Kennedy/Jenks Consultants
200 S.W. Market St. Suite 500
Portland, Oregon 97201
503-423-4000
503-295-4901 (Fax)

City of Canby
Stormwater Master Plan

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Prepared for

City of Canby
111 NW 2nd Avenue
Canby, Oregon 97013

K/J Project No. 1291029.00

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List of Acronyms

%	Percent
“	inch
BMP	best management practice
cfs	cubic feet per second
CIP	Capital Improvement Project
City	City of Canby
County	Clackamas County
Ct	Court
CWS	Clean Water Services
DMA	Designated Management Agency
Dr	Drive
EDL	Effluent Discharge Limits
ENR CCI	Engineering News Record Construction Cost Index
Ft^2	Square feet
GIS	Geographic Information System
GWPD	Groundwater Protectiveness Demonstration
HDPE	high-density polyethylene
HGL	Hydraulic Grade Line
Hwy	Highway
ID	identification
Kennedy/Jenks	Kennedy/Jenks Consultants
LF	Lineal foot
LID	low impact development
LS	Lump sum
Metro	The elected regional government for the Portland metropolitan area.
N	North
NCCF	non-construction cost factor
NE	Northeast

List of Acronyms (con't)

NW	Northwest
O&M	Operation and Maintenance
ODOT	Oregon Department of Transportation
ORDEQ	Oregon Department of Environmental Quality
Pl	Place
PVC	polyvinyl chloride
Rd	road
SCS	Soil Conservation Service
SDMP	Storm Drainage Master Plan
SSA	Storm and Sanitary Analysis
St	Street
SWMM	Stormwater Management Model (United States Environmental Protection Agency)
SWMP	Stormwater Master Plan
USGS	U.S. Geological Survey
UGB	urban growth boundary
UIC	underground injection control
WPCF	Water Pollution Control Facility
WQF	water quality flow
WQV	water quality volume

Section 1: Executive Summary

1.1 Purpose

This Stormwater Master Plan (SWMP) update was prepared to evaluate the City of Canby's (City) storm drainage system. The master plan is a key component in the decision process for both continued maintenance and improvements to the storm drainage system. This master plan is an update to the previous 1994 master plan. The scope of this document includes an evaluation of the existing stormwater system which is comprised of six collection systems with direct surface discharges, approximately 400 drywells, also known as underground injection controls (UICs), and one existing detention pond. Conceptual sizing of a proposed treatment wetland is also included as part of this document. The analysis of the existing storm drainage system, including runoff and conveyance modeling, as well as condition assessment, was used to compile a list of Capital Improvement Projects (CIP) for the City to use for budgeting purposes.

1.2 Study Area and Population

The storm drainage system serves nearly all of the homes and businesses within the City limits as well as minor developed areas outside the City's limits but within the urban growth boundary (UGB). The study area generally corresponds to the area within the City limits that is currently developed, plus consideration of additional drainage basins that flow into the City limits along North (N) Maple Drive (Dr), and into Willow Creek. Population estimates were developed using the annual population estimates prepared by Metro. The City of Canby's population is expected to approximately double over the 20-year time frame of this stormwater management plan. Based on this population projection, the majority of the area designated as residential in the comprehensive plan is expected to be developed, and future development was modeled as such.

1.3 Planning Projections

Storm flow simulations were primarily based on current land use. The comprehensive plan zoning map from the 2007 City Comprehensive Plan was used to determine future land use projections for system modeling and evaluation. Because City design guidelines require that private development not increase offsite discharge rates and volumes over pre-development levels, the City's storm drainage systems are generally only managing runoff from the right of way. This design requirement was adhered to when determining basin area for stormwater runoff modeling purposes. In general, it is assumed that future development in the northwestern part of the City and south of Highway (Hwy) 99E will not contribute additional surface discharges, but some areas where new street development is anticipated in the vicinity of Northeast (NE) Territorial Road (Rd) were added to the future surface drainage system model.

1.4 Conveyance System Analysis

Current conveyance system conditions were attained and documented through anecdotal information provided through meetings with the City of Canby Public Works employees and other City staff and through the City's existing Geographic Information System (GIS) data. In general, this existing system appears to be in good condition. However, City staff noted specific locations where flooding occurs or where the pipes are in poor conditions. This is detailed on Table 3.6.

Current and future system requirements have also been evaluated through hydrologic and hydraulic simulation. Generally, the modeling demonstrated that most of the existing surface drainage systems have adequate capacity, with the exception of the Canby Downtown system. The Canby Downtown system drains along NW 2^{nd} Ave and the model showed surcharging and flooding in the system due to shallow pipe slopes, reverse pipe slopes, and undersized pipes. The conveyance system analysis identified required pipe sizes to address this issue and for anticipated future systems, and these are included in the CIPs.

1.5 UIC Evaluation

The City of Canby's stormwater system includes 384 UICs. A retrofit analysis of these UICs, including groundwater mapping, modeling and risk assessment, was completed to determine which UICs can be brought into compliance through protectiveness modeling, and identifying retrofits for the UICs that are out of compliance. The evaluation of the UIC's can be found in Appendix C. The majority of the City's UIC's (357) appear to be functioning well and through modeling have been demonstrated to be protective of groundwater quality.

A total of eight UICs were identified as out of compliance and requiring decommissioning and retrofit, either through groundwater modeling, proximity to potential pollutant sources, or because they are not in use. The City has identified six UIC s that exhibit failure characteristics such as draining slowly or flooding, and seven UICs that have wet feet (permanent water at the bottom) and these are also addressed through CIPs. An additional 20 UICs were identified as having potential high groundwater. These UICs were demonstrated through modeling to be protective of groundwater quality and do not need to be decommissioned, although they have some higher potential risk than UICs that are not connected to groundwater.

1.6 Capital Improvement Program

Current and future conveyance capacity has been evaluated using Autodesk Storm and Sanitary Analysis 2012 (SSA). Flow projections for existing land use and buildout conditions indicate capacity issues for both the short term (e.g. 0-5 years) and long term (e.g. 5 -20 years) of the system. A total of 24 CIPs have been developed based on the current and future needs of the City. These CIPs are ranked as high, medium, or low priority based on the assessed urgency of need. High priority CIPs are to be completed in a 0 to 5 year timeframe, medium priority in 6 to10 years and low priority in 11 to 20 years. Recommendations for storm system improvements are compiled into the CIP as shown in Table 1.1. For a full description of the CIP projects, see Section 6.

Table 1.1 Capital Improvement Project Summary Sheet

CIP #	Project	Estimated Cost	Priority
1	N Baker Dr.	$180,000	High
2	NW 10th Ave. from N Locust St. to N Pine St.	$1,330,000*	High
3	SE Hazeldell Way	$90,000	High
4	SW 13th Ave near Canby High School	$30,000	High
5	UIC E-8 and E-11 Decommission	$50,000	High
6	NW 2nd Ave and N Ivy St. UIC Decommission	$40,000	High
7	Cinema Parking Lot UIC Decommission	$40,000	High
8	S Ivy St	$730,000*	Medium
9	N Maple St. at Maple St. Park	$30,000	Medium
10	N Maple St. and NW 34th Pl.	$30,000	Medium
11	NW 13th Ave from N Ash St. to N Birch St.	$30,000	Medium
12	NW 9th Ave from N Ash St. to N Birch St.	$420,000	Medium
13	N Knights Bridge Rd	$130,000	Medium
14	NW 2nd Ave from N Cedar St. to NW Baker Dr	$690,000	Medium
15	NW 3rd Ave from N Cedar St. to. N Grant St.	$670,000	Medium
16	N Holly St.	$310,000	Medium
17	N Juniper St. and NE 5th Ave	$30,000	Low
18	N Baker St. and N Alder St.	$30,000	Low
19	N Cedar St.	$10,000	Low
20	S Pine St. and SE 2nd Ave	$30,000	Low
21	Police Station/NW 3rd Ave Pond	$30,000	Low
22	Fish Eddy Wetland Flow Monitoring	$30,000	Low
23	Fish Eddy Wetland	$670,00	Low
24	Knight's Bridge Runoff Treatment	$50,000	Low
1-24	**All Capital Improvement Projects**	**$5,680,000**	**TOTAL**
1-7	High Priority Projects	$1,760,000	High
8-16	Medium Priority Projects	$3,040,000	Medium
17-24	Low Priority Projects	$880,000	Low

Other Costs for Budgeting Purposes

24	Comprehensive Survey of Existing System	$10,000/yr	Other
25	Operation and Maintenance (O&M) Manual	$30,000	Other
26	System Flow Monitoring	$10,000/yr	Other

* This CIP or a portion of this CIP is within Clackamas County's jurisdiction and is not the responsibility of the City of Canby until the City assumes responsibility for the road and stormwater system.

Section 2: Introduction

2.1 Background

The City of Canby completed a Storm Drainage Master Plan (SDMP) in 1994. However, due to changing conditions and regulations, particularly with respect to UICs, the 1994 plan is considered outdated and an updated Stormwater Master Plan is required. In addition, Canby is a Designated Management Agency (DMA) and a new SWMP is mandated in the implementation manual. The City expects to receive an individual UIC Water Pollution Control Facility (WPCF) permit from Oregon Department of Environmental Quality (ORDEQ) in late 2013 and has the goal of adopting the SWMP before issuance of that permit.

2.2 Authorization

On 15 January 2013, Kennedy/Jenks Consultants (Kennedy/Jenks) was authorized by the City of Canby to update the City's existing Storm Drainage Master Plan.

2.3 Purpose for Study

The purpose of this current SWMP is to analyze the City of Canby's existing stormwater system for current hydraulic adequacy, as well as to model and analyze the system hydraulic performance under existing and future land use scenarios. Identified existing hydraulic deficiencies include known areas of recurring flooding and pipes identified as in need of maintenance, undersized, or in unknown condition. This updated SWMP is intended to aid the City in properly planning and budgeting for needed improvements to address key issues required in maintaining an effective and efficient storm drainage system, as well as meeting future needs.

2.4 Scope of Work

The scope of work needed to accomplish this study was defined by the City of Canby and Kennedy/Jenks' project team. These objectives aided in guiding the project and were used as a measure of project accomplishment. The scope of work included:

- Gather and review stormwater system inventory data pertinent to this study.
- Review existing system data including the existing system map.
- Describe the existing stormwater system as well as an inventory of known problem areas.
- Determine the capacity of the existing detention pond at the west end of NW 3rd Ave.
- Update and develop the system service areas and basin boundaries.
- Develop a hydrologic and hydraulic model of the capacity of the existing collection system that discharges to surface water. Develop flow projections for the existing and future

conditions for the six discrete drainage basins where stormwater is discharged to surface water.

- Calculate the required size of a treatment wetland proposed on the Fish Eddy property at the Willow Creek outfall.
- Identify and prioritize capacity deficiencies and needs.
- Groundwater depth modeling and protectiveness demonstration.
- Identify UICs that require retrofit and list the proposed retrofit solution for each of these UICs.
- Develop a CIP identifying recommended capital improvements and estimated project costs for a 20-year planning period.
- Prepare a Stormwater Master Plan report summarizing the findings and recommendations.

Section 3: Study Area Characteristics

3.1 General

The City of Canby is located 25 miles south of Portland, Oregon in the northern Willamette Valley. The City is characterized by near level terrain, on a low terrace between the Willamette River to the north and the Molalla River to the south and west. The City covers approximately 2,084 acres within City limits, and approximately 3,384 acres within the UGB. The 2010 census estimates 15,829 citizens in the City.

Much of the text in Sections 3.1 and 3.2 is taken directly from the 1994 City of Canby Storm Drainage Master Plan completed by Curran-McLeod, Inc. Consulting Engineers [SDMP 1994}. The text has been edited and updated, where appropriate.

3.1.1 Economy

The City lies in the heart of very productive agricultural lands and accordingly, the City's economic growth has traditionally been tied to local agriculture. The study area is surrounded by large farm tracts which produce a variety of products, most notably row crops, fruits, nuts, and berries, as well as nursery stock. More recently, the City has experienced growth due to the demand for housing from the nearby metropolitan area.

Economic development resulting in employment opportunities in the urban area of the City has been slow. This is due, in part, to the demographics of the population and the relatively close location of other larger employment centers which encourages commuting. The City has taken steps towards diversification into commercial, industrial, professional, and other service related opportunities to actively promote employment opportunities within the City.

Canby's small town, rural atmosphere also attracts individuals desiring to escape more congested urban areas. This is reflected by the fact that approximately 75 percent of the working residents in Canby commute to nearby communities such as Wilsonville, Salem or the Greater Portland area for employment. Again, a contributing factor to this phenomenon is the lack of locally available employment opportunities and the willingness of individuals to commute from the Canby area to other communities.

3.2 Existing Environment

3.2.1 Geography

The City of Canby is located in the Willamette Valley which is the population center of the State of Oregon. The Willamette Valley is nestled between the Coast and Cascade Mountain Ranges and is from 20 to 40 miles wide and 130 miles long extending from Eugene-Springfield in the south to Portland in the north [SDMP 1994a].

The Canby area is bordered to the east and south by the Molalla River and to the north by the Willamette River. The Molalla River converges with the Willamette River northwest of the City.

The Willamette River is the major waterway in the Willamette Valley draining a total of 11,200 square miles [SDMP 1994a]. The Willamette provides significant recreational opportunities, serves as a major transportation link, provides water for agricultural, municipal and industrial uses and provides habitat for significant wildlife populations.

The City is accessible from the Interstate 5 corridor and Interstate 205 via state Hwy 99E. The transportation system provides direct connections to the surrounding communities of Oregon City, Woodburn, and Salem as well as the greater Portland metropolitan area. The City is also on the main north-south Union Pacific Rail line.

3.2.2 Topography

The community of Canby is located on a relatively flat terrace overlooking the Willamette and Molalla Rivers. With few exceptions, only gentle changes in the topography of less than 30 feet occur within the City limits and UGB (between 120 to 170 feet above mean sea level). This can be easily observed throughout the residential, industrial and commercial districts.

South and West of the City, the landscape drops abruptly at the Molalla River to an elevation of approximately 80 feet. This is very evident when approaching the City from the west on Knights Bridge Rd or Hwy 99E or from the south across Goods Bridge on Ivy Street (St). This drop along the Molalla River establishes a natural boundary for the area.

At the northern extreme of the UGB, where the UGB extends to the Willamette River, the topography gradually changes. This area slopes to the Willamette River, dropping from an elevation of approximately 130 feet to an elevation of 100 feet at the City's wastewater treatment facility.

To the east of Canby, the topography changes very little until beyond the UGB, where the ground has undulating gentle hills in the southeastern areas and steep rocky cliffs in the northeastern areas along the Willamette River.

3.2.3 Geology

The area we know today as the Willamette Valley was at one time part of the Oligocene ocean continental shelf extending from the cascades to beyond the current coast line. With uplifting of the Coast Range during the Miocene epoch, the trough like shape of the Willamette Valley was created. Later, during the Miocene, basaltic lava from fissures and vents in northeastern Oregon poured into the Willamette Valley where they reached as far south as Salem [SDMP 1994a]. Volcanic activity was renewed along the western edge of the Cascade Mountains at the close of the Miocene epoch, culminating in the deposition of the Sardine Formation. This formation is composed of agglomerates, flow breccias, mud flows, and lavas.

Low lands and lakes were formed by folding that occurred during the Pliocene. The low lands and lakes later filled with sediments from upland areas. These sediments, the Sandy River Mudstone and overlaying Troutdale Formation were then subjected to warping and subsequent erosion. Also, during the Pliocene, volcanism preceded the viscous lavas, agglomerates, and flow breccias of the Boring Lava. In present-day areas of Clackamas County (County) where these lava flows are exposed and thin, weathering has nearly completely altered the lava flows to red clay soils.

Following the Boring Lava event, gravel and finer sediments were deposited in the Portland area. Glacial floods produced channeled scab lands and deltaic deposits of torrentially cross-bedded gravels. Continued erosion by Willamette Valley river systems, earthquakes and weathering of the area through geologic time, continued to carve and shape the Willamette Valley into its shape today. In the Canby area, this has resulted in a terrace overlooking the Willamette and Molalla Rivers.

The Oregon Department of Geology and Mineral Industries identified the prominent Surficial Geologic Units in the Canby Area [SDMP 1994b]. The results of the study were published in 1979. From the study, there are predominantly Alluviums and Lacustrine Sediments underlying the Canby urban area. The Lacustrine Sediments are unconsolidated cross-bedded to graded sedimentary beds deposited by the late Pleistocene glacial floods. There are three classifications of these sediments: Qws, Qls and Qdg. The characteristics of the Alluvium and Lacustrine Sediments are identified in Table 3.1 and the approximate limits of each of these series are depicted in Figure 1.

Table 3.1 Surficial Geologic Units

Classification	Characteristics
	Lacustrine Sediments
Qws	Lacustrine fine sandy silt and clay deposited up to 350 feet elevation. Beds range from a few inches to several feet thick. Occurs along the valleys of the Tualatin and other tributaries of the Willamette River.
Qls	Lacustrine and fluvial unconsolidated stratified to cross bedded sand and silt and occasional lenses of pebbles to gravel. Occurs along the Willamette River. Equivalent to lacustrine sands (Qls) and sand and silt deposits (Qs).
Qdg	Deltaic deposits of sand, gravel, and boulders up to 8 feet in diameter; torrential cross-bedding.
	Alluvium
Qal/Pt	Unconsolidated sand, gravel, and cobbles within stream channels and on adjacent flood plains; sandy silt up to 10 feet thick overlies gravel on flood plains.

In general, the Lacustrine Sediments are well drained when on terraces. However, when located in concave areas and within drainage ways, they are poorly drained. From Figure 1 it can be seen that the Alluvium is located adjacent to the Willamette River. This area has, in the past, been subject to flooding and has a high groundwater table. Thus, the Alluvium surficial unit is anticipated to be poorly drained during winter wet weather periods.

Additional description of the existing local geology underlying the City of Canby is provided in GSI's report "Groundwater Protectiveness Demonstrations and Risk Prioritization for Underground Injection Control (UIC) Devices" attached as Appendix C. This describes the geology where the City's UICs are located as the "coarse-grained facies of the catastrophic flood deposits (unit Qfc)."

3.2.4 Soils

United States Department of Agriculture, Soil Conservation Service (SCS) information was used in identifying soil types in Canby. The approximate extent of each soil type is shown in Figure 2. SCS classifications indicate the effects the soil features will have on the infiltration and drainage of stormwater in the Canby area [City of Canby, 2012]. Soil characteristics were used to calculate infiltration rates during modeling of pervious areas.

As can be seen from Figure 2, the predominate soils are Canderly Sandy Loam (12A) and Latourell Loam (53A, 53B, and 53C) with small inclusions of Amity Silt loam (3) and McBee Silt Clay Loam (56), Newberg Fine Sandy Loam (67), Newberg Loam (68), Willamette Silt Loam (68A) and Xerochrepts and Haploxerolls (92F).

3.2.4.1 Canderly Sandy Loam, (12A)

The Canderly Series consists of deep somewhat excessively drained soils on terraces to a depth of 60 inches or more. These soils have moderately rapid permeability of from 2.0 to 6.0 inches per hour and have an available water capacity of about 5.5 to 7.0 inches. Runoff from this series is slow and the likelihood of erosion is slight. Depth to ground water is greater than 60 inches. This soil series is of a capability Class II.

3.2.4.2 Latourell Loam Series (53A, 53B and 53C)

Latourell Loam Series are deep well drained soils, having a moderate permeability of from 0.6 to 2.0 inches per hour in the first 48 inches of the soil to a permeability similar to that of the Canderly soils of about 2.0 to 6.0 inches at depths greater than 48 inches. These soils are typically more than 60 inches deep. Latourell soils have greater water availability than that of the Canderly Series with a range of about 8 to 12 inches. Runoff from this series is also slow and the potential for erosion is again slight. Depth to groundwater is generally greater than 60 inches. This soil has capability rating Class I.

3.2.4.3 Amity Silt Loam (3)

Only two small inclusions of Amity Silt Loam are present in the Canby area. These are somewhat poorly drained soils having a moderately slow permeability of from 0.6 to 2.0 inches per hour in the upper layer declining from 0.2 to 0.6 inches per hour in the lower. This soil can extend to a depth of 60 inches or more. The Depth to Water Table is from 6 to 18 inches in the winter and spring. Capability rating for this soil type is a Class II.

3.2.4.4 McBee Silty Clay Loam (56)

The McBee Silt Clay Loam is a deep moderately well drained soil having a moderate permeability of from 0.6 to 2.0 inches per hour. Runoff is slow and erosion is unlikely. However, the water table is generally at a depth of from 24 to 36 inches in the winter and early spring and the soils are located in areas subject to occasional but brief periods of flooding in the winter.

Only one isolated location northeast of Canby is characterized as this soil type. Capability rating for this series is Class II.

3.2.5 Groundwater Supplies

The Canby area is bounded on three sides by river basins that are at slightly lower elevations. As a result, groundwater is plentiful and accessible at relatively shallow depths. Due to the topography and soil composition, groundwater is estimated to be similar to the pattern documented for the Willamette River Valley. This indicates that there is an unconfined shallow aquifer that varies dramatically with the seasonal weather, and a much deeper confined aquifer with higher quality water supplies. There are a substantial number of domestic wells within the Canby area. The majority of these wells exceed 100 foot depths, although numerous wells are logged at as little as 30 feet to 40 feet deep. Well logs indicate a confining barrier is present, varying in depth but less than 100 feet below the ground surface. Refer to the report “Groundwater Protectiveness Demonstrations and Risk Prioritization for Underground Injection Control (UIC) Devices” prepared by GSI and attached as Appendix C for more detailed analysis of groundwater at the City of Canby.

3.2.6 Climate

The climate in the study area is temperate. Summers are warm with daily temperatures averaging from 70 to 75 degrees with occasional hot days where the temperature exceeds 100 degrees. Winters are cool with average daily temperatures of about 40 degrees Fahrenheit. Freezing temperatures occur periodically throughout the winter with lows in the teens [SDMP 1994c].

Due to the temperate climate, snowfall is rare, but can occur annually. Accumulations of from 1 to 4 inches of snow have been recorded during December and January [SDMP 1994c]. Unofficially, accumulations of as much as 12 inches of snow have been reported in the Canby area for short periods of time.

Normal rainfall, as recorded at the North Willamette Experimental Station, for the period from 1961 through 1990, averages 41 inches per year [SDMP 1994d]. The driest months of the year are July and August, with normal precipitation of less than one inch per month. During these two months, several weeks may pass without precipitation. The balance of the year, frequent rain showers occur with the wettest months being November, December and January. During these three months, rainfall normally exceeds six inches per month.

3.2.7 Flooding (Risk Analysis)

Due to Canby's natural terrace above the major waterways and the City's isolation from larger contributing area, flooding from the rivers is a relatively infrequent occurrence. Flooding has occurred for short periods of time in the lower elevations along the Willamette River, Molalla River contributing streams near the City limits, and on Willow Creek at NE Territorial Rd. As a result, access to and from Canby is obstructed; however, the study area is generally not impacted directly.

Numerous areas of poor drainage are documented in this report. These areas of poor drainage can exhibit ponded water during heavy precipitation. These areas are localized and typically occur due to obstruction of drywells or catch basins. An inventory of drywells, identified by the City as exhibiting failure characteristics, is presented in Table 3.2. These drywells may not be performing adequately due to high groundwater, slow infiltration rates, or large contributing drainage basins. Excessive overland flow is generally very limited due to the high permeability and water capacity of the native soils.

Table 3.2 Drywells/UICs Observed to Exhibit Failure Characteristics

Drywell ID #	Location	Condition
No ID	N Baker St and N Alder St	Slow Draining
B-8	NW 9th Ave. and N Ash St.	Slow Draining
B-11	NW 9th Ave. and N Cedar St	Slow Draining
D-61	NE 12th Ave	Slow Draining
A-5	SW 13th Ave and S Cedar	Slow Draining
D-63	NW 10th and N. Pine St	Slow Draining, Standing Water
D-26	NE 12th Pl and N Pine St	Slow Draining, Potential for Standing Water
D-28	NE 11th Pl and N Pine St	Potential for Standing Water
D-23	NE 13th Ave and N Pine St	Slow Draining, Standing water
D-31	NE 10th Ave and N Oak St	Slow Draining, Standing water
D-35	N Maple St at Maple St City Park	Slow Draining, Potential for Standing Water
F-7	N Birch St and NW Territorial Rd	Slow Draining, Potential for Standing Water

Due to the relatively short duration of the impacts of flooding and the minimal risk to property, a low intensity storm is used as the design standard for future stormwater infrastructure sizing. The City of Canby's Public Works design standards require conveyance systems to be designed to pass the 10-year storm event without surcharge, and the 25-year event with surcharge while maintaining the hydraulic grade lines below the manhole lids. The 10-year return frequency design storm is used as the standard for calculating runoff rates and volumes, which is, in turn, used for sizing all conveyance system improvements. The cost escalates dramatically with the selection of a less frequent storm event, as a result of requiring larger size conveyance systems.

Recent flooding events described by the City are generally of localized areas flooding during larger rainfall events and consist of street and intersection flooding. The areas that flood are most commonly associated either with a relatively large drainage basin contributing to a single drywell, or with a drywell that has been identified as having wet feet or being slow draining. In some cases, lack of maintenance of adjacent privately-owned stormwater systems was identified as contributing to flooding. Specific characteristics of areas that flood are described as part of the relevant CIP in Section 6. Flooding generally subsides within approximately six hours, but can last for two or three days and impairs access.

3.3 System Mapping

As part of the SWMP, system maps have been developed using the most current GIS data and incorporating available record drawings. These maps incorporate many aspects central to the Master Planning effort including; roadways, rivers, parcels, comprehensive plan zones, and other local information. Additionally, more stormwater-system specific information was added to the GIS map when available. These additions include existing pipes, manholes, catch basins, UICs, major drainage basins, and subbasins. The existing and future conveyance system maps developed, using the GIS, are the primary source for development of the hydrologic and hydraulic models.

It should be noted that the GIS system is considered a “work in progress”. While a significant amount of information was added to the GIS as part of this project, including pipe sizes, location of UIC’s, manholes and catchbasins, as well as surveyed invert elevations of selected manholes, it is not all inclusive. Further, the information available from record drawings was incomplete. Therefore, as more information is gathered, it should be added to the GIS system and the mapping updated. This is common practice for mapping and GIS systems.

3.4 Service Area and Basin Boundaries

The comprehensive plan, acknowledged in 1984 and updated in 2007, identifies the total area within the UGB as 3,384 acres. The area is divided into three generic land use types: residential, commercial and industrial. Residential land use includes the comprehensive plan designations of low density residential, medium density residential and high density residential acres as well as private residential, public owned, and flood/steep slopes due to the runoff characteristics. The commercial area includes downtown commercial, convenience commercial, residential commercial, highway commercial and commercial manufacturing. Industrial acreage includes light and heavy industrial.

The residential community has a mixture of low, medium, and high density land uses. Low density residential zoning allows for construction of one single family dwelling per lot, whereas medium density allows for construction of duplexes or triplexes on each lot, and high density permits construction of multifamily dwellings, boarding houses, or placement of no more than eight manufactured homes, per acre. Commercial development, in the urban area, has occurred in the existing downtown area and along Hwy 99E. Canby continues to encourage light industrial development within the urban area. However, less than 30 percent of the designated light industrial area has been developed. No heavy industry has been developed in the urban area. All areas of future development that were modeled in this plan as contributing additional stormwater runoff to surface stormwater systems were residential, and the right of way area was assumed to be the same for both low, medium, and high density.

Analysis and modeling for the SWMP includes all area within the UGB draining to City outfalls or UICs. Delineation of specific drainage basins and runoff modeling was completed for six discrete surface drainage outfalls. Areas draining to UICs or surface infiltration locations were not delineated. City design guidelines require that private development not increase offsite discharge rates and volumes over pre-development levels. The City’s storm drainage systems are generally designed to manage runoff from the right-of-way. Due to this design standard, the models of subbasins for analysis were defined primarily based on the right-of-way and assumed to be 100 percent (%) impervious with exceptions noted below.

The general basin boundaries for these major surface basins and the modeled impervious subbasin boundaries are shown in Figure 3. In general, it is assumed that future development in the northwestern part of the City and south of Hwy 99E will not contribute additional surface discharges to City conveyance systems. Areas, where construction of new streets to service future residential development anticipated in the vicinity of NE Territorial Road, were added to the future surface drainage system model.

The basin boundary for N Maple Street includes an upslope agricultural area that drains toward the street and is known to contribute runoff to the City's conveyance system. These agricultural areas were assumed to be 100% pervious crops. Two surface drainage basins for Hwy 99E are shown on Figure 3. These basins contain both impervious and pervious land cover and drain to discrete surface discharge points. These drainage basins were not included in the conveyance system analysis because they are owned and maintained by Oregon Department of Transportation (ODOT). The South Hwy 99E basin drains to the Molalla River and the North Highway 99E basin eventually drains to Willow Creek. The three other drainage basins draining to Willow Creek were not included in the conveyance analysis either, but were defined based on their current land use for the purposes of modeling runoff contributing to the proposed Fish Eddy

3.4.1 Storm Drainage Basins

The following six surface drainage basins with discrete outfalls were delineated and modeled. Current City staff concerns include; current areas experiencing flooding events, ongoing problematic maintenance needs, and areas of expected growth guided the modeling efforts. The model examines the current hydraulic capacity of each system and provides an understanding of future system hydraulics for systems that are anticipated to require additional capacity.

The basin areas presented below represent the total area contributing runoff to the modeled conveyance system. It is generally assumed that all rain falling on private land is infiltrated or otherwise managed onsite. Thus, the basin areas generally correspond to the area within the street right of way, with exceptions noted below. This area is assumed to represent the impervious surface directly connected to surface conveyance systems. The overall basin boundaries and the modeled basin boundaries are presented in Figure 3.

3.4.1.1 Canby Downtown

This storm system drains the downtown commercial area and is the oldest part of the City's stormwater infrastructure. The system discharges to the Molalla River in the western part of the community. The total area of the existing basin as determined using GIS mapping, and as modeled, is 48.9 acres. In addition to the typical right of way areas, this basin also includes the entire developed downtown area between Northwest (NW) 1st Avenue (Ave) and NW 3rd Ave, between NW Elm Street and N Ivy Street. A portion of land located near Wait Park drains to existing UICs and infiltration areas and does not contribute to total basin flow, so it is therefore not included in the basin. The outfall from the Canby Downtown basin is a surface water discharge point via a detention pond, grassy swale, and natural drainage way adjacent to the new police station at the west end of NW 3rd Ave.

3.4.1.2 N Baker Drive

The N Baker Dr basin drains to a pipe outfall that discharges runoff collected from the south end of N Baker Dr, and extends from NW 6th Place (Pl), and NW 6th Ave. The area of the existing basin, as modeled, is 2.1 acres. The outfall is near the steep bank on the Molalla River, upstream of Knights Bridge Road. There is limited data for the conveyance system in this area and there may be additional outfalls that are not currently mapped. CIP #1 addresses known issues in this basin.

3.4.1.3 Knights Bridge Road

This system drains approximately 1500 feet of Knights Bridge Road and immediate adjacent areas of intersecting streets. The total area modeled for the existing basin is 3.5 acres. The outfall for this storm system is an open pipe located underneath the bridge on which Knights Bridge Rd crosses the Molalla River. The pipe is supported by the bridge structure underneath the road way and has a free outfall approximately 30 feet above the Molalla River, above the approximate centerline of the river. The conveyance system includes a Contech Model CDS2015-4 hydrodynamic separator for particulate separation, located near N Ash St at a manhole prior to the discharge point.

3.4.1.4 Southwest Berg Parkway

This conveyance system drains the 3.2 acres of Southwest (SW) Berg Parkway to an outfall near the Molalla River.

3.4.1.5 N Maple Street

This system collects stormwater in a number of UICs connected in series along N Maple Street, which is located in the northern part of the City. Runoff from adjacent agricultural land is also collected at the north end of N Maple Street in a second pipe system that is connected underneath NE 34th Pl. The combined runoff then discharges directly to the Willamette River beyond the end of NE 34th Pl. The impervious area of the basin consists of a total of 5.6 acres, and the agricultural area contributing to the basin is 123 acres. The system discharges through a deeply buried pipe at approximately the water surface elevation of the Willamette River.

3.4.1.6 Redwood/Willow Creek

This system drains N Redwood St and adjacent side streets from NE Territorial Rd to Hwy 99E, as well as a portion of the Molalla Forest Rd, and parts of the residential streets on the east side of N Pine St and NE Territorial Rd from N Maple St to Willow Creek. The existing drainage basin modeled for this report consists of 22.2 acres of impervious. The system's outfall is to Willow Creek on the north side of NE Territorial Road, where it combines with flow from the main stem of Willow Creek and flows onto the Fish Eddy property.

3.4.2 Existing Stormwater System

3.4.2.1 Conveyance System

The existing conveyance systems throughout the City are comprised of gravity storm drainage pipes, open drainage ways or ditches, trench drains, and UICs. The existing stormwater system is shown in Figure 4. Approximately 125,000 feet of storm pipeline has been identified in the system inventory. Approximately half of this pipe drains to surface outfalls and half drains to infiltration areas, including UICs. The City is responsible for slightly less than 108,800 feet of pipe; however, the exact amount is unknown due to the fact that the current mapping does not distinguish private and county owned infiltration structures or minor surface systems from those that are owned by the City. Less than 1.5 percent of the pipelines in the system have documented diameter or pipe material type. Known diameters of existing pipes range from 4 inches to 36 inches. The majority of pipes that have known diameters are between 10 and 24 inches.

Approximately 16,000 feet of pipeline within the City is under the jurisdiction of ODOT or County, and captures and conveys runoff through surface drainage systems on Hwy 99E within City limits. Clackamas County is also responsible for an unknown number of UICs within the City, but these are not clearly defined and the locations are not known exactly. Pipeline that is not the responsibility of the City is not included in modeling in this report. Table 3.3 summarizes pipe in the system.

Table 3.3 Storm System Inventory

Category	Total Length of Pipe (feet)
Within City Limits	124,800
Surface Basins	60,800
Infiltration Systems/Private Systems	64,000
Total City Responsibility (Approximate)	108,800
ODOT/Clackamas County (Hwy 99E and S Ivy St)	16,000

3.4.2.2 Existing Drywells/UICs

The City has identified 384 existing UICs which are shown in Figure 5. Of these UICs, 284 are named and labeled with their identification (ID), and 100 are not named or labeled and are given the ID “None” in the City’s database. The majority of the City’s UIC’s (357) appear to be functioning well and, through modeling, have been demonstrated to be protective of groundwater quality. Refer to the GSI Water Solutions report “Groundwater Protectiveness Demonstrations and Risk Prioritization for Underground Injection Control (UIC) Devices” (GWPD) attached as Appendix C for more detail concerning UICs and groundwater modeling.

A total of eight UICs have been identified for decommissioning and these are listed in Table 3.4. The UICs that the City has observed to exhibit failure characteristics were previously listed in Table 3.2, but not all of these require decommissioning as discussed below. GSI classified six UICs as high risk and requiring decommissioning. The Groundwater Protectiveness

Demonstration (GWPD) modeling identified four high risk UICs that must be decommissioned to protect groundwater quality, due to their vertical and horizontal separation from seasonal high groundwater and a water well, respectively. Two UICs were identified by the City as being located close to potential pollutant sources and these are also classified as high risk and to be decommissioned. Finally, the City identified two additional UICs to be decommissioned based on other reasons. All eight of these UICs are addressed in the CIPs in Section 6. One additional UIC at NE 2nd Ave and N Ivy St is also recommended for decommissioning because of its proximity to a UIC identified as high risk. It has been included in CIP # 7, but not listed in Table 3.4.

Table 3.4: Drywells/UICs to Be Decommissioned

Drywell ID #	Location	Reason
D-63	NW 10th Ave and N Pine St	High Risk – GWPD (wet feet)
None	NE 2nd Ave and N Ivy St	High Risk – GWPD
None	NW 3rd Ave and N Holly St	High Risk – GWPD
D-48	NW 3rd Ave and N Holly St	High Risk – GWPD
E-8	1480 NE Territorial Rd (WWTP)	High Risk – Pollutant Source
E-11	1490 NE Territorial Rd (PW Complex)	High Risk – Pollutant Source
None	NW 2nd Ave Cinema Complex Parking Lot	Not Currently in Use
None	N 1st Ave and N Ivy St	Required to be rebuilt or decommissioned

The City has identified seven UICs that have wet feet (listed in Table 3.2), and groundwater modeling identified an additional 20 UICs that have the potential for wet feet based on the depth below surface of the groundwater table. One of the seven wet feet UICs identified by the City (D-63) is listed above in the high risk category. The remaining are assigned to the moderate risk category because they are close to or directly connected to groundwater. The moderate risk UICs, as demonstrated through modeling, were determined to be protective of groundwater quality but have some higher potential risk than UICs that are not connected to groundwater. ORDEQ does not require decommissioning of these UICs.

3.4.2.3 Stormwater Management Structures

In addition to the conveyance system and UICs, several other stormwater management structures are located within the City, including two detention basins and one Contech Model CDS2015-4 hydrodynamic separator. The City and private developers have implemented low impact development (LID) best management practices (BMPs) in several areas. These practices include pervious pavement, vegetated swales, gravel or rock swales, and rain gardens. These structures and their locations are listed in Table 3.5.

Table 3.5: Stormwater Management Structures

Location	Stormwater Feature/Structure
Central System, NW 3rd Ave	Detention Pond (Police Station Area)
N Knights Bridge Rd at N Ash St	Contech Model CDS2015-4 hydrodynamic separator
N Knights Bridge Rd, Grant Street to N Ash St	Pervious Pavement on North Side of Street
NW Territorial Rd at N Birch St	Swale, Rock, located on the South Side of Territorial Rd and on Both Sides of N Birch St
NW Territorial Rd at N Juniper St	Rock Swale
N Juniper St at N Juniper Pl (S)	Rock Swale
NW 13th Ave between N Ash St and N Birch St	Pervious Pavement , both sides of the street (~100 ft)
NW 9th Ave at N Elm St	250 ft of Pervious Pavement, both sides of the street
NW 9th Ave at N Cedar Ct	Pervious Pavement, both sides of the street
NW Hawthorne Ct at NW 13th Ave	Pervious Pavement
N Baker St at N Alder St	Vegetated Swale
NW 11th Ave N Alder St to N Ash St	Vegetated Swales, Both Sides of Street
NE 11th Ave east of N Ivy St	Pervious Pavement, South Side ~ 1 block
N Locust St at NW 9th Ave	50 ft Pervious Pavement, West Side (w/ Gravel Shoulders)
Auburn Farms Subdivision, NE 22nd Ave at N Locust St	Vegetated Swales and Trench Drains
NW Territorial Rd at N Redwood St	Proposed Dog Park with Pervious Parking Lot
NW Territorial Rd at Willow Creek	Proposed 'Fish Eddy' Constructed Stormwater Treatment Wetland
Fairgrounds Area, NE 4th Ave	Pervious Pavement
Wait City Park	Pervious Pavement in all Parking Surrounding Park
NW 3rd Ave at N Elm St	Pervious Pavement
Apollo Subdivision near NW 1st Ave	Vegetated Swale
NW 1st Ave and N Fir St	Vegetated Swales
NW 8th Place	Pervious Pavement
NE 19th Ave Cul-de-sac	Pervious Pavement
NE 10th Ave at NE Oak Circle	Pervious Pavement
NE Laurelwood Loop	Pervious Pavement
NE Vine St	Pervious Pavement
Canby Cinema NE 2nd Ave at N Knott St	Rain Gardens, Swales on NE 2nd Ave and Pervious Parking Lot
Countryside Living NW 2nd Ave at N Fir St	Rain Gardens
Fred Meyer and Commercial Development S Sequoia Parkway and SE 1st Ave	Retention Pond (private) near Spring Source of Willow Creek, receives overflow from Fred Meyer private drywells and some public right-of-way

Location	Stormwater Feature/Structure
SE Hazeldell	Trench Drains
S Sequoia Parkway at SE Township Rd	Vegetated swales and Trench Drains
S Walnut St	Vegetated swales
S Pine St	Pervious Pavement
SW 13th Ave	Pervious Pavement
NW Territorial Rd at N Redwood St	Proposed Dog Park with Pervious Parking Lot
NW Territorial Rd at Willow Creek	Proposed 'Fish Eddy' Constructed Stormwater Treatment Wetland

3.4.2.4 Known Issues with the Existing System

Existing stormwater system mapping data was reviewed during a meeting with City Public Works employees, the City environmental service manager, and planning staff. This review was used to identify known features, understand the history of the system, and identify issues and problems within the current system. Table 3.6 lists areas identified as having problems and summarizes the issues or known deficiencies. The areas are generally listed starting at Hwy 99E on the west side of the City and proceeding in a clockwise direction.

Table 3.6: Stormwater System Issues and Known Deficiencies

Location	Identified Issues or Deficiencies
End of NW 3rd Ave, near the Police Station	This system is likely to require a redesign when the police station access is paved, but current conditions create no issues
NW 2nd Ave (downtown)	This section pipe is currently is satisfactory condition until N Fir St, downstream of which, it is likely to be undersized and in poor condition
NW 3rd Ave	This section of pipe is in poor condition
NW 6th Ave (downtown)	This 8" pipe is contains large amounts of tree roots, may be undersized, and should be replaced
N Baker Dr, NW Baker St to NW 6th Pl *	40-foot high steep bank of Molalla River is unstable, and the areas surrounding have many different types of pipes. The pipes are shallow and bursting may heave roads. This is a high priority area for repair, and most likely will require connecting the basin to Knights Bridge Rd pipe system.
N Knights Bridge Road, Molalla River to N Aspen Ct	The hydrodynamic separator for this segment of pipe has had de-gritting issues, but otherwise functions well with regular street sweeping 8" concrete pipe in area in poor condition with misaligned joints 8" corrugated pipe in area is in poor condition and deteriorating catch basins tie in to the main line at blind tees flooding runs down Grant St and unplugging the line is difficult
NW 9th Ave at NW Ash Ave	This area incurs flooding due to failure of UICs, with relatively large drainage areas contributing to each UIC. The flooding often requires 6 hours to 3 days to recede. Flooding has currently been mitigated through cleaning of the drywells; however this is not expected to be a permanent solution.
NW 9th Ave at NW Aspen Ct	The area incurs flooding due to failure of UICs, with relatively large drainage area to each UIC. The flooding often requires 6 hours to 3 days to recede. Flooding has currently been mitigated through cleaning of the drywells; however this is not expected to be a permanent solution.
NW 9th Ave at N Holly St	Buried 90 degree pipe bend needs manhole added
NW 9th Ave between N Holly St and N Ivy St	The sedimentation manhole in this area creates flooding
NW 13th Ave between N Ash St and N Birch St	Limited pervious pavement has been constructed in this area, but flooding still occurs along the west side of the street.

Location	Identified Issues or Deficiencies
10th Ave from N Locust St to N Pine St *	Significant flooding currently occurs in this area and is expected to be worse once the existing gravel shoulders are paved. The road is owned by Clackamas County and any projects will require coordination with the County.
N Maple St at NW 34th Pl	ORDEQ may require a hydrodynamic separator for this outfall – the pipe is approximately 16 feet deep at the last manhole
NW Territorial Rd at N Holly St *	Trench drains in the private development are anticipated to fail and are located along property lines and therefore inaccessible.
12th Ave and N Pine St *	Flooding occurs in this area
NE 11th Ave at N Pine St	Flooding occurs in this area
N Maple St at NE 10th Ave, Maple St Park	Maple St Park's parking lot floods and the area currently has gravel shoulders
NE 5th Ave and N Juniper St	Flooding occurs in this area (City would like to redo sidewalks and paving with pervious).
Canby Cinema NE 2nd Ave at N Knott St	The existing rain gardens are experience maintenance problems; plugging and trash
SE Hazeldell Way *	This area incurs flooding 2 -3" deep in the vicinity of the trench drains
S Pine St at SE 2nd Ave, behind Canby Builders	Flooding occurs in this area - Unmaintained UICs in adjacent private development
S Ivy St at SE 2nd Ave	This area consists of a county system with no manholes and inaccessible; it is probably full of mud, ties into ODOT system along 99E - unknown condition
SW 2nd Ave	The pipe in this area is in marginal condition and contains some roots
SW 7th Ave and S Ivy St	Flooding occurs in this area and pipe in poor condition
SW 13th Ave and S Cedar St	Flooding occurs at UIC A-5 (the adjacent right-of-way have adequate space available for a swale)
SW 13th Ave south of Canby High School	Flooding occurs in this area (easement may be available for a swale)
Village on the Lochs Area, S Elm St	Flooding occurs at UIC A-11 when private UICs in the adjacent park fail

Note:

* indicates an area identified as high priority

Section 4: Basis of Planning

4.1 Introduction

The City has implemented programs, rules, policies and standards regarding stormwater management and design. Additionally, any private development must meet the requirements set forth by the City code. The City of Canby requires that all stormwater from private lots, except the downtown core area including 1st, 2nd and 3rd streets, is managed on-site.

4.2 Land Use and Development Area Projections

Conveyance system flows are based on land use information provided by the City and population projections completed by METRO. While storm flow estimates are governed predominantly by land use and impervious coverage, the population information is provided here to substantiate the expected timing of changes to land uses within the UGB. In general, it is assumed that future development in the northwestern part of the City and south of Hwy 99E will be managed with infiltration facilities and will not contribute additional surface discharges. Areas where new street development is anticipated in the vicinity of NE Territorial Road were added to the future surface drainage system model.

The 2010 census places the City of Canby's population at 15,829. The METRO Regional Population and Employment Range Forecasts [March 2009] report presents an estimated range of annual percentage growth rates for the Portland Metropolitan Statistical Area of 1.37% to 1.70%. Table 4.1 presents the census population for the City of Canby from 2010, as well as forecasted population for the City between the years 2015 and 2030, in five-year increments. The City population was predicted in five-year increments using both the low and high range growth rates estimated by METRO. The City of Canby's population is expected to approximately double over the 20-year time frame of this stormwater management plan. Based on this population projection, it was concluded that the majority of the area designated as residential in the comprehensive plan would be developed during this time frame, thus, future development was modeled as described above.

Table 4.1 City of Canby Population Forecast

Year	Predicted Population Low Range	Predicted Population High Range
2010	15,829	15,829
2015	17,998	18,520
2020	20,464	21,668
2025	23,268	25,352
2030	26,456	29,662
2035	30,080	34,705
2040	34,201	40,605

4.3 Flow Analysis

The SSA modeling software simulates rainfall-runoff response through land use analysis. Existing and expected future land use is used in conjunction with rainfall and hydrologic inputs to simulate the storm flow within a specified drainage basin or subbasin. The City's storm drainage systems are generally only managing runoff from the right of way, and were modeled as such. The modeled runoff is then time delayed and attenuated by traveling through the storm drainage conveyance and storage network. A complete discussion of flow generation by Storm and Sanitary Analysis can be found in Section 5.2.2

4.4 Guiding Principles

The following guiding principles are to be applied to the development of stormwater solutions and to the development of the overall master plan for the City of Canby:

- For the City of Canby's stormwater conveyance system, follow Public Works design guidelines and provide capacity to pass the 10-year storm events without surcharge, and provide capacity to pass a 25-year storm event with surcharge, but keeping the hydraulic grade line below the manhole lids.
- Use a combination of hard-piped system and open drainageway/natural systems to provide current and long-term stormwater service to the City of Canby.
- Whenever possible, design to the following preferences for stormwater management practices expressed by the City of Canby's Public Works employees:
    - Piping to centralized treatment is the preferred option, where logical. These centralized locations include the proposed treatment wetland at the Fish Eddy property.
    - The preferred LID alternatives to UICs are, in preferred order: pervious pavement, swales, rain gardens (if designed for easy maintenance and located toward the back of sites).
    - Buried stormwater management options such as trench drains should not be considered.
    - Swales need to be designed and located in such a way as to not present a safety hazard.
    - Curb extensions or 'Bump outs' on streets should not be considered as an option because residents will not appreciate the loss of parking spots.
    - UICs are ideal if they can be connected in series or 'daisy-chained' together to help manage overflows and reduce flooding.

4.5 UIC Retrofit Alternatives

UICs that are identified as requiring decommissioning based on the July 2012 UIC WPCF permit and results of the GWPD modeling will require retrofits to manage the stormwater previously directed to the UIC. The eight UICs in the City of Canby and the reason they require decommissioning are described in Section 3.4.2.2. Generally, UICs do not meet OAR 340-044 because of one of the following four conditions:

1. UIC intercepts groundwater
2. Insufficient vertical separation distance between the bottom of the UIC and high seasonal groundwater
3. Proximity to drinking water well with sampling results that exceed Effluent Discharge Limits (EDL) in permit
4. Monitoring results not meeting EDL in permit.

Retrofit alternatives are limited, based on the reason that a UIC requires decommissioning. Retrofit alternatives, the conditions when they can be used, and limitations on their use are provided in Table 4.2.

Table 4.2 UIC Retrofit Alternatives

Retrofit Option	Applicable Conditions*	Limitations
Add Surface Infiltration Facility	1, 2, 3, 4	
pervious pavement		not for driving surface
vegetated swale		
rain garden		
curb extensions (green street)		
Add Pretreatment	1, 2, 3, 4	
stormwater planter		
sediment manhole		
proprietary filtration (eg.Imbrium, Contech)		
Modify the Existing UIC	1, 2	
shallow the existing UIC		
shallow the existing UIC and add additional shallow sump for capacity		
Decommission and Replace UIC	2, 3	
new shallow sump UIC		
horizontal infiltration trench or perforated pipe		not under driving surface
Decommission and Redirect to Existing Stormwater System	1, 2, 3, 4	

<u>Notes:</u>

1 UIC intercepts groundwater

2 insufficient vertical separation distance between the bottom of the UIC and high seasonal groundwater

3 proximity to drinking water well with sampling results that exceed Effluent Discharge Limits (EDL) in permit

4 monitoring results not meeting EDL in permit

Section 5: Conveyance System Analysis

5.1 Introduction

City wide storm water infrastructure has been investigated through a combination of existing document review, discussions with City staff, site visits, and limited system survey. Six collection and conveyance systems with discrete surface outfalls were modeled. Current City staff concerns, current areas experiencing flooding events, ongoing problematic maintenance needs, and areas of expected growth guided the modeling efforts. The model examines the current hydraulic capacity of each system and provides an understanding of future system hydraulics for systems that are anticipated to require additional capacity. The model was built and analyzed using Autodesk Storm and Sanitary Analysis 2012 (SSA) and the EPA Stormwater Management Model (SWMM) hydrology model. Development of the model and interpretation of the output is discussed in this section.

In general, the SSA model has two main functions: flow generation and flow conveyance. The model generates a runoff hydrograph based on a simulated rainfall event. The flow conveyance function consists of a junction and link representation of the pipes, manholes, and other structures typically found in storm water collection systems. Flows represented by the runoff hydrograph are injected over time into the appropriate junction, typically a manhole, and are dynamically routed through the collection system, allowing flow propagation to attenuate through the system, and to generate an overall hydrograph at a terminal outfall. Outfalls are usually real structures, such as river discharges.

The model was set up and adjusted to produce results that generally agreed with observations by City staff. However, much of the observed flooding is believed to be related to UICs and there was no infiltration included in the modeling. It is also important to note that flow monitoring during actual flood events was not included in this study. Therefore, the model has not been calibrated against actual events. The results from the model may vary from the actual conditions, and the City may wish to verify the model output by performing flow monitoring in the future.

5.2 Conveyance System Model

A model was developed for each of the six surface collection and conveyance systems with surface outfalls. The model was developed starting with the GIS data provided by the City for the existing storm water system, which included some elevation information for manhole and UIC rims, and general pipe locations. The GIS data was updated with available record and design drawings; however, the diameter, material type, inverts and slopes of less than two percent of the system is recorded. Some additional data was collected through survey of top of manhole rim elevations and measure-downs to pipe inverts.

Model pipes, basins, and junctions were defined to provide a realistic representation of the most likely existing system, but do not correspond exactly to the existing conditions or to the network represented in the GIS, in some cases. The model is understood to contain the 'best available' information. Assumptions were made as to pipe slope, invert elevations and diameter, when necessary, to best represent the existing conditions reported by the City. Pipes, junctions and subbasins were defined in the GIS using the existing system information as a guide. Their geometric dimensions such as length, area and slope were calculated and the elements were imported into SSA.

5.2.1 Conveyance System Assumptions

All pipe was assumed to have a Manning's n of 0.015, which is an average value for concrete sewer pipe with inlets, manholes, etc. New pipe was also conservatively assigned a Manning's n of 0.015, although all new pipe is required by City design standards to be either polyvinyl chloride (PVC) or high-density polyethylene (HDPE), which may have a lower Manning's n and consequently a higher capacity. The minimum cover for all existing and new pipes was assumed to be 30 inches, which also aligns with City design standards for paved areas. The available grade constraint typically set the design pipe slope and pipes were designed to eliminate or minimize surcharge without specifically checking for the minimum required flow velocity of three feet per second. The minimum diameter used for new pipe was 12 inches.

When unknown, manholes were assumed to be seven feet deep to the lowest pipe invert with a 1.5 foot deep sump, which is the minimum sump depth required by City design standards. Where new manholes were modeled, they were generally spaced at the required maximum 500-foot spacing in the absence of other criteria. Each manhole was modeled to flood a circle of 50 foot diameter (an area of 123 square feet) which is the width of the right-of-way. The flooded depth is returned to the manhole when flows subside. UICs were in some cases used as junction locations and assumed to have a 27 foot sump depth with an initial water surface elevation 10 feet below the rim, but no infiltration was modeled at any point in the conveyance system.

5.2.2 Model Subbasins

Surface drainage subbasins were delineated starting from the basin delineations in the 1994 SDMP and are named to be consistent with that plan. The existing pipe network and catch basin locations in the GIS were used to further define the contributing area. U.S. Geological Survey (USGS)-Metro 10-foot interval contour data and manhole and UIC rim elevations were used to delineate basin boundaries. Flow directions, catch basin locations, and street level drainage breaks were in many cases identified using Google Maps satellite or street level imagery. The City of Canby requires that all stormwater from private lots, except the downtown core area including 1st, 2nd and 3rd streets, be managed on-site. The area contributing runoff to the City's systems is, therefore, estimated based on the area of the public right-of-way.

The entire area of the right-of-way is assumed to be impervious surface. An impervious surface is one in which water is unable to infiltrate. Impervious surfaces are typically man-made structures such as roadways, sidewalks and roofs. The assumption of 100% imperviousness in the right-of-way accounts for the area of paved roads and sidewalks, driveways and other impervious surfaces within the public right-of-way. The assumption also allows for some run-on to the right-of way from adjacent pervious surfaces, such as private driveways.

Basins were digitized in the GIS either using the tax lot boundaries to define the right-of-way limits, or crossing tax lot boundaries where other information was available. Basins were developed for all right-of-way areas draining to surface systems. In the case of N Maple St, the adjacent agricultural area was also modeled, based on contour data and satellite imagery and routed to the conveyance system.

Based on the Comprehensive Plan Zones map, new residential development was assumed to take place on both sides of the north end of N Pine St between NE 15th Ave and NE Territorial Road, and on the east side of N Redwood Road from NE 12th Ave to NE 18th Pl. A basin representing a 500-foot or 750-foot long by 50-foot wide impervious right of way was assumed for each major tax lot and street intersection, with the length depending on the tax lot depth. Other future basins were existing basins that are anticipated to be connected to existing systems as CIPs.

5.2.3 Additional Basin Characteristics

Basin characteristics must be input into the model. These inputs are presented and discussed in the following list.

- Area – Subbasins were delineated and digitized in the GIS, which was also used to calculate their area in acres and square feet.
- Flow Path Length – The longest flow path length, or the maximum distance, in feet, a drop of rainfall must travel over land before it enters the conveyance system is used to calculate the time of concentration of the basin. The time of concentration is used to determine the speed of runoff response. Flow path length was either approximately digitized or estimated for all drainage basins, based on the longest distance from a basin boundary to an inlet.
- Equivalent Width – The equivalent width of each basin was approximated by dividing the area of each basin in square feet by the longest flow path length in the basin in feet. Equivalent width is used by the model in estimating time of concentration.
- Percent Slope – The average percent slope of the basin was estimated. Elevations along the flow path were used to define the estimated change in elevation where contour data was available. In many locations, the City is flat and level and a default basin slope of 0.01 percent was assumed.
- Percentage Depressions - Depth of depression storage within the basin. Storage depth is defined for both impervious, 0.08 inches, and pervious, 0.20 inches, portions of the basin.
- Curve Number – The SCS curve number represents the type of land cover and soil in the basin and is used in the EPA SWMM model only for infiltration, which occurs only in basins with less than 100% impervious cover.

5.2.4 Flow Generation Information

The design rainfall event is defined as precipitation intensity over a specified time period, which produces a total rainfall amount in inches. A SCS Type IA 24-hr storm distribution was used, with precipitation amounts for Clackamas County. The 10-year and 25-year storms were modeled. The EPA SWMM model simulates rainfall on the defined basin surface, first filling the depression storage, then calculating the runoff rate to model junctions using Manning's equation and the time of concentration for the basin. A hydrograph is developed for each basin and routed through the pipe network junction by junction.

5.3 Hydraulic Criteria

5.3.1 Hydraulic Grade Line (HGL) Evaluation Technique

Modeled output is used to evaluate the hydraulic capacity of total pipe networks, subbasin networks, and individual pipe segments. In order to evaluate such a large quantity of output data, evaluation criteria must be developed. This criteria tests output data and determines the point at which the available capacity no longer allows for an acceptable factor of safety. These areas triggering a greater capacity are then translated into CIP projects.

The factor of safety associated with a piping configuration is typically related to the ratio between level of flow during a peak event divided by the total water level provided in the pipe, or more frequently called the d/D. For example, a ratio of 0.5 represents a half full pipe and a ratio of 1 represents a full pipe. As the ratio approaches 1, the factor of safety against an overflow event is decreased.

The modeling effort used two design storm events to develop an understanding of both the system as it functions now, and how it could react during a future storm event. Both current and future conditions were modeled using a 10-year, 24-hour 25-year, 24-hour design event. The basic criteria for evaluating whether a modeled system was adequate are that it does not surcharge during the 10-year event and that it does not flood during the 25-year event. Model outputs have been tabulated and are provided in Appendix A. Surcharged means that a pipe's maximum to total depth ratio is greater than 1.0, i.e. the water surface elevation exceeds the crown of the pipe. Flood means that the water surface elevation in a junction (manhole) exceeds the manhole rim elevation.

5.3.2 Sizing Evaluations

Flows developed in both the 10-year and 25-year modeling scenarios have shown surcharging events to be present in some systems. Most often, this surcharging is directly related to insufficient capacity in downstream piping. As downstream surcharging develops, an upstream backwater wave begins. As this backwater builds, upstream piping begins to surcharge as well. When a surcharge event is developed in the model, typically many pipe segments are affected. While numerous pipes may have contributed to the surcharging, it is important to note where the surcharging and backwater wave began, as it directly affects where the CIP project should focus. Additionally, since CIP projects are not always completed in one phase, it is important to begin improvements at the downstream end of the piping network. As downstream capacities are increased, surcharging related to backwater waves will be reduced.

5.4 Modeled Systems

Six surface outfall drainage systems were modeled in the City of Canby. Four additional systems were modeled for proposed CIP projects or anticipated futures conditions. Basic modeled-system characteristics are presented in Table 5.1 and model results are described briefly below. Much of the City is serviced by UICs connected to relatively short pipe segments or other small private systems, these areas are also presented in Table 5.1. The basins managed by ODOT along Hwy 99E drain to surface outfalls but were also not modeled. A complete basin specific map for each model and detailed model inputs and results are included in Appendix A.

Table 5.1: Modeled Basins

Basin Name	Description	Area (Acres)	Outfall
Canby Downtown	Existing system discharges to detention pond	48.9	Detention pond, swale to Molalla River
Canby Downtown CIP	Increase pipe sizes, add basin for decommissioned UICs	52.3	“
N Baker Dr	South end of N Baker Dr	2.1	Molalla River
Knights Bridge Rd	Existing system	3.5	Molalla River under bridge
Knights Bridge Rd plus N Baker St	Add pipe to collect N Baker St	4.3	“
Knights Bridge Rd and N Baker Rd plus NW 9th Ave	Add pipe to connect slow draining UICs on NW 9th and increase mainline pipe size	7.8	“
S Berg Parkway	Small basin in SW of City	3.2	Molalla River
North Maple St	Both agricultural and impervious area, includes area outside UGB	123 agricultural, 5.6 impervious 128.6 total	Willamette River
Redwood/Willow Creek	West of Willow Creek, including Territorial Rd	22.2	Willow Creek
10th Ave and Pine St to NE Territorial Rd plus Redwood/Willow Creek	Adds N 10th Ave and Pine St through to Territorial Rd	31.5	“
Redwood/Willow Creek plus N Pine St and N Redwood Rd New Development	Adds right-of-way for new residential development on available parcels	41.1	“
TOTAL MODELED	**EXISTING**	**209**	**Surface**
TOTAL MODELED	**FUTURE**	**239**	**Surface**
NOT MODELED	**UICs/Private/ODOT**	**3158**	**Infiltration-Surface**

5.4.1.1 Canby Downtown

This storm system drains the downtown commercial area and is the oldest part of the City of Canby's stormwater infrastructure. The discharge for this basin is to the Molalla River in the western part of the community. The area of the existing basin as modeled is 48.9 acres. There is an area within this basin in the vicinity of Wait Park that drains to drywells or infiltrates, and this area is not included as part of the basin. The outfall from the Canby downtown basin is a surface water discharge point via a detention pond, grassy swale, and natural drainage way adjacent to the new police station at the west end of NW 3rd Ave.

The model shows surcharging and flooding within the system due to shallow slopes in the existing pipe system and reverse slopes on the branch along N Ivy St. The mainline pipes along NW 3rd Ave were modeled as 30-inch diameter to reduce the flooding represented for existing conditions to a reasonable level, although other evidence suggests they might only be 20-inch diameter. There may be significant exfiltration through failing pipe systems that is not captured by the model.

5.4.1.2 Canby Downtown CIP

This model of a CIP adds an additional drainage basin on the south side of Wait Park to account for the decommissioning of drywells in the area. Pipe sizes are increased in several locations to eliminate flooding and minimize surcharging. The pipe segment on N Fir St between NW 3rd Ave and NW 2nd Ave is eliminated and a new pipe is added on NW 3rd Ave between N Fir St and N Cedar St.

5.4.1.3 N Baker Drive

The N Baker Dr basin drains to a pipe outfall that discharges runoff collected from the south end of N Baker Dr, NW 6th Pl, and NW 6th Ave. The area of the existing basin, as modeled, is 2.1 acres. The outfall is near the steep bank on the Molalla River, upstream of Knights Bridge Road. There is limited data for the conveyance system in this area and there may be additional outfalls that are not mapped. CIP #1 addresses issues in this basin. All pipes were assumed to have a 10-inch diameter and there were no issues identified in the modeling of this basin, except that one pipe has a reverse slope.

5.4.1.4 Knights Bridge Rd

This system drains approximately 1500 feet of Knights Bridge Road and immediate areas of intersecting streets. The area of the existing basin, as modeled, is 3.5 acres. The outfall of this storm system is a pipe located underneath the bridge on which Knights Bridge Rd crosses the Molalla River. The pipe is supported by the bridge structure underneath the roadway and has a free outfall approximately 30 feet above the Molalla River, above the approximate centerline of the river. The conveyance system includes a Contech Model CDS2015-4 hydrodynamic separator for particulate removal located at approximately N Ash St at the second-last manhole before the discharge point. The existing system model shows the system is adequate.

5.4.1.5 Knights Bridge Rd plus N Baker Rd

This model adds approximately 700 feet of N Baker Dr to the Knights Bridge Rd system. The new basin area is approximately 0.8 acres. The design standard 12-inch pipe was used and the model showed no other changes were needed.

5.4.1.6 Knights Bridge Rd and N Baker Rd including NW 9th Ave

This model adds another 3.5 acres to the system by connecting four slow-draining UICs on NW 9th Ave through a pipe along N Aspen St. The model shows that the Knights Bridge Rd mainline to the outfall needs to be increased from 12 to 18 inches to prevent flooding. However, no infiltration was assumed through the existing UICs and these would be expected to reduce peak flows somewhat. UIC flow modeling was beyond the scope of this plan; however, it should be completed to accurately determine the future required pipe size along Knights Bridge Rd.

5.4.1.7 S Berg Parkway

This conveyance system drains the 3.2 acres of SW Berg Parkway to an outfall near the Molalla River. Pipes were assumed to be 10-inch diameter and no issues were identified by modeling of this basin.

5.4.1.8 North Maple Street

This system collects stormwater in a series of daisy-chained UICs along Maple Street, which is located in the northern part of the City, which then combines with runoff from adjacent agricultural land and discharges directly to the Willamette River beyond the end of NE 34th Pl. The existing impervious area of this basin is 5.6 acres, and the contributing agricultural area is 123 acres. The system discharges through a deeply buried pipe at approximately the water surface elevation of the Willamette River.

The model of this system demonstrated slight surcharging and flooding of the manhole located adjacent to NE 31st Pl, with the remainder of the system functioning well. However, no infiltration was assumed through the existing UICs and which would be expected, and would reduce peak flows. In addition, the City has not observed any flooding problems in this basin, so it assumed the conveyance system is adequate. If more accurate flow rate estimates are required for future sizing of potential treatment devices in this basin, the reduction in flow rate due to the existing UICs should be considered in the model.

5.4.1.9 Redwood/Willow Creek

This system drains N Redwood St and adjacent side streets from NE Territorial Rd to Hwy 99E, a portion of the Molalla Forest Rd, some of the residential streets on the east side of N Pine St and NE Territorial Rd from N Maple St to Willow Creek. The modeled existing drainage basin is 22.2 acres. This system’s outfall is to Willow Creek on the north side of NE Territorial Rd where it combines with flow from the main stem of Willow Creek and flows onto the Fish Eddy property. No issues were identified in the modeling of this system and it appears to be adequately sized.

5.4.1.10 N 10th Ave and N Pine St to NE Territorial Rd plus Redwood/Willow Creek

This model adds basins, manhole junctions and pipes to represent development of N 10th Ave and N Pine St and decommissioning of one UIC in the area. The additional basin area is 9.3 acres. Flows are piped along N Pine St to Territorial Rd. The model shows the existing system has adequate capacity for the new flows that are proposed to be added to the existing pipe along NE Territorial Rd.

5.4.1.11 Redwood/Willow Creek plus N Pine St and N Redwood Rd New Development

This model adds conceptual basins to account for runoff from potential future development. Pipes are added along N Pine St to connect to NE Territorial Rd and new connections are added along Redwood Rd from the basins to the east. The additional drainage area is 11.7 acres and the model shows the system has adequate capacity for the additional runoff.

5.1 Water Quality Facility Sizing Criteria

The design criteria for water quality facilities are specified in the City of Canby Public Works Design Standards for storm drainage design in Section 4.310 c. The standards require that water quality facilities be designed to the criteria in the Clean Water Services (CWS) Design Manual Chapter 4, paragraph 4.06. City of Canby design standards also required that overflow or bypass be provided for the storm events from the 25-year to 100-year 24-hour storm event such that the facility does not over-top or exceed the capacity of the overflow.

Chapter 4.06 of the CWS Design and Construction Standards specify that a wetland should have a permanent pool volume equal to 0.55 times the water quality volume (WQV), a detention volume equal to 1.0 times the water quality volume, and a drawdown time of 48 hours. The water quality volume is based on “a dry weather storm event totaling 0.36 inches of precipitation falling in 4 hours with an average storm return period of 96 hours.” The calculation for the required WQV used by CWS is 0.36 inches times the impervious area draining to the facility, with the amount of impervious area depending on the size and type of new development. The water quality flow rate (WQF) is calculated as the WQV divided by 14,400 seconds (equivalent to four hours). CWS also specifies other detailed design criteria that should be followed during wetland design.

Through discussion with CWS staff, it was determined that the above criteria and design guidelines are not applicable to a large regional treatment facility, and no alternative method has been developed by CWS for modeling the above specified design storm. Water quality facility evaluations were, therefore, completed using the SSA model and a 6-month 24-hour Type 1A SCS design storm, which was assumed to be 1.91 inches of rainfall or 72% of the 2-year 24-hour storm. This is a commonly used criteria for sizing water quality facilities and is expected to treat approximately 90% of annual stormwater runoff. The total volume flowing to the treatment location for the above storm is assumed to be the water quality volume.

5.2 Existing Detention Pond Capacity

The volumetric capacity of the existing detention pond below the west end of NW 3rd Ave near the police station was estimated based on field observations and satellite imagery. The approximate depth of the pond was observed to be four feet, measured from the high water mark to the water surface. The water surface was observed on 14 February 2013 to cover approximately one quarter of the pond bottom only a few inches deep, in the lowest southeast corner of the pond, although the pond bottom appears to be relatively flat and level. The four-foot water depth in the pond would be controlled by the invert elevation of the swale leaving the east end of the pond, and ultimately by the three 36-inch culverts at the end of the swale. However, infiltration appears to be significant because the observed water level was significantly below this control point during the rainiest part of the year. The approximate surface area of the pond at the four-foot depth, digitized on satellite imagery, is 18,000 square feet. The approximate volume of the pond, as it is currently functioning, is therefore 72,000 cubic feet (1.7 acre-feet).

5.2.1 Detention Pond Capacity Analysis

The Canby Downtown stormwater models previously discussed in Sections 5.4.1.1 and 5.4.1.2 were used to analyze the detention pond capacity. Model assumptions for the pond and swale were based on visual observations and photographs obtained during one site visit and anecdotal reports of pond performance. Because of the many assumptions made to develop the model, results should be interpreted with caution; however, they generally indicate that the pond has sufficient capacity to convey the design storms for current and future conditions, and provides a level of water quality treatment.

A storage node representing the pond was added to the models. The pond was assumed to have an area of 18,000 square feet from an elevation of 110 feet to a maximum depth of 6 feet. The swale was assumed to leave the pond at elevation 113, have a bottom width of 10 feet, a depth of 3 feet with 4 to 1 side slopes, and a length of 625 feet. The swale was assumed to have a relatively flat slope so the inverts of the 3 culverts at the end of the swale were assumed to be at elevation 112.9. The road surface was assumed to be 3 feet above the top of the culvert. Refer to Appendix D for detailed model inputs and results.

Infiltration was not generally considered during modeling for this stormwater master plan, but because the modeled detention pond would contain a permanent pool, which has been observed not to occur, an exfiltration (equivalent to infiltration out of the pond) rate was assumed for the pond area. The assumed exfiltration rate was based on the design long term infiltration rate recommended by GeoPacific Engineers following infiltration testing in May 2003 at the Valentine meadows development located at NW Cedar and NW 3rd Ave. The exfiltration rate for the pond was modeled as 3.5 inches per hour, which is the recommended rate of 7 inches per hour divided by a safety factor of 2.

The previously discussed conveyance criteria of designing to provide capacity to pass a 25-year storm event with surcharge but without flooding was used as a capacity criteria for the pond. Specifically, the criteria was to keep the hydraulic grade line below the road surface at the three 36-inch culverts. For the detention pond, this criterion was also checked for the 100-year storm. The culverts do not flow full, and the freeboard is not less than 4.5 feet for any of the modeled scenarios, indicating that the pond has excess capacity. Refer to Appendix D for model results.

The previously discussed criterion for water quality volume is that detention facilities be designed to contain the volume of the 6-month 24-hour storm. For existing conditions at the detention pond, this volume is 326,200 cubic feet, which is larger than the calculated pond volume. However, additional treatment volume and performance is provided by the swale through storage and vegetative filtration etc. Also, a significant portion of the runoff volume entering the pond is observed to infiltrate (the model indicates approximately 60% of the WQ storm exfiltrates for existing conditions), and, therefore, also receives treatment. More complex modeling would be required to accurately represent the true water quality benefit of the pond and swale, and this would require additional data collection beyond the scope of the current stormwater master plan.

5.3 Proposed Fish Eddy Wetland Sizing

A stormwater treatment wetland is proposed on the Fish Eddy property at approximately 1570 NE Territorial Rd and adjacent to the existing Willow Creek. This treatment wetland is part of a restoration of the entire Fish Eddy property to native seasonal wetland and wet prairie habitat as part of the Willamette Wayside Properties SWMP. The current plan estimates a total of 1,435,000 square feet (33 acres) is to be restored on the property, including the area for the treatment wetland.

5.3.1 Treatment Wetland Drainage Basins

The stormwater treatment wetland is sized to treat stormwater runoff from properties south of Territorial Rd, including Willow Creek. The wetland is sized to treat the existing conditions, plus any new impervious right-of-way, based on the assumption that new development will maintain discharges at or below existing levels. The watershed draining to Willow Creek at the proposed wetland location is approximately 393 acres and is estimated to be approximately 25% impervious when fully built out. The modeled subbasins used for sizing the wetland are shown on Figure 6 and described below.

The existing piped system discharging to Willow Creek on the north side of NE Territorial Rd drains areas including the existing piped systems along N Redwood Rd and NE Territorial Rd, the proposed piped system for NE 10th Ave and N Pine St, and potential new development at the north end of N Pine St and the west side of Redwood Rd. These subbasins total approximately 31 acres and are assumed to be 100% impervious. The existing Willow Creek subdivision, the undeveloped and rural area along Willow Creek between NE Territorial Rd and Hwy 99E, and the north side of Hwy 99E drain through Willow Creek to the Willow Creek Wetlands. These areas are modeled as a single basin of 120 acres that is 15% impervious. The Willow Creek wetlands discharge through a weir structure to two 36-inch culverts under NE Territorial Rd to join runoff from the piped systems on NE Territorial.

The north fork of Willow Creek also receives runoff from the basin, southeast of Hwy 99E via an existing culvert under the highway. This basin is approximately 142 acres and assumed to be 5% impervious. The south fork of Willow Creek receives runoff from two basins southwest of Hwy 99E, through an existing private detention pond located south of Hwy 99E. The first is an approximately 72 acre and 40% impervious piped system draining the privately owned commercial area adjacent to Hwy 99E. The second basin is the ODOT and County pipe and ditch system that drains S Ivy St and a portion of Hwy 99E. This basin is 18 acres and assumed to be 95% impervious.

5.3.2 Conceptual Treatment Wetland Size

The SSA model inputs and results are presented in Appendix E for the 6-month 24-hour water quality storm. The modeled flow at the Willow Creek outfall to the proposed Fish Eddy Treatment Wetland averages 6.7 cubic feet per second (cfs) and has a peak of 18.8 cfs. The modeled total runoff volume to this outfall, which is the water quality volume, is 814,000 cubic feet. Note that because of limited data, assumptions concerning channel dimensions, outlet structures, etc. were made based on satellite imagery and limited photographs or observations in the field while developing the model and basin characteristics. These estimates are conceptual only and should be refined prior to further design.

The depth of a stormwater treatment volume varies throughout the wetland, with deeper areas in the forebay and varied depths of the permanent pool to promote different zones of treatment and diverse vegetation; however, for conceptual sizing, the average depth can be assumed to be approximately three feet. Dividing the above water quality volume by three feet gives a required wetland surface area of approximately 6.2 acres. With the required side slopes, perimeter areas that are expected to flood during large storm events and additional volumes for sediment retention, the approximate disturbed area required for construction of the treatment wetland is 12 acres, or approximately two times the required surface area for treatment.

It is recommended that a flow monitoring station be established in Willow Creek North of NE Territorial Rd to characterize the rainfall runoff response for the existing watershed for one or two seasons prior to designing the treatment wetland. The collected data can be used to calibrate the hydrologic and hydraulic model so it can be used to size the wetland to be an efficient and cost-effective stormwater treatment facility.

Section 6: Capital Improvement Projects

CIPs have been identified and developed based on anecdotal information provided by City staff, as well as through stormwater runoff and conveyance modeling. The model results have been used to develop pipe alignments and pipe sizes required to pass the design storm basin flows. The following improvements incorporate the estimated costs for the recommended capital improvement projects over the 20-year planning period. The CIP projects are intended to correct existing storm system deficiencies and provide the capacity to accommodate anticipated growth and development.

6.1 System Evaluation Approach

To evaluate the existing system, identify areas of inadequate capacity, and develop a list of CIPs, Kennedy/Jenks conducted interviews with City personnel and staff for anecdotal information, and used SSA modeling to evaluate existing stormwater infrastructure. Storm events were modeled for individual basins to analyze existing pipe networks and identify pipes susceptible to surcharging and areas prone to flooding. Future storm event flows were also developed in SSA, as discussed in Section 5, where additional development is anticipated during the planning period.

Required CIPs have been established by evaluating the severity of surcharging or flooding that develops in insufficiently sized pipe. These areas, as described below, are places where pipe upsizing or replacement is required. Timing of the CIPs is dependent on the future build out of the basins and priorities identified by the City. By evaluating different yearly flows, specific projects can be phased and scheduled as required by the growing population. The following sections discuss the specific projects and project basins that were developed from interviews with City personnel and the modeling work. Each section will discuss the basin assessment, the evaluation, the recommended improvements, the required phasing, and an estimated cost to complete the project.

6.2 System Evaluation Results

A total of 24 stormwater infrastructure improvement projects have been developed, including two flow-monitoring installations. While the CIP descriptions provided herein are aimed at increasing the capacity and functionality of the system through hydraulic upgrades, other non-CIP related activities are also recommended in this section, including comprehensive surveying of the existing system, development of an operation and maintenance manual, further coordination with Clackamas County and ODOT, and continued monitoring of storm conditions to aid in understanding changes over time.

6.3 Estimates of Probable Cost

Each CIP has an associated estimate of probable cost. These costs were developed through a combination of recent similar project costs, RS Means 2013, and applying the May 2013 Engineering News Record Construction Cost Index (ENR CCI) of 9440.52. A pipeline cost estimating tool has been developed and included in Appendix B. This tool uses line items associated with typical pipeline construction costs to develop an overall project specific cost. The cost listed on the CIP summary sheets represents the total project cost. For all CIP components, the total cost was calculated by multiplying the base cost of construction times a non-construction cost factor (NCCF). This NCCF includes contractor mobilization, contractor overhead and profit, and a planning level contingency. These planning level cost estimates should be expected to range from 30% less than to 50% more than actual project costs. Total project costs also include 10% for contractor overhead and profit, 20% for contingency, and 25% for engineering and administration costs. In addition, the project costs have been rounded up to the nearest $10,000. Detailed cost estimates for each of the CIPs can be found in Appendix B.

6.3.1 Pipe Costs

The line item costs for various pipe diameters are included in the cost estimates for each CIP in Appendix B. These costs include the total construction and installation cost per lineal foot (LF), including excavation, backfill, pipe placement, saw cutting, traffic control, and asphalt replacement. The cost assumes that pipe will be installed within the asphalt paved roadway, and at an average depth of 8 feet.

6.3.2 UIC Decommissioning

The lump sum (LS) cost for decommissioning a UIC assumes that the UIC to be decommissioned is free of pollution, and that the decommissioning includes the plugging of all inlet pipes to the UIC, backfilling the UIC with rock, removing the cone of the UIC manhole, and providing asphalt restoration. Note that if pollution is found and remediation is required, the cost for decommissioning could increase significantly. This will be highly dependent upon the type of pollution found and the extent of the pollution. Therefore, it is difficult to estimate potential remediation costs.

6.4 Project Prioritization

Prioritizing CIPs is an important step to creating a better functioning stormwater system. The overriding factors for stormwater project prioritization are severity of current conditions and implementing downstream improvements first. The current conditions model results can be used to evaluate those areas that require immediate response. These system parameters were used to define the priority ranking as seen on each CIP summary sheet.

6.5 Project Implementation

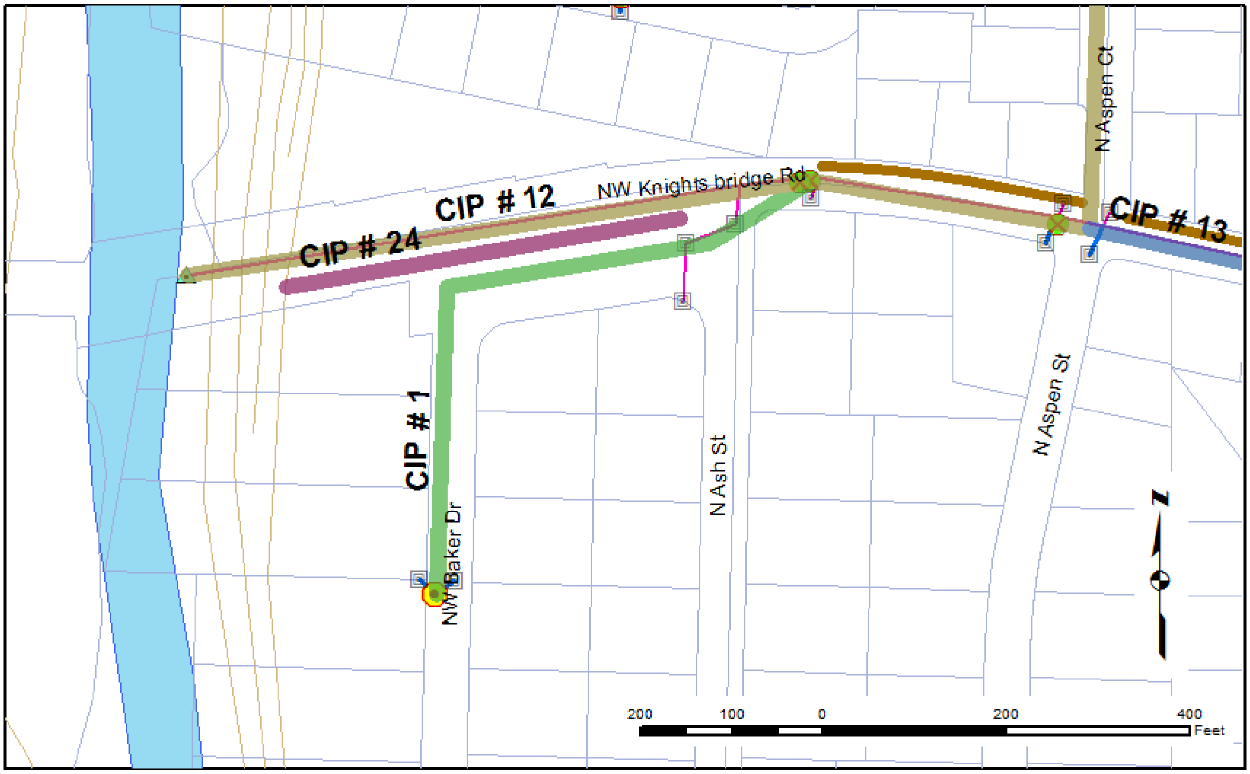
A descriptive breakdown of each CIP is presented below. Planning level estimates of probable cost are provided for individual CIP projects in Appendix B**.**

6.5.1 N Baker Dr. CIP #1

Currently, stormwater is observed to seep from the steep bank of the Molalla River, along the residential area of N Baker Dr. between NW 6th Pl. and N Baker St. It is believed that this seepage may be due to an inadequately sized storm system and along with overland flow of stormwater to the river is causing bank erosion. The UIC for this area lacks adequate capacity and is slow draining. The stormwater pipes in this area are also in poor condition

Recommended Improvements

The recommended improvements consist of replacement and rerouting of the existing stormwater pipes in the area. The new piping will consist of approximately 800 LF of 12–inch storm pipe and the installation of three stormwater manholes. The pipe alignment will begin at the existing UIC on N Baker Dr. and will follow N Baker Dr. until it connects to the existing stormwater conveyance system located at the intersection of N Baker Dr. and N Ash St. Catch basins should be installed on either side of the street at each manhole location. The installation of this new stormwater system will divert stormwater runoff away from the existing outfall and unstable bank, and convey it to the existing Contech Model CDS2015-4 hydrodynamic separator treatment system located at N Knights Bridge Rd. Total project costs are estimated to be $180,000.



CIP #1: N Baker Dr.

Description: Address and mitigate the current Molalla River bank erosion concerns, as well as address the surrounding UIC capacity and pipe failures by installing approximately 800 LF of 12" diameter HDPE pipe, 3 manholes, and 8 catch basins

Location: N Baker Dr. from the existing UIC to Knights Bridge Rd.

Existing System: Slow draining UIC with insufficient capacity; Molalla River outfall; Storm drain pipes in poor condition

Proposed System: Add new 12" diameter HDPE storm drain pipe, and reroute the stormwater runoff to the Contech Model CDS2015-4 hydrodynamic separator treatment system at N. Knights Bridge Rd.

Pipe Length: Approximately 800 LF

Cost: $180,000

Priority: High

Schedule: 0 to 5 years

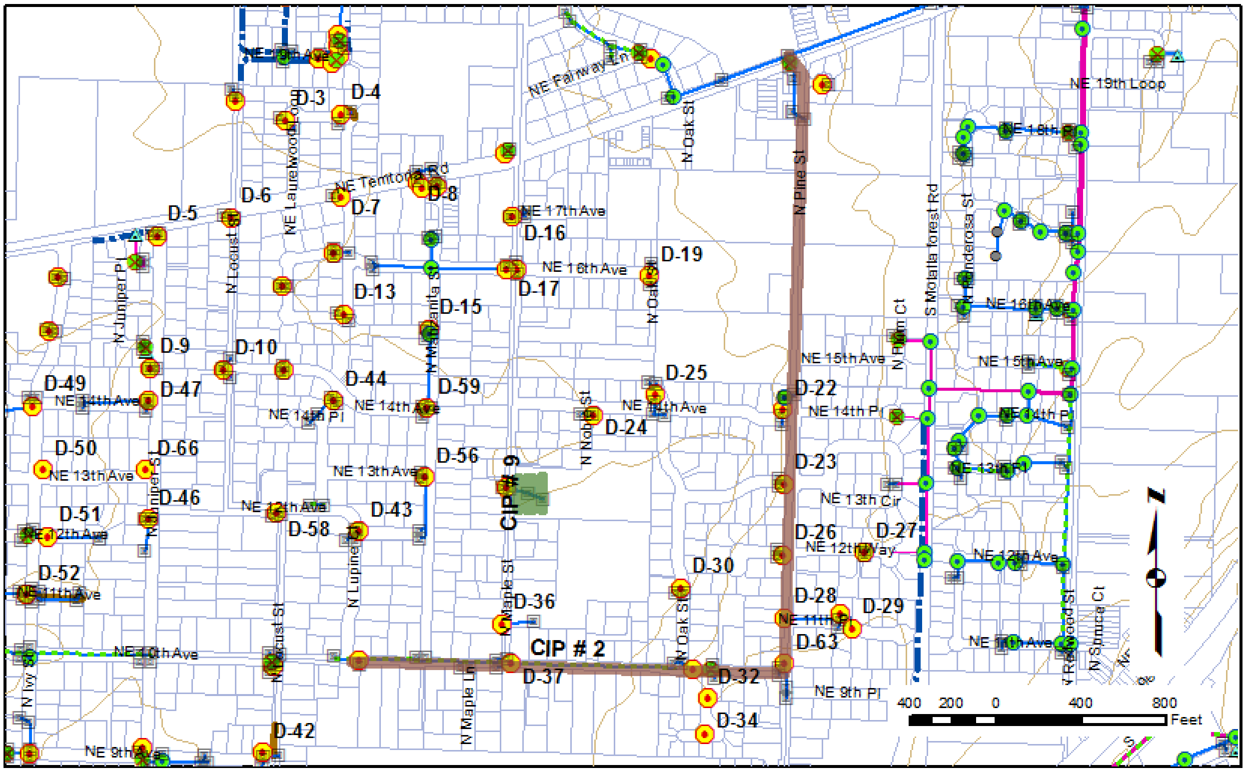
6.5.2 10th Ave. from N Locust St. to N Pine St. CIP #2

This street currently has gravel shoulders and no curb or sidewalks. Large puddles form during storm events, and the roadway often becomes flooded with stormwater runoff. This is due to the lack of capacity, poor infiltration abilities of the existing UICs. It is anticipated that when curb and gutter and sidewalks are added, problems with flooding will be worse. Furthermore, groundwater modeling and City observations have demonstrated that four UICs within this corridor are likely to have wet feet and should be decommissioned. Finally, UIC D-63 at the intersection of NE 10th Ave and N Pine St has been identified as high risk and must be decommissioned.

Recommended Improvements

The recommended improvements consist of decommissioning five UICs (D-31, D-63, D-28, D-26, D-23) and replacing the existing piping on NE 10th Ave from N Locust to N Pine St with new 12-inch and 18-inch piping. This piping will connect the remaining UICs in series from Locust to Pine St. At Pine St, a new pipeline alignment will convey runoff north on Pine St to Territorial Rd. The new alignment on Pine St will consist of 24-inch pipe. The system will also require the installation of fourteen new manholes. The project will require approximately 1,650 LF of 12" pipe, 370 LF of 18" pipe and 2,900 LF of 24" pipe. The total project cost is estimated at $1,330,000.

N Pine St. is currently under the jurisdiction of Clackamas County. Approximately 60 percent of the cost of the proposed improvement s should be allocated to Clackamas County based on the length of piping along N Pine St relative to the total length of piping for the CIP, and because four of the five drywells to be decommissioned are located on N Pine St. Clackamas County's share of the CIP is approximately $801,000 and the City of Canby's share of the CIP is approximately $529,000.



CIP #2: 10th Ave. from N Locust St. to N Pine St.

Description: Significant flooding occurs between the streets of N Locust St. and N Pine St. on NW 10th Ave. Groundwater modeling demonstrated that four UICs along this stretch that have wet feet and should be decommissioned, and one UIC that is high risk and must be decommissioned. The recommendation is to decommission the five UICs, install a new pipeline from N. Locust to N Pine St. on 10th, and along N Pine St. to Territorial Rd.

Location: 10th Ave. from N. Locust St. to N. Pine St, N. Pine St. from 10th Ave to Territorial Rd.

Existing System: Currently this area is served by UICs that are slow draining and lack capacity, and has undeveloped gravel shoulders. Furthermore, groundwater modeling demonstrated that one of the existing UICs is out of compliance and requires decommissioning. The condition of the existing piping that conveys flows to the existing UICs is unknown.

Proposed System: Decommission the five UICs and add 1,650 LF of 12” pipe and 370 LF of 18” pipe along NE 10th Ave. Install a new stormwater pipeline along N Pine St from NE 10th Ave to NE Territorial Rd, consisting of approximately 2,900 LF of 24” pipe. Approximately 14 new manholes will be required.

Cost: Total: $1,330,000 Total, City of Canby: $ 529,000, Clackamas County: $801,000

Priority: High

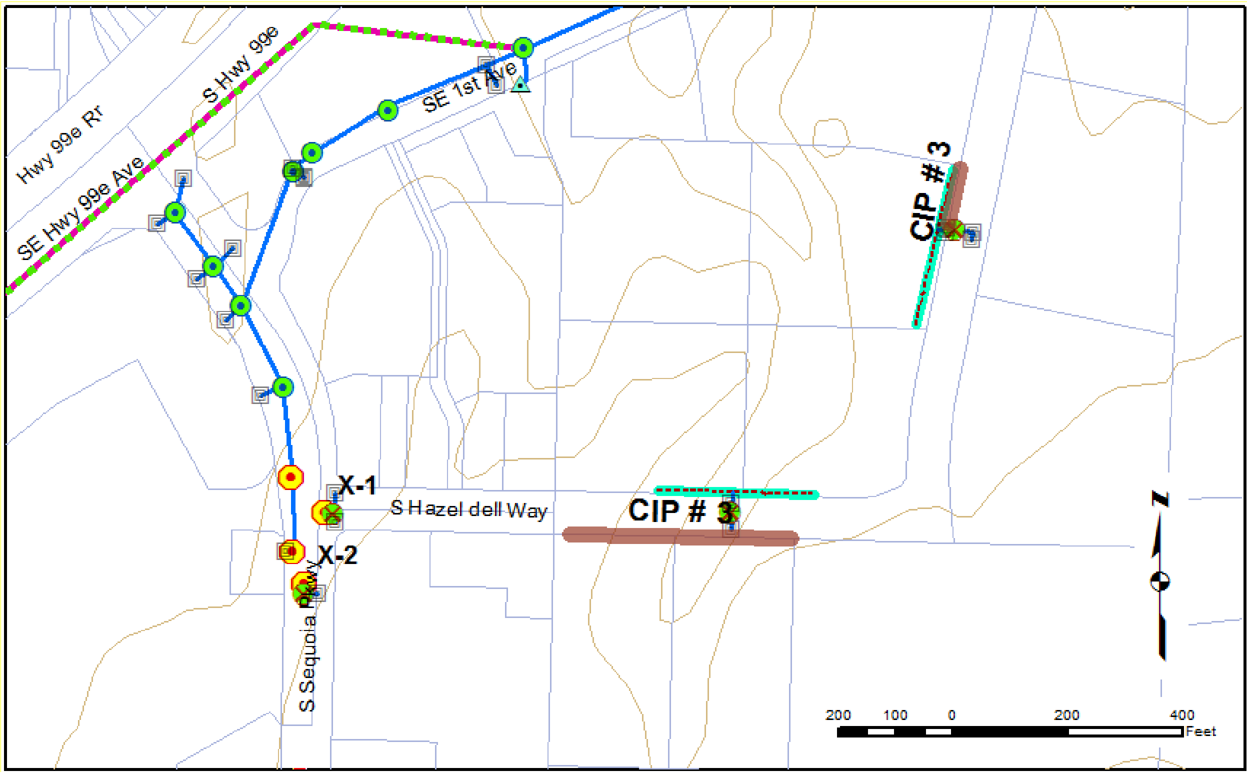
Schedule: 0 to 5 years.

6.5.3 SE Hazeldell Way CIP #3

Current conditions create flooding of the roadway along Hazeldell Way in the vicinity of the existing trench drains. The current stormwater infrastructure in the area consists of UICs and a piped system to the north, and sediment manholes and trench drains in the remainder of the area. The south end of S Sequoia Parkway, near S Walnut St. previously also had flooding problems but these have been resolved by repairing the trench drains. Groundwater modeling has demonstrated that the seasonal high groundwater table is relatively high in this area, at 20 feet depth.

Recommended Improvements

The recommended improvements consist of installing vegetated swales within the right-of-way of Hazeldell Way, on the opposite side of the street from the existing catch basins and catch drains. The total approximate length of swale to be installed is 400 feet (ft). The estimated cost of the project is $90,000.



CIP #3: SE Hazeldell Way

Description: Flooding on SE Hazeldell Way. The recommendation is to install vegetated swales to provide some detention and more infiltration capacity.

Location: SE Hazeldell Way

Existing System: The existing system consists of UICs and a piped system or sediment manholes and trench drains. Groundwater modeling has demonstrated the water table is relatively high in this area.

Proposed System: The proposed system is to install vegetated swales in the right-of-way across from the existing trench drains to provide more infiltration capacity and mitigate flooding. The estimate includes one 300 LF swale and one 100 LF swale. The total length of swales to be installed is approximately 400 ft.

Cost: $90,000

Priority: High

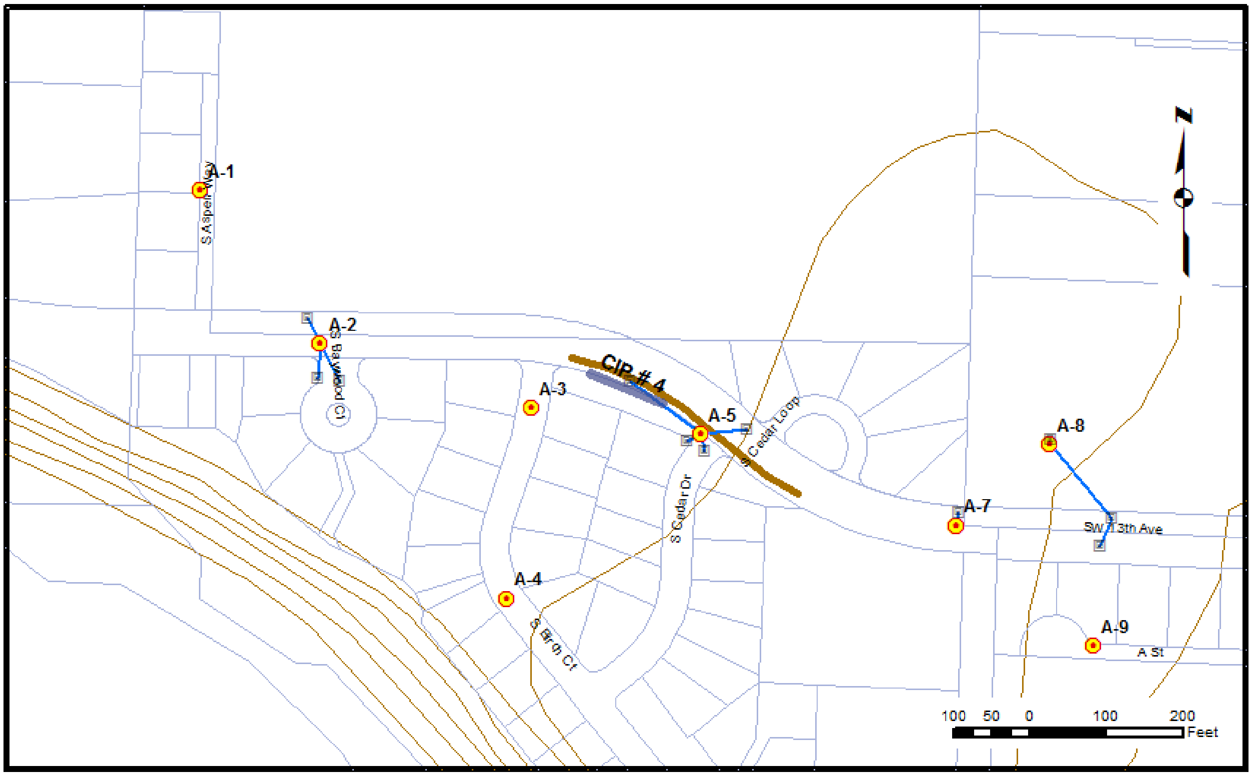
Schedule: 0 to 5 years.

6.5.4 SW 13th Ave near Canby High School CIP #4

Currently, flooding occurs along this corridor because the existing UIC A-5 has insufficient capacity. The area often becomes a overloaded with stormwater runoff.

Recommended Improvements

The recommended improvements consist of utilizing the width of the right-of-way and installing a swale for stormwater retention and infiltration to increase the infiltration capacity in the area. The estimated cost of the project is $30,000.



CIP #4: SW 13th Ave near Canby High School

Description: This corridor has flooding issues, and the existing UICs have insufficient capacity to handle the stormwater runoff.

Location: SW 13th Ave near Canby High School

Existing System: The existing system consists of UICs

Proposed System: The proposed system is to create a 100 LF stormwater retention and infiltration swale in the right-of-way between S Birch Ct. and S Cedar Dr.

Cost: $30,000

Priority: High

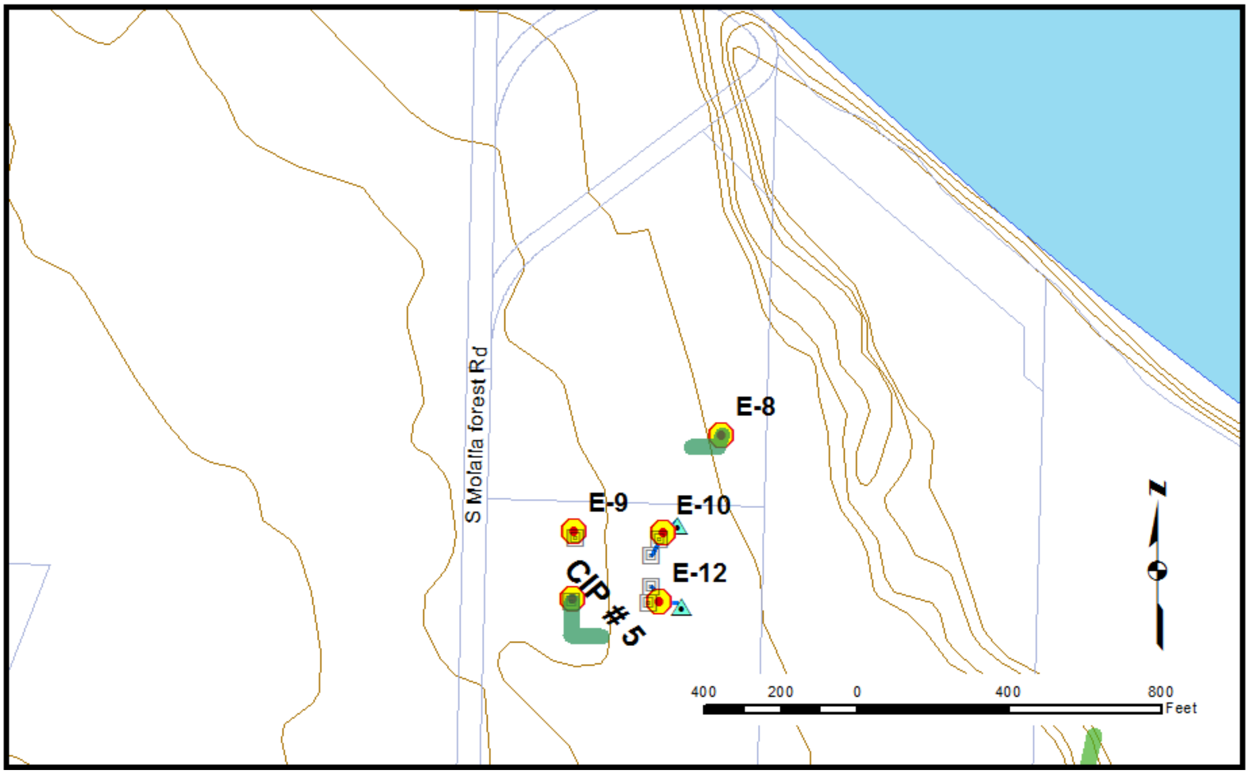
Schedule: 0 to 5 years.

6.5.5 UIC E-8 and E-11 Decommission #5

Currently, there are two existing UICs (UIC E-8 and E-11) located within the property occupied by the Canby WWTP which are out of compliance and are required to be decommissioned.

Recommended Improvements

The recommended improvement is to decommission the two out of compliance UICs and install rain garden infiltration swales near the location of the decommissioned UICs. The estimated cost of the project is $50,000.



CIP #5: UIC E-8 and E-11 Decommission

Description: The existing conditions consist of two UICs (E-8 and E-11) which are currently out of compliance and require decommissioning.

Location: City of Canby WWTP property

Existing System: The existing system consists of two UICs which are out of compliance.

Proposed System:

The proposed system is to decommission the UICs which are out of compliance (E-8 and E-11) and install rain garden infiltration swales near the location of the decommissioned UICs.

Cost: $50,000

Priority: High

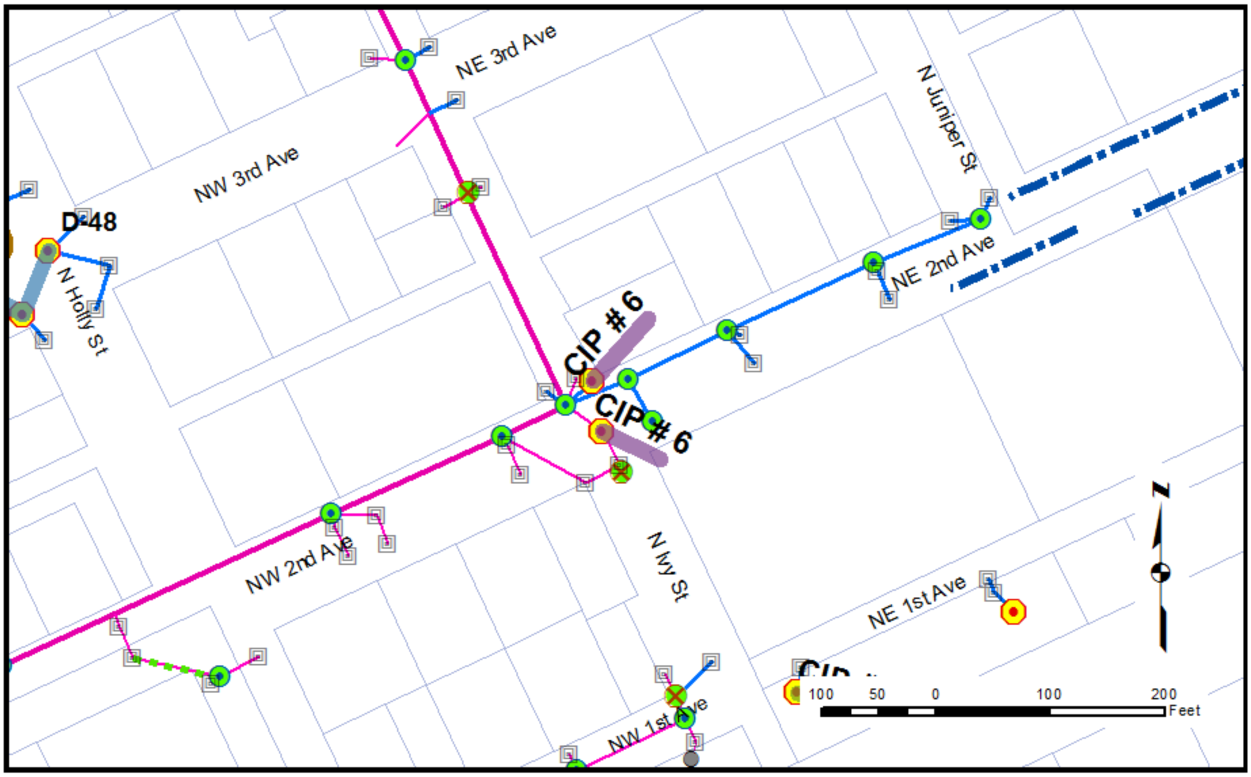
Schedule: 0 to 5 years.

6.5.6 NW 2nd Ave and N Ivy St. UIC Decommission #6

Currently, there is one existing UIC (unnamed) located near the intersection of NW 2nd Ave and N Ivy St, which is out of compliance and is required to be decommissioned, and one UIC (unnamed) at the intersection identified to have wet feet and is recommended to be decommissioned.

Recommended Improvements

The recommended improvement is decommissioning of two UICs. New stormwater infrastructure is not anticipated to be required to handle existing stormwater runoff following the decommissioning of these UICs because runoff will be handled by the downtown stormwater system, which is addressed in CIPs #14 and #15. The estimated cost of the project is $40,000.



CIP #6: NW 2nd Ave and N Ivy St. UIC Decommission

Description: The existing area has one UIC that has been determined to be out of compliance.

Location: NW 2nd Ave and N Ivy St

Existing System: The existing system consists of one UIC which is out of compliance.

Proposed System: The proposed system is to decommission the UIC. No Additional stormwater infrastructure is required.

Cost: $40,000

Priority: High

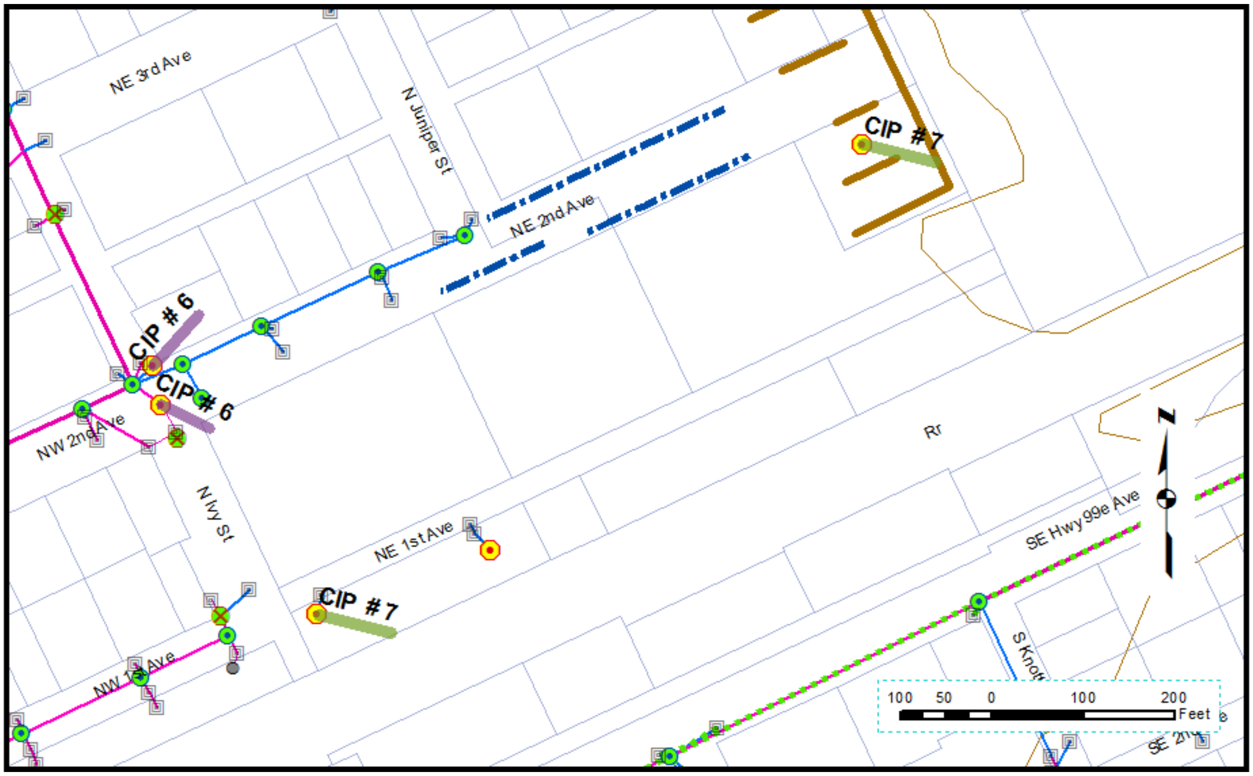
Schedule: 0 to 5 years.

6.5.7 Cinema and NW 1st and Ivy St UIC Decommission #7

Currently, there is one existing UIC (unnamed) located in the Canby Cinema parking lot near the intersection of NW 2nd Ave and N Knott St as well as one UIC located at the intersection NE 1st Ave and N Ivy St. The UIC located in the Cinema parking lot is unregistered and no longer in use and should be decommissioned, while the City would like to decommission the UIC located at NE 1st and N Ivy St. The exact location of the UICs to be decommissioned is uncertain and should be confirmed as the first step of this CIP.

Recommended Improvements

The recommended improvement is confirming the location of and decommissioning two UICs. No new or additional stormwater infrastructure is anticipated to be required to handle existing flows, as one of the UICs being decommissioned is currently not being used, and the other is located near existing stormwater infrastructure with excess capacity. The estimated cost of the project is $40,000.



CIP #7: Cinema Parking Lot UIC Decommission and NW 1st and Ivy St UIC Decommission

Description: The existing area has one UIC that is unregistered and is currently not used as well as one UIC that is requested to be decommissioned by the City.

Location: Cinema Parking Lot near NW 2nd Ave and N Knott St. and the intersection of NW 1st Ave and N Ivy St.

Existing System: The existing system consists of one UIC that is unregistered and no longer in use as well as one UIC that is requested to be decommissioned by the City.

Proposed System:

The proposed system is to decommission two UICs. No additional stormwater infrastructure is required.

Cost: $40,000

Priority: High

Schedule: 0 to 5 years.

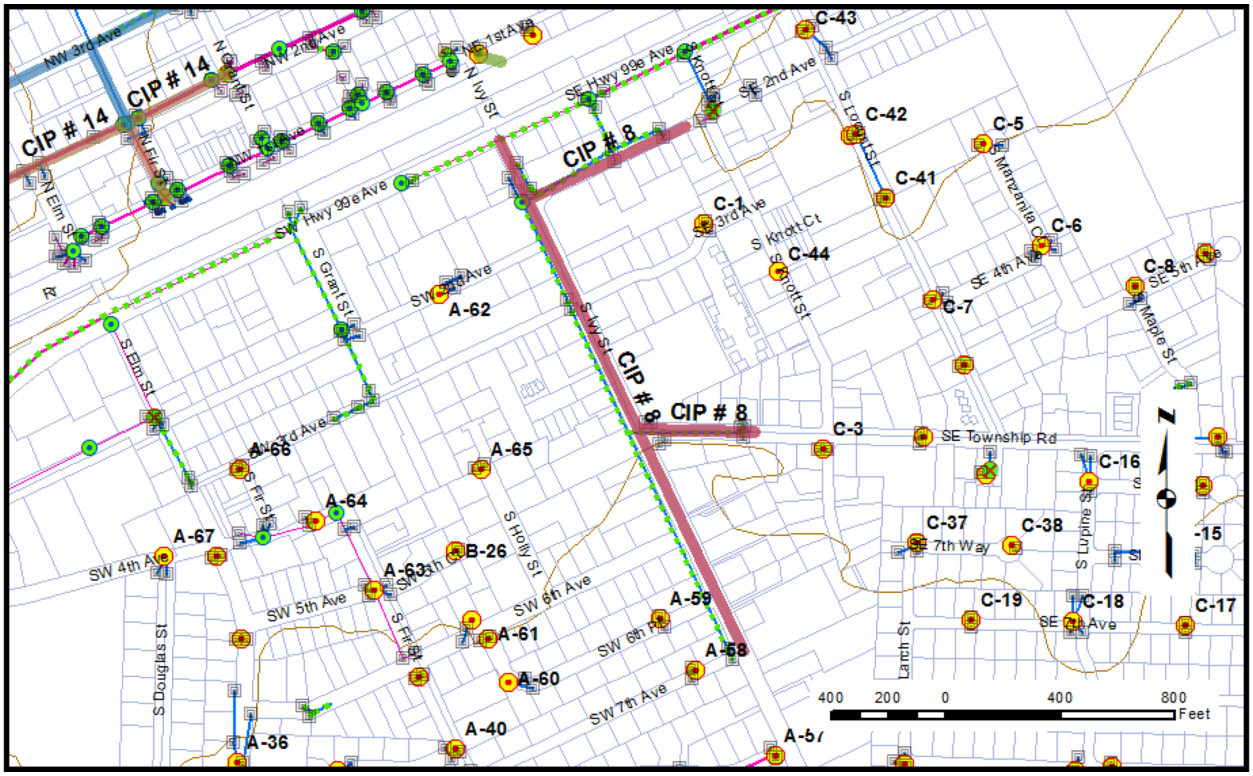
6.5.8 S Ivy St. CIP #8

Currently, periodic and minor flooding occurs in this area. The roadway can become overloaded with stormwater runoff. The current stormwater infrastructure consists of a county-owned pipe system with no manholes that is inaccessible and may be full of mud. The system ties into the ODOT system along 99E but the condition of the system is completely unknown. Groundwater modeling indicates this area may have high groundwater. The roadway in this area is under county jurisdiction and all CIPs will require coordination with the County authority. S Ivy St south of approximately SW 7th Ave is served by existing drywells that are functioning well.

Recommended Improvements

The recommended improvements consist of installing pavement piped stormwater system including new manholes and catch basins along S Ivyt St. Because of the large number of unknowns this system was not modeled. The drainage basin served is approximately 5 acres and all pipe was assumed to be 18" diameter. Approximately 3000 feet of piping is required. The estimated cost of the project is $730,000. Because S Ivy St. is currently under Clackamas County Jurisdiction this cost is considered the responsibility of Clackamas County.

In addition to the unknowns along S Ivy St., The ODOT system along Hwy 99E is also of unknown condition and capacity and was not evaluated as part of this Stormwater Master Plan. Therefore, future investigation, and analysis is require d before proceeding with this CIP.



CIP #8: S Ivy St

Description: The existing drainage system lacks sufficient capacity, and periodic flooding occurs, and the condition of the system is unknown. The recommendation is to install pavement new piped system tied into the ODOT Hwy 99E system.

Location: S Ivy St. from Hwy 99E to approximately SW 7th Ave.

Existing System: The existing system consists of capacity county owned and maintained pipe system of unknown condition and capacity.

Proposed System: The proposed system is to install a piped stormwater system. Approximately 3,000 LF of pipe is required.

Cost: $730,000 (preliminary estimate) responsibility of Clackamas County

Priority: Medium

Schedule: 6 to 10 years.

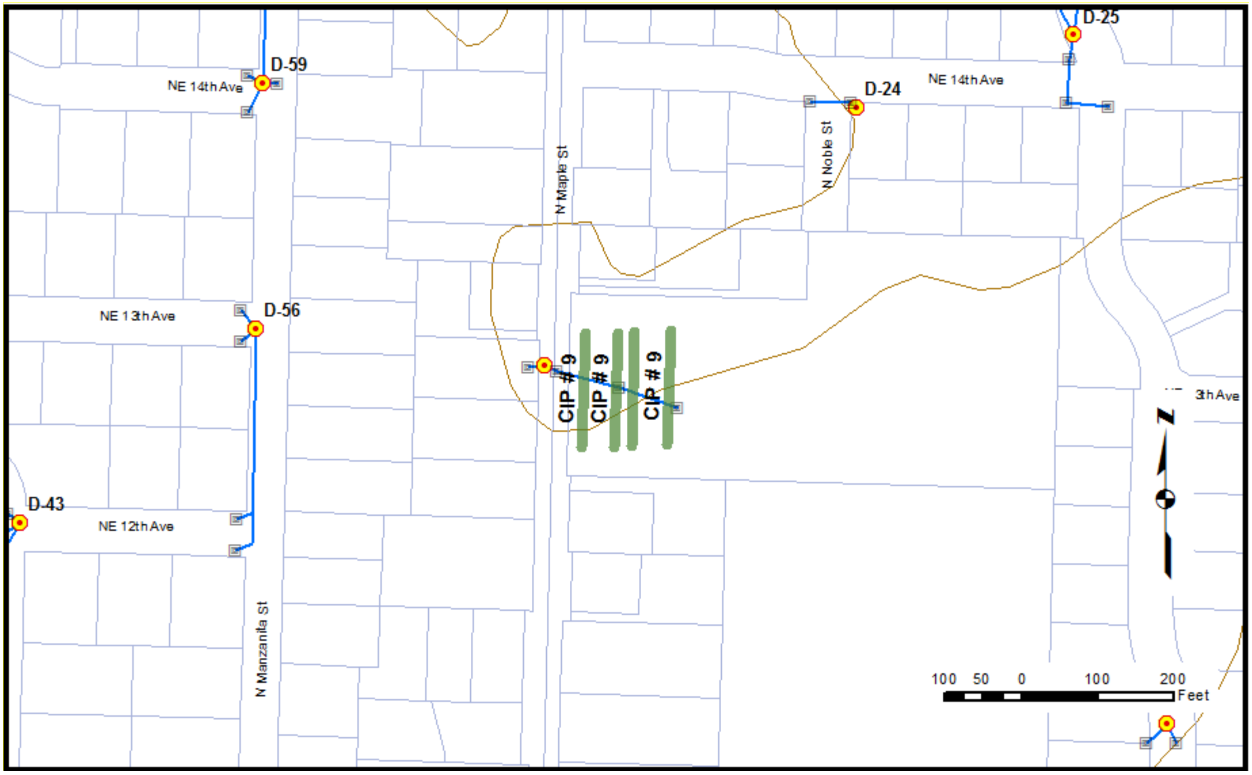
Requirements: Further investigation into the condition and capacity of this system and the ODOT system on Hwy 99E should be completed prior ot proceeding with this CIP.

6.5.9 N Maple St. at Maple St. Park CIP #9

Currently, minor and periodic flooding occurs in the parking lot of the Maple St Park. The current stormwater infrastructure consists of a slow draining UIC with insufficient capacity.

Recommended Improvements

The recommended improvements consist of installing pervious pavement within the parking lot to increase infiltration capacity. The pervious pavement should be located wihtin the parking areas. The total approximate area of pervious pavement to be installed is 4,800 ft^2. The estimated cost of the project is $30,000.



CIP #9: N Maple St. at Maple St. Park

Description: The existing UIC drainage system lacks sufficient capacity, and periodic and minor flooding occurs. The recommendation is to install pervious pavement to increase infiltration capacity.

Location: N Maple St. at the Maple St. Park

Existing System: The existing system consists of a UIC.

Proposed System: The proposed system is to install pervious pavement in parking area, to provide more infiltration capacity and mitigate flooding. The total pervious pavement to be installed is approximately 4,800 ft^2.

Cost: $30,000

Priority: Medium

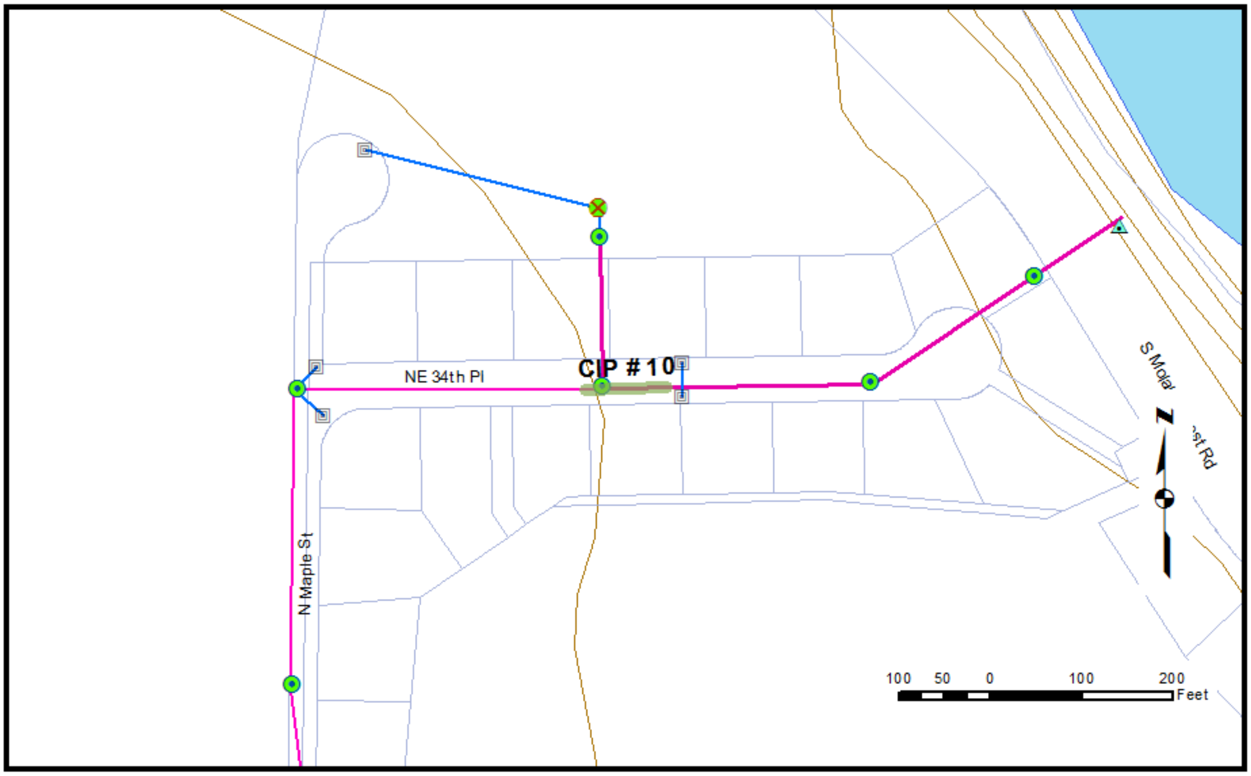
Schedule: 6 to 10 years.

6.5.10 N Maple St. and NW 34th Pl CIP #10

The current stormwater infrastructure collects runoff from the surrounding residential area as well as runoff from the adjacent farm fields and outfalls to the Willamette River. Although the quality of the stormwater has not been analyzed, it is reasonable to assume that the stormwater has high turbidity due to the runoff from the adjacent agriculture fields. It is anticipated that ORDEQ may require the installation of a flow-through treatment system prior to the outfall.

Recommended Improvements

The recommended improvement consists of installing a flow through treatment system at the third manhole upstream from the Willamette River outfall approximately half way along NE 34th Pl. This location will treat the majority of the basin but the storm system is not excessively deep. The estimated cost of the project is $30,000.



CIP #10: N Maple St. and N 34th Pl

Description: The existing drainage system collects runoff from the surrounding residential area, as well from nearby farm fields and outfalls to the Willamette River. It is assumed that the outfall will be required to have flow through treatment in the future.

Location: N Maple St. and N 34th Pl

Existing System: The existing system consists of UICs connected in series via stormwater pipes and manholes; the system outfalls to the Willamette River.

Proposed System: The proposed system involves the installation of a flow-through treatment system for suspended solids removal, at the manhole upstream from the existing outfall approximately half way along NE 34th Pl.

Cost: $30,000

Priority: Medium

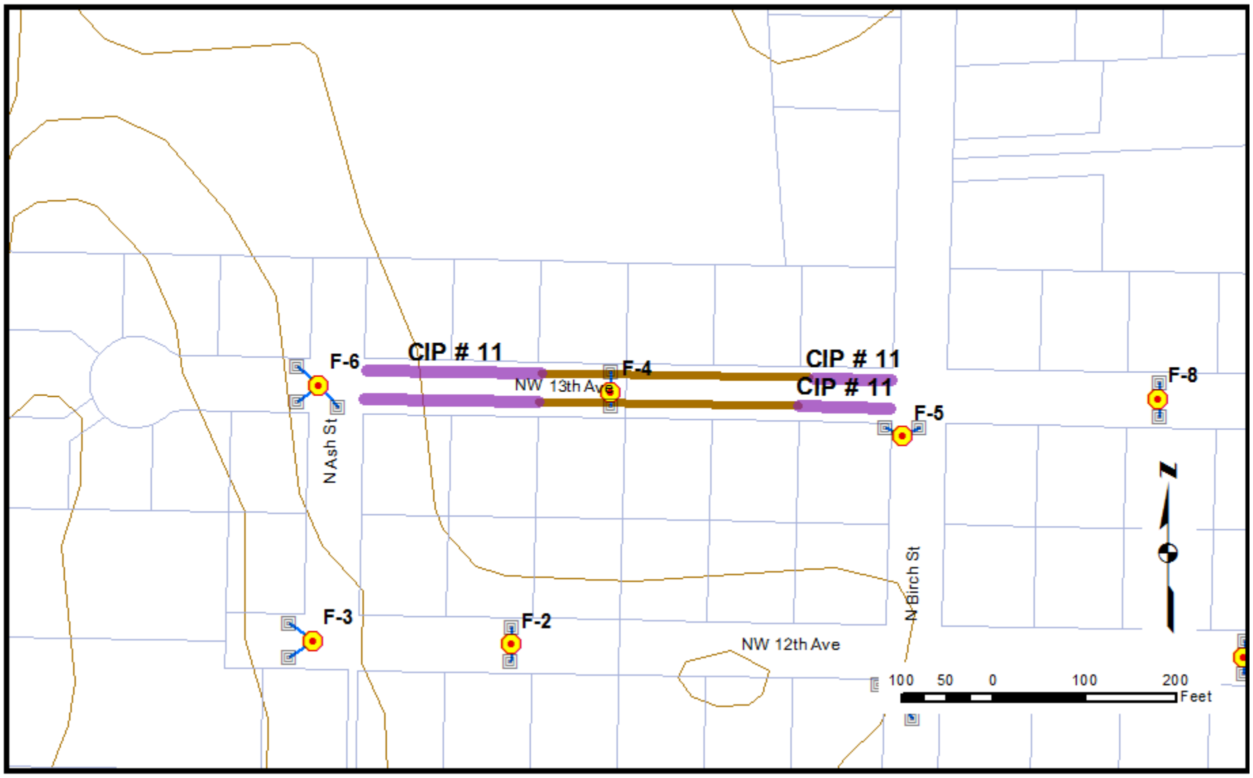
Schedule: 6 to 10 years. The priority level of this project may change, depending on ORDEQ requirements.

6.5.11 NW 13th Ave from N Ash to N Birch St. CIP #11

Currently, periodic and minor flooding occurs along this corridor. The roadway can become overloaded with stormwater runoff. The current stormwater infrastructure consists of UICs with insufficient capacity and some pervious pavement.

Recommended Improvements

The recommended improvements consist of installing additional pervious pavement within the existing right-of-way, within the bike lane, road shoulder, and parking lane to increase infiltration capacity. The pervious pavement should be located on NW 13th Ave, between the N Ash St and N Birch St. The total approximate area of pervious pavement to be installed is 4,800 ft^2. The estimated cost of the project is $40,000.



CIP #11: NW 13th Ave from N Ash to N Birch St.

Description: The existing UIC drainage system lacks sufficient capacity, and periodic flooding occurs. The recommendation is to increase the infiltration capacity of the area through the installment of stretches of pervious pavement.

Location: Along NW 13th Ave from N Ash to N Birch St.

Existing System: The existing system consists of UICs and some pervious pavement.

Proposed System: The proposed system is to install pervious pavement in the bike lane, shoulder and parking lanes of all the street, to provide more infiltration capacity and mitigate flooding. The total pervious pavement to be installed is approximately 4,800 ft^2.

Cost: $30,000

Priority: Medium

Schedule: 6 to 10 years.

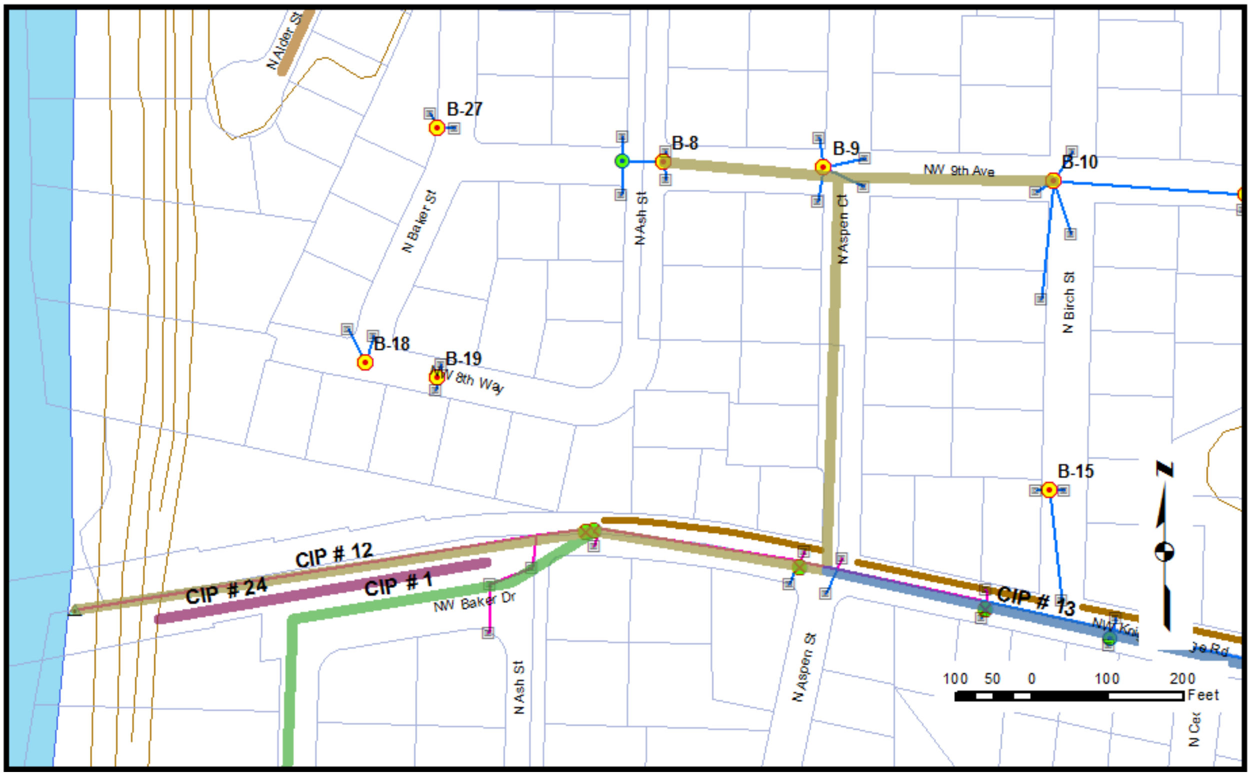
6.5.12 NW 9th Ave from N Ash to N Birch St. CIP #12

Flooding has occured in this area, especially at each intersection. The flooding has currently been mitigated through installation of pervious pavement on NW 9th Ave. near N Cedar Ct. and through cleaning of the drywells, however it is anticipated that flooding is likely to occur again in the future. The roadway can become overloaded with stormwater runoff. The current stormwater infrastructure consists of slow draining UICs with insufficient capacity. Depending on the storm intensity this area experiences, flooding can remain in the area for six hours, up to three days.

Recommended Improvements

The recommended improvements consist of connecting this drainage area to the stormwater line located along N Knights Bridge Rd. The existing UICs should be connected in series along NW 9th St, and then piped down N Aspen Court (Ct) to tie into the stormwater conveyance system on N Knights Bridge Rd. An accurate survey of elevations in this system should be completed to confirm the proposed construction can be accommodated within existing grades prior to initiating any other activity for this CIP.

The additional runoff to the Knights Bridge Rd system will require that the storm drain pipe along Knights Bridge Rd from N Aspen Ct to the outfall be upsized as well. The total approximate length of pipe to be installed is 500 LF of 12” diameter and 1,500 LF of 18” diameter HDPE pipe. The estimated cost of the project is $420,000.



CIP #12: NW 9th Ave from N Ash to N Cedar St.

Description: The existing UIC drainage system lacks sufficient capacity and is slow draining. Insufficient capacity leads to flooding, especially at the intersections. The recommendation is to connect the existing UICs via storm drain piping, and connect the drainage area to the Knights Bridge drainage and outfall system. This will require that the storm drain pipe along Knights Bridge Rd from N Aspen Ct to the outfall be upsized as well.

Location: NW 9th Ave from N Ash to N Birch St, from N Aspen St to Knights Bridge Rd, and from Knights Bridge road to the outfall to the Molalla located on Knights Bridge.

Existing System: The existing system consists of UICs. UIC's B-11 and B-10 are connected by existing piping, which may be 10" diameter concrete.

Proposed System: The proposed system for mitigating the existing flooding consists of 12" diameter HDPE pipe connecting the existing UICs in series along NW 9th St from N Ash to N Birch St. The NW 9th UICs will be connected to NW Knights Bridge Rd via an 18" diameter HDPE pipe along N Aspen Ct, connecting this drainage basin to the existing system on NW Knights Bridge Rd, which will require upsizing to an 18" diameter HDPE pipe from N Aspen Ct to the outfall. The total length of pipe to be installed is approximately 500 LF of 12" diameter HDPE and approximately 1,500 LF of 18" diameter HDPE.

Cost: $420,000

Priority: High

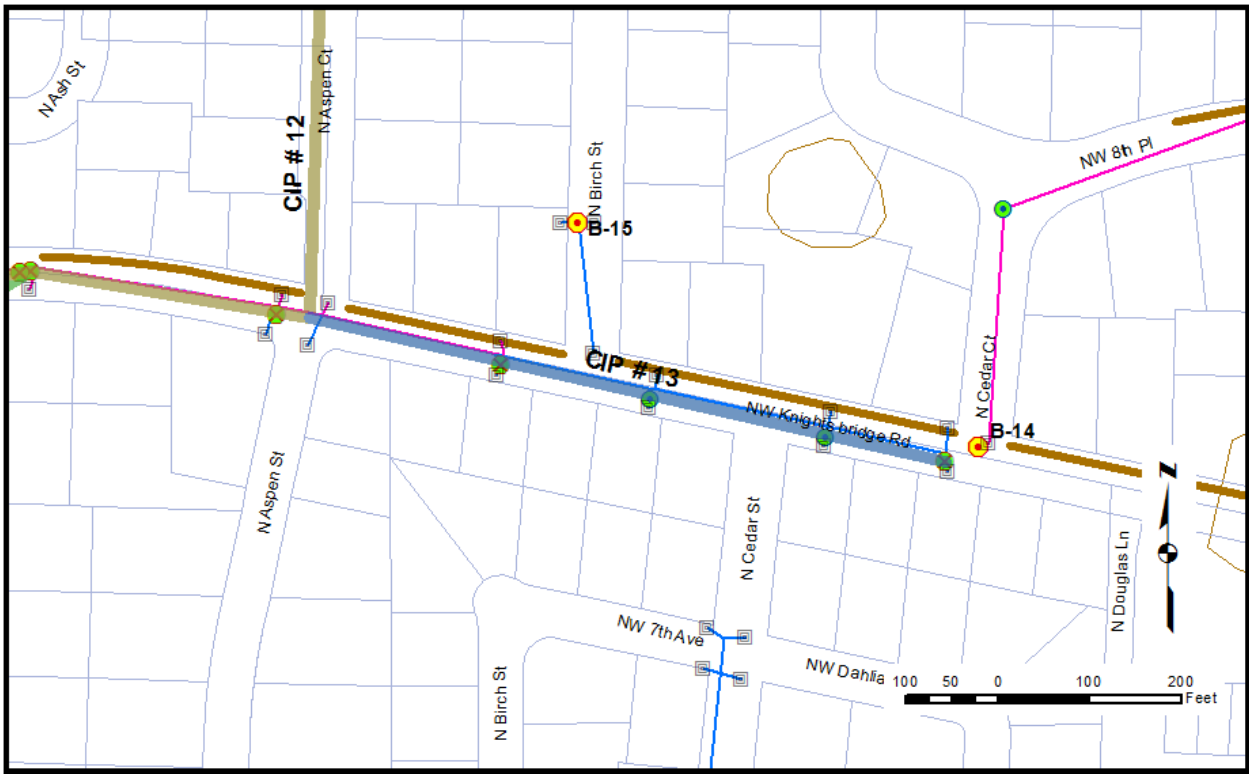
Schedule: 0 to 5 years. A survey of the existing system should be completed before initiating any other work.

6.5.13 N Knights Road CIP #13

The existing 8” diameter pipe located beneath Knights Bridge Rd from N Aspen Ct to N Cedar Ct currently has misaligned joints and has been infiltrated with tree roots. This greatly reduces the capacity of the existing pipe and the efficiency of the system.

Recommended Improvements

The recommended improvements consist of replacing the existing 8” diameter pipe with new 12” diameter HDPE pipe. The alignment should tie into the existing alignment at Knights Bridge Rd and N Aspen Ct., which outfalls to the Molalla River. The alignment consists of approximately 750 LF of 12” diameter HDPE. The estimated cost of the project is $130,000.



CIP #13: N Knights Bridge Rd

Description: The existing 8" diameter storm drain pipe has misaligned joints and has been infiltrated by tree roots.

Location: N Knights Bridge Rd from N Cedar Ct. to N Aspen Ct.

Existing System: The existing system consists of 8" concrete and corrugated storm drain pipe.

Proposed System: The proposed system includes removing the existing 8" diameter pipe and replacing the alignment with 12" diameter HDPE pipe, tying into the existing Knights Bride Rd alignment at N Aspen Ct. The project consists of approximately 750 LF of 8" diameter HDPE pipe.

Cost: $130,000

Priority: Medium

Schedule: 6 to 10 years.

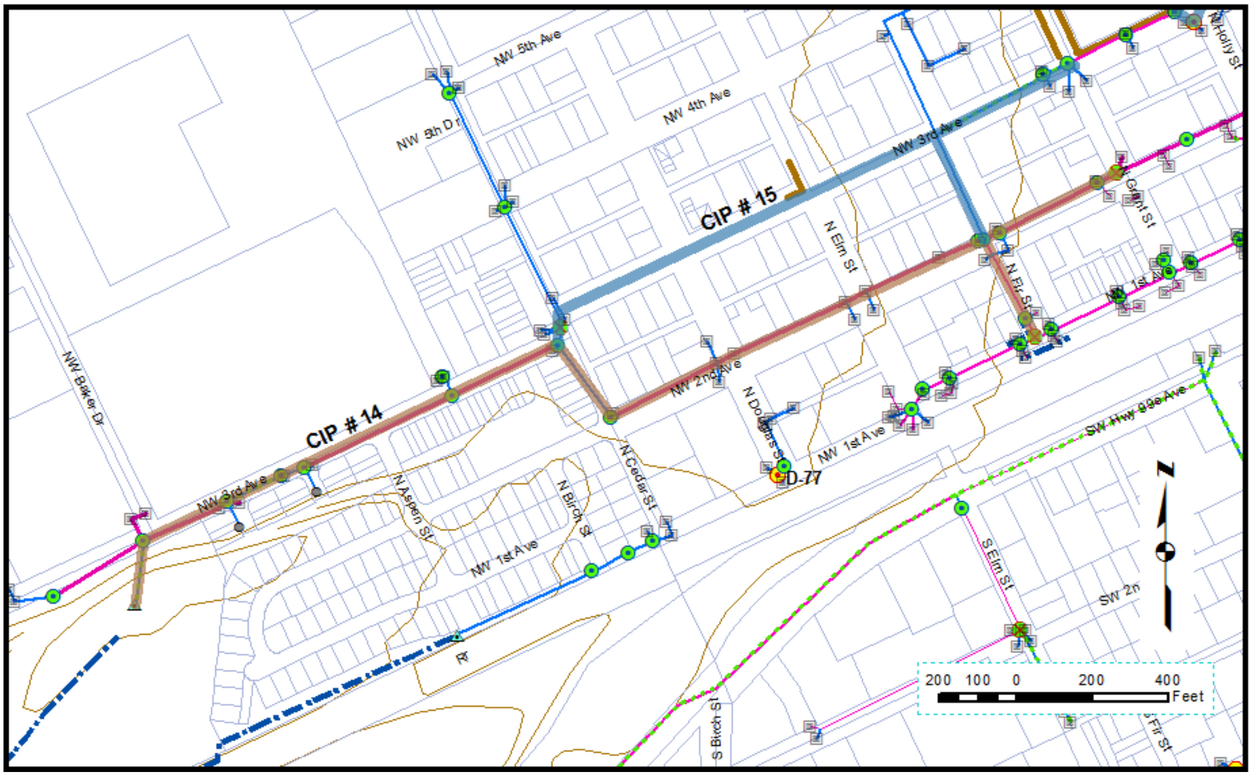
6.5.14 NW 2nd Ave from N Grant St. to NW Baker Dr. CIP #14

Stormwater runoff and conveyance modeling for this area of downtown Canby has shown that the existing infrastructure is undersized, which can lead to surcharging in the pipes and manholes as well as flooding in the area. Furthermore, the existing pipe is old, is in bad condition, has roots, has been observed to be flowing full and needs replacement.

Recommended Improvements

Prior to initiating any capital improvements to the downtown system (CIP #15 and CIP #16) the city should conduct flow monitoring to verify and calibrate model results. Hydrologic and hydraulic models are particularly sensitive in areas with relatively flat slopes such as found in the downtown areas. Flow monitoring should be conducted at the outfall to the detention pond on NW 3rd Ave, at the intersection of NW 2nd Ave and N Cedar St. and at the intersection of NW 3rd Ave and N Fir St. If model results calibrated through flow monitoring show the existing pipe size is adequate, then cleaning and lining of the existing pipe may be an alternative lower cost option. To be conservative, the estimated cost for pipe replacement with a larger size is presented in this CIP.

The recommended improvements consist of increasing the capacity of the existing conveyance system alignment by replacing the existing pipe with larger diameter pipe. The total approximate length of pipe to be installed is 400 LF of 30" diameter HDPE pipe and 1,350 LF of 36" HDPE. The estimated cost of the project is $690,000.



CIP #14: NW 2nd Ave from N Grant St. to NW Baker Dr.

Description: This area is an integral part of the downtown stormwater conveyance system, and currently consists of multiple pipe segments of different diameters and slopes. Stormwater runoff modeling for this portion of the downtown system demonstrated a lack of conveyance capacity in the existing system, causing surcharging in the pipes and manholes, with the potential for flooding.

Location: NW 2nd Ave from N Cedar St. to N Grant St., and NW 3rd Ave from N Grant St. to NW Baker Dr.

Existing System: The existing system consists of piped storm drain systems of various sizes and slopes.

Proposed System: The proposed system is to increase the capacity of the existing system. The CIP includes upsizing the existing pipe from N Grant St to N Fir St. from 21" with approximately 400 LF of 30" HDPE, as well as replacing the existing 30" pipe from N Fir St. to NW 3rd and NW Baker Dr. at the outfall to the pond with approximately 1,350 LF of 36" HDPE.

Cost: $690,000

Priority: Medium

Schedule: 6 to 10 years.

6.5.15 NW 3rd Ave from N Cedar St. to N Holly St. CIP #15

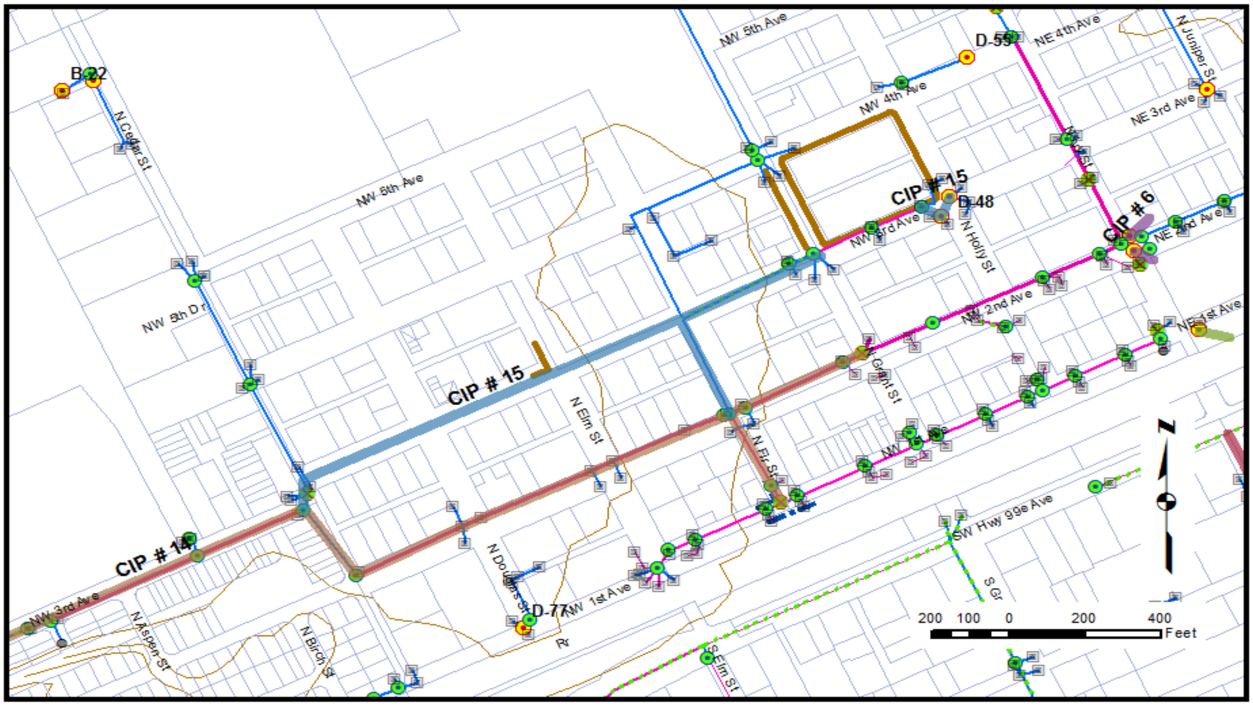
Stormwater runoff and conveyance modeling for this area of downtown Canby has shown that the existing infrastructure is undersized, which can lead to surcharging in the pipes and manholes as well as flooding in the area. To help mitigate surcharging and flooding in the existing conveyance system, CIP #16 involves the installation of a new stormwater conveyance pipeline along NW 3rd Ave from N Holly St. to N Cedar St.

Recommended Improvements

Prior to initiating any capital improvements to the downtown system (CIP #15 and CIP #16) the city should conduct flow monitoring to verify and calibrate model results. Hydrologic and hydraulic models are particularly sensitive in areas with relatively flat slopes such as found in the downtown areas. Flow monitoring should be conducted at the outfall to the detention pond on NW 3rd Ave, at the intersection of NW 2nd Ave and N Cedar St. and at the intersection of NW 3rd Ave and N Fir St. Model results calibrated through flow monitoring will provide more accurate estimates of required pipe sizes.

Decommissioning of the two UICs that is described below should be completed only after the pipe improvements have been completed so that significant flooding problems are not created in the downtown area. In addition, this project should not be completed until CIP #15 has been completed, increasing the capacity of NW 3rd St between N Cedar St. and N Baker Dr.

The recommended improvements consist of mitigating the flow from existing stormwater conveyance structures by installing a new stormwater pipeline along NW 3rd Ave between N Holly St and N Cedar St. The CIP also involves the decommission of two UICs identified for removal by GSI, as well as the capping of an existing 21" stormwater pipe at NW 3rd Ave and N Fir St. The total approximate length of pipe to be installed is 150 LF of 18" diameter HDPE pipe, 400 LF of 20" diameter HDPE, 1,200 LF of 30" diameter HDPE pipe and 2 new manholes and eight new catch basins. The estimated cost of the project is $670,000.



CIP #15: NW 3rd Ave from N Cedar St. to N Holly St.

Description: Stormwater runoff and conveyance modeling for the existing downtown Canby basin shows a lack of adequate capacity, which can lead to pipe and manhole surcharging as well as flooding.

Location: NW 3rd Ave from N Cedar to N Holly St.

Existing System: The current system on NW 3rd St. includes two UICs located near Wait Park at the corner of NW 3rd Ave and N Holly St, a relatively new pipe system connecting catch basins and manholes near NW 3rd and N Grant St to the UICs at N Holly St. and an older pipe under NW 3rd between N Grant St. and N Fir St. The majority of the stormwater runoff in the downtown Canby is currently handled by the conveyance infrastructure of various sizes along NW 2nd Ave.

Proposed System: The proposed system is a new stormwater conveyance pipeline along NW 3rd Ave from N Grant St. to N Cedar St., connecting to the existing system located at NW 3rd and Cedar St. The CIP also includes decommissioning the two existing UICs located near Wait Park at the corner of NW 3rd Ave and N Holly St. The system will consist of approximately 150 LF of 18" diameter HDPE pipe in the area of N Holly to N Grant St, approximately 400 LF of 24" diameter HDPE pipe from N Grant St. to N Fir St. as well as 1,200 LF of 30" diameter HDPE from N Fir St. to N Cedar St. The project also includes the installation of 2 new manholes and 8 new catch basins.

Cost: $670,000

Priority: Medium

Schedule: 6 to 10 years.

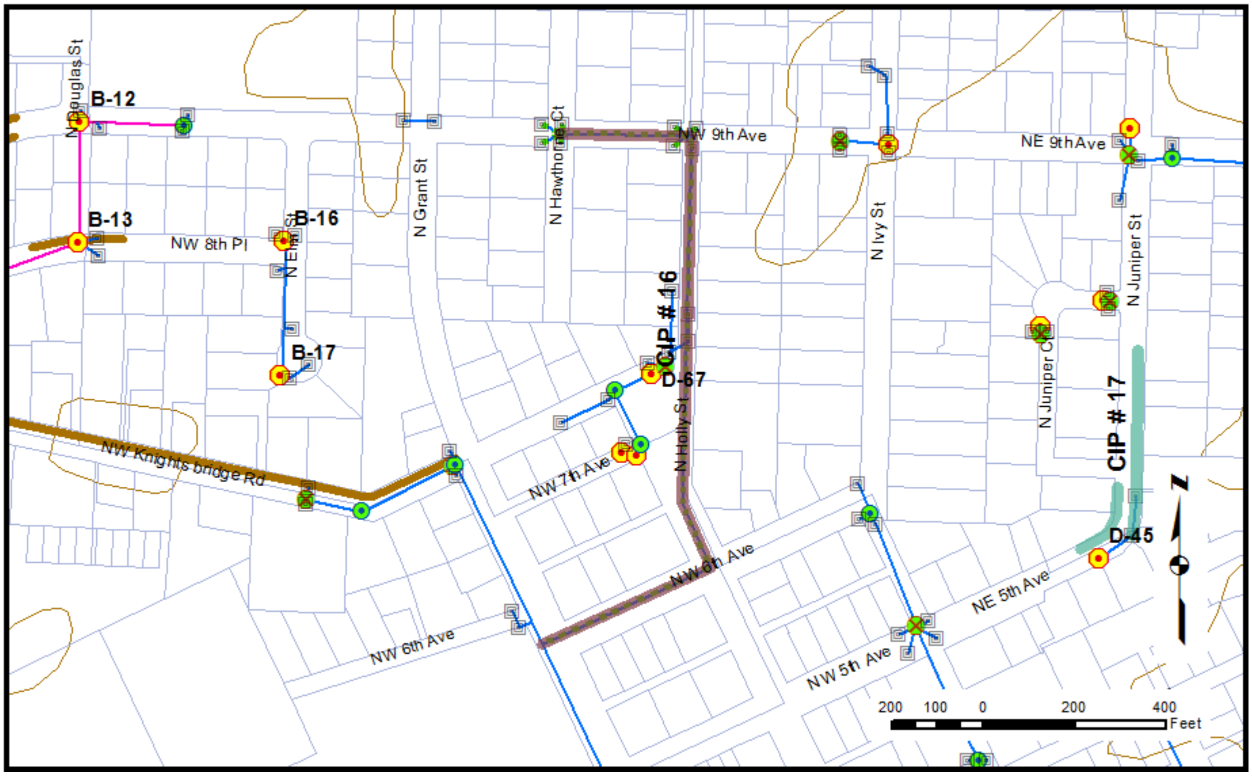
Requirements: This project should not be completed until CIP #15 has been completed, increasing the capacity of NW 3rd St between N Cedar St. and N Baker Dr.

6.5.16 N Holly St. CIP #16

The N Holly St. drainage basin CIP project extends from NW 9th Ave to NW 6th Ave along N Holly, and then extends to N Grant St. Stormwater runoff and conveyance modeling for this area has shown that the existing infrastructure is undersized, which can lead to surcharging in the pipes and manholes as well as flooding in the area.

Recommended Improvements

The recommended improvements consist of increasing the capacity of the existing conveyance system alignment by replacing the existing pipe with larger diameter pipe. The total approximate length of pipe to be installed is 1,650 LF of 12” diameter HDPE pipe as well as the installation of three manholes. The estimated cost of the project is $310,000.



CIP #167: N Holly St.

Description: The N Holly St. drainage basin CIP project extends from NW 9th Ave to NW 6th Ave along N Holly St., and then extends to N Grant St. Stormwater runoff modeling for this portion of the downtown system demonstrated a lack of conveyance capacity in the existing system, causing surcharging in the pipes and manholes, and a potential for flooding.

Location: N Holly St from NW 9th Ave to NW 6th Ave and from N Holly St to N Grant St along NW 6th Ave.

Existing System: The existing system consists of an 8" diameter piped storm drain system.

Proposed System: The proposed system is to increase the capacity of the existing system. The CIP includes upsizing the existing pipe from N Holly St from NW 9th Ave to NW 6th Ave and from N Holly St. to N Grant St. along NW 6th Ave with approximately 1,650 LF of 12" HDPE.

Cost: $310,000

Priority: Medium

Schedule: 6 to 10 years.

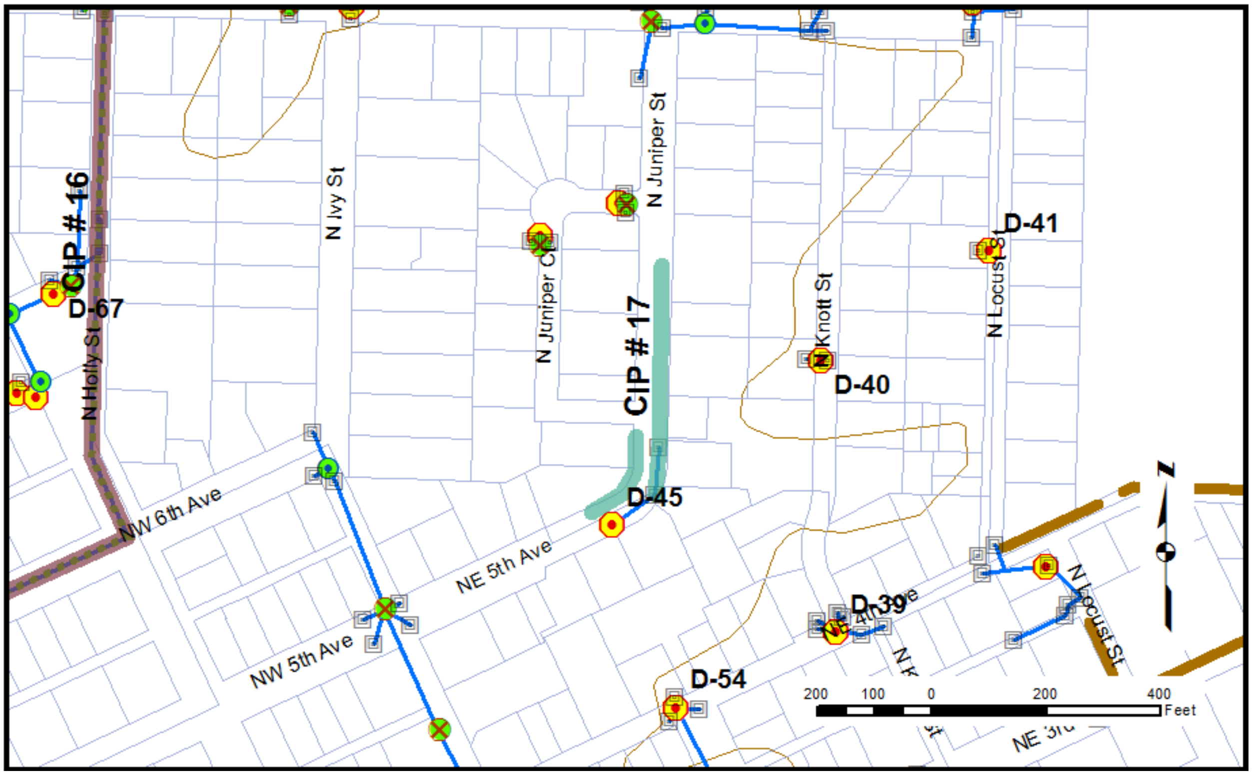
Requirements: This project should not be completed until CIP #15 and CIP #16 have been completed, increasing the capacity on NW 3rd St.

6.5.17 N Juniper and NE 5th Ave #17

The area near N Juniper and NE 5th Ave is currently underserved by stormwater infrastructure, with only one UIC in the area. The roadway can become overloaded with stormwater runoff. Furthermore, according to the City Public Works Department, the pedestrian and transportation infrastructure in the area is in need of a redesign and rehabilitation, including the installation of new roadway shoulders and sidewalks.

Recommended Improvements

The recommended improvement to N Juniper and NE 5th Ave is the installation of approximately 4,800 ft^2 of pervious pavement within the roadway shoulder and parking lane. To save costs, the CIP should be coupled with future roadway improvement projects. The estimated cost of the project is $30,000.



CIP #17: N Juniper and NE 5th Ave

Description: The existing area is underserved by stormwater infrastructure, and currently is susceptible to flooding. The roadway is in need of improvements according to City Public Works staff, including the repaving and sidewalks.

Location: N Juniper and NE 5th Ave

Existing System: The existing system consists of one UIC in the vicinity of the flooding.

Proposed System: The proposed system includes coupling the stormwater improvement project with transportation infrastructure improvements, and installing approximately 4,800 ft^2 of pervious pavement to infiltrate stormwater.

Cost: $30,000

Priority: Medium

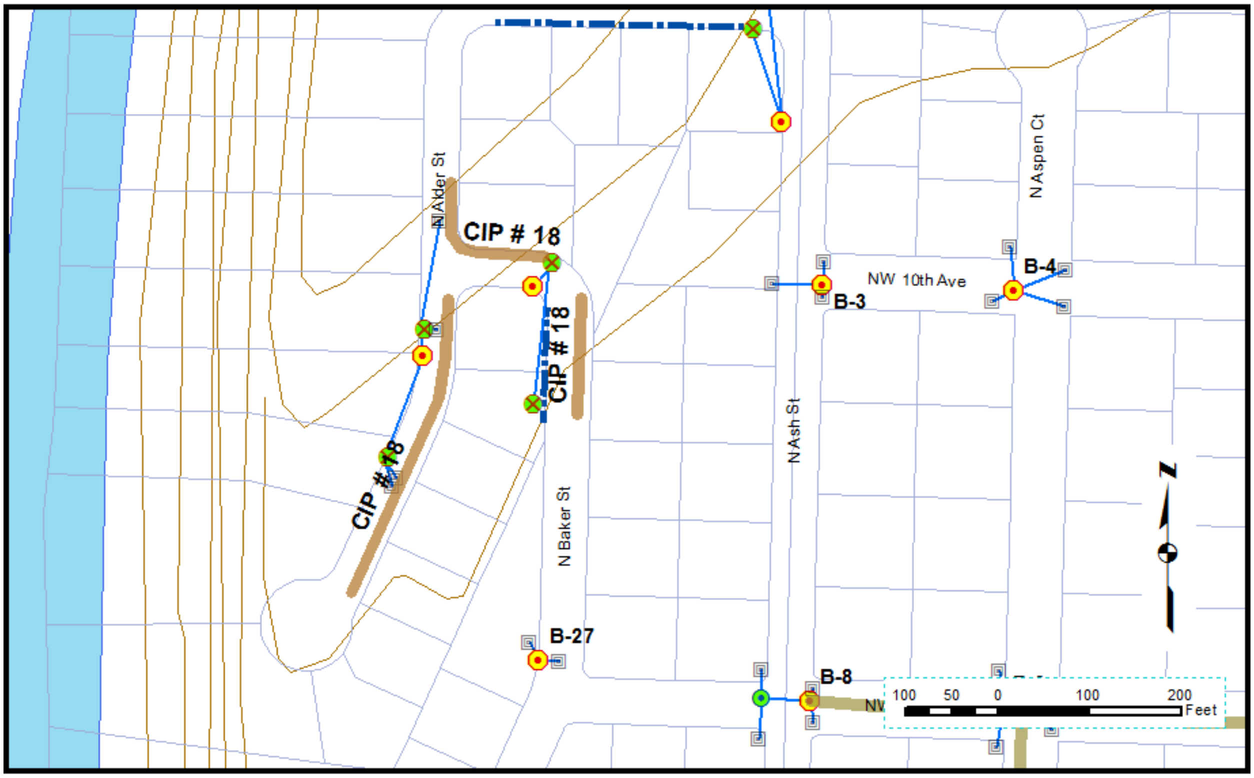
Schedule: 6 to 10 years.

6.5.18 N Baker St. and N Alder St.#18

The intersection of N Alder and N Baker St is currently underserved by stormwater infrastructure, with two slow draining UICs in the area. The roadway can become overloaded with stormwater runoff.

Recommended Improvements

The recommended improvement to N Alder and N Baker St is the installation of approximately 4,800 ft^2 of pervious pavement within the roadway shoulder and parking lane. The estimated cost of the project is $30,000.



CIP #18: N Alder and N Baker St.

Description: The existing area is underserved by stormwater infrastructure, and currently is susceptible to flooding.

Location: N Alder and N Baker St.

Existing System: The existing system consists of two slow draining UICs.

Proposed System: The proposed system includes the installation of approximately 4,800 ft^2 of pervious pavement to infiltrate stormwater.

Cost: $30,000

Priority: Medium

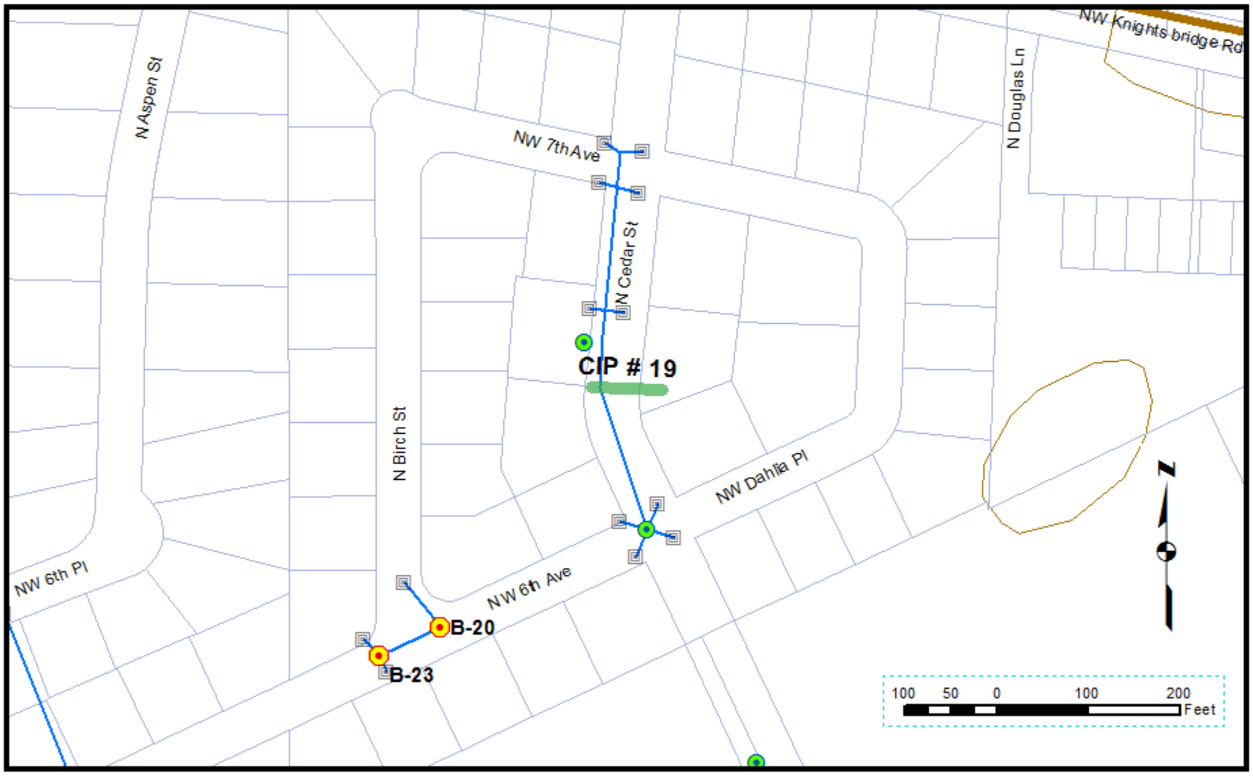
Schedule: 6 to 10 years.

6.5.19 N Cedar CIP #19

An existing storm drain line along N Cedar St from NW 7th Ave to NW Dahlia Pl drains to a UIC at the intersection of NW Dahlia Pl. The current line consists of catch basins and a UIC, but does not have any existing manholes along the line, which creates difficult conditions for cleaning the pipe.

Recommended Improvements

The recommended improvement to the N Cedar line is to install a new manhole within the alignment, mid-block, between NW 7th Ave and NW Dahlia Pl at the location of existing catch basins to allow for cleaning of the storm drain line. The existing catch basins should connect to the new man holes. The estimated cost of the project is $10,000.



CIP #19: N Cedar St.

Description: The existing area consists of a storm drain line draining to a UIC; the alignment has no manholes along the long stretch of pipe, creating maintenance difficulties.

Location: N Cedar St between NW 7th Ave and NW Dahlia Pl.

Existing System: The existing system consists of two slow draining UICs

Proposed System: The proposed system includes installing a new manhole within the alignment, mid-block between NW 7th Ave and NW Dahlia Pl, at the location of existing catch basin connection.

Cost: $10,000

Priority: Low

Schedule: 10 to 20 years.

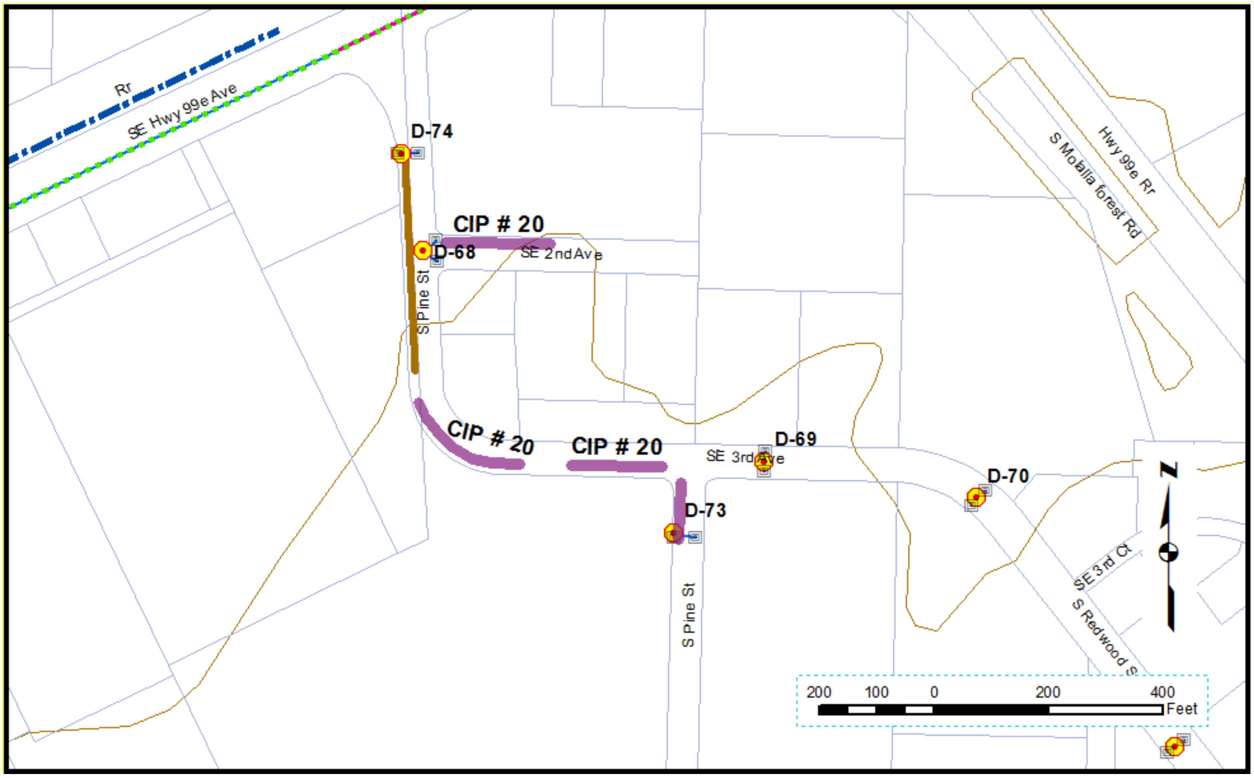
6.5.20 S Pine St. and SE 2nd Ave and SE 3rd Ave CIP #20

Currently, significant flooding occurs at the intersections of S Pine St. and SE 2nd Ave and S Pine St. and SE 3rd Ave. The roadway often becomes overloaded with stormwater runoff. The current stormwater infrastructure consists of unmaintained and poorly functioning privately owned UICs.

Recommended Improvements

A phased approach is recommended to address this issue. Because the problem is suspected to be unmaintained private UICs, the City should first clean and maintain these private UICs one time and monitor the performance of the stormwater system. If the flooding is resolved for the short term, then the private UICs will have been confirmed to be the likely cause of the problem. The City can then work with the owners of the private system to correctly maintain their system, or proceed with the CIP and secure funding as appropriate.

Assuming the system is not resolved through maintenance, the recommended improvements consist of installing pervious pavements within the bike lane, shoulder and parking lanes of the roadways. The total approximate area of pervious pavement to be installed is 4,800 ft^2. The estimated cost of the project is $30,000.



CIP #20: S Pine St. and SE 2nd Ave

Description: Significant flooding occurs at the intersections of S Pine St. and SE 2nd Ave and S Pine St. and SE 3rd Ave. The recommendation is to install stretches of pervious pavement following a phased maintenance approach.

Location: Intersection of S Pine St. and SE 2nd Ave and S Pine St. and SE 3rd Ave.

Existing System: The existing system consists of privately owned UICs which are poorly maintained and have insufficient capacity.

Proposed System: The proposed system is pervious pavement in the bike lane, shoulder and parking lanes to provide more infiltration capacity and mitigate flooding, if necessary following maintenance of existing facilities. The total pervious pavement to be installed is approximately 4,800 ft^2.

Cost: $30,000

Priority: High

Schedule: 0 to 5 years.

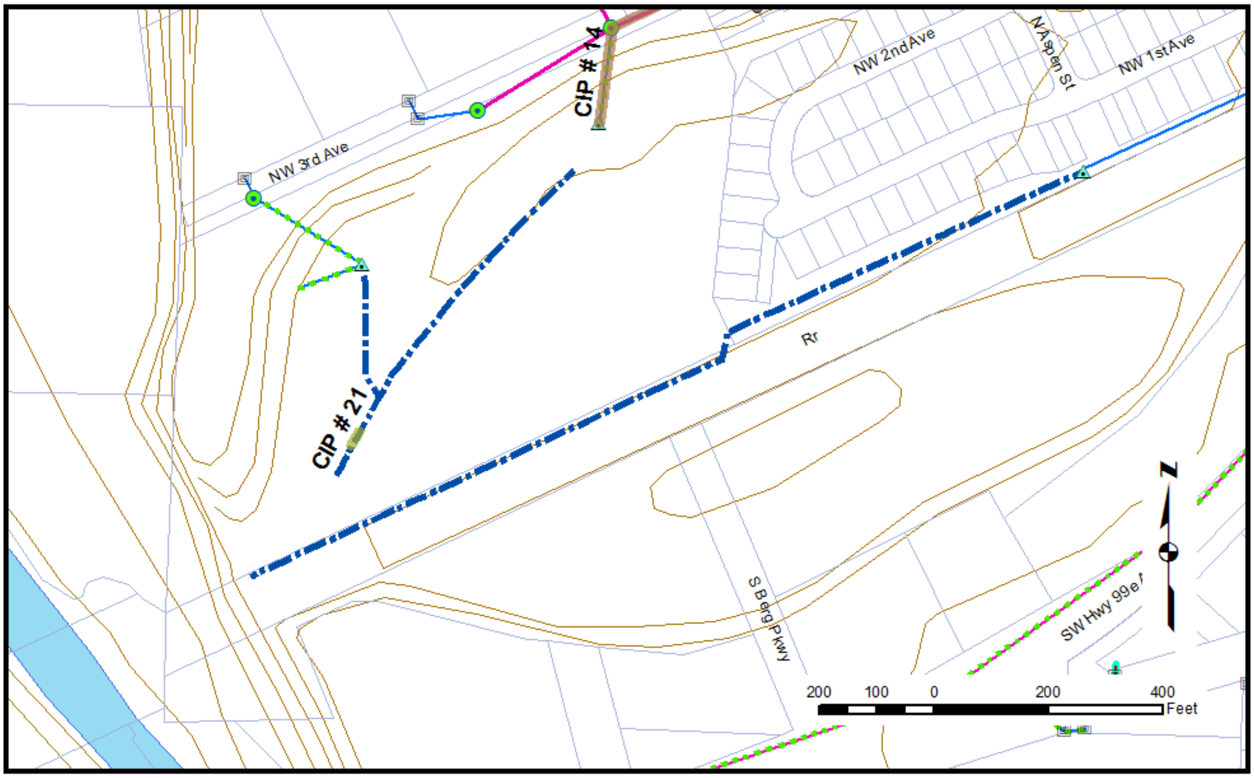
Requirements: This CIP should only be implemented following a phased approach beginning with maintenance of the private UICs.

6.5.21 Police Station/NW 3rd Ave Pond CIP #21

The City of Canby currently utilizes a constructed stormwater detention pond within an empty field near the Police Station located at NW 3rd Ave. The Detention pond flows to a stormwater conveyance swale which outfalls to the Molalla River.

Recommended Improvements

The recommended improvements to this detention pond is to install a flow monitoring system located just upstream of the existing culverts. This flow monitoring system will allow the City to quantify the discharge occurring from the stormwater pond. The system should consist of constructed concrete channel or flume, a stilling well connected to the concrete channel, and a flow measurement data logger. The estimated cost of the project is $30,000.



CIP #21: Police Station/NW 3rd Ave Pond

Description: The existing area consists of a stormwater detention pond and a grassy swale used to convey stormwater to an outfall on the Molalla River.

Location: The swale across an undeveloped field near the Police Station on NW 3rd Ave.

Existing System: The existing system consists of a stormwater detention pond and a grassy swale used for stormwater conveyance and discharge

Proposed System: The proposed system includes the installation of a flow monitoring and data collection system located within the grassy swale, just upstream of the existing culverts. The system should consist of a concrete channel and flume, along with a stilling well and a flow measurement data logger.

Cost: $30,000

Priority: Low

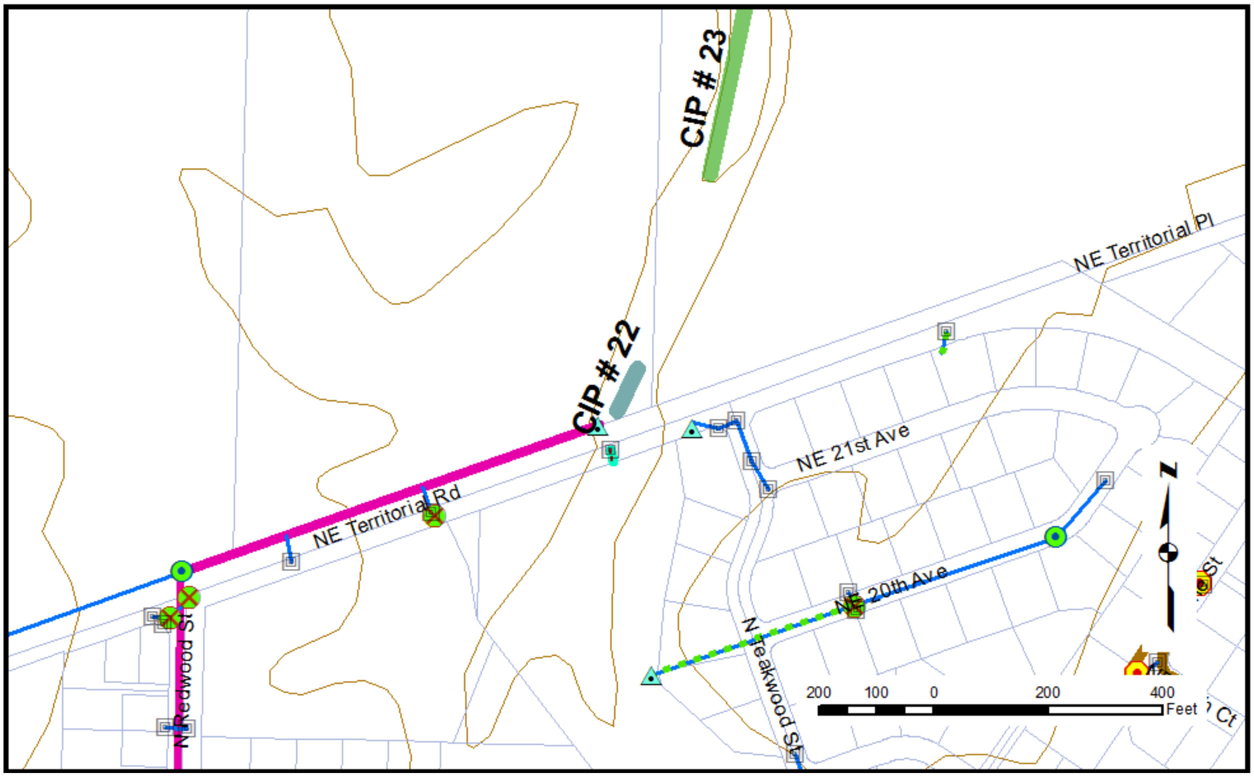
Schedule: 10 to 20 years.

6.5.22 Fish Eddy Wetland Flow Monitoring #22

Willow Creek is an existing creek which collects stormwater, conveys stormwater runoff to the location of the future Fish Eddy wetland.

Recommended Improvements

The recommended improvements to this stormwater conveyance creek are to install a flow monitoring system located within Willow Creek, just North of NE Territorial Rd. This flow monitoring system will allow the City to quantify the discharge occurring from the stormwater pond, and provide guidance for sizing the future Fish Eddy Wetland. The system should consist of constructed concrete channel or flume, a stilling well connected to the concrete channel, and a flow measurement data logger. The estimated cost of the project is $30,000.



CIP #22: Fish Eddy Wetland Flow Monitoring

Description: The existing area consists of Willow Creek, which collects and conveys stormwater runoff through the future Fish Eddy Wetland site.

Location: Willow Creek, North of NE Territorial Rd

Existing System: Willow Creek

Proposed System: The proposed system includes the installation of a flow monitoring and data collection system located within Willow Creek, North of NE Territorial Rd. The system should consist of a concrete channel and flume, along with a stilling well and a flow measurement data logger.

Cost: $30,000

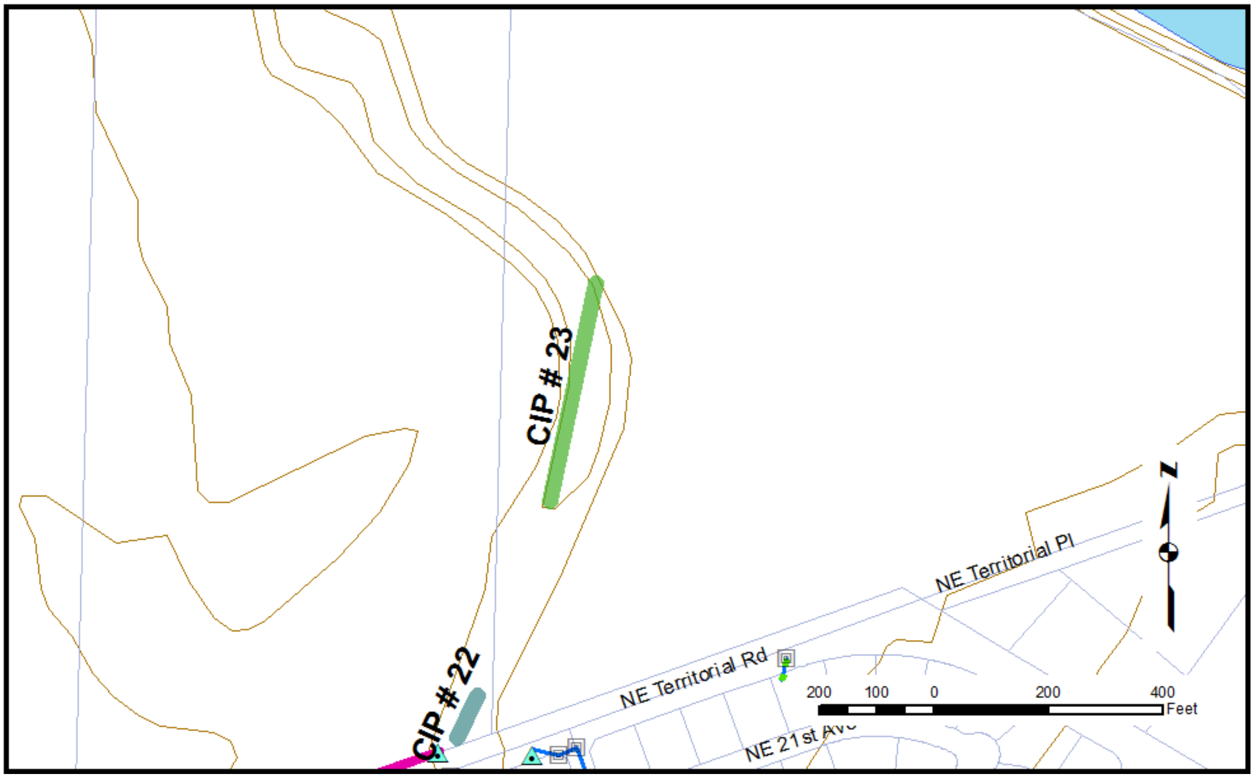
Priority: Low

Schedule: 10 to 20 years.

6.5.23 Fish Eddy Wetland #23

Recommended Improvements

A stormwater treatment wetland is proposed on the Fish Eddy property north of NE Territorial Rd and adjacent to the existing Willow Creek, as discussed in Section 5.3. This treatment wetland is part of a restoration of the entire Fish Eddy property to native seasonal wetland and wet prairie habitat as part of the Willamette Wayside Properties Master Plan. The estimated cost for the treatment wetland is $670,000.



CIP #23: Fish Eddy Wetland

Description: A stormwater treatment wetland as part of the Willamette Wayside Master Plan

Location: Willow Creek, north of NE Territorial Rd

Existing System: Willow Creek and agricultural field.

Proposed System: A stormwater Treatment Wetland sized to provide water quality treatment, based on the 6-month, 24-hour storm as described in Section 5.3. The cost estimate is for design and construction of the treatment wetland and does not include interpretive features, boardwalks etc., or surrounding landscaping.

Cost: $670,000

Priority: Low

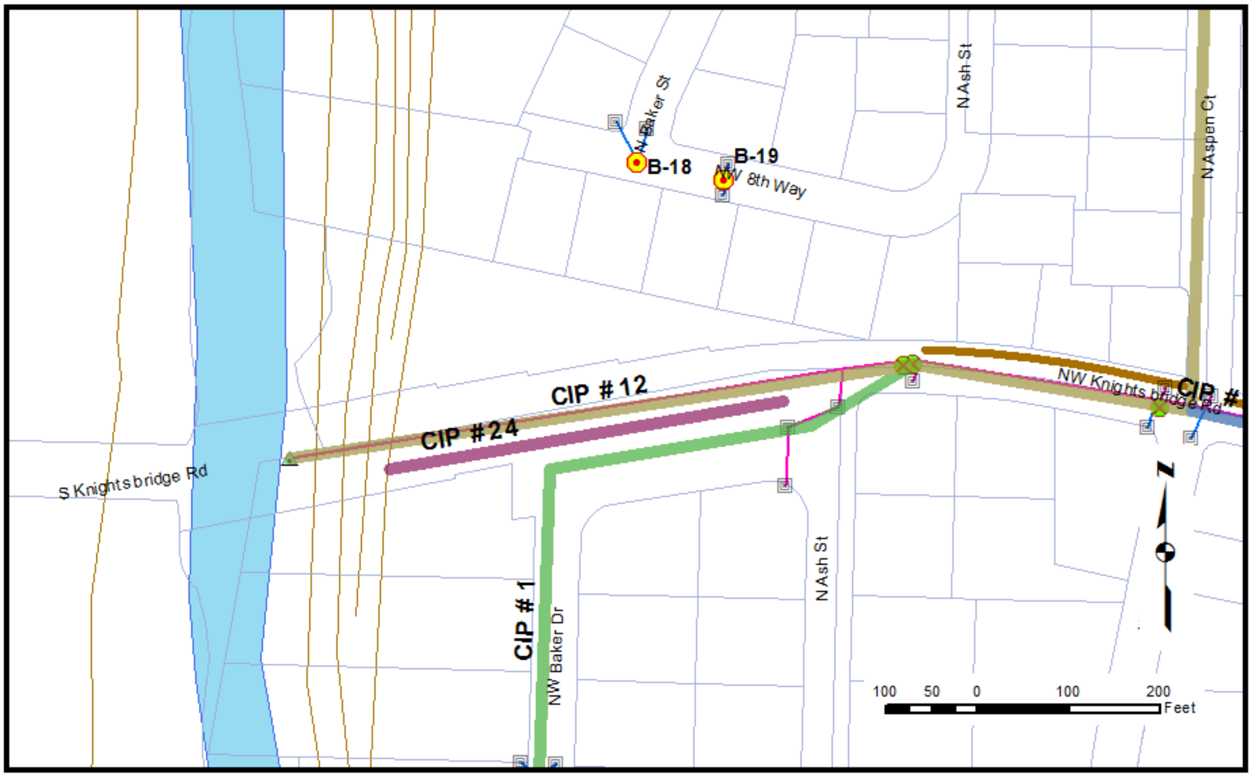
Schedule: 10 to 20 years.

6.5.24 Knight's Bridge Runoff Treatment #24

Currently, runoff from the Knight's Bridge basin flows through a Contech Model CDS2015-4 hydrodynamic separator treatment system and then freely discharges to the Molalla River from a pipe attached to the underside of Knight's Bridge.

Recommended Improvements

A stormwater treatment flow through swale is proposed on the City-owned property on the south side of Knight's Bridge Rd, west of Ash St. This stormwater treatment swale is proposed in anticipation of future DEQ stormwater discharge requirements. The estimated cost for the treatment swale is $50,000.



CIP #24: Knight's Bridge Runoff Treatment

Description: A flow through stormwater treatment swale to treat stormwater runoff from the Knight's Bridge basin prior to discharge to the Molalla River.

Location: City owned property on the South side of N Knight's Bridge Rd, West of Ash St.

Existing System: The basin currently utilizes a Contech Model CDS2015-4 hydrodynamic separator treatment system and then outfalls to the Molalla River via a pipe beneath Knight's Bridge.

Proposed System: A flow-through stormwater treatment swale designed to treat for anticipated DEQ stormwater discharge permit requirements.

Cost: $50,000

Priority: Low

Schedule: 10 to 20 years.

6.6 Other Considerations

While compiling data and analyzing the existing stormwater system during the development of this Stormwater Master Plan, other projects, in addition to capital improvement projects, were identified. These other considerations are detailed below.

6.6.1 Comprehensive Survey of Existing System

When collecting and analyzing available existing data for this SWMP, it became apparent that there are numerous areas throughout town where the available data is incomplete, or where multiple sources of information do not agree. It is recommended that the City implement a plan to collect a comprehensive survey of the existing stormwater infrastructure system. Comprehensive survey data will allow for updated and accurate mapping and inventorying of the existing stormwater infrastructure, which will help identify future CIPs and aid in locating potential problem areas. It is recommended that the City plan to add the cost of surveying to the annual stormwater budget. A reasonable schedule would be to collect survey data for one basin per year. The anticipated cost for the collection of survey data is $10,000/year. This cost includes the mobilization of one survey crew for five days, for the purposes of collecting stormwater infrastructure data (Manhole locations, rim elevations, Pipe diameters, pipe invert elevations, catch basins, pipe lengths, UIC locations, UIC RIM elevations, and UIC depth), as well as submitting this data electronically in a format that can be used by the City to update their GIS system mapping.

6.6.2 Operation and Maintenance (O&M) Manual

Proper operation and maintenance of infrastructure and devices of a system can prolong its life, improve its performance, and minimize future capital costs. It is recommended that the City create a formal O&M manual for its stormwater infrastructure. The O&M should include an inventory of the existing system, a cleaning schedule for all infrastructures, and a replacement schedule. The anticipated cost for the development of an O&M manual is $30,000. The O&M manual should be updated and be revised as survey data for the existing system is collected.

6.6.3 System Flow Monitoring

During modeling of storm events and stormwater infrastructure conveyance for the SWMP, general assumptions were made. These assumptions, including basin sizes, pipe conditions, and stormwater flow paths, when coupled together, can have a profound effect on the model output. Because little to no historical stormwater flow data was available, it was not possible to calibrate the model and these assumptions, to ensure that the model is providing accurate flow estimates. It is recommended that the City implement a stormwater infrastructure flow monitoring program, which will allow for calibration of the stormwater runoff modeling, and ensure that the CIP projects recommended within this report are accurately sized. The anticipated cost for this ongoing system flow monitoring program is $10,000/year.

6.6.4 Coordination with Clackamas County and Oregon Department of Transportation

During data collection for the SWMP, it was determined that very little data pertaining to the stormwater infrastructure located beneath the County and ODOT roads was available. Furthermore, these roadways often were problem areas susceptible to flooding. It is recommended that the City make an effort to coordinate data collection as well as roadway and stormwater infrastructure improvements in these areas with the County and ODOT.

Section 7: Stormwater Master Plan

7.1 Introduction

The Stormwater Master Plan for the City of Canby is comprised of the following elements:

1. Flow directions to be implemented in conjunction with new development
2. Direction in the form of the type of systems to be implemented
3. Direction in the magnitude and timing of capital improvements to the storm water system
4. Guidance for the City to provide to private entities in order to construct elements of the stormwater system consistent with City standards and this Storm Water Master Plan.

7.2 Capital Improvement Program

The Capital Improvement Program is summarized and prioritized in Table 7.1.

Table 7.1 CIP Summary Sheet

Capital Improvement Projects Summary Sheet (cont.)				Priority		
#	**Project Name**	**Total Project Cost 2013**	**Clackamas County Share**	**High 0-5 years**	**Medium 6-10 years**	**Low 11-20 years**
1	N Baker Dr.	$180,000		**X**		
2	NW 10th Ave. from N Locust St. to N Pine St.	$1,330,000	$801,000	**X**		
3	SE Hazeldell Way	$90,000		**X**		
4	SW 13th Ave near Canby High School	$30,000		**X**		
5	UIC E-8 and E-11 Decommission	$50,000		**X**		
6	NW 2nd Ave and N Ivy St. UIC Decommission	$40,000		**X**		
7	Cinema Parking Lot and NW 1st and Ivy St UIC Decommission	$40,000		**X**		
8	S Ivy St	$730,000	$730,000		**X**	
9	N Maple St. at Maple St. Park	$30,000			**X**	
10	N Maple St. and NW 34th Pl.	$30,000			**X**	
11	NW 13th Ave from N Ash St. to N Birch St.	$30,000			**X**	
12	NW 9th Ave from N Ash St. to N Cedar St.	$420,000			**X**	
13	N Knights Bridge Rd	$130,000			**X**	

Capital Improvement Projects Summary Sheet (cont.)				Priority		
#	**Project Name**	**Total Project Cost 2013**	**Clackamas County Share**	**High 0-5 years**	**Medium 6-10 years**	**Low 11-20 years**
14	NW 2nd Ave from N Cedar St. to NW Baker Dr..	$690,000			**X**	
15	NW 3rd Ave from N Cedar St. to N Holly St.	$670,000			**X**	
16	N Holly St.	$310,000			**X**	
17	N Juniper St. and NE 5th Ave	$30,000				**X**
18	N Alder St. and N Baker St.	$30,000				**X**
19	N Cedar St.	$10,000				**X**
20	S Pine St. and NSE 2nd Ave	$30,000				**X**
20	Police Station/NW 3rd Ave Pond	$30,000				**X**
21	Fish Eddy Wetland Flow Monitoring	$30,000				**X**
22	Fish Eddy Wetland	$670,000				**X**
23	Knight's Bridge Runoff Treatment	$50,000				**X**
	TOTAL	**$5,680,000**	**$1,531,000**	**7 Projects**	**9 Projects**	**8 Projects**

Additional Costs for Budgeting Purposes

25	Comprehensive Survey of Existing System	$10,000/yr		**X**
26	Operation and Maintenance (O&M) Manual	$30,000		**X**
27	System Flow Monitoring	$10,000/yr		**X**

Section 8: References

City of Canby Comprehensive Plan, City of Canby Planning Department, originally published 1984, updated January 2007

City of Canby Public Works Design Standards, City of Canby, June 2012

City of Canby Willamette Wayside Properties Master Plan, University of Oregon. Dept. of Planning, Public Policy and Management, Community Planning Workshop, August 2004

METRO 2009. Executive Summary, 20 and 50 year Regional Population and Employment Range Forecasts, METRO Research Center, March 2009

SDMP 1994. Curran-McLeod, Inc. Consulting Engineers, City of Canby Storm Drainage Master Plan, December 1994.

SDMP 1994a. Orr, Elizabeth L., William N. Orr and Ewart M. Baldwin, Geology of Oregon, Kendall/Hunt Publishing Co., Dubuque, Iowa, 1992

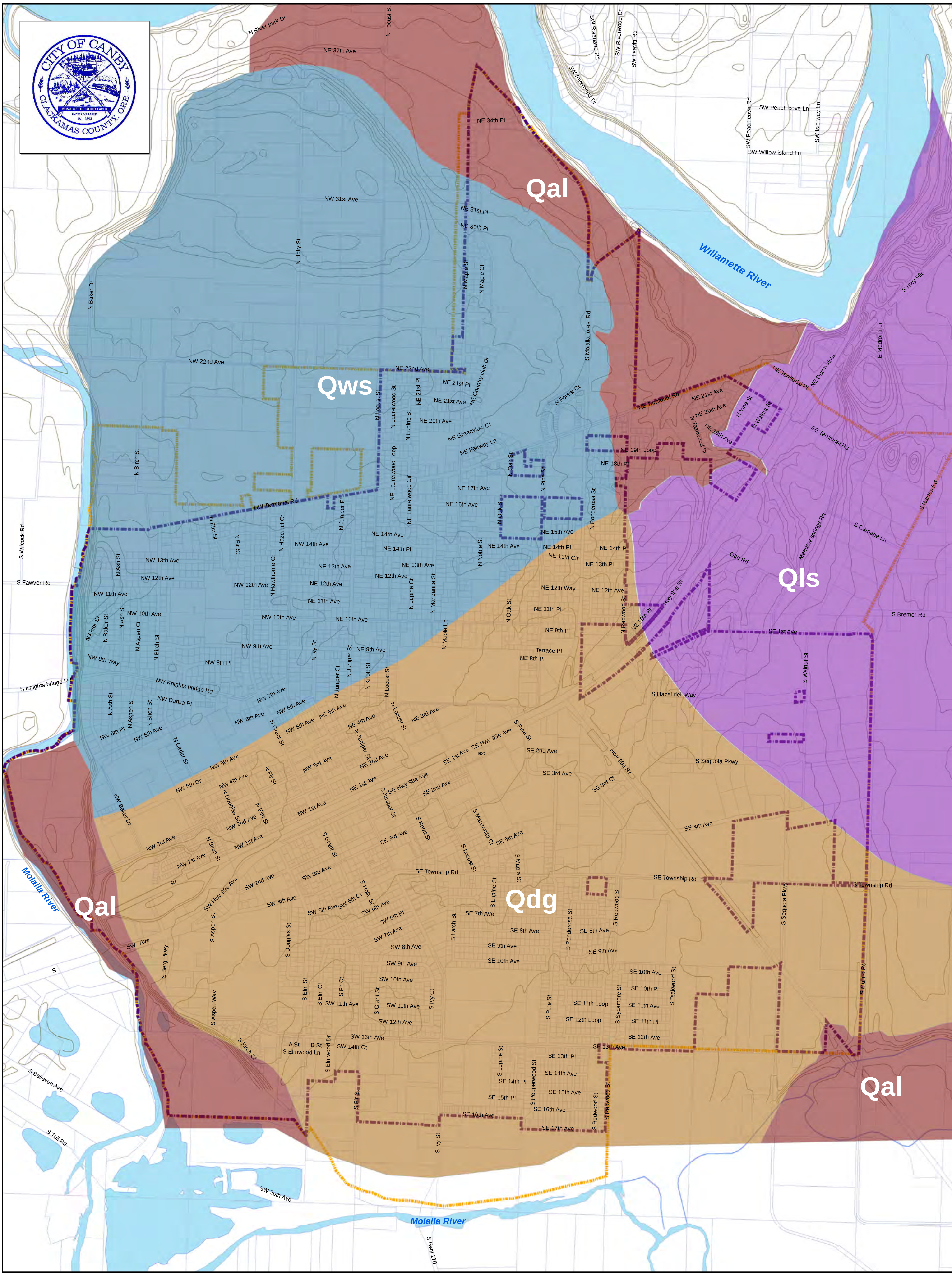
SDMP 1994b. Oregon Department of Geology and Mineral Industries, Geologic Hazards Canby and Oregon City Quadrangle Bulletin Number 99,179, 1979.

SDMP 1994c. US Department of Agriculture, Soil Conservation Service, Soil Survey of Clackamas County, OR, 1982

SDMP 1994d. US Weather Service, North Willamette Experimental Station

United States Census Bureau, 2010 Census, http://www.census.gov/ accessed February 2013

Figures



Legend

Surficial Geologic Units

Classification

- Qws
- Qls
- Qdg
- Qal

- City Limits
- Urban Growth Boundary
- Taxlots
- 10 Foot Contours - Metro/USGS

The data used to prepare this map is based on limited record and design drawings and field mapping. This information is the best currently available, however there are known innacuracies and deficiencies. The City will continue to update and refine the mapping of the stormwater system in an adapative management process.

0 250 500 1,000 1,500 2,000 Feet

Kennedy/Jenks Consultants

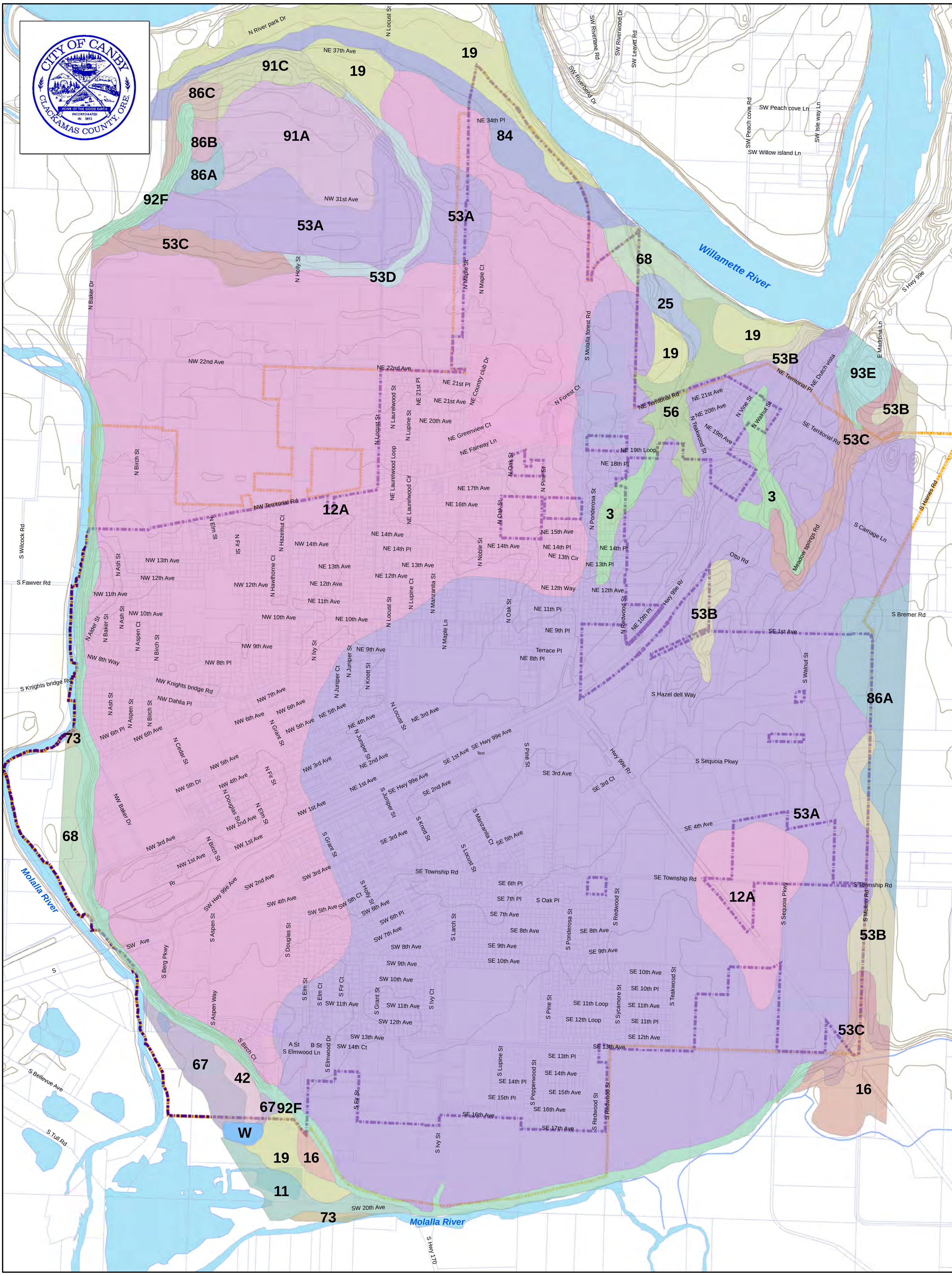
CITY OF CANBY
STORMWATER MASTER PLAN

SURFICIAL GEOLOGIC UNITS

FIGURE 1

17 September 2013

K/J Project Number 01291029.00



Kennedy/Jenks Consultants

CITY OF CANBY
STORMWATER MASTER PLAN

SOIL CONSERVATION SERVICE (SCS)
SOIL CLASSIFICATION

FIGURE 2

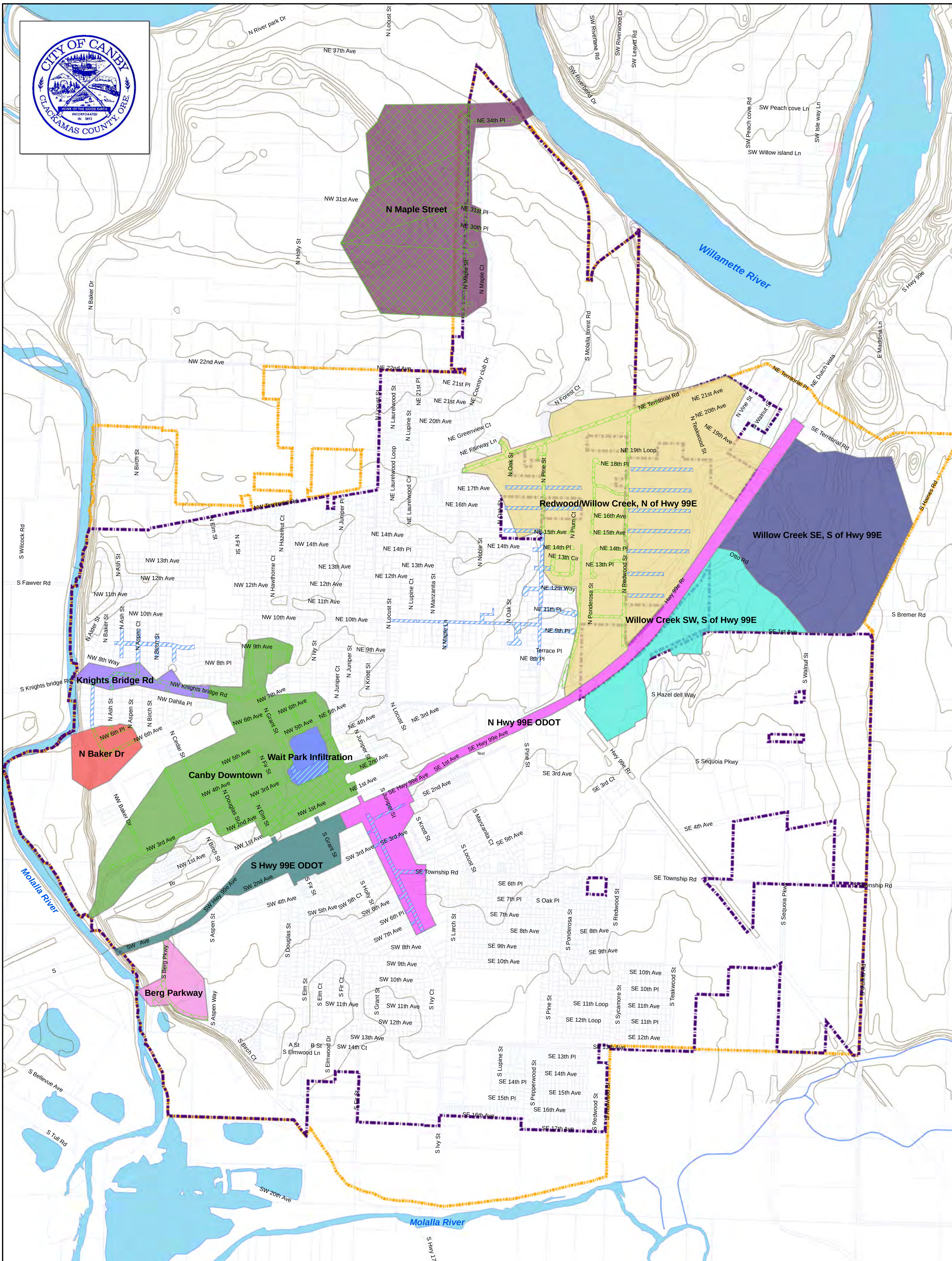
17 September 2013
K/J Project Number 01291029.00

The data used to prepare this map is based on limited record and design drawings and field mapping. This information is the best currently available, however there are known innacuracies and deficiencies. The City will continue to update and refine the mapping of the stormwater system in an adapative management process.

0 250 500 1,000 1,500 2,000 Feet

Legend

- 10 Foot Contours - Metro/USGS
- City Limits
- Urban Growth Boundary
- Taxlots



Legend

Subbasin

- Canby Downtown
- Wait Park Infiltration
- N Baker Dr
- Knights Bridge Rd
- Berg Parkway
- N Maple Street
- Redwood/Willow Creek, N of Hwy 99E
- Willow Creek SE, S of Hwy 99E
- Willow Creek SW, S of Hwy 99E
- N Hwy 99E ODOT
- S Hwy 99E ODOT

- ModelSubbasins
- FutureSubbasins
- City Limits
- Urban Growth Boundary
- Taxlots
- 10 Foot Contours - Metro/USGS

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0 250 500 1,000 1,500 2,000 Feet

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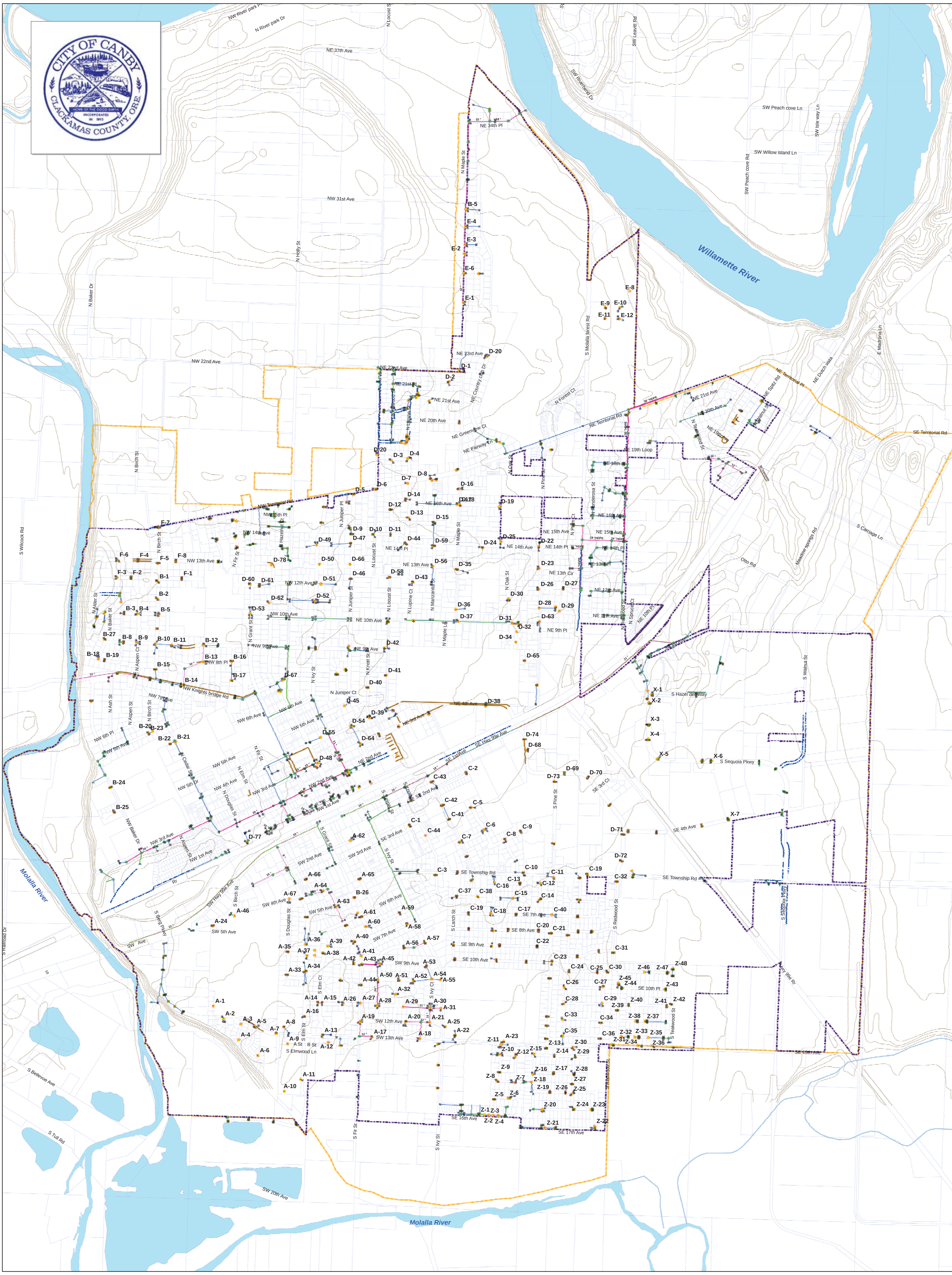
CITY OF CANBY
STORMWATER MASTER PLAN

MAJOR BASINS AND SUBBASINS

FIGURE 3

17 September 2013
K/J Project Number 01291029.00



Legend

- Catch Basin
- Storm Manhole
- Sediment Manhole
- Drywell/UIC
- Cleanout
- Outfall
- Trench Drain Lines
- Swale
- Pervious Pavement

Pipe - Unverified

- Assumed Location

Pipe - Data Diameter, in.

- Unknown Diameter
- 4
- 6
- 8
- 10
- 12
- 15
- 18
- 20
- 21
- 24
- 27
- 30
- 36

- City Limits
- Urban Growth Boundary
- Taxlots
- 10 Foot Contours - Metro/USGS

The data used to prepare this map is based on limited record and design drawings and field mapping. This information is the best currently available, however there are known innaccuracies and deficiencies. The City will continue to update and refine the mapping of the stormwater system in an adapative management process.

N

0 250 500 1,000 1,500 2,000 Feet

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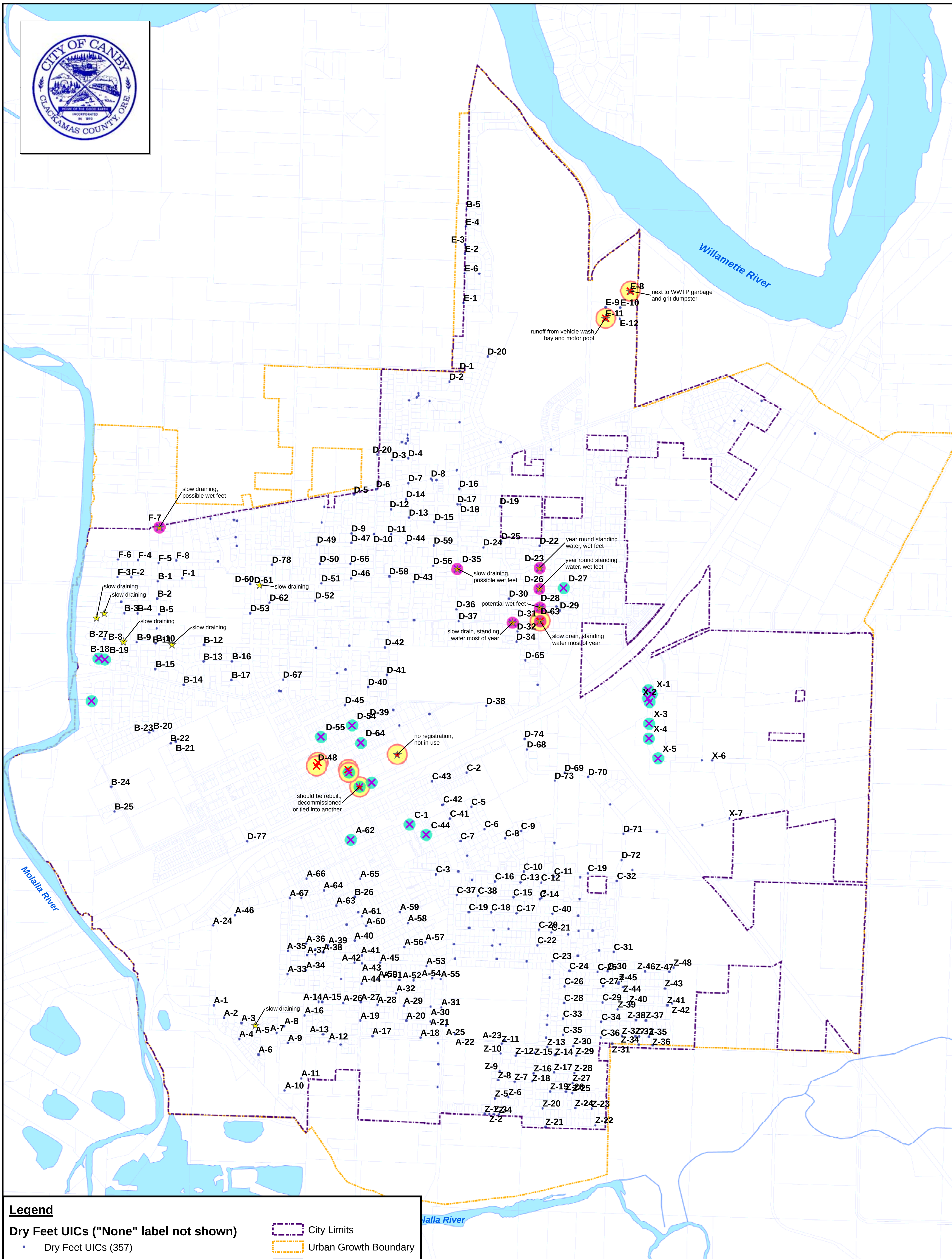
CITY OF CANBY
STORMWATER MASTER PLAN

EXISTING STORMWATER SYSTEM

FIGURE 4

1 October 2013

K/J Project Number 01291029.00



Legend

Dry Feet UICs ("None" label not shown)
- Dry Feet UICs (357)

Wet Feet UICs
- City - Dry Feet / GW Model - Wet Feet (20)
- City & GW Model - Wet Feet (7)

City Identified Need to Decommission
- yes
- probably
- maybe

Groundwater Protectiveness Demonstration Risk Priority
- High (Must be decommissioned) (6)
- Moderate (26)

UICs To Decommission
- UICs To Decommission (9)

- City Limits
- Urban Growth Boundary
- Taxlots

The data used to prepare this map is based on limited record and design drawings and field mapping. This information is the best currently available, however there are known innacuracies and deficienicies. The City will continue to update and refine the mapping of the stormwater system in an adapative management process.

0 250 500 1,000 1,500 2,000 Feet

N

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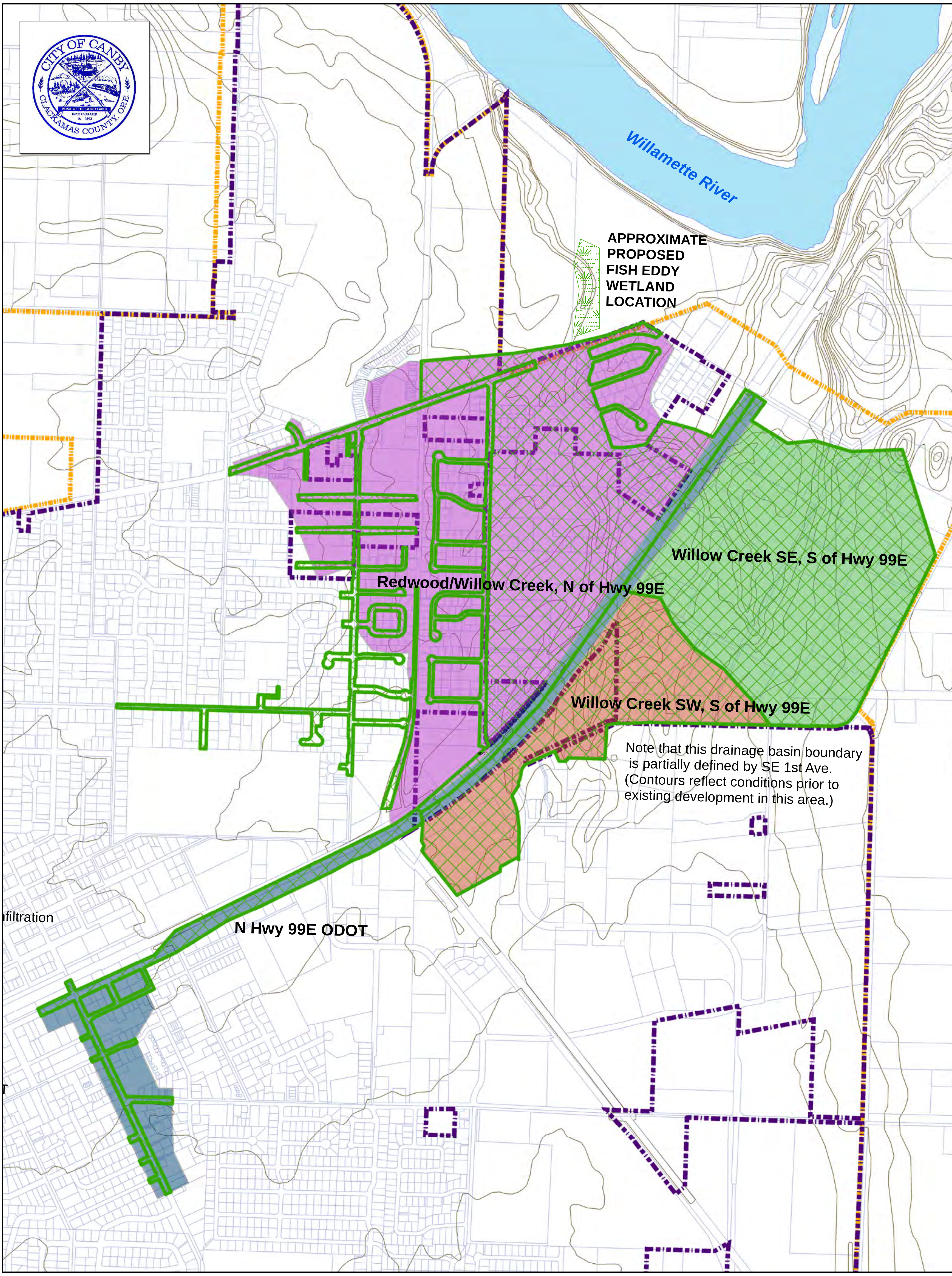
CITY OF CANBY
STORMWATER MASTER PLAN

UIC DECOMMISSIONING/RISK PRIORITY

FIGURE 5

26 September 2013
K/J Project Number 01291029.00



Legend

- FishEddyBasins
- Redwood/Willow Creek, N of Hwy 99E
- Willow Creek SE, S of Hwy 99E
- Willow Creek SW, S of Hwy 99E
- N Hwy 99E ODOT
- City Limits
- Urban Growth Boundary
- Taxlots
- 10 Foot Contours - Metro/USGS

The data used to prepare this map is based on limited record and design drawings and field mapping. This information is the best currently available, however there are known innaccuracies and deficienicies. The City will continue to update and refine the mapping of the stormwater system in an adapative management process.

0 250 500 1,000 1,500 2,000 Feet

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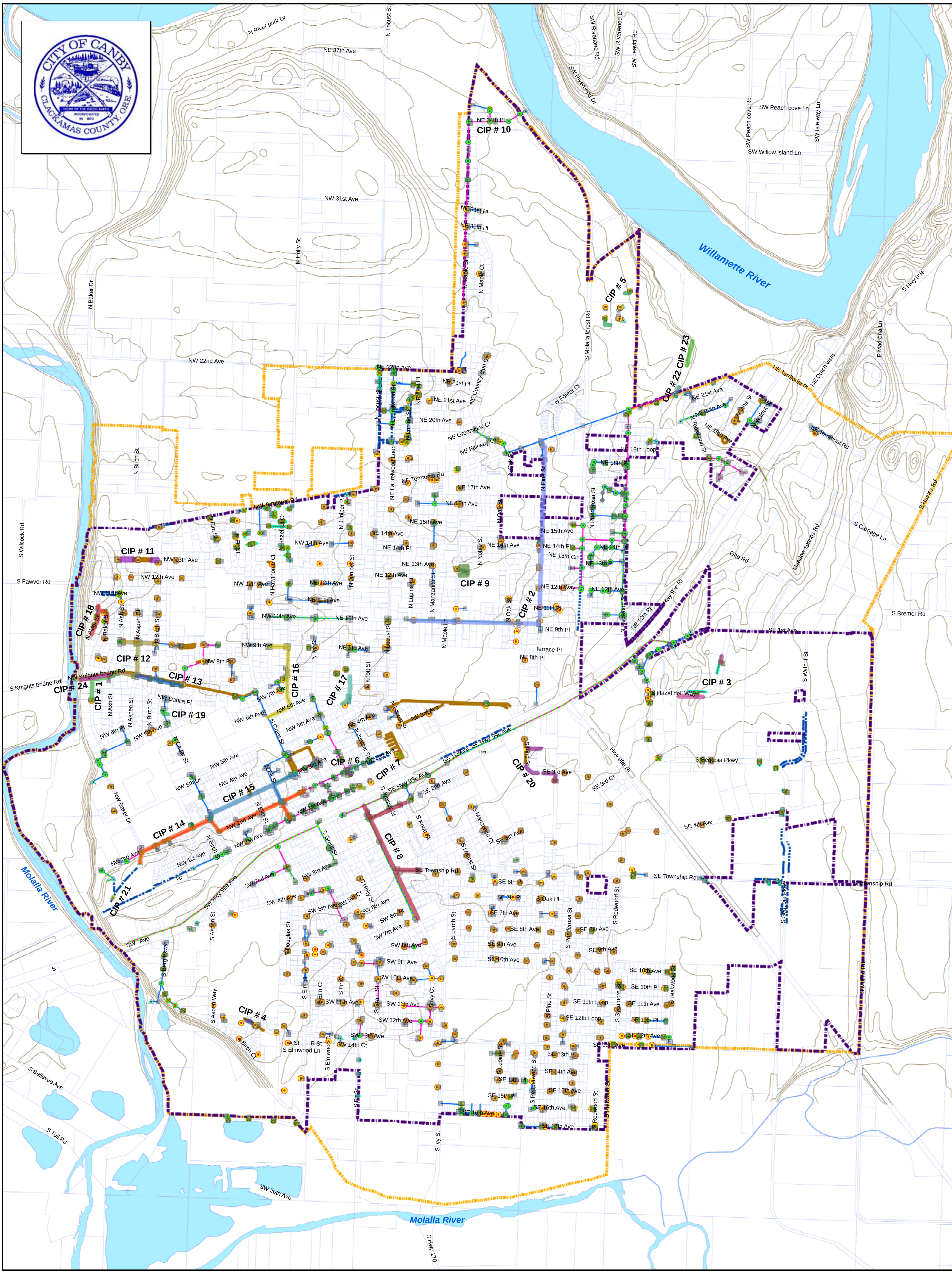
CITY OF CANBY
STORMWATER MASTER PLAN

PROPOSED FISH EDDY WETLAND MODEL BASINS

FIGURE 6

17 September 2013

K/J Project Number 01291029.00



Legend

- Catch Basin
- Storm Manhole
- Sediment Manhole
- Drywell/UIC
- Cleanout
- Outfall
- Trench Drain Lines
- Swale
- Pervious Pavement

Pipe - Unverified

- Assumed Location

Pipe - Data

- Diameter Labeled on Map
- Unknown Diameter

- City Limits
- Urban Growth Boundary
- Taxlots
- 10 Foot Contours - Metro/USGS

The data used to prepare this map is based on limited record and design drawings and field mapping. This information is the best currently available, however there are known innaccuracies and deficienicies. The City will continue to update and refine the mapping of the stormwater system in an adapative management process.

N

0 250 500 1,000 1,500 2,000 Feet

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CITY OF CANBY
STORMWATER MASTER PLAN

CAPITAL IMPROVEMENT PROJECTS APPROXIMATE LOCATIONS

FIGURE 7

26 September 2013
K/J Project Number 01291029.00

Appendix A

Hydrologic/Hydraulic Modeling Input - Output/Results

Provided on CD

Appendix B

CIP Cost Estimates

City of Canby

Stormwater CIP #1

N Baker Dr.

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115	800	$92,000
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$92,000
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000	8	$8,000
Manholes		EA	$3,850	3	$12,000
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastructure					$20,000

Grand SubTotal				**$112,000**
Contractor O&P	10%			$11,200
			Subtotal	**$123,200**
Contingency	20%			$24,640
Engineering and Admin	25%			$30,800
Estimated Project Cost				**$180,000**

City of Canby

Stormwater CIP #2

NW 10th Ave. from N Locust St. to N Pine St.

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115	1,650	$190,000
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136	370	$50,000
24-inch diameter	24	LF	$167	2,900	$484,000
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$724,000
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850	14	$54,000
UIC Decommission		EA	$10,000	5	$50,000
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastructure					$104,000

Grand SubTotal			**$828,000**
Contractor O&P	10%		$82,800
		Subtotal	**$910,800**
Contingency	20%		$182,160
Engineering and Admin	25%		$227,700
Estimated Project Cost			**$1,330,000**

City of Canby

Stormwater CIP #3

SE Hazeldell Way

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$0
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26	2,000	$52,000
Subtotal, Infiltration					$52,000
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastrucutre					$0

Grand SubTotal			**$52,000**
Contractor O&P	10%		$5,200
		Subtotal	**$57,200**
Contingency	20%		$11,440
Engineering and Admin	25%		$14,300
Estimated Project Cost			**$90,000**

City of Canby

Stormwater CIP #4

SW 13th Ave Near Canby High School

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$0
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26	500	$13,000
Subtotal, Infiltration					$13,000
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$0		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastructure					$0

Grand SubTotal			**$13,000**
Contractor O&P	10%		$1,300
		Subtotal	**$14,300**
Contingency	20%		$2,860
Engineering and Admin	25%		$3,575
Estimated Project Cost			**$30,000**

City of Canby
Stormwater CIP #5
UIC E-8 and E-11 Decommission

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$0
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26	300	$7,800
Subtotal, Infiltration					$7,800
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000	2	$20,000
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastrucutre					$20,000

Grand SubTotal			**$27,800**
Contractor O&P	10%		$2,780
		Subtotal	**$30,580**
Contingency	20%		$6,116
Engineering and Admin	25%		$7,645
Estimated Project Cost			**$50,000**

City of Canby

Stormwater CIP #6

NW 2nd Ave and N Ivy St. UIC Decommission

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$0
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000	2	$20,000
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastrucutre					$20,000

Grand SubTotal			**$20,000**
Contractor O&P	10%		$2,000
		Subtotal	**$22,000**
Contingency	20%		$4,400
Engineering and Admin	25%		$5,500
Estimated Project Cost			**$40,000**

City of Canby

Stormwater CIP #7

Cinema Parking Lot Decommission

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$0
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000	2	$20,000
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastrucutre					$20,000

Grand SubTotal			**$20,000**
Contractor O&P	10%		$2,000
		Subtotal	**$22,000**
Contingency	20%		$4,400
Engineering and Admin	25%		$5,500
Estimated Project Cost			**$40,000**

City of Canby
Stormwater CIP #8
S Ivy St

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136	3,000	$407,000
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$407,000
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000	16	$16,000
Manholes		EA	$3,850	8	$31,000
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastrucutre					$47,000

Grand SubTotal			**$454,000**
Contractor O&P	10%		$45,400
		Subtotal	**$499,400**
Contingency	20%		$99,880
Engineering and Admin	25%		$124,850
Estimated Project Cost			**$730,000**

City of Canby
Stormwater CIP #19
Maple St Park

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$0
Infiltration					
Pervious Pavement		SF	$3	4,800	$14,400
Swales		SF	$26		$0
Subtotal, Infiltration					$14,400
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastrucutre					$0

Grand SubTotal			**$14,400**
Contractor O&P	10%		$1,440
		Subtotal	**$15,840**
Contingency	20%		$3,168
Engineering and Admin	25%		$3,960
Estimated Project Cost			**$30,000**

City of Canby

Stormwater CIP #10

N Maple St. and NW 34th Pl.

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136	20	$3,000
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$3,000
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850	1	$4,000
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000	1	$10,000
Subtotal, Infrastrucutre					$14,000

Grand SubTotal			**$17,000**
Contractor O&P	10%		$1,700
		Subtotal	**$18,700**
Contingency	20%		$3,740
Engineering and Admin	25%		$4,675
Estimated Project Cost			**$30,000**

City of Canby

Stormwater CIP #11

NW 13th Ave from N Ash St. to N Birch St.

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$0
Infiltration					
Pervious Pavement		SF	$3	4,800	$14,400
Swales		SF	$26		$0
Subtotal, Infiltration					$14,400
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastrucutre					$0

Grand SubTotal			**$14,400**
Contractor O&P	10%		$1,440
		Subtotal	**$15,840**
Contingency	20%		$3,168
Engineering and Admin	25%		$3,960
Estimated Project Cost			**$30,000**

City of Canby

Stormwater CIP #12

NW 9th Ave from N Ash St. to N Cedar St.

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115	500	$58,000
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136	1,500	$203,000
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$261,000
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastrucutre					$0

Grand SubTotal			**$261,000**
Contractor O&P	10%		$26,100
		Subtotal	**$287,100**
Contingency	20%		$57,420
Engineering and Admin	25%		$71,775
Estimated Project Cost			**$420,000**

City of Canby
Stormwater CIP #13
N Knights Bridge Road

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102	750	$77,000
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$77,000
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastrucutre					$0

Grand SubTotal			**$77,000**
Contractor O&P	10%		$7,700
		Subtotal	**$84,700**
Contingency	20%		$16,940
Engineering and Admin	25%		$21,175
Estimated Project Cost			**$130,000**

City of Canby

Stormwater CIP #14

NW 2nd Ave from N Grant St. to NW Baker Dr.

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233	400	$93,000
36-inch diameter	36	LF	$249	1,350	$337,000
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$430,000
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Storm Pipe					$0

Grand SubTotal			**$430,000**
Contractor O&P	10%		$43,000
		Subtotal	**$473,000**
Contingency	20%		$94,600
Engineering and Admin	25%		$118,250
Estimated Project Cost			**$690,000**

City of Canby

Stormwater CIP #15

NW 3rd Ave from N Cedar St. to N Holly St.

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136	140	$19,000
24-inch diameter	24	LF	$167	400	$67,000
30-inch diameter	30	LF	$233	1,200	$280,000
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$366,000
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000	2	$2,000
Manholes		EA	$3,850	8	$31,000
UIC Decommission		EA	$10,000	2	$20,000
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastrucutre					$53,000

Grand SubTotal			**$419,000**
Contractor O&P	10%		$41,900
		Subtotal	**$460,900**
Contingency	20%		$92,180
Engineering and Admin	25%		$115,225
Estimated Project Cost			**$670,000**

City of Canby
Stormwater CIP #16
N Holly St.

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115	1,650	$190,000
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$190,000

Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0

Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Storm Pipe					$0

Grand SubTotal			**$190,000**
Contractor O&P	10%		$19,000
		Subtotal	**$209,000**
Contingency	20%		$41,800
Engineering and Admin	25%		$52,250
Estimated Project Cost			**$310,000**

City of Canby
Stormwater CIP #17
N Juniper St. and NE 5th Ave

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$0
Infiltration					
Pervious Pavement		SF	$3	4,800	$14,400
Swales		SF	$26		$0
Subtotal, Infiltration					$14,400
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Storm Pipe					$0

Grand SubTotal			**$14,400**
Contractor O&P	10%		$1,440
		Subtotal	**$15,840**
Contingency	20%		$3,168
Engineering and Admin	25%		$3,960
Estimated Project Cost			**$30,000**

City of Canby

Stormwater CIP #18

N Baker St. and N Alder St.

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$0
Infiltration					
Pervious Pavement		SF	$3	4,800	$14,400
Swales		SF	$26		$0
Subtotal, Infiltration					$14,400
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Storm Pipe					$0

Grand SubTotal			**$14,400**
Contractor O&P	10%		$1,440
		Subtotal	**$15,840**
Contingency	20%		$3,168
Engineering and Admin	25%		$3,960
Estimated Project Cost			**$30,000**

City of Canby
Stormwater CIP #19
N Cedar St.

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115	20	$2,000
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$2,000
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850	1	$4,000
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Storm Pipe					$4,000

Grand SubTotal			**$6,000**
Contractor O&P	10%		$600
		Subtotal	**$6,600**
Contingency	20%		$1,320
Engineering and Admin	25%		$1,650
Estimated Project Cost			**$10,000**

City of Canby

Stormwater CIP #20

S Pine St. and SE 2nd Ave

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$0
Infiltration					
Pervious Pavement		SF	$3	4,800	$14,400
Swales		SF	$26		$0
Subtotal, Infiltration					$14,400
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$3,850		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastrucutre					$0

Grand SubTotal			**$14,400**
Contractor O&P	10%		$1,440
		Subtotal	**$15,840**
Contingency	20%		$3,168
Engineering and Admin	25%		$3,960
Estimated Project Cost			**$30,000**

City of Canby

Stormwater CIP #20

Police Station/NW 3rd Ave Pond Flow Monitoring

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79	20	$2,000
6-inch diameter	6	LF	$85		$0
8-inch diameter	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter	36	LF	$249		$0
42-inch diameter	42	LF	$273		$0
48-inch diameter	48	LF	$288		$0
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$2,000
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000	1	$1,000
Concrete Channel	1	EA	$10,000	1	$10,000
UIC Decommission		EA	$10,000		$0
Data Logger		EA	$2,000	1	$2,000
Subtotal, Storm Pipe					$13,000

Grand SubTotal			**$15,000**
Contractor O&P	10%		$1,500
		Subtotal	**$16,500**
Contingency	20%		$3,300
Engineering and Admin	25%		$4,125
Estimated Project Cost			**$30,000**

City of Canby

Stormwater CIP #21

Fish Eddy Wetland Flow Monitoring

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79	20	$2,000
6-inch diameter	6	LF	$85		$0
"Conventional" Treatmer	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter					
-inch diameter					
-inch diameter					36
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$2,000
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26		$0
Subtotal, Infiltration					$0
Infrastructure					
Catchbasins		EA	$1,000	1	$1,000
Concrete Channel	1	EA	$10,000	1	$10,000
UIC Decommission		EA	$10,000		$0
Data Logger		EA	$2,000	1	$2,000
Subtotal, Storm Pipe					$13,000

Grand SubTotal			**$15,000**
Contractor O&P	10%		$1,500
		Subtotal	**$16,500**
Contingency	20%		$3,300
Engineering and Admin	25%		$4,125
Estimated Project Cost			**$30,000**

City of Canby
Stormwater CIP #22
Fish Eddy Wetland

Drainage Area	393	Acres	Approx. 25% impervious	
WQ Flow Rate	6.7	cfs	3,007	gpm
Facility Volume (WQ Vol)	814,000	cu-ft		

"Conventional" Treatment Wetland

Area required = WQ Vol / 3-foot average depth:

271,333	sq-ft
6.23	ac
2.5	hectares surface area

$485,058	Total Present Cost Estimate Weiss et. al. 2007 (2005 dollars, US Zone 1 Rainfall -adjusted to 2013 dollars, US Zone 7 Rainfall)
$575,049	Cost Estimate Kadlec & Wallace Treatment Wetlands (1996 dollars - adjusted to 2013 dollars using ENR CCI)

Subtotal $530,054
(Average Estimated Cost for Construction Overhead and Contingency)

Grand SubTotal			**$530,054**
Contractor O&P	included in estimates		-
		Subtotal	$530,054
Contingency	included in estimates		-
Engineering and Admin	25%		$132,513
Estimated Project Cost			**$670,000**

City of Canby
Stormwater CIP #23
Knight's Bridge Runoff Treatment

Item	Diameter	Unit	Unit cost	Quantity	Total
Storm Pipes					
4-inch diameter	4	LF	$79		$0
6-inch diameter	6	LF	$85		$0
"Conventional" Treatmer	8	LF	$102		$0
10-inch diameter	10	LF	$108		$0
12-inch diameter	12	LF	$115		$0
15-inch diameter	15	LF	$123		$0
18-inch diameter	18	LF	$136		$0
24-inch diameter	24	LF	$167		$0
30-inch diameter	30	LF	$233		$0
36-inch diameter					
-inch diameter					
-inch diameter					36
54-inch diameter	54	LF	$343		$0
Subtotal, Storm Pipe					$0
Infiltration					
Pervious Pavement		SF	$3		$0
Swales		SF	$26	1,000	$26,000
Subtotal, Infiltration					$26,000
Infrastructure					
Catchbasins		EA	$1,000		$0
Manholes		EA	$0		$0
UIC Decommission		EA	$10,000		$0
In-Line Treatment		EA	$10,000		$0
Subtotal, Infrastructure					$0

Grand SubTotal			**$26,000**
Contractor O&P	10%		$2,600
		Subtotal	**$28,600**
Contingency	20%		$5,720
Engineering and Admin	25%		$7,150
Estimated Project Cost			**$50,000**

Appendix C

Groundwater Protectiveness Demonstrations and Risk Prioritization for Underground Injection Control (UIC) Devices

GSI
Water Solutions, Inc.

Technical Memorandum

REGISTERED OREGON MATTHEW V. KOHLBECKER G2248 GEOLOGIST

EXP. 4/14

To: Darvin Tramel – City of Canby

From: Matt Kohlbecker, RG – GSI Water Solutions
Ari Petrides, PhD – GSI Water Solutions

Cc: Gordon Munro, PE – Kennedy Jenks Consultants
Alan Flemming – Kennedy/Jenks Consultants

Date: August 29, 2013

Re: **Groundwater Protectiveness Demonstrations and Risk Prioritization for Underground Injection Control (UIC) Devices, City of Canby, Oregon**

This technical memorandum (TM) presents a Groundwater Protectiveness Demonstration (GWPD) for Underground Injection Control (UIC) devices in the City of Canby (City), Oregon (Figure 1). The GWPD was conducted to support the City's 2013 Stormwater Master Plan and UIC Water Pollution Control Facilities (WPCF) permit application.

1. Introduction

A UIC is device that infiltrates fluids into the subsurface. The City of Canby (City) owns 384 UIC devices that manage stormwater mainly from public rights-of-way (ROW) and adjacent properties in residential areas. The UICs are typically 4-foot diameter vertical structures that range from approximately 26 to 28 feet deep. The locations of the City's UICs are shown in Figure 2.

UICs are regulated by the Oregon Department of Environmental Quality (DEQ). Because the City's UICs infiltrate only stormwater from residential, commercial, and roadway areas, DEQ considers them to be Class V injection systems and regulates them under Oregon Administrative Rules (OAR) 340-044-0011(5)(d). The City applied for a UIC WPCF permit (the permit) for its UICs on December 30, 2008. In July 2012, DEQ issued a draft UIC WPCF permit template (the permit template) that will be used as the basis for developing the City's permit, which the City expects to receive in the fall of 2013.

The permit is designed to protect groundwater to its highest beneficial use. As such, the permit template stipulates that the City address UICs that are within 500 feet of a public drinking water or irrigation supply well, or inside the 2-year time of travel of a public water supply well.

55 SW Yamhill Street, Suite 300 Portland, OR 97204 P: 503.239.8799 F: 503.239.8940 info@gsiws.com www.gsiws.com

Options for addressing these UICs include developing a GWPD, retrofit the UIC, or decommission the UIC. A GWPD is an evaluation of whether beneficial use of groundwater is adversely impacted by stormwater pollutants as a result of infiltration. The City has chosen to develop GWPD models to identify which UICs are protective of groundwater, and to prioritize future UIC decommissioning and retrofitting based on the GWPD and other considerations (i.e., UIC functionality and other risk factors). This TM summarizes the GWPD models, which simulate attenuation of stormwater pollutants in the subsurface (i.e., after infiltration from a UIC). Two GWPDs were conducted:

- **Unsaturated Zone GWPD**. Unsaturated zone GWPDs are based on modeling pollutant fate and transport *vertically* through the *unsaturated* soils beneath a UIC. The objective of the unsaturated zone GWPD is to calculate the vertical distance required for pollutants to attenuate to background levels (which is considered to be the method reporting limit [MRL]), called the vertical protective separation distance. If the vertical separation distance at a UIC is greater than the protective separation distance, then the UIC is demonstrated to be protective and does not need to be retrofit or decommissioned. If the vertical separation distance at a UIC is less than the protective separation distance, then groundwater protectiveness must be demonstrated using a different method, the UIC must be retrofit, or the UIC must be decommissioned.

- **Saturated Zone GWPD.** A saturated zone GWPD consists of modeling *horizontal* pollutant fate and transport through *saturated* soils. The model is used to demonstrate that that the UIC does not adversely impact groundwater users by delineating a waste management area (WMA) around the UIC. A WMA is the "area where waste or material that could become waste if released to the environment, is located or has been located" [OAR 340-040-0010(19)]. In the context of stormwater infiltration from a UIC, the WMA is the location where groundwater contains stormwater pollutants above background levels. The objective of the saturated zone GWPD is to calculate the horizontal distance required for pollutant concentrations to decline to zero. This horizontal distance replaces the default horizontal separation distance in the permit template (i.e., 500 feet or 2-year time of travel).

GWPDs have been conducted by several municipalities in Oregon, including the Cities of Gresham, Portland, Bend, Redmond, Eugene, and Milwaukie; Clackamas County Water Environment Services; and Lane County. Results of the GWPD models apply to stormwater with pollutant concentrations typical of stormwater runoff from urban ROWs, and do not apply to releases of pollutants to the environment (i.e., spills). The model results will be considered along with other relevant to groundwater protectiveness factors, permit requirements, and the City's goals and policies to develop a strategy for addressing the City of Canby's UICs.

1.1 Objectives

The objectives of this TM are:

- Locate water wells in the City, and the number of UICs that are within the default setbacks to water wells that are specified within the permit template (500 feet of a water well or the 2-year time of travel).
- Determine the depth to seasonal high groundwater in the City, and UICs that intersect the seasonal high water table.

- Present technical documentation for the unsaturated zone and saturated zone GWPD models, and identify the protective vertical and horizontal separation distances for the City's UICs.
- Identify whether each UIC is protective of groundwater, based on the protective separation distances calculated by the GWPD models and the City's internal risk management goals.

The main text of this TM provides an overview of the UIC system and GWPD models. Additional technical details are provided in Attachment A (technical documentation for determining depth to seasonal high groundwater and water well locations), Attachment B (technical documentation for the unsaturated zone GWPD model), and Attachment C (technical documentation for the saturated zone GWPD model).

2. Geology and Hydrogeology

Input parameters for the GWPD models are based on the physical characteristics of the soils in Canby. This section summarizes the geologic and hydrogeologic characteristics of the soils with the objective of informing model input parameters.

2.1 Geology

The City's UICs are located in the coarse-grained facies of the catastrophic flood deposits (unit Qfc), shown on the geologic map in Figure 2. Locally, the Qfc is identified as the Canby fan by Piper (1942). The Canby fan is an alluvial fan originating from an erosional gap near the City of Oregon City. During the catastrophic floods that occurred approximately 13,000 to 15,000 years ago, a flow restriction downstream of the current location of the City of Portland caused floodwaters to backflow south into the Willamette Valley and spill southward into Canby and Wilsonville (O'Conner et al., 2001).

Locally, the Qfc consists of an up to 120 feet thick bouldery, sandy gravel that is capped with several feet of sand and silt (O'Conner et al., 2001). Shallow (i.e., approximately 20 feet deep) borings advanced as a part of geotechnical investigations in Canby indicate that the shallow unsaturated zone ranges from a coarse gravel with trace silt and sand to a silty, sandy gravel (Northwest Geotech, 2006a, 2006b, 2006c; Geotech Solutions, 2004; GeoDesign, 2007). Interbedded silty, sandy gravel lenses are noted within the coarse gravels in some of the shallow logs (GeoDesign, 2007). Well driller logs for monitoring wells and geotechnical holes indicate that below 20 feet, the unsaturated zone is primarily sand and gravel with layers of "gravelly silts" (CLAC 57878) and "clay layers" (CLAC 1529) (note that the "clays" on well driller logs are most likely silts because there are few true clays in the Portland basin, and silts are easily mistaken for clays).

2.2 Hydrogeology

A map showing groundwater elevation in the Qfc unit is provided in Figure 3. Groundwater flows toward the Willamette and Molalla Rivers, and away from topographic highs in the east and north areas of town. A map of depth to seasonal high groundwater, which is used for evaluating whether a UIC has sufficient protective vertical separation distance, is provided in Figure 4.

Technical documentation for development of the water level maps is provided in Attachment A. Hydrogeologic properties of unsaturated zone and saturated zone soils are summarized in Table 1 (unsaturated zone) and Table 4 (saturated zone), and are discussed in detail in Attachment B (unsaturated zone soils) and Attachment C (saturated zone soils).

3. Water Well Locations and Setbacks Between UICs and Water Wells

Water wells in the City were located based on the Oregon Water Resources Department (OWRD) online water rights database and the OWRD online well log query. A UIC and water well location map is provided in Figure 5. Technical documentation of the methods used to located water wells is provided in Attachment A.

Based on the permit template, UICs within 500 feet of a water well or the 2-year time of travel must be addressed with a GWPD, be retrofit, or be decommissioned. The 2-year time-of-travel zone (DEQ, 2012) or 500 foot buffer for each water well is shown in Figure 5 to indicate the default setback conditions in the permit template. A total of 189 UICs (shown in green in Figure 5) is within the default setbacks between UICs and water wells, and need to be addressed.

4. Groundwater Protectiveness Demonstrations

This section provides an overview of the unsaturated zone (Section 4.1) and saturated zone (Section 4.2) GWPD models. Detailed technical documentation for input parameters, the governing equations, and conservative assumptions for the GWPD are provided in Attachment B (unsaturated zone GWPD) and Attachment C (saturated zone GWPD).

Both models simulate pollutant fate and transport over time based on user-provided input parameters. During transport in the subsurface, pollutant concentrations are reduced by microbial action (biodegradation), dispersion, and sorption on aquifer solids. The objective of the modeling was to calculate the vertical and horizontal transport distances necessary to attenuate pollutants to below zero (i.e., MRL). Pollutant fate and transport are simulated for organic pollutants pentachlorophenol (PCP); di(2-ethylhexyl)phthalate (DEHP); benzo(a)pyrene; and the metal lead. These pollutants are among the most mobile, toxic, and environmentally persistent in their respective chemical classes (GSI, 2008), and are the most likely pollutants in their respective chemical classes to exceed regulatory standards for stormwater at UICs (Kennedy/Jenks, 2009).

4.1 Unsaturated Zone GWPD

The unsaturated zone GWPD model simulates pollutant fate and transport in soils below the bottom of the UIC and above the seasonal high groundwater table. The model is based on the 1-dimensional (1-D) advection dispersion equation, and is implemented in a Microsoft Excel spreadsheet. Model input parameters are summarized in Table 1 (soil properties) and Table 2 (pollutant properties). The input parameters for the unsaturated zone GWPD are varied to evaluate two scenarios for pollutant fate and transport: (1) the average scenario, which is represented by the central tendency or expected mean value of the input parameter, and (2) the reasonable maximum scenario, which is an upper bound on what could occur, but is considered unlikely to occur because of compounding conservatism.

Table 3 presents the minimum protective vertical separation distances under the average and reasonable maximum scenarios of the unsaturated zone GWPD model. The average scenario represents most reasonably likely conditions, and is used for regulatory compliance. Pollutant selected for modeling included those that are consistently present in stormwater and represent a cross section of chemical types. PCP migrates farther than the other pollutants that were modeled because it is more mobile in the environment. Therefore, the protective vertical separation distance at City UICs is conservatively based on PCP. Under the average scenario, the minimum protective vertical separation distance is 1.4 feet. However, GSI recommends adding 1.1 feet to the model-calculated vertical separation distance to account for natural variation of seasonal groundwater high elevations over time[1]. Therefore, GSI recommends using a protective separation distance of 2.5 feet for the minimum separation distance at vertical UICs.

The reasonable maximum scenario represents the worst-case conditions, and is characterized by compounding conservatism of input variables. The purpose of the reasonable maximum scenario is to evaluate model sensitivity, and it is not used for regulatory compliance. As is shown in Table 3, the protective separation distances under the worst-case "reasonable maximum scenario" are larger than the protective separation distances under the most likely "average scenario."

4.2 Saturated Zone GWPD

The saturated zone GWPD simulates pollutant fate and transport in saturated soils below the water table. The conceptual model for the saturated zone GWPD assumes that the UIC intersects the seasonal high groundwater table such that the UIC extends 5 feet below the water table. The saturated zone GWPD model is based on a conservative, 3-D numerical groundwater model (MODFLOW) that is coupled with a pollutant fate and transport model (MT3D) to simulate pollutant attenuation by dilution, dispersion, biodegradation, and retardation. Model input parameters are summarized in Table 4 (soil properties) and Table 5 (pollutant properties).

Table 6 presents the protective horizontal separation distances based on the saturated zone GWPD model. PCP migrates farther than the other pollutants that were modeled because it is more mobile and persistent in the environment. Therefore, the protective horizontal separation distance at City UICs is conservatively based on PCP. The protective horizontal separation distance is 267 feet.

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[1] The protective vertical separation distance is a separation from the seasonal high groundwater elevation. However, the seasonal high groundwater elevation fluctuates annually. The factor of safety accounts for these annual fluctuations in seasonal groundwater high, and was calculated using a prediction interval. A prediction interval contains a specified percent of the data from a distribution. For example, the upper 90 percent prediction interval for seasonal high groundwater elevation at a well contains 90 percent of the observed seasonal groundwater highs.

Groundwater elevation measurements from State of Oregon observation well CLAC 54227 (located in T3S R1W Section 24DD) were downloaded from the OWRD online groundwater elevation database. The period of record for CLAC 54227 is 1998 to 2012, and the well completed in the Qfc. The seasonal high groundwater elevation for each calendar year was identified, and one-sided nonparametric prediction interval was calculated using Equation 3.11 in Helsel and Hirsch (2002). Data from a calendar year was used only if data from February through May were available, which is when the seasonal groundwater high typically occurs. Also, data from 1999 through 2001 was excluded from the analysis because water levels appear to be outliers from the remainder of the data. The prediction interval for CLAC 54227 was 1.1 feet greater than their median seasonal high groundwater elevations. Therefore, annual variation in seasonal high groundwater elevations is expected to be within 1.1 feet (LANE 8029) of the median seasonal high groundwater elevation 90 percent of the time. The measure of safety was conservatively chosen to be 1.1 feet.

5. Conclusion and Recommendations

The GWPD models indicate that UICs are protective if they meet at least one of the following two conditions: (1) vertical separation distance between the UIC and seasonal high groundwater is more than 2.5 feet or (2) horizontal separation distance between a UIC and water well is more than 267 feet. UICs that do not meet one of these two conditions must be retrofit, decommissioned, or demonstrated to be protective using a different method.

Relative risk posed by UICs in the City is summarized in Table 7, and shown in Figure 6. UICs are classified according to the following risk categories:

- **Red = High Risk.** The following types of UICs are designated as high risk:
  - UICs that do not have the horizontal or vertical criteria for protectiveness. Specifically, the vertical separation distance is less than 2.5 feet <u>and</u> horizontal separation distance is less than 267 feet at these UICs.
  - UICs that drain areas where the stormwater potentially has a high pollutant load (UIC E-8, which is located near the garbage and grit dumpster at the wastewater treatment plant, and UIC E-11, which is located near a vehicle wash bay at the motor pool).

  A total of six high-risk UICs were identified.

- **Yellow = Moderate Risk.** These UICs are protective because they have more than 267 feet of horizontal separation distance; therefore, DEQ does not require decommissioning or retrofit of these UICs. However, because these are wet feet UICs, they are considered to pose a higher risk to groundwater and are candidates for retrofit. Wet-feet UICs were identified using two methods: (1) information provided by the public works staff indicating that the UICs contained water (Darvin Tramel, personal communication, May 6, 2013), and (2) comparison of UIC depth to the depth to seasonal high groundwater in Figure 4 (i.e., UIC depth is greater than depth to seasonal high groundwater at wet-feet UICs).

  The method used to identify wet-feet UICs is provided in Table 7. A total of six wet-feet UICs were identified on the basis of information provided by the public works staff, and 20 UICs were identified on the basis of the depth to seasonal high groundwater map. GSI recommends site visits to the wet-feet UICs identified during April (the time of seasonal high groundwater) to confirm that they are wet-feet UICs.

- **Green = Low Risk.** UICs that meet at least one of the two conditions for protectiveness listed above. At these UICs, the vertical separation distance is greater than 2.5 feet <u>or</u> horizontal separation distance is greater than 267 feet.

A total of six high-risk and 26 moderate-risk UICs were identified. The high-risk and moderate-risk UICs are located primarily along Northwest 3rd Avenue and North Holly, North Pine, and Northeast 10th, and the wastewater treatment plant. The high-risk UICs may be addressed by retrofit, decommissioning, or an alternative GWPD, which may include:

- Demonstration to DEQ that the nearby water well is no longer being used for domestic purposes, or has been decommissioned.

- Demonstration that the UIC is outside of the capture zone of the water well (a capture zone is the area of groundwater that drains to a UIC).
- Demonstration that the water well is constructed in a manner that is protective against stormwater infiltration (i.e., based on the locations of the well seal and well screen).

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Tables

Table 1

Unsaturated Zone GWPD Model Input Parameters – Soil Properties

City of Canby

Input Parameter	Units	Average Scenario	Reasonable Maximum Scenario	Data Source and Location of Technical Documentation
Total Porosity (η)	-	0.325	0.325	Midrange porosity for a gravel, Freeze and Cherry (1979) Table 2.4. Appendix B, Section 2.1.1.
Effective Porosity (η_e)	-	0.20	0.20	In the range of specific yields for a gravel (Johnson, 1967). Appendix B, Section 2.1.1.
Bulk Density (ρ_b)	g/cm^3	1.79	1.79	Calculated by equation 8.26 in Freeze and Cherry (1979). Appendix B, Section 2.1.2.
Dispersivity (α)	m/d	5% of transport distance	5% of transport distance	Calculated based on Gelhar (1985). Appendix B, Section 2.1.3.
Pore Water Velocity (v)	m/d	0.42	0.73	Based on 8 specific capacity tests at water wells conducted at water wells that are less than 50 feet deep. Average scenario uses the median of permeability measurements, reasonable maximum scenario uses the 95% UCL on the mean of permeability measurements. Appendix B, Section 2.1.4.

Notes

g/cm^3 = grams per cubic centimeter

m/d = meters per day

95% UCL = 95% Upper Confidence Limit on the mean, based on the 95% H-UCL, which assumes a lognormal distribution of K

(-) = input parameter units are dimensionless

Table 2

Unsaturated Zone GWPD Model Input Parameters – Pollutant Properties

City of Canby

Input Parameter	Units	Pollutant	Average Scenario	Reasonable Maximum Scenario	Data Source and Location of Technical Documentation
Initial Concentration	μg/L	PCP	10	10	Action Level in City of Eugene UIC WPCF Permit
		DEHP	60	60	Action Level in City of Gresham UIC WPCF Permit
		B(a)P	2	2	Action Level in City of Eugene UIC WPCF Permit
		Lead	500	500	Action Level in City of Eugene UIC WPCF Permit
Organic Carbon Partitioning Coefficient (K_{oc})	L/Kg	PCP	703	703	EPA (1996), assuming a pH of 6.6 based on groundwater pH measured at USGS observation wells. Appendix B, Section 2.3.1.
		DEHP	12,200	12,200	Calculated based on equations in Roy and Griffin (1985). Appendix B, Section 2.3.1.
		B(a)P	282,185	282,185	
Distribution Coefficient (K_d)	L/Kg	PCP	5.4	1.0	Calculated based on Equation 5.12 in Watts (1998). Appendix B, Section 2.3.2.
		DEHP	94.4	16.7	Calculated based on Equation 5.12 in Watts (1998). Appendix B, Section 2.3.2.
		B(a)P	2,184	387	Calculated based on Equation 5.12 in Watts (1998). Appendix B, Section 2.3.2.
		Lead	1,200,000	535,000	
Half Life (h)	d	PCP	31.4	49.9	Literature values. Appendix B, Section 2.3.3.
		DEHP	46.2	69.3	Literature values. Appendix B, Section 2.3.3.
		B(a)P	533	2,666	Literature values. Appendix B, Section 2.3.3.
Retardation Factor (R)	-	PCP	30.9	6.3	Calculated based on Equation (9.14) in Freeze and Cherry (1979). Appendix B, Section 2.3.4.
		DEHP	521	93	
		B(a)P	12,022	2,129	
		Lead	6,600,000	2,900,000	

Notes

d = days

L/Kg = Liters per Kilogram

mg/L = micrograms per liter

DEHP = di(2-ethylhexyl) phthalate

(-) = input parameter units are dimensionless

PCP = pentachlorophenol

B(a)P = benzo(a)pyrene

H = horizontal UIC

V = vertical UIC

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Table 3

Unsaturated Zone GWPD - Protective Vertical Separation Distances

City of Canby

Pollutant	MRL (µg/L)	Minimum Protective Vertical Separation Distance (feet)		
		Average Scenario	Reasonable Maximum Scenario	Recommended Value [3]
Lead [1]	0.1	< 0.1	< 0.1	2.5
Benzo(a)pyrene	0.01	< 0.1	< 0.1	
PCP	0.04	1.4	11.4	
DEHP	1	< 0.1	0.66	

Notes:

MRL = method reporting limit

µg/L = micrograms per liter

PCP = pentachlorophenol

DEHP = di(2-ethylhexyl)phthalate

[1] Metals transport simulations are longer than 12.80 days because metals do not biodegrade over time. Metals transport simulations assume 1000 years of transport at 12.80 days per year = 12,800 days of transport.

[2] The vertical separation distance in the unsaturated zone that is necessary for pollutant concentrations to attenuate to below the method reporting limit.

[3] "Recommended Value" is based on PCP, which migrates further than the other pollutants that were modeled. The Recommeded Value was calculated by adding the minimum protective vertical separation distance for PCP under the average scenario (1.4 feet) to a safety measure of 1.1 feet. The safety measure accounts for uncertainties in the seasonal high groundwater elevation contour map and natural variation of seasonal high groundwater elevations over time.

Table 4

Saturated Zone GWPD Model Input Parameters – Soil Properties

City of Canby

Input Parameter	Units	Base Model and DB Sensitivity Runs	Data Source and Location of Technical Documentation
Total Porosity (η)	-	0.325	Midrange porosity for a gravel, Freeze and Cherry (1979) Table 2.4. Appendix C, Section 2.4.1.
Effective Porosity (η_e)	-	0.20	Range of specific yields for a gravel in Johnson (1967). Appendix C, Section 2.4.1.
Hydraulic Conductivity (K)	ft/d	26.6	Median hydraulic conductivity calculated from well tests available on OWRD well logs in the Missoula Flood Deposits (Qfc) Appendix C, Section 2.4.1.
Hydraulic Gradient (h)	ft/ft	0.011	Based on groundwater elevation contour map in the City of Canby
Bulk Density (ρ_b)	g/cm^3	1.79	Calculated by equation 8.26 in Freeze and Cherry (1979). Appendix B, Section 2.1.2.
Longitudinal Dispersivity (α_L)	ft	17.93	Calculated using Xu and Eckstein (1995). aL = (3.28)(0.83)[log(Lp/3.28)]2.414. A transport distance (L_p) of 500 feet was used in the calculation). Appendix C, Section 2.4.1.
Transverse Dispersivity (y-direction)	ft	5.92	Calculated using EPA (1986). $a_T = 0.33(a_L)$. Appendix C, Section 2.4.1.
Vertical Dispersivity (z-direction)	ft	1.79	Calculated using EPA (1986). $a_v = 0.10(a_L)$. Appendix C, Section 2.4.1.

Notes

g/cm^3 = grams per cubic centimeter

ft/d = feet per day

ft = feet

DB Sensitivity Runs = Drainage Basin Sensitivity Runs

(-) = input parameter units are dimensionless

Table 5

Saturated Zone GWPD Model Input Parameters – Pollutant Properties

City of Canby

Input Parameter	Units	Pollutant	Base Model - Near Vertical UIC	Base Model - Distal From Vertical UIC	Data Source and Location of Technical Documentation
Initial Concentration	µg/L	PCP	10	10	Action Level in City of Eugene UIC WPCF Permit
		DEHP	60	60	Action Level in City of Gresham UIC WPCF Permit
		B(a)P	2	2	Action Level in City of Eugene UIC WPCF Permit
		Lead	500	500	Action Level in City of Eugene UIC WPCF Permit
Organic Carbon Partitioning Coefficient (K_{oc})	L/Kg	PCP	703	703	EPA (1996), assuming a pH of 6.6 from USGS monitoring wells. Appendix B, Section 2.3.1.
		DEHP	12,200	12,200	Calculated based on equations in Roy and Griffin (1985). Appendix B, Section 2.3.1.
		B(a)P	282,185	282,185	
Distribution Coefficient (K_d)	L/Kg	PCP	8.2	1.3	Calculated based on Equation 5.12 in Watts (1998). Appendix B, Section 2.3.2.
		DEHP	142	22.3	Calculated based on Equation 5.12 in Watts (1998). Appendix B, Section 2.3.2.
		B(a)P	3,293	515	Calculated based on Equation 5.12 in Watts (1998). Appendix B, Section 2.3.2.
		Lead	1,000,000	1,000,000	
Half Life (h)	d	PCP	46	46	Literature values. Appendix C, Section 2.4.2.
		DEHP	10	10	Literature values. Appendix C, Section 2.4.2.
		B(a)P	587	587	Literature values. Appendix C, Section 2.4.2.
Retardation Factor (R)	-	PCP	74	6.95	Calculated based on Equation (9.14) in Freeze and Cherry (1979). Appendix B, Section 2.3.4.
		DEHP	1,260	124	
		B(a)P	29,471	2,800	
		Lead	5,500,000	5,500,000	

Notes

d = days

L/Kg = Liters per Kilogram

mg/L = micrograms per liter

DEHP = di(2-ethylhexyl) phthalate

(-) = input parameter units are dimensionless

PCP = pentachlorophenol

B(a)P = benzo(a)pyrene

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P:\Portland\390-Kennedy_Jenks\002-Canby_Risk_Model\Reports-Memos\TABLES\Main Text\TABLE 5 - POLLUTANT PROPERTY INPUT PARMS sz

Table 6

Saturated Zone GWPD -- Protective Horizontal Separation Distance

City of Canby

Pollutant	Minimum Protective Horizontal Separation Distance (feet)
Lead	5
Benzo(a)pyrene	33
PCP	267
DEHP	67

Notes:

DEHP = di(2-ethylhexyl)phthalate

PCP = pentachlorophenol

Table 7

Relative Risk at UICs

City of Canby

UIC ID	UIC Address	Vertical Separation Distance (feet)	Distance to Nearest Water Well (feet)	Nearest Water Well ID
High Risk				
D-63	N Pine and NE 10th	< 0.0 (1)	69.1	CLAC 12047
None	NE 2nd and N Ivy	-8.05	233.6	CLAC 12792
None	NW 3rd and N Holly	-11.29	83.8	CLAC 9668
D-48	NW 3rd and N Holly	-10.83	144	CLAC 9668
E-8	1480 NE Territorial (WWTP)	6.9	1109	Cert 55066
E-11	1490 NE Territorial (PW Complex)	9.0	626	Cert 55066
Moderate Risk				
D-28	NW 11th Ave and N Pine St	< 0.0 (1)	278	CLAC 12047
D-35	NW 14th Ave and N Oak St	< 0.0 (1)	295	CLAC 9703
D-31	NW 10th Ave and N Oak St	< 0.0 (1)	408	CLAC 12047
D-26	NW 12th Ave and N Pine St	< 0.0 (1)	574	CLAC 12047
D-23	NW 13th Ave and N Pine St	< 0.0 (1)	617	CLAC 9675
F-7	N Birch and Territorial	< 0.0 (1)	684	Claim GR2913
None	NW 2nd Ave. and N Ivy St.	-8.14 (2)	272.01	C12792
A-62	SW 2nd Ave	-0.52 (2)	304.05	C12048
D-54	NE 4th Ave and N Juniper St	-0.49 (2)	415.20	C12266
D-55	NW 4th Ave and N Ivy St	-5.24 (2)	439.93	C12792
D-64	NE 3rd Ave and N Juniper St	-2.22 (2)	446.20	C12792
X-1	S Hazel Dell Way and S Sequoia Py	-13.91 (2)	527.48	C12103
X-2	S Hazel Dell Way and S Sequoia Py	-13.40 (2)	544.42	C12103
None	NE 1st Ave and N Ivy St	-6.86 (2)	556.65	C12792
None	S Hazel Dell Way and S Sequoia Py	-13.50 (2)	570.37	C12103
None	S Hazel Dell Way and S Sequoia Py	-13.82 (2)	582.14	Cert 30448
B-18	N Baker St and N 8th Way	-12.85 (2)	624.82	Inchoate T7068
X-3	S Sequoia Parkway	-11.78 (2)	640.45	C12103
None	NE 1st Ave	-4.91 (2)	647.97	C12792
B-19	N Baker St and N 8th Way	-9.92 (2)	704.36	Inchoate T7068
D-27	NE 12th Way	-1.14 (2)	708.36	C12047
C-44	S Knott Ct and S Knott St	-0.05 (2)	726.39	C64610
X-5	S Sequoia Parkway	-6.00 (2)	788.46	C12104
X-4	S Sequoia Parkway	-10.02 (2)	789.50	C12103
None	N Baker Drive	-25.77 (2)	930.90	C12038
C-1	SE 3rd Avenue and S Knott St	-1.93 (2)	1004.26	C12048

Notes

[1] Wet feet conditions based on observations by City public works department

[2] Wet feet conditions based on depth to seasonal high grounwater map

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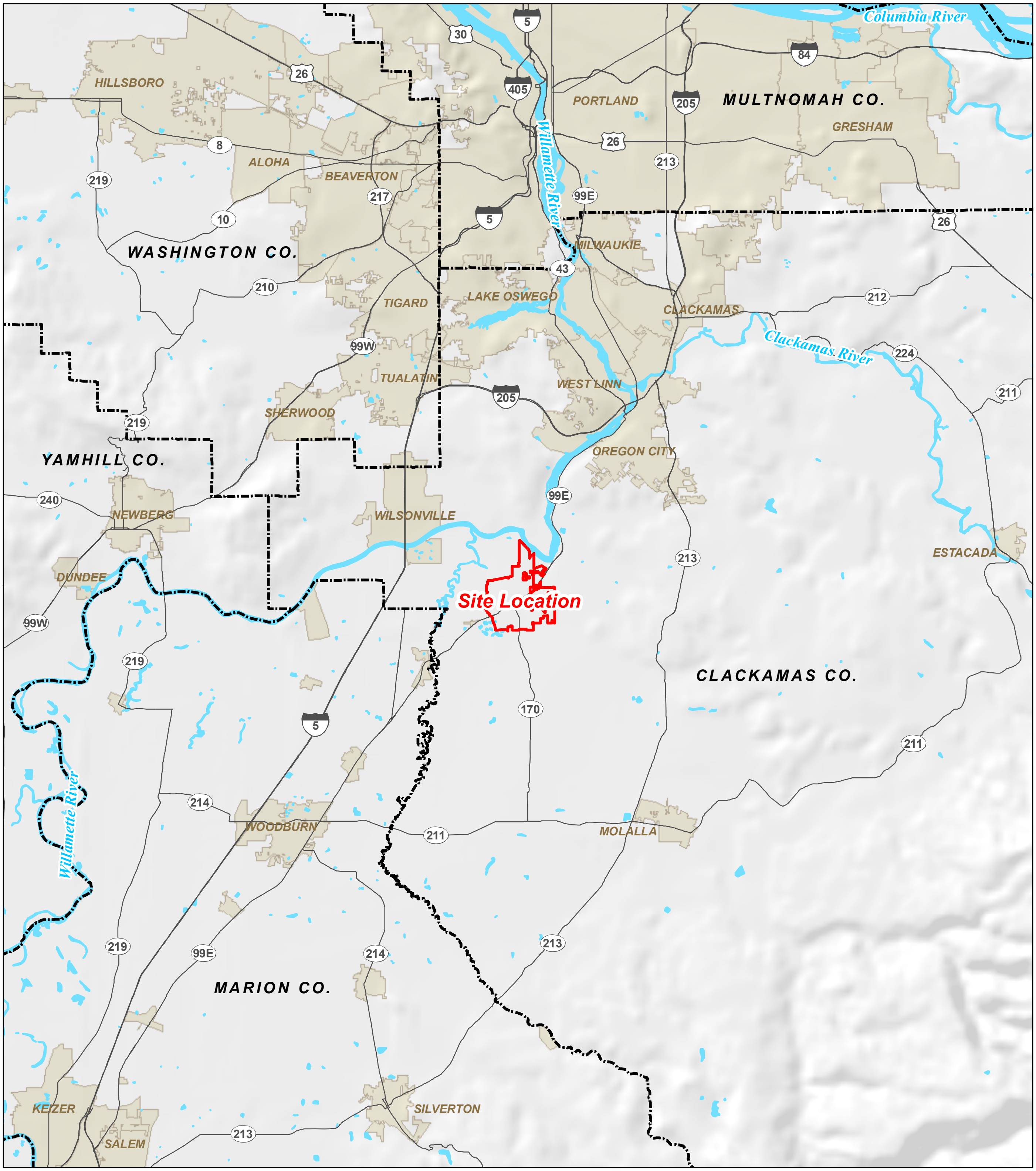
Figures



LEGEND

- Site Location
- Cities
- Counties
- Highways
- Major Waterbodies

MAP NOTES:
Date: August 28, 2013
Data Sources: METRO RLIS, USGS, ESRI

FIGURE 1

Site Location

City of Canby

Groundwater Protectiveness Demonstration

N

0 2.5 5
Miles

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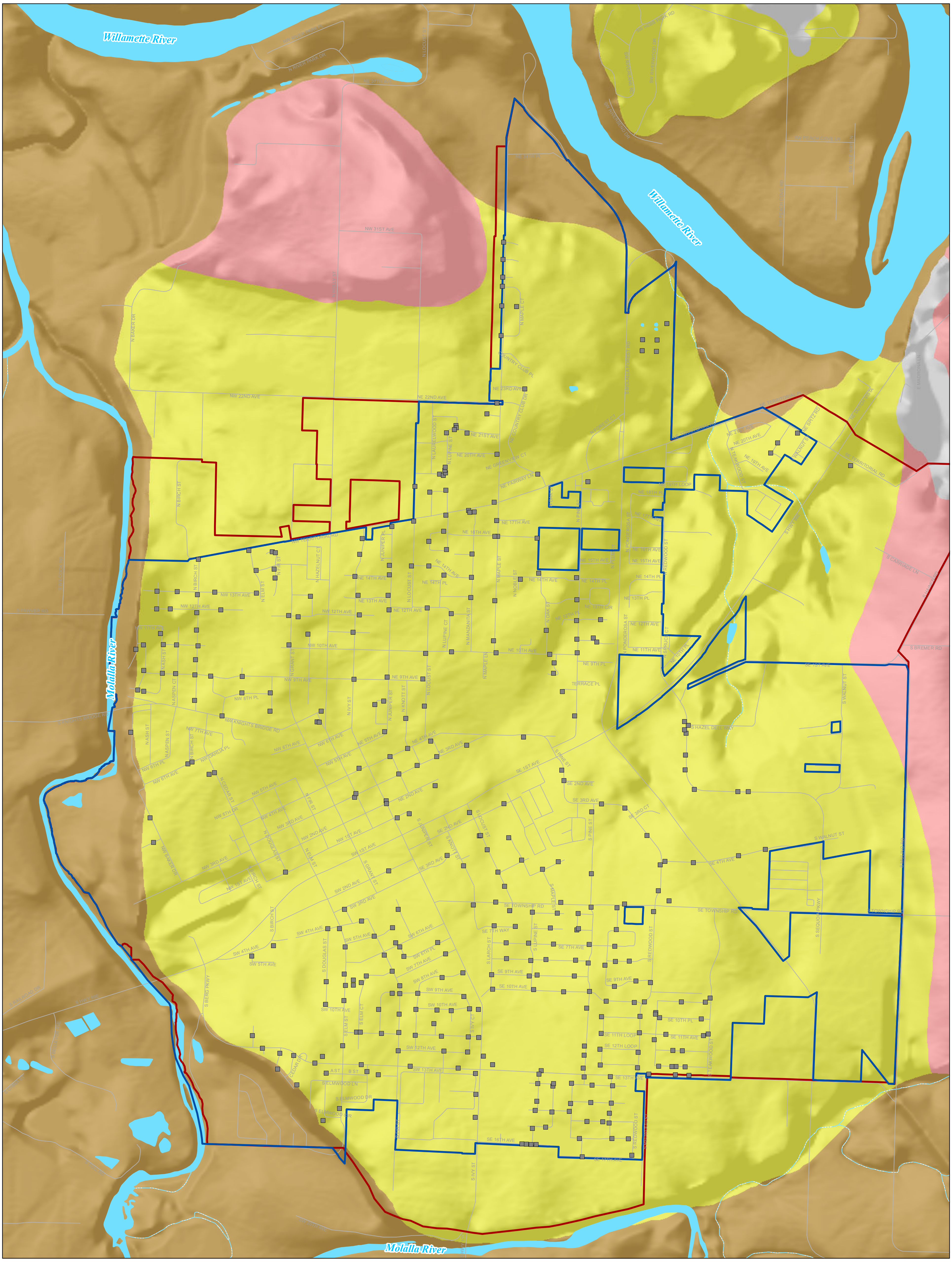

LEGEND

▪ City UIC

Surficial Geology

Catastrophic Flood Deposits

- Qff - Catastrophic flood deposits, fine grained facies
- Qfc - Catastrophic flood deposits, coarse grained facies
- Miocene Columbia River Basalt Group Lavas

Additional Quaternary Surficial Deposits

- Other Quaternary Surficial Deposits

All Other Features

- Canby City Limits
- Canby Urban Growth Boundary
- Streets
- Watercourses
- Waterbodies

MAP NOTES:
Date: August 28, 2013
Data Sources: UICs from City of Canby (May 2013), OWRD, DOGAMI, USGS, METRO RLIS, OGIC
Wet Feet Well E-7 is not shown on the map.

N

0 750 1,500 2,250
Feet

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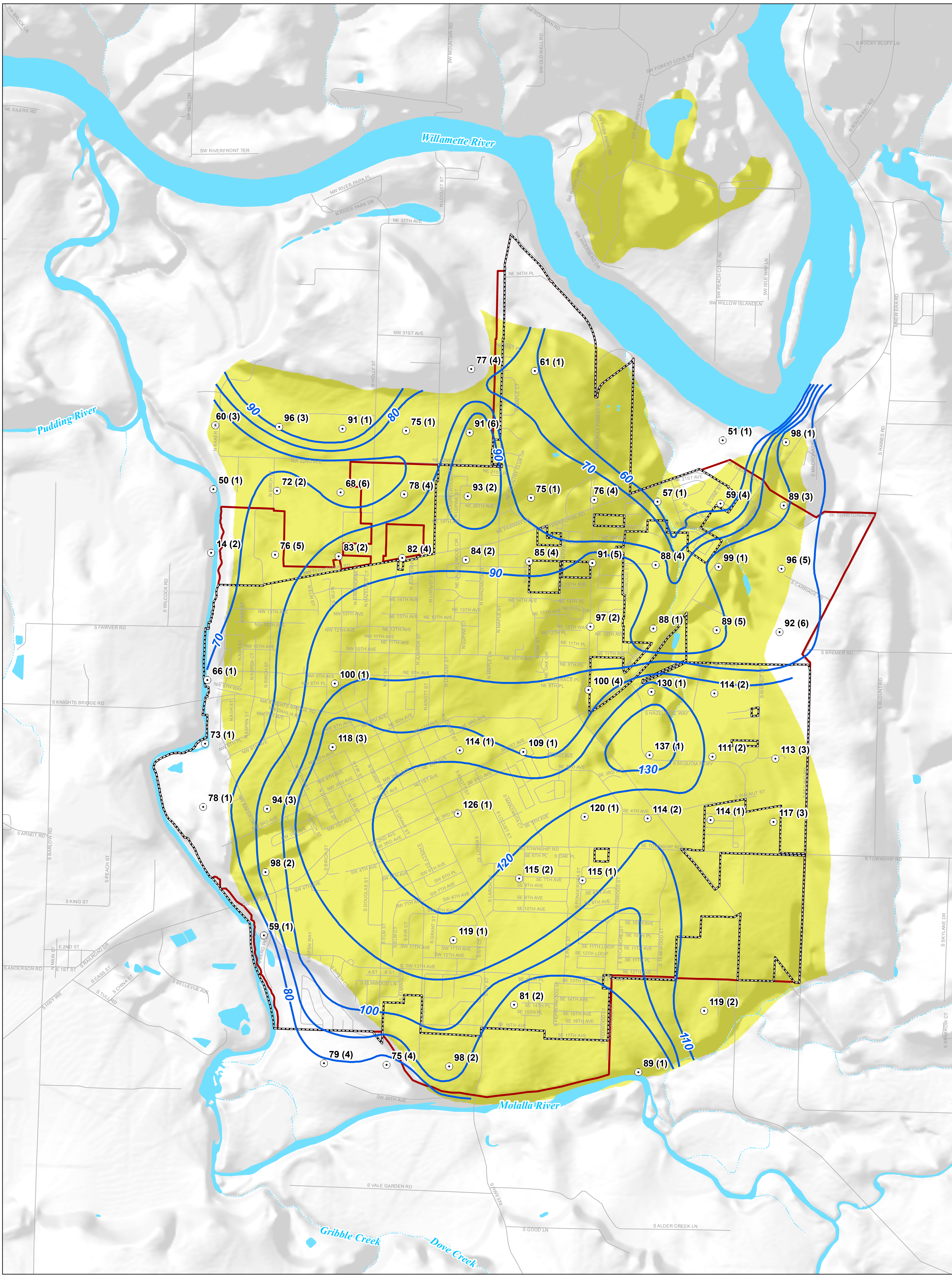
FIGURE 2

Surficial Geology and UICs

City of Canby
Groundwater Protectiveness Demonstration



LEGEND

Well Location (centroid of 1/4 1/4 section)
- Number immediately adjacent to well represents groundwater elevation
- Number in parenthesis represents number of observations

Static Groundwater Elevation Contours

All Other Features

Qfc - Catastrophic flood deposits, coarse grained facies

Canby City Limits

Canby Urban Growth Boundary

Streets

Watercourses

Waterbodies

MAP NOTES:
Date: August 28, 2013
Data Sources: OWRD, USGS, METRO RLIS, OGIC,
Elevation data based on NGVD 1927 vertical datum

N

0 1,000 2,000 3,000
Feet

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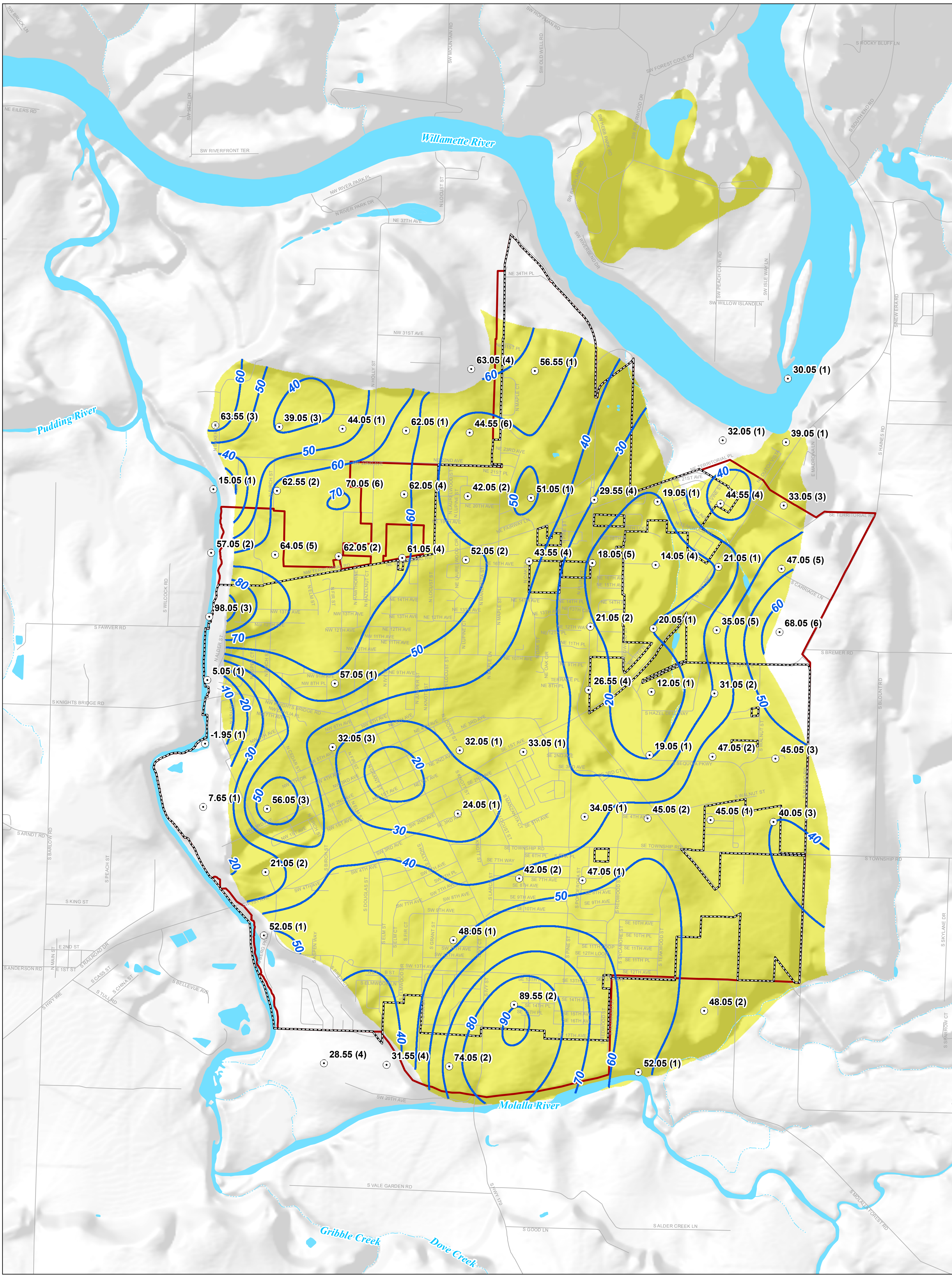
FIGURE 3

Static Groundwater Elevation

City of Canby
Groundwater Protectiveness Demonstration



LEGEND

Well Location (centroid of 1/4 1/4 section)
- Number represents depth to groundwater
- Number in parenthesis represents number of observations

Static Depth to Water Contours

Streets

Watercourses

Waterbodies

All Other Features

Qfc - Catastrophic flood deposits, coarse grained facies

Canby City Limits

Canby Urban Growth Boundary

MAP NOTES:
Date: August 28, 2013
Data Sources: OWRD, USGS, METRO RLIS, OGIC,
Elevation data based on NGVD 1927 vertical datum

N

0 1,000 2,000 3,000
Feet

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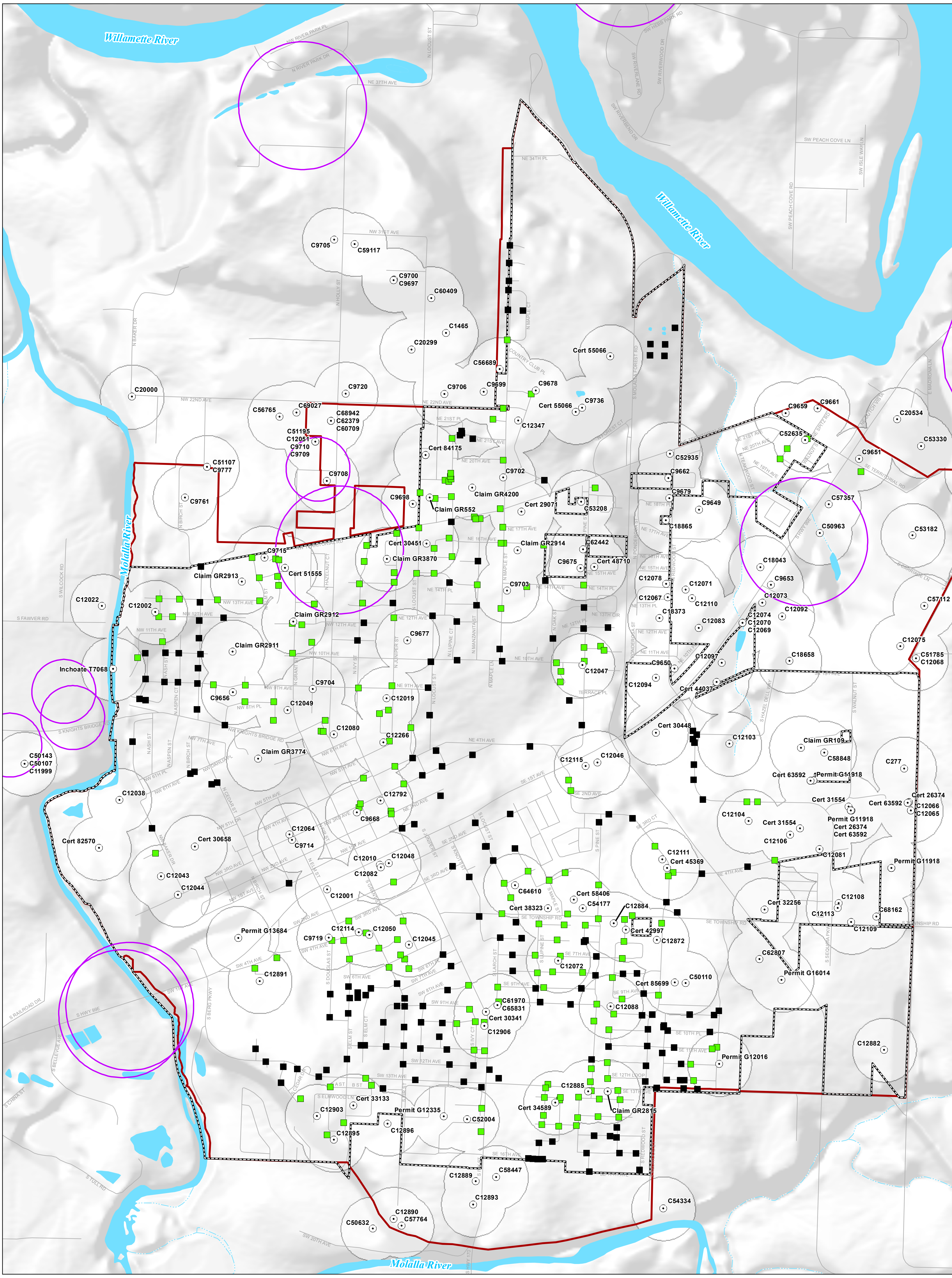
FIGURE 4

Depth to Seasonal High Groundwater

City of Canby
Groundwater Protectiveness Demonstration



LEGEND

- UICs within 500 feet of Water Well
- UICs not within 500 feet of Water Well

All Other Features

- Water Wells
- Water Wells - 500 ft Buffers
- Two Year Time of Travel Zones
- Canby City Limits
- Canby Urban Growth Boundary
- Streets
- Watercourses
- Waterbodies

NOTE:
Two year time of travel zones are from the Oregon Department of Environmental Quality's 2005 source water assessment results for public water systems in Oregon. Wells associated with the two year time of travel zones are not included in the 2005 data, and therefore are not shown on this map.

N

0 750 1,500 2,250
Feet

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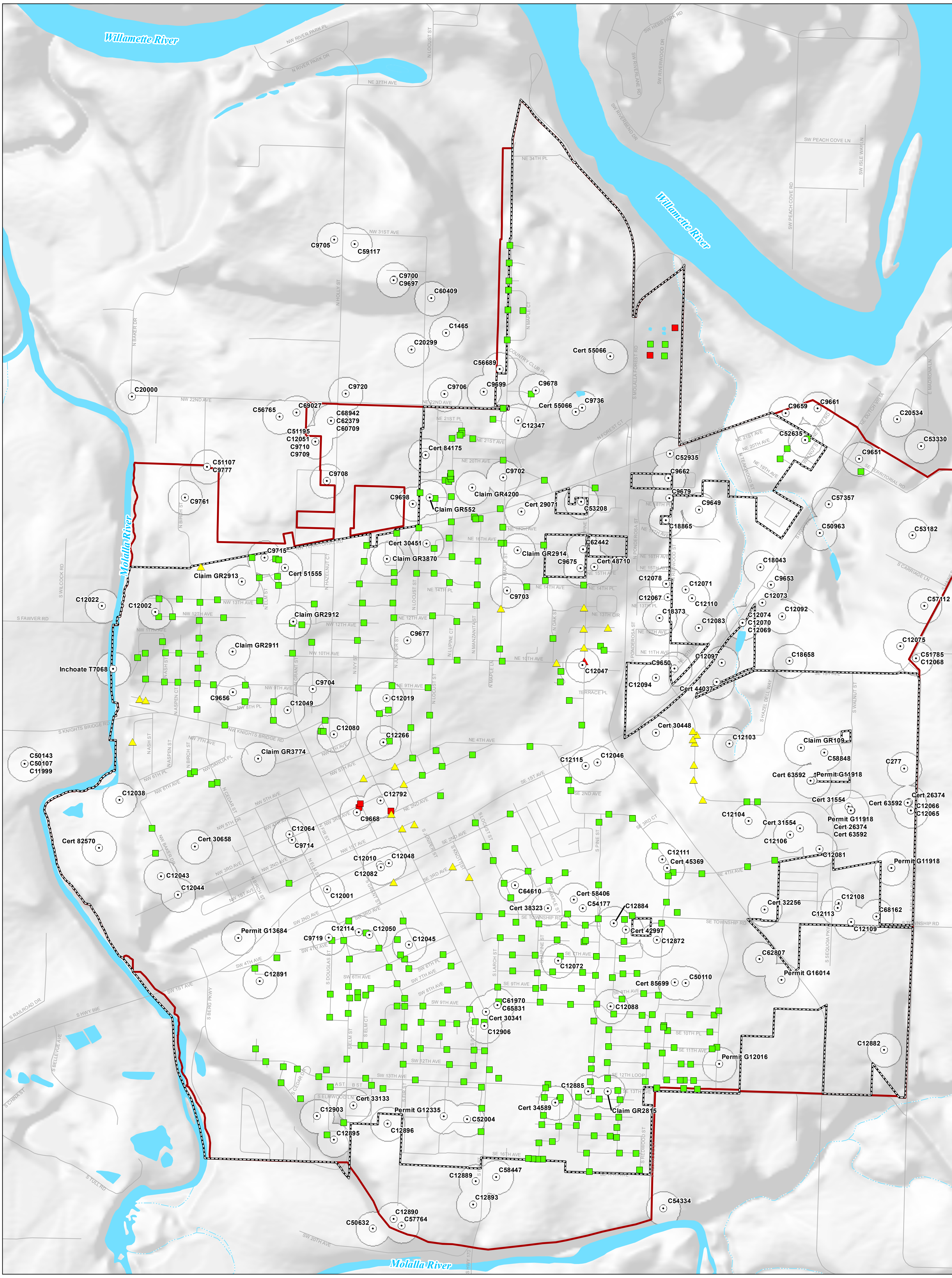
MAP NOTES:
Date: August 28, 2013
Data Sources: UICs from City of Canby (May 2013), OWRD, DOGAMI, USGS, METRO RLIS, OGIC, OR DEQ

FIGURE 5
UIC and Water Well Location Map
City of Canby



LEGEND

UICs
- Wet Feet
- Dry Feet
- High Risk
- Moderate Risk
- Low Risk

All Other Features
- Water Wells
- Water Wells - 267 ft Buffers
- Canby City Limits
- Urban Growth Boundary

- Streets
- Watercourses
- Waterbodies

FIGURE 6

UIC Risk Prioritization

City of Canby
Groundwater Protectiveness Demonstration

MAP NOTES:
Date: August 28, 2013
Data Sources: UICs from City of Canby (May 2013), OWRD, USGS, METRO RLIS, OGIC,
UICs E-8 and E-11 are protective based on model results. However, they are identified as "high risk" because they drain areas that potentially have a high pollutant load.

N

0 750 1,500 2,250
Feet

GSI Water Solutions, Inc.

Attachments

Attachment A – Technical Documentation for Water Well Location Map and Groundwater Elevation Map

1. Water Well Location Map

Water wells were located using online databases that are maintained by the Oregon Water Resources Department (OWRD). The following two databases were used:

- **OWRD Water Rights Database.** The OWRD maintains a database of water rights in Oregon (OWRD, 2013a). Groundwater rights are associated with a well, and the wells location is relatively accurate (e.g., located based on distance and bearing from a section corner).

  GSI downloaded groundwater rights from the database in March 2013. The water rights are identified as "TYPE-XXX," with "TYPE" indicating whether the water right is a claim, permit, or certificate, and "XXX" indicating the claim, permit, or certificate number. For example, "Permit G13684" is the water well identified on Permit G13684.

- **OWRD On-Line Well Log Query Database.** When a monitoring well, geotechnical hole, or water well is drilled, the well driller submits a record of the well to the State of Oregon. The records (i.e., well logs) are available in an on-line well log query database (OWRD, 2013b). Well logs include well location information, which exhibits a wide range of accuracy. For example, the accuracy of a well location may be as good as to a specific property by address or tax lot (accuracy of +/- 100 feet) or may be as poor as to a section (accuracy of +/- 5,280 feet).

  GSI downloaded well logs for water wells from the database in March 2013. Well logs that were accurate to a property address or tax lot were plotted using the tax lot or address on the water well location map. Decommissioned wells were not included in the analysis. The wells were identified as "C-XXX," with "C" indicating the county identifier from the online well log query database (i.e., Clackamas), and "XXX" indicating the well identification number from the online database. For example, C9706 indicates CLAC 9706 in the online well query.

The water well location map is presented in Figure 5 of the attached technical memorandum titled *Groundwater Protectiveness Demonstrations and Risk Prioritization for Underground Injection Control (UIC) Devices, City of Canby, Oregon.*

2. Depth to Groundwater Map

The depth to groundwater map is a tool that is used to understand the relative risk from Underground Injection Controls (UIC) based on vertical separation distance to seasonal high groundwater and to prioritize UICs for retrofit or decommissioning. As such, it is an approximation of the groundwater table with uncertainties related to the fact that wells are completed at different depths, water levels were measured during different times of the year, and water levels may have been influenced by the drilling activity.

Depths to seasonal high groundwater were determined using the following methodology:

- **Tabulate depths to static groundwater.** Water well logs were downloaded from the OWRD online well log database (OWRD, 2013b). The following types of well logs were used:
    - Well location information included the quarter-quarter section (specifically, wells without quarter-quarter section information were excluded).
    - Well was completed in unconsolidated sediments (specifically, wells completed in basalt were excluded).
    - The well type was a water well.
    - Well depth was less than 200 feet.

  Static groundwater depths for each quarter-quarter section within the City of Canby's urban grown boundary were tabulated on the basis of information from the water well logs. If there was more than one groundwater depth for a quarter-quarter section, the median groundwater depth was calculated.

  Ideally, depth to seasonal high groundwater would be evaluated only by using groundwater measurements from March and April, when groundwater levels are highest. However, the data tabulation used static groundwater depths that were measured during all times of the year to provide better coverage throughout the City.

- **Calculate groundwater elevation.** Depths to groundwater were converted to groundwater elevation based on a U.S. Geological Survey 10-meter digital elevation model (DEM). Groundwater elevations were contoured by hand with the objective of identifying outliers in the static water level dataset (i.e., groundwater data that do not fit the general pattern of groundwater levels across the City). A single outlier was identified (CLAC 12101) and excluded from the maps.

- **Contour depth to groundwater.** Depth to groundwater was contoured using a spline interpolation technique in geographic information system (GIS) software.

- **Calculate depth to seasonal high groundwater.** Because groundwater measurements were collected throughout the year, they are not representative of the depth to seasonal high groundwater and need to be adjusted. The depth to seasonal groundwater high map was developed by subtracting 2.95 feet from each groundwater depth. The 2.95 feet measurement is based on Snyder (2008), who calculated a seasonal fluctuation (i.e., the difference between the maximum and minimum groundwater levels) of 5.9 feet based on recharge and effective porosity in the Unconsolidated Sedimentary Aquifer. Using half of 5.9 feet (i.e., 2.95 feet) has the effect of converting the median depth to groundwater to a depth to seasonal high groundwater.

In the depth to groundwater contour map, 155 static groundwater elevations were used.

References

OWRD. 2013a. Water Right Information Search. Accessed by GSI in March 2013. Oregon Water Resources Department (OWRD). Available online at: http://www.oregon.gov/owrd/pages/WR/wris.aspx

OWRD. 2013b. Well Log Query. Accessed by GSI in March 2013. Oregon Water Resources Department (OWRD). Available online at: http://apps.wrd.state.or.us/apps/gw/well_log/Default.aspx

Snyder, D.T. 2008. Estimated Depth to Ground Water and Configuration of the Water Table in the Portland, Oregon, Area. U.S. Geological Survey Scientific Investigations Report 2008–5059, 40 p. (Available at http://pubs.usgs.gov/sir/2008/5059/)

Attachment B – Technical Documentation for the Unsaturated Zone GWPD

1. Pollutant Fate and Transport Processes

An Underground Injection Control (UIC) device allows stormwater to infiltrate into the unsaturated zone (i.e., variably saturated soils above the water table). The stormwater is transported downward by matric forces that hold the water close to mineral grain surfaces. During transport, pollutant concentrations are attenuated by the following processes:

- **Volatilization.** Volatilization is pollutant attenuation by transfer from the dissolved phase to the vapor phase. Because soil pores in the unsaturated zone are only partially filled with water, chemicals with a high vapor pressure volatilize into the vapor phase. The propensity of a pollutant to volatilize is described by the Henry's constant. Volatilization is not significant at depths below most UIC bottoms (EPA, 2001), so volatilization is not included in the unsaturated zone Groundwater Protectiveness Demonstration (GWPD).

- **Adsorption.** Adsorption is pollutant attenuation by partitioning of substances in the liquid phase onto the surface of a solid substrate. Physical adsorption is caused mainly by Van der Waals forces and electrostatic forces between the pollutant molecule and the ions of the solid substrate molecule's surface. For organic pollutants, the unsaturated zone GWPD simulates adsorption is a function of f_{oc} (fraction organic compound) and K_{oc} (organic carbon partitioning coefficient). For metals, the unsaturated zone GWPD uses stormwater analytical data to estimate adsorption.

- **Degradation.** Degradation is pollutant attenuation by biotic and abiotic processes. Abiotic degradation includes hydrolysis, oxidation-reduction, and photolysis. Biotic degradation involves microorganisms metabolizing pollutants through biochemical reactions.

- **Dispersion.** Dispersion is pollutant attenuation from porewater mixing, which occurs because of differences in subsurface permeability.

2. Pollutant Fate and Transport Input Parameters

The unsaturated zone GWPD consists of a 1-dimensional analytical model (Advection Dispersion Equation) that simulates the effects of adsorption, degradation, and dispersion based on user-specified input parameters from scientific references and available regulatory guidance. Input parameters to the unsaturated zone GWPD model include soil properties, organic carbon content in the subsurface, and pollutant properties, as described in the following sections:

- Soil properties
  - Total porosity and effective porosity (Section 2.1.1)
  - Soil bulk density (Section 2.1.2)
  - Dispersion coefficient and dispersivity (Section 2.1.3)
  - Average linear pore water velocity (Section 2.1.4)
- Organic carbon content of the subsurface
  - Fraction organic carbon (Section 2.2.1)
- Pollutant properties
  - Organic carbon partitioning coefficient (Section 2.3.1)
  - Distribution coefficient (Section 2.3.2)
  - Degradation rate constant and half life (Section 2.3.3)
  - Retardation factor (Section 2.3.4)

Some of the input parameters for the unsaturated zone GWPD are varied to evaluate two scenarios for pollutant fate and transport: (1) the average scenario, which is represented by the central tendency or expected mean value of the input parameter, and (2) the reasonable maximum scenario, which is an upper bound on what could occur, but is considered unlikely to occur because of compounding conservatism.

Pollutant fate and transport are simulated for organic pollutants pentachlorophenol (PCP); di(2-ethylhexyl)phthalate (DEHP); benzo(a)pyrene; and the metal lead. These pollutants are detected regularly in stormwater, and of those, are among the most mobile, toxic, and/or environmentally persistent in their respective chemical classes (GSI, 2008), and are the most likely pollutants in their respective chemical classes to exceed regulatory standards for stormwater at UICs (Kennedy/Jenks, 2009).

2.1 Soil Properties

Soil properties include total porosity, effective porosity, soil bulk density, dispersivity/dispersion coefficient, and average linear pore water velocity.

2.1.1 Total Porosity (η) and Effective Porosity (η_e)

Total porosity is the percent of pore space in a material. Porosities are correlated with soil type (e.g., sand, silt, gravel), and were estimated from Table 2.4 of Freeze and Cherry (1979). Specifically, the midrange porosity of a gravel was used because the City of Canby's UICs are completed in gravels of the Qfc geologic unit. Effective porosity is the percent of pore space through which flow occurs, as was estimated as 0.20 for the Qfc. The value of 0.20 is within the range of specific yields for gravels reported by Johnson (1967) (specific yield is approximately equivalent to effective porosity). It should be noted that this effective porosity is conservatively lower than the effective porosity of the Qfc gravels reported in other studies (e.g., 0.31 was used in Snyder et al. [1998]).

2.1.2 Soil Bulk Density (ρ_b)

Bulk density is the density of a soil, including soil particles and pore space. According to Freeze and Cherry (1979), bulk density is calculated from total porosity by the following formula:

$$\rho_b = 2.65(1-\eta) \tag{B.1}$$

2.1.3 Dispersion Coefficient (*D*) and Dispersivity (α)

Dispersion is the spreading of a pollutant plume caused by differential advection. The dispersion coefficient, *D*, is defined as:

$$D = \alpha\ v \tag{B.2}$$

where:

v is average linear pore water velocity (L/T), and
α is longitudinal dispersivity (L).

The dispersivity (and therefore the dispersion coefficient) is a scale-dependent parameter. According to a review of tracer tests conducted under saturated conditions, dispersivity is estimated as (Gelhar et al., 1992):

$$\alpha \le \frac{L}{10} \tag{B.3}$$

where:

L is the length scale of transport (L).

However, according to a review of tracer tests conducted in the unsaturated zone, dispersivity can be significantly less than would be estimated by Equation (B.3) (Gehlar et al., 1985):

$$\frac{L}{10} \le \alpha \le \frac{L}{100} \tag{B.4}$$

Because the unsaturated zone under the UICs is at near-saturated conditions, the model assumes that $\alpha = \frac{L}{20}$, which is conservatively less than saturated dispersivity, but is on the high end of the reported range in unsaturated dispersivity.

2.1.4 Average Linear Pore Water Velocity (*v*)

Average linear pore water velocity is the rate that water moves vertically through the unsaturated zone, and is directly proportional to soil moisture content (i.e., pore water velocity increases as soil moisture content increases). Soil moisture content is the percent of water in soil, and is equal to or less than porosity. The unsaturated zone GWPD conservatively assumes that soils are fully saturated, which is likely representative of actual conditions because of the near-constant infiltration of water during the rainy season.

Darcy's Law is (Stephens, 1996):

$$v = -K_u\left(\frac{\partial \psi}{\partial y} + \frac{\partial y}{\partial y}\right) \tag{B.5}$$

where:

v is specific discharge (L/T),

K_u is unsaturated hydraulic conductivity (L/T),

$\left(\frac{\partial \psi}{\partial y}\right)$ is the pressure gradient (L/L), and

$\left(\frac{\partial y}{\partial y}\right)$ is the head gradient (L/L).

In the unsaturated zone, $\left(\frac{\partial y}{\partial y}\right)$ = 1. When the unsaturated zone is stratified and pressure head is averaged over many layers (which is the case in the alluvial sediments in the vicinity of the City of Canby), $\left(\frac{\partial \psi}{\partial y}\right)$ = 0. Under these conditions, Equation (B.5) reduces to (Stephens, 1996):

$$v = -K_u \tag{B.6}$$

Hydraulic conductivity was calculated on the basis of specific capacity tests reported on drillers' logs. Logs for water wells were downloaded from the Oregon Water Resources Department (OWRD) online well log query (OWRD, 2013). The following specific capacity test data were analyzed: (1) wells that are completed at less than 50 feet below ground surface (bgs) and (2) wells located in the Qfc as mapped by DOGAMI (2009). Hydraulic conductivity was calculated from pumping rate and drawdown measured during well testing by the following formula (Driscoll, pg. 1021, 1986):

$$K = \frac{\frac{2000}{7.481}\frac{Q}{s}}{b} \tag{B.7}$$

Where:

Q is pumping rate (gallons per minute)

s is drawdown (feet)

b is aquifer thickness (feet)

In Equation (B.7), 2000 and 7.481 are conversion factors. The water wells were completed in gravel interbedded with finer grained sediments (e.g., sand and silt). The thickness of gravels on each well log was conservatively used for b in Equation (B.7) (i.e., the thickness of silt and sand interbeds were not included in calculation of b). Hydraulic conductivities and input values to Equation (B.7) are presented in Table B-1.

Average linear pore water velocity is calculated by dividing Equation (B.6) by the effective porosity of 0.20.

2.2 Organic Carbon Content in the Subsurface

The organic carbon content in the subsurface is parameterized by fraction organic carbon, a dimensionless measure of the quantity of organic carbon in soil (i.e., g_{carbon} / g_{soil}). Carbon in unsaturated soil beneath a UIC is derived from two sources:

- Organic carbon incorporated into sediments during deposition
- Particulate matter (e.g., degraded leaves, pine needles, and pollen) that is filtered out of stormwater and accumulates in unsaturated soil adjacent to UICs as stormwater infiltrates from the UIC

The unsaturated zone GWPD conservatively considers only organic carbon that accumulates in the unsaturated zone soils as a result of filtering of particulate matter from stormwater.

2.2.1 Fraction Organic Carbon (f_{oc})

As stormwater infiltrates into the unsaturated zone surrounding the UIC, the organic carbon is filtered out of solution. The f_{oc} in soil increases over time because of the ongoing addition of organic carbon. An estimate of f_{oc} based on the accumulation of carbon in unsaturated soil was derived by calculating the grams of organic carbon added to unsaturated materials surrounding the UIC during a 10-year period. A 10-year accumulation period is conservative because scientific literature evaluating the longevity of organic material in bioretention cells indicates that it lasts about 20 years before it begins to degrade (Weiss et al., 2008). The following equations were used in the analysis:

$$I = (A)(p)(1-e) \tag{B.8}$$

$$CL = (I)(C)(t)\left(\frac{1\,\text{liter}}{1{,}000\,\text{cm}^3}\right)\left(\frac{1\,\text{gram}}{1{,}000\ \text{milligrams}}\right) \tag{B.9}$$

$$\rho_{oc} = \frac{CL}{SV} \tag{B.10}$$

$$f_{oc} = \frac{\rho_{oc}}{\rho_b + \rho_{oc}} \tag{B.11}$$

where:

I = Average annual stormwater infiltration volume (cubic feet per year)
A = Area of a typical UIC catchment (square feet)
p = Precipitation (feet per year)
e = Evaporative loss factor (dimensionless). The infiltration volumes assumed an evaporative loss factor of 24 percent, based on Snyder et al. (1994).
CL = Organic carbon loaded into the unsaturated zone beneath a UIC during a 10-year period (grams)
C = Average total organic carbon (TOC) concentration in stormwater (milligrams per liter)
t = Time of carbon loading (years)
ρ_{oc} = Organic carbon weight per unit unsaturated zone material volume (grams per cubic centimeter)

SV = Material volume into which the organic carbon would accumulate because of filtration and adsorption (cubic centimeters).
f_{oc} = Fraction organic carbon (dimensionless)
ρ_b = Bulk density (grams per cubic centimeter)

SV is assumed to be the volume of soil from 3 feet above the UIC bottom to 5 feet below the base of the UIC, extending 1 foot from the radius of the UIC (i.e., SV is about 5 million cubic centimeters).

Calculations of f_{oc}, based on the filtering of TOC for the average and reasonable maximum scenarios, are shown in Tables B-2 through B-4. First, the average annual precipitation was calculated from rain gages (Table B-2) and used to calculate the volume of stormwater that infiltrates into a UIC (Table B-3) by Equation (B.8). Next, an average TOC concentration was calculated on the basis of stormwater samples collected at UICs in Clackamas County (Water Environment Services [WES], and the Cities of Gresham, Canby, Portland ,and Milwaukie (Table B-4). Because TOC concentration in stormwater varies during the year (i.e., highest during leaf fall in the fall and lowest during winter), a weighted TOC concentration was calculated. Data from multiple jurisdictions were used to obtain data from the entire year. The average TOC concentration was used to calculate the grams of carbon added to the unsaturated zone surrounding the UIC during a 10-year period by Equation (B.9), mass of organic carbon per unit volume of material surrounding the UIC (ρ_{oc}) by Equation (B.10), and convert ρ_{oc} to f_{oc} by Equation (B.11) (Table B-5).

2.3 Pollutant Properties

Pollutant properties include the organic carbon partitioning coefficient, distribution coefficient, degradation rate constant/half life, and retardation factor.

2.3.1 Organic Carbon Partitioning Coefficient (K_{oc})

The organic carbon partitioning coefficient (K_{oc}) is pollutant specific, and governs the degree to which the pollutant will partition between the organic carbon and water phases. Higher K_{oc} values indicate that the pollutant has a higher tendency to partition in the organic carbon phase, and lower K_{oc} values indicate that the pollutant will have a higher tendency to partition in the water phase.

K_{oc} was assigned differently for PCP and other organic pollutants, according to the following criteria:

- **PCP.** The K_{oc} for PCP is pH dependent, so K_{oc}s for the average and reasonable maximum scenarios were estimated on the basis of the pH. At 12 groundwater wells completed in shallow groundwater of the Unconsolidated Sedimentary Aquifer (which includes the Qfc), pH has been measured within the City of Portland from 1997 to 2007. The maximum values at each well were average (pH = 6.6) and used to calculate a Koc of 703 L/Kg. Because the K_{oc} for PCP decreases at pH increases, using maximum pH is conservative.

- **All Organic Pollutants except PCP.** For the average scenario, K_{oc} was estimated from empirical regression equations relating K_{oc} to the octanol water partitioning coefficient (K_{ow}) and/or pollutant solubility. For the reasonable maximum scenario, K_{oc} was assumed to be either the lowest-reported scientific literature value or the K_{oc} calculated by empirical equations, whichever was lower (i.e., more conservative).

2.3.2 Distribution Coefficient (K_d)

For organic pollutants, the distribution coefficient, K_d, was estimated from the following equation (e.g., Watts, 1998):

$$K_d = f_{oc}K_{oc} \tag{B.12}$$

For metals, K_d was estimated from equations in Bricker (1998). The most important solid phases for sorption of metals in environmental porous media are clays, organic matter, and iron/manganese oxyhydroxides (Langmuir et al., 2004). The distribution of a trace metal between dissolved and sorbed phases is described by the following equation:

$$K_d = \frac{C_s}{C_w} \tag{B.13}$$

where:

C_s is the concentration of the metal adsorbed on the solid phase (M/L^3), and
C_w is the dissolved concentration (M/L^3).

The value of K_d for metals can depend on a number of environmental factors, including the nature and abundance of the sorbing solid phases, dissolved metal concentration, pH, redox conditions, and water chemistry. Measured K_d values for a given metal range over several orders of magnitude depending on the environmental conditions (Allison and Allison, 2005). Therefore, site-specific K_d values are preferred for metals over scientific literature-reported K_ds. K_d values can be determined empirically for a particular situation from Equation (B.13) (Bricker, 1998). The partitioning coefficients were estimated from total and dissolved lead concentrations and total suspended solids (TSS) data in stormwater collected in 2012 at UICs owned by the City of Milwaukie. Sorbed concentrations were calculated by normalizing the particulate lead concentrations to the concentration of TSS. For each sample, an apparent K_d value was calculated for lead from the following equation:

$$K_d = \frac{\left([Me]_t - [Me]_d\right)}{[Me]_d \times TSS} \times 10^6 \tag{B.14}$$

where:

$[Me]_t$ is total lead concentration (M/L^3), and
$[Me]_d$ is dissolved lead concentration (M/L^3)

Note that in Equation (B.14), lead concentration is in micrograms per liter, and TSS is in units of milligrams per liter.

Although the K_ds are determined from systems containing lower concentrations of sorbing particle surfaces than is typical of stormwater infiltrating through a soil column, this is considered to be conservative because (1) the low levels of suspended solids in the stormwater may result in nonlinear sorption regime, in which case calculated K_d values may be significantly lower than would be expected in a higher surface area environment (i.e., the unsaturated zone), and (2) site-specific K_ds calculated in the stormwater already account for the effect of dissolved organic carbon, which could lower apparent K_d values by complexing with trace metals, and thereby shifting the partitioning to the solution. The K_d for lead was 1,200,000 liters per kilogram (L/Kg) for the average scenario and 535,000 L/Kg for the reasonable maximum scenario. This K_d is consistent with K_d values calculated for lead by the City of Portland (GSI, 2008) and City of Gresham (GSI, 2011) using stormwater data at each City's UICs and Equation (B.14).

2.3.3 Degradation Rate Constant (*k*) and Half Life (*h*)

Degradation rate is a chemical-specific, described by a first-order rate constant, and depends on whether the unsaturated zone is aerobic or anaerobic. The organic pollutants evaluated in the unsaturated zone GWPD are biodegradable under aerobic conditions (Aronson et al., 1999; MacKay, 2006); therefore, it is expected that these compounds will biodegrade to some extent within the unsaturated zone after discharging from the UIC. Metals are not discussed in this section because they do not undergo biodegradation.

Aerobic biodegradation rate constants were compiled from a review of the scientific literature, including general reference guides as well as compound-specific studies. The review included degradation in soils, surface water, groundwater, and sediment. Soil aerobic degradation rates were considered to be most representative of UIC field conditions and these are summarized for each of the compounds of interest. First-order rate constants are generally appropriate for describing biodegradation under conditions where the substrate is limited and there is no growth of the microbial population (reaction rate is dependent on substrate concentration rather than microbial growth). Because of the low concentrations of the organic pollutants detected in stormwater, it is appropriate to consider biodegradation as a pseudo-first-order rate process for the UIC unsaturated zone scenario.

The ranges of biodegradation rates representative of conditions expected to be encountered in the unsaturated zone beneath UICs are summarized in Table B-6. Summary statistics provided in Table B-6 include number of measurements, minimum, maximum, mean, and 25th and 50th percentile (median) values. For the average scenario, the median biodegradation rate (benzo(a)pyrene and DEHP) or 10 percent of the average biodegradation rate (PCP) was used. For the reasonable maximum, the 25th percentile biodegradation rate (benzo(a)pyrene and DEHP) or the minimum biodegradation rate (PCP) was used.

The half-life of a pollutant is the time required for pollutant concentration decline to one half of its initial value. Half-life is calculated by the following formula:

$$h = \frac{\ln(2)}{k} \qquad \text{(B.15)}$$

where:

k is the first-order rate constant (T^{-1}), and
h is the half-life (T)

2.3.4 Retardation Factor (*R*)

The retardation factor, *R*, is the ratio between the rate of pollutant movement and the rate of pore water movement. For example, a retardation factor of 2 indicates that pollutants move twice as slow as pore water. The retardation factor is estimated by equation 9.14 of Freeze and Cherry (1979):

$$R = 1 + \frac{(\rho_b)(K_d)}{\eta} \tag{B.16}$$

where:

ρ_b is soil bulk density (M/L^3),
K_{oc} is the organic carbon partitioning coefficient (L^3/M),
f_{oc} is fraction organic carbon (dimensionless), and
η is total porosity (dimensionless).

3. Governing Equation for Unsaturated Zone GWPD

A 1-dimensional (1-D) pollutant fate and transport equation was used to estimate the magnitude of pollutant attenuation during transport through the unsaturated zone. This constant source Advection-Dispersion Equation (ADE) incorporates adsorption, degradation (biotic and abiotic), and dispersion to estimate pollutant concentration at the water table (e.g., Watts, 1998). This equation is provided below:

$$\frac{C(y,t)}{C_0} = \frac{1}{2}\left[\left(e^{A_1}\right)erfc(A_2) + \left(e^{B_1}\right)erfc(B_2)\right] \tag{B.17}$$

where:

$$A_1 = \left(\frac{y}{2D'}\right)\left(v' - \sqrt{(v')^2 + 4D'k'}\right)$$

$$A_2 = \frac{y - t\sqrt{(v')^2 + 4D'k'}}{2\sqrt{D't}}$$

$$B_1 = \left(\frac{y}{2D'}\right)\left(v' + \sqrt{(v')^2 + 4D'k'}\right)$$

$$B_2 = \frac{y + t\sqrt{(v')^2 + 4D'k'}}{2\sqrt{D't}}$$

$$v' = \frac{v}{R}$$

$$D' = \frac{D}{R}$$

$$k' = \frac{k}{R}$$

and:

y is distance in the vertical direction (L),
v is average linear pore water velocity (L/T),

D is the dispersion coefficient (L^2/T),
R is the retardation factor (dimensionless),
k is the first-order degradation constant (T^{-1}),
t is average infiltration time (T),
C_0 is initial pollutant concentration (M/L^3),
$C(y, t)$ is pollutant concentration at depth y and time t (M/L^3), and
erfc is complementary error function used in partial differential equations

Equation (B.17) is an exact solution to the 1-D ADE. The exact solution can be used for both short (i.e., less than 3.5 meters) and long transport distances (more than 35 meters; Neville and Vlassopoulos, 2008). An approximate solution to the 1-D ADE also has been developed, and can be used only for long transport distances. The unsaturated zone GWPD uses the exact solution to the ADE.

With the exception of infiltration time (t), the input parameters were described in Section 2. Infiltration time is the length of time during the year that stormwater infiltrates into a UIC and, therefore, migrates downward through the unsaturated zone. Because stormwater infiltrates into UICs only when the precipitation rate exceeds a threshold value, the infiltration time is dependent on the occurrence of rain events equal to or greater than this amount. The Oregon Department of Environmental Quality (DEQ) (DEQ, 2005) permit fact sheet for the City of Portland UIC Water Pollution Control Facilities (WPCF) permit assigns a threshold precipitation rate of 0.08 inch/hour for stormwater to infiltrate into UICs. The unsaturated zone GWPD conservatively assumes that stormwater infiltrates into UICs at one-half of the threshold precipitation rate (i.e., 0.04 inch/hour). Precipitation and infiltration times from 1998 to 2012 in the City of Portland are shown in Table B-2.

The key assumptions in applying this equation include:

- Transport is 1-D vertically downward from the bottom of the UIC to the water table.
- The stormwater infiltration rate into the UIC is constant and maintains a constant head within the UIC to drive the water into the unsaturated soil.
- Pollutant concentrations in water discharging into the UIC are uniform and constant throughout the period of infiltration.
- The pollutant undergoes equilibrium sorption (instantaneous and reversible) following a linear sorption isotherm.
- The pollutant is assumed to undergo a first-order transformation reaction involving biotic degradation.
- The pollutant does not undergo transformation reactions in the sorbed phase (i.e., no abiotic or biotic degradation).
- There is no portioning of the pollutant to the gas phase in the unsaturated zone.
- The soil is initially devoid of the pollutant.

The unsaturated zone GWPD provides a conservative simulation of pollutant fate and transport for the following reasons:

- In the model, pollutant concentrations are higher than what typically is observed in stormwater. For example, the concentration of PCP (the most mobile and persistent of the common stormwater pollutants) in the model is higher than any of the PCP concentrations observed during 7 years of stormwater discharge monitoring (more than 1,400 stormwater samples) in the City of Portland. The PCP concentration is also 10 times higher than the U.S. Environmental Protection Agency's (EPA) maximum contaminant level (MCL).

- The model does not include pre-treatment upstream of the UIC (e.g., attenuation caused by processes in the sedimentation manhole, vegetated facilities, etc.).

- The model does not take into account pollutant attenuation that occurs while in the UIC (i.e. through adsorption to sediment or organic matter that falls out of solution, or volatilization as water cascades into the UIC from the end-of-pipe) before entering the surrounding soil. The model also does not take into account filtering of pollutants that are sorbed to particulates during transport through the unsaturated zone.

- The model uses conservative parameters for estimating pollutant attenuation. For example, the first-order rate constant for PCP (which governs pollutant attenuation by microbial activity) is 10 percent of the average of scientific literature values.

- Pollutant attenuation is a directional process that occurs in three dimensions. However, the unsaturated zone model simulates pollutant attenuation in only one dimension, which underestimates pollutant attenuation.

- At a typical vertical UIC, most stormwater infiltrates horizontally through the weep holes in the sides of the UIC several feet above the UIC bottom, and then migrates vertically downward. The models assume that stormwater flows only vertically downward from the bottom of the UIC, thereby underestimating the travel distance of stormwater through the unsaturated zone.

- In reality, stormwater flows are highly variable and short in duration, resulting in varying water levels within the UIC depending on the infiltration capacity of the formation. Thus, the UIC periodically will fill with water and then drain during the wet season. The model assumes pollutant fate and transport occurs constantly for the time period during the wet season that the UIC likely contains water. This approach is conservative because it minimizes attenuation by microbial activity, and maximizes the infiltration that would be expected to reach the water table.

- Pollutant concentrations are assumed to be constant, while in reality they are variable throughout storm events. This likely over-predicts the concentration throughout the duration of a storm event.

4. Unsaturated Zone GWPD Results

The unsaturated zone GWPD model input, calculations and results are provided in Table B-7.

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Table B-1

Hydraulic Conductivity Estimates from Specific Capacity Tests at Water Wells

City of Canby

OWRD Well Log ID	Well Depth (ft)	Saturated Gravel Thickness b (ft)	Pumping Rate Q (gpm)	Test Duration t (hrs)	Drawdown s (ft)	Hydraulic Conductivity K_H (ft/day)	Hydraulic Conductivity K_V (ft/day)
Catastrophic Flood Deposits (Qfc)							
9668	42	21	30	1	10	38.2	0.38
9679	47	13	50	1	24	42.8	0.43
9689	50	28	30	3	14	20.5	0.20
9695	40	26	30	2	15	20.6	0.21
9696	46	23	30	3	10	34.9	0.35
12060	47	13	25	1	41	12.5	0.13
12879	50	35	50	0	20	19.1	0.19
12880	40	20	40	3	10	53.5	0.53
					Median (ft/day)	**27.72**	**0.27**
					Median (m/day)	**8.45**	**0.083**
					95% UCL (ft/day)	**47.98**	**0.48**
					95% UCL (m/day)	**14.6**	**0.15**

Notes:

ft = feet

ft/day = feet per day

gpm = gallons per minute

hrs = hours

m = meters

UCL = upper confidence limit

Table B-2

Precipitation, 1998 - 2012

City of Canby, Oregon

Year	Precipitation exceeding 0.04 inch/hour (inches)	Precipitation (feet)	Hours With ≥ 0.04 inches/hr intensity (hours)	Days with ≥ 0.04 inches/hr intensity (days)
2012	46.89	3.9	561.0	23.4
2011	32.51	2.7	411.7	17.2
2010	36.87	3.1	460.7	19.2
2009	23.16	1.9	291.0	12.1
2008	20.21	1.7	270.0	11.3
2007	22.03	1.8	303.3	12.6
2006	29.30	2.4	379.7	15.8
2005	24.33	2.0	307.7	12.8
2004	15.22	1.3	204.7	8.5
2003	24.76	2.1	331.0	13.8
2002	14.00	1.2	181.7	7.6
2001	15.08	1.3	218.3	9.1
2000	16.87	1.4	244.3	10.2
1999	29.62	2.5	422.0	17.6
1998	20.02	1.7	243.7	10.2
Maximum	*46.9*	*3.91*	*561*	*23.4*
Minimum	*14.0*	*1.17*	*182*	*7.6*
Average	*24.7*	*2.06*	*322*	*13.4*
Median	*23.2*	*1.93*	*303*	*12.6*
Geomean	***23.3***	***1.94***	***307***	***12.8***

Notes

Data is from rain gages at 2033 SE Harney Street in Milwaukie; Oregon; the Sylvania PCC Rain Gage at 12000 SW 49th Avenue in Lake Oswego, Oregon; and the Riverdale High School Rain Gage at 9806 SW Boones Ferry Drive. Hourly rainfall amounts for input into the model were calculated by averaging data from the three rain gages. These rain gages were used because they are the closest gages to Canby with hourly precipitation data. Data are available on-line from the City of Portland HYDRA rainfall network.

Table B-3

Unsaturated Zone GWPD Stormwater Infiltration Volume

City of Canby

Impervious Area, *A* (ft^2)	**Annual Precipitation, *P*** (Geometric Mean, 1998 - 2012) (ft/yr)	**Evaporative Loss Factor, *e*** (-)	**Infiltration Volume, *I*** (ft^3/yr)	**Infiltration Volume, *I*** (cm^3/yr)
20,832	1.94	0.24	30,773 (1)	8.71E+08 (1)

Notes

(1) Calculated by the following equation from Snyder (1994): $I = (A)(P)(1-e)$

ft = feet

cm = centimeters

yr = year

GSI Water Solutions, Inc.

Table B-4

Total Organic Carbon in Stormwater

City of Canby, Oregon

		TOC Concentrations				**Average Scenario** (calculated using mean TOC)			**Reasonable Maximum Scenario** (calculated using minimum TOC)		
Time Period	Months	N	Min (mg/L)	Max (mg/L)	Mean (mg/L)	Weighting		Weighted Mean TOC (mg/L)	Weighting		Weighted Mean TOC (mg/L)
Fall	Oct, Nov (1)	15	3.1	55.4	20.5	2 / 9	22%	8.21	2 / 9	22%	1.44
Winter	Dec, Jan, Feb, Mar (2)	66	0.25	9.7	2.6	4 / 9	44%		4 / 9	44%	
Spring	Apr, May, June (3)	27	1.9	23.8	7.6	3 / 9	33%		3 / 9	33%	

Notes

(1) Data from Clackamas County WES

(2) Data from City of Gresham and Canby

(3) Data from City of Portland and City of Milwaukie

mg/L = milligrams per liter

TOC = total organic carbon

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Table B-5

Unsaturated Zone GWPD Fraction Organic Carbon

City of Canby

	CL Calculation					*SV Calculation*					ρ_{oc} *Calculation*	f_{oc} *Calculation*	
	Infiltration Volume (cm^3/yr)	**Carbon Concentration** (mg TOC/1000 cm^3)	**Time** (years)	**Conversion Factor for ug to g**	**CL**	**UIC radius** (cm)	**Radius of Carbon Accumulation + UIC radius** (cm)	**3' Above base volume** (cm^3)	**5' Below base volume** (cm^3)	**Total Volume** (cm^3)	ρ_{oc} (g TOC per cm^3 soil)	**Bulk Density** (g/cm^3)	f_{oc}
Average Scenario	8.71E+08	8.21	10	1,000,000	71,532	60.96	91.44	1,333,723	4001170.42	5,334,894	0.013408278	1.79	0.007435
Reasonable Maximum Scenario	8.71E+08	1.44	10	1,000,000	12,548	60.96	91.44	1,333,723	4001170.42	5,334,894	0.00235206	1.79	0.001312

Notes

TOC = total organic carbon

cm = centimeters

mg = milligrams

ug = micrograms

g = grams

yr = year

Average scenario uses the average TOC concentration, reasonable maximum scenario uses the minimum TOC concentration

Horizontal UIC calculations assume 1 feet of radial transport for TOC accumulation

Equations:

$$CL = (I)(C)(t)\left(\frac{1 \text{ liter}}{1{,}000 \text{ cm}^3}\right)\left(\frac{1 \text{ gram}}{1{,}000 \text{ milligrams}}\right)$$

$$\rho_{oc} = \frac{CL}{SV}$$

$$f_{oc} = \frac{\rho_{oc}}{\rho_b + \rho_{oc}}$$

CL = Organic carbon loaded into the unsaturated zone beneath a UIC during a 10-year period

I = Average annual stormwater infiltration volume

C = TOC concentration in stormwater

t = time of carbon loading

ρ_{oc} = Organic carbon weight per unit unsaturated zone material volume

SV = material volume into which the organic carbon would accumulate because of filtration and adsorption

f_{oc} = fraction organic carbon

ρ_b = bulk density

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Table B-6

Unsaturated Zone GWPD Biodegradation Rates

City of Canby

Compound	First-Order Biodegradation Rate (day^{-1})					
	N	*Median*	*Mean*	*Maximum*	*25 th percentile*	*Minimum*
Benzo(a)pyrene [1]	38	0.0013	0.0021	0.015	0.00026	ND
Di-(2-ethylhexyl)phthalate [2]	34	0.015	0.021	0.082	0.01	0.004
PCP [3]	10	0.206	0.221	0.361	0.1695	0.139

Notes

[1] Rate constants under aerobic conditions in soil were compiled from Aronson et al. (1999) Ashok et al. (1995); Bossart and Bartha (1986); Carmichael and Pfaender (1997); Coover and Sims (1987); Deschenes et al. (1996); Grosser et al. (1991); Grosser et al. (1995); Howard et al. (1991); Keck et al. (1989); Mackay et al. (2006); Mueller et al. (1991); Park et al. (1990); and Wild and Jones (1993).

[2] From Dorfler et al. (1996); Efroymson and Alexander (1994); Fairbanks et al. (1985); Fogel et al. (1995); Maag and Loekke (1990); Mayer and Sanders (1973); Ruedel et al. (1993); Schmitzer et al. (1988); Scheunert et al. (1987) and Shanker et al. (1985).

[3] From Schmidt et al. (1999) and D'Angelo and Reddy (2000)

Table B-7. Pollutant Fate and Transport

Groundwater Protectiveness Demonstration, City of Canby

	Parameter	Symbol	Units	Metals: Lead – Average Scenario	Metals: Lead – Reasonable Maximum Scenario	PAHs: Benzo(a)pyrene – Average Scenario	PAHs: Benzo(a)pyrene – Reasonable Maximum Scenario	SVOCs: PCP – Average Scenario	SVOCs: PCP – Reasonable Maximum Scenario	SVOCs: di-(2-ethylhexyl) phthalate – Average Scenario	SVOCs: di-(2-ethylhexyl) phthalate – Reasonable Maximum Scenario
UIC Properties	Distance Needed to Reach MRLs	y	m	0.00245	0.0095	0.00107	0.0106	0.43	3.62	0.0217	0.2128
		y	ft	0.00803	0.031	0.00352	0.03475	1.39	11.87	0.071	0.70
	Concentration	C_0	mg/L	0.50 [1]	0.50 [1]	0.002 [1]	0.002 [1]	0.01 [1]	0.01 [1]	0.06 [1]	0.06 [1]
	Infiltration Time	t	d	12,800 [2]	12,800 [2]	12.8 [3]	12.8 [3]	12.8 [3]	12.8 [3]	12.8 [3]	12.8 [3]
Pollutant Properties	First-Order Rate Constant	k	d^{-1}			1.30E-03 [4]	2.60E-04 [5]	2.21E-02 [6]	1.39E-02 [7]	1.50E-02 [4]	1.00E-02 [5]
	Half-Life	h	d			533.2 [8]	2666.0 [8]	31.4 [8]	49.9 [8]	46.2 [8]	69.3 [8]
Physical and Chemical Soil Properties	Soil Porosity	η	-	0.325 [9]	0.325 [9]	0.325 [9]	0.325 [9]	0.325 [9]	0.325 [9]	0.325 [9]	0.325 [9]
	Soil Bulk density	ρ_b	g/cm^3	1.79 [10]	1.79 [10]	1.79 [10]	1.79 [10]	1.79 [10]	1.79 [10]	1.79 [10]	1.79 [10]
	Fraction Organic Carbon	f_{oc}	-			0.0074 [11]	0.00130 [11]	0.0074 [11]	0.00130 [11]	0.0074 [11]	0.00130 [11]
	Organic Carbon Partition Coefficient	K_{oc}	L/kg			282,185 [12]	282,185 [12, 13]	703 [14]	703 [14]	12,200 [12]	12,200 [12, 13]
	Distribution Coefficient	K_d	L/kg	1,203,704 [15]	535,040 [16]	2,098 [17]	367 [17]	5.2 [17]	0.9 [17]	90.7 [17]	15.9 [17]
	Effective Porosity	η_e	-	0.20 [18]	0.20 [18]	0.20 [18]	0.20 [18]	0.20 [18]	0.20 [18]	0.20 [18]	0.20 [18]
	Pore Water Velocity	v	m/d	0.42 [19]	0.73 [19]	0.42 [19]	0.73 [19]	0.42 [19]	0.73 [19]	0.42 [19]	0.73 [19]
Calculations	Retardation Factor	R	-	6,625,003	2,944,779	11,548	2,020	29.8	6.0	500	88
	Dispersion Coefficient	D	m^2/d	5.17E-05	3.47E-04	2.27E-05	3.87E-04	8.98E-03	1.32E-01	4.59E-04	7.77E-03
	Normalized Dispersion	D'	m^2/d	7.80E-12	1.18E-10	1.96E-09	1.91E-07	3.02E-04	2.19E-02	9.18E-07	8.80E-05
	Normalized Velocity	v'	m/d	6.38E-08	2.48E-07	3.66E-05	3.61E-04	1.42E-02	1.21E-01	8.45E-04	8.27E-03
	Normalized Degradation	k'	d^{-1}	0.00E+00	0.00E+00	1.13E-07	1.29E-07	7.42E-04	2.31E-03	3.00E-05	1.13E-04
	A_1	-	-	0.00E+00	0.00E+00	-3.30E-06	-3.77E-06	-2.22E-02	-6.87E-02	-7.71E-04	-2.91E-03
	A_2	-	-	2.58E+00	2.58E+00	1.91E+00	1.91E+00	1.95E+00	1.94E+00	1.59E+00	1.59E+00
	e^{A1}	-	-	1.00E+00	1.00E+00	1.00E+00	1.00E+00	9.78E-01	9.34E-01	9.99E-01	9.97E-01
	$erfc(A_2)$	-	-	2.64E-04	2.64E-04	7.02E-03	7.04E-03	5.70E-03	5.97E-03	2.43E-02	2.42E-02
	B_1	-	-	2.00E+01	2.00E+01	2.00E+01	2.00E+01	2.00E+01	2.01E+01	2.00E+01	2.00E+01
	B_2	-	-	5.16E+00	5.16E+00	4.86E+00	4.86E+00	4.89E+00	4.89E+00	4.75E+00	4.75E+00
	e^{B1}	-	-	4.85E+08	4.85E+08	4.85E+08	4.85E+08	4.96E+08	5.20E+08	4.86E+08	4.87E+08
	$erfc(B_2)$	-	-	2.84E-13	2.84E-13	6.19E-12	6.21E-12	4.89E-12	4.64E-12	1.89E-11	1.88E-11
	Concentration Immediately Above Water Table	C	mg/L	1.00E-04	1.00E-04	1.00E-05	1.00E-05	4.00E-05	4.00E-05	1.00E-03	1.00E-03
	MRL	C	mg/L	1.00E-04	1.00E-04	1.00E-05	1.00E-05	4.00E-05	4.00E-05	1.00E-03	1.00E-03
	Action Level	C	mg/L	5.00E-01 [20]		2.00E-03 [20]		1.00E-02 [20]			

NOTES *(SEE APPENDIX B FOR CITATIONS)*

[1] Equal to the action level in Table 1 of the City of Eugene UIC WPCF permit (lead, pentachlorophenol, benzo(a)pyrene) or the City of Gresham UIC WPCF permit (DEHP).

[2] Infiltration time for lead is 1,000 years (1,000 years at 12.8 days per year = 12.800 days)

[3] Infiltration time is the number of hours (converted to days) during the year that stormwater infiltrates into the UIC. Stormwater infiltration is conservatively assumed to occur when the precipitation rate is $\geq$ 0.04 inches/hour.

[4] Median biodegradation rate from a review of scientific literature (see Table B-4 for references).

[5] 25th percentile biodegradation rate from a review of scientific literature (seeTable B-4 for references).

[6] 10 percent of the average biodegradation rate of PCP under aerobic conditions (see Table B-4 for references).

[7] 10 percent of the minimum biodegradation rate of PCP under aerobic conditions (see Table B-4 for references).

[8] Calculated from the following formula: $C_t = C_0e^{-kt}$, where C_t is concentration at time t, C_0 is initial concentration, t is time, and k is biodegradation rate.

[9] Midrange of porosity for gravel in Freeze and Cherry (Table 2.4, pg. 37, 1979)

[10] Calculated by formula 8.26 in Freeze and Cherry (1979): $\rho_b = 2.65(1-\eta)$.

[11] Estimate of f_{oc} based on loading of TOC in stormwater; see Appendix B text for details.

[12] Calculated from the equation of Roy and Griffin (1985), which relates K_{oc} (soil organic carbon-water partitioning coefficient) to water solubility and K_{ow} (octanol-water partitioning coefficient) as presented in Fetter (1994).

[13] Because the K_{oc}s reported in field studies were all higher than K_{oc}s calculated from K_{ow} (i.e., field-study K_{oc}s were less conservative), the reasonable maximum scenario uses the K_{oc} calculated by Roy and Griffin (1985)

[14] The Koc for PCP is pH-dependent. Soil and groundwater pH are in equilibrium; therefore, soil pH can be estimated from groundwater pH. Ph has been measured at 12 USGS wells screened at or near the water table in Portland on the east side of the Willamette River from 1997 to 2007. The average maximum groundwater pH at the USGS wells is 6.6. The PCP organic carbon partitioning coefficient when pH = 6.6 is 703 L/Kg.

[15] Median K_d for lead, calculated using stormwater analytical data collected by the City of Milwaukie in spring of 2012 and an equation from Brickner (1998)

[16] 10th percentile K_d for lead, calculated using stormwater analytical data collected by the City of Milwaukie in spring of 2012 and an equation from Brickner (1998)

[17] K_d calculated from the following equation: $Kd = (f_{oc})(K_{oc})$ (e.g., Watts, pg. 279, 1998).

[18] Within the range of specific yields for gravels from Johnson (1967) (specific yield is approximately equal to effective porosity)

[19] Based on the median (average scenario) or 95% UCL (reasonable maximum scenario) hydraulic conductivity of the USA hydrogeologic unit from analysis of specific capacity tests reported on driller logs (see text for details).

[20] Action Levels from Table 1 of the City of Eugene UIC WPCF permit.

Attachment C – Technical Documentation for the Saturated Zone GWPD

This attachment provides technical documentation of the methods used to delineate a waste management area (WMA) for Underground Injection Control (UIC) devices in the City of Canby. A WMA is the "area where waste or material that could become waste if released to the environment, is located or has been located" (Oregon Administrative Rules [OAR] 340-040-0010(19)). In the context of stormwater infiltration from a UIC, the WMA is the location where groundwater contains stormwater pollutants above background levels (the method reporting limit [MRL]). The waste management area will be used as saturated zone Groundwater Protectiveness Demonstration (GWPD) to demonstrate that UICs are protective of water wells in accordance with the July 2012 draft UIC Water Pollution Control Facilities (WPCF) permit template (the permit template).

1. Introduction

Pollutant fate and transport from a typical wet foot UIC was simulated with a transient 3-dimensional (3-D) finite difference numerical model for groundwater flow and pollutant fate and transport. The UIC was simulated as an injection well that infiltrates stormwater into the aquifer during a 35-year period. Pollutant infiltration was simulated only during years 3 to 35 (32 years total) so that the hydraulics associated with the transient injection simulations stabilized before pollutant injection began. Pollutant concentrations were estimated directly downgradient of the UIC in the direction of groundwater flow.

Pollutant fate and transport are simulated for organic pollutants pentachlorophenol (PCP); di(2-ethylhexyl)phthalate (DEHP); benzo(a)pyrene; and the metal lead. These pollutants are detected regularly in stormwater, and of those, are among the most mobile, toxic, and/or environmentally persistent in their respective chemical classes (GSI, 2008), and are the most likely pollutants in their respective chemical classes to exceed regulatory standards for stormwater at UICs (Kennedy/Jenks, 2009).

The pollutant fate and transport modeling conservatively estimates pollutant fate and transport so that it can be applied to all UICs with less than the protective vertical separation distance established by the unsaturated zone GWPD (i.e., see Attachment B; the protective vertical separation distance is 2.5 feet). The conservative modeling assumptions for the saturated zone GWPD included the following:

- The UIC was assumed to discharge directly to groundwater.
- Pollutant concentrations downgradient of the UIC were measured directly downgradient of the direction of groundwater flow, which is where the highest concentrations occur.

- Groundwater flow direction was constant and did not exhibit seasonal changes, which underestimates dilution of the pollutants (i.e., because seasonal changes in groundwater flow direction increase the volume of the mixing zone between UIC discharges and groundwater).

- The input concentration for PCP (the driver for determining the WMA) was equal to the action level in the permit template, which is greater than any observed PCP concentration from stormwater sampling at UICs in Kennedy/Jenks (2009).

- Pollutant transport and aquifer parameters were selected as averages based on field studies.

- Stormwater infiltration was assumed to occur when the rainfall intensity was equal to or exceeded 0.04 inch per hour, which is half of the intensity threshold of 0.08 inch per hour for runoff to occur (DEQ, 2005b).

2. Saturated Zone Groundwater Protectiveness Demonstration Modeling

The following model runs were conducted as a part of the saturated zone GWPD:

- **Base Model.** A single model run (i.e., "base model") was conducted using input parameters based on average conditions to represent the central tendency or expected mean value of the input parameter. The objective of the base model run was to determine which pollutant (i.e., PCP, DEHP, lead, or benzo(a)pyrene) travels the farthest from the UIC and, therefore, would be used to identify the WMA.

- **Sensitivity Analyses.** Additional model runs were conducted to evaluate the sensitivity of model results to input parameter values. Sensitivity analyses were conducted only for PCP, which was determined to travel the farthest from the UIC in the base model (i.e., because it is the most persistent and mobile of the pollutants that were modeled).

2.1 Model Software

Model software included a groundwater flow model and a pollutant fate and transport model. Groundwater flow was simulated using the 3-D finite difference U.S. Geological Survey block centered numerical groundwater flow model MODFLOW-2000. MODFLOW divides an aquifer into discrete cubes (known as cells) and solves the groundwater flow equation for groundwater elevation in each cell by minimizing mass balance errors in between the cells. The groundwater model output includes groundwater velocity at each cell. The groundwater flow equation was solved using the Pre Conditioned Conjugant Gradient 2 package (PCG2). The velocities output by MODFLOW are used by the 3-D pollutant fate and transport code MT3D to simulate reactive pollutant transport. Particle advection was simulated using the total variation diminishing (TVD) solution scheme.

Groundwater Vistas version 6.27 (build 17) was used as a pre- and post-processor for model input and output, respectively.

2.2 Model Boundaries

Numerical groundwater models simulate groundwater and pollutant movement over a user-specified area. The edges of the area are called boundaries. Different types of model boundaries are used to create flow conditions that mimic real-world groundwater flow. The upgradient and downgradient model boundaries were assigned constant head boundaries (i.e., groundwater elevation is constant over time). Lateral boundaries were no-flow boundaries oriented parallel to the direction of groundwater flow (i.e., groundwater flows parallel to and does not cross the boundary).

2.3 Spatial and Temporal Discretization

The model is divided into cells (i.e., spatially discretized) and time units (i.e., temporally discretized). Spatial and temporal model discretization is summarized in Table C-1.

The areal extent of the model domain (2,000 feet by 400 feet) was selected to maximize computational efficiency. Trial simulations with a larger model domain (approximately 10,000 feet by 10,000 feet) were conducted to confirm that the areal extent of the 2,000 feet by 400 feet model domain did not affect simulation results. Cell sizes in the area of pollutant transport were chosen based on maintaining a Peclet number of less than 2 to prevent artificial oscillation (Huyakorn and Pinder, 1983). For simulation of pollutant transport, the MT3D time step was chosen to be 10 percent of the MODFLOW time step to achieve a Courant number of 1, which is in the range of zero to 2 necessary to prevent numerical dispersion (Van Ganutchen, 1994). Numerical dispersion is spreading of a pollutant plume caused by interpolation errors in between time steps. Numerical dispersion is undesirable because it is an artifact of the numerical solution scheme (as opposed to dispersion caused by physical properties of the aquifer).

2.4 Model Input Parameters

Model input parameters include aquifer properties and pollutant properties, and are summarized in Table C-2, Table C-3, and Table C-4.

2.4.1 Aquifer Properties

Aquifer properties are hydraulic characteristics of the aquifer that govern groundwater flow, and are summarized in Table C-2. Based on a geologic map from the Oregon Department of Geology and Mineral Industries (DOGAMI), the City's UICs are located in the coarse-grained facies of the catastrophic Missoula Flood Deposits (unit Qfc). The aquifer properties used in the saturated zone GWPD are representative of hydrogeologic conditions in the Qfc.

Hydraulic Gradient

Hydraulic gradient is the slope of the water table. Hydraulic gradient (0.011 foot/foot) was calculated on the basis of the groundwater elevation contour map for the Qfc in Canby. Methods for developing the groundwater elevation contour map are summarized in Attachment A.

Hydraulic Conductivity

Hydraulic conductivity describes the ease with which groundwater moves through subsurface soils. Hydraulic conductivities were calculated from specific capacity test data on well logs using the following equations from Driscoll (page 1021, 1986):

$$T = 2000 * \left(\frac{Q}{s}\right) \tag{C.1}$$

$$K = \frac{1}{7.481}\frac{T}{b} \tag{C.2}$$

Where:

T = transmissivity (gallons per day per foot)
Q = pumping rate (gallons per minute)
s = drawdown (feet)
K = hydraulic conductivity (feet per day), and
b = aquifer thickness (feet)

The numbers 2000 and 7.481 in Equations (C.1) and (C.2), respectively, are conversion factors. Hydraulic conductivities in the Qfc are summarized in Table C-3. The median hydraulic conductivity for the Qfc geologic unit is 26.6 feet per day.

Saturated Thickness

Saturated thickness is the portion of a hydrogeologic unit that is saturated with groundwater. The saturated thickness of 47 feet was calculated on the basis of well logs from the OWRD online well log query (OWRD, 2013).

Porosity, Effective Porosity, and Specific Yield

Porosity is a weight-based percentage of void space in a soil. Porosity (0.325) was the midrange for a gravel from Freeze and Cherry (1979) to represent the gravels of the Qfc. The effective porosity and specific yield (0.20) were taken from Johnson (1967). It should be noted that this effective porosity is conservatively lower than the effective porosity of the Qfc gravels reported in other studies farther north, near Portland, Oregon, and Vancouver, Washington (Snyder et al., [1998]; and Snyder [2008] estimated an effective porosity of 0.31 for the Unconsolidated Sedimentary Aquifer (USA), which includes the Qfc unit].

Dispersivity

Dispersivity (α) is related to the spreading of a solute plume as pollutants are transported by groundwater. Solutes spread during transport because some solute particles move faster than the average groundwater flow velocity and other solute particles move slower than the average groundwater flow velocity. The spreading of a solute occurs in three dimensions, and is called dispersion.

Dispersivity is scale-dependent, and increases with increasing pollutant transport distance. The U.S. Environmental Protection Agency (EPA) recommends using the equation of Xu and Eckstein (1995) to calculate a longitudinal dispersivity (i.e., dispersivity parallel to the direction of groundwater flow; EPA, 1996). The calculated longitudinal dispersivity is 17.93 feet, based on a transport distance of 500 feet. Following recommendations in EPA (1996), transverse dispersivity (the horizontal dispersivity perpendicular to longitudinal dispersivity) was set as

33 percent of longitudinal dispersivity, and vertical dispersivity was set as 10 percent of longitudinal dispersivity. The equations used to calculate dispersivity are shown in Table C-3.

Stormwater Infiltration Volume

Calculations for stormwater infiltration volumes are shown in Table C-4. Stormwater infiltration volume was estimated from the following equation (e.g., Snyder, 1994):

$$I = (A)(p)(1-e) \tag{C.3}$$

Where:

- I = Annual stormwater infiltration volume (cubic feet per year)
- A = Impervious area within a UIC drainage basin (square feet)
- p = Precipitation that runs off into the UIC (feet per day)
- e = Evaporative loss factor

Impervious Area (A)

Average impervious area is 20,823 ft^2, based on delineating the impervious area at 14 of the City's UICs. The calculations included the impervious area of driveways that drain into City rights of way.

Precipitation That Runs Off into a UIC (p)

Based on the City of Portland's WPCF permit evaluation report, runoff into a UIC occurs when the rainfall intensity exceeds 0.08 inch per hour (DEQ, 2005b). For the purpose of infiltration calculations, it was conservatively assumed that all precipitation that falls during a storm intensity of greater than or equal to 0.04 inch per hour runs off into UICs. As shown in Table C-4, approximately 1.94 feet of precipitation are produced annually by storm intensities greater than or equal to 0.04 inch per hour. Precipitation data are from 1998 to 2012. Hourly precipitation from three rain gauges in the City of Portland HYDRA rainfall network—2033 SE Harney Street in Milwaukie, 12000 SW 49th Avenue in Lake Oswego, and 9806 SW Boones Ferry Drive in Lake Oswego—were averaged. These rain gauges were used because they were the closest rain gauges to the City of Canby that had hourly precipitation data.

Infiltration Volumes (I)

As shown in Table C-4, the annual infiltration volume in an average UIC drainage basin is estimated to be approximately 30,715 ft^3. The infiltration volumes assumed an evaporative loss factor, e, of 24 percent, which is a recommended value from Snyder et al. (1994).

Stormwater Infiltration Time

Stormwater infiltration time is shown in Table C-4. On average, precipitation intensity is equal to or exceeds 0.04 inch per hour for about 307 hours per year. In the model, the UIC is estimated to discharge the entire year's volume of stormwater runoff during 8 months, with an alternating series of 1-day-long rain events followed by 2-day-long dry periods. This method of inputting runoff into the model produced a reasonable hydraulic head in the UIC during discharge. A simplifying assumption in the modeling was that stormwater discharges were not assumed to occur from June through September.

Fraction Organic Carbon

Fraction organic carbon (f_{oc}) is a dimensionless measure of organic carbon content in a material (i.e., g_{carbon} / g_{soil}). Organic pollutants primarily sorb to organic carbon; therefore, organic pollutant retardation is directly proportional to fraction organic carbon.

Carbon in saturated soil beneath a UIC is derived from two sources:

- Organic carbon incorporated into the soil when the soil is deposited (i.e., "background f_{oc})
- Particulate matter (e.g., degraded leaves, pine needles, pollen, etc.) that is filtered out of stormwater and accumulates in soil adjacent to UICs as stormwater discharges from the UIC.

The model included f_{oc} from both sources.

The background f_{oc} was estimated to be 0.001826 g_{carbon}/g_{soil} based on the average total organic carbon (TOC) in three soil samples that were collected from temporary borings in the USA near the City of Gresham's UIC area (GSI, 2013). The USA hydrogeologic unit includes the Qfc geologic unit and therefore is a reasonable estimate to use for the Qfc geologic unit in the City of Canby.

An estimate of f_{oc} based on accumulation of TOC from stormwater around a UIC by filtration and sorption was determined by calculating the grams of organic carbon added to the saturated zone around the UIC during a 10-year period. The approach also was used to calculate grams of organic carbon added to the unsaturated zone as a part of the City's unsaturated zone GWPD (see Attachment B). The following equations were used in the analysis:

$$I = (A)(p)(1-e) \tag{C.4}$$

$$CL = (t)\left[\sum_{i=1}^{n} I_i C\right] \frac{1\ \text{liter}}{1{,}000\ \text{cm3}} \frac{1\ \text{gram}}{1{,}000{,}000\ \text{milligrams}} \tag{C.5}$$

$$\rho_{oc} = \frac{CL}{SV} \tag{C.6}$$

$$f_{oc} = \frac{\rho_{oc}}{\rho_b + \rho_{oc}} \tag{C.7}$$

Where the variables in Equation (C.4) were identified previously, and:

CL = Organic carbon loaded into the saturated zone beneath a UIC during a 10-year period (grams)

C = TOC concentration in stormwater (milligrams per liter)

t = Time of carbon loading (years)

ρ_{oc} = Organic carbon weight per unit saturated zone material volume (grams per cubic centimeter)

SV = Material volume into which the organic carbon would accumulate because of filtration and adsorption (assumed to be the volume of the grid cell(s) where the UIC is located) (cubic centimeters)

f_{oc} = Fraction organic carbon (g_{carbon}/g_{soil})

ρ_b = Bulk density (grams per cubic centimeter)

Calculation of f_{oc}, based on the filtering of TOC as suspended solids is shown in Table C-5 for the different impervious areas within UIC drainage basins. First, the volume of stormwater that infiltrates into a UIC each month was calculated by Equation (C.4). Next, Equation (C.5) was used to calculate the grams of carbon added to the saturated zone surrounding the UIC during a 10-year period. Equation (C.6) was used to calculate the mass of organic carbon per unit volume of material surrounding the UIC (ρ_{oc}), and Equation (C.7) was used to convert ρ_{oc} to f_{oc}.

2.4.2 Pollutant Properties

Pollutant properties used in the base model are summarized in Table C-6. With the exception of half-life, the data sources for calculating pollutant properties for saturated transport are the same as those used for unsaturated transport (see Attachment B). The wet-feet transport simulations used half-lives that were the midrange of field studies for pollutant degradation in aerobic groundwater from Howard et al. (1991).

3. Results

Results of the saturated zone GWPD model are summarized in Table C-7.

3.1 Base Model

PCP travels farther than the other modeled pollutants because it is more mobile in the environment. Therefore, the WMA for City UICs is conservatively based on PCP. Under the conservative modeling approach that was used for the saturated zone GWPD, PCP requires 267 feet of horizontal transport to attenuate below the MRL. The protective horizontal separation distance for City UICs is 267 feet.

3.2 Sensitivity Analyses

Sensitivity analyses were conducted on hydraulic conductivity and effective porosity:

- **Hydraulic Conductivity.** Hydraulic conductivity was increased and decreased by 1 order of magnitude from the median value calculated from specific capacity tests that was used in the base model (26.6 feet per day [ft/day]). Hydraulic conductivity and WMA are directly proportional. Increasing the hydraulic conductivity 1 order of magnitude to 266 ft/day increases the WMA by 73 feet. Decreasing the hydraulic conductivity by 1 order of magnitude to 2.6 ft/day decreases the WMA by 152 feet. However, the hydraulic gradient is inversely correlated to hydraulic conductivity and would reduce the magnitude of the effect of hydraulic conductivity.

- **Effective Porosity.** Effective porosity was increased to 0.30 from the value of 0.20 used in the base model. Effective porosity and WMA are inversely proportional. Increasing effective porosity to 0.30 decreases the WMA by 62 feet. Snyder et al. (1998) and Snyder (2008) consider 0.31 to be the effective porosity of the Qfc gravels farther north of Canby.

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Table C-1

Model Discretization

City of Canby

Variable	**Reference**
Spatial Discretization	
Horizontal x -extent	2000 feet
Horizontal y -extent	400 feet
Vertical Exent	55 feet
Number of Rows	36
Number of Columns	162
Number of Layers	4
Total Number of Cells	23,328
Cell Size	5 feet to 20 feet
Temporal Discretization	
Simulation Length	35 years (32 years of pollutant loading)
Number of Time Steps	13,140
MODFLOW Time Step Length	1 day
MT3D Time Step Length	0.1 day

Table C-2
Aquifer Properties
City of Canby

Variable	Symbol	Units	Value	Reference
Hydraulic Gradient	h	feet/foot	0.011	Based on Water Table Map for the city of Canby
Hydraulic Conductivity	K_h	feet/day	26.6	Median hydraulic conductivity calculated from well tests available on OWRD well logs in the Missoula Flood Deposits (Qfc)
Anisotropy	$K_h:K_v$	dimensionless	10:1	Freeze and Cherry (1979)
Average Hydrogeologic Unit Thickness	b_{HGU}	feet	80	Based on Qfc thickness in driller logs
Average Depth to Groundwater	DTW	feet bgs	33	Average depth to groundnwater calculated using monitored wells in the area
Average Saturated Thickness	b	feet	47	Calculated from hydrogeologic unit thickness and depth to water
Porosity	η	dimensionless	0.325 (Qfc Deposits)	Midrange of porosity for gravel in Freeze and Cherry (Table 2.4, pg. 37, 1979)
Effective Porosity	η_e	dimensionless	0.20	USGS David Morgan (1996) Upper Sedimentary Unit
Specific Yield	S_y	dimensionless	0.20	USGS David Morgan (1996) Upper Sedimentary Unit
Longitudinal Dispersivity	α_L	feet	17.93	Calculated using Xu and Eckstein (1995). $a_L = (3.28)(0.83)[\log(L_p/3.28)]^{2.414}$. A transport distance ($L_p$) of 500 feet was used in the calculation)
Transverse Dispersivity (y-direction)	α_T	feet	5.92	Calculated using EPA (1986). $a_T = 0.33(a_L)$
Vertical Dispersivity (z-direction)	α_V	feet	1.79	Calculated using EPA (1986). $a_v = 0.10(a_L)$
Fraction Organic Carbon	f_{oc}	dimensionless	0.01360	f_{oc} near UIC due to carbon loading from stormwater. See text for calculations and Table C-5
			0.001826	f_{oc} in native sediments, based on TOC measurements of unconsolidated sediments of the USA in Gresham, Oregon (GSI, 2012), which includes the Qfc

Note:
bgs = below ground surface
USGS = United States Geological Survey
DTW = depth to groundwater
OWRD = Oregon Water Resources Department
EPA = Environmental Protection Agency
Qfc = Coarse grained catastrophic flood deposits
UIC = Underground Injection Control
USA = Unconsolidated Sedimentary Aquifer
TOC = Total Organic Carbon

Table C-3

Hydraulic Conductivity Estimates from Specific Capacity Tests

City of Canby

OWRD Well Log ID	**Well Depth** (ft)	**Aquifer Thickness, b** (ft)	**Pumping Rate, Q** (gpm)	**Test Duration, t** (hrs)	**Drawdown, s** (ft)	**Hydraulic Conductivity, K** (ft/day)
Quaternary/Floodplain Deposits (Qfc)						
CLAC_277	120	34	28	10	6	37
CLAC_12047	98	58	30	3	40	3
CLAC_12049	100	40	30	5	8	25
CLAC_12050	132	52	30	2	40	4
CLAC_12051	300	9	700	4	161	129
CLAC_12051	300	9	650	2	158	122
CLAC_12051	300	9	600	13	104	171
CLAC_12052	237	2	210	24	80	351
CLAC_12052	237	2	110	26	48	306
CLAC_12053	79	49	40	2	10	22
CLAC_12060	47	17	25	1	41	10
CLAC_12061	67	18	35	4	15	35
CLAC_12065	285	14	300	5	81	71
CLAC_12065	285	14	360	2	87	79
CLAC_12067	102	38	25	4	48	4
CLAC_12068	135	33	24	17	10	19
CLAC_12072	125	50	20	1	23	5
CLAC_12073	80	47	30	1	21	8
CLAC_12074	237	23	300	1	50	70
CLAC_12074	237	23	400	1	66	70
CLAC_12074	237	23	500	5	77	75
CLAC_12076	285	12	300	2	76	88
CLAC_12076	285	12	400	4	83	107
CLAC_12076	285	12	500	6	104	107
CLAC_12079	140	12	85	4	55	34
CLAC_12081	620	105	200	33	144	4
CLAC_12089	73	51	45	8	35	7
CLAC_12097	192	45	300	1	45	40
CLAC_12097	192	45	200	1	30	40
CLAC_12100	201	55	100	2	95	5
CLAC_12101	140	6	35	2	130	12
CLAC_12102	110	26	70	4	20	36
CLAC_12103	93	66	50	1	85	2
CLAC_12105	113	38	88	1	22	28
CLAC_12105	113	38	102	1	27	27
CLAC_12105	113	38	128	1	32	28
CLAC_12105	113	38	140	1	37	27
CLAC_12105	113	38	180	4	52	24

Table C-3

Hydraulic Conductivity Estimates from Specific Capacity Tests

City of Canby

OWRD Well Log ID	**Well Depth** (ft)	**Aquifer Thickness, b** (ft)	**Pumping Rate, Q** (gpm)	**Test Duration, t** (hrs)	**Drawdown, s** (ft)	**Hydraulic Conductivity, K** (ft/day)
CLAC_12106	125	75	200	2	32	22
CLAC_12108	112	52	220	2	60	19
CLAC_12108	112	52	180	2	40	23
CLAC_12110	59	38	8	1	28	2
CLAC_12115	132	71	155	4	28	21
CLAC_12115	132	71	200	3	43	18
CLAC_58848	115	39	25	2	8	21
CLAC_12003	40	12	412	2	16	555
CLAC_12025	421	72	403	2	33	45
CLAC_12038	190	34	225	2	59	30
CLAC_9679	47	13	50	2	24	43
CLAC_9679	48	13	30	2	18	34
CLAC_9680	62	17	50	2	31	25
CLAC_9680	62	17	30	2	21	22
CLAC_9680	62	17	20	2	16	20
CLAC_62807	310	101	400	2	105	10
CLAC_12877	90	35	40	2	10	31
CLAC_12880	40	21	40	2	10	51
CLAC_12897	101	34	40	2	29	11
CLAC_12886	81	2	33	2	8	551
CLAC_12872	90	50	30	2	16	10

Table C-4
Infiltration Volume and Rate
City of Canby

Impervious Area in UIC Drainage Catchment (ft^2)	**Infiltration Time** (Annual Number of Hours with Precipitation ≥ 0.04 inches/hour [1]) (hours)	**Infiltration Time** (Annual Number of Days with Precipitation ≥ 0.04 inches/hour [1]) (days)	**Annual Precipitation ≥ 0.04 inches/hour [1]** (ft)	**Annual Infiltration Volume [2]** (ft^3)
20,832	307	12.79	1.94	30,715

Notes
(1) Based on precipitation records from 1998 to 2011. Values calculated using the geometric mean.

(2) Assumes an evaporative loss factor of 24%.

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Table C-5

Saturated Zone GWPD Carbon Loading Calculations

City of Canby

Impervious Area (ft^2)	**UIC Type**	**Annual Infiltration Volume** (cm^3/yr)	**TOC Concentration** (mg/L)	**Time** (years)	**Conversion Factor**	**Grams Carbon Added Over 10 Years** (g)	**Cell Width** (cm)	**Cell Length** (cm)	**Cell Depth** (cm)	**Aquifer Volume** (cm^3)	**g TOC per cm^3/soil** (g/cm^3)	**Bulk Density** (g/cm^3)	f_{oc} (-)
20,832	Vertical UIC	869,743,575	8.21	10.00	1,000,000	74,798	152.39	152.40	152.39	3539260.38	0.02	1.79	0.0117

Notes

mg/L = milligrams per liter

cm^3/yr = cubic centimeters per year

g = grams

cm = centimeters

g/cm^3 = grams per cubic centimeter

Table C-6

Pollutant Properties

City of Canby

Variable	Symbol	Units	Pollutant	Value	Reference
Organic Carbon Partitioning Coefficient	K_{oc}	L/kg	B(a)P	282,185	Calculated by Roy and Griffin (1985), which relates Koc to solubility in water
			PCP	703	The K_{oc} for PCP is pH-dependent. pH has been measured at 6 shallow water wells (30 - 78 feet below ground surface) that are completed in the sand and gravels, Lane County, Oregon (Craner, Table 9, 2006). The average pH at monitoring wells was 6.8. When pH = 6.8, the K_{oc} for PCP is 592 L/kg.
			DEHP	12,200	Calculated by Roy and Griffin (1985), which relates Koc to solubility in water
Distribution Coefficient	K_d	L/kg	Lead	1,000,000	Calculated by the equation of Bricker (1988), which calculates Kd based on concentrations of total metals, dissolved metals, and TSS. See Appendix B.
			B(a)P	515 (Native Sediments) 3,292.8 (Near vertical UIC, reflects loading from stormwater)	Calculated from the relationship: $K_d = (f_{oc})(K_{oc})$ (Watts, 1998)
			PCP	Native Sediments: 1.3 8.2 (Near vertical UIC, reflects loading from stormwater)	Calculated from the relationship: $K_d = (f_{oc})(K_{oc})$ (Watts, 1998)
			DEHP	22.3 (Native Sediments) 142.36 (Near UIC, reflects loading from stormwater)	Calculated from the relationship: $K_d = (f_{oc})(K_{oc})$ (Watts, 1998)
Retardation Factor	R	dimensionless	Lead	5,507,693	Calculated from the relationship: $R = 1 + (\rho_b)(K_d)/(\eta)$. Based on a bulk density (ρ_b) of 1.79 g/cm^3, calculated from porosity using equation 8.26 of Freeze and Cherry (1979).
			B(a)P	2,839 (Native Sediments) 29,471 (Near vertical UIC, reflects loading from stormwater)	Calculated from the relationship: $R = 1 + (\rho_b)(K_d)/(\eta)$. Based on a bulk density (ρ_b) of 1.79 g/cm^3, calculated from porosity using equation 8.26 of Freeze and Cherry (1979).
			PCP	Native Sediments: 6.95 74.4 (Near vertical UIC, reflects loading from stormwater)	Calculated from the relationship: $R = 1 + (\rho_b)(K_d)/(\eta)$. Based on a bulk density (ρ_b) of 1.79 g/cm^3, calculated from porosity using equation 8.26 of Freeze and Cherry (1979).
			DEHP	124 (Native Sediments) 1275.12 (Near UIC, reflects loading from stormwater)	Calculated from the relationship: $R = 1 + (\rho_b)(K_d)/(\eta)$. Based on a bulk density (ρ_b) of 1.79 g/cm^3, calculated from porosity using equation 8.26 of Freeze and Cherry (1979).
Half Life	h	days	B(a)P	587	Based on midrange observed biodegradation rate for B(a)p in aerobic groundwater (Howard et al., 1991)
			PCP	46	Based on observed biodegradation rate for PCP in aerobic groundwater (Howard et al., 1991)
			DEHP	10	Based on observed biodegradation rate for DEHP in aerobic groundwater (Howard et al., 1991)
Input Concentration	C_{AL}	ug/L	Lead	500	
			B(a)P	2	
			PCP	10	
			DEHP	60	

Table C-7

Model Simulation Results

City of Canby

	Horizontal Distance for Pollutants to Attenuate to Below MRL or MCL *(WMA is Based on the MRL)*							
	PCP		DEHP		B(a)P		Lead	
	MRL (0.04 ug/L)	MCL (1.0 ug/L)	MRL (0.962 ug/L)	MCL (6.0 ug/L)	MRL (0.01 ug/L)	MCL (0.2 ug/L)	MRL (0.1 ug/L)	MCL (15.0 ug/L)
Base Model (Used to Delineate WMA)								
Hydraulic Conductivity = 26.6 ft/day Effective Porosity = 0.20	267	53	67	33	33	18	5	< 1
Sensitivity analysis								
High hydraulic conductivity (K=266 ft/day)	340	18						
Low hydraulic conductivity (K= 2.6 ft/day)	115	55						
High effective porosity (n=0.3)	205	47						

Notes:

MRL= Method Reporting Limit

MCL= Maximum Contaminant Level

Appendix D

Detention Pond Hydrologic/Hydraulic Modeling Input - Output/Results

Provided on CD

Appendix E

Fish Eddy Wetland Sizing Model Results

Provided on CD