.0	0%	1.6	2.4	2.8	3.4	1.6	2.4	2.8	3.4
N0330	BN0330	6.0	3.8	85.0	86.0	50.0	55.0	10%	1.8	2.7	3.2	3.9	1.9	2.8	3.3	4.0
N0350	BN0350	3.4	3.9	87.0	87.0	55.0	55.0	0%	1.1	1.6	1.9	2.3	1.1	1.6	1.9	2.3
N0360	BN0360	5.5	3.0	83.0	85.0	50.0	55.0	10%	1.5	2.4	2.8	3.5	1.7	2.5	3.0	3.7
N0400	BN0400	3.4	9.0	89.0	90.0	50.0	55.0	10%	1.1	1.6	1.9	2.3	1.1	1.6	1.9	2.3
N0402	BN0402	2.7	6.4	89.0	90.0	50.0	55.0	10%	0.9	1.3	1.5	1.8	0.9	1.3	1.5	1.9
N0403	BN0403	2.4	1.2	90.0	90.0	55.0	55.0	0%	0.8	1.2	1.4	1.7	0.8	1.2	1.4	1.7
N0404	BN0404	1.3	7.0	89.0	90.0	50.0	55.0	10%	0.4	0.6	0.7	0.9	0.4	0.6	0.7	0.9
N0405	BN0405	5.1	7.7	89.0	90.0	50.0	55.0	10%	1.6	2.4	2.8	3.5	1.7	2.5	2.9	3.5
N0420	BN0420	5.2	7.4	90.0	90.0	55.0	55.0	0%	1.7	2.5	2.9	3.6	1.7	2.5	2.9	3.6
N0422	BN0422	4.4	6.5	90.0	90.0	55.0	55.0	0%	1.4	2.1	2.5	3.0	1.4	2.1	2.5	3.0
N0460	BN0460	3.4	5.9	90.0	90.0	55.0	55.0	0%	1.1	1.7	1.9	2.4	1.1	1.7	1.9	2.4
N0470	BN0470	4.5	2.0	90.0	90.0	55.0	55.0	0%	1.5	2.2	2.5	3.1	1.5	2.2	2.5	3.1
N0510	BN0510	7.1	2.6	85.0	85.0	55.0	55.0	0%	2.2	3.2	3.8	4.7	2.2	3.2	3.8	4.7
N0520	BN0520	4.0	19.7	88.0	88.0	60.0	60.0	0%	1.3	1.9	2.2	2.7	1.3	1.9	2.2	2.7
N0600	BN0600	6.3	11.0	90.0	90.0	65.0	65.0	0%	2.2	3.1	3.7	4.5	2.2	3.1	3.7	4.5
N1100	BN1100	9.8	0.8	84.0	87.0	40.0	55.0	38%	2.1	3.6	4.4	5.6	2.7	4.3	5.1	6.4
N1400	BN1400	5.2	1.0	76.0	87.0	10.0	55.0	450%	0.4	1.0	1.3	1.8	1.5	2.3	2.7	3.4
N1800	BN1800	8.4	7.0	85.0	86.0	45.0	51.0	13%	2.4	3.7	4.3	5.4	2.5	3.8	4.5	5.6
N99991	BN99991	4.0	6.8	78.0	84.0	35.0	55.0	57%	0.9	1.5	1.8	2.2	1.2	1.8	2.1	2.6

Table B-1. Hydrologic Input Data and Peak Flow Results																
				Curve Number		Impervious Area (%)			Existing Land Use Scenario Subbasin Peak Flow (cubic feet per second)				Future Land Use Scenario Subbasin Peak Flow (cubic feet per second)			
Subbasin	Inlet Node	Area (acre)	Average Slope (%)	Existing Land Use	Future Land Use	Existing Land Use	Future Land Use	Percent Increase	2yr 24hr Design Storm	10yr 24hr Design Storm	25yr 24hr Design Storm	100yr 24hr Design Storm	2yr 24hr Design Storm	10yr 24hr Design Storm	25yr 24hr Design Storm	100yr 24hr Design Storm
BASIN O																
O0400	BO0400	4.1	13.6	78.0	86.0	15.0	50.0	233%	0.7	1.3	1.6	2.1	1.2	1.9	2.2	2.7
O0700	BO0700	2.8	0.0	92.0	92.0	75.0	75.0	0%	1.0	1.4	1.7	2.0	1.0	1.4	1.7	2.0
O0712	BO0712	1.1	6.1	78.0	80.0	50.0	55.0	10%	0.3	0.5	0.6	0.7	0.3	0.5	0.6	0.7
O0715	BO0715	9.9	6.5	86.0	87.0	50.0	55.0	10%	2.9	4.4	5.2	6.5	3.1	4.6	5.4	6.7
O0750	BO0750	4.2	0.3	86.0	86.0	50.0	50.0	0%	1.2	1.9	2.2	2.7	1.2	1.9	2.2	2.7
O1100	BO1100	1.8	6.6	86.0	87.0	50.0	55.0	10%	0.5	0.8	1.0	1.2	0.6	0.8	1.0	1.2
O1300	BO1300	2.9	11.4	86.0	87.0	50.0	55.0	10%	0.9	1.3	1.6	1.9	0.9	1.4	1.6	2.0
O1800	BO1800	5.2	7.1	86.0	87.0	50.0	55.0	10%	1.5	2.3	2.8	3.4	1.6	2.4	2.8	3.5
O1810	BO1810	6.6	25.3	87.0	88.0	50.0	55.0	10%	2.0	3.1	3.6	4.4	2.1	3.1	3.7	4.5
O1922	BO1922	4.0	6.0	84.0	88.0	40.0	55.0	38%	1.0	1.7	2.0	2.5	1.3	1.9	2.2	2.7
O1932	BO1932	5.6	3.9	94.0	94.0	85.0	85.0	0%	2.1	3.0	3.4	4.1	2.1	3.0	3.4	4.1
O1933	BO1933	5.2	2.6	86.0	89.0	45.0	55.0	22%	1.5	2.3	2.7	3.3	1.7	2.5	2.9	3.6
O1940	BO1940	4.2	12.0	86.0	88.0	50.0	55.0	10%	1.3	1.9	2.3	2.8	1.3	2.0	2.3	2.9
O1970	BO1970	4.0	5.7	83.0	88.0	35.0	55.0	57%	0.9	1.6	1.9	2.4	1.2	1.9	2.2	2.7
O2000	BO2000	3.6	23.9	86.0	89.0	40.0	55.0	38%	1.1	1.6	1.9	2.4	1.2	1.8	2.1	2.5
O2150	BO2150	1.8	6.7	89.0	90.0	50.0	55.0	10%	0.6	0.9	1.0	1.3	0.6	0.9	1.0	1.3
O2170	BO2170	3.5	0.5	87.0	90.0	40.0	55.0	38%	0.9	1.5	1.7	2.2	1.1	1.7	1.9	2.4
O2300	BO2300	2.0	37.2	86.0	89.0	40.0	55.0	38%	0.6	0.9	1.0	1.3	0.6	1.0	1.1	1.4
BASIN P																
P0110	BP0110	2.3	1.0	86.0	87.0	50.0	55.0	10%	0.6	1.0	1.2	1.5	0.7	1.0	1.2	1.5
P0500	BP0500	2.5	21.4	83.0	87.0	35.0	55.0	57%	0.6	1.0	1.2	1.5	0.8	1.2	1.4	1.7
P0600	BP0600	2.9	19.4	82.0	87.0	35.0	55.0	57%	0.7	1.2	1.4	1.7	0.9	1.4	1.6	1.9
BASIN Q																
Q0200	BQ0200	3.0	18.4	86.0	89.0	40.0	55.0	38%	0.9	1.3	1.6	2.0	1.0	1.5	1.7	2.1
Q0400	BQ0400	5.6	6.5	90.0	90.0	55.0	55.0	0%	1.8	2.7	3.1	3.9	1.8	2.7	3.1	3.9
BASIN R																
R0120	BR0120	1.9	0.4	86.0	87.0	50.0	55.0	10%	0.5	0.9	1.0	1.3	0.6	0.9	1.0	1.3
R0200	BR0200	3.5	5.0	86.0	87.0	50.0	55.0	10%	1.1	1.6	1.9	2.3	1.1	1.7	1.9	2.4
BASIN S																
S0500	BS0500	2.7	6.2	86.0	87.0	50.0	55.0	10%	0.8	1.2	1.5	1.8	0.9	1.3	1.5	1.8
S0530	BS0530	2.9	5.5	81.0	83.0	50.0	55.0	10%	0.8	1.2	1.5	1.8	0.9	1.3	1.5	1.9
S0900	BS0900	3.1	12.4	84.0	86.0	50.0	55.0	10%	0.9	1.4	1.6	2.0	1.0	1.4	1.7	2.1
S1100	BS1100	3.5	10.7	87.0	88.0	50.0	55.0	10%	1.1	1.6	1.9	2.3	1.1	1.7	1.9	2.4
BASIN T																
T0100	BT0100	3.2	3.5	80.0	85.0	35.0	55.0	57%	0.7	1.2	1.4	1.8	1.0	1.5	1.7	2.1
T0500	BT0500	2.7	0.9	77.0	82.0	40.0	55.0	38%	0.5	0.9	1.2	1.5	0.7	1.2	1.4	1.7
T0900	BT0900	2.8	1.5	86.0	89.0	45.0	55.0	22%	0.7	1.2	1.4	1.8	0.9	1.3	1.5	1.9
BASIN U																
U0300	BU0300	3.8	5.8	82.0	83.0	50.0	55.0	10%	1.1	1.6	1.9	2.4	1.1	1.7	2.0	2.5
BASIN V																
V0102	BV0102	1.3	1.0	80.0	81.0	50.0	55.0	10%	0.4	0.5	0.6	0.8	0.4	0.6	0.7	0.8
V0110	BV0110	1.7	15.5	73.0	84.0	20.0	55.0	175%	0.2	0.5	0.6	0.8	0.5	0.8	0.9	1.1
V0210	BV0210	0.6	15.8	68.0	77.0	20.0	43.0	115%	0.1	0.2	0.2	0.3	0.2	0.2	0.3	0.3
V0400	BV0400	0.2	3.5	65.0	71.0	10.0	28.0	180%	0.0	0.1	0.1	0.1	0.0	0.1	0.1	0.1
V0520	BV0520	1.4	7.6	82.0	84.0	50.0	55.0	10%	0.4	0.6	0.7	0.9	0.4	0.7	0.8	0.9
V0530	BV0530	2.2	10.7	86.0	87.0	50.0	55.0	10%	0.7	1.0	1.2	1.4	0.7	1.0	1.2	1.5
V0600	BV0600	1.3	0.3	76.0	81.0	40.0	55.0	38%	0.2	0.4	0.5	0.7	0.3	0.5	0.6	0.8
V0700	BV0700	0.6	3.3	72.0	81.0	30.0	55.0	83%	0.1	0.2	0.2	0.3	0.2	0.3	0.3	0.4
V0710	BV0710	1.1	7.3	80.0	82.0	50.0	55.0	10%	0.3	0.5	0.6	0.7	0.3	0.5	0.6	0.7
V0720	BV0720	4.1	6.9	82.0	84.0	50.0	55.0	10%	1.2	1.8	2.1	2.6	1.2	1.9	2.2	2.7
V1000	BV1000	3.9	12.5	80.0	81.0	50.0	55.0	10%	1.1	1.7	2.0	2.5	1.2	1.7	2.0	2.5
V1100	BV1100	3.8	0.7	80.0	81.0	50.0	55.0	10%	0.9	1.5	1.8	2.3	1.0	1.6	1.9	2.4
V1200	BV1200	3.9	1.1	80.0	81.0	50.0	55.0	10%	1.0	1.6	1.9	2.4	1.1	1.7	2.0	2.5
V1600	BV1600	2.6	5.4	84.0	86.0	50.0	55.0	10%	0.8	1.1	1.3	1.7	0.8	1.2	1.4	1.7
V5000	BV5000	0.9	2.1	61.0	81.0	0.0	53.0		0.0	0.1	0.2	0.3	0.3	0.4	0.5	0.6
V5200	BV5200	0.4	1.1	80.0	81.0	50.0	55.0	10%	0.1	0.2	0.2	0.2	0.1	0.2	0.2	0.2
V5300	BV5300	3.4	1.7	80.0	81.0	50.0	55.0	10%	0.9	1.4	1.7	2.1	0.9	1.5	1.7	2.2
BASIN X																
X0100	BX0100	4.6	7.3	82.0	89.0	30.0	60.0	100%	1.0	1.8	2.1	2.7	1.5	2.2	2.6	3.2
BASIN Z																
Z0300	BZ0300	0.4	0.0	77.0	77.0	65.0	65.0	0%	0.1	0.2	0.2	0.3	0.1	0.2	0.2	0.3
Z0400	BZ0400	1.1	0.7	79.0	79.0	65.0	65.0	0%	0.3	0.5	0.6	0.7	0.3	0.5	0.6	0.7
Z0500	BZ0500	1.0	1.7	83.0	83.0	65.0	65.0	0%	0.3	0.5	0.5	0.7	0.3	0.5	0.5	0.7
Z0600	BZ0600	2.9	0.0	83.0	86.0	65.0	77.0	18%	0.8	1.2	1.4	1.8	0.9	1.4	1.6	2.0
Z0710	BZ0710	1.7	1.0	85.0	85.0	65.0	65.0	0%	0.6	0.8	1.0	1.2	0.6	0.8	1.0	1.2
Z0830	BZ0830	0.3	4.0	96.0	96.0	90.0	90.0	0%	0.1	0.2	0.2	0.2	0.1	0.2	0.2	0.2
Z2010	BZ2010	4.0	3.0	67.0	67.0	15.0	15.0	0%	0.3	0.8	1.1	1.5	0.3	0.8	1.1	1.5

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
BASIN A																		
Outfall	A0101					27.47		28.5		28.5		28.5		8.9	10.2	9.5	10.8	
A0102	A0101	174	12'' DIA	4	4.5%	35.35	27.47	37.9	28.5	65.5	28.5	70.8	28.5	9.6	11.1	10.3	11.9	Existing 10-yr
A0103	A0102	231	12'' DIA	8	5.1%	47.16	35.35	49.1	37.9	68.4	65.5	74.0	70.8	4.4	5.2	4.6	5.4	Existing 10-yr
A0104	A0103	43	12'' DIA	10	8.4%	50.82	47.21	51.8	49.1	69.0	68.4	74.6	74.0	5.1	6.2	5.4	6.5	Existing 10-yr
Outfall	A0110					27.99		29.5		29.5		29.5		9.8	10.0	9.8	10.0	
A0120	A0110	313	18'' DIA	5	0.8%	30.57	27.99	33.3	29.5	38.7	29.5	38.7	29.5	9.8	10.0	9.8	10.0	Existing 10-yr
A0130	A0120	90	18'' DIA	8	2.0%	32.32	30.57	33.8	33.3	41.3	38.7	41.3	38.7	10.2	10.2	10.2	10.2	Existing 10-yr
A0140	A0130	340	1.5' H, 2'BW, 2 SS Channel	10	0.2%	32.87	32.32	34.4	33.8	43.7	41.3	43.8	41.3	13.3	14.3	13.3	14.3	Existing 10-yr
A0150	A0140	402	18'' DIA	8	0.6%	35.13	32.87	39.7	34.4	47.7	43.7	47.7	43.8	14.6	15.7	14.6	15.7	Existing 10-yr
A0170	A0150	1063	18'' DIA	12	1.4%	49.81	35.43	53.1	39.7	56.5	47.7	56.6	47.7	12.6	12.7	12.0	13.0	Existing 10-yr
A0180	A0170	139	18'' DIA	5	0.3%	50.23	49.86	53.6	53.1	56.8	56.5	56.9	56.6	7.2	7.9	6.6	7.8	Existing 10-yr
A0191	A0190	309	18'' DIA	4	0.1%	50.61	50.23	56.7	53.6	56.9	56.8	57.1	56.9	2.8	3.2	2.8	3.2	Existing 10-yr
Outfall	A0100					45.00		50.0		46.2		46.2		87.4	93.0	90.4	94.7	
A0200	A0100	1098	See cross-section	1248	0.5%	50.16	45.00	55.2	50.0	52.7	46.2	52.7	46.2	87.9	93.5	91.0	94.9	--
A0300	A0200	40	48'' DIA	51	0.4%	50.33	50.16	58.6	55.2	53.3	52.7	53.4	52.7	85.1	90.2	87.9	91.2	--
A0310	A0300	469	12'' DIA	9	6.8%	82.00	50.33	85.0	58.6	82.5	53.3	82.5	53.4	5.1	5.9	5.2	6.1	--
A0320	A0310	252	12'' DIA	4	1.4%	85.50	82.00	88.0	85.0	88.8	82.5	89.2	82.5	5.1	6.0	5.3	6.2	Existing 10-yr
A0330	A0320	129	12'' DIA	3	0.5%	86.20	85.50	89.6	88.0	91.4	88.8	92.0	89.2	5.2	6.1	5.4	6.2	Existing 10-yr
A0400	A0300	9	48'' DIA	51	0.4%	50.37	50.33	58.6	58.6	53.5	53.3	53.5	53.4	80.3	84.3	82.7	85.2	--
A0410	A0400	100	12'' DIA	4	1.3%	51.70	50.37	56.7	58.6	55.9	53.5	56.7	53.5	4.3	4.8	4.3	4.8	--
A0420	A0410	730	12'' DIA	2	0.3%	54.00	51.70	57.0	56.7	57.0	55.9	57.0	56.7	1.7	2.0	1.7	2.0	Existing 10-yr
A0500	A0400	39	48'' DIA	51	0.4%	50.54	50.37	58.3	58.6	53.9	53.5	54.0	53.5	77.2	79.6	78.7	80.5	--
A0600	A0500	26	48'' DIA	53	0.5%	50.66	50.54	58.7	58.3	54.2	53.9	54.3	54.0	74.8	76.1	75.6	76.6	--
A0700	A0600	153	48'' DIA	41	0.3%	51.09	50.66	60.4	58.7	55.5	54.2	55.6	54.3	74.8	76.1	75.6	76.6	--
A0900	A0700	207	48'' DIA	45	0.3%	51.77	51.09	59.7	60.4	57.4	55.5	57.6	55.6	74.8	76.1	75.6	76.6	--
A1000	A0900	123	48'' DIA	45	0.3%	52.18	51.77	60.4	59.7	58.6	57.4	58.7	57.6	74.9	76.1	75.6	76.6	--
A1100	A1000	34	48'' DIA	110	2.0%	52.85	52.18	61.4	60.4	58.9	58.6	59.0	58.7	71.6	71.6	71.7	71.7	--
A1200	A1100	83	4',4', Box	146	0.6%	53.38	52.85	60.0	61.4	59.0	58.9	59.1	59.0	71.9	71.9	71.9	71.9	--
A1300	A1200	17	51.96'' DIA	141	0.6%	53.49	53.38	60.0	60.0	59.0	59.0	59.2	59.1	71.6	71.6	71.7	71.7	--
A1500	A1300	38	48'' DIA	55	0.5%	53.68	53.49	60.4	60.0	59.3	59.0	59.4	59.2	70.7	70.7	70.8	70.8	--
A1600	A1500	65	48'' DIA	54	0.5%	53.99	53.68	60.9	60.4	59.8	59.3	59.9	59.4	70.7	70.7	70.8	70.8	--
A1700	A1600	240	48'' DIA	54	0.5%	55.14	53.99	59.1	60.9	61.4	59.8	61.4	59.9	111.2	103.9	105.3	108.7	Existing 10-yr
A1800	A1700	97	See cross-section	429	-0.2%	54.97	55.14	60.0	59.1	61.4	61.4	61.4	61.4	122.7	119.0	119.5	120.7	Existing 10-yr
A1820	A1800	424	12'' DIA	2	0.4%	56.87	54.97	58.7	60.0	66.9	61.4	69.0	61.4	4.5	5.3	5.0	5.9	Existing 10-yr
A1900	A1800	41	60'' DIA	234	2.7%	56.09	54.97	61.1	60.0	61.6	61.4	61.6	61.4	99.1	99.1	96.8	104.3	Existing 10-yr
A2000	A1900	873	See cross-section	907	0.9%	63.80	56.09	66.8	61.1	65.9	61.6	65.9	61.6	90.8	99.2	97.0	104.4	--
A2100	A2000	512	See cross-section	96	0.6%	67.04	63.80	70.0	66.8	69.5	65.9	69.5	65.9	97.4	101.8	100.4	106.7	--
A2200	A2100	23	36'' DIA	50	0.6%	67.17	67.04	72.8	70.0	72.8	69.5	72.8	69.5	88.8	96.9	94.9	103.1	--
A2210	A2200	136	12'' DIA	2	0.2%	67.73	67.42	71.3	72.8	71.3	72.8	71.3	72.8	2.2	2.6	2.6	3.0	Future 10-yr
A2300	A2200	119	36'' DIA	50	0.6%	67.98	67.32	72.9	72.8	72.4	72.8	72.6	72.8	87.1	95.1	93.1	99.7	--
A2400	A2300	94	48'' DIA	58	0.6%	68.50	67.98	72.5	72.9	73.3	72.4	73.8	72.6	87.1	95.0	93.0	99.7	Existing 10-yr
A2500	A2400	60	See cross-section	2178	0.9%	69.04	68.50	73.0	72.5	73.3	73.3	73.8	73.8	87.2	95.0	93.1	99.7	Existing 10-yr
A2600	A2500	40	48'' DIA	56	0.5%	69.25	69.04	77.3	73.0	73.8	73.3	74.4	73.8	87.2	95.0	93.3	99.7	--

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
A2700	A2600	27	4',5', Box	194	0.6%	69.42	69.25	77.2	77.3	73.9	73.8	74.4	74.4	87.2	95.0	93.1	99.6	--
A2710	A2700	93	12'' DIA	3	0.8%	72.73	71.94	75.4	77.2	73.9	73.9	74.5	74.4	1.2	1.4	1.3	1.5	--
A2720	A2710	68	12'' DIA	3	0.8%	73.40	72.83	76.2	75.4	73.9	73.9	74.5	74.5	1.2	1.4	1.3	1.5	--
A2800	A2700	35	4',5', Box	196	0.6%	69.64	69.42	75.2	77.2	73.9	73.9	74.5	74.4	86.3	93.9	92.1	98.6	--
A2900	A2800	13	36'' DIA	36	0.3%	69.68	69.64	75.2	75.2	74.2	73.9	74.7	74.5	86.3	93.9	92.1	98.6	--
A2920	A2900	231	24'' DIA	9	0.1%	70.51	70.18	76.6	75.2	78.4	74.2	79.0	74.7	30.8	32.0	31.2	32.3	Existing 10-yr
A2930	A2920	231	24'' DIA	11	0.2%	71.06	70.51	76.2	76.6	82.0	78.4	82.6	79.0	28.8	29.5	29.0	29.8	Existing 10-yr
A2940	A2930	102	24'' DIA	7	0.1%	71.12	71.01	76.1	76.2	83.6	82.0	84.2	82.6	28.7	29.5	28.9	29.7	Existing 10-yr
A2950	A2940	105	24'' DIA	11	0.2%	71.37	71.12	75.2	76.1	85.2	83.6	85.9	84.2	28.6	29.4	28.7	29.5	Existing 10-yr
A2960	A2950	212	18'' DIA	3	0.1%	71.52	71.32	73.9	75.2	100.7	85.2	101.5	85.9	28.5	29.3	28.6	29.4	Existing 10-yr
A2965	A2960	330	18'' DIA	4	0.2%	72.11	71.52	77.7	73.9	125.0	100.7	126.0	101.5	28.8	29.7	29.0	29.9	Existing 10-yr
A2966	A2965	396	12'' DIA	6	2.3%	84.29	75.11	88.1	77.7	128.9	125.0	130.2	126.0	4.5	5.2	4.6	5.4	Existing 10-yr
A2967	A2966	32	12'' DIA	9	5.6%	86.08	84.29	88.1	88.1	129.2	128.9	130.6	130.2	4.9	5.7	5.0	5.8	Existing 10-yr
A2968	A2967	34	12'' DIA	2	0.4%	86.31	86.18	88.4	88.1	129.3	129.2	130.6	130.6	1.7	2.0	1.8	2.1	Existing 10-yr
A2969	A2968	214	12'' DIA	7	3.2%	93.25	86.46	96.1	88.4	129.6	129.3	130.9	130.6	2.2	2.6	2.3	2.7	Existing 10-yr
A2970	A2965	411	18'' DIA	2	0.0%	72.24	72.11	77.0	77.7	146.3	125.0	147.4	126.0	24.3	24.5	24.3	24.5	Existing 10-yr
A2975	A2970	74	18'' DIA	6	-0.3%	72.02	72.24	78.2	77.0	150.2	146.3	151.2	147.4	24.2	24.3	24.2	24.4	Existing 10-yr
A2980	A2975	216	18'' DIA	9	-0.7%	70.60	72.02	79.0	78.2	161.5	150.2	162.6	151.2	24.6	24.8	24.6	24.9	Existing 10-yr
A2986	A2980	252	12'' DIA	2	0.4%	71.95	71.00	77.6	79.0	161.7	161.5	162.8	162.6	6.3	6.3	6.3	6.3	Existing 10-yr
A2987	A2986	116	12'' DIA	3	0.6%	72.85	72.11	75.1	77.6	161.8	161.7	162.9	162.8	5.7	5.7	5.7	5.7	Existing 10-yr
A3000	A2900	128	36'' DIA	36	0.3%	70.05	69.68	74.9	75.2	75.0	74.2	75.8	74.7	55.9	62.3	61.2	66.6	Existing 10-yr
A3010	A3000	49	48'' DIA	72	-0.9%	69.62	70.05	74.7	74.9	75.1	75.0	75.9	75.8	37.0	42.0	40.9	44.7	Existing 10-yr
A3020	A3010	30	48'' DIA	29	0.1%	69.66	69.62	74.8	74.7	75.2	75.1	76.0	75.9	37.0	42.2	41.0	44.8	Existing 10-yr
A3030	A3020	310	48'' DIA	37	0.2%	70.47	69.76	75.0	74.8	75.8	75.2	76.8	76.0	35.4	40.3	39.2	42.9	Existing 10-yr
A3050	A3030	46	48'' DIA	34	0.2%	70.56	70.47	75.1	75.0	75.9	75.8	76.9	76.8	35.6	40.5	39.4	43.0	Existing 10-yr
A3100	A3000	325	18'' DIA	4	-0.2%	69.56	70.05	74.6	74.9	86.9	75.0	89.0	75.8	20.7	22.4	21.7	23.2	Existing 10-yr
A3200	A3100	378	18'' DIA	0	0.0%	69.56	69.56	75.1	74.6	99.8	86.9	102.6	89.0	19.5	21.0	20.3	21.7	Existing 10-yr
A3300	A3200	299	18'' DIA	7	0.4%	70.78	69.56	75.3	75.1	108.1	99.8	111.5	102.6	17.8	19.2	18.6	19.9	Existing 10-yr
A3400	A3300	162	18'' DIA	2	0.1%	70.86	70.78	75.6	75.3	112.7	108.1	116.5	111.5	17.7	19.1	18.5	19.8	Existing 10-yr
A3410	A3400	154	18'' DIA	2	0.1%	70.94	70.86	75.4	75.6	112.9	112.7	116.7	116.5	6.1	5.1	6.1	5.5	Existing 10-yr
A34102	A3410	39	15'' DIA	11	2.9%	72.32	71.19	74.8	75.4	112.9	112.9	116.7	116.7	1.6	2.2	2.6	4.3	Existing 10-yr
A3420	A3410	210	18'' DIA	9	0.7%	72.43	70.94	75.7	75.4	113.0	112.9	116.8	116.7	2.8	3.0	2.9	3.0	Existing 10-yr
A3430	A3420	236	12'' DIA	8	4.8%	83.82	72.43	85.0	75.7	114.2	113.0	118.1	116.8	3.1	2.8	2.7	3.0	Existing 10-yr
A3440	A3430	75	12'' DIA	6	3.1%	86.11	83.82	87.4	85.0	114.5	114.2	118.4	118.1	3.4	2.8	2.9	3.0	Existing 10-yr
A3450	A3440	344	12'' DIA	5	1.7%	92.07	86.11	94.1	87.4	115.8	114.5	119.8	118.4	4.1	4.2	4.2	4.2	Existing 10-yr
A3470	A3450	219	12'' DIA	4	1.3%	95.10	92.17	97.0	94.1	116.1	115.8	120.1	119.8	2.3	2.2	2.3	2.3	Existing 10-yr
A3480	A3470	57	6'' DIA	0	0.7%	95.59	95.20	96.9	97.0	117.1	116.1	121.1	120.1	2.0	2.2	2.1	2.2	Existing 10-yr
A3490	A3480	198	12'' DIA	5	2.0%	99.21	95.19	101.3	96.9	117.2	117.1	121.2	121.1	1.9	2.2	2.0	2.4	Existing 10-yr
A3500	A3400	117	24'' DIA	24	1.1%	72.18	70.86	75.4	75.6	113.1	112.7	117.0	116.5	19.9	16.9	17.4	15.9	Existing 10-yr
A3600	A3500	646	24'' DIA	38	2.8%	90.44	72.18	94.8	75.4	115.1	113.1	119.1	117.0	20.1	17.2	18.3	17.3	Existing 10-yr
A3610	A3600	21	12'' DIA	13	11.8%	92.94	90.44	94.8	94.8	115.1	115.1	119.1	119.1	2.3	2.2	2.2	1.3	Existing 10-yr
A3620	A3610	56	12'' DIA	2	0.3%	93.23	93.04	95.1	94.8	115.1	115.1	119.2	119.1	2.2	2.1	2.3	1.6	Existing 10-yr
A3640	A3630	35	8.04'' DIA	0	0.1%	93.38	93.33	95.7	95.1	115.3	115.1	119.3	119.2	2.7	3.2	2.9	3.4	Existing 10-yr
A3700	A3600	10	24'' DIA	58	6.6%	91.09	90.44	95.1	94.8	115.1	115.1	119.2	119.1	18.5	16.6	16.7	17.0	Existing 10-yr

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10-yr max water surface elevation (ft)		Future 10-yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10-yr	Existing 25-yr	Future 10-yr	Future 25-yr	
A3710	A3700	102	12'' DIA	5	1.7%	93.09	91.39	95.6	95.1	115.2	115.1	119.2	119.2	2.7	2.6	2.4	3.0	Existing 10-yr
A3720	A3710	18	12'' DIA	9	5.4%	94.05	93.09	96.0	95.6	115.2	115.2	119.2	119.2	2.71*	2.05*	2.35*	2.33*	Existing 10-yr
A3720A	A3640	29	6'' DIA	1	1.8%	94.30	93.78	96.0	95.7	115.2	115.3	119.2	119.3	0.92*	1.13*	1.03*	1.24*	Existing 10-yr
A3730	A3720	60	12'' DIA	6	2.5%	95.73	94.25	97.5	96.0	115.2	115.2	119.3	119.2	2.0	2.6	2.4	3.0	Existing 10-yr
A3740	A3730	78	12'' DIA	7	3.8%	98.78	95.83	100.3	97.5	115.2	115.2	119.3	119.3	1.6	2.0	1.9	2.3	Existing 10-yr
A3741	A3740	13	12'' DIA	4	13.5%	100.52	98.78	101.5	100.3	115.3	115.2	119.3	119.3	2.4	2.9	2.7	3.1	Existing 10-yr
A3742	A3741	224	1' H, 1'BW, 2 SS Channel	23	9.2%	121.10	100.52	122.1	101.5	121.4	115.3	121.4	119.3	1.6	1.9	1.7	2.0	--
A3743	A3742	142	12'' DIA	2	0.5%	121.76	121.10	123.5	122.1	122.4	121.4	122.4	121.4	1.6	1.9	1.7	2.0	--
A3744	A3743	127	12'' DIA	1	0.2%	122.05	121.86	123.6	123.5	122.9	122.4	122.9	122.4	1.6	1.9	1.7	2.0	--
A3800	A3700	9	24'' DIA	61	7.5%	92.02	91.35	95.7	95.1	115.1	115.1	119.2	119.2	16.9	17.5	17.0	17.9	Existing 10-yr
A3900	A3800	222	24'' DIA	27	1.4%	95.20	92.02	98.2	95.7	115.6	115.1	119.7	119.2	16.8	17.7	17.3	18.3	Existing 10-yr
A3910	A3900	55	12'' DIA	4	1.5%	96.03	95.20	97.7	98.2	115.7	115.6	119.8	119.7	2.3	2.7	2.7	3.2	Existing 10-yr
A4100	A3900	251	18'' DIA	10	0.8%	97.22	95.20	100.3	98.2	117.8	115.6	122.1	119.7	16.9	17.6	17.2	17.7	Existing 10-yr
A4110	A4100	43	15'' DIA	3	0.3%	97.34	97.22	100.0	100.3	117.8	117.8	122.1	122.1	2.7	2.6	2.4	2.3	Existing 10-yr
A4120	A4110	73	15'' DIA	10	2.2%	99.03	97.44	100.7	100.0	117.8	117.8	122.1	122.1	1.4	1.4	1.5	1.9	Existing 10-yr
A4300	A4100	338	18'' DIA	22	4.5%	112.58	97.22	115.7	100.3	121.6	117.8	126.2	122.1	15.9	17.5	16.8	17.8	Existing 10-yr
A4400	A4300	97	18'' DIA	33	10.1%	122.53	112.73	124.6	115.7	123.5	121.6	127.7	126.2	15.9	17.5	16.6	18.0	Existing 10-yr
A4500	A4400	171	18'' DIA	20	3.5%	128.65	122.63	131.0	124.6	129.5	123.5	129.8	127.7	13.1	14.3	13.5	14.9	Future 10-yr
A4600	A4500	42	18'' DIA	17	2.7%	130.09	128.95	132.9	131.0	131.1	129.5	131.1	129.8	13.1	14.3	13.5	14.6	--
A4700	A4600	250	18'' DIA	29	7.8%	149.49	130.19	151.9	132.9	150.2	131.1	150.2	131.1	13.1	14.3	13.5	14.6	--
A4900	A4700	202	18'' DIA	26	6.0%	161.67	149.59	165.2	151.9	162.4	150.2	162.4	150.2	11.7	12.5	11.9	12.8	--
A5000	A4900	67	18'' DIA	19	3.1%	163.80	161.72	168.7	165.2	164.7	162.4	164.7	162.4	11.7	12.5	11.9	12.8	--
A5010	A5000	357	12'' DIA	2	0.5%	165.36	163.63	168.8	168.7	168.8	164.7	168.8	164.7	2.7	3.1	2.8	3.2	--
A5020	A5010	226	12'' DIA	8	4.5%	175.55	165.36	180.5	168.8	176.0	168.8	176.0	168.8	2.7	3.1	2.8	3.2	--
A5100	A5000	51	12'' DIA	7	13.1%	170.68	164.05	172.7	168.7	172.0	164.7	172.1	164.7	8.5	10.2	10.0	11.7	--
A5100A	F1800	125	18'' DIA	28	7.1%	169.58	160.71	172.7	163.1	169.8	160.1	169.9	160.3	1.4	3.0	2.8	4.4	--
A1902	A1901	65	24'' DIA	51	17.7%	77.82	66.50	80.8	68.5	78.2	66.9	78.2	66.9	4.6	5.4	4.7	5.5	--
A1920	A1901	126	24'' DIA	38	2.6%	69.79	66.50	75.2	68.5	70.8	66.9	70.9	66.9	15.7	18.6	18.8	21.3	--
Wetland_In	Olson_Wetland	100	See cross-section	2021	7.6%	66.50	58.90	68.5	61.4	66.9	61.4	66.9	61.4	20.2	23.9	23.5	26.8	--
A1930	A1920	106	24'' DIA	71	9.6%	80.06	69.99	88.3	75.2	80.7	70.8	80.8	70.9	15.7	18.6	18.8	21.3	--
A1940	A1930	94	24'' DIA	25	1.1%	81.32	80.26	87.3	88.3	82.4	80.7	82.5	80.8	14.2	16.6	15.8	18.2	--
A1950	A1940	37	24'' DIA	11	0.2%	81.60	81.52	87.8	87.3	83.1	82.4	83.2	82.5	14.2	16.6	15.8	19.8	--
A1951	A1950	25	18'' DIA	14	1.7%	82.11	81.70	87.8	87.8	83.1	83.1	83.2	83.2	2.7	3.2	2.8	3.3	--
A1952	A1951	134	18'' DIA	7	0.4%	82.70	82.16	87.9	87.8	83.4	83.1	83.4	83.2	2.7	3.2	2.8	3.3	--
A1953	A1952	51	12'' DIA	3	0.5%	82.95	82.70	87.7	87.9	83.6	83.4	83.6	83.4	1.7	2.0	1.8	2.1	--
A1954	A1953	249	12'' DIA	2	0.2%	83.63	83.05	90.1	87.7	84.4	83.6	84.5	83.6	1.7	2.0	1.8	2.1	--
A1960	A1950	24	24'' DIA	30	1.7%	82.10	81.70	87.8	87.8	83.2	83.1	83.3	83.2	11.5	13.5	13.1	16.1	--
A1970	A1960	311	24'' DIA	5	0.0%	82.35	82.20	87.1	87.8	84.4	83.2	84.7	83.3	11.5	13.5	13.1	15.8	--
A1975	A1970	356	21'' DIA	6	0.1%	83.05	82.55	85.4	87.1	86.1	84.4	86.9	84.7	11.5	13.5	13.1	15.2	Existing 10-yr
A1980	A1975	71	18'' DIA	5	0.2%	83.22	83.05	85.4	85.4	86.7	86.1	87.7	86.9	10.0	11.7	11.5	13.3	Existing 10-yr
A1985	A1980	199	18'' DIA	2	0.0%	83.36	83.27	86.7	85.4	88.4	86.7	89.9	87.7	10.1	11.9	11.6	13.5	Existing 10-yr
A1987	A1985	53	12'' DIA	4	1.0%	84.04	83.51	86.0	86.7	88.6	88.4	90.2	89.9	2.6	3.2	3.2	3.7	Existing 10-yr
A1988	A1987	225	12'' DIA	7	3.6%	92.30	84.14	97.4	86.0	92.7	88.6	92.9	90.2	2.6	3.2	3.2	3.7	--
A1989	A1988	227	12'' DIA	3	0.7%	94.05	92.40	98.1	97.4	94.8	92.7	94.9	92.9	2.6	3.2	3.2	3.7	--

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom Width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
A1990	A1985	383	15'' DIA	4	0.4%	85.02	83.46	89.0	86.7	89.5	88.4	91.6	89.9	3.8	4.5	4.6	5.4	Existing 10-yr
A1992	A1990	70	12'' DIA	1	0.2%	85.24	85.12	89.0	89.0	90.3	89.5	92.7	91.6	3.8	4.6	4.7	5.5	Existing 10-yr
Outfall	A2993					72.40		73.9		73.4		73.4		6.2	7.6	7.2	8.6	
A2994	A2993	382	1' H, 1'BW, 2 SS Channel	6	0.6%	74.61	72.40	75.6	73.9	75.1	73.4	75.1	73.4	1.1	1.2	1.1	1.3	--
A2995	A2994	147	12'' DIA	3	0.5%	75.35	74.61	77.4	75.6	75.8	75.1	75.8	75.1	1.1	1.2	1.1	1.3	--
A2996	A2995	49	1' H, 1'BW, 2 SS Channel	9	1.3%	76.00	75.35	77.0	77.4	76.4	75.8	76.4	75.8	1.1	1.2	1.1	1.3	--
A2998	A2996	103	12'' DIA	5	2.1%	78.15	76.00	81.2	77.0	78.5	76.4	78.5	76.4	1.1	1.2	1.1	1.3	--
A2999	A2998	186	12'' DIA	8	4.4%	86.56	78.35	96.3	81.2	86.8	78.5	86.8	78.5	1.1	1.2	1.1	1.3	--
BASIN B																		
Outfall	B0100					5.98		10.5		10.5		10.5		43.4	51.5	48.1	56.4	
B0200	B0100	86	54'' DIA	434	5.0%	10.28	5.98	31.4	10.5	11.2	10.5	11.3	10.5	43.4	51.5	48.1	56.4	--
B0300	B0200	164	54'' DIA	140	0.5%	18.96	18.14	52.6	31.4	20.7	11.2	20.8	11.3	43.4	51.5	48.1	56.4	--
B0400	B0300	62	54'' DIA	87	0.2%	19.28	19.16	53.0	52.6	21.3	20.7	21.4	20.8	43.4	51.5	48.1	56.4	--
B0500	B0400	43	54'' DIA	434	4.5%	27.11	25.16	53.9	53.0	28.1	21.3	28.1	21.4	43.4	51.5	48.1	56.4	--
B0600	B0500	46	54'' DIA	482	6.0%	35.90	33.11	55.7	53.9	36.8	28.1	36.9	28.1	43.4	51.5	48.1	56.4	--
B0610	B0600	207	12'' DIA	3	0.9%	47.97	46.16	57.0	55.7	48.5	36.8	48.6	36.9	2.0	2.4	2.3	2.7	--
B0630	B0620	278	12'' DIA	3	0.6%	52.49	50.80	58.7	57.3	52.9	50.3	53.0	50.4	1.1	1.4	1.4	1.7	--
B0640	B0630	65	12'' DIA	3	0.7%	53.09	52.62	58.3	58.7	53.5	52.9	53.6	53.0	1.1	1.4	1.4	1.7	--
B0700	B0600	96	54'' DIA	28	0.0%	39.94	39.96	55.6	55.7	42.1	36.8	42.2	36.9	40.2	47.6	44.4	52.0	--
B0800	B0700	88	54'' DIA	236	1.4%	41.41	40.14	53.8	55.6	42.8	42.1	42.9	42.2	40.2	47.6	44.4	52.0	--
B0900	B0800	291	54'' DIA	61	0.1%	42.00	41.72	53.4	53.8	44.2	42.8	44.3	42.9	39.3	46.6	43.4	50.9	--
B0901	B0900	439	15'' DIA	6	0.8%	49.89	46.35	53.9	53.4	50.5	44.2	50.5	44.3	2.5	2.9	2.5	2.9	--
B0910	B0900	247	42'' DIA	35	0.1%	45.06	44.76	55.0	53.4	46.2	44.2	46.3	44.3	9.4	11.1	9.9	11.6	--
B0911	B0910	440	18'' DIA	8	0.6%	48.46	45.91	55.0	55.0	49.0	46.2	49.0	46.3	2.3	2.7	2.4	2.8	--
B0920	B0910	273	30'' DIA	19	0.2%	45.65	45.06	54.9	55.0	46.7	46.2	46.7	46.3	7.2	8.4	7.5	8.8	--
B0921	B0920	450	15'' DIA	4	0.4%	47.67	45.87	53.8	54.9	48.2	46.7	48.3	46.7	1.7	2.0	1.8	2.1	--
B0930	B0920	245	30'' DIA	18	0.2%	46.15	45.65	53.9	54.9	47.0	46.7	47.0	46.7	4.7	5.6	5.0	5.8	--
B0931	B0930	445	18'' DIA	6	0.3%	47.59	46.27	51.3	53.9	48.1	47.0	48.2	47.0	1.7	2.0	1.7	2.0	--
B0940	B0930	251	12'' DIA	3	0.9%	49.98	47.82	60.0	53.9	50.6	47.0	50.6	47.0	2.4	2.9	2.6	3.0	--
B1000	B0900	547	48'' DIA	80	0.3%	43.68	42.00	55.1	53.4	45.2	44.2	45.4	44.3	25.6	30.5	29.1	34.1	--
B1001	B1000	247	18'' DIA	9	0.6%	48.82	47.25	54.0	55.1	49.1	45.2	49.1	45.4	0.5	0.7	0.6	0.7	--
B1010	B1000	537	36'' DIA	9	0.0%	46.26	46.17	57.8	55.1	48.0	45.2	48.2	45.4	10.0	12.0	12.7	14.8	--
B1011	B1010	244	15'' DIA	7	1.2%	49.16	46.26	54.6	57.8	49.6	48.0	49.6	48.2	1.6	1.9	2.1	2.4	--
B1020	B1010	276	24'' DIA	14	0.4%	49.59	48.50	56.4	57.8	50.5	48.0	50.7	48.2	6.5	7.8	8.2	9.5	--
B1030	B1020	221	24'' DIA	19	0.7%	51.29	49.81	57.3	56.4	52.0	50.5	52.0	50.7	4.5	5.4	5.5	6.4	--
B1040	B1030	299	24'' DIA	11	0.3%	52.04	51.29	58.2	57.3	52.9	52.0	53.0	52.0	4.5	5.4	5.5	6.4	--
B1050	B1040	259	18'' DIA	9	0.7%	53.91	52.04	62.7	58.2	54.4	52.9	54.5	53.0	2.2	2.6	2.6	3.1	--
B1060	B1050	259	15'' DIA	4	0.4%	55.00	53.91	60.9	62.7	55.6	54.4	55.7	54.5	2.2	2.6	2.6	3.1	--
B1070	B1060	261	12'' DIA	8	4.9%	67.94	55.16	73.7	60.9	68.3	55.6	68.3	55.7	2.2	2.6	2.6	3.1	--
B1100	B1000	275	36'' DIA	69	1.1%	46.73	43.68	56.1	55.1	47.6	45.2	47.7	45.4	13.8	16.3	14.4	17.0	--
B1200	B1100	223	36'' DIA	43	0.4%	47.65	46.73	55.4	56.1	48.8	47.6	48.8	47.7	12.9	15.2	13.5	15.8	--
B1300	B1200	296	36'' DIA	36	0.3%	48.51	47.65	55.9	55.4	49.6	48.8	49.6	48.8	10.5	12.4	10.9	12.8	--
B1400	B1300	223	30'' DIA	53	1.6%	52.08	48.61	61.5	55.9	52.7	49.6	52.8	49.6	7.9	9.3	8.2	9.6	--
B1500	B1400	270	30'' DIA	18	0.2%	52.71	52.16	61.2	61.5	53.8	52.7	53.9	52.8	7.9	9.3	8.2	9.6	--

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
B1501	B1500	282	12'' DIA	4	1.2%	55.96	52.71	60.9	61.2	56.4	53.8	56.4	53.9	1.7	2.0	1.8	2.1	--
B1510	B1500	265	18'' DIA	8	0.6%	54.22	52.71	62.7	61.2	54.8	53.8	54.8	53.9	2.1	2.5	2.2	2.6	--
B1520	B1510	403	15'' DIA	4	0.4%	55.91	54.23	61.1	62.7	56.3	54.8	56.3	54.8	1.0	1.2	1.1	1.2	--
B1600	B1500	481	21'' DIA	8	0.2%	53.74	52.62	60.6	61.2	54.7	53.8	54.7	53.9	4.1	4.8	4.2	4.9	--
B1700	B1600	362	18'' DIA	8	0.6%	55.86	53.85	59.9	60.6	56.5	54.7	56.5	54.7	3.1	3.6	3.2	3.7	--
B6020	B0610	297	12'' DIA	3	0.5%	49.65	48.12	57.3	57.0	50.3	48.5	50.4	48.6	2.0	2.4	2.3	2.7	--
BASIN C																		
Outfall	C0100					14.18		16.2		16.2		16.2		24.8	27.7	27.4	30.5	
Overflow Outfall	Z0200					49.81		51.3		50.7		50.7		4.9	6.3	5.9	7.3	
C0200	C0100	101	24'' DIA	42	3.2%	17.38	14.18	19.4	16.2	18.5	16.2	18.6	16.2	24.8	27.7	27.4	30.5	--
C0300	C0200	223	24'' DIA	12	0.3%	18.05	17.38	20.1	19.4	21.7	18.5	22.4	18.6	24.8	27.8	27.4	30.5	Existing 10-yr
C0400	C0300	35	24'' DIA	124	29.3%	27.90	18.05	32.5	20.1	28.5	21.7	28.5	22.4	24.8	27.8	27.4	30.5	--
C0500	C0400	125	24'' DIA	99	17.2%	49.08	27.90	61.0	32.5	49.8	28.5	49.8	28.5	24.8	27.8	27.4	30.5	--
C0600	C0500	68	36'' DIA	48	0.5%	49.46	49.14	59.4	61.0	51.0	49.8	51.0	49.8	24.0	26.7	25.9	28.8	--
C0700	C0600	385	36'' DIA	44	0.4%	51.24	49.72	61.3	59.4	52.8	51.0	52.9	51.0	24.0	26.7	25.9	28.8	--
C0800	C0700	29	36'' DIA	61	-0.8%	51.02	51.24	61.9	61.3	53.0	52.8	53.1	52.9	28.8	33.0	31.8	36.1	--
C0800A	Z0200	92	18'' DIA	5	0.8%	50.52	49.81	61.9	51.3	51.7	50.7	51.9	50.7	4.9	6.3	5.9	7.3	--
C0810	C0800	279	18'' DIA	10	0.8%	52.67	50.52	58.1	61.9	53.0	53.0	53.1	53.1	0.9	1.0	0.9	1.1	--
C0900	C0800	17	36'' DIA	17	0.1%	51.37	51.36	62.0	61.9	53.1	53.0	53.2	53.1	28.0	32.0	30.9	35.0	--
C1000	C0900	233	36'' DIA	45	0.4%	52.36	51.37	58.5	62.0	54.1	53.1	54.2	53.2	28.0	32.0	30.9	35.0	--
C1100	C1000	283	30'' DIA	39	0.9%	54.83	52.42	60.3	58.5	56.3	54.1	56.5	54.2	26.7	30.5	29.5	33.4	--
C1300	C1100	262	30'' DIA	17	0.2%	55.24	54.83	60.2	60.3	57.5	56.3	57.9	56.5	25.4	28.9	28.0	31.6	--
C1301	C1300	232	15'' DIA	5	0.5%	56.31	55.26	59.8	60.2	59.8	57.5	59.9	57.9	5.4	5.9	6.3	7.0	Existing 10-yr
C1302	C1301	221	12'' DIA	3	0.5%	57.68	56.49	60.3	59.8	61.8	59.8	64.0	59.9	4.3	4.8	5.1	5.7	Existing 10-yr
C1303	C1302	231	12'' DIA	2	0.3%	58.45	57.68	62.1	60.3	64.2	61.8	67.1	64.0	3.8	4.4	4.6	5.0	Existing 10-yr
C1304	C1303	281	12'' DIA	3	0.6%	60.12	58.45	64.6	62.1	65.1	64.2	68.4	67.1	2.7	3.1	3.2	3.4	Existing 10-yr
C1305	C1304	242	12'' DIA	3	0.8%	61.95	60.12	67.1	64.6	67.1	65.1	69.6	68.4	2.3	2.7	2.8	3.3	Existing 10-yr
C1310	C1300	456	24'' DIA	14	0.3%	56.89	55.36	62.4	60.2	58.0	57.5	58.4	57.9	8.1	9.9	9.3	11.0	--
C1320	C1310	270	24'' DIA	16	0.5%	58.11	56.89	63.0	62.4	59.0	58.0	59.0	58.4	6.4	7.7	6.9	8.3	--
C1330	C1320	71	18'' DIA	14	1.7%	59.31	58.11	65.1	63.0	59.8	59.0	59.9	59.0	3.9	4.6	4.1	4.8	--
C1340	C1330	172	18'' DIA	13	1.4%	61.64	59.31	69.2	65.1	62.2	59.8	62.2	59.9	3.9	4.6	4.1	4.8	--
C1360	C1350	245	15'' DIA	10	2.2%	67.15	61.79	71.8	69.2	67.7	62.2	67.7	62.2	3.9	4.7	4.1	5.0	--
C1370	C1360	43	12'' DIA	4	1.3%	67.71	67.15	72.7	71.8	68.5	67.7	68.5	67.7	3.9	4.6	4.1	4.8	--
C1380	C1370	248	12'' DIA	4	1.4%	71.21	67.71	77.0	72.7	71.6	68.5	71.6	68.5	1.5	1.8	1.6	1.8	--
C1400	C1300	282	24'' DIA	20	0.7%	57.25	55.24	66.2	60.2	58.4	57.5	58.5	57.9	12.0	13.2	12.6	13.7	--
C1500	C1400	225	24'' DIA	16	0.5%	58.48	57.41	66.6	66.2	59.7	58.4	59.7	58.5	10.9	11.8	11.4	12.3	--
C1600	C1500	93	24'' DIA	19	0.7%	60.73	60.10	66.4	66.6	61.8	59.7	61.8	59.7	10.9	11.8	11.4	12.3	--
C1700	C1600	165	24'' DIA	20	0.7%	61.88	60.73	68.2	66.4	62.9	61.8	63.0	61.8	10.9	11.8	11.4	12.3	--
C1800	C1700	282	15'' DIA	5	0.5%	64.87	63.52	74.8	68.2	70.1	62.9	70.6	63.0	9.4	10.1	9.9	10.5	--
C1810	C1800	204	12'' DIA	5	1.5%	68.03	65.03	73.1	74.8	70.2	70.1	70.8	70.6	1.2	1.4	1.3	1.5	--
C1820	C1810	35	12'' DIA	5	1.7%	71.33	70.72	73.0	73.1	71.7	70.2	71.7	70.8	1.2	1.4	1.3	1.5	--
C1830	C1820	179	12'' DIA	3	0.9%	72.86	71.33	76.1	73.0	73.3	71.7	73.3	71.7	1.2	1.4	1.3	1.5	--
C1900	C1800	322	15'' DIA	5	0.6%	66.81	64.89	76.6	74.8	75.0	70.1	76.0	70.6	8.3	8.9	8.7	9.2	--
C2000	C1900	335	18'' DIA	8	0.5%	68.62	66.91	75.4	76.6	76.2	75.0	77.3	76.0	7.0	7.3	7.3	7.4	Existing 10-yr

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
C2010	C2000	12	12'' DIA	14	15.7%	70.51	68.62	75.4	75.4	76.2	76.2	77.3	77.3	2.0	2.3	2.0	2.1	Existing 10-yr
C2020	C2010	155	12'' DIA	3	0.8%	72.47	71.16	74.8	75.4	76.5	76.2	77.6	77.3	2.0	2.3	2.0	2.3	Existing 10-yr
C2030	C2020	106	12'' DIA	3	0.7%	73.20	72.47	76.7	74.8	76.7	76.5	77.8	77.6	2.0	2.4	2.0	2.4	Existing 10-yr
C2200	C2000	183	18'' DIA	6	0.3%	69.16	68.62	74.9	75.4	76.6	76.2	77.7	77.3	6.1	6.2	6.2	6.2	Existing 10-yr
C2300	C2200	101	18'' DIA	15	1.9%	71.22	69.26	74.8	74.9	76.8	76.6	77.9	77.7	13.2	14.6	14.5	15.6	Existing 10-yr
C2400	C2300	10	18'' DIA	15	2.0%	71.42	71.22	75.4	74.8	76.7	76.8	77.9	77.9	8.12*	9.56*	9.09*	10.15*	Existing 10-yr
C2400A	A3050	391	48'' DIA	36	0.2%	71.42	70.56	75.4	75.1	76.7	75.9	77.9	76.9	35.56*	40.45*	39.36*	43.02*	Existing 10-yr
C2500	C2400	9	27'' DIA	8	0.2%	71.47	71.45	75.5	75.4	77.0	76.7	78.2	77.9	27.6*	31.12*	30.47*	33.09*	Existing 10-yr
C2500A	C2300	13	18'' DIA	15	2.0%	71.47	71.22	75.5	74.8	77.0	76.8	78.2	77.9	13.16*	14.63*	14.46*	15.57*	Existing 10-yr
C2510	C2500	128	12'' DIA	4	1.1%	72.94	71.53	74.9	75.5	77.1	77.0	78.3	78.2	1.3	1.5	1.4	1.7	Existing 10-yr
BASIN D																		
Outfall	D0100					52.79		55.0		54.8		54.8		36.1	39.5	37.5	41.0	
D0200	D0100	33	27'' DIA	29	3.0%	53.77	52.79	61.4	55.0	56.3	54.8	56.5	54.8	36.1	39.5	37.5	41.0	--
D0210	D0200	227	21'' DIA	13	0.7%	55.55	53.94	62.4	61.4	56.4	56.3	56.5	56.5	3.6	4.3	3.8	4.4	--
D0220	D0210	264	21'' DIA	13	-0.7%	53.80	55.55	60.6	62.4	56.5	56.4	56.6	56.5	2.4	2.9	2.5	3.0	--
D0240	D0220	238	18'' DIA	13	1.7%	60.21	56.25	61.9	60.6	60.6	56.5	60.7	56.6	2.4	2.9	2.5	3.0	--
D0300	D0200	466	24'' DIA	24	1.1%	59.07	53.77	64.3	61.4	65.8	56.3	66.7	56.5	32.5	35.4	33.7	36.6	Existing 10-yr
D0310	D0300	597	24'' DIA	10	0.2%	60.58	59.30	63.9	64.3	67.3	65.8	68.2	66.7	12.0	13.5	12.4	13.8	Existing 10-yr
D0320	D0310	712	18'' DIA	5	0.3%	62.51	60.58	64.4	63.9	70.6	67.3	71.7	68.2	7.8	8.6	8.0	8.7	Existing 10-yr
D0330	D0320	117	18'' DIA	6	0.3%	62.84	62.51	66.3	64.4	71.1	70.6	72.2	71.7	7.5	8.3	7.7	8.4	Existing 10-yr
D0340	D0330	108	15'' DIA	15	5.7%	68.98	62.84	72.5	66.3	72.5	71.1	73.3	72.2	7.5	8.3	7.7	8.4	Existing 10-yr
D0350	D0340	460	15'' DIA	7	1.1%	74.23	69.19	78.1	72.5	78.1	72.5	78.1	73.3	6.4	7.5	6.6	7.8	Future 10-yr
D0360	D0350	256	12'' DIA	10	7.1%	92.75	74.55	97.3	78.1	93.3	78.1	93.3	78.1	5.3	6.3	5.5	6.5	--
D0370	D0360	268	12'' DIA	7	3.5%	102.21	92.75	107.6	97.3	102.8	93.3	102.8	93.3	4.0	4.7	4.2	4.9	--
D0400	D0300	464	24'' DIA	11	0.2%	60.51	59.38	66.3	64.3	69.2	65.8	70.3	66.7	23.4	24.6	25.0	24.7	Existing 10-yr
D0410	D0400	301	21'' DIA	7	0.2%	61.13	60.51	65.6	66.3	69.3	69.2	70.5	70.3	5.1	5.5	5.8	5.1	Existing 10-yr
D0420	D0410	214	18'' DIA	7	-0.4%	60.57	61.41	65.7	65.6	69.4	69.3	70.5	70.5	4.4	4.8	4.3	4.4	Existing 10-yr
D0440	D0430	263	18'' DIA	9	0.8%	63.26	61.20	67.9	65.7	69.5	69.4	70.6	70.5	2.7	2.9	2.3	2.7	Existing 10-yr
D0500	D0400	320	24'' DIA	11	0.2%	61.56	60.83	67.6	66.3	70.9	69.2	72.1	70.3	21.7	21.7	21.7	21.6	Existing 10-yr
D0600	D0500	138	24'' DIA	6	0.1%	61.70	61.59	66.2	67.6	71.6	70.9	72.9	72.1	16.6	17.7	17.2	18.4	Existing 10-yr
D0700	D0600	758	21'' DIA	11	0.5%	65.25	61.76	69.4	66.2	78.0	71.6	79.7	72.9	16.2	16.8	16.8	17.3	Existing 10-yr
D0800	D0700	378	24'' DIA	36	2.5%	74.91	65.30	77.4	69.4	79.1	78.0	80.8	79.7	12.5	13.2	12.6	13.4	Existing 10-yr
D0900	D0800	378	12'' DIA	6	2.5%	84.52	74.91	88.5	77.4	107.4	79.1	109.3	80.8	10.5	10.5	10.4	10.5	Existing 10-yr
D1000	D0900	189	12'' DIA	11	9.6%	102.46	84.53	106.1	88.5	119.1	107.4	121.1	109.3	10.1	9.8	10.2	9.7	Existing 10-yr
D1100	D1000	92	12'' DIA	6	3.0%	105.28	102.55	110.2	106.1	123.3	119.1	125.3	121.1	8.6	9.0	8.8	9.2	Existing 10-yr
D1120	D1100	706	12'' DIA	7	2.9%	126.00	105.28	128.0	110.2	126.3	123.3	126.3	125.3	0.9	1.1	1.0	1.1	--
D1200	D1100	321	12'' DIA	6	3.1%	115.25	105.39	119.3	110.2	133.7	123.3	135.9	125.3	7.4	7.8	7.6	8.1	Existing 10-yr
D1310	D1300	172	12'' DIA	8	4.8%	123.42	115.25	125.7	119.3	135.3	133.7	137.5	135.9	5.2	6.2	5.4	6.1	Existing 10-yr
D1320	D1310	56	12'' DIA	7	3.4%	125.33	123.45	127.9	125.7	135.5	135.3	137.7	137.5	3.7	4.0	3.7	4.2	Existing 10-yr
D1330	D1320	162	12'' DIA	7	3.3%	130.73	125.37	133.5	127.9	136.1	135.5	138.3	137.7	3.3	3.9	3.4	4.0	Existing 10-yr
D1340	D1330	297	12'' DIA	6	3.0%	139.68	130.73	141.7	133.5	140.2	136.1	140.2	138.3	3.3	3.9	3.5	4.1	--
D1500	D1300	20	8.04'' DIA	1	11.5%	117.54	115.25	118.3	119.3	142.1	133.7	144.7	135.9	3.3	3.5	3.5	3.6	Existing 10-yr
D1600	D1500	50	0.71' H, 1'BW, 3 SS Channel	10	5.4%	120.28	117.54	121.3	118.3	142.3	142.1	144.9	144.7	5.0	5.5	5.2	5.9	Existing 10-yr
D1700	D1600	198	12'' DIA	8	5.2%	130.54	120.28	134.3	121.3	143.3	142.3	146.0	144.9	5.2	6.1	5.6	6.5	Existing 10-yr

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10-yr max water surface elevation (ft)		Future 10-yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10-yr	Existing 25-yr	Future 10-yr	Future 25-yr	
D1800	D1700	505	12'' DIA	7	3.5%	150.03	132.34	153.0	134.3	150.5	143.3	150.6	146.0	3.3	3.9	3.6	4.2	--
D1900	D1800	131	12'' DIA	4	1.0%	151.39	150.03	154.4	153.0	151.9	150.5	151.9	150.6	2.0	2.3	2.0	2.4	--
D2100	D2000	160	12'' DIA	9	5.4%	160.24	151.59	166.1	154.4	160.6	151.9	160.6	151.9	2.0	2.3	2.0	2.4	--
D2200	D2100	184	12'' DIA	2	0.2%	160.87	160.44	163.7	166.1	161.7	160.6	161.8	160.6	2.0	2.3	2.0	2.4	--
BASIN E																		
Outfall	E0100					56.55		58.6		58.0		58.1		21.7	24.3	23.7	26.0	
E0200	E0100	145	24'' DIA	24	3.7%	61.97	56.55	65.8	58.6	63.4	58.0	63.6	58.1	20.9	23.5	22.9	25.1	--
E0210	E0200	93	15'' DIA	4	1.4%	63.63	62.32	66.4	65.8	64.2	63.4	64.2	63.6	1.8	2.0	1.8	2.0	--
E0300	E0200	390	24'' DIA	8	0.4%	63.59	62.12	68.1	65.8	69.5	63.4	71.2	63.6	15.1	16.5	17.3	18.5	Existing 10-yr
E0400	E0300	304	18'' DIA	6	0.4%	64.73	63.59	72.2	68.1	75.7	69.5	79.4	71.2	15.2	16.6	17.3	18.6	Existing 10-yr
E0500	E0400	228	18'' DIA	16	2.4%	70.22	64.73	75.8	72.2	79.5	75.7	84.6	79.4	13.7	14.8	15.9	17.0	Existing 10-yr
E0520	E0500	301	15'' DIA	5	0.7%	75.03	73.05	79.4	75.8	80.0	79.5	85.0	84.6	3.8	4.6	3.8	4.1	Existing 10-yr
E0521	E0520	30	12'' DIA	4	1.2%	75.67	75.31	79.0	79.4	80.1	80.0	85.0	85.0	2.4	2.9	2.6	2.9	Existing 10-yr
E0522	E0521	177	12'' DIA	6	2.8%	80.74	75.77	89.0	79.0	81.2	80.1	87.4	85.0	2.4	2.9	2.6	2.9	--
E0523	E0522	405	12'' DIA	3	0.8%	83.85	80.79	89.2	89.0	84.5	81.2	89.2	87.4	2.4	2.8	2.4	2.8	--
E0600	E0500	60	18'' DIA	23	4.8%	75.03	72.13	78.1	75.8	80.1	79.5	85.6	84.6	10.8	12.3	13.6	14.5	Existing 10-yr
E0700	E0600	87	18'' DIA	25	5.7%	80.00	75.03	82.5	78.1	81.1	80.1	87.0	85.6	10.8	12.1	13.5	14.4	Future 10-yr
E0800	E0700	277	18'' DIA	7	1.5%	84.10	80.00	87.6	82.5	92.0	81.1	102.6	87.0	10.8	12.0	13.6	14.6	Existing 10-yr
E0900	E0800	120	12'' DIA	8	5.0%	90.49	84.42	94.5	87.6	102.1	92.0	118.7	102.6	10.9	12.2	14.4	15.5	Existing 10-yr
E1000	E0900	214	12'' DIA	8	4.8%	100.88	90.54	104.3	94.5	108.3	102.1	128.1	118.7	6.8	7.1	8.5	8.9	Existing 10-yr
E1100	E1000	219	12'' DIA	7	4.0%	109.81	100.98	112.8	104.3	114.8	108.3	138.8	128.1	6.8	7.1	8.3	8.6	Existing 10-yr
E1200	E1100	239	12'' DIA	7	3.1%	117.36	109.86	120.9	112.8	121.9	114.8	150.6	138.8	7.4	7.8	8.9	9.2	Existing 10-yr
E1300	E1200	252	12'' DIA	8	4.1%	127.60	117.41	131.0	120.9	131.0	121.9	163.2	150.6	7.1	9.0	10.1	10.7	Future 10-yr
E1400	E1300	251	12'' DIA	8	4.7%	139.49	127.65	142.9	131.0	140.1	131.0	170.8	163.2	5.3	6.7	8.4	8.4	Future 10-yr
E1500	E1400	250	12'' DIA	9	5.2%	152.52	139.54	156.3	142.9	153.1	140.1	178.8	170.8	5.3	6.7	9.0	9.7	Future 10-yr
E1600	E1500	175	12'' DIA	9	5.8%	162.80	152.57	165.1	156.3	163.4	153.1	185.1	178.8	5.3	6.7	10.4	11.6	Future 10-yr
E1700	E1600	211	12'' DIA	9	6.3%	176.19	162.88	177.7	165.1	176.5	163.4	186.1	185.1	2.2	2.8	3.9	4.5	Future 10-yr
E1800	E1700	249	15'' DIA	17	6.3%	191.90	176.19	194.8	177.7	192.2	176.5	192.3	186.1	2.2	2.8	3.9	4.5	--
E1900	E1800	301	18'' DIA	19	2.8%	201.39	192.88	204.4	194.8	201.7	192.2	201.9	192.3	2.2	2.8	3.9	4.5	--
E2000	E1900	109	18'' DIA	22	4.2%	207.06	202.44	209.1	204.4	207.4	201.7	207.5	201.9	2.2	2.8	3.9	4.5	--
E2100	E2000	150	12'' DIA	8	4.2%	213.42	207.06	216.8	209.1	213.8	207.4	213.9	207.5	2.2	2.8	3.9	4.5	--
E2200	E2100	182	12'' DIA	7	3.4%	219.70	213.47	222.2	216.8	220.1	213.8	220.3	213.9	2.2	2.8	3.9	4.5	--
BASIN F																		
Outfall	F0100					109.62		113.5		111.2		111.2		14.3	16.4	16.1	18.0	
F0200	F0100	207	15'' DIA	5	0.6%	111.11	109.96	114.7	113.5	118.5	111.2	120.3	111.2	12.2	13.9	13.6	15.1	Existing 10-yr
F0300	F0200	166	15'' DIA	8	1.6%	113.82	111.16	116.5	114.7	122.2	118.5	125.1	120.3	9.7	11.2	11.0	12.3	Existing 10-yr
F0400	F0300	114	15'' DIA	5	0.7%	114.61	113.86	117.7	116.5	124.7	122.2	128.4	125.1	9.8	11.2	11.0	12.4	Existing 10-yr
F0500	F0400	73	15'' DIA	11	3.0%	117.18	114.96	119.5	117.7	126.0	124.7	130.1	128.4	8.7	10.0	9.9	11.1	Existing 10-yr
F0600	F0500	186	15'' DIA	9	2.0%	120.81	117.18	124.9	119.5	129.2	126.0	134.4	130.1	9.3	10.6	10.4	11.9	Existing 10-yr
F0610	F0600	406	12'' DIA	10	7.9%	153.13	121.11	157.7	124.9	153.5	129.2	153.5	134.4	3.0	3.5	3.3	3.8	--
F0620	F0610	45	12'' DIA	17	22.8%	163.21	153.13	165.6	157.7	163.5	153.5	163.5	153.5	2.2	2.6	2.4	2.8	--
F0630	F0620	184	12'' DIA	12	11.9%	185.30	163.61	188.0	165.6	185.6	163.5	185.6	163.5	2.2	2.6	2.4	2.8	--
F0640	F0630	182	12'' DIA	6	2.4%	189.70	185.30	191.2	188.0	190.0	185.6	190.0	185.6	1.2	1.4	1.3	1.5	--
F0650	F0640	182	6'' DIA	1	2.4%	194.13	189.70	195.2	191.2	198.5	190.0	200.1	190.0	1.3	1.5	1.4	1.6	Existing 10-yr

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H/V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
F0700	F0600	37	15'' DIA	16	6.6%	123.49	121.06	126.5	124.9	129.5	129.2	134.8	134.4	7.0	7.9	7.8	9.0	Existing 10-yr
F0710	F0700	186	12'' DIA	10	7.3%	137.08	123.59	138.6	126.5	137.3	129.5	137.3	134.8	1.1	1.3	1.3	1.5	--
F0720	F0710	328	12'' DIA	12	10.8%	172.32	137.13	174.4	138.6	172.5	137.3	172.5	137.3	0.7	0.9	0.9	1.1	--
F0900	F0700	279	12'' DIA	6	3.3%	132.75	123.59	136.2	126.5	136.2	129.5	143.8	134.8	5.9	7.3	7.1	8.2	Future 10-yr
F1000	F0900	32	12'' DIA	5	2.1%	133.44	132.75	136.4	136.2	136.5	136.2	144.4	143.8	4.0	6.1	5.9	7.1	Existing 10-yr
F1100	F1000	392	12'' DIA	7	4.2%	150.01	133.44	160.5	136.4	150.5	136.5	158.5	144.4	4.0	6.1	5.8	7.1	--
F1500	F1400	71	12'' DIA	11	9.7%	156.84	150.01	160.9	160.5	157.1	150.5	157.2	158.5	1.4	3.0	2.8	3.9	--
F1600	F1500	49	12'' DIA	3	0.9%	157.46	157.04	160.4	160.9	157.9	157.1	158.2	157.2	1.4	3.0	2.8	4.2	--
F1700	F1600	33	12'' DIA	7	3.7%	158.60	157.36	161.7	160.4	158.9	157.9	159.0	158.2	1.4	3.0	2.8	4.2	--
F1800	F1700	92	12'' DIA	4	1.0%	159.66	158.70	163.1	161.7	160.1	158.9	160.3	159.0	1.4	3.0	2.8	4.4	--
BASIN G																		
Outfall	G0100					48.58		50.1		50.1		50.1		1.6	2.0	2.7	3.1	
G0300	G0100	49	18'' DIA	10	3.0%	50.04	48.58	52.5	50.1	50.5	50.1	50.6	50.1	1.6	2.0	2.7	3.1	--
G0400	G0300	308	12'' DIA	3	0.5%	51.56	50.04	55.5	52.5	52.1	50.5	52.4	50.6	1.6	2.0	2.7	3.1	--
G0500	G0400	76	12'' DIA	3	0.6%	52.08	51.66	59.2	55.5	52.4	52.1	52.4	52.4	0.6	0.6	0.6	0.8	--
G0600	G0500	66	12'' DIA	3	0.5%	52.39	52.08	59.6	59.2	52.7	52.4	52.7	52.4	0.6	0.7	0.6	0.7	--
G0700	G0600	279	12'' DIA	3	0.5%	53.90	52.39	58.0	59.6	54.2	52.7	54.2	52.7	0.6	0.7	0.6	0.7	--
BASIN H																		
Outfall	H0100					43.15		44.7		44.7		44.7		8.3	9.5	10.2	11.0	
H0200	H0100	409	18'' DIA	16	2.3%	52.73	43.15	58.6	44.7	53.5	44.7	53.6	44.7	8.6	9.5	10.2	11.0	--
H0300	H0200	47	18'' DIA	29	7.2%	56.13	52.78	58.9	58.6	56.7	53.5	56.8	53.6	8.7	9.5	10.2	11.0	--
H0400	H0300	327	18'' DIA	6	0.3%	57.30	56.37	60.4	58.9	60.4	56.7	60.2	56.8	9.1	9.1	9.8	10.6	--
H0500	H0400	115	18'' DIA	7	0.4%	57.80	57.30	59.3	60.4	59.9	60.4	61.1	60.2	8.3	9.2	9.9	10.7	Existing 10-yr
H0510	H0500	147	1.5' H, 1.5'BW, 2 SS Channel	21	1.0%	59.32	57.80	61.3	59.3	59.9	59.9	61.1	61.1	0.9	1.0	0.9	1.0	--
H0520	H0510	91	24'' DIA	11	0.8%	60.06	59.32	63.1	61.3	60.5	59.9	61.1	61.1	0.9	1.0	0.9	1.0	--
H0700	H0500	42	18'' DIA	5	0.7%	58.11	57.80	59.6	59.3	60.6	59.9	62.2	61.1	7.8	9.1	9.9	10.9	Existing 10-yr
H0710	H0700	49	1.25' H, 1.5'BW, 10 SS Channel	39	0.6%	58.40	58.11	59.7	59.6	60.6	60.6	62.2	62.2	3.0	3.6	4.2	4.9	Existing 10-yr
H0720	H0710	119	15'' DIA	8	1.7%	60.37	58.40	68.7	59.7	60.8	60.6	62.4	62.2	2.4	2.8	3.0	3.5	--
H0800	H0700	169	1.5' H, 1.5'BW, 2 SS Channel	28	1.7%	61.00	58.11	63.0	59.6	61.7	60.6	62.4	62.2	5.1	6.0	6.4	6.9	--
H0810	H0800	24	1.5' H, 1'BW, 2 SS Channel	19	-0.9%	60.78	61.00	62.3	63.0	61.7	61.7	62.4	62.4	0.7	0.9	0.9	1.0	Future 10-yr
H0820	H0810	56	18'' DIA	9	0.7%	61.20	60.78	62.7	62.3	61.7	61.7	62.4	62.4	0.7	0.9	0.9	1.0	--
H0830	H0820	34	1.5' H, 1.5'BW, 2 SS Channel	22	1.6%	61.75	61.20	63.3	62.7	61.9	61.7	62.4	62.4	0.4	0.5	0.5	0.6	--
H0840	H0830	579	12'' DIA	2	0.3%	63.70	61.75	67.9	63.3	64.0	61.9	64.1	62.4	0.4	0.5	0.5	0.6	--
H0850	H0840	142	12'' DIA	4	1.0%	65.40	63.93	69.1	67.9	65.6	64.0	65.7	64.1	0.4	0.5	0.5	0.6	--
H0860	H0850	188	12'' DIA	3	0.9%	67.13	65.49	70.8	69.1	67.4	65.6	67.4	65.7	0.4	0.5	0.5	0.6	--
H0870	H0860	88	12'' DIA	3	0.9%	67.97	67.21	71.5	70.8	68.2	67.4	68.2	67.4	0.4	0.5	0.5	0.6	--
H1000	H0800	143	12'' DIA	3	-0.7%	59.94	61.00	62.3	63.0	63.8	61.7	65.9	62.4	4.4	5.1	5.8	6.1	Existing 10-yr
H1200	H1000	143	12'' DIA	3	0.7%	61.10	60.14	64.2	62.3	65.9	63.8	69.6	65.9	4.4	5.2	5.9	6.3	Existing 10-yr
H1300	H1200	148	12'' DIA	2	0.5%	61.86	61.15	65.5	64.2	66.3	65.9	70.1	69.6	1.9	2.2	2.3	2.6	Existing 10-yr
H1400	H1300	120	12'' DIA	3	0.9%	63.06	61.96	66.2	65.5	66.7	66.3	70.5	70.1	1.9	2.3	2.4	2.7	Existing 10-yr
H1500	H1400	166	12'' DIA	4	1.2%	65.15	63.16	68.7	66.2	67.2	66.7	71.1	70.5	1.9	2.3	2.5	2.9	Future 10-yr
H1600	H1500	117	12'' DIA	6	3.2%	68.97	65.25	71.3	68.7	69.4	67.2	71.6	71.1	1.9	2.3	2.5	3.0	Future 10-yr
H1700	H1600	138	12'' DIA	2	0.4%	69.46	68.97	71.4	71.3	70.2	69.4	72.1	71.6	1.9	2.3	2.5	2.9	Future 10-yr
H1800	H1700	142	12'' DIA	2	0.4%	69.96	69.46	73.4	71.4	70.7	70.2	73.4	72.1	2.0	2.3	2.5	2.9	--

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
H1900	H1800	141	12'' DIA	2	0.3%	70.45	69.96	73.6	73.4	71.2	70.7	73.6	73.4	2.0	2.3	2.5	2.8	--
H2000	H1900	75	12'' DIA	2	0.3%	70.71	70.45	74.4	73.6	71.5	71.2	74.1	73.6	2.0	2.3	2.5	2.8	--
H2100	H2000	183	12'' DIA	3	0.6%	71.46	70.36	75.7	74.4	72.1	71.5	74.6	74.1	2.0	2.3	2.5	2.8	--
BASIN I																		
Outfall	I0100					62.81		64.8		63.2		63.3		6.8	7.7	7.3	8.5	
I0200	I0100	91	24'' DIA	68	8.4%	70.46	62.81	75.6	64.8	70.9	63.2	70.9	63.3	7.2	8.1	7.7	8.9	--
I0400	I0300	97	24'' DIA	12	0.3%	70.72	70.46	76.4	75.6	71.7	70.9	71.7	70.9	6.5	7.5	7.0	8.0	--
I0500	I0400	101	24'' DIA	40	3.2%	73.96	70.77	78.0	76.4	74.5	71.7	74.5	71.7	6.5	7.5	7.0	8.0	--
I0510	I0500	108	18'' DIA	16	2.4%	76.67	74.06	81.5	78.0	77.1	74.5	77.1	74.5	2.9	3.3	2.9	3.3	--
I0600	I0500	121	24'' DIA	36	2.6%	77.06	73.96	80.5	78.0	77.5	74.5	77.5	74.5	3.6	4.2	4.1	4.7	--
I0700	I0600	117	21'' DIA	26	2.5%	80.00	77.06	83.5	80.5	80.5	77.5	80.5	77.5	3.6	4.2	4.1	4.7	--
I0800	I0700	120	21'' DIA	24	2.2%	82.67	80.00	86.3	83.5	83.1	80.5	83.2	80.5	3.6	4.2	4.1	4.7	--
I0900	I0800	118	21'' DIA	26	2.5%	85.77	82.77	89.2	86.3	86.2	83.1	86.2	83.2	3.6	4.2	4.1	4.7	--
I1000	I0900	121	21'' DIA	21	1.9%	88.03	85.77	92.1	89.2	88.5	86.2	88.6	86.2	3.6	4.2	4.1	4.7	--
I1010	I1000	43	12'' DIA	11	9.2%	92.38	88.43	94.2	92.1	92.7	88.5	92.7	88.6	2.1	2.4	2.1	2.4	--
I1020	I1010	202	12'' DIA	3	0.9%	94.15	92.43	96.5	94.2	94.7	92.7	94.7	92.7	2.1	2.4	2.1	2.4	--
I1030	I1020	201	12'' DIA	3	0.9%	95.87	94.15	99.0	96.5	96.5	94.7	96.5	94.7	2.1	2.4	2.1	2.4	--
I1100	I1000	203	21'' DIA	16	1.2%	90.73	88.38	95.1	92.1	91.1	88.5	91.1	88.6	1.5	1.8	2.0	2.3	--
I1200	I1100	198	21'' DIA	18	1.5%	93.68	90.73	97.2	95.1	94.0	91.1	94.1	91.1	1.5	1.8	2.0	2.3	--
I1300	I1200	202	21'' DIA	21	1.6%	96.69	93.53	100.8	97.2	97.0	94.0	97.1	94.1	1.5	1.8	2.0	2.3	--
I1400	I1300	160	21'' DIA	18	1.3%	98.74	96.69	105.7	100.8	99.1	97.0	99.1	97.1	1.5	1.8	2.0	2.3	--
BASIN J																		
Basin J drains to Basin A at A3050																		
J0100	C2500	7	27'' DIA	22	-1.8%	71.40	71.53	76.2	75.5	77.4	77.0	78.6	78.2	39.8	44.7	43.9	47.5	Existing 10-yr
J0200	J0100	291	36'' DIA	15	0.2%	71.90	71.40	76.9	76.2	80.9	77.4	82.9	78.6	40.1	45.1	44.3	47.9	Existing 10-yr
J0300	J0200	124	36'' DIA	108	2.3%	74.78	71.90	79.8	76.9	81.0	80.9	83.0	82.9	19.8	22.9	23.0	24.9	Existing 10-yr
J0400	J0300	126	36'' DIA	330	25.1%	105.32	74.78	112.6	79.8	105.8	81.0	105.8	83.0	19.8	22.9	23.0	24.9	--
J0500	J0400	81	36'' DIA	152	4.6%	111.04	107.32	116.9	112.6	111.8	105.8	111.8	105.8	19.8	22.9	23.0	24.9	--
J0600	J0500	255	36'' DIA	19	0.1%	112.63	112.44	121.1	116.9	114.5	111.8	114.7	111.8	19.8	22.9	23.0	24.9	--
J0800	J0600	203	18'' DIA	22	4.5%	122.38	113.33	128.3	121.1	123.5	114.5	123.6	114.7	19.8	22.9	23.0	24.9	--
J0900	J0800	248	18'' DIA	23	4.6%	133.90	122.42	140.5	128.3	134.9	123.5	135.0	123.6	17.0	19.6	20.1	21.5	--
J1000	J0900	230	18'' DIA	20	3.6%	142.20	133.90	148.9	140.5	143.3	134.9	143.5	135.0	17.0	19.6	20.1	21.5	--
J1300	J1200	85	18'' DIA	16	2.0%	144.01	142.28	146.9	148.9	145.8	143.3	146.9	143.5	17.0	19.6	20.1	21.6	--
J1400	J1300	225	18'' DIA	16	2.2%	149.10	144.11	155.3	146.9	150.3	145.8	155.3	146.9	15.7	17.9	18.6	20.8	--
J1600	J1400	257	1.5' H, 1'BW, 2 SS Channel	52	7.8%	169.30	149.42	171.8	155.3	170.2	150.3	170.3	155.3	15.8	18.0	18.6	20.8	--
J1700	J1600	102	18'' DIA	13	1.3%	170.67	169.30	175.0	171.8	172.1	170.2	173.0	170.3	13.8	15.8	16.5	18.4	--
J1710	J1700	39	12'' DIA	4	1.5%	171.77	171.20	174.9	175.0	172.4	172.1	173.2	173.0	2.8	3.4	3.2	3.7	--
J1720	J1710	182	12'' DIA	3	0.8%	173.48	171.97	176.8	174.9	174.0	172.4	174.0	173.2	1.6	1.9	1.8	2.1	--
J1730	J1720	103	12'' DIA	6	2.2%	175.98	173.68	179.4	176.8	176.3	174.0	176.4	174.0	1.6	1.9	1.8	2.1	--
J1740	J1730	175	12'' DIA	6	2.6%	180.73	176.18	184.1	179.4	181.1	176.3	181.1	176.4	1.6	1.9	1.8	2.1	--
J1750	J1740	66	12'' DIA	6	2.4%	182.55	'180.93	186.2	184.1	182.9	181.1	182.9	181.1	1.6	1.9	1.8	2.1	--
J1800	J1700	87	18'' DIA	16	2.1%	172.70	170.87	176.4	175.0	173.6	172.1	174.0	173.0	11.0	12.4	13.3	14.7	--
J1900	J1800	136	18'' DIA	19	3.0%	177.03	172.90	182.0	176.4	177.9	173.6	178.0	174.0	11.0	12.4	13.3	14.7	--
J2000	J1900	179	18'' DIA	31	8.1%	191.69	177.23	196.5	182.0	192.3	177.9	192.4	178.0	11.0	12.4	13.3	14.7	--

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
J2100	J2000	58	18'' DIA	13	1.5%	192.75	191.89	197.4	196.5	193.8	192.3	193.9	192.4	10.5	11.7	12.0	13.1	--
J2200	J2100	149	18'' DIA	9	0.7%	193.83	192.85	198.3	197.4	195.6	193.8	196.1	193.9	10.5	11.7	12.0	13.1	--
J2300	J2200	42	18'' DIA	19	3.2%	195.35	194.03	199.2	198.3	196.3	195.6	196.5	196.1	10.5	11.7	12.0	13.1	--
J2400	J2300	48	12'' DIA	6	2.5%	196.74	195.51	199.8	199.2	198.9	196.3	199.1	196.5	7.9	8.6	8.2	8.8	--
J2500	J2400	327	12'' DIA	5	1.6%	201.93	196.65	205.1	199.8	215.0	198.9	216.3	199.1	8.3	8.8	8.5	9.0	Existing 10-yr
J2510	J2500	250	12'' DIA	7	4.2%	212.70	202.18	214.7	205.1	216.2	215.0	217.6	216.3	3.7	4.1	4.0	4.3	Existing 10-yr
J2530	J2520	66	12'' DIA	5	1.8%	213.90	212.70	217.4	214.7	216.3	216.2	217.7	217.6	1.5	1.7	1.7	2.1	Future 10-yr
J2540	J2530	45	12'' DIA	6	2.5%	215.03	213.90	219.6	217.4	216.3	216.3	217.8	217.7	1.5	1.7	1.6	2.1	--
J2550	J2540	103	12'' DIA	6	2.9%	217.97	215.03	220.0	219.6	218.3	216.3	218.3	217.8	1.5	1.7	1.6	1.9	--
J2560	J2550	364	12'' DIA	6	3.0%	229.26	218.32	233.9	220.0	229.6	218.3	229.6	218.3	1.5	1.7	1.6	1.9	--
J2610	J2500	406	12'' DIA	3	0.8%	205.23	202.06	207.7	205.1	224.9	215.0	226.6	216.3	5.9	6.2	6.0	6.4	Existing 10-yr
J2700	J2610	242	12'' DIA	4	1.4%	208.65	205.23	212.6	207.7	229.8	224.9	231.8	226.6	6.3	6.9	6.5	7.1	Existing 10-yr
J2710	J2700	209	12'' DIA	10	7.5%	224.31	208.74	229.4	212.6	230.3	229.8	232.3	231.8	2.7	3.2	2.8	3.3	Existing 10-yr
J2800	J2700	140	12'' DIA	5	2.3%	211.97	208.74	214.6	212.6	230.9	229.8	233.0	231.8	4.7	5.4	5.0	5.6	Existing 10-yr
J2900	J2800	183	12'' DIA	7	4.3%	219.93	212.13	227.5	214.6	232.5	230.9	234.6	233.0	4.7	5.4	5.0	5.7	Existing 10-yr
J3000	J2900	76	12'' DIA	9	7.0%	225.20	219.93	230.0	227.5	233.1	232.5	235.3	234.6	4.8	5.7	5.1	5.9	Existing 10-yr
J3100	J3000	112	12'' DIA	9	6.9%	233.19	225.50	237.4	230.0	233.8	233.1	237.4	235.3	3.6	4.2	3.8	4.4	--
J3200	J3100	87	12'' DIA	12	12.1%	243.87	233.34	248.3	237.4	244.2	233.8	244.3	237.4	3.6	4.2	3.8	4.4	--
J3210	J3200	126	12'' DIA	9	5.9%	251.38	243.97	256.1	248.3	251.7	244.2	251.7	244.3	1.7	2.0	1.8	2.1	--
J3300	J3200	108	12'' DIA	15	16.0%	260.96	243.97	263.9	248.3	261.2	244.2	261.2	244.3	1.9	2.2	2.0	2.4	--
J3400	J3300	205	12'' DIA	14	13.9%	289.34	261.11	293.5	263.9	289.6	261.2	289.6	261.2	1.9	2.2	2.0	2.4	--
J3500	J3400	279	12'' DIA	7	3.9%	300.34	289.41	305.2	293.5	300.7	289.6	300.7	289.6	1.9	2.2	2.0	2.4	--
J3600	J3500	103	12'' DIA	3	0.9%	301.42	300.48	303.6	305.2	302.0	300.7	302.0	300.7	1.9	2.2	2.0	2.4	--
J5000	J0200	136	18'' DIA	4	0.2%	72.11	71.90	76.3	76.9	84.5	80.9	86.6	82.9	17.9	19.6	18.5	20.1	Existing 10-yr
J5100	J5000	47	15'' DIA	11	2.8%	73.71	72.38	77.4	76.3	87.8	84.5	90.1	86.6	17.9	19.6	18.4	19.8	Existing 10-yr
J5200	J5100	228	18'' DIA	13	1.4%	77.13	73.84	80.4	77.4	94.0	87.8	96.5	90.1	18.3	19.5	18.5	19.8	Existing 10-yr
J5300	J5200	31	18'' DIA	8	0.6%	77.30	77.13	80.6	80.4	94.8	94.0	97.3	96.5	18.8	20.8	19.6	21.0	Existing 10-yr
J5500	J5300	55	24'' DIA	71	8.6%	82.00	77.30	85.0	80.6	95.0	94.8	97.6	97.3	17.7	19.5	18.3	20.0	Existing 10-yr
J5600	J5500	124	2' H, 1'BW, 2 SS Channel	126	11.0%	95.50	82.00	97.5	85.0	96.4	95.0	97.8	97.6	17.7	20.3	18.7	20.7	Future 10-yr
J5700	J5600	29	24'' DIA	40	10.8%	98.60	95.50	100.6	97.5	99.5	96.4	99.6	97.8	18.0	21.1	19.3	21.7	--
J5800	J5700	112	2' H, 1'BW, 2 SS Channel	120	11.0%	110.85	98.60	116.9	100.6	111.7	99.5	111.7	99.6	19.9	22.8	21.3	23.0	--
J5820	J5800	100	12'' DIA	4	1.5%	112.34	110.85	118.4	116.9	112.7	111.7	112.7	111.7	1.2	1.5	1.3	1.5	--
J5900	J5800	79	24'' DIA	11	0.3%	111.05	110.85	118.0	116.9	112.7	111.7	112.8	111.7	16.3	19.0	17.4	19.7	--
J6000	J5900	99	24'' DIA	11	0.2%	111.37	111.15	116.8	118.0	113.2	112.7	113.4	112.8	16.2	19.0	17.1	19.7	--
J6100	J6000	90	24'' DIA	25	1.3%	112.50	111.37	118.7	116.8	113.8	113.2	113.8	113.4	16.2	19.0	17.0	19.7	--
J6110	J6100	173	24'' DIA	13	0.3%	113.76	113.18	116.9	118.7	114.7	113.8	114.7	113.8	5.5	6.2	5.6	6.3	--
J6120	J6110	167	24'' DIA	12	0.3%	114.32	113.86	117.1	116.9	115.3	114.7	115.3	114.7	5.5	6.2	5.6	6.3	--
J6130	J6120	246	12'' DIA	2	0.5%	115.72	114.52	118.3	117.1	121.0	115.3	121.3	115.3	5.6	6.3	5.7	6.4	Existing 10-yr
J6140	J6130	42	12'' DIA	2	0.4%	115.97	115.82	118.3	118.3	121.4	121.0	121.7	121.3	3.2	3.6	3.3	3.7	Existing 10-yr
J6160	J6140	110	12'' DIA	1	0.1%	116.18	116.07	120.0	118.3	122.2	121.4	122.6	121.7	3.3	3.7	3.4	3.8	Existing 10-yr
J6170	J6160	54	12'' DIA	3	0.9%	116.78	116.28	122.7	120.0	122.7	122.2	123.0	122.6	3.3	3.9	3.4	4.1	Future 10-yr
J6180	J6170	242	12'' DIA	10	8.1%	136.32	116.85	145.6	122.7	136.7	122.7	136.7	123.0	3.3	3.9	3.4	4.1	--
J6200	J6100	115	24'' DIA	33	2.1%	115.49	113.08	120.0	118.7	116.2	113.8	116.2	113.8	8.7	10.3	9.2	10.8	--
J6300	J6200	168	24'' DIA	41	2.9%	120.54	115.59	125.6	120.0	121.2	116.2	121.2	116.2	8.7	10.3	9.2	10.8	--

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
J6310	J6300	169	18" DIA	16	8.4%	134.60	120.54	139.1	125.6	135.0	121.2	135.0	121.2	2.3	2.7	2.4	2.8	--
J6320	J6310	119	12" DIA	10	7.1%	143.02	134.60	147.2	139.1	143.4	135.0	143.4	135.0	2.3	2.7	2.4	2.8	--
J6330	J6320	35	12" DIA	4	1.6%	143.59	143.02	147.3	147.2	144.1	143.4	144.1	143.4	2.3	2.7	2.4	2.8	--
J6331	J6330	161	12" DIA	3	0.7%	144.69	143.64	148.4	147.3	145.4	144.1	145.4	144.1	2.3	2.7	2.4	2.8	--
J6400	J6300	89	18" DIA	20	3.4%	123.76	120.74	128.4	125.6	124.3	121.2	124.4	121.2	6.4	7.6	6.9	8.1	--
J6500	J6400	185	18" DIA	32	9.5%	141.82	124.36	145.6	128.4	142.3	124.3	142.3	124.4	6.4	7.6	6.9	8.1	--
J6600	J6500	140	18" DIA	9	0.7%	143.04	142.10	146.2	145.6	143.9	142.3	143.9	142.3	5.3	6.2	5.6	6.6	--
J6700	J6600	149	15" DIA	6	0.7%	144.33	143.22	148.1	146.2	145.1	143.9	145.1	143.9	3.9	4.6	4.1	4.8	--
J6800	J6700	94	15" DIA	17	6.8%	150.77	144.36	153.6	148.1	151.2	145.1	151.2	145.1	3.9	4.6	4.1	4.8	--
J6900	J6800	190	15" DIA	11	3.2%	156.97	150.87	161.3	153.6	157.5	151.2	157.5	151.2	3.9	4.6	4.1	4.8	--
J7000	J6900	170	15" DIA	13	3.8%	165.22	158.72	169.0	161.3	165.7	157.5	165.7	157.5	3.9	4.6	4.1	4.8	--
J7100	J7000	107	15" DIA	6	0.8%	166.21	165.32	170.1	169.0	167.0	165.7	167.0	165.7	3.9	4.6	4.1	4.8	--
BASIN K																		
Outfall	K0000					207.00		210.0		209.5		209.5		10.0	12.1	12.1	14.3	
K0100	K0000	50	30" DIA	200	23.0%	218.20	207.00	225.1	210.0	218.6	209.5	218.6	209.5	10.0	12.1	12.1	14.3	--
K0110	K0100	76	24" DIA	57	5.9%	222.88	218.40	230.0	225.1	223.2	218.6	223.2	218.6	3.4	4.1	3.5	4.1	--
K0120	K0110	54	18" DIA	19	3.1%	226.96	225.28	231.3	230.0	227.4	223.2	227.4	223.2	3.4	4.1	3.5	4.1	--
K0130	K0120	304	15" DIA	8	1.5%	231.68	227.16	237.3	231.3	232.1	227.4	232.1	227.4	2.0	2.4	2.1	2.4	--
K0140	K0130	168	15" DIA	18	7.9%	245.16	231.83	253.2	237.3	245.3	232.1	245.3	232.1	0.7	0.8	0.7	0.9	--
K0150	K0140	161	12" DIA	14	16.5%	271.47	245.28	277.1	253.2	271.6	245.3	271.6	245.3	0.7	0.8	0.7	0.9	--
K0160	K0150	137	12" DIA	12	11.4%	287.19	271.67	290.8	277.1	287.4	271.6	287.4	271.6	0.7	0.8	0.7	0.9	--
K0200	K0100	76	24" DIA	16	0.5%	218.70	218.30	228.4	225.1	219.6	218.6	219.7	218.6	6.6	8.0	8.6	10.2	--
K0300	K0200	81	18" DIA	45	19.2%	234.00	218.70	242.0	228.4	234.3	219.6	234.3	219.7	5.0	5.9	5.3	6.2	--
K0400	K0300	119	12" DIA	21	33.5%	271.66	234.00	280.0	242.0	272.0	234.3	272.0	234.3	5.0	5.9	5.3	6.2	--
K0500	K0400	247	12" DIA	14	16.2%	311.50	271.96	315.3	280.0	311.9	272.0	311.9	272.0	5.0	5.9	5.3	6.2	--
K0600	K0500	17	12" DIA	6	2.7%	312.12	311.65	315.8	315.3	312.7	311.9	312.7	311.9	3.4	4.0	3.5	4.1	--
K0700	K0600	303	12" DIA	3	0.8%	314.67	312.12	319.6	315.8	315.6	312.7	316.5	312.7	3.4	4.0	3.5	4.1	--
K0710	K0700	35	12" DIA	5	2.2%	315.52	314.74	319.8	319.6	315.9	315.6	316.2	316.5	1.5	1.7	1.5	1.8	--
K0720	K0710	198	12" DIA	5	1.8%	319.58	316.02	322.5	319.8	320.0	315.9	320.0	316.2	1.5	1.7	1.5	1.8	--
K0800	K0700	351	12" DIA	5	1.6%	320.28	314.72	323.1	319.6	320.7	315.6	320.8	316.5	1.9	2.3	2.0	2.3	--
K0900	K0800	167	12" DIA	3	0.7%	321.52	320.38	324.6	323.1	322.1	320.7	322.1	320.8	1.9	2.3	2.0	2.3	--
BASIN L																		
Basin L drains to Basin A at A5100																		
L0100	A5100	158	18" DIA	29	7.7%	181.86	169.68	184.2	172.7	182.4	172.0	182.5	172.1	8.5	10.2	10.0	11.7	--
L0200	L0100	286	18" DIA	23	4.8%	195.71	181.91	198.6	184.2	196.3	182.4	196.4	182.5	7.7	9.2	9.0	10.6	--
L0300	L0200	96	18" DIA	23	5.0%	200.68	195.91	203.1	198.6	201.3	196.3	201.3	196.4	7.7	9.2	9.0	10.6	--
L0400	L0300	112	18" DIA	24	5.3%	206.78	200.88	210.1	203.1	207.4	201.3	207.4	201.3	7.7	9.2	9.0	10.6	--
L0500	L0400	332	18" DIA	11	1.1%	210.77	206.98	218.8	210.1	211.6	207.4	211.7	207.4	6.6	7.8	7.2	8.5	--
L0510	L0500	303	15" DIA	24	14.5%	254.32	210.97	258.5	218.8	254.6	211.6	254.6	211.7	2.7	3.1	2.8	3.3	--
L0520	L0510	123	15" DIA	27	15.9%	273.69	254.42	278.7	258.5	274.0	254.6	274.0	254.6	2.7	3.1	2.8	3.3	--
L0530	L0520	92	15" DIA	22	11.7%	284.43	273.72	288.6	278.7	284.7	274.0	284.7	274.0	2.7	3.1	2.8	3.3	--
L0540	L0530	118	15" DIA	18	7.2%	293.01	284.50	296.1	288.6	293.3	284.7	293.4	284.7	2.7	3.1	2.8	3.3	--
L0550	L0540	349	15" DIA	15	5.5%	312.10	293.06	317.3	296.1	312.4	293.3	312.4	293.4	1.7	2.0	1.8	2.1	--
L0560	L0550	89	15" DIA	13	4.3%	316.41	312.60	325.6	317.3	316.7	312.4	316.7	312.4	1.7	2.0	1.8	2.1	--

Table B-2. Hydraulic Model Parameters and Results																		
Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
L0570	L0560	137	12'' DIA	7	4.3%	322.25	316.41	327.5	325.6	322.6	316.7	322.6	316.7	1.7	2.0	1.8	2.1	--
L0600	L0500	188	18'' DIA	26	6.4%	222.96	210.97	226.3	218.8	223.3	211.6	223.3	211.7	2.4	2.8	2.5	3.0	--
L0700	L0600	155	18'' DIA	18	3.2%	228.07	223.06	233.0	226.3	228.4	223.3	228.4	223.3	2.4	2.8	2.5	3.0	--
L0800	L0700	132	12'' DIA	12	11.5%	243.23	228.17	249.1	233.0	243.4	228.4	243.5	228.4	1.1	1.3	1.2	1.5	--
L0900	L0800	67	12'' DIA	9	7.0%	248.01	243.34	252.4	249.1	248.2	243.4	248.3	243.5	1.1	1.3	1.2	1.5	--
L1000	L0900	233	12'' DIA	14	14.0%	280.21	248.05	283.7	252.4	280.4	248.2	280.4	248.3	1.1	1.3	1.2	1.5	--
BASIN M																		
Drains to Clackamas County Stormdrain																		
M0200	M0100	189	15'' DIA	6	2.8%	283.82	278.50	286.4	279.8	284.7	279.8	284.8	279.8	5.1	5.7	5.3	5.9	--
M0300	M0200	194	15'' DIA	4	1.2%	286.27	283.92	289.0	286.4	289.0	284.7	289.3	284.8	5.2	5.7	5.3	5.9	Existing 10-yr
M0400	M0300	130	15'' DIA	2	0.2%	286.68	286.37	288.1	289.0	291.2	289.0	291.6	289.3	4.6	5.2	4.8	5.3	Existing 10-yr
M0500	M0400	29	15'' DIA	5	0.5%	286.88	286.73	288.7	288.1	291.3	291.2	291.7	291.6	4.7	5.3	4.9	5.5	Existing 10-yr
M0510	M0500	138	9.96'' DIA	2	0.8%	288.27	287.18	289.5	288.7	291.9	291.3	292.3	291.7	1.6	1.8	1.6	1.9	Existing 10-yr
M0600	M0500	151	12'' DIA	5	1.6%	289.30	286.88	291.6	288.7	292.4	291.3	292.9	291.7	4.2	4.5	4.3	4.5	Existing 10-yr
M0610	M0600	148	12'' DIA	5	1.9%	292.24	289.50	293.7	291.6	292.7	292.4	293.3	292.9	1.9	2.2	1.9	2.3	--
M0700	M0600	94	12'' DIA	3	0.8%	290.16	289.40	291.9	291.6	292.6	292.4	293.1	292.9	2.51*	2.64*	2.61*	2.7*	Existing 10-yr
M0700A	M0510	134	12'' DIA	3	1.7%	290.61	288.27	291.9	289.5	292.6	291.9	293.1	292.3	1.55*	1.75*	1.61*	1.86*	Existing 10-yr
M0800	M0700	82	12'' DIA	2	0.2%	290.45	290.26	292.1	291.9	293.2	292.6	293.7	293.1	3.3	3.9	3.4	4.0	Existing 10-yr
M0900	M0800	129	12'' DIA	6	2.7%	294.00	290.55	296.0	292.1	294.4	293.2	294.5	293.7	2.4	2.9	2.4	2.9	--
M1000	M0900	101	12'' DIA	6	2.7%	296.69	294.00	298.7	296.0	297.1	294.4	297.1	294.5	2.4	2.9	2.4	2.9	--
BASIN N																		
Drains to Clackamas County Stormdrain																		
N0200	N0100	82	24'' DIA	24	1.1%	220.57	219.67	223.2	221.6	223.4	221.6	223.6	221.7	34.3	38.1	35.1	38.9	Existing 10-yr
N0300	N0200	454	24'' DIA	27	1.4%	227.12	220.67	231.4	223.2	233.9	223.4	234.4	223.6	34.5	38.4	35.3	39.2	Existing 10-yr
N0310	N0300	165	18'' DIA	35	10.9%	245.63	227.68	249.6	231.4	246.1	233.9	246.1	234.4	6.6	7.8	6.9	8.1	--
N0320	N0310	165	15'' DIA	21	10.9%	263.59	245.63	267.4	249.6	264.1	246.1	264.1	246.1	6.6	8.3	7.1	8.8	--
N0330	N0320	21	15'' DIA	7	1.0%	263.80	263.59	268.8	267.4	264.9	264.1	264.9	264.1	6.6	7.8	6.9	8.1	--
N0340	N0330	84	12'' DIA	5	2.2%	265.60	263.80	270.0	268.8	266.3	264.9	266.3	264.9	4.0	4.7	4.1	4.8	--
N0350	N0340	287	12'' DIA	10	8.2%	288.95	265.60	292.7	270.0	289.4	266.3	289.4	266.3	4.0	4.7	4.1	4.8	--
N0360	N0350	277	12'' DIA	5	1.7%	293.59	288.95	297.3	292.7	294.1	289.4	294.1	289.4	2.4	2.8	2.5	3.0	--
N0400	N0300	101	21'' DIA	14	0.8%	227.90	227.14	230.7	231.4	236.6	233.9	237.3	234.4	26.1	28.9	26.7	29.5	Existing 10-yr
N0402	N0400	489	12'' DIA	2	0.4%	230.09	228.08	233.0	230.7	245.3	236.6	246.4	237.3	5.4	6.4	5.6	6.5	Existing 10-yr
N0403	N0402	300	12'' DIA	12	11.9%	265.55	230.09	270.0	233.0	266.0	245.3	266.0	246.4	4.2	4.9	4.3	5.0	--
N0404	N0403	54	12'' DIA	9	6.9%	269.44	265.73	272.8	270.0	269.8	266.0	269.8	266.0	3.0	3.5	3.1	3.6	--
N0405	N0404	295	12'' DIA	7	3.7%	280.39	269.47	285.2	272.8	280.8	269.8	280.8	269.8	2.4	2.8	2.5	2.9	--
N0410	N0400	67	12'' DIA	4	1.1%	228.85	228.08	231.1	230.7	239.7	236.6	240.4	237.3	8.4	9.5	8.4	9.4	Existing 10-yr
N0420	N0410	376	12'' DIA	10	8.4%	260.40	228.95	271.2	231.1	261.2	239.7	261.2	240.4	8.4	10.1	8.4	10.1	--
N0421	N0420	226	12'' DIA	4	1.1%	263.36	260.77	266.7	271.2	263.9	261.2	263.9	261.2	2.1	2.5	2.1	2.5	--
N0422	N0421	187	12'' DIA	11	9.2%	280.67	263.49	283.2	266.7	281.0	263.9	281.0	263.9	2.1	2.5	2.1	2.5	--
N0430	N0420	231	12'' DIA	6	2.8%	273.16	266.71	278.2	271.2	273.7	261.2	273.7	261.2	3.8	4.4	3.8	4.4	--
N0440	N0430	177	12'' DIA	9	6.3%	284.41	273.28	286.7	278.2	284.9	273.7	284.9	273.7	3.8	4.4	3.8	4.4	--
N0450	N0440	103	12'' DIA	6	2.9%	287.58	284.56	290.4	286.7	288.2	284.9	288.2	284.9	3.8	4.4	3.8	4.4	--
N0460	N0450	158	12'' DIA	2	0.5%	288.30	287.58	292.0	290.4	292.0	288.2	292.0	288.2	3.8	4.4	3.8	4.4	--

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
N0470	N0460	180	12'' DIA	8	4.5%	296.43	288.30	300.9	292.0	296.8	292.0	296.8	292.0	2.2	2.5	2.2	2.5	--
N0500	N0400	268	21'' DIA	5	0.1%	228.34	228.08	231.5	230.7	238.1	236.6	239.0	237.3	12.5	13.7	13.0	14.3	Existing 10-yr
N0510	N0500	538	18'' DIA	27	6.8%	264.92	228.54	269.2	231.5	265.4	238.1	265.4	239.0	5.2	6.0	5.2	6.0	--
N0520	N0510	232	12'' DIA	5	2.4%	270.49	264.99	273.6	269.2	270.9	265.4	270.9	265.4	1.9	2.2	1.9	2.2	--
N0600	N0500	40	12'' DIA	3	0.7%	229.20	228.94	231.5	231.5	239.9	238.1	241.0	239.0	8.2	8.9	8.9	9.4	Existing 10-yr
N0700	N0600	163	12'' DIA	3	0.8%	230.56	229.30	233.0	231.5	243.1	239.9	245.2	241.0	6.4	7.0	7.1	7.6	Existing 10-yr
N0900	N0800	321	12'' DIA	1	0.2%	231.07	230.56	234.0	233.0	250.9	243.1	255.4	245.2	6.1	6.7	6.8	7.3	Existing 10-yr
N1000	N0900	134	12'' DIA	1	0.1%	231.25	231.07	233.2	234.0	254.3	250.9	259.7	255.4	6.9	7.4	7.8	7.4	Existing 10-yr
N1100	N1000	162	12'' DIA	11	9.5%	246.55	231.30	249.6	233.2	258.3	254.3	264.8	259.7	7.0	7.8	8.2	8.1	Existing 10-yr
N1200	N1100	173	12'' DIA	3	0.8%	248.01	246.55	250.5	249.6	259.6	258.3	266.6	264.8	4.3	4.7	5.1	5.1	Existing 10-yr
N1300	N1200	250	12'' DIA	5	2.1%	253.30	248.01	256.3	250.5	261.5	259.6	269.2	266.6	4.6	5.1	5.3	5.5	Existing 10-yr
N1400	N1300	328	15'' DIA	7	0.9%	256.37	253.30	258.4	256.3	262.3	261.5	270.2	269.2	4.6	5.7	6.4	7.1	Existing 10-yr
N1500	N1400	113	15'' DIA	15	5.1%	262.07	256.37	265.6	258.4	262.5	262.3	270.4	270.2	3.7	4.3	4.0	4.6	Future 10-yr
N1600	N1500	68	15'' DIA	11	2.9%	264.07	262.07	271.8	265.6	264.6	262.5	270.5	270.4	3.7	4.4	3.8	4.6	--
N1700	N1600	175	15'' DIA	7	1.3%	266.26	264.07	272.3	271.8	266.9	264.6	271.3	270.5	3.7	4.3	3.8	4.5	--
N1800	N1700	102	15'' DIA	12	3.6%	269.91	266.26	275.1	272.3	270.4	266.9	270.8	271.3	3.7	4.3	3.8	4.5	--
BASIN O																		
Outfall To Pond	O0400					138.33		142.1		139.2		139.2		20.0	21.9	20.9	22.7	
O0600	O0400	257	1' H, 1'BW, 2 SS Channel	24	10.7%	165.67	138.33	168.7	142.1	166.6	139.2	166.6	139.2	18.8	20.3	19.1	20.5	--
O0700	O0600	89	36'' DIA	89	1.6%	167.14	165.67	170.8	168.7	168.1	166.6	168.1	166.6	18.8	20.4	19.1	20.5	--
O0710	O0700	35	15'' DIA	11	2.8%	169.11	168.14	171.7	170.8	171.5	168.1	171.6	168.1	16.6	17.7	16.8	17.9	--
O0711	O0710	47	12'' DIA	6	2.4%	170.27	169.16	172.5	171.7	178.9	171.5	179.3	171.6	14.8	15.7	15.0	15.9	Existing 10-yr
O0712	O0711	67	12'' DIA	6	2.4%	171.95	170.37	174.5	172.5	180.0	178.9	180.5	179.3	4.9	5.5	5.1	5.6	Existing 10-yr
O0713	O0712	212	12'' DIA	7	3.7%	179.96	172.05	183.7	174.5	183.1	180.0	183.7	180.5	4.4	5.3	4.6	5.2	--
O0714	O0713	244	12'' DIA	6	2.3%	185.99	180.36	189.5	183.7	186.7	183.1	186.8	183.7	4.4	5.4	4.6	5.4	--
O0715	O0714	235	12'' DIA	7	3.8%	194.94	185.99	199.0	189.5	195.5	186.7	195.5	186.8	4.4	5.2	4.6	5.4	--
O0720	O0710	96	12'' DIA	5	1.7%	171.37	169.71	173.9	171.7	171.8	171.5	171.8	171.6	1.9	2.2	1.9	2.2	--
O0730	O0720	39	12'' DIA	5	1.6%	172.04	171.42	175.5	173.9	172.5	171.8	172.5	171.8	1.9	2.2	1.9	2.2	--
O0740	O0730	140	12'' DIA	7	3.8%	177.43	172.14	181.3	175.5	177.8	172.5	177.8	172.5	1.9	2.2	1.9	2.2	--
O0750	O0740	219	12'' DIA	6	2.4%	182.80	177.53	186.9	181.3	183.2	177.8	183.2	177.8	1.9	2.2	1.9	2.2	--
O0800	O0700	11	36'' DIA	90	1.7%	167.33	167.14	171.0	170.8	168.1	168.1	168.1	168.1	0.8	1.0	0.8	1.0	--
O0900	O0800	36	18'' DIA	16	2.9%	168.38	167.33	171.4	171.0	168.6	168.1	168.6	168.1	0.8	1.0	0.8	1.0	--
O1000	O0900	10	18'' DIA	16	2.9%	168.98	168.68	171.9	171.4	169.2	168.6	169.2	168.6	0.8	1.0	0.8	1.0	--
O1100	O1000	39	18'' DIA	20	4.3%	170.85	169.18	173.1	171.9	171.0	169.2	171.1	169.2	0.8	1.0	0.8	1.0	--
O1300	O0711	278	12'' DIA	11	9.1%	195.68	170.37	198.7	172.5	201.2	178.9	202.1	179.3	10.3	10.7	10.5	10.9	Existing 10-yr
O1400	O1300	9	15'' DIA	7	1.1%	195.98	195.88	198.2	198.7	201.4	201.2	202.3	202.1	9.9	10.0	9.9	10.2	Existing 10-yr
O1500	O1400	26	15'' DIA	18	7.5%	197.99	196.03	200.4	198.2	201.9	201.4	202.9	202.3	9.7	9.9	9.8	10.1	Existing 10-yr
O1600	O1500	105	12'' DIA	9	5.7%	204.00	197.99	206.5	200.4	209.3	201.9	210.5	202.9	9.5	9.9	9.7	10.0	Existing 10-yr
O1700	O1600	88	12'' DIA	9	6.5%	209.66	204.00	212.2	206.5	215.5	209.3	216.9	210.5	9.5	9.8	9.7	10.0	Existing 10-yr
O1800	O1700	143	12'' DIA	4	1.4%	211.62	209.66	214.1	212.2	225.6	215.5	227.4	216.9	10.0	10.4	10.3	10.8	Existing 10-yr
O1810	O1800	263	12'' DIA	7	3.4%	220.76	211.77	222.8	214.1	226.4	225.6	228.2	227.4	3.1	3.6	3.1	3.7	Existing 10-yr
O1900	O1800	373	15'' DIA	5	0.5%	213.49	211.62	217.6	214.1	229.1	225.6	231.1	227.4	7.0	7.2	7.1	7.4	Existing 10-yr
O1910	O1900	44	15'' DIA	3	0.3%	214.05	213.94	217.6	217.6	229.3	229.1	231.2	231.1	5.4	5.5	5.5	5.6	Existing 10-yr
O1911	O1910	157	12'' DIA	9	5.4%	222.72	214.18	224.7	217.6	229.3	229.3	231.2	231.2	0.6	0.7	0.6	0.6	Existing 10-yr

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
01920	01910	46	15" DIA	3	0.3%	214.18	214.05	216.4	217.6	229.4	229.3	231.4	231.2	4.7	4.9	4.8	4.9	Existing 10-yr
01921	01920	41	12" DIA	4	1.1%	214.82	214.38	216.5	216.4	229.4	229.4	231.4	231.4	1.4	1.7	1.7	1.9	Existing 10-yr
01922	01921	85	12" DIA	7	3.6%	218.07	214.97	221.0	216.5	229.5	229.4	231.5	231.4	1.7	2.0	1.9	2.2	Existing 10-yr
01930	01920	294	12" DIA	3	0.5%	215.81	214.33	218.1	216.4	231.4	229.4	233.4	231.4	3.7	3.8	3.7	3.9	Existing 10-yr
01931	01930	189	12" DIA	1	0.0%	215.90	215.81	216.9	218.1	231.7	231.4	233.7	233.4	3.8	5.0	4.0	5.2	Existing 10-yr
01932	01931	223	1' H, 1'BW, 2 SS Channel	2	0.0%	215.99	215.90	217.0	216.9	244.4	231.7	247.1	233.7	4.6	5.3	4.7	5.5	Existing 10-yr
01933	01932	232	12" DIA	2	0.4%	216.88	215.99	218.2	217.0	244.5	244.4	247.2	247.1	2.3	2.7	2.5	2.9	Existing 10-yr
01940	01930	247	12" DIA	4	1.3%	219.57	216.41	223.1	218.1	231.9	231.4	234.0	233.4	3.3	3.7	3.6	4.1	Existing 10-yr
01950	01940	280	12" DIA	4	1.1%	222.53	219.59	226.6	223.1	232.0	231.9	234.1	234.0	1.6	2.1	1.9	2.4	Existing 10-yr
01960	01950	84	12" DIA	6	3.2%	225.18	222.53	227.4	226.6	232.1	232.0	234.2	234.1	1.6	1.9	2.0	2.3	Existing 10-yr
01970	01960	146	12" DIA	5	2.1%	228.49	225.48	230.0	227.4	232.1	232.1	234.2	234.2	1.6	1.9	1.9	2.2	Existing 10-yr
02000	01900	283	12" DIA	6	3.0%	222.84	214.29	225.2	217.6	231.1	229.1	233.1	231.1	5.4	5.9	5.7	6.0	Existing 10-yr
02100	02000	29	12" DIA	6	2.7%	223.77	222.98	226.0	225.2	231.3	231.1	233.2	233.1	3.2	3.7	3.7	4.0	Existing 10-yr
02110	02100	143	12" DIA	9	5.6%	231.80	223.87	234.1	226.0	232.2	231.3	233.6	233.2	2.3	2.7	2.5	3.0	--
02120	02110	200	8.04" DIA	3	8.1%	248.35	232.11	249.6	234.1	248.8	232.2	248.8	233.6	2.3	2.7	2.5	3.0	--
02130	02120	102	8.04" DIA	3	7.1%	255.65	248.42	256.9	249.6	256.1	248.8	256.1	248.8	2.3	2.7	2.5	3.0	--
02140	02130	47	8.04" DIA	4	9.8%	260.27	255.70	260.9	256.9	260.6	256.1	260.7	256.1	2.3	2.7	2.5	3.0	--
02150	02140	343	0.67' H, 1'BW, 1 SS Channel	8	12.1%	301.54	260.27	303.5	260.9	301.9	260.6	301.9	260.7	2.3	2.7	2.5	3.0	--
02160	02150	104	12" DIA	3	0.5%	302.06	301.54	304.4	303.5	302.6	301.9	302.6	301.9	1.4	1.7	1.7	1.9	--
02170	02160	518	12" DIA	2	0.4%	304.00	302.06	306.5	304.4	304.6	302.6	304.7	302.6	1.5	1.7	1.7	1.9	--
02200	02100	388	12" DIA	4	1.0%	227.56	223.87	231.2	226.0	231.3	231.3	233.3	233.2	1.0	1.5	1.9	1.8	Existing 10-yr
02300	02200	127	12" DIA	3	0.7%	228.52	227.66	232.3	231.2	231.6	231.3	233.3	233.3	0.9	1.1	1.2	1.1	Future 10-yr
BASIN P																		
Outfall	P0100					133.77		136.8		134.2		134.2		3.1	3.7	3.6	4.2	
P0110	P0100	76	12" DIA	12	10.6%	141.77	133.77	144.9	136.8	142.0	134.2	142.0	134.2	1.0	1.2	1.0	1.2	--
P0200	P0100	105	12" DIA	4	1.0%	134.84	133.77	136.5	136.8	135.4	134.2	135.5	134.2	2.2	2.6	2.5	3.0	--
P0400	P0200	86	12" DIA	3	0.7%	135.54	134.94	139.9	136.5	136.2	135.4	136.2	135.5	2.2	2.6	2.5	3.0	--
P0500	P0400	59	12" DIA	4	1.5%	136.51	135.59	138.4	139.9	137.0	136.2	137.1	136.2	2.2	2.6	2.5	3.0	--
P0600	P0500	32	12" DIA	4	1.6%	137.01	136.51	139.6	138.4	137.4	137.0	137.4	137.1	1.2	1.4	1.4	1.6	--
BASIN Q																		
Drains to Clackamas County Stormdrain																		
Q0200	Q0100	32	12" DIA	15	17.9%	216.99	211.38	219.9	215.2	217.3	212.2	217.4	212.2	4.0	4.7	4.1	4.9	--
Q0300	Q0200	241	12" DIA	15	18.1%	260.05	217.09	265.5	219.9	260.3	217.3	260.3	217.4	2.7	3.1	2.7	3.1	--
Q0400	Q0300	300	12" DIA	9	6.1%	278.58	260.21	282.1	265.5	279.0	260.3	279.0	260.3	2.7	3.1	2.7	3.1	--
BASIN R																		
Drains to Clackamas County Stormdrain																		
R0110	R0100	69	12" DIA	6	2.4%	281.05	279.39	283.3	282.2	281.3	279.6	281.3	279.6	0.9	1.0	0.9	1.0	--
R0120	R0110	121	12" DIA	4	1.1%	282.60	281.25	285.4	283.3	282.9	281.3	282.9	281.3	0.9	1.0	0.9	1.0	--
R0200	R0100	241	12" DIA	6	2.9%	286.27	279.19	288.8	282.2	286.6	279.6	286.6	279.6	1.6	1.9	1.7	1.9	--
BASIN S																		
Outfall	S0100					274.19		275.7		274.2		274.2		5.4	6.4	5.6	6.6	
S0200	S0100	44	12" DIA	11	8.2%	278.27	274.69	284.9	275.7	278.8	274.2	278.8	274.2	5.4	6.4	5.6	6.6	--
S0300	S0200	67	18" DIA	16	8.2%	283.78	278.27	293.5	284.9	284.4	278.8	284.4	278.8	5.4	6.4	5.6	6.6	--

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
S0400	S0300	80	18'' DIA	17	2.5%	286.08	284.03	291.9	293.5	286.7	284.4	286.7	284.4	5.4	6.4	5.6	6.6	--
S0500	S0400	14	18'' DIA	23	4.7%	286.86	286.18	292.1	291.9	287.4	286.7	287.4	286.7	5.4	6.4	5.6	6.6	--
S0510	S0500	49	12'' DIA	7	3.9%	288.86	286.96	292.6	292.1	289.1	287.4	289.2	287.4	1.2	1.5	1.3	1.5	--
S0520	S0510	161	12'' DIA	6	3.3%	294.37	289.06	300.0	292.6	294.7	289.1	294.7	289.2	1.2	1.5	1.3	1.5	--
S0530	S0520	66	12'' DIA	9	5.9%	298.32	294.47	301.8	300.0	298.6	294.7	298.6	294.7	1.2	1.5	1.3	1.5	--
S0600	S0500	138	12'' DIA	4	1.1%	288.44	286.91	291.1	292.1	289.1	287.4	289.1	287.4	3.0	3.5	3.1	3.6	--
S0700	S0600	338	12'' DIA	5	1.8%	294.45	288.44	297.7	291.1	295.0	289.1	295.0	289.1	3.0	3.5	3.1	3.6	--
S0800	S0700	32	12'' DIA	6	2.5%	295.24	294.45	298.1	297.7	295.8	295.0	295.8	295.0	3.0	3.5	3.1	3.6	--
S0900	S0800	39	12'' DIA	5	1.8%	296.05	295.34	299.0	298.1	296.6	295.8	296.6	295.8	3.0	3.5	3.1	3.6	--
S1000	S0900	76	12'' DIA	2	0.3%	296.44	296.25	299.3	299.0	297.1	296.6	297.1	296.6	1.6	1.9	1.7	1.9	--
S1100	S1000	116	12'' DIA	5	2.0%	299.40	297.09	300.8	299.3	299.8	297.1	299.8	297.1	1.6	1.9	1.7	1.9	--
BASIN T																		
Drains to Clackamas County Stormdrain																		
T0100	T0000	282	12'' DIA	13	13.5%	291.85	254.00	294.1	257.0	292.2	255.0	292.2	255.0	3.3	4.0	3.9	4.6	--
T0200	T0100	473	12'' DIA	8	4.4%	312.55	291.95	315.3	294.1	312.9	292.2	312.9	292.2	2.1	2.5	2.4	2.9	--
T0300	T0200	46	12'' DIA	7	4.2%	314.66	312.75	316.4	315.3	315.0	312.9	315.1	312.9	2.1	2.5	2.4	2.9	--
T0400	T0300	116	12'' DIA	6	2.9%	318.27	314.86	320.1	316.4	318.7	315.0	318.7	315.1	2.1	2.5	2.4	2.9	--
T0500	T0400	276	12'' DIA	3	0.5%	319.58	318.32	321.7	320.1	320.3	318.7	320.4	318.7	2.1	2.5	2.5	2.9	--
T0600	T0500	140	12'' DIA	4	1.4%	321.64	319.68	323.6	321.7	322.0	320.3	322.0	320.4	1.2	1.4	1.3	1.5	--
T0700	T0600	181	12'' DIA	5	1.7%	324.83	321.84	326.3	323.6	325.2	322.0	325.2	322.0	1.2	1.4	1.3	1.5	--
T0800	T0700	97	12'' DIA	3	0.5%	325.44	324.93	327.3	326.3	325.9	325.2	325.9	325.2	1.2	1.4	1.3	1.5	--
T0900	T0800	170	12'' DIA	3	0.5%	326.32	325.44	328.2	327.3	326.8	325.9	326.8	325.9	1.2	1.4	1.3	1.5	--
BASIN U																		
Drains to Clackamas County Stormdrain																		
U0200	U0100	30	12'' DIA	5	2.0%	290.90	290.29	292.7	292.5	291.3	290.8	291.3	290.8	1.6	1.9	1.7	2.0	--
U0300	U0200	125	9.96'' DIA	1	0.2%	291.30	291.00	293.1	292.7	292.4	291.3	292.6	291.3	1.6	1.9	1.7	2.0	--
BASIN V																		
Outfall	V0101					277.86		278.9		278.0		278.0		0.5	0.6	0.6	0.7	
V0102	V0101	122.8	12'' DIA	10.6	8.5%	288.29	277.86	292.5	278.9	288.4	278.0	288.5	278.0	0.5	0.6	0.6	0.7	--
Outfall	V0110					267.99		270.0		268.3		268.3		2.2	4.2	3.1	5.1	
Outfall	V0100					260.30		262.1		261.1		261.1		8.9	9.0	9.0	9.1	
V0200	V0100	44	21'' DIA	22	6.6%	263.22	260.30	273.6	262.1	264.0	261.1	264.0	261.1	8.9	9.0	9.0	9.1	--
V0210	V0200	387	12'' DIA	8	5.1%	282.85	263.22	285.6	273.6	283.0	264.0	283.0	264.0	0.2	0.2	0.2	0.3	--
V0300	V0200	80	12'' DIA	9	5.9%	268.70	264.02	276.2	273.6	269.5	264.0	269.5	264.0	8.7	8.8	8.7	8.8	--
V0400	V0300	78	12'' DIA	8	4.6%	272.40	268.80	279.8	276.2	274.1	269.5	274.1	269.5	8.69*	8.81*	8.73*	8.84*	--
V0400A	V0110	163	24'' DIA	42	3.6%	273.80	267.99	279.8	270.0	274.1	268.3	274.1	268.3	1.67*	3.56*	2.3*	4.14*	--
V0500	V0400	66	15'' DIA	8	1.4%	273.32	272.40	278.0	279.8	275.6	274.1	275.9	274.1	10.3	12.3	11.0	12.9	--
V0510	V0500	33	12'' DIA	7	3.4%	274.50	273.37	278.3	278.0	275.7	275.6	276.0	275.9	1.6	1.9	1.7	2.0	--
V0520	V0510	76	12'' DIA	12	10.5%	282.44	274.50	287.1	278.3	282.7	275.7	282.7	276.0	1.6	1.9	1.7	2.0	--
V0530	V0520	105	12'' DIA	11	10.5%	293.35	282.44	297.5	287.1	293.6	282.7	293.6	282.7	1.0	1.2	1.0	1.2	--
V0600	V0500	63	15'' DIA	13	4.0%	278.21	275.67	280.7	278.0	278.9	275.6	279.0	275.9	8.7	10.4	9.3	10.9	--
V0700	V0600	247	15'' DIA	10	2.4%	284.14	278.21	291.8	280.7	285.0	278.9	285.0	279.0	8.3	9.9	8.8	10.3	--
V0710	V0700	30	12'' DIA	7	3.9%	286.75	285.59	291.9	291.8	287.1	285.0	287.2	285.0	2.2	2.6	2.3	2.8	--

Table B-2. Hydraulic Model Parameters and Results

Up and downstream model node names		Length (ft)	Size/Type H = Height, BW = Bottom width, SS = Side slope (H:V)	Capacity (cfs)	Slope (%)	Invert elevation (ft)		Ground elevation (ft)		Existing 10 yr max water surface elevation (ft)		Future 10 yr max water surface elevation (ft)		Peak flow values at upstream node (cfs)*				When flooding (Max WSE > ground elevation)
Name/US node	DS node					US	DS	US	DS	US	DS	US	DS	Existing 10 yr	Existing 25 yr	Future 10 yr	Future 25 yr	
V0720	V0710	285	12'' DIA	7	3.7%	297.64	287.00	301.1	291.9	298.0	287.1	298.0	287.2	1.8	2.1	1.9	2.2	--
V1000	V0700	265	15'' DIA	6	0.8%	286.39	284.14	291.0	291.8	287.4	285.0	287.4	285.0	5.9	7.0	6.2	7.3	--
V1100	V1000	243	12'' DIA	4	1.1%	289.12	286.44	295.5	291.0	290.4	287.4	290.7	287.4	4.3	5.1	4.4	5.2	--
V1200	V1100	109	12'' DIA	4	1.2%	290.53	289.22	293.9	295.5	291.1	290.4	291.3	290.7	2.7	3.2	2.9	3.4	--
V1400	V1200	363	12'' DIA	5	1.8%	297.03	290.58	299.4	293.9	297.4	291.1	297.4	291.3	1.1	1.3	1.2	1.4	--
V1500	V1400	82	12'' DIA	2	0.3%	297.24	297.03	299.9	299.4	297.8	297.4	297.8	297.4	1.1	1.3	1.2	1.4	--
V1600	V1500	71	12'' DIA	6	2.6%	299.20	297.34	301.3	299.9	299.5	297.8	299.5	297.8	1.1	1.3	1.2	1.4	--
Outfall	V5000					247.23		248.2		247.4		247.4		1.7	2.0	2.0	2.4	
V5100	V5000	173	12'' DIA	16	20.3%	281.61	247.23	288.8	248.2	281.8	247.4	281.8	247.4	1.6	1.8	1.6	1.9	--
V5200	V5100	211	12'' DIA	8	4.9%	296.27	285.96	300.2	288.8	296.6	284.9	296.6	284.9	1.6	1.8	1.6	1.9	--
V5300	V5200	312	12'' DIA	7	3.5%	307.43	296.47	310.0	300.2	307.7	296.6	307.8	296.6	1.4	1.7	1.5	1.7	--
BASIN X																		
Outfall	X0100					72.00		74.0		73.3		73.3		11.4	11.4	11.4	11.4	
X0100	X0110	93	2' H, 3'BW, 1 SS Channel	42	1.1%	72.00	71.00	74.0	73.0	73.3	73.1	73.3	73.1	11.4	11.4	11.4	11.4	--
X0110	X0120	47	24'' DIA	15	0.4%	71.00	70.80	73.0	72.8	73.1	72.8	73.1	72.8	23.1	23.1	23.1	23.1	Existing 10-yr
BASIN Z																		
Outfall	Z0700					52.72		53.7		53.7		53.7		0.8	1.0	0.8	1.0	
Z0710	Z0700	75	12'' DIA	9	5.4%	56.77	52.72	59.6	53.7	57.0	53.7	57.0	53.7	0.8	1.0	0.8	1.0	--
Outfall	Z2000					32.05		33.1		33.1		33.1		1.2	1.2	1.2	1.2	
Z2010	Z2000	86	12'' DIA	3	0.5%	32.48	32.05	35.2	33.1	33.4	33.1	33.4	33.1	1.2	1.2	1.2	1.2	--

Maximum flow values were modified in instances where two pipes share the same US node. In these cases maximum flow is provided for the conduit. All other maximum flow values pertain to the US node.

Appendix C: CIP Cost Summaries

City of Gladstone - Stormwater Master Plan
Capital Improvement Project
Preliminary Engineering Unit Cost

ITEM	UNIT	UNIT COST ($)
Water Quality Facility Installation		
General Earthwork & Off-site removal	CY	$18
Clearing Brush	AC	$1,850
Clear and Grub brush including stumps	AC	$6,500
Clear and Grub brush including stumps - 50' Width	LF	$80
Amended Soils and Mulch	CY	$40
Jute Matting, Biodegradeable	SY	$10
Rip-Rap, Class 50	CY	$60
Drain Rock	CY	$40
Pond Outflow Control Structure	EA	$6,000
Water Quality Facility Plantings	SF	$3
Water Quality Facility Plantings with Trees	SF	$6
Rain Garden	SF	$25
Stormwater Planter	SF	$37
Beehive Overflow	EA	$1,500
Structure Installation		
Precast Concrete Manhole (48", 0-8' deep)	EA	$3,500
Precast Concrete Manhole (48", 9-12' deep)	EA	$5,100
Precast Concrete Manhole (48", 13-20' deep)	EA	$8,300
Precast Concrete Manhole (60", 0-8' deep)	EA	$5,000
Precast Concrete Manhole (60", 9-12' deep)	EA	$9,000
Precast Concrete Manhole (72", 0-8' deep)	EA	$7,000
Precast Concrete Manhole (72", 9-12' deep)	EA	$13,000
Curb Inlet	EA	$1,900
Concrete Ditch Inlet (0-8' deep)	EA	$2,000
Catch Basin	EA	$2,300
Connection to Existing Structure	EA	$1,000
Abandon Existing Manhole	EA	$440
Abandon Existing Pipe, no excavation (15-18")	FT	$22
Abandon Existing Pipe, no excavation (21"-24")	FT	$27
Abandon Existing Pipe, no excavation (27"-36")	FT	$36
Flow bypass	EA	$10,000
Outfall Improvements	EA	$5,000
Concrete Fill - Catch basin disconnection	CY	$140
Drywell (48", 20-25' deep)	EA	$10,000
Restoration/ Resurfacing		
Riparian Planting	SF	$3
4-foot Chain Link Fence	LF	$21
Hydroseed	AC	$2,300
Stream Bed Gravel	CY	$40
Fish Removal	EA	$3,000
Project Totals		
Project Sub-Total		
Mobilization/Demobilization (10%)	LS	10%
Erosion Control (2%)	LS	2%
Construction Contingency (30%)	LS	30%
Construction Cost Estimate		
Engineering and Permitting (%) *	LS	Varies by project (20-40%)
Construction Administration (%)	LS	5%
Total Project Engineering and Construction Cost		

* Engineering and permitting costs will be documented in each project's write-up.

City of Gladstone - Stormwater Master Plan Costs
PIPE INSTALLATION with Asphalt

Storm Drain Pipe Construction Cost per Linear Foot																
	Diameter (inches)															
Cover Depth (feet)	12	12-RCP	18	18-RCP	24	24-RCP	30	30-RCP	36	36-RCP	42	42-RCP	48	48-RCP	54	60
2-5	$78	$97	$122	$141	$161	$194	$209	$297	$259	$377	$316	$415	$370	$490	$470	$556
5-10	$107	$125	$162	$181	$213	$246	$273	$361	$336	$453	$404	$503	$470	$590	$582	$680
10-15	$135	$153	$202	$221	$265	$298	$337	$425	$412	$529	$492	$591	$571	$690	$695	$805
15-20	$163	$181	$242	$261	$317	$350	$401	$489	$488	$606	$580	$679	$671	$790	$807	$929

Breakdown of Linear Foot Cost																
Depth of Cover (ft)	12	12	18	18	24	24	30	30	36	36	42	42	48	48	54	60
Sub Task																
Pipe + Bed (ft)	2	2	2.0	2.0	2.5	2.5	3.0	3.0	3.5	3.5	4.0	4.0	4.5	4.5	5.0	5.5
Width (ft)	2	2	3	3	4	4	5	5	6	6	7	7	8	8	9	10
Bedding (ft)	0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.4
Shoring (lf)	$ 4	$ 4	$ 4	$ 4	$ 4	$ 4	$ 4	$ 4	$ 4	$ 4	$ 4	$ 4	$ 4	$ 4	$ 4	$ 4
Sawcutting and Asphalt Removal (lf)	$ 17	$ 17	$ 24	$ 24	$ 31	$ 31	$ 38	$ 38	$ 45	$ 45	$ 52	$ 52	$ 59	$ 59	$ 66	$ 73
Trench Excavation (CY)	$ 25	$ 25	$ 25	$ 25	$ 25	$ 25	$ 25	$ 25	$ 25	$ 25	$ 25	$ 25	$ 25	$ 25	$ 25	$ 25
Trench Backfill (CY)	$ 40	$ 40	$ 40	$ 40	$ 40	$ 40	$ 40	$ 40	$ 40	$ 40	$ 40	$ 40	$ 40	$ 40	$ 40	$ 40
HDPE Piping unless noted concrete (lf)	$ 13	$ 31	$ 23	$ 42	$ 27	$ 60	$ 37	$ 125	$ 48	$ 165	$ 61	$ 160	$ 71	$ 190	$ 123	$ 159
Asphalt Restoration (lf)	$ 13	$ 13	$ 20	$ 20	$ 27	$ 27	$ 34	$ 34	$ 40	$ 40	$ 47	$ 47	$ 54	$ 54	$ 60	$ 67
Cover (CY)																
2-5	0.5	0.5	0.8	0.8	1.1	1.1	1.5	1.5	1.9	1.9	2.3	2.3	2.8	2.8	3.3	3.9
5-10	0.9	0.9	1.3	1.3	1.9	1.9	2.4	2.4	3.0	3.0	3.6	3.6	4.3	4.3	5.0	5.7
10-15	1.2	1.2	1.9	1.9	2.6	2.6	3.3	3.3	4.1	4.1	4.9	4.9	5.8	5.8	6.7	7.6
15-20	1.6	1.6	2.4	2.4	3.3	3.3	4.3	4.3	5.2	5.2	6.2	6.2	7.3	7.3	8.3	9.4
Cost ($/LF)																
2-5	$78	$97	$122	$141	$161	$194	$209	$297	$259	$377	$316	$415	$370	$490	$470	$556
5-10	$107	$125	$162	$181	$213	$246	$273	$361	$336	$453	$404	$503	$470	$590	$582	$680
10-15	$135	$153	$202	$221	$265	$298	$337	$425	$412	$529	$492	$591	$571	$690	$695	$805
15-20	$163	$181	$242	$261	$317	$350	$401	$489	$488	$606	$580	$679	$671	$790	$807	$929

CIP A-1. Rinearson Creek Stream Enhancement

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
Channel Maintenance/Clearing				
Clear and Grub brush including stumps - 50' Width	500	LF	$ 80	$ 40,000
Flow bypass	1	EA	$ 10,000	$ 10,000
Fish Removal	1	EA	$ 3,000	$ 3,000
Rip-Rap, Class 50	50	CY	$ 60	$ 3,000
Olson Wetlands Improvements				
General Earthwork & Off-site removal	1,200	CY	$ 18	$ 21,600
Amended Soils and Mulch	300	CY	$ 40	$ 12,000
Water Quality Facility Plantings with Trees	7,500	SF	$ 6	$ 45,000
Water Quality Facility Plantings	10,500	SF	$ 3	$ 31,500
Flow bypass	1	EA	$ 10,000	$ 10,000
Jute Matting, Biodegradeable	2,000	SY	$ 10	$ 20,000
Capital Expense Sub-Total				$ 196,100
Mobilization/Demobilization	10%	LS		$ 19,610
Traffic Control/Utility Relocation	0%	LS		$ -
Erosion Control	10%	LS		$ 19,610
Construction Cost Sub-Total				$ 235,320
Construction Contingency	30%	LS		$ 70,596
Capital Expense Total				$ 305,916
Administrative Expenses				
Engineering and Permitting	30%	LS		$ 91,775
Construction & General Administration	5%	LS		$ 15,296
Administrative Expense Total				$ 107,071
Capital Implementation Cost Total				**$ 412,987**

CIP A-2. Portland Avenue Bypass and Upstream Improvements

A-2.1 Portland Avenue High Flow Bypass

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
HDPE Pipeline (48", 5-10' Deep)	1,400	LF	$ 470	$ 658,503
HDPE Pipeline (48", 10-15' Deep)	1,250	LF	$ 571	$ 713,319
RCP Outfall Pipeline (48", 10-15' Deep)	500	LF	$ 590	$ 294,930
Outfall Improvement - On Clackamas	1	EA	$ 50,000	$ 50,000
Precast Concrete Manhole (72", 9-12' deep)	13	EA	$ 13,000	$ 169,000
Capital Expense Sub-Total				$ 1,885,752
Mobilization/Demobilization	10%	LS		$ 188,575
Traffic Control/Utility Relocation	2%	LS		$ 37,715
Erosion Control	2%	LS		$ 37,715
Construction Cost Sub-Total				$ 2,149,757
Construction Contingency	30%	LS		$ 644,927
Capital Expense Total				$ 2,794,685
Administrative Expenses				
Engineering and Permitting	30%	LS		$ 838,405
Construction & General Administration	5%	LS		$ 139,734
Administrative Expense Total				$ 978,140
Capital Implementation Cost Total				**$ 3,772,824**

A-2.2 Sanitary Sewer Disconnect

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
Retrofit Combined Manhole C0500	1	LS	$ 5,000	$ 5,000
Catch Basin	12	EA	$ 2,300	$ 27,600
HDPE Inlet Leads (12", 2-5' Deep)	120	LF	$ 78	$ 9,414
Capital Expense Sub-Total				$ 42,014
Mobilization/Demobilization	10%	LS		$ 4,201
Traffic Control/Utility Relocation	2%	LS		$ 840
Erosion Control	2%	LS		$ 840
Construction Cost Sub-Total				$ 47,895
Construction Contingency	30%	LS		$ 14,369
Capital Expense Total				$ 62,264
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 12,453
Construction & General Administration	5%	LS		$ 3,113
Administrative Expense Total				$ 15,566
Capital Implementation Cost Total				**$ 77,830**

CIP A-2. Portland Avenue Bypass and Upstream Improvements

A-2.3 Portland Avenue Pipe Replacement/Realignment North of Jersey

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
RCP Pipeline (24", 2-5' Deep)	416	LF	$ 194	$ 80,713
RCP Pipeline (36", 5-10' Deep)	153	LF	$ 453	$ 69,340
HDPE Pipeline (42", 5-10' Deep)	1,168	LF	$ 404	$ 471,669
Catch Basin	8	EA	$ 2,300	$ 18,400
RCP Inlet Leads (12", 2-5' Deep)	80	LF	$ 78	$ 6,276
Precast Concrete Manhole (60", 0-8' deep)	2	EA	$ 5,000	$ 10,000
Precast Concrete Manhole (72", 9-12' deep)	5	EA	$ 13,000	$ 65,000
Capital Expense Sub-Total				$ 721,397
Mobilization/Demobilization	10%	LS		$ 72,140
Traffic Control/Utility Relocation	2%	LS		$ 14,428
Erosion Control	2%	LS		$ 14,428
Construction Cost Sub-Total				$ 822,393
Construction Contingency	30%	LS		$ 246,718
Capital Expense Total				$ 1,069,111
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 213,822
Construction & General Administration	5%	LS		$ 53,456
Administrative Expense Total				$ 267,278
Capital Implementation Cost Total				**$ 1,336,388**

A-2.4 Duniway to Barclay Pipe Replacement/Realignment

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
RCP Pipeline (12", 2-5' Deep)	116	LF	$ 97	$ 11,217
HDPE Pipeline (18", 5-10' Deep)	252	LF	$ 162	$ 40,765
HDPE Pipeline (24", 5-10' Deep)	692	LF	$ 213	$ 147,465
HDPE Pipeline (30", 5-10' Deep)	385	LF	$ 273	$ 105,098
Precast Concrete Manhole (48", 0-8' deep)	6	EA	$ 3,500	$ 21,000
Concrete Ditch Inlet (0-8' deep)	1	EA	$ 2,000	$ 2,000
Capital Expense Sub-Total				$ 327,545
Mobilization/Demobilization	10%	LS		$ 32,754
Traffic Control/Utility Relocation	2%	LS		$ 6,551
Erosion Control	2%	LS		$ 6,551
Construction Cost Sub-Total				$ 373,401
Construction Contingency	30%	LS		$ 112,020
Capital Expense Total				$ 485,421
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 97,084
Construction & General Administration	5%	LS		$ 24,271
Administrative Expense Total				$ 121,355
Capital Implementation Cost Total				**$ 606,777**

CIP A-3. High School Stormdrain Improvements and Detention Pond				
Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
RCP Pipeline (30", 2-5' Deep)	718	LF	$ 297	$ 213,100
Precast Concrete Manhole (48", 0-8' deep)	4	EA	$ 3,500	$ 14,000
Pond Outflow Control Structure	1	EA	$ 6,000	$ 6,000
General Earthwork - Excavation	5,815	CY	$ 18	$ 104,667
Amended Soils and Mulch	1,336	CY	$ 40	$ 53,425
Drain Rock	1,336	CY	$ 40	$ 53,425
Water Quality Facility Plantings	7,770	SF	$ 3	$ 23,311
4-foot Chain Link Fence	650	LF	$ 21	$ 13,650
RCP Pipeline (36", 2-5' Deep)	740	LF	$ 377	$ 278,862
Precast Concrete Manhole (60", 0-8' deep)	4	EA	$ 5,000	$ 20,000
RCP Pipeline (48", 2-5' Deep)	389	LF	$ 490	$ 190,440
Precast Concrete Manhole (72", 0-8' deep)	3	EA	$ 7,000	$ 21,000
Capital Expense Sub-Total				$ 991,879
Mobilization/Demobilization	10%	LS		$ 99,188
Traffic Control/Utility Relocation	2%	LS		$ 19,838
Erosion Control	2%	LS		$ 19,838
Construction Cost Sub-Total				$ 1,130,742
Construction Contingency	30%	LS		$ 339,223
Capital Expense Total				$ 1,469,964
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 293,993
Construction & General Administration	5%	LS		$ 73,498
Administrative Expense Total				$ 367,491
Capital Implementation Cost Total				**$ 1,837,456**

CIP A-4. High School Rain Garden

General Earthwork & Off-site removal	15	CY	$	18	$	270
Amended Soils and Mulch	15	CY	$	40	$	600
Water Quality Facility Plantings	200	SF	$	3	$	600
Rain Garden	200	SF	$	25	$	5,000
Capital Expense Sub-Total					$	6,470
Mobilization/Demobilization	10%	LS			$	647
Erosion Control	2%	LS			$	129
Construction Cost Sub-Total					$	7,246
Construction Contingency	30%	LS			$	2,174
Capital Expense Total					$	9,420
Administrative Expenses						
Engineering and Permitting	20%	LS			$	1,884
Construction & General Administration	5%	LS			$	471
Administrative Expense Total					$	2,355
Capital Implementation Cost Total					**$**	**11,775**

CIP A-5. Tyron Rain Garden				
Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
General Earthwork & Off-site removal	250	CY	$ 18	$ 4,500
Amended Soils and Mulch	250	CY	$ 40	$ 10,000
Water Quality Facility Plantings	3,330	SF	$ 3	$ 9,990
Rain Garden	3,330	SF	$ 25	$ 83,250
HDPE Leads (12", 2-5' Deep)	100	LF	$ 97	$ 9,670
Capital Expense Sub-Total				$ 117,410
Mobilization/Demobilization	10%	LS		$ 11,741
Traffic Control/Utility Relocation	2%	LS		$ 2,348
Erosion Control	2%	LS		$ 2,348
Construction Cost Sub-Total				$ 133,847
Construction Contingency	30%	LS		$ 40,154
Capital Expense Total				$ 174,001
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 34,800
Construction & General Administration	5%	LS		$ 8,700
Administrative Expense Total				$ 43,500
Capital Implementation Cost Total				**$ 217,501**

CIP A-6. Glen Echo Pipeline Realignment

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
RCP Pipeline (12", 2-5' Deep)	203	LF	$ 97	$ 19,629
HDPE Pipeline (24", 2-5' Deep)	625	LF	$ 161	$ 100,639
Catch Basin	2	EA	$ 2,300	$ 4,600
HDPE Inlet Leads (12", 2-5' Deep)	20	LF	$ 78	$ 1,569
Precast Concrete Manhole (48", 0-8' deep)	3	EA	$ 3,500	$ 10,500
Abandon Existing Pipe, no excavation (15-18")	620	FT	$ 22	$ 13,330
Abandon Existing Manhole	3	EA	$ 440	$ 1,320
Capital Expense Sub-Total				$ 151,587
Mobilization/Demobilization	10%	LS		$ 15,159
Traffic Control/Utility Relocation	2%	LS		$ 3,032
Erosion Control	2%	LS		$ 3,032
Construction Cost Sub-Total				$ 172,809
Construction Contingency	30%	LS		$ 51,843
Capital Expense Total				$ 224,652
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 44,930
Construction & General Administration	5%	LS		$ 11,233
Administrative Expense Total				$ 56,163
Capital Implementation Cost Total				**$ 280,815**

CIP A-7. Meldrum Bar Bioswale

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
420 LF of Bioswale and 340 LF of Channel Restoration				
Abandon Existing Pipe, no excavation (15-18")	402	FT	$ 22	$ 8,643
RCP Pipeline (18", 2-5' Deep)	15	LF	$ 141	$ 2,110
Concrete Ditch Inlet (0-8' deep)	1	EA	$ 2,000	$ 2,000
Precast Concrete Manhole (48", 0-8' deep)	1	EA	$ 3,500	$ 3,500
General Earthwork & Off-site removal	917	CY	$ 18	$ 16,500
Amended Soils and Mulch	324	CY	$ 40	$ 12,971
Jute Matting, Biodegradeable	2,151	SY	$ 10	$ 21,507
Stream Bed Gravel	103	CY	$ 40	$ 4,102
Water Quality Facility Plantings	17,511	SF	$ 3	$ 52,532
Capital Expense Sub-Total				$ 123,864
Mobilization/Demobilization	10%	LS		$ 12,386
Traffic Control/Utility Relocation	2%	LS		$ 2,477
Erosion Control	2%	LS		$ 2,477
Construction Cost Sub-Total				$ 141,205
Construction Contingency	30%	LS		$ 42,361
Capital Expense Total				$ 183,566
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 36,713
Construction & General Administration	5%	LS		$ 9,178
Administrative Expense Total				$ 45,892
Capital Implementation Cost Total				**$ 229,458**

CIP A-8. Riverdale Drainage Improvements

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
Stormwater Planter	1,765	SF	$ 37	$ 65,305
Water Quality Facility Plantings	1,765	SF	$ 3	$ 5,295
Catch Basin	3	EA	$ 2,300	$ 6,900
HDPE inlet Leads (12", 2-5' Deep)	300	LF	$ 97	$ 29,009
Concrete Fill - Catch basin disconnection	9	CY	$ 140	$ 1,260
Drywell (48", 20-25' deep)	3	EA	$ 10,000	$ 30,000
Precast Concrete Manhole (48", 13-20' deep)	1	EA	$ 8,300	$ 8,300
Capital Expense Sub-Total				$ 146,069
Mobilization/Demobilization	10%	LS		$ 14,607
Traffic Control/Utility Relocation	2%	LS		$ 2,921
Erosion Control	5%	LS		$ 7,303
Construction Cost Sub-Total				$ 170,901
Construction Contingency	30%	LS		$ 51,270
Capital Expense Total				$ 222,171
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 44,434
Construction & General Administration	5%	LS		$ 11,109
Administrative Expense Total				$ 55,543
Capital Implementation Cost Total				**$ 277,713**

CIP B-1. Basin B Drainage Improvements				
Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
Gloucester Street				
Stormwater Planter	745	SF	$ 37	$ 27,565
Water Quality Facility Plantings	745	SF	$ 3	$ 2,235
Beehive Overflow	10	EA	$ 1,500	$ 15,000
HDPE Leads (12", 2-5' Deep)	250	LF	$ 97	$ 24,174
Connection to Existing Structure	10	EA	$ 1,000	$ 10,000
Arlington Street				
Stormwater Planter	1,335	SF	$ 37	$ 49,395
Water Quality Facility Plantings	1,335	SF	$ 3	$ 4,005
Concrete Fill - Catch basin disconnection	6	CY	$ 140	$ 840
Capital Expense Sub-Total				$ 133,214
Mobilization/Demobilization	10%	LS		$ 13,321
Traffic Control/Utility Relocation	2%	LS		$ 2,664
Erosion Control	2%	LS		$ 2,664
Construction Cost Sub-Total				$ 151,864
Construction Contingency	30%	LS		$ 45,559
Capital Expense Total				$ 197,423
Administrative Expenses				
Engineering and Permitting	30%	LS		$ 59,227
Construction & General Administration	5%	LS		$ 9,871
Administrative Expense Total				$ 69,098
Capital Implementation Cost Total				**$ 266,521**

CIP F-1. Caldwell to Hull Pipe Replacement/Realignment

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
RCP Pipeline (24", 2-5' Deep)	1,347	LF	$ 194	$ 261,348
HDPE Inlet Leads (12", 2-5' Deep)	60	LF	$ 78	$ 4,707
Precast Concrete Manhole (48", 0-8' deep)	8	EA	$ 3,500	$ 28,000
Catch Basin	6	EA	$ 2,300	$ 13,800
Capital Expense Sub-Total				$ 307,855
Mobilization/Demobilization	10%	LS		$ 30,785
Traffic Control/Utility Relocation	2%	LS		$ 6,157
Erosion Control	2%	LS		$ 6,157
Construction Cost Sub-Total				$ 350,954
Construction Contingency	30%	LS		$ 105,286
Capital Expense Total				$ 456,241
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 91,248
Construction & General Administration	5%	LS		$ 22,812
Administrative Expense Total				$ 114,060
Capital Implementation Cost Total				**$ 570,301**

CIP H-1. System H Channel Improvement

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
General Earthwork & Off-site removal	185	CY	$ 18	$ 3,330
Clearing Brush	0.5	AC	$ 1,850	$ 925
Riparian Planting	5,000	SF	$ 3	$ 15,000
Capital Expense Sub-Total				$ 19,255
Mobilization/Demobilization	10%	LS		$ 1,926
Traffic Control/Utility Relocation	2%	LS		$ 385
Erosion Control	2%	LS		$ 385
Construction Cost Sub-Total				$ 21,951
Construction Contingency	30%	LS		$ 6,585
Capital Expense Total				$ 28,536
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 5,707
Construction & General Administration	5%	LS		$ 1,427
Administrative Expense Total				$ 7,134
Capital Implementation Cost Total				**$ 35,670**

CIP J-1. Cornell at Landon Pipe Replacement/Realignment

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
HDPE Pipeline (18", 5-10' Deep)	334	LF	$ 162	$ 54,030
HDPE Pipeline (30", 5-10' Deep)	906	LF	$ 273	$ 247,321
Precast Concrete Manhole (48", 0-8' deep)	7	EA	$ 3,500	$ 24,500
Catch Basin	4	EA	$ 2,300	$ 9,200
HDPE Inlet Leads (12", 2-5' Deep)	40	LF	$ 78	$ 3,138
Abandon Existing Pipe, no excavation (15-18")	397	FT	$ 22	$ 8,536
Capital Expense Sub-Total				$ 346,725
Mobilization/Demobilization	10%	LS		$ 34,672
Traffic Control/Utility Relocation	2%	LS		$ 6,934
Erosion Control	2%	LS		$ 6,934
Construction Cost Sub-Total				$ 395,266
Construction Contingency	30%	LS		$ 118,580
Capital Expense Total				$ 513,846
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 102,769
Construction & General Administration	5%	LS		$ 25,692
Administrative Expense Total				$ 128,461
Capital Implementation Cost Total				**$ 642,307**

CIP J-2. Oatfield Road Pipe Replacement

Description	Quantity	Unit		Unit Cost (2013)		2013 Cost
Capital Expenses						
RCP Pipeline (18", 2-5' Deep)	790	LF	$	141	$	111,118
RCP Pipeline (24", 2-5' Deep)	623	LF	$	194	$	120,876
Precast Concrete Manhole (48", 0-8' deep)	7	EA	$	3,500	$	24,500
Capital Expense Sub-Total					$	256,494
Mobilization/Demobilization	10%	LS			$	25,649
Traffic Control/Utility Relocation	2%	LS			$	5,130
Erosion Control	2%	LS			$	5,130
Construction Cost Sub-Total					$	292,403
Construction Contingency	30%	LS			$	87,721
Capital Expense Total					$	380,124
Administrative Expenses						
Engineering and Permitting	20%	LS			$	76,025
Construction & General Administration	5%	LS			$	19,006
Administrative Expense Total					$	95,031
Capital Implementation Cost Total					**$**	**475,155**

CIP M-1. Crownview Drive Pipe Replacement

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
HDPE Pipeline (18", 2-5' Deep)	542	LF	$ 122	$ 65,937
HDPE Inlet Leads (12", 2-5' Deep)	40	LF	$ 78	$ 3,138
Precast Concrete Manhole (48", 0-8' deep)	3	EA	$ 3,500	$ 10,500
Catch Basin	4	EA	$ 2,300	$ 9,200
Capital Expense Sub-Total				$ 88,775
Mobilization/Demobilization	10%	LS		$ 8,878
Traffic Control/Utility Relocation	2%	LS		$ 1,776
Erosion Control	2%	LS		$ 1,776
Construction Cost Sub-Total				$ 101,204
Construction Contingency	30%	LS		$ 30,361
Capital Expense Total				$ 131,565
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 26,313
Construction & General Administration	5%	LS		$ 6,578
Administrative Expense Total				$ 32,891
Capital Implementation Cost Total				**$ 164,456**

CIP N-1. Kraxberger Middle School Bioswale at Webster Road				
Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
Bioswale				
HDPE Pipeline (12", 2-5' Deep)	30	LF	$ 141	$ 4,220
Concrete Ditch Inlet (0-8' deep)	1	EA	$ 2,000	$ 2,000
Precast Concrete Manhole (48", 0-8' deep)	1	EA	$ 3,500	$ 3,500
500 Linear Feet of Open Channel				
General Earthwork & Off-site removal	444	CY	$ 18	$ 8,000
Amended Soils and Mulch	176	CY	$ 40	$ 7,027
Jute Matting, Biodegradeable	1,165	SY	$ 10	$ 11,652
Stream Bed Gravel	56	CY	$ 40	$ 2,222
Water Quality Facility Plantings	9,487	SF	$ 3	$ 28,460
Pipe Replacement				
RCP Pipeline (24", 2-5' Deep)	270	LF	$ 194	$ 52,386
HDPE Inlet Leads (12", 2-5' Deep)	40	LF	$ 78	$ 3,138
Catch Basin	4	EA	$ 2,300	$ 9,200
Precast Concrete Manhole (48", 0-8' deep)	3	EA	$ 3,500	$ 10,500
RCP Pipeline (36", 5-10' Deep)	905	LF	$ 377	$ 341,165
Precast Concrete Manhole (60", 0-8' deep)	5	EA	$ 5,000	$ 25,000
Capital Expense Sub-Total				$ 508,470
Mobilization/Demobilization	10%	LS		$ 50,847
Traffic Control/Utility Relocation	2%	LS		$ 10,169
Erosion Control	2%	LS		$ 10,169
Construction Cost Sub-Total				$ 579,656
Construction Contingency	30%	LS		$ 173,897
Capital Expense Total				$ 753,553
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 150,711
Construction & General Administration	5%	LS		$ 37,678
Administrative Expense Total				$ 188,388
Capital Implementation Cost Total				**$ 941,942**

CIP N-2. System N Inlet Replacement				
Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
HDPE Pipeline & Inlet Leads (12", 2-5' Deep)	530	LF	$ 78	$ 41,577
Curb Inlet	8	EA	$ 1,900	$ 15,200
Precast Concrete Manhole (48", 0-8' deep)	4	EA	$ 3,500	$ 14,000
Connection to Existing Structure	7	EA	$ 1,000	$ 7,000
Capital Expense Sub-Total				$ 77,777
Mobilization/Demobilization	10%	LS		$ 7,778
Traffic Control/Utility Relocation	2%	LS		$ 1,556
Erosion Control	2%	LS		$ 1,556
Construction Cost Sub-Total				$ 88,665
Construction Contingency	30%	LS		$ 26,600
Capital Expense Total				$ 115,265
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 23,053
Construction & General Administration	5%	LS		$ 5,763
Administrative Expense Total				$ 28,816
Capital Implementation Cost Total				**$ 144,081**

CIP O-1. Ridgewood and Oatfield to Pond

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
RCP Pipeline (18", 2-5' Deep)	384	LF	$ 141	$ 54,012
RCP Pipeline (24", 2-5' Deep)	1,105	LF	$ 194	$ 214,395
HDPE Inlet Leads (12", 2-5' Deep)	60	LF	$ 78	$ 4,707
Precast Concrete Manhole (48", 0-8' deep)	6	EA	$ 3,500	$ 21,000
Catch Basin	6	EA	$ 2,300	$ 13,800
Precast Concrete Manhole (60", 0-8' deep)	2	EA	$ 5,000	$ 10,000
Bioswale				
263 Linear Feet of Open Channel				
General Earthwork & Off-site removal	321	CY	$ 18	$ 5,786
Amended Soils and Mulch	92	CY	$ 40	$ 3,696
Jute Matting, Biodegradeable	613	SY	$ 10	$ 6,129
Stream Bed Gravel	29	CY	$ 40	$ 1,169
Water Quality Facility Plantings	4,990	SF	$ 3	$ 14,970
Capital Expense Sub-Total				$ 349,664
Mobilization/Demobilization	10%	LS		$ 34,966
Traffic Control/Utility Relocation	2%	LS		$ 6,993
Erosion Control	2%	LS		$ 6,993
Construction Cost Sub-Total				$ 398,616
Construction Contingency	30%	LS		$ 119,585
Capital Expense Total				$ 518,201
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 103,640
Construction & General Administration	5%	LS		$ 25,910
Administrative Expense Total				$ 129,550
Capital Implementation Cost Total				**$ 647,752**

CIP O-2. Church Pond Retrofit Evaluation						
Planning Expenses						
Conduct survey and functional evaluation	1	LS	$	15,000	$	15,000
Planning Cost Total					**$**	**15,000**

General CIP. Green Streets Pilot Project

Description	Quantity	Unit	Unit Cost (2013)	2013 Cost
Capital Expenses				
Green Streets in Public ROW (Calculated per 500' block, Type A/B soils)				
Stormwater Planter	1,490	SF	$ 37	$ 55,130
HDPE Overflow Connections (12", 2-5' Deep)	180	LF	$ 78	$ 14,120
Beehive Overflow	6	EA	$ 1,500	$ 9,000
Connection to Existing Structure	6	EA	$ 1,000	$ 6,000
Capital Expense Sub-Total				$ 69,250
Mobilization/Demobilization	10%	LS		$ 6,925
Traffic Control/Utility Relocation	2%	LS		$ 1,385
Erosion Control	2%	LS		$ 1,385
Construction Cost Sub-Total				$ 78,945
Construction Contingency	30%	LS		$ 23,684
Capital Expense Total				$ 102,629
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 20,526
Construction & General Administration	5%	LS		$ 5,131
Administrative Expense Total				$ 25,657
Capital Implementation Cost Total				**$ 128,286**
Green Streets in Public ROW (Calculated per 500' block, Type C/D soils)				
Stormwater Planter	1,965	SF	$ 37	$ 72,705
HDPE Overflow Connections (12", 2-5' Deep)	240	LF	$ 78	$ 18,827
Beehive Overflow	8	EA	$ 1,500	$ 12,000
Connection to Existing Structure	8	EA	$ 1,000	$ 8,000
Capital Expense Sub-Total				$ 111,532
Mobilization/Demobilization	10%	LS		$ 11,153
Traffic Control/Utility Relocation	2%	LS		$ 2,231
Erosion Control	2%	LS		$ 2,231
Construction Cost Sub-Total				$ 127,147
Construction Contingency	30%	LS		$ 38,144
Capital Expense Total				$ 165,291
Administrative Expenses				
Engineering and Permitting	20%	LS		$ 33,058
Construction & General Administration	5%	LS		$ 8,265
Administrative Expense Total				$ 41,323
Capital Implementation Cost Total				**$ 206,613**

Appendix D: CIP Hydraulic Results

Table D-1 CIP Hydraulic Table

System Conduit**							Invert Elevation (ft)		Ground Elevation (ft)		CIP Fut 10 yr Max Water Surface Elevation (ft)		CIP Fut 25 yr Max Water Surface Elevation (ft)		Peak Flow Values at Upstream Node (cfs)*		Duration of Predicted Flooding, 25-yr Design Storm, 36-Hour Model Run (hrs)	
US Node Name	DS Node Name	CIP Name	Length (ft)	Size/Type H = Height, BW = Bottom Width, SS = Side Slope (H:V)	Capacity (cfs)	Slope (%)	US	DS	US	DS	US	DS	US	DS	CIP Fut 10 yr	CIP Fut 25 yr	Future Land Use Condition, Without CIP	Future Land Use Conditions, With CIP
BASIN A																		
Outfall	A0101											28.5		28.5	8.7	9.8		
A0102	A0101	-	174	12'' DIA	4	4.5%	35.35	27.47	37.9	0.0	63.9	28.5	73.4	28.5	9.4	10.7	4.3	4.3
A0103	A0102	-	231	12'' DIA	8	5.1%	47.16	35.35	49.1	37.9	65.6	63.9	75.7	73.4	3.3	3.9	2.9	2.9
A0104	A0103	-	43	12'' DIA	10	8.4%	50.82	47.21	51.8	49.1	65.9	65.6	76.2	75.7	3.9	4.7	2.7	2.7
Outfall	A0110										29.1	29.1	29.2	29.2	8.4	9.3	-	-
A0120	A0110	-	313	18'' DIA	5	0.8%	30.57	27.99	33.3	30.0	35.8	29.1	37.4	29.2	8.4	9.4	6.0	6.0
A0130	A0120	-	90	18'' DIA	8	2.0%	32.32	30.57	33.8	33.3	37.7	35.8	39.8	37.4	10.2	11.5	6.8	7.0
A0140	A0130	CIP A-7	340	1.5' H, 2.5'BW, 3 SS Channel	14	0.2%	32.87	32.32	34.4	33.8	37.9	37.7	40.0	39.8	21.3	22.7	6.3	6.2
A0150	A0140	CIP A-7	402	1.5' H, 2'BW, 3 SS Channel	24	0.6%	35.13	32.87	39.7	34.4	39.8	37.9	40.4	40.0	15.9	18.0	4.2	2.0
A0170	A0150	-	1064	18'' DIA	12	1.4%	49.81	35.43	53.1	39.7	51.0	39.8	54.6	40.4	11.7	13.5	2.5	0.7
A0180	A0170	-	139	18'' DIA	5	0.3%	50.23	49.86	53.6	53.1	51.4	51.0	55.0	54.6	5.5	6.6	2.4	0.7
A0191	A0190	-	309	18'' DIA	4	0.1%	50.61	50.23	56.7	53.6	51.7	51.4	56.7	55.0	2.8	3.2	2.2	0.0
Outfall	A0100										46.1	46.1	46.1	46.1	76.7	78.7	-	-
A0200	A0100	-	1100	See cross-section	157	0.5%	50.16	45.00	55.2	0.0	53.2	46.1	53.4	46.1	78.7	82.3	-	-
A0300	A0200	-	40	48'' DIA	51	0.4%	50.33	50.16	58.6	55.2	53.6	53.2	53.9	53.4	75.6	78.7	-	-
A0310	A0300	-	469	12'' DIA	9	6.8%	82.00	50.33	85.0	58.6	82.5	53.6	82.6	53.9	5.2	6.1	-	-
A0320	A0310	-	252	12'' DIA	4	1.4%	85.50	82.00	88.0	85.0	89.2	82.5	91.2	82.6	5.3	6.2	1.4	1.4
A0330	A0320	-	129	12'' DIA	3	0.5%	86.20	85.50	89.6	88.0	92.0	89.2	95.0	91.2	5.4	6.2	1.5	1.5
A0400	A0300	-	9	48'' DIA	51	0.4%	50.37	50.33	58.6	58.6	53.7	53.6	53.9	53.9	70.4	72.7	-	-
A0410	A0400	-	100	12'' DIA	4	1.3%	51.70	50.37	56.7	58.6	55.2	53.7	56.7	53.9	4.3	4.7	0.0	-
A0420	A0410	-	730	12'' DIA	2	0.3%	54.00	51.70	57.0	56.7	57.0	55.2	57.8	56.7	1.7	2.0	1.0	1.1
A0500	A0400	-	39	48'' DIA	51	0.4%	50.54	50.37	58.3	58.6	54.0	53.7	54.2	53.9	66.3	68.4	-	-
A0600	A0500	-	26	48'' DIA	53	0.5%	50.66	50.54	58.7	58.3	54.2	54.0	54.4	54.2	63.5	66.0	-	-
A0700	A0600	-	153	48'' DIA	41	0.3%	51.09	50.66	60.4	58.7	55.1	54.2	55.4	54.4	63.5	66.0	-	-
A0900	A0700	-	207	48'' DIA	45	0.3%	51.77	51.09	59.7	60.4	56.5	55.1	56.8	55.4	63.5	66.0	-	-
A1000	A0900	-	123	48'' DIA	45	0.3%	52.18	51.77	60.4	59.7	57.3	56.5	57.7	56.8	63.5	66.0	-	-
A1100	A1000	-	34	48'' DIA	109	2.0%	52.85	52.18	61.4	60.4	57.5	57.3	57.9	57.7	61.1	63.2	-	-
A1200	A1100	-	83	4',4', Box	145	0.6%	53.38	52.85	60.0	61.4	57.5	57.5	58.0	57.9	61.2	63.2	-	-
A1300	A1200	-	17	51.96'' DIA	142	0.6%	53.49	53.38	60.0	60.0	57.6	57.5	58.0	58.0	61.2	63.2	-	-
A1500	A1300	-	38	48'' DIA	55	0.5%	53.68	53.49	60.4	60.0	57.8	57.6	58.3	58.0	59.9	61.6	-	-
A1600	A1500	-	65	48'' DIA	53	0.5%	53.99	53.68	60.9	60.4	58.1	57.8	58.6	58.3	60.1	61.6	-	-
A1700	A1600	-	240	48'' DIA	54	0.5%	55.14	53.99	59.1	60.9	59.3	58.1	59.9	58.6	96.7	94.3	11.7	-
A1800	A1700	CIP A-1	97	See cross-section	--	-0.2%	54.97	55.14	59.4	59.1	59.3	59.3	59.9	59.9	116.9	132.1	12.2	-
A1820	A1800	-	425	12'' DIA	2	0.4%	56.87	54.97	58.7	59.4	66.7	59.3	69.9	59.9	5.0	5.9	18.3	5.0
A1850	A1800	CIP A-1	307	See cross-section	--	1.5%	60.62	56.09	63.1	59.4	62.5	59.3	62.5	59.9	68.2	77.2		-
A1900	A1800	ABANDON	41	60'' DIA	--	2.7%	56.09	54.97	61.1	59.4							-	
A2000_1	A1900	CIP A-1	567	See cross-section	300	0.6%	63.80	60.62	66.8	61.1	65.5	59.2	65.6	59.9	45.1	50.9	-	-
A2100	A2000	CIP A-1	512	See cross-section	96	0.6%	67.04	63.80	70.0	66.8	68.7	65.5	68.8	65.6	42.5	48.6	-	-
A2200	A2100	-	23	36'' DIA	50	0.6%	67.17	67.04	72.8	70.0	69.3	68.7	69.5	68.8	42.5	47.6	0.0	-
A2210	A2200	-	136	12'' DIA	2	0.2%	67.73	67.42	71.3	72.8	70.0	69.3	70.5	69.5	2.6	3.0	0.8	-
A2300	A2200	-	119	36'' DIA	49	0.6%	67.98	67.32	72.9	72.8	70.0	69.3	70.2	69.5	40.3	44.9	0.9	-
A2400	A2300	-	94	48'' DIA	58	0.6%	68.50	67.98	72.5	72.9	70.8	70.0	71.0	70.2	42.9	48.0	-	-
A2500	A2400	-	60	See cross-section	91	0.9%	69.04	68.50	73.0	72.5	70.9	70.8	71.1	71.0	45.2	50.9	-	-
A2600	A2500	-	40	48'' DIA	56	0.5%	69.25	69.04	77.3	73.0	71.6	70.9	71.8	71.1	49.1	55.8	-	-
A2700	A2600	-	27	4',5', Box	197	0.6%	69.42	69.25	77.2	77.3	71.6	71.6	71.8	71.8	46.0	54.1	-	-

Table D-1. CIP Hydraulic Model Parameters and Results																		
System Conduit**							Invert Elevation (ft)		Ground Elevation (ft)		CIP Fut 10 yr Max Water Surface Elevation (ft)		CIP Fut 25 yr Max Water Surface Elevation (ft)		Peak Flow Values at Upstream Node (cfs)*		Duration of Predicted Flooding, 25-yr Design Storm, 36-Hour Model Run (hrs)	
US Node Name	DS Node Name	CIP Name	Length (ft)	Size/Type H = Height, BW = Bottom Width, SS = Side Slope (H:V)	Capacity (cfs)	Slope (%)	US	DS	US	DS	US	DS	US	DS	CIP Fut 10 yr	CIP Fut 25 yr	Future Land Use Condition, Without CIP	Future Land Use Conditions, With CIP
A2710	A2700	-	94	12'' DIA	3	0.8%	72.73	71.94	75.4	77.2	73.2	71.6	73.2	71.8	1.3	1.5	0.0	-
A2720	A2710	-	68	12'' DIA	3	0.8%	73.40	72.83	76.2	75.4	73.8	73.2	73.9	73.2	1.3	1.5	-	-
A2800	A2700	-	35	4',5', Box	195	0.6%	69.64	69.42	75.2	77.2	71.5	71.6	71.8	71.8	46.0	52.6	0.6	-
A2900	A2800	-	13	36'' DIA	36	0.3%	69.68	69.64	75.2	75.2	71.9	71.5	72.1	71.8	38.8	43.3	1.1	-
A2920	A2900	-	231	24'' DIA	9	0.1%	70.51	70.18	76.6	75.2	72.5	71.9	72.9	72.1	12.4	14.0	4.3	-
A2930	A2920	-	231	24'' DIA	11	0.2%	71.06	70.51	76.2	76.6	72.8	72.5	73.2	72.9	8.2	9.2	33.3	-
A2940	A2930	-	102	24'' DIA	7	0.1%	71.12	71.01	76.1	76.2	72.9	72.8	73.4	73.2	8.2	9.2	34.3	-
A2950	A2940	-	105	24'' DIA	11	0.2%	71.37	71.12	75.2	76.1	73.0	72.9	73.6	73.4	8.2	9.2	35.1	-
A2960	A2950	-	212	18'' DIA	3	0.1%	71.52	71.32	73.9	75.2	74.3	73.0	75.2	73.6	8.2	9.4	35.9	1.3
A2965	A2960	-	330	18'' DIA	4	0.2%	72.11	71.52	77.7	73.9	76.3	74.3	77.7	75.2	8.3	9.2	35.9	0.0
A2966	A2965	-	396	12'' DIA	6	2.3%	84.29	75.11	88.1	77.7	85.2	76.3	90.3	77.7	6.0	6.8	35.1	0.9
A2967	A2966	-	32	12'' DIA	9	5.6%	86.08	84.29	88.1	88.1	86.7	85.2	91.3	90.3	6.0	7.0	35.2	1.1
A2968	A2967	-	34	12'' DIA	2	0.4%	86.31	86.18	88.4	88.1	87.0	86.7	91.4	91.3	2.3	2.7	35.1	1.1
A2969	A2968	-	214	12'' DIA	7	3.2%	93.25	86.46	96.1	88.4	93.7	87.0	93.7	91.4	2.3	2.7	34.2	-
A2962	A3300	CIP A-2	385	30'' DIA	27	0.4%	69.90	68.30	77.7	75.3	71.4	70.9	71.7	71.1	17.5	17.9		-
A2970	A2965	CIP A-2	402	24'' DIA	14	0.4%	71.54	69.90	77.0	77.7	73.9	76.3	74.0	77.7	17.5	17.9	36.0	-
A2975	A2970	CIP A-2	74	24'' DIA	14	0.4%	71.83	71.54	78.2	77.0	74.3	73.9	74.5	74.0	17.5	17.9	36.0	-
A2980	A2975	CIP A-2	216	24'' DIA	11	0.3%	72.37	71.83	79.0	78.2	75.6	74.3	77.9	74.5	17.5	17.9	36.0	-
A2986	A2980	CIP A-2	252	18'' DIA	5	0.2%	72.87	72.37	77.6	79.0	77.0	75.6	77.6	77.9	3.3	3.3	36.0	0.0
A2987	A2986	CIP A-2	116	12'' DIA	2	0.2%	73.10	72.87	75.1	77.6	76.3	77.0	76.6	77.6	2.5	2.9	36.0	2.2
A3000	A2900	-	128	36'' DIA	36	0.3%	70.05	69.68	74.9	75.2	72.1	71.9	72.3	72.1	52.3	59.9	2.3	-
A3000_2	C1307	CIP A-2	43	48'' DIA	101	0.5%	63.80	63.60	74.9	74.9	66.7	66.5	67.0	66.8	89.9	101.7		-
A3020	A3000	CIP A-3	79	48'' DIA	48	0.1%	70.14	70.05	74.8	74.9	72.4	72.1	72.6	72.3	52.2	59.9	2.4	-
A3020	A3000	CIP A-3	79	48'' DIA	48	0.1%	70.14	70.05	74.8	74.9	72.4	72.1	72.6	72.3	52.2	59.9	2.4	-
A3030	A3020	CIP A-3	310	48'' DIA	47	0.1%	70.47	70.14	75.0	74.8	73.2	72.4	73.5	72.6	51.4	59.2	2.5	-
A3050	A3030	CIP A-3	41	36'' DIA	31	0.2%	70.56	70.47	75.1	75.0	73.3	73.2	73.6	73.5	36.7	41.8	2.5	-
A3100	A3000	CIP A-2	327	42'' DIA	66	0.4%	65.60	64.30	74.6	74.9	68.3	72.1	68.6	72.3	64.4	71.3	19.5	-
A3200	A3100	CIP A-2	378	42'' DIA	66	0.4%	67.10	65.60	75.1	74.6	69.8	68.3	70.1	68.6	61.8	68.2	21.5	-
A3300	A3200	CIP A-2	299	42'' DIA	66	0.4%	68.30	67.10	75.3	75.1	70.9	69.8	71.1	70.1	57.5	63.2	22.3	-
A3400	A3300	CIP A-2	163	42'' DIA	73	0.5%	69.10	68.30	75.6	75.3	71.2	70.9	71.4	71.1	40.0	45.3	22.4	-
A3410	A3400	CIP A-2	153	36'' DIA	56	0.7%	70.70	69.60	75.4	75.6	72.3	71.2	72.4	71.4	31.1	34.8	22.5	-
A34102	A3410	-	39	15'' DIA	7	1.3%	72.32	71.80	74.8	75.4	72.7	72.3	72.8	72.4	1.6	1.8	22.6	-
A3420	A3410	CIP A-2	210	24'' DIA	22	1.0%	72.73	70.70	75.7	75.4	75.7	72.3	76.5	72.4	27.6	30.9	22.3	0.9
A3430	A3420	-	30	12'' DIA	8	4.8%	83.82	82.40	85.0	75.7	84.5	75.7	84.5	76.5	6.1	6.8	15.0	-
A3440	A3430	-	75	12'' DIA	6	3.1%	86.11	83.82	87.4	85.0	86.8	84.5	86.9	84.5	5.3	5.9	14.0	-
A3450	A3440	-	344	12'' DIA	5	1.7%	92.07	86.11	94.1	87.4	95.2	86.8	97.0	86.9	5.6	6.1	12.3	1.5
A3470	A3450	-	219	12'' DIA	4	1.3%	95.10	92.17	97.0	94.1	97.0	95.2	99.1	97.0	3.3	3.6	11.5	1.2
A3480	A3470	-	57	6'' DIA	0	0.7%	95.59	95.20	96.9	97.0	102.2	97.0	105.4	99.1	1.9	2.4	11.7	2.9
A3490	A3480	-	198	12'' DIA	5	2.0%	99.21	95.19	101.3	96.9	102.6	102.2	105.9	105.4	2.0	2.4	10.6	1.7
A3500	A3400	CIP A-2	116	24'' DIA	20	0.8%	72.18	71.30	75.4	75.6	73.1	71.2	73.2	71.4	8.9	10.5	22.4	-
A3600	A3500	-	646	24'' DIA	37	2.8%	90.44	72.18	94.8	75.4	91.0	73.1	91.0	73.2	5.5	6.5	11.8	-
A3610	A3600	-	21	12'' DIA	12	11.8%	92.94	90.44	94.8	94.8	93.2	91.0	93.2	91.0	2.2	2.5	11.8	-
A3620	A3610	-	56	12'' DIA	2	0.3%	93.23	93.04	95.1	94.8	94.0	93.2	94.0	93.2	2.2	2.5	11.7	-
A3640	A3630	-	35	8.04'' DIA	0	0.1%	93.38	93.33	95.7	95.1	95.0	94.0	95.4	94.0	2.9	3.4	11.5	-
A3700	A3600	-	10	24'' DIA	55	6.6%	91.09	90.44	95.1	94.8	91.4	91.0	91.5	91.0	3.3	4.0	11.7	-
A3710	A3700	-	102	12'' DIA	5	1.7%	93.09	91.39	95.6	95.1	93.6	91.4	93.7	91.5	2.3	2.9	11.5	-
A3720	A3710	-	18	12'' DIA	9	5.4%	94.05	93.09	96.0	95.6	94.4	93.6	94.5	93.7	2.3*	2.9*	11.4	-

Table D-1. CIP Hydraulic Model Parameters and Results																		
System Conduit**							Invert Elevation (ft)		Ground Elevation (ft)		CIP Fut 10 yr Max Water Surface Elevation (ft)		CIP Fut 25 yr Max Water Surface Elevation (ft)		Peak Flow Values at Upstream Node (cfs)*		Duration of Predicted Flooding, 25-yr Design Storm, 36-Hour Model Run (hrs)	
US Node Name	DS Node Name	CIP Name	Length (ft)	Size/Type H = Height, BW = Bottom Width, SS = Side Slope (H:V)	Capacity (cfs)	Slope (%)	US	DS	US	DS	US	DS	US	DS	CIP Fut 10 yr	CIP Fut 25 yr	Future Land Use Condition, Without CIP	Future Land Use Conditions, With CIP
A3720A	A3640	-	29	6'' DIA	1	1.8%	94.30	93.78	96.0	95.7	94.4	95.0	94.5	95.4	0.7*	0.9*	11.4	-
A3730	A3720	-	60	12'' DIA	6	2.5%	95.73	94.25	97.5	96.0	96.1	94.4	96.1	94.5	1.7	2.0	10.9	-
A3740	A3730	-	78	12'' DIA	7	3.8%	98.78	95.83	100.3	97.5	99.1	96.1	99.1	96.1	1.7	2.0	10.1	-
A3741	A3740	-	13	12'' DIA	4	13.9%	100.52	98.78	101.5	100.3	101.0	99.1	101.0	99.1	1.7	2.0	9.8	-
A3742	A3741	-	224	1' H, 1'BW, 2 SS Channel	24	9.2%	121.10	100.52	122.1	101.5	121.4	101.0	121.4	101.0	1.7	2.0	3.4	-
A3743	A3742	-	142	12'' DIA	2	0.5%	121.76	121.10	123.5	122.1	122.4	121.4	122.4	121.4	1.7	2.0	2.9	-
A3744	A3743	-	127	12'' DIA	1	0.2%	122.05	121.86	123.6	123.5	122.9	122.4	123.0	122.4	1.7	2.0	2.9	-
A3800	A3700	-	9	24'' DIA	48	7.5%	92.02	91.35	95.7	95.1	92.2	91.4	92.2	91.5	1.0	1.1	11.5	-
A3900	A3800	ABANDON	225	24'' DIA	--	1.4%	95.20	92.02	98.2	95.7							10.7	
A3910	A3900	ABANDON	57	12'' DIA	--	1.5%	96.03	95.20	97.7	98.2							10.9	
A3915	A3920	CIP A-6	203	12'' DIA	4	1.0%	95.42	93.39	97.7	97.4	96.1	93.5	96.2	93.6	2.7	3.2		-
A3920	A3427	CIP A-6	516	24'' DIA	38	2.8%	92.39	77.86	97.4	83.9	93.5	79.3	93.6	79.7	21.6	24.2		-
A4100	A3900	ABANDON	338	18'' DIA	--	0.6%	97.22	95.20	100.3	98.2							10.0	
A4100_2	A3920	CIP A-6	110	24'' DIA	37	2.6%	95.27	92.39	100.3	97.4	96.3	93.5	96.4	93.6	18.9	21.0	10.0	-
A4110	A4100	-	43	15'' DIA	3	0.3%	97.34	97.22	100.0	100.3	98.0	96.3	98.1	96.4	2.3	2.7	10.5	-
A4120	A4110	-	73	15'' DIA	10	2.2%	99.03	97.44	100.7	100.0	99.4	98.0	99.4	98.1	1.3	1.6	10.3	-
A4300_1	A4100	-	223	18'' DIA	22	4.5%	112.58	102.45	115.7	100.3	113.5	96.3	113.6	96.4	16.6	18.3	6.7	-
A4300_2	A4100	-	115	18'' DIA	22	4.6%	102.45	97.22	115.7	100.3	113.5	96.3	113.6	96.4	16.6	18.3	6.7	-
A4400	A4100	-	97	18'' DIA	33	10.1%	122.53	112.73	105.5	100.3	103.4	96.3	103.5	96.4	16.6	18.3		-
A4500	A4400	-	171	18'' DIA	20	3.5%	128.65	122.63	131.0	124.6	129.6	123.3	129.6	123.3	13.5	14.6	2.5	-
A4600	A4500	-	42	18'' DIA	17	2.7%	130.09	128.95	132.9	131.0	131.1	129.6	131.2	129.6	13.5	14.6	2.0	-
A4700	A4600	-	250	18'' DIA	29	7.8%	149.49	130.19	151.9	132.9	150.2	131.1	150.2	131.2	13.5	14.6	-	-
A4900	A4700	-	202	18'' DIA	26	6.0%	161.67	149.59	165.2	151.9	162.4	150.2	162.4	150.2	11.9	12.8	-	-
A5000	A4900	-	67	18'' DIA	19	3.1%	163.80	161.72	168.7	165.2	164.7	162.4	164.7	162.4	11.9	12.8	-	-
A5010	A5000	-	357	12'' DIA	2	0.5%	165.36	163.63	168.8	168.7	168.8	164.7	168.8	164.7	2.8	3.2	0.0	0.0
A5020	A5010	-	226	12'' DIA	8	4.5%	175.55	165.36	180.5	168.8	176.0	168.8	176.0	168.8	2.8	3.2	-	-
A5100	A5000	-	51	12'' DIA	7	13.1%	170.68	164.05	172.7	168.7	172.1	164.7	172.3	164.7	10.0	11.7	-	-
A5100A	F1800	-	111	18'' DIA	31	8.0%	169.58	160.71	172.7	163.1	169.9	160.3	170.0	161.0	2.8	4.4	-	-
Wetland_In	A1901	-	312	See cross-section	469	1.6%	65.50	60.62	69.0	69.0	66.1	66.1	66.1	66.1	23.5	26.6	-	-
A1902	A1901	-	65	24'' DIA	53	17.7%	77.82	66.50	80.8	69.0	78.2	66.1	78.3	66.1	4.7	5.5	-	-
A1920	A1901	-	132	24'' DIA	37	2.5%	69.79	66.50	75.2	69.0	70.8	66.1	70.9	66.1	18.8	21.1	-	-
A1930	A1920	-	106	24'' DIA	72	9.6%	80.06	69.99	88.3	75.2	80.8	70.8	80.8	70.9	18.8	21.2	-	-
A1940	A1930	-	94	24'' DIA	25	1.1%	81.32	80.26	87.3	88.3	82.5	80.8	82.6	80.8	15.8	18.5	-	-
A1950	A1940	-	37	24'' DIA	11	0.2%	81.60	81.52	87.8	87.3	83.2	82.5	83.4	82.6	15.8	20.8	-	-
A1951	A1950	-	25	18'' DIA	14	1.6%	82.11	81.70	87.8	87.8	83.2	83.2	83.3	83.4	2.8	3.4	-	-
A1952	A1951	-	134	18'' DIA	7	0.4%	82.70	82.16	87.9	87.8	83.4	83.2	83.4	83.3	2.8	3.3	-	-
A1953	A1952	-	51	12'' DIA	3	0.5%	82.95	82.70	87.7	87.9	83.6	83.4	83.6	83.4	1.8	2.1	-	-
A1954	A1953	-	249	12'' DIA	2	0.2%	83.63	83.05	90.1	87.7	84.5	83.6	84.6	83.6	1.8	2.1	-	-
A1960	A1950	-	24	24'' DIA	30	1.7%	82.10	81.70	87.8	87.8	83.3	83.2	83.6	83.4	13.1	16.1	-	-
A1970	A1960	-	311	24'' DIA	5	0.0%	82.35	82.20	87.1	87.8	84.7	83.3	87.1	83.6	13.1	15.9	0.1	0.1
A1975	A1970	-	356	21'' DIA	6	0.1%	83.05	82.55	85.4	87.1	86.9	84.7	87.9	87.1	13.1	15.3	1.8	1.8
A1980	A1975	-	71	18'' DIA	5	0.2%	83.22	83.05	85.4	85.4	87.7	86.9	89.0	87.9	11.5	13.4	1.9	1.9
A1985	A1980	-	199	18'' DIA	2	0.0%	83.36	83.27	86.7	85.4	89.9	87.7	91.9	89.0	11.6	13.5	1.8	1.8
A1987	A1985	-	53	12'' DIA	4	1.0%	84.04	83.51	86.0	86.7	90.3	89.9	92.5	91.9	3.2	3.7	2.1	2.1
A1988	A1987	-	225	12'' DIA	7	3.6%	92.30	84.14	97.4	86.0	92.9	90.3	97.4	92.5	3.2	3.7	-	0.0
A1989	A1988	-	227	12'' DIA	3	0.7%	94.05	92.40	98.1	97.4	94.9	92.9	98.1	97.4	3.2	3.7	0.0	0.0
A1990	A1985	-	383	15'' DIA	4	0.4%	85.02	83.46	89.0	86.7	91.6	89.9	94.3	91.9	4.6	5.4	1.6	1.6

Table D-1. CIP Hydraulic Model Parameters and Results

System Conduit**							Invert Elevation (ft)		Ground Elevation (ft)		CIP Fut 10 yr Max Water Surface Elevation (ft)		CIP Fut 25 yr Max Water Surface Elevation (ft)		Peak Flow Values at Upstream Node (cfs)*		Duration of Predicted Flooding: 25-yr Design Storm, 36-Hour Model Run (hrs)	
US Node Name	DS Node Name	CIP Name	Length (ft)	Size/Type H = Height, BW = Bottom Width, SS = Side Slope (H:V)	Capacity (cfs)	Slope (%)	US	DS	US	DS	US	DS	US	DS	CIP Fut 10 yr	CIP Fut 25 yr	Future Land Use Condition, Without CIP	Future Land Use Conditions, With CIP
A1992	A1990	-	70	12'' DIA	1	0.2%	85.24	85.12	89.0	89.0	92.7	91.6	95.8	94.3	4.7	5.5	1.7	1.7
Outfall	A2993										73.4	73.4	73.4	73.4	7.2	8.6	-	-
A2994	A2993	-	382	1' H, 1'BW, 2 SS Channel	6	0.6%	74.61	72.40	75.6	0.0	75.1	73.4	75.1	73.4	1.1	1.3	-	-
A2995	A2994	-	147	12'' DIA	3	0.5%	75.35	74.61	77.4	75.6	75.8	75.1	75.8	75.1	1.1	1.3	-	-
A2996	A2995	-	49	1' H, 1'BW, 2 SS Channel	8	1.3%	76.00	75.35	77.0	77.4	76.4	75.8	76.4	75.8	1.1	1.3	-	-
A2998	A2996	-	103	12'' DIA	5	2.1%	78.15	76.00	81.2	77.0	78.5	76.4	78.5	76.4	1.1	1.3	-	-
A2999	A2998	-	186	12'' DIA	8	4.4%	86.56	78.35	96.3	81.2	86.8	78.5	86.8	78.5	1.1	1.3	-	-
BASIN B - No Modeled CIPs																		
BASIN C																		
Outfall	C0100										16.7	16.7	16.9	16.9	118.4	134.0	-	-
Overflow Outfall	Z0200										50.6	50.6	50.7	50.7	3.9	5.1	-	-
C0200	C0100	CIP A-2	101	48'' DIA	162	1.2%	15.38	14.18	19.4	0.0	17.9	16.7	18.1	16.9	118.4	134.0	-	-
C0300	C0200	CIP A-2	223	48'' DIA	79	0.3%	16.05	15.38	20.1	19.4	19.5	17.9	20.3	18.1	118.4	134.0	2.8	0.4
C0400	C0300	CIP A-2	35	48'' DIA	658	20.3%	23.00	16.05	32.5	20.1	24.2	19.5	24.2	20.3	118.4	134.0	-	-
C0500	C0400	CIP A-2	125	48'' DIA	592	15.4%	45.62	26.51	60.6	32.5	46.8	24.2	46.9	24.2	118.4	134.0	-	-
C0600	C0500	-	68	36'' DIA	48	0.5%	49.46	49.14	59.4	60.6	50.9	46.8	51.0	46.9	21.9	24.4	-	-
C0700	C0600	-	385	36'' DIA	44	0.4%	51.24	49.72	61.3	59.4	52.8	50.9	52.8	51.0	21.9	24.4	-	-
C0800	C0700	-	29	36'' DIA	61	-0.8%	51.02	51.24	61.9	61.3	52.9	52.8	53.0	52.8	25.9	29.5	-	-
C0800A	Z0200	-	82	18'' DIA	5	0.9%	50.52	49.81	61.9	0.0	51.5	50.6	51.7	50.7	3.9	5.1	-	-
C0810	C0800	-	279	18'' DIA	9	0.8%	52.67	50.52	58.1	61.9	53.0	52.9	53.1	53.0	0.9	1.1	-	-
C0900	C0800	-	17	36'' DIA	17	0.1%	51.37	51.36	62.0	61.9	53.0	52.9	53.2	53.0	24.9	28.4	-	-
C1000	C0900	-	233	36'' DIA	45	0.4%	52.36	51.37	58.5	62.0	54.0	53.0	54.1	53.2	24.9	28.4	-	-
C1100	C1000	-	283	30'' DIA	39	0.9%	54.83	52.42	60.3	58.5	56.2	54.0	56.4	54.1	23.6	26.8	-	-
C1200	C0500	CIP A-2	179	48'' DIA	134	0.8%	47.25	45.82	60.8	60.6	49.8	46.8	50.0	46.9	95.2	107.9		-
C1300	C1100	-	262	30'' DIA	17	0.2%	55.24	54.83	60.2	60.3	57.2	56.2	57.5	56.4	22.1	25.0	-	-
C1301	C1300	-	232	15'' DIA	5	0.5%	56.31	55.26	59.8	60.2	57.3	57.2	57.6	57.5	1.3	1.5	1.2	-
C1302_2	C1301	CIP A-2	801	48'' DIA	130	0.8%	53.40	47.25	60.3	59.8	55.9	57.3	56.2	57.6	95.2	107.9	2.2	-
C1303	C1302	CIP A-2	231	48'' DIA	133	0.8%	55.25	53.40	62.1	60.3	57.7	55.9	58.0	56.2	94.4	107.0	2.1	-
C1304	C1303	CIP A-2	281	48'' DIA	134	0.8%	57.50	55.25	64.6	62.1	60.0	57.7	60.2	58.0	92.5	104.8	2.1	-
C1305	C1304	CIP A-2	242	48'' DIA	105	0.5%	58.90	57.70	67.1	64.6	61.8	60.0	62.1	60.2	92.5	104.8	1.8	-
C1306	C1305	CIP A-2	511	48'' DIA	112	0.6%	62.00	59.10	75.0	67.1	64.7	61.8	65.0	62.1	89.9	101.7		-
C1307	C1306	CIP A-2	368	48'' DIA	99	0.4%	63.60	62.00	74.9	75.0	66.5	64.7	66.8	65.0	90.2	101.7		-
C1310	C1300	-	456	24'' DIA	14	0.3%	56.89	55.36	62.4	60.2	58.1	57.2	58.3	57.5	9.3	11.1	-	-
C1320	C1310	-	270	24'' DIA	16	0.5%	58.11	56.89	63.0	62.4	59.0	58.1	59.1	58.3	6.9	8.3	-	-
C1330	C1320	-	71	18'' DIA	14	1.7%	59.31	58.11	65.1	63.0	59.9	59.0	59.9	59.1	4.1	4.8	-	-
C1340	C1330	-	172	18'' DIA	13	1.4%	61.64	59.31	69.2	65.1	62.2	59.9	62.3	59.9	4.1	4.8	-	-
C1360	C1350	-	245	15'' DIA	10	2.2%	67.15	61.79	71.8	69.2	67.7	62.2	67.8	62.3	4.1	5.0	-	-
C1370	C1360	-	43	12'' DIA	4	1.3%	67.71	67.15	72.7	71.8	68.5	67.7	68.6	67.8	4.1	4.8	-	-
C1380	C1370	-	248	12'' DIA	4	1.4%	71.21	67.71	77.0	72.7	71.6	68.5	71.7	68.6	1.6	1.8	-	-
C1400	C1300	-	283	24'' DIA	20	0.7%	57.25	55.24	66.2	60.2	58.4	57.2	58.4	57.5	11.6	12.5	-	-
C1500	C1400	-	225	24'' DIA	16	0.5%	58.48	57.41	66.6	66.2	59.6	58.4	59.7	58.4	10.4	11.1	-	-
C1600	C1500	-	93	24'' DIA	19	0.7%	60.73	60.10	66.4	66.6	61.8	59.6	61.8	59.7	10.4	11.1	-	-
C1700	C1600	-	165	24'' DIA	20	0.7%	61.88	60.73	68.2	66.4	62.9	61.8	63.0	61.8	10.4	11.1	-	-
C1800	C1700	-	282	15'' DIA	5	0.5%	64.87	63.52	74.8	68.2	69.3	62.9	69.9	63.0	8.8	9.3	-	-
C1810	C1800	-	204	12'' DIA	5	1.5%	68.03	65.03	73.1	74.8	69.6	69.3	70.2	69.9	1.3	1.5	-	-
C1820	C1810	-	35	12'' DIA	5	1.7%	71.33	70.72	73.0	73.1	71.7	69.6	71.7	70.2	1.3	1.5	-	-

Table D-1. CIP Hydraulic Model Parameters and Results																		
System Conduit**							Invert Elevation (ft)		Ground Elevation (ft)		CIP Fut 10 yr Max Water Surface Elevation (ft)		CIP Fut 25 yr Max Water Surface Elevation (ft)		Peak Flow Values at Upstream Node (cfs)*		Duration of Predicted Flooding, 25-yr Design Storm, 36-Hour Model Run (hrs)	
US Node Name	DS Node Name	CIP Name	Length (ft)	Size/Type H = Height, BW = Bottom Width, SS = Side Slope (H:V)	Capacity (cfs)	Slope (%)	US	DS	US	DS	US	DS	US	DS	CIP Fut 10 yr	CIP Fut 25 yr	Future Land Use Condition, Without CIP	Future Land Use Conditions, With CIP
C1830	C1820	-	179	12'' DIA	3	0.9%	72.86	71.33	76.1	73.0	73.3	71.7	73.3	71.7	1.3	1.5	-	-
C1900	C1800	-	322	15'' DIA	5	0.6%	66.81	64.89	76.6	74.8	73.4	69.3	75.6	69.9	7.5	7.7	0.9	-
C2000	C1900	-	335	18'' DIA	8	0.5%	68.62	66.91	75.4	76.6	74.2	73.4	74.9	75.6	6.1	6.1	2.3	-
C2010	C2000	-	12	12'' DIA	15	15.7%	70.51	68.62	75.4	75.4	74.2	74.2	75.0	74.9	2.0	2.4	2.3	-
C2020	C2010	-	155	12'' DIA	3	0.8%	72.47	71.16	74.8	75.4	74.8	74.2	75.6	75.0	2.0	2.4	2.7	0.9
C2030	C2020	-	106	12'' DIA	3	0.7%	73.20	72.47	76.7	74.8	75.0	74.8	76.7	75.6	2.0	2.4	1.9	0.0
C2200	C2000	-	183	18'' DIA	6	0.3%	69.16	68.62	74.9	75.4	74.4	74.2	75.0	74.9	5.8	5.8	2.7	0.5
C2300	C2200	-	101	18'' DIA	15	1.9%	71.22	69.26	74.8	74.9	74.5	74.4	75.1	75.0	8.1	8.2	2.8	0.7
C2400	C2300	-	10	18'' DIA	16	2.0%	71.42	71.22	75.4	74.8	74.5	74.5	75.1	75.1	3.7*	3.8*	2.6	-
C2400A_1	A3050	CIP A-3	247	36'' DIA	30	0.2%	71.30	70.80	75.4	75.1	74.5	73.3	75.1	73.6	36.8*	41.8*	2.6	-
C2500	C2400	CIP A-3	9	36'' DIA	71	1.1%	71.40	71.30	75.5	75.4	74.5	74.5	75.2	75.1	37.0*	39.8*	2.6	-
C2500A	C2300	-	13	18'' DIA	15	2.0%	71.47	71.22	75.5	74.8	74.5	74.5	75.2	75.1	7.2*	7.2*	2.6	-
C2510	C2500	-	128	12'' DIA	4	1.1%	72.94	71.53	74.9	75.5	74.7	74.5	75.4	75.2	1.4	1.7	3.1	0.8
BASIN D - No Modeled CIPs																		
BASIN E																		
Outfall	E0100										57.8	57.8	57.9	57.9	30.9	35.0	-	-
PE0200	E0100	Sized, but not included in CIP	145	30'' DIA	58	2.0%	59.44	56.55	65.8	0.0	60.7	57.8	60.8	57.9	30.1	34.0		-
E0210	PE0200	-	93	15'' DIA	4	1.4%	63.63	62.32	66.4	65.8	64.2	60.7	64.3	60.8	1.8	2.0	-	-
PE0300	PE0200	Sized, but not included in CIP	390	30'' DIA	29	0.5%	61.59	59.64	68.1	65.8	63.3	60.7	63.5	60.8	23.7	26.6		-
PE0400	PE0300	Sized, but not included in CIP	304	30'' DIA	40	1.0%	64.73	61.79	72.2	68.1	66.1	63.3	66.2	63.5	23.7	26.7	6.1	-
PE0500	E0400	Sized, but not included in CIP	228	30'' DIA	63	2.4%	70.22	64.73	75.8	72.2	71.2	66.1	71.3	66.2	21.5	24.1	5.9	-
E0520	E0500	-	301	15'' DIA	5	0.7%	75.03	73.05	79.4	75.8	75.8	71.2	75.9	71.3	3.8	4.4	4.9	-
E0521	E0520	-	30	12'' DIA	4	1.2%	75.67	75.31	79.0	79.4	76.2	75.8	76.3	75.9	2.4	2.8	5.0	-
E0522	E0521	-	177	12'' DIA	6	2.8%	80.74	75.77	89.0	79.0	81.2	76.2	81.2	76.3	2.4	2.8	-	-
E0523	E0522	-	405	12'' DIA	3	0.8%	83.85	80.79	89.2	89.0	84.5	81.2	84.6	81.2	2.4	2.8	0.9	-
PE0600	E0500	Sized, but not included in CIP	60	24'' DIA	40	3.1%	72.30	70.44	78.1	75.8	73.2	71.2	73.3	71.3	17.6	19.7	5.3	-
PE0700	E0600	Sized, but not included in CIP	87	24'' DIA	52	5.2%	77.00	72.50	82.5	78.1	77.8	73.2	77.9	73.3	17.6	19.7		-
PE0800	PE0700	Sized, but not included in CIP	277	24'' DIA	36	2.5%	84.10	77.20	87.6	82.5	85.1	77.8	85.2	77.9	17.6	19.7	7.7	-
PE0900	E0800	Sized, but not included in CIP	120	24'' DIA	53	5.0%	90.49	84.42	94.5	87.6	91.3	85.1	91.3	85.2	17.7	19.7	7.5	-
E1000	E0900	-	214	18'' DIA	24	4.8%	100.88	90.54	104.3	94.5	101.6	91.3	101.6	91.3	9.9	10.5	6.8	-
E1100	E1000	-	219	12'' DIA	7	4.0%	109.81	100.98	112.8	104.3	117.6	101.6	119.6	101.6	9.9	10.5	6.4	4.0
E1200	E1100	-	239	12'' DIA	7	3.1%	117.36	109.86	120.9	112.8	134.6	117.6	138.8	119.6	10.1	10.7	6.3	4.3
E1300	E1200	-	252	12'' DIA	7	4.1%	127.60	117.41	131.0	120.9	152.9	134.6	159.2	138.8	10.8	11.6	6.1	4.2
E1400	E1300	-	251	12'' DIA	8	4.7%	139.49	127.65	142.9	131.0	163.7	152.9	171.3	159.2	8.4	8.5	5.6	3.9
E1500	E1400	-	250	12'' DIA	8	5.2%	152.52	139.54	156.3	142.9	174.6	163.7	183.6	171.3	9.2	9.8	5.1	3.6
E1600	E1500	-	175	12'' DIA	9	5.8%	162.80	152.57	165.1	156.3	182.5	174.6	192.5	183.6	10.5	11.6	4.8	3.4
E1700	E1600	-	211	12'' DIA	9	6.3%	176.19	162.88	177.7	165.1	183.7	182.5	193.8	192.5	3.9	4.5	3.7	2.5
E1800	E1700	-	249	15'' DIA	17	6.3%	191.90	176.19	194.8	177.7	192.3	183.7	194.3	193.8	3.9	4.5	1.2	-
E1900	E1800	-	301	18'' DIA	19	2.8%	201.39	192.88	204.4	194.8	201.9	192.3	201.9	194.3	3.9	4.5	-	-
E2000	E1900	-	109	18'' DIA	23	4.3%	207.06	202.44	209.1	204.4	207.5	201.9	207.5	201.9	3.9	4.5	-	-
E2100	E2000	-	151	12'' DIA	8	4.2%	213.42	207.06	216.8	209.1	214.0	207.5	214.0	207.5	3.9	4.5	-	-
E2200	E2100	-	182	12'' DIA	7	3.4%	219.70	213.47	222.2	216.8	220.3	214.0	220.3	214.0	3.9	4.5	-	-
BASIN F																		
Outfall	F0105										111.7	111.7	111.7	111.7	13.9	17.5		-
F0110	F0105	CIP F-1	622	24'' DIA	24	1.1%	116.57	109.60	120.7	111.7	117.7	111.7	117.8	111.7	13.9	17.5		-
F0200	F0100	-	207	15'' DIA	5	0.6%	111.11	109.96	114.7	112.0	114.7	111.6	114.7	111.6	4.9	5.7	2.9	-

Table D-1. CIP Hydraulic Model Parameters and Results

System Conduit**							Invert Elevation (ft)		Ground Elevation (ft)		CIP Fut 10 yr Max Water Surface Elevation (ft)		CIP Fut 25 yr Max Water Surface Elevation (ft)		Peak Flow Values at Upstream Node (cfs)*		Duration of Predicted Flooding, 25-yr Design Storm, 36-Hour Model Run (hrs)	
US Node Name	DS Node Name	CIP Name	Length (ft)	Size/Type H = Height, BW = Bottom Width, SS = Side Slope (H:V)	Capacity (cfs)	Slope (%)	US	DS	US	DS	US	DS	US	DS	CIP Fut 10 yr	CIP Fut 25 yr	Future Land Use Condition, Without CIP	Future Land Use Conditions, With CIP
F0300	F0200	-	166	15'' DIA	8	1.6%	113.82	111.16	116.5	114.7	114.2	114.7	114.2	114.7	1.6	1.9	2.9	-
F0400	F0300	-	114	15'' DIA	5	0.7%	114.61	113.86	117.7	116.5	115.1	114.2	115.1	114.2	1.6	1.9	2.9	-
F0500	F0400	-	73	15'' DIA	--	3.0%	117.18	114.96	119.5	117.7	117.2	115.1	117.2	115.1	0.0	0.0	2.8	-
F0600	F0500	Disconnect US End	179	15'' DIA	--	2.0%	120.81	117.18	124.9	119.5								
F0600_2	F0110						120.81	116.57		120.7		117.7		117.8	12.4	15.7	2.5	-
F0610	F0600	-	403	12'' DIA	10	8.0%	153.13	121.11	157.7	124.9	153.5	121.8	153.6	122.0	3.3	3.8	-	-
F0620	F0610	-	45	12'' DIA	17	22.8%	163.21	153.13	165.6	157.7	163.5	153.5	163.5	153.6	2.4	2.8	-	-
F0630	F0620	-	184	12'' DIA	12	11.9%	185.30	163.61	188.0	165.6	185.6	163.5	185.6	163.5	2.4	2.8	-	-
F0640	F0630	-	182	12'' DIA	6	2.4%	189.70	185.30	191.2	188.0	190.0	185.6	190.1	185.6	1.3	1.5	-	-
F0650	F0640	-	182	6'' DIA	1	2.4%	194.13	189.70	195.2	191.2	200.1	190.0	203.0	190.1	1.4	1.6	2.2	2.2
F0700	F0600	CIP F-1	31	24'' DIA	48	4.6%	122.49	121.06	126.5	124.9	123.1	121.8	123.2	122.0	9.1	11.9	2.4	-
F0710	F0700	-	186	12'' DIA	10	7.3%	137.08	123.59	138.6	126.5	137.3	123.1	137.3	123.2	1.3	1.5	1.0	-
F0720	F0710	-	328	12'' DIA	11	10.8%	172.32	137.13	174.4	138.6	172.5	137.3	172.5	137.3	0.9	1.1	-	-
F0900	F0700	CIP F-1	279	24'' DIA	41	3.3%	132.75	123.59	136.2	126.5	133.3	123.1	133.4	123.2	7.9	10.4	2.2	-
F1000	F0900	CIP F-1	32	24'' DIA	34	2.1%	133.44	132.75	136.4	136.2	134.0	133.3	134.1	133.4	5.8	7.9	2.0	-
F1100	F1000	-	392	12'' DIA	7	4.2%	150.01	133.44	160.5	136.4	150.7	134.0	158.7	134.1	5.8	7.9	0.9	-
F1500	F1400	-	71	12'' DIA	11	9.7%	156.84	150.01	160.9	160.5	157.2	150.7	157.5	158.7	2.8	4.4	0.9	-
F1600	F1500	-	49	12'' DIA	3	0.9%	157.46	157.04	160.4	160.9	158.2	157.2	158.7	157.5	2.8	4.4	1.0	-
F1700	F1600	-	34	12'' DIA	7	3.7%	158.60	157.36	161.7	160.4	159.0	158.2	159.3	158.7	2.8	4.4	0.9	-
F1800	F1700	-	92	12'' DIA	4	1.0%	159.66	158.70	163.1	161.7	160.3	159.0	161.0	159.3	2.8	4.4	0.8	-
BASIN G - No Modeled CIPs																		
BASIN H - No Modeled CIPs																		
BASIN I - No Modeled CIPs																		
BASIN J																		
Basin J drains to Basin A at A3030 & C2500																		
J0200	C2500	CIP A-3	299	36'' DIA	27	0.2%	71.90	71.40	76.9	75.5	75.5	74.5	76.4	75.2	38.7	45.1	2.9	-
J0300	J0200	-	124	36'' DIA	104	2.3%	74.78	71.90	79.8	76.9	76.0	75.5	76.5	76.4	34.4	40.0	2.2	-
J0400	J0300	-	126	36'' DIA	344	25.1%	105.32	74.78	112.6	79.8	106.0	76.0	106.0	76.5	34.4	40.0	-	-
J0500	J0400	-	81	36'' DIA	149	4.6%	111.04	107.32	116.9	112.6	112.0	106.0	112.1	106.0	34.4	40.0	-	-
J0600	J0500	-	255	36'' DIA	19	0.1%	112.63	112.44	121.1	116.9	115.2	112.0	115.6	112.1	34.4	40.0	-	-
J0800	J0600	CIP J-1	329	30'' DIA	67	2.8%	122.38	113.33	128.3	121.1	123.6	115.2	123.8	115.6	34.4	40.0	0.0	-
J0900	J0800	CIP J-1	248	30'' DIA	73	3.3%	130.45	122.38	140.5	128.3	131.6	123.6	131.7	123.8	31.5	36.6	-	-
J1000	J0900	ABANDON	222	18'' DIA	--	3.7%	142.20	133.90	147.2	140.5								
J1200_2	J0900	CIP J-1	245	30'' DIA	66	2.6%	136.70	130.45	148.9	140.5	137.9	131.6	138.1	131.7	31.5	36.6	-	-
J1300	J1200	CIP J-1	85	30'' DIA	63	2.2%	138.55	136.70	146.9	148.9	139.8	137.9	139.9	138.1	31.5	36.6	1.0	-
J1400	J1300	-	225	30'' DIA	89	4.7%	149.10	138.55	155.3	146.9	150.0	139.8	150.0	139.9	23.1	26.8	0.8	-
J1600	J1400	-	257	1.5' H, 1'BW, 2 SS Channel	52	7.8%	169.30	149.42	171.8	155.3	170.4	150.0	170.4	150.0	23.1	26.8	-	-
J1700	J1600	-	102	18'' DIA	13	1.3%	170.67	169.30	175.0	171.8	174.5	170.4	175.9	170.4	21.0	24.4	-	0.8
J1710	J1700	-	39	12'' DIA	4	1.5%	171.77	171.20	174.9	175.0	174.9	174.5	176.3	175.9	3.2	3.7	-	1.0
J1720	J1710	-	182	12'' DIA	3	0.8%	173.48	171.97	176.8	174.9	175.3	174.9	176.8	176.3	1.8	2.1	-	0.1
J1730	J1720	-	103	12'' DIA	6	2.2%	175.98	173.68	179.4	176.8	176.4	175.3	177.1	176.8	1.8	2.1	-	-
J1740	J1730	-	175	12'' DIA	6	2.6%	180.73	176.18	184.1	179.4	181.1	176.4	181.1	177.1	1.8	2.1	-	-
J1750	J1740	-	66	12'' DIA	6	2.4%	182.55	180.93	186.2	184.1	182.9	181.1	183.0	181.1	1.8	2.1	-	-
J1800	J1700	-	87	18'' DIA	16	2.1%	172.70	170.87	176.4	175.0	176.9	174.5	179.1	175.9	17.9	20.8	-	1.0
J1900	J1800	-	136	18'' DIA	19	3.0%	177.03	172.90	182.0	176.4	181.3	176.9	184.0	179.1	17.9	21.0	-	0.8
J2000	J1900	-	179	18'' DIA	31	8.1%	191.69	177.23	196.5	182.0	192.5	181.3	192.7	184.0	17.9	21.0	-	-

Table D-1. CIP Hydraulic Model Parameters and Results

System Conduit[a,b]							Invert Elevation (ft)		Ground Elevation (ft)		CIP Fut 10 yr Max Water Surface Elevation (ft)		CIP Fut 25 yr Max Water Surface Elevation (ft)		Peak Flow Values at Upstream Node (cfs)[c]		Duration of Predicted Flooding, 25-yr Design Storm, 36-Hour Model Run (hrs)	
US Node Name	DS Node Name	CIP Name	Length (ft)	Size/Type H = Height, BW = Bottom Width, SS = Side Slope (H:V)	Capacity (cfs)	Slope (%)	US	DS	US	DS	US	DS	US	DS	CIP Fut 10 yr	CIP Fut 25 yr	Future Land Use Condition, Without CIP	Future Land Use Conditions, With CIP
J2100	J2000	CIP J-2	58	24'' DIA	27	1.5%	192.75	191.89	197.4	196.5	193.9	192.5	194.0	192.7	16.5	19.4	-	-
J2200	J2100	CIP J-2	148	24'' DIA	18	0.7%	193.83	192.85	198.3	197.4	195.3	193.9	195.5	194.0	16.5	19.4	-	-
J2300	J2200	CIP J-2	42	24'' DIA	40	3.2%	195.35	194.03	199.2	198.3	196.3	195.3	196.4	195.5	16.5	19.4	-	-
J2400	J2300	CIP J-2	48	24'' DIA	36	2.5%	196.74	195.51	199.8	199.2	197.6	196.3	197.6	196.4	12.4	14.5	-	-
J2500	J2400	CIP J-2	327	24'' DIA	29	1.6%	201.93	196.65	205.1	199.8	202.9	197.6	202.9	197.6	12.4	14.5	5.2	-
J2510	J2500	-	253	12'' DIA	7	4.2%	212.70	202.18	214.7	205.1	213.2	202.9	213.3	202.9	3.7	4.3	2.9	-
J2530	J2520	-	66	12'' DIA	5	1.8%	213.90	212.70	217.4	214.7	214.3	213.2	214.3	213.3	1.6	1.9	2.1	-
J2540	J2530	-	45	12'' DIA	6	2.5%	215.03	213.90	219.6	217.4	215.4	214.3	215.4	214.3	1.6	1.9	1.3	-
J2550	J2540	-	103	12'' DIA	6	2.9%	217.97	215.03	220.0	219.6	218.3	215.4	218.4	215.4	1.6	1.9	1.3	-
J2560	J2550	-	364	12'' DIA	6	3.0%	229.26	218.32	233.9	220.0	229.6	218.3	229.6	218.4	1.6	1.9	-	-
J2610	J2500	CIP J-2	408	18'' DIA	9	0.8%	205.23	202.06	207.7	205.1	206.4	202.9	207.1	202.9	8.7	10.2	5.7	-
J2700	J2610	CIP J-2	242	18'' DIA	12	1.4%	208.65	205.23	212.6	207.7	209.5	206.4	209.6	207.1	7.9	9.3	4.9	-
J2710	J2700	-	209	12'' DIA	10	7.5%	224.31	208.74	229.4	212.6	224.7	209.5	224.7	209.6	2.8	3.3	2.6	-
J2800	J2700	CIP J-2	140	18'' DIA	16	2.3%	211.97	208.74	214.6	212.6	212.6	209.5	212.6	209.6	5.1	6.0	4.7	-
J2900	J2800	-	183	12'' DIA	7	4.3%	219.93	212.13	227.5	214.6	220.5	212.6	220.6	212.6	5.1	6.0	3.2	-
J3000	J2900	-	76	12'' DIA	9	7.0%	225.20	219.93	230.0	227.5	225.7	220.5	225.8	220.6	5.1	6.0	3.0	-
J3100	J3000	-	112	12'' DIA	9	6.9%	233.19	225.50	237.4	230.0	233.6	225.7	233.7	225.8	3.8	4.4	1.8	-
J3200	J3100	-	87	12'' DIA	13	12.1%	243.87	233.34	248.3	237.4	244.3	233.6	244.3	233.7	3.8	4.4	-	-
J3210	J3200	-	126	12'' DIA	9	5.9%	251.38	243.97	256.1	248.3	251.7	244.3	251.7	244.3	1.8	2.1	-	-
J3300	J3200	-	108	12'' DIA	14	16.0%	260.96	243.97	263.9	248.3	261.2	244.3	261.2	244.3	2.0	2.4	-	-
J3400	J3300	-	205	12'' DIA	13	13.9%	289.34	261.11	293.5	263.9	289.6	261.2	289.6	261.2	2.0	2.4	-	-
J3500	J3400	-	279	12'' DIA	7	3.9%	300.34	289.41	305.2	293.5	300.7	289.6	300.7	289.6	2.0	2.4	-	-
J3600	J3500	-	103	12'' DIA	3	0.9%	301.42	300.48	303.6	305.2	302.0	300.7	302.0	300.7	2.0	2.4	-	-
J5000	J0200	ABANDON	126	18'' DIA	--	0.2%	72.11	71.90	76.3	76.9								
J5100_2	J5050	CIP A-3	468	30'' DIA	28	0.5%	73.71	71.55	77.4	75.5	75.0	73.5	75.1	73.8	14.7	16.8	3.3	-
J5200	J5100	-	228	18'' DIA	13	1.4%	77.13	73.84	80.4	77.4	80.1	75.0	80.5	75.1	14.7	16.8	3.1	0.2
J5300	J5200	-	31	18'' DIA	8	0.6%	77.30	77.13	80.6	80.4	80.6	80.1	81.2	80.5	14.7	16.9	3.2	0.6
J5500	J5300	-	55	24'' DIA	68	8.6%	82.00	77.30	85.0	80.6	82.6	80.6	82.6	81.2	11.5	13.1	2.7	-
J5600	J5500	-	122	2' H, 1'BW, 2 SS Channel	128	11.1%	95.50	82.00	97.5	85.0	96.2	82.6	96.3	82.6	11.6	13.1	1.5	-
J5700	J5600	-	30	24'' DIA	40	10.4%	98.60	95.50	100.6	97.5	99.3	96.2	99.4	96.3	11.8	13.3	1.0	-
J5800	J5700	-	112	2' H, 1'BW, 2 SS Channel	131	11.0%	110.85	98.60	116.9	100.6	111.6	99.3	111.6	99.4	12.9	14.1	-	-
J5820	J5800	-	100	12'' DIA	4	1.5%	112.34	110.85	118.4	116.9	112.7	111.6	112.7	111.6	1.3	1.5	-	-
J5900	J5800	-	79	24'' DIA	11	0.3%	111.05	110.85	118.0	116.9	112.3	111.6	112.4	111.6	10.2	11.6	-	-
J6000	J5900	-	99	24'' DIA	11	0.2%	111.37	111.15	116.8	118.0	112.7	112.3	112.8	112.4	10.2	11.6	-	-
J6100	J6000	-	90	24'' DIA	25	1.3%	112.50	111.37	118.7	116.8	113.4	112.7	113.5	112.8	10.2	11.6	-	-
J6110	J6100	-	173	24'' DIA	13	0.3%	113.76	113.18	116.9	118.7	114.7	113.4	114.7	113.5	5.6	6.3	-	-
J6120	J6110	-	167	24'' DIA	12	0.3%	114.32	113.86	117.1	116.9	115.3	114.7	115.3	114.7	5.6	6.3	-	-
J6130	J6120	-	246	12'' DIA	2	0.5%	115.72	114.52	118.3	117.1	121.3	115.3	123.2	115.3	5.7	6.4	2.0	2.0
J6140	J6130	-	42	12'' DIA	2	0.4%	115.97	115.82	118.3	118.3	121.7	121.3	123.7	123.2	3.3	3.7	2.1	2.1
J6160	J6140	-	110	12'' DIA	1	0.1%	116.18	116.07	120.0	118.3	122.6	121.7	124.9	123.7	3.4	3.8	1.8	1.8
J6170	J6160	-	54	12'' DIA	3	0.9%	116.78	116.28	122.7	120.0	123.0	122.6	125.5	124.9	3.4	4.1	1.3	1.3
J6180	J6170	-	242	12'' DIA	10	8.1%	136.32	116.85	145.6	122.7	136.7	123.0	136.8	125.5	3.4	4.1	-	-
J6200	J6100	-	115	24'' DIA	34	2.1%	115.49	113.08	120.0	118.7	115.9	113.4	115.9	113.5	2.4	2.8	-	-
J6300	J6200	-	168	24'' DIA	39	2.9%	120.54	115.59	125.6	120.0	120.9	115.9	120.9	115.9	2.4	2.8	-	-
J6310	J6300	-	169	18'' DIA	17	8.4%	134.60	120.54	139.1	125.6	135.0	120.9	135.0	120.9	2.4	2.8	-	-
J6320	J6310	-	118	12'' DIA	9	7.1%	143.02	134.60	147.2	139.1	143.4	135.0	143.4	135.0	2.4	2.8	-	-
J6330	J6320	-	35	12'' DIA	5	1.6%	143.59	143.02	147.3	147.2	144.1	143.4	144.2	143.4	2.4	2.8	-	-

Table D-1. CIP Hydraulic Model Parameters and Results

System Conduit**							Invert Elevation (ft)		Ground Elevation (ft)		CIP Fut 10 yr Max Water Surface Elevation (ft)		CIP Fut 25 yr Max Water Surface Elevation (ft)		Peak Flow Values at Upstream Node (cfs)*		Duration of Predicted Flooding, 25-yr Design Storm, 36-Hour Model Run (hrs)	
US Node Name	DS Node Name	CIP Name	Length (ft)	Size/Type H = Height, BW = Bottom Width, SS = Side Slope (H:V)	Capacity (cfs)	Slope (%)	US	DS	US	DS	US	DS	US	DS	CIP Fut 10 yr	CIP Fut 25 yr	Future Land Use Condition, Without CIP	Future Land Use Conditions, With CIP
J6331	J6330	-	161	12'' DIA	3	0.7%	144.69	143.64	148.4	147.3	145.4	144.1	145.5	144.2	2.4	2.8	-	-
J6400	J6300	ABANDON	89	18'' DIA	--	3.4%	123.76	120.74	128.4	125.6							-	-
J6500	J6400	ABANDON	176	18'' DIA	--	10.0%	141.82	124.36	145.6	128.4							-	-
J6500_2	J1300	CIP J-1	334	18'' DIA	9	0.7%	141.90	139.55	145.6	146.9	142.9	139.8	143.0	139.9	6.9	8.1	-	-
J6600	J6500	-	140	18'' DIA	9	0.7%	143.04	142.10	146.2	145.6	143.9	142.9	144.0	143.0	5.6	6.6	-	-
J6700	J6600	-	149	15'' DIA	6	0.7%	144.33	143.22	148.1	146.2	145.1	143.9	145.2	144.0	4.1	4.8	-	-
J6800	J6700	-	94	15'' DIA	17	6.8%	150.77	144.36	153.6	148.1	151.2	145.1	151.2	145.2	4.1	4.8	-	-
J6900	J6800	-	190	15'' DIA	12	3.2%	156.97	150.87	161.3	153.6	157.5	151.2	157.5	151.2	4.1	4.8	-	-
J7000	J6900	-	170	15'' DIA	13	3.8%	165.22	158.72	169.0	161.3	165.7	157.5	165.8	157.5	4.1	4.8	-	-
J7100	J7000	-	107	15'' DIA	6	0.8%	166.21	165.32	170.1	169.0	167.0	165.7	167.1	165.8	4.1	4.8	-	-
BASIN K - No Modeled CIPs																		
BASIN L - No Modeled CIPs																		
BASIN M																		
Drains to Clackamas County Stormdrain																		
M0200	M0100	CIP M-1	189	18'' DIA	19	2.8%	283.82	278.50	286.4	0.0	284.4	279.8	284.5	279.8	6.1	7.2	-	-
M0300	M0200	CIP M-1	194	18'' DIA	8	0.5%	284.88	283.92	289.0	286.4	285.9	284.4	286.0	284.5	6.1	7.2	1.6	-
M0500	M0400	CIP M-1	29	18'' DIA	8	0.5%	285.68	285.53	288.7	288.1	286.6	286.5	286.7	286.6	5.3	6.3	2.4	-
M0510	M0500	-	138	9.96'' DIA	2	0.8%	288.27	287.18	289.5	288.7	288.5	286.6	288.7	286.7	0.5	1.1	2.2	-
M0600	M0500	-	151	12'' DIA	5	1.6%	289.30	286.88	291.6	288.7	290.2	286.6	290.6	286.7	4.9	5.2	1.9	-
M0610	M0600	-	148	12'' DIA	5	1.9%	292.24	289.50	293.7	291.6	292.7	290.2	292.7	290.6	1.9	2.3	1.1	-
M0700	M0600	-	94	12'' DIA	3	0.8%	290.16	289.40	291.9	291.6	290.9	290.2	291.1	290.6	3.0*	3.1*	1.8	-
M0700A	M0510	-	134	12'' DIA	3	1.7%	290.61	288.27	291.9	289.5	290.9	288.5	291.1	288.7	0.4*	1.1*	1.8	-
M0800	M0700	-	82	12'' DIA	2	0.2%	290.45	290.26	292.1	291.9	291.8	290.9	292.1	291.1	3.4	4.0	1.9	0.2
M0900	M0800	-	129	12'' DIA	6	2.7%	294.00	290.55	296.0	292.1	294.4	291.8	294.5	292.1	2.4	2.9	0.5	-
M1000	M0900	-	101	12'' DIA	6	2.7%	296.69	294.00	298.7	296.0	297.1	294.4	297.2	294.5	2.4	2.9	-	-
BASIN N																		
Drains to Clackamas County Stormdrain																		
N0200	N0100	CIP N-1	82	36'' DIA	67	1.0%	219.48	218.67	223.2	222.5	222.3	221.6	222.3	221.6	43.9	49.9	1.6	-
N0300	N0200	CIP N-1	454	36'' DIA	67	1.0%	224.22	219.68	231.4	223.2	226.0	222.3	226.2	222.3	42.9	49.9	1.8	-
N0310	N0300	-	165	18'' DIA	34	10.9%	245.63	227.68	249.6	231.4	246.1	226.0	246.1	226.2	6.9	8.1	-	-
N0320	N0310	-	165	15'' DIA	21	10.9%	263.59	245.63	267.4	249.6	264.1	246.1	264.1	246.1	7.1	8.8	-	-
N0330	N0320	-	21	15'' DIA	6	1.0%	263.80	263.59	268.8	267.4	264.9	264.1	265.0	264.1	6.9	8.1	-	-
N0340	N0330	-	84	12'' DIA	5	2.2%	265.60	263.80	270.0	268.8	266.3	264.9	266.4	265.0	4.1	4.8	-	-
N0350	N0340	-	287	12'' DIA	10	8.2%	288.95	265.60	292.7	270.0	289.4	266.3	289.4	266.4	4.1	4.8	-	-
N0360	N0350	-	277	12'' DIA	5	1.7%	293.59	288.95	297.3	292.7	294.1	289.4	294.2	289.4	2.5	3.0	-	-
N0400	N0300	CIP N-1	101	36'' DIA	67	1.0%	225.43	224.42	230.7	231.4	227.0	226.0	227.1	226.2	33.7	39.2	2.3	-
N0402_1	N0400	-	187	12'' DIA	2	0.4%	230.09	229.32	233.0	230.7	235.2	227.0	237.0	227.1	5.6	6.5	2.9	1.8
N0403	N0402	-	300	12'' DIA	12	11.9%	265.55	230.09	270.0	233.0	266.0	235.2	266.0	237.0	4.3	5.0	-	-
N0404	N0403	-	54	12'' DIA	9	6.9%	269.44	265.73	272.8	270.0	269.8	266.0	269.9	266.0	3.1	3.6	-	-
N0405	N0404	-	295	12'' DIA	7	3.7%	280.39	269.47	285.2	272.8	280.8	269.8	280.8	269.9	2.5	2.9	-	-
N0410	N0400	-	67	12'' DIA	4	1.1%	228.85	228.08	231.1	230.7	232.5	227.0	233.8	227.1	8.4	9.9	2.3	1.6
N0420	N0410	-	376	12'' DIA	10	8.4%	260.40	228.95	271.2	231.1	261.1	232.5	261.2	233.8	8.4	9.9	-	-
N0421	N0420	-	226	12'' DIA	4	1.1%	263.36	260.77	266.7	271.2	263.9	261.1	264.0	261.2	2.1	2.5	0.7	-
N0422	N0421	-	187	12'' DIA	11	9.2%	280.67	263.49	283.2	266.7	281.0	263.9	281.0	264.0	2.1	2.5	-	-
N0430	N0420	-	231	12'' DIA	6	2.8%	273.16	266.71	278.2	271.2	273.7	261.1	273.8	261.2	3.8	4.4	-	-

System Conduit**							Invert Elevation (ft)		Ground Elevation (ft)		CIP Fut 10 yr Max Water Surface Elevation (ft)		CIP Fut 25 yr Max Water Surface Elevation (ft)		Peak Flow Values at Upstream Node (cfs)*		Duration of Predicted Flooding, 25-yr Design Storm, 36-Hour Model Run (hrs)	
US Node Name	DS Node Name	CIP Name	Length (ft)	Size/Type H = Height, BW = Bottom Width, SS = Side Slope (H:V)	Capacity (cfs)	Slope (%)	US	DS	US	DS	US	DS	US	DS	CIP Fut 10 yr	CIP Fut 25 yr	Future Land Use Condition, Without CIP	Future Land Use Conditions, With CIP
N0440	N0430	-	177	12'' DIA	9	6.3%	284.41	273.28	286.7	278.2	284.9	273.7	284.9	273.8	3.8	4.4	-	-
N0450	N0440	-	103	12'' DIA	6	2.9%	287.58	284.56	290.4	286.7	288.2	284.9	288.2	284.9	3.8	4.4	-	-
N0460	N0450	-	158	12'' DIA	2	0.5%	288.30	287.58	292.0	290.4	292.0	288.2	292.0	288.2	3.8	4.4	0.0	0.0
N0470	N0460	-	180	12'' DIA	8	4.5%	296.43	288.30	300.9	292.0	296.8	292.0	296.8	292.0	2.2	2.5	-	-
N0500	N0400	CIP N-1	268	36'' DIA	44	0.4%	226.77	225.63	231.5	230.7	228.3	227.0	228.5	227.1	23.6	27.4	2.6	-
N0510	N0500	-	538	18'' DIA	27	6.8%	264.92	228.54	269.2	231.5	265.4	228.3	265.4	228.5	5.2	6.0	-	-
N0520	N0510	-	232	12'' DIA	5	2.4%	270.49	264.99	273.6	269.2	270.9	265.4	270.9	265.4	1.9	2.2	-	-
N0600	N0500	CIP N-1	40	24'' DIA	16	0.5%	227.97	227.77	231.5	231.5	229.3	228.3	229.4	228.5	13.0	15.0	5.0	-
N0700	N0600	CIP N-1	163	24'' DIA	22	1.0%	229.56	227.97	233.0	231.5	230.5	229.3	230.6	229.4	9.9	11.4	5.8	-
N0900	N0800	-	321	12'' DIA	--	0.2%	231.07	230.56	234.0	233.0	231.1	230.5	231.1	230.6	0.0	0.0	7.6	-
N1050_Swale	N0800	CIP N-1	500	2' H, 2'BW, 3 SS Channel	34	0.3%	232.00	230.56	237.2	233.0	233.5	230.5	233.6	230.6	10.0	11.7		-
N1000	N0900	-	134	12'' DIA	--	0.1%	231.25	231.07	233.2	234.0	231.3	231.1	231.3	231.1	0.0	0.0	12.5	-
N1100	N1000	-	122	12'' DIA	11	9.5%	246.55	235.04	249.6	233.2	247.3	231.3	247.4	231.3	10.0	11.5	4.8	-
N1200	N1100	-	173	12'' DIA	3	0.8%	248.01	246.55	250.5	249.6	251.8	247.3	253.0	247.4	5.8	6.6	4.8	1.8
N1300	N1200	-	250	12'' DIA	5	2.1%	253.30	248.01	256.3	250.5	257.8	251.8	260.8	253.0	5.9	6.8	4.5	1.6
N1400	N1300	-	328	15'' DIA	6	0.9%	256.37	253.30	258.4	256.3	260.3	257.8	264.0	260.8	6.1	7.2	4.3	1.6
N1500	N1400	-	113	15'' DIA	15	5.1%	262.07	256.37	265.6	258.4	262.5	260.3	264.6	264.0	3.8	4.5	3.2	-
N1600	N1500	-	68	15'' DIA	11	2.9%	264.07	262.07	271.8	265.6	264.6	262.5	264.8	264.6	3.8	4.5	2.2	-
N1700	N1600	-	175	15'' DIA	7	1.3%	266.26	264.07	272.3	271.8	266.9	264.6	267.0	264.8	3.8	4.5	2.2	-
N1800	N1700	-	102	15'' DIA	12	3.6%	269.91	266.26	275.1	272.3	270.4	266.9	270.4	267.0	3.8	4.5	1.5	-
BASIN O																		
Outfall To Pond	O0400										139.2	139.2	139.3	139.3	33.3	38.7	-	-
O0600	O0400	CIP O-1	263	2' H, 2'BW, 3 SS Channel	209	10.5%	165.67	138.33	168.7	0.0	166.5	139.2	166.6	139.3	31.4	36.5	-	-
O0700	O0600	-	89	36'' DIA	90	1.6%	167.14	165.67	170.8	168.7	168.4	166.5	168.5	166.6	31.4	36.5	-	-
O0710	O0700	CIP O-1	35	24'' DIA	33	2.0%	168.03	167.34	171.7	170.8	169.5	168.4	169.9	168.5	29.2	33.9	0.8	-
O0711	O0710	CIP O-1	47	24'' DIA	33	2.0%	169.17	168.23	172.5	171.7	170.6	169.5	170.8	169.9	27.3	31.7	10.1	-
O0712	O0711	-	67	12'' DIA	6	2.4%	171.95	170.37	174.5	172.5	172.7	170.6	172.8	170.8	5.1	6.0	5.8	-
O0713	O0712	-	212	12'' DIA	7	3.7%	179.96	172.05	183.7	174.5	180.5	172.7	180.6	172.8	4.6	5.4	1.2	-
O0714	O0713	-	244	12'' DIA	6	2.3%	185.99	180.36	189.5	183.7	186.7	180.5	186.8	180.6	4.6	5.4	0.6	-
O0715	O0714	-	235	12'' DIA	7	3.8%	194.94	185.99	199.0	189.5	195.5	186.7	195.6	186.8	4.6	5.4	-	-
O0720	O0710	-	96	12'' DIA	5	1.7%	171.37	169.71	173.9	171.7	171.8	169.5	171.9	169.9	1.9	2.2	-	-
O0730	O0720	-	39	12'' DIA	5	1.6%	172.04	171.42	175.5	173.9	172.5	171.8	172.5	171.9	1.9	2.2	-	-
O0740	O0730	-	140	12'' DIA	7	3.8%	177.43	172.14	181.3	175.5	177.8	172.5	177.8	172.5	1.9	2.2	-	-
O0750	O0740	-	219	12'' DIA	6	2.4%	182.80	177.53	186.9	181.3	183.2	177.8	183.2	177.8	1.9	2.2	-	-
O0800	O0700	-	11	36'' DIA	84	1.7%	167.33	167.14	171.0	170.8	168.4	168.4	168.5	168.5	0.8	1.0	-	-
O0900	O0800	-	36	18'' DIA	17	2.9%	168.38	167.33	171.4	171.0	168.6	168.4	168.6	168.5	0.8	1.0	-	-
O1000	O0900	-	10	18'' DIA	17	2.9%	168.98	168.68	171.9	171.4	169.2	168.6	169.2	168.6	0.8	1.0	-	-
O1100	O1000	-	39	18'' DIA	21	4.3%	170.85	169.18	173.1	171.9	171.1	169.2	171.1	169.2	0.8	1.0	-	-
O1300	O0711	CIP O-1	278	24'' DIA	69	9.2%	194.73	169.37	198.7	172.5	195.5	170.6	195.6	170.8	22.2	25.8	4.8	-
O1400	O1300	CIP O-1	9	24'' DIA	33	2.2%	194.93	194.73	198.2	198.7	196.1	195.5	196.2	195.6	20.9	24.2	5.1	-
O1500	O1400	CIP O-1	26	24'' DIA	50	4.8%	196.39	195.13	200.4	198.2	197.3	196.1	197.4	196.2	20.9	24.2	4.3	-
O1600	O1500	CIP O-1	105	24'' DIA	54	5.6%	202.50	196.59	206.5	200.4	203.4	197.3	203.4	197.4	20.9	24.2	5.7	-
O1700	O1600	CIP O-1	88	24'' DIA	56	6.2%	208.16	202.70	212.2	206.5	209.0	203.4	209.1	203.4	20.9	24.2	5.7	-
O1800	O1700	CIP O-1	143	24'' DIA	27	1.5%	210.44	208.36	214.1	212.2	211.8	209.0	211.9	209.1	20.9	24.2	12.5	-
O1810	O1800	-	264	12'' DIA	7	3.4%	220.76	211.77	222.8	214.1	221.3	211.8	221.3	211.9	3.1	3.7	5.5	-
O1900	O1800	CIP O-1	373	24'' DIA	17	0.5%	212.49	210.44	217.6	214.1	214.0	211.8	214.2	211.9	15.3	17.7	10.0	-
O1910	O1900	CIP O-1	44	18'' DIA	9	0.7%	213.00	212.69	217.6	217.6	214.4	214.0	214.8	214.2	10.1	11.6	10.3	-

Table D-1. CIP Hydraulic Model Parameters and Results

System Conduit**							Invert Elevation (ft)		Ground Elevation (ft)		CIP Fut 10 yr Max Water Surface Elevation (ft)		CIP Fut 25 yr Max Water Surface Elevation (ft)		Peak Flow Values at Upstream Node (cfs)*		Duration of Predicted Flooding, 25-yr Design Storm, 36-Hour Model Run (hrs)	
US Node Name	DS Node Name	CIP Name	Length (ft)	Size/Type H = Height, BW = Bottom Width, SS = Side Slope (H:V)	Capacity (cfs)	Slope (%)	US	DS	US	DS	US	DS	US	DS	CIP Fut 10 yr	CIP Fut 25 yr	Future Land Use Condition, Without CIP	Future Land Use Conditions, With CIP
01911	01910	-	157	12'' DIA	8	5.4%	222.72	214.18	224.7	217.6	223.0	214.4	223.0	214.8	1.3	1.5	6.3	-
01920	01910	CIP O-1	46	18'' DIA	9	0.7%	213.53	213.20	216.4	217.6	214.7	214.4	215.2	214.8	8.8	10.1	11.8	-
01921	01920	-	41	12'' DIA	4	1.1%	214.82	214.38	216.5	216.4	215.3	214.7	215.4	215.2	1.9	2.2	11.6	-
01922	01921	-	85	12'' DIA	7	3.6%	218.07	214.97	221.0	216.5	218.4	215.3	218.5	215.4	1.9	2.2	7.8	-
01930	01920	CIP O-1	294	18'' DIA	9	0.7%	215.81	213.73	218.1	216.4	216.8	214.7	216.9	215.2	6.9	7.9	11.4	-
01931	01930	-	189	12'' DIA	1	0.0%	215.90	215.81	216.9	218.1	218.2	216.8	218.6	216.9	3.1	3.4	22.6	5.9
01932	01931	-	223	1' H, 1'BW, 2 SS Channel	2	0.0%	215.99	215.90	217.0	216.9	221.4	218.2	224.0	218.6	2.3	2.7	22.8	5.6
01933	01932	-	232	12'' DIA	2	0.4%	216.88	215.99	218.2	217.0	221.8	221.4	224.4	224.0	2.5	2.9	21.7	4.7
01940	01930	-	247	12'' DIA	4	1.3%	219.57	216.41	223.1	218.1	220.4	216.8	223.1	216.9	3.9	4.5	8.2	0.0
01950	01940	-	280	12'' DIA	4	1.1%	222.53	219.59	226.6	223.1	223.0	220.4	223.1	223.1	1.9	2.2	7.0	-
01960	01950	-	84	12'' DIA	6	3.2%	225.18	222.53	227.4	226.6	225.6	223.0	225.6	223.1	1.9	2.2	6.8	-
01970	01960	-	146	12'' DIA	5	2.1%	228.49	225.48	230.0	227.4	228.9	225.6	229.0	225.6	1.9	2.2	5.8	-
02000	01900	-	283	12'' DIA	6	3.0%	222.84	214.29	225.2	217.6	223.5	214.0	223.7	214.2	5.2	6.1	6.3	-
02100	02000	-	29	12'' DIA	6	2.7%	223.77	222.98	226.0	225.2	224.3	223.5	224.4	223.7	3.5	4.1	6.1	-
02110	02100	-	143	12'' DIA	8	5.6%	231.80	223.87	234.1	226.0	232.2	224.3	232.2	224.4	2.5	3.0	3.3	-
02120	02110	-	200	8.04'' DIA	4	8.1%	248.35	232.11	249.6	234.1	248.8	232.2	248.8	232.2	2.5	3.0	-	-
02130	02120	-	102	8.04'' DIA	3	7.1%	255.65	248.42	256.9	249.6	256.1	248.8	256.2	248.8	2.5	3.0	-	-
02140	02130	-	47	8.04'' DIA	4	9.8%	260.27	255.70	260.9	256.9	260.7	256.1	260.7	256.2	2.5	3.0	-	-
02150	02140	-	343	0.67' H, 1'BW, 1 SS Channel	8	12.1%	301.54	260.27	303.5	260.9	301.9	260.7	301.9	260.7	2.5	3.0	-	-
02160	02150	-	104	12'' DIA	3	0.5%	302.06	301.54	304.4	303.5	302.6	301.9	302.7	301.9	1.7	1.9	-	-
02170	02160	-	518	12'' DIA	2	0.4%	304.00	302.06	306.5	304.4	304.7	302.6	304.7	302.7	1.7	1.9	-	-
02200	02100	-	388	12'' DIA	4	1.0%	227.56	223.87	231.2	226.0	227.9	224.3	227.9	224.4	1.0	1.1	4.5	-
02300	02200	-	127	12'' DIA	3	0.7%	228.52	227.66	232.3	231.2	228.9	227.9	228.9	227.9	1.0	1.1	3.9	-
BASIN P																		
Outfall	P0100										134.1	134.1	134.2	134.2	3.6	4.2	-	-
P0110	P0100	-	76	12'' DIA	13	10.6%	141.77	133.77	144.9	136.8	142.0	134.1	142.0	134.2	1.0	1.2	-	-
P0200	P0100	-	105	12'' DIA	4	1.0%	134.84	133.77	136.5	136.8	135.5	134.1	135.5	134.2	2.5	3.0	-	-
P0400	P0200	-	86	12'' DIA	3	0.7%	135.54	134.94	139.9	136.5	136.2	135.5	136.3	135.5	2.5	3.0	-	-
P0500	P0400	-	59	12'' DIA	4	1.5%	136.51	135.59	138.4	139.9	137.1	136.2	137.1	136.3	2.5	3.0	-	-
P0600	P0500	-	32	12'' DIA	5	1.6%	137.01	136.51	139.6	138.4	137.4	137.1	137.4	137.1	1.4	1.6	-	-
BASIN Q - No Modeled CIPs																		
BASIN R - No Modeled CIPs																		
BASIN S - No Modeled CIPs																		
BASIN T - No Modeled CIPs																		
BASIN U - No Modeled CIPs																		
BASIN V - No Modeled CIPs																		
BASIN X - No Modeled CIPs																		
BASIN Z - No Modeled CIPs																		

** Maximum flow values were modified in instances where two pipes share the same US node. In these cases maximum flow is provided for the conduit. All other maximum flow values pertain to the US node.*

***Existing node names were maintained in CIP development. Cost estimates assume replacement of any affected existing node and the addition of new nodes when needed. Invert elevations and conduit size reflect preliminary design of the CIPs.*