SECTION III

SECTION III – SYSTEM ANALYSIS

The objective of this section is to evaluate if the water system facilities are capable of supplying sufficient quality and quantity of water to meet existing and projected demands as identified in Section II. Facility performance and design criteria are established as the basis for evaluating existing facilities and proposed improvements. The majority of the standards identified below are based on regulatory requirements and DOH guidance. System specific standards or standards that deviate from recommendations in DOH guidance are referenced where applicable. In addition, City standards will be used where applicable. Water System Standards are addressed in further detail in Section VII.

WATER QUALITY CRITERIA

The City's water quality must comply with the provisions of the federal Safe Drinking Water Act (SDWA). DOH water quality provisions contained in Chapter 246-290 WAC generally incorporate the SDWA requirements. However, due to resource and legal process constraints, the DOH rules tend to lag behind SDWA implementation. Therefore, compliance with the current state rules does not guarantee compliance with all current federal rules. Some of the analysis in this section may evaluate proposed federal rules as well as current state statutes.

WATER DEMAND AND SERVICE CRITERIA

Population and water demand estimates presented in Section II were developed in accordance with the DOH Water System Design Manual (WSDM). The demand projections identified in Section II are utilized to determine minimum requirements for source, storage and distribution capacities. The following is a summary of applicable capacity requirements and minimum standards for service as defined in Chapter 246-290 WAC and the DOH WSDM.

Source and Pumping Capacity

Overall source requirements are dependent upon the capacity and reliability of the source facilities. Source capacity and the capacity of source pumping facilities must equal or exceed Maximum Day Demand (MDD) (WAC 246-290-222). Systems with a single source of supply must be capable of supplying the MDD with standby storage. Equalizing storage must be provided if source capacity is less than the Peak Hour Demand (PHD). Source pumping should not exceed instantaneous withdrawal rights allowed by the Department of Ecology (DOE).

Distribution and Transmission Performance

Minimum design pressures must be maintained under PHD and MDD plus fire flow conditions (WAC 246-290-222). Systems shall be designed to provide a minimum pressure of 30 psi measured at the meter or property line under PHD conditions (WAC 246-290-230). This design pressure must be met at the condition where equalizing storage is depleted. During routine operating conditions, approved design pressures shall be maintained, but in no case shall service pressure be less than 20 psi (WAC 246-290-420). When fire flow is required, 20-psi design pressure must be maintained at the operating hydrant and in all other areas of the distribution system. Positive pressures must be maintained at all points in the distribution under actual fire flow operating conditions. New water mains shall be a minimum of 8-inch for dead end mains and 6-inch diameter for looped mains. Transmission main pressure should be maintained above 5 psi at all times.

Storage Capacity

Storage shall be sufficient for the 6-year planning period and should be designed to meet the requirements for the 20-year planning period. Critical components are operating, standby, equalizing, fire and dead storage as discussed in the following.

- Operating Storage: Operating storage is defined as the volume between the pump on and off settings. This volume will vary according to two main factors: 1) the sensitivity of the water level sensors controlling the source pumps, and 2) the volume required to prevent excessive cycling (starting and stopping) of the pump motor(s).
- Standby Storage: Standby storage provides system demands during emergency conditions. Standby storage is typically sized to equal two times Average Day Demand (ADD) for single source systems. For multiple source systems standby storage is that which provides a margin of system reliability during emergency conditions and is sized to be the greater of 1) twice the average day demand minus the source capacity with the largest source out of service, or 2) 200 gallons/equivalent residential unit.

- Equalizing Storage: Equalizing storage volume is provided to make up any difference between the PHD and the capacity of the sum of all sources. Equalizing storage must be provided in accordance with WAC 246-290-235(2). Equation 9-1 of the WSDM is utilized to calculate required equalizing storage volumes.

  WSDM Equation 9-1: ES = (PHD - Q_S) (150 min.)

  Where: ES = Equalizing storage component, in gallons

  PHD = Peak hourly demand, in gpm-as defined in Chapter5 of the WSDM manual

  Q_S = Sum of all installed and active supply source capacities except emergency supply, in gpm. See Section 9.1.1 of the WSDM for source definitions

- Fire Flow Storage: Fire flow storage must be equal to the greatest required fire flow within the service area multiplied by the required duration. Fire flow storage may be nested within standby storage if allowed by the local fire protection authority. Applicable fire flow, duration and local standards are provided in the discussion of fire flow criteria below.
- Dead Storage: Dead storage is the volume of stored water not available to all consumers at the minimum design pressures. The DOH WSDM Section 9.0.6 states, Dead storage (DS) is the volume of stored water not available to all consumers at the minimum design pressure (WAC 246-290-230(5) and (6)). In the case of Woodland the base elevation of the reservoirs, 160', provides greater than 30 psi to all services within the distribution system except for a few residences near the reservoir that are served by individual booster pumps so there is no dead storage in Woodland's Reservoirs.

DISINFECTION CONTACT TIME

The City of Woodland is required by WAC 246-290-662 to provide continuous disinfection. The level of disinfection required is based upon the removal credit received for the filtration process. The level of inactivation is based upon CT calculations and is based upon chlorine concentration, contact time, pH and temperature. CT values are calculated each day based upon peak hourly flow. For a system serving a population more than 3,300, the residual chlorine concentration of the water entering the distribution system must be monitored continuously and the lowest value recorded each day.

FIRE FLOW CRITERIA

Fire flow requirements are specific to both the water system performance and to specific building projects. The Washington Survey and Rating Bureau (WSRB) classify municipalities according to fire fighting capabilities and

establish guidelines for rating overall fire protection. The insurance rating of a local district is based upon a variety of criteria distributed among three key components. Ten percent (10%) of the rating is based upon Emergency Communications, forty percent (40%) is based upon the reliability of the water system and its ability to provide required fire flows and fifty percent (50%) is based upon the fire department itself.

The WSRB's Public Protection Classification (PPC) rating influences fire insurance premiums. Our current PPC from Washington Survey and Ratings is a 6 (on a scale of 1-10, with 1 being the best and 10 being no fire protection). The current rating is conditional upon proximity of hydrants. If a residential property is more than 1,000 feet, or a commercial is more than 300 feet, the rating is reduced to 8A for insurance purposes. A municipality should request a new rating evaluation when significant upgrades to fire protection capabilities are completed.

Water system performance requirements are either based on statutes or are established by the City Council in the form of performance goals. WAC 246-293-640 sets specific fire flow performance requirements for public water systems. For reference and comparison, these performance requirements are summarized in Table III-1.

Table III-1 Minimum Fire Flow per WAC 246-293-640

Development Classification	Minimum Fire Flow Requirement (*)
Rural	None
Residential	500 gpm for 30 minutes
Commercial and Multi- Family Structures greater than 4,000 sf	750 gpm for 60 minutes (**)
Industrial	1,000 gpm for 60 minutes (**)

(*) Minimum fire flow requirements are in addition to requirements for normal domestic maximum use.
(**) Commercial and industrial building may be subject to higher flow requirements when evaluated on an individual basis by the fire protection authority.

The City's water system design and construction standards identifies a fire flow goal of 1,000 gpm for residential areas, 1,500 gpm for high density residential and commercial and 2,000 gpm for industrial areas. These are higher than the required minimum flows for systems that serve more than 1,000 connections. The City goal for fire flow storage will be 2,000 gpm for 120 minutes.

Site specific fire flow requirements are implemented by the local fire protection authority through the building permit review process. If site specific flow requirements are greater than what is available at the site, it is the responsibility

of the property owner and/or developer to provide the improvements (whether to the system or on-site) that are required for the specific building proposal.

WATER QUALITY ANALYSIS

The City is required to test water quality parameters ranging from monthly coliform samples to sampling inorganic chemicals once every three years or as required by the Water Quality Monitoring Report. The following briefly discusses the various water quality parameters and recent results. A history of water quality sample results is provided in Appendix C. Table III-2 provides a summary of water quality monitoring requirements and results. Future monitoring schedules are addressed in Section VI.

EPA passed the Stage 2 Disinfection Byproducts Rule (DBP) and the Long Term 2 Enhanced Surface Water Treatment Rule (LT2) in 2006. The Stage 2 DBP rule builds on earlier rules that addressed disinfection byproducts to improve drinking water quality and provide additional public health protection from disinfection byproducts. The rule tightened compliance monitoring requirements for two groups of DBP's, trihalomethanes (TTHM) and haloacetic acids (HAA5). The rule is intended to reduce potential cancer and reproductive and developmental health risks from DBP's in drinking water which form when disinfectants are used to control microbial pathogens. The rule requires the water system to meet maximum contaminant levels as an average at each monitoring location instead of system wide as in previous rules. The City submitted the Initial Distribution System Evaluation (IDSE) report to verify the most effective DBP sampling sites and was approved in 2010. The system is considered in compliance with the IDSE reporting requirements. However, the city has not yet submitted a Stage 2 DBP Rule Compliance Monitoring Plan. Approval must be gained prior to the beginning of compliance monitoring fourth quarter 2013.

The purpose of the Long Term 2 Enhanced Surface Water Treatment Rule (LT2) is to reduce illness linked with the contaminant Cryptosporidium and other disease causing microorganisms in drinking water. The rule supplements existing regulations by targeting additional Cryptosporidium treatment requirements to higher risk systems. The rule also contains provisions to reduce risks for uncovered finished water reservoirs and to ensure that systems maintain microbial protection when they take steps to decrease the formation of disinfection byproducts. The rule applies to all public water systems that use surface water or ground water that is under the direct influence of surface water. The rule requires systems to initially monitor their water sources to determine treatment requirements. Filtered water systems will be classified into one of four treatment categories based on their

monitoring results. Systems serving fewer than 10,000 people did not begin monitoring until October 2008. Woodland's conducted source water monitoring in 2008 and 2009 and all tests were absent of total and fecal coliforms. Woodland will need to begin a second round of source water monitoring in 2017. Based on the results of the initial round of sampling this additional monitoring is not expected to trigger the need for additional treatment.

Table III-2 Water Quality Monitoring Summary

Parameter	Monitoring Requirements	Results
Coliform	Monthly coliform samples have been consistently collected in accordance with the regulations.	There have been no positive coliform samples since 2004.
Volatile Organic Chemicals (VOC)	One VOC sample is required between Jan 2011-Dec 2013.	VOC was sampled in October 2012, no exceedances.
Nitrates	Nitrate samples are required annually.	Nitrate samples were collected in October 2012, no exceedances.
Inorganic Chemicals (IOC)	One complete IOC sample is required between Jan 2011-Dec 2019.	The last complete IOC was done in June 2007. IOC is not required per the Water Quality monitoring Report in 2012.
Synthetic Organic Chemicals (SOC)	SOC monitoring requirements are based on DOH rating of the source susceptibility.	Herbicides and Insecticides were both last tested in January of 2001. One Herbicide sample is required between Jan 2011-Dec 2013. A waiver is granted for Insecticides – no sampling is required thru Dec 2013.
Radionuclides	Radionuclide samples are required every four years.	The last samples were taken in June 2010, no exceedances.
Asbestos	DOH policy requires asbestos sampling every nine years if more than 10% of the distribution system consists of AC pipe.	Asbestos was last sampled in October 2010. One asbestos sample is required in 2013.
Disinfection By-Products (DBPs)	Quarterly monitoring is required for TTHMs and HAA5	THM and HAA5 were sampled mid November 2012, results are pending. The September 2012 sample for THM level was 16.6 ug/l and HAA5 level was 6.7 ug/l.
Lead and Copper	LCR 1 set of 20 samples is required on distribution between Jan 2010- Dec 2012.	The last lead/copper samples were taken in June 2012. The 90% for copper was 0.5 mg/l and the 90% for lead was 0.0026 mg/l.

SYSTEM DESCRIPTION AND ANALYSIS

This section provides a description of the general condition and physical capacity of each water system component. The existing capacity is compared to the existing and projected water demands identified in Section II. This analysis, in coordination with other sections of this WSP Update, results in the identification of the extent and timing of facility deficiencies. Each identified deficiency is summarized at the end of this Section. Recommendations are discussed in more detail in the Operations Program in Section VI and Capital Improvement Program in Section VIII. A schematic map of the water system facilities is provided at the end of this chapter.

Source

The City of Woodland owns and operates a Ranney Well on the Lewis River. The Ranney includes intake laterals located east of the Ranney well under the Lewis River. The City is currently using laterals 2, 3B, and 4B. In 2007 laterals 1, 2, 3, 3B, and 4B were cleaned and evaluated to optimize the number of laterals being used. Lateral 1 was found to produce poor quality (black) water and was subsequently closed. Lateral 3 appears to have a break at a distance from the well (about 95 feet out) and is producing sand. This lateral was also closed to prevent sand migration into the well and pumps.

Currently, Laterals 2, 3B and 4B are being used to produce water and they include only 143 lineal feet of screen (total) since the first 49-56 feet of each line is blank pipe. At design entrance velocities of 1-2 feet per minute, this means that the mechanical capacity of the well should be something between 550-1,100 gallons per minute (gpm). The testing following the maintenance work in 2007 indicated that yields up to 3.4 million gallons per day (mgd) or about 2,400 gpm, should be possible, indicating that additional well screen would be needed to match the mechanical capacity to the projected yields. If the well is to produce flows of 3.4 mgd, additional footage of well screen would be required to maintain suitable entrance velocities and extend the interval between required maintenance (screen cleaning and redevelopment) events.

Plugging with precipitated iron has necessitated repeated cleaning and redevelopment of the laterals. The rate of well screen plugging increases when the well is pumped at flow rates above the mechanical capacity of the well. Since its installation, the collector well has required some type of cleaning and redevelopment at a frequency averaging about six years. It is likely that the collector well will continue to require similar maintenance in the future every five years or less, especially in view of the limited lengths of screen that are now available for developing the raw water supply.

Because the potential yield of the collector well exceeds its present mechanical capacity, it is recommended to install additional laterals in this well. Laterals with larger diameter wire-wrapped stainless-steel well screens could be installed that have a larger percentage of open area which can help lower screen entrance velocities. Newer pipe projection procedures (since the 1980's) allow better elevation control over the horizontal orientation of the laterals than was possible during the original construction of the well. To construct the additional laterals the work will be done in the winter when the Ranney Well only pumps water for about 8 hours a day. In addition a generator

should be installed at the Ranney Well. With three pumps and a dedicated generator, the Ranney Well should be considered as a multiple source for WSDOH design storage calculations. Based upon general assumptions, the budget cost to install 2-3 new lateral well screens is $ 750,000 and a new generator along with associated electrical improvements is $250,000. To complete this work the City secured a $973,000 Public Works Trust Fund (PWTF) in 2012. Combined with a City Match this is sufficient funding to complete this project in 2013 and 2014.

As the first step for this project, two exploratory borings will be installed in the vicinity of the collector well to identify potential aquifer zones suitable for installation of the lateral screens. One or more hydraulic interval pumping test will be performed on zones within the aquifer identified during the drilling to differentiate the suitability between the zones with respect to water quality and gradation to optimize future well performance. The cost of this project is around $40,000 and the City has hired Layne to complete this project in December of 2012. To help pay for this study the City received $12,000 in Rural Development Funds from Cowlitz County.

In 2007 the City completed electrical improvements and installed three 100 HP vertical turbine pumps with variable frequency drives of which two can operate at the same time. Each pump has a capacity of 1,050 gpm at 303' of total dynamic head. These pumps deliver water through approximately 4,200 feet of 12-inch main to the water treatment plant which is located adjacent to the storage reservoirs. The 12-inch main was installed as part of the water treatment plant project to improve raw water supply. The source water capacity at the Ranney Well is 1,200 - 1,400 gpm but the City has water rights for 2,800 gpm. Once additional laterals are added in the Ranney Well to provide up to 2,800 gpm, the Ranney Well will have the capacity to meet the City's needs for the next 20 years. With one pump out of service, two pumps could provide 2,100 gpm, which is sufficient to meet the current 20 year planning period estimated MDD. With all three pumps on the maximum water right capacity of 2,800 gpm could be provided which is more than enough to meet the Peak Hour Demand (PHD) of 2,229 in 2033. PHD can also be met through providing equalizing storage. Figure III-1 on the next page provides a graphical representation of projected Maximum Daily Demands compared to the current and projected pumping capacities of the Ranney Well and treatment plant.

Due to the high iron content in the source water, the transmission line from the Ranney Well to the treatment plant gets buildup of oxidized iron in the pipe which can cause headloss problems. The City has set up a pigging system to run foam pigs from the Ranney Well up to the treatment plant where they discharge into the backwash lagoons. This is done 1-2 times a year to maintain capacity and minimize headloss through the pipe.

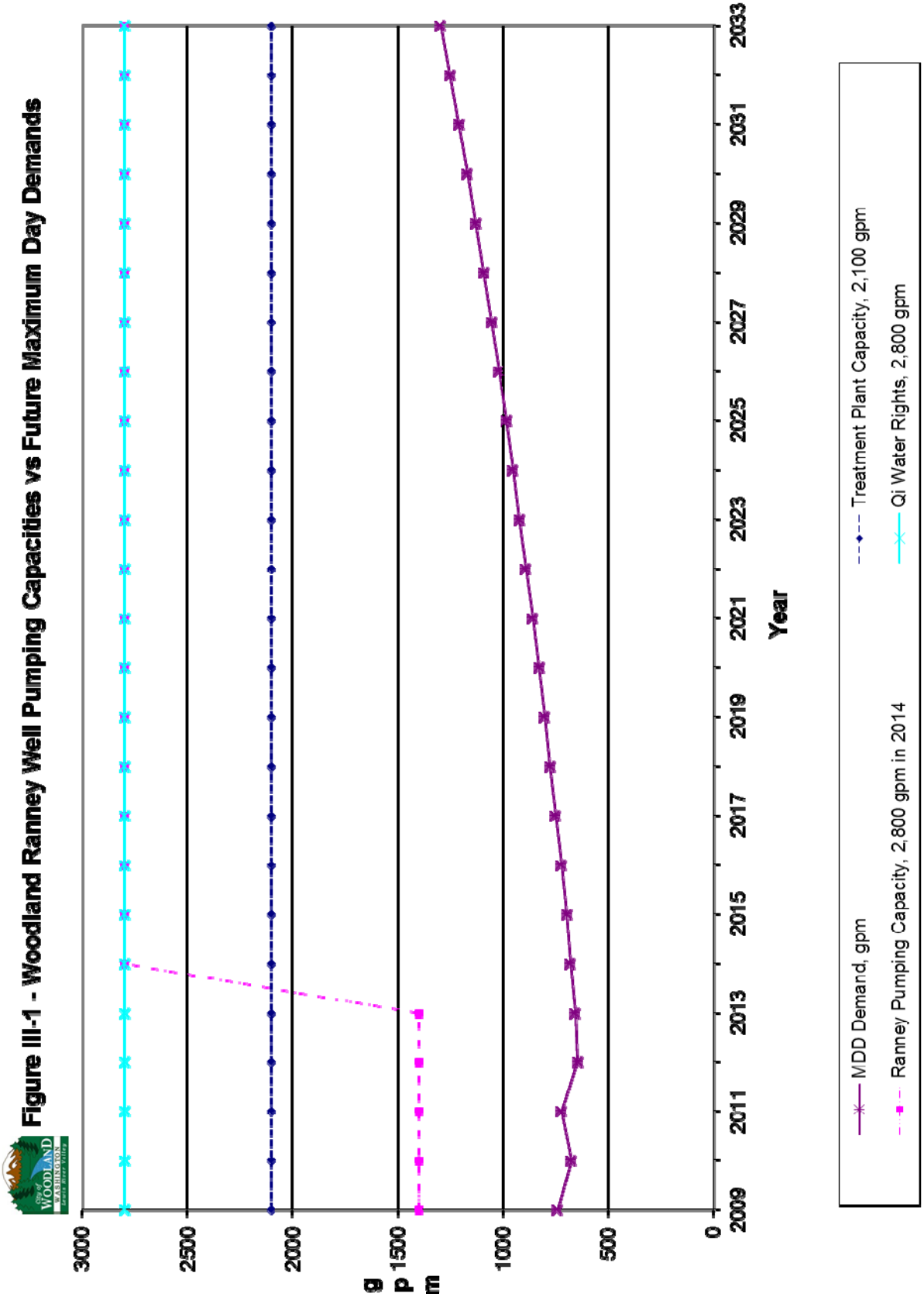


Figure III-1 - Woodland Ranney Well Pumping Capacities vs Future Maximum Day Demands
3000
2500
2000
1500
1000
500
0
gpm
2009
2011
2013
2015
2017
2019
2021
2023
2025
2027
2029
2031
2033
Year
MDD Demand, gpm
Treatment Plant Capacity, 2,100 gpm
Ranney Pumping Capacity, 2,800 gpm in 2014
Qi Water Rights, 2,800 gpm

Treatment

In 1999 the City completed construction of a surface water treatment plant (WTP). The primary purpose of the water treatment plant is to remove the high levels of iron that exist in the source water pumped from the Ranney Well. The 1999 plant had a design capacity of 2 MGD with a 1 MGD (700 gpm) capacity from each treatment train. In 2007 the treatment plant was expanded to 3 MGD by adding a third treatment train.

Raw water is pumped to the WTP from the Ranney Well located along the Lewis River. The raw water is pretreated by pH adjustment and chlorine addition to oxidize iron present in the raw water supply. The filtration system consists of three Microfloc filtration units with upflow clarifiers. Prior to entering the filtration units the City's chemical feed system can provide sodium hypochlorite to oxidize iron, polymer for primary coagulation, aluminum sulfate for secondary coagulation and a Non-Ionic polymer as a filter aid. Filtration of the raw water occurs through the treatment units to remove both turbidity and oxidized iron. Post filtration chlorine is added for disinfection with adequate contact time to achieve the required Giardia and virus reductions through filtration and inactivation. The finished water is fluoridated and also receives pH adjustment with soda ash to provide distribution system corrosion control. Finished water flows into the 169,000 gallon clearwell designed to maintain a minimum of 155,000 gallons for chlorine contact. Baffling of the clearwell was provided to increase contact time.

The treatment system has worked well since being put into operation in 1999 with typical finished water iron levels being non-detectable. Based on projected demands the plant has enough treatment capacity over the 20 year planning period.

The existing treatment plant does have an ongoing ventilation problem. Improvements have been installed in the soda ash room but additional improvements are needed to prolong the life of the equipment. The primary problem is associated with chlorine off gassing during filter backwash cycles as a result of injecting sodium hypochlorite ahead of the filtration units to oxidize the iron in the raw water. It is recommended to install panels directly over the treatment units to introduce air into the covered basins at one end and pull the air out of the other end and exhaust this air out to the atmosphere through the roof. This would likely require 2 fans, one on each side of the filters and then both fans would exhaust when a basin undergoes backwashing. It is anticipated that the cost of simply upsizing the current ventilation system would be similar to installing panels however, panels directly over the treatment units would be more efficient in exhausting the chlorine off gassing during a filter backwash. The off-

gassing has also caused problems with the various electrical components in the plant. Wiring and contacts are corroding and some control panels are no longer used. It is recommended that the electrical system, including SCADA and telemetry, is upgraded as part of the ventilation project. The estimated cost of this project is $203,000 including sales tax.

The plant does not have a combined turbidimeter. Due to the effluent piping layout installing a combined turbidimeter is problematic. For compliance monitoring purposes the City takes the highest reading recorded between the three turbidimeters on any given day.

Storage

The City of Woodland has three existing reservoirs. Reservoir No.1 was constructed in 1912 and is not used. Reservoir No. 2 was constructed in 1962 has an overflow elevation of 179 and has a capacity of 500,000 gallons. Reservoir No. 2 is a concrete reservoir with a newer roof that was constructed in 2005.

Reservoir No. 3 was constructed in 1990 also has an overflow elevation of 179 and has a capacity of 1,100,000 gallons. Reservoir No. 3 is a glass lined bolted steel reservoir with an aluminum geodesic roof that was constructed in 1990. Reservoir No. 3 is still in good condition and does not require improvements.

DOH requires that water systems evaluate the capacity as well as the physical condition of each storage facility. The analysis must address constraints within the six and twenty year planning periods. As discussed previously in this Section, storage requirements are dependent upon capacity and reliability of the source of supply. The City of Woodland is in the process of improving the Ranney Well to provide source capacity sufficient to meet the 20 year projected MDD. As part of the source capacity improvement project a dedicated generator will be installed at the Ranney Well. When this project is completed the Ranney Well will have three pumps with redundant power and only two pumps will be required to meet Maximum Day Demands (MDD) through 2033. The City believes the Ranney Well will have sufficient redundancy to be considered a multiple source for calculations of standby storage. This means that rather than requiring two times the average daily demand for storage, the City would be required to supply 200 gallons per ERU. Table III-3 below lists the projected storage component volumes over the next 20 years. Fireflow storage of 2,000 gpm for 120 minutes was not nested inside the stand-by storage. No equalization storage is required since the source capacity of the Ranney Well will be greater than the 20 year PHD once the source improvements are completed. Including the source improvements at the source scheduled to be completed

in 2014, the City will have adequate reservoir storage through 2027. The storage deficit in 2033 is 257,000 gallons (2,037,000 total required).

The Water System Plan recommends that a minimum 500,000 gallon reservoir be installed by 2027 to meet storage requirements. The new reservoir would be placed at the existing water treatment plant site.

Table III-3 Woodland Storage requirements in gallons

Year	Operational Storage	Equalization Storage	Stand-By Storage	Fireflow Storage	Total Storage	Storage Deficit
2009	288,141	0	864,200	240,000	1,392,341	-387,659
2010	288,141	0	787,000	240,000	1,315,141	-464,859
2011	288,141	0	841,600	240,000	1,369,741	-410,259
2012	288,141	0	751,381	240,000	1,279,522	-500,478
2013	288,141	0	766,167	240,000	1,294,308	-485,692
2014	288,141	0	793,050	240,000	1,321,191	-458,809
2015	288,141	0	812,405	240,000	1,340,546	-439,454
2016	288,141	0	842,918	240,000	1,371,059	-408,941
2017	288,141	0	873,430	240,000	1,401,571	-378,429
2018	288,141	0	903,942	240,000	1,432,083	-347,917
2019	288,141	0	934,454	240,000	1,462,595	-317,405
2020	288,141	0	964,967	240,000	1,493,108	-286,892
2021	288,141	0	1,001,178	240,000	1,529,319	-250,681
2022	288,141	0	1,037,390	240,000	1,565,531	-214,469
2023	288,141	0	1,073,601	240,000	1,601,742	-178,258
2024	288,141	0	1,109,812	240,000	1,637,953	-142,047
2025	288,141	0	1,146,024	240,000	1,674,165	-105,835
2026	288,141	0	1,186,135	240,000	1,714,276	-65,724
2027	288,141	0	1,227,649	240,000	1,755,790	-24,210
2028	288,141	0	1,270,617	240,000	1,798,758	18,758
2029	288,141	0	1,315,089	240,000	1,843,230	63,230
2030	288,141	0	1,361,117	240,000	1,889,258	109,258
2031	288,141	0	1,408,756	240,000	1,936,897	156,897
2032	288,141	0	1,458,062	240,000	1,986,203	206,203
2033	288,141	0	1,509,095	240,000	2,037,236	257,236

Distribution System

Water distribution maps are located at the end of this chapter. Distribution pipe size and materials (where available) are provided in the hydraulic model information. The City currently tracks pipe installation and replacement through as-built drawings. As-built drawings are maintained in a file at the Public Works Department and uploaded to the City's GIS system. The water system map is updated through WSP Updates and when water system projects are completed. Minor repairs and modifications by City staff are documented through notes and revisions to existing maps. The City has also completed mapping of all system hydrants in the GIS system. In the future the City will add valves and other components to the mapping system. The GIS system provide detailed information for the piping including pipe size, material, year of installation and any other noted information. Table III-4 below provides a breakdown of pipe sizes and materials for the existing system.

Table III-4 Woodland Water System Pressure Pipe Inventory

Diameter (in.)	PVC Pipe (ft)	AC Pipe (ft)	Cast Iron (ft)	Steel (ft)	Ductile Iron (ft)	All Materials (ft)
2	0	0	458	748	0	1,206
3	28	0	0	0	0	28
4	10,607	0	497	0	0	11,104
6	34,706	6,140	6,441	0	722	48,009
8	49,844	1,039	4,620	0	1,017	56,520
10	3,761	2,762	4,079	0	282	10,884
12	47,546	4,651	1,220	0	1,929	55,346
16	1,569	0	0	0	0	1,569
All Dia.	148,061	14,592	17,315	748	3,950	184,666
%	80.2	7.9	9.4	0.4	2.1	

Approximately 8% of the distribution system is asbestos cement (AC) pipe. The life expectancy of AC pipe is significantly reduced in areas with hydric soils and/or where groundwater is present within the pipe zone for extended periods. Inspection of the AC pipe over the past decade during service installations indicates that there is little evidence AC pipe deterioration. Therefore, there is no recommendation for system wide AC pipe replacement at this time. However, it is recommended that the condition of AC distribution materials is carefully examined and documented during any future repairs or new service installations.

Hydraulic Analysis

Water systems are required to complete a hydraulic analysis to evaluate existing system performance, identify deficiencies, aid in identifying needed system improvements, and document any special operational needs. WAC 246-290-230 requires that the minimum size for transmission or distribution mains must be determined by a hydraulic analysis. Sizing mains using a hydraulic analysis must, at a minimum, consider the two demand scenarios specified in WAC 246-290-230. DOH also recommends that the design of distribution mains not exceed a maximum velocity of eight feet per second (fps) under PHD conditions. Maximum velocities of greater than eight fps may occur under fire flow conditions, for short sections of mains, or for piping within pump and valve station facilities.

The model used for this analysis is WaterCAD by Haestad Methods. The previous model created during the 2006 Water System Plan was used as a base for the model. Pipes and nodes were input using an overlay of the City's GIS map. Elevations for nodes were obtained using USGS maps and aerial survey information. System demands presented in Section II are apportioned evenly throughout the system nodes. The model results are provided in Appendix C. The model was calibrated using hydrant flow test data done in 2009, 2010, and 2012. Calibration was done using the Darwin Calibrator in WaterCAD. Actual fire hydrant results in nine different locations throughout the City were inputted. The program then optimized pipe roughness coefficients to determine the roughness that best fit those results.

The model was analyzed using three different physical capacities. The first capacity is the existing water system. In the 2018 analysis it was assumed the Ranney Well project in 2014 improved source capacity to match the plant capacity so the pump output increased to 2,100 gpm in the model. In 2033 system capacity is further improved by the addition of 500,000 gallons of storage at the same location and elevation as the existing reservoirs. The 2033 fire flow model also included distribution system improvements identified in this chapter needed to improve fire flows and redundancy in the system.

<u>PHD Model Results:</u> The PHD criteria for the distribution model runs is to ensure that the PHD can be provided at the minimum required pressure of 30 psi for design conditions and at least 20 psi for current operating conditions in all parts of the system. For these model runs the reservoir elevation was set at elevation 175.0 which is approximately the bottom of equalizing storage and 24 hour extended run simulations were done. The system was modeled using the projected six and twenty year PHD. Model results indicate all locations within the existing

system are capable of meeting the 30 psi design criteria except near the reservoirs. The Year 2033 minimum system pressure under PHD demands is approximately 54 psi.

2018 MDD + 500 gpm Fire Flow Model Results: Next, the existing system was modeled using the 2018 MDD in addition to minimum fire flows of 500 gpm. The lowest pressure residual allowed at the hydrant was 34 psi. This is higher than the minimum of 20 psi but this is the lowest residual pressure read during the last 3 sets of fire hydrant tests so it was used to help the model more accurately show actual fire flows in the system. The 2018 fire flow model was used to evaluate the system at the DOH required minimum fire flows of 500 gpm. The results were then used to prioritize the required distribution improvements.

2033 MDD + 1,000 gpm Fire Flow Model Results: Next, the existing system was modeled using the 2033 MDD in addition to minimum fire flows of 1,000 gpm. The lowest pressure residual allowed at the hydrant was 34 psi. This is higher than the minimum of 20 psi but this is the lowest residual pressure read during the last 3 sets of fire hydrant tests so it was used to help the model more accurately show actual fire flows in the system. Nodes that did not represent where hydrants would be used were eliminated from this analysis. For example, multiple apartment complexes and one mobile home park have domestic water lines in addition to fire lines. The domestic lines did not provide adequate fire flow but they are not required to as the adjacent fire lines are properly sized. Nodes at the end of cul-de-sacs and other areas that did not need hydrants were also removed. The existing 4" line on Goerig west of the railroad tracks was removed as that is a domestic line only and not intended for fire flow. Improvements to the pipe system were then made to the model until all applicable nodes had fire flows of 1,000 gpm.

A residual "design" pressure of 20 psi must be maintained throughout the distribution system for fire flow conditions. For "operating" conditions, 20 psi must be maintained at the operating hydrant and at least positive pressure must be maintained throughout the system (WAC 246-290-420(3)). For the fire flow analysis the reservoir elevation was set at elevation 175 which is approximately the bottom of equalizing and fire flow storage. The model indicates that the majority of the existing system is capable of providing minimum fire flow goals under "design" conditions. The following is a description of the current, 6 year and 20 year projections and which areas do not meet the DOH required minimums and the City's fire flow goals.

2018 MDD plus DOH Minimum Fire Flows

In the 2018 analysis the only area that does not meet the required DOH minimum fire flows under design conditions is along Lakeshore Drive and Island Aire Drive. The solution is to install approximately 2,700 linear feet of 8" water main within Horseshoe Lake Park and along Lakeshore Drive. Upsizing to 8" would not provide the 1,000 gpm needed in the 2033 MDD analysis.

20-year MDD plus 1,000 gpm Minimum Fire Flows

By increasing the fire flow requirement to 1,000 gpm several areas of the system were found to be deficient in fire flows. For the existing system the following areas would not meet a fire flow requirement of 1,000 gpm:

- To meet 1,000 gpm the Lakeshore Drive Project requires 2,062' of 12" water main from Davidson and Goerig to Lakeshore and Island Aire. The project also requires 616' of 8" water main along Lakeshore south of Island Aire and 458' of 6" water main at the end of Island Aire.
- Along Scott Avenue and SR 503 between Goerig Street and Scott Avenue Drive. It is recommended that approximately 1,400 linear feet of 12" water main be installed along Scott from Goerig to SR 503 and 1,000' of 12" water main along SR 503 from Scott to Goerig to meet the City's fire flow goals as well as for reliability purposes. Currently there is a 12" water main that feeds east from the reservoir site to the east side of the water system and a 6" water line that feeds the east side of the system along Scott Avenue and SR 503. If a problem were to occur or if the 12" needed to be isolated for maintenance the 6" water line can not supply the necessary water to the east side of the system.
- Along SR 503 from E. Scott to Gun Club. This project would require 2,314 feet of 12" water main to replace the existing 6" water main.
- Along SR 503 at the end of the water system between Salmon Street and McCraken Road. To meet the City's fire flow goals would require approximately 1,200 linear feet of 12" water main.
- Along 5th north of Bozarth. To meet the City's fire flow goals at the primary school would require approximately 650 feet of 8" water main to replace existing 6" main.
- Along South Pekin to serve the southernmost end of the City. This project would require the installation of 871' of 12" water main from Marty to Windflower and 579' of 12" water main from Lake to just south of the Lilac Gardens.
- The replacement of 964' of existing 6" water main to the sewer treatment plant with a 12" water main.

Another water main recommended to be installed for reliability is a 12" water main from the end of the existing 12" on West Scott Avenue under the railroad tracks and connect to the 12" at the intersection of Guild Road and North Pekin Road. This would require construction of approximately 800 linear feet of 12" with a railroad bore crossing.

Scott Hill Booster Station

Near the treatment plant and reservoirs are several homes that have individual booster pumps to provide sufficient pressure to their homes. In addition, the City in 2011 purchased over 40 acres for a future park right next to the water treatment plant. To serve the homes and future park a booster pump station is needed. There is sufficient room at the treatment plant site to place the station. It is anticipated this station will be built when the park is developed. The timeline for park development is uncertain but the hope is to complete the park within 5 years.

OVERALL CAPACITY

DOH requires that water systems provide an analysis of the system to determine overall physical service capacity in terms of ERU's. Table III-5 presents the capacities of the existing system components and of the combined water system based on WSDM formulas and guidance.

Table III-5 Existing Water System Capacity Limitations

Existing System Component	ERU Capacity
1,400 gpm Source (MDD)	8,129
2,100 gpm Treatment (MDD)	12,194
2,800 gpm Water Rights (Qi)	16,258
2,186 acre-feet Water Rights (Qa)	10,549
1,600,000 gallons of Storage	6,452

The limiting system components are currently the source capacity and storage. The Ranney Well Pumps are large enough, there just isn't enough water coming from the laterals to match water rights. Storage is a secondary limitation that needs to be addressed within the 20 year time frame of this plan. This WSP recommends improvements for the Ranney Well and treatment plant within 6 years.

Table III-6 presents the capacities of the recommended 20-year system components and of the combined water system based on WSDM formulas and guidance.

Table III-6 Proposed 20-year Water System Capacity Limitations

Proposed 20-year System Component	ERU Capacity
2,800 gpm Source (MDD)	16,258
2,100 gpm Treatment (3.0 MGD) (MDD)	12,194
2,800 gpm Water Rights (Qi)	16,258
2,186 acre-feet Water Rights (Qa)	10,549
2,100,000 gallons of Storage	8,468

SUMMARY OF SYSTEM DEFICIENCIES

Table III-7 provides a summary of the system deficiencies and recommended alternatives. Priorities for implementation, project descriptions and planning level cost estimates are discussed in the Capital Improvement Program in Section VIII and the Financial Program in Section IX. Figure VIII-1, Capital Improvement Program Map, in Section VIII shows the general area of where these improvements would be located.

Table III-7 Summary of Deficiencies

Deficiency	Recommended Action
Source of Supply	
A) Ranney Well Pumping Rate does not meet water rights	Install new laterals so water availability matches existing water rights. Install generator at Ranney Well.
Treatment	
B) Treatment Plant Ventilation	Install additional ventilation at the Treatment Plant to exhaust high levels of chlorine. Upgrade electrical systems that are outdated and have been corroded by chlorine off-gas.
Storage	
C) Storage Capacity	Implement 500,000 gallons of storage by 2028 (total storage required in 2033 2,024,000).
Transmission	
No Deficiency	Transmission facilities are expected to be serviceable at least through the 20-year planning period.
Distribution	
D) Lakeshore Drive	The area along Lakeshore Drive and Island Aire Drive is needed to be upgraded to 12", 8", and 6" PVC to meet DOH minimum fire flows. Project requires 2,100' of 12", 600' of 8", and 450' of 6" water main.
E) Scott/Goerig/SR 503 Triangle	Install approximately 1,400 linear feet of 12" water main along Scott from Goerig to SR 503 and 1,000' of 12" water main along SR 503 from Scott to Goerig.
F) SR 503, Scott – Gun Club	This project would require 2,314 feet of 12" water main to replace the existing 6" water main.
G) SR 503, Salmon - McCraken	Installation of 1,200 linear feet of 12" water main.
H) 5th Street north of Bozarth	To meet the City's fire flow goals at the primary school would require approximately 650 feet of 8" water main to replace existing 6" main.
I) South Pekin Project	This project would require the installation of 871' of 12" water main from Marty to Windflower and 579' of 12" water main from Lake to just south of the Lilac Gardens.
J) Wastewater Plant	Replace 964' of existing 6" water main to the sewer treatment plant with a 12" water main.
K) West Scott Connector	West Scott Avenue under railroad tracks – Install approximately 800 feet of 12" PVC.
L) Scott Hill Booster Station	Install booster pump station at treatment plant site to serve Scott Hill Park and surrounding homes.

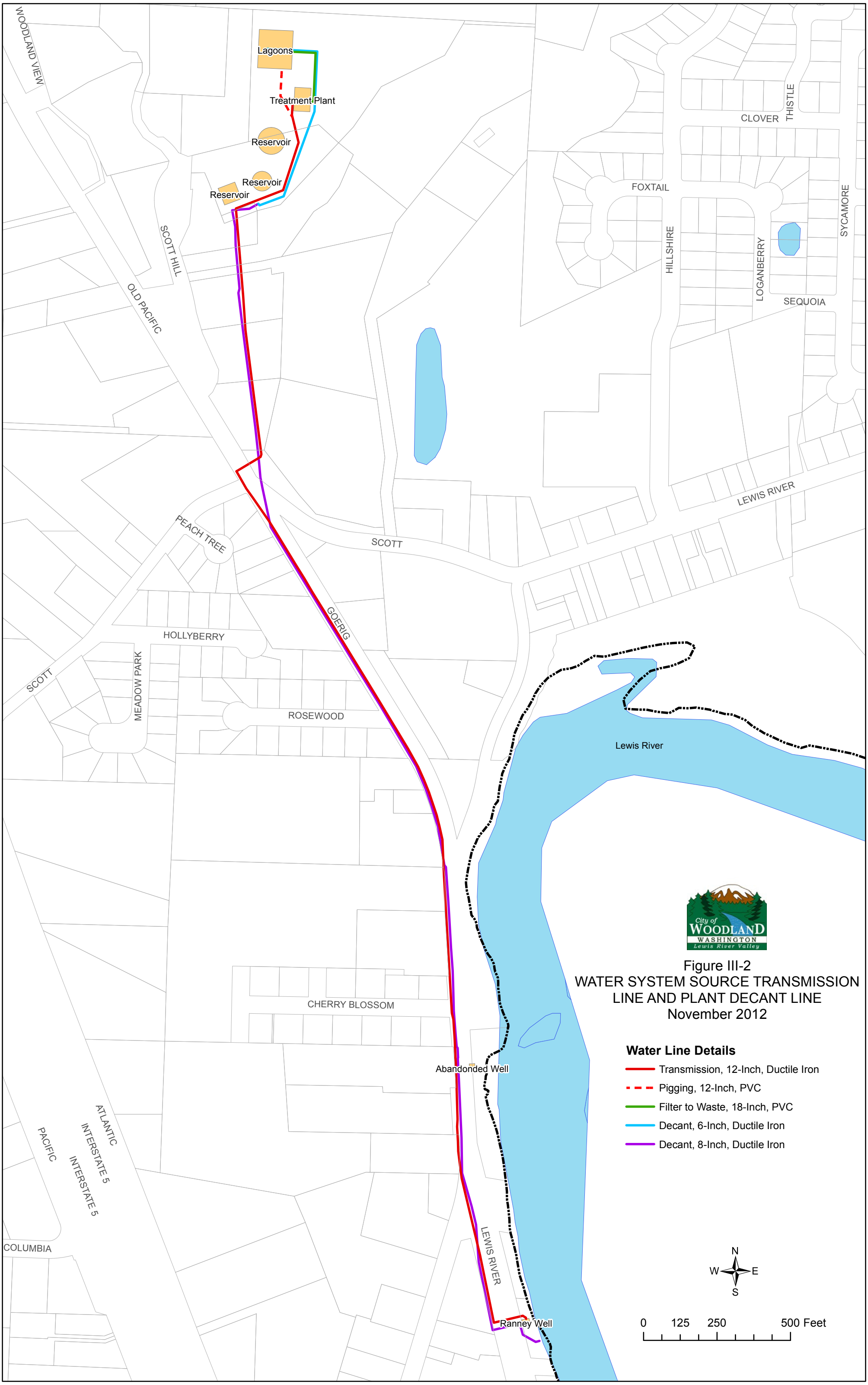
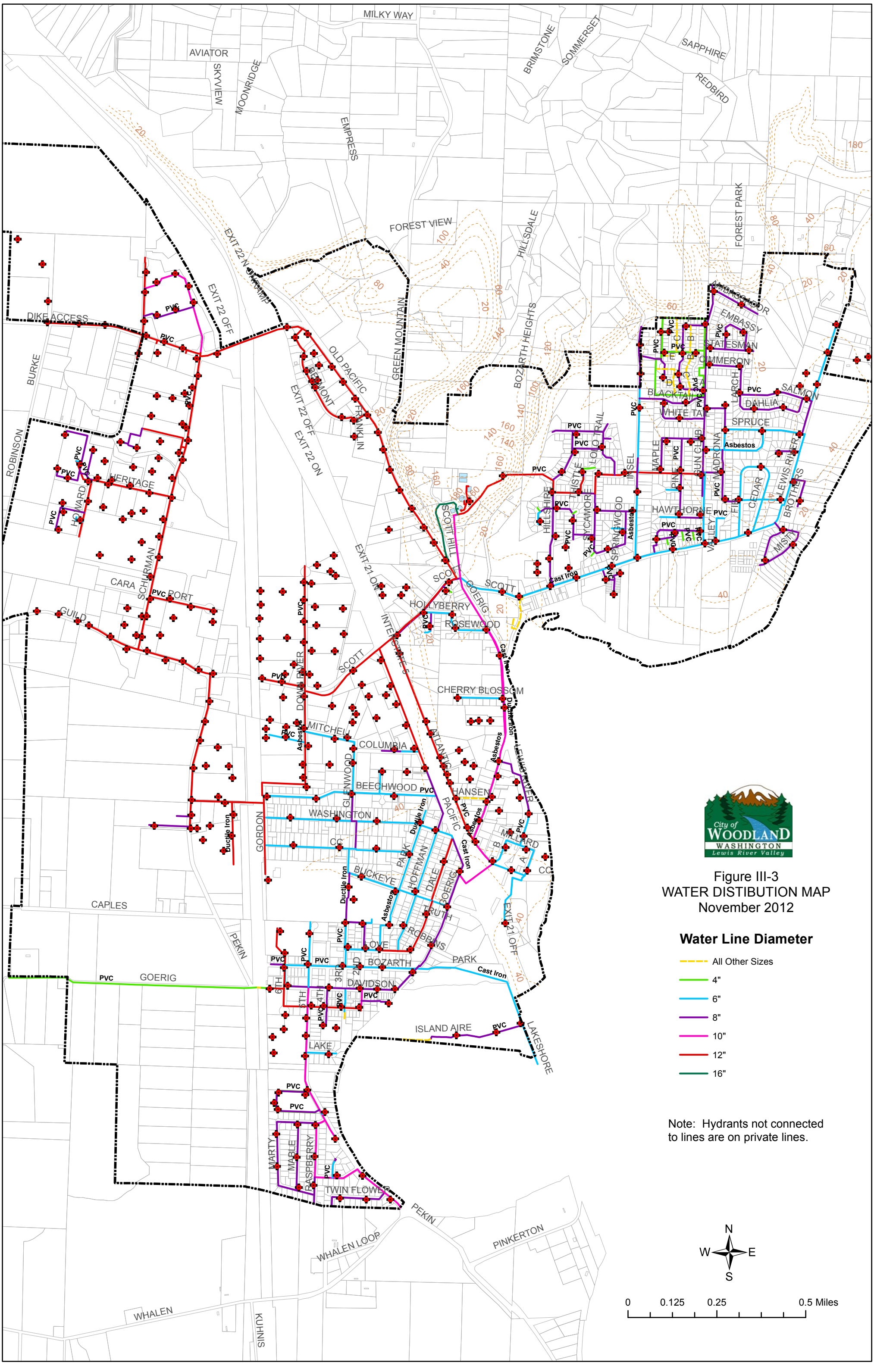
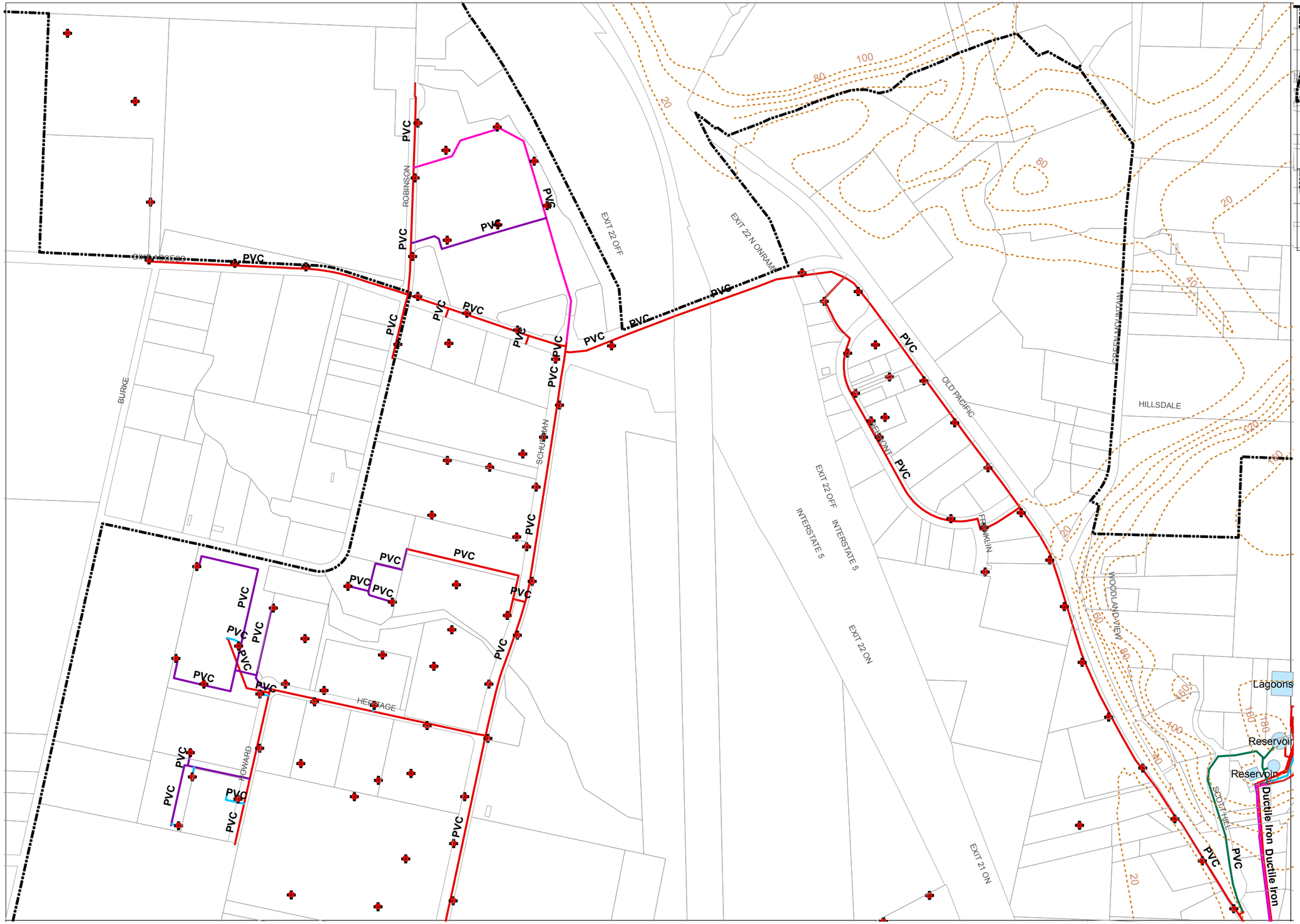


Figure III-2
WATER SYSTEM SOURCE TRANSMISSION LINE AND PLANT DECANT LINE
November 2012


MILKY WAY
AVIATOR
SKYVIEW
MOONRIDGE
EMPRESS
BRIMSTONE
SOMMERSET
SAPPHIRE
REDBIRD
FOREST VIEW
HILLSDALE
FOREST PARK
EXIT 22 ON RAMP
EXIT 22 OFF
DIKE ACCESS
BURKE
ROBINSON
HERITAGE
HOWARD
CARA
SCHURMAN
PORT
GUILD
OLD PACIFIC
BEAUMONT
FRANKLIN
EXIT 22 OFF
EXIT 22 ON
GREEN MOUNTAIN
BOZARTH HEIGHTS
EXIT 21 ON
SCOTT HILL
SCOTT
GOERIG
HOLLYBERRY
ROSEWOOD
INTERSTATE 5
CHERRY BLOSSOM
DOWER RIVER
MITCHELL
COLUMBIA
GLENWOOD
BEECHWOOD
WASHINGTON
GORDON
CC
BUCKEYE
PARK
HOFFMAN
DALE
TRUTH
ROBBINS
LOVE
BOZARTH
DAVIDSON
PARK
ATLANTIC
PACIFIC
HANSEN
EAST RIVER
MILLARD
EXIT 21 OFF
CAPLES
PEKIN
GOERIG
6TH
5TH
4TH
3RD
2ND
LAKE
ISLAND AIRE
LAKESHORE
MARTY
MABLE
RASPBERRY
TWIN FLOWER
PEKIN
WHALEN LOOP
PINKERTON
WHALEN
KUHNIS
EMBASSY
STATESMAN
COMMERON
BLACKTAIL
WHITE TAIL
LARCH
SALMON
DAHLIA
SPRUCE
MADRONA
GUN CLUB
MAPLE
PINE
INSEL
THISTLE
LOLO TRAIL
HILLSHIRE
SYCAMORE
SPRINGWOOD
HAWTHORNE
VALLEY
FIR
CEDAR
LEWIS RIVER
BROTHERS
MISTY
PVC
Asbestos
Ductile Iron
Cast Iron
City of WOODLAND WASHINGTON Lewis River Valley
Figure III-3
WATER DISTIBUTION MAP
November 2012
Water Line Diameter
All Other Sizes
4"
6"
8"
10"
12"
16"
Note: Hydrants not connected to lines are on private lines.
N
W
E
S
0
0.125
0.25
0.5 Miles



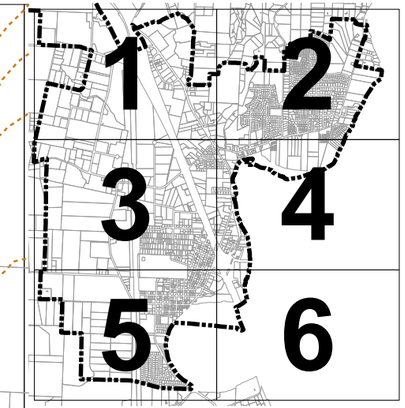


Figure III-3.1
WATER DISTIBUTION MAP
November 2012

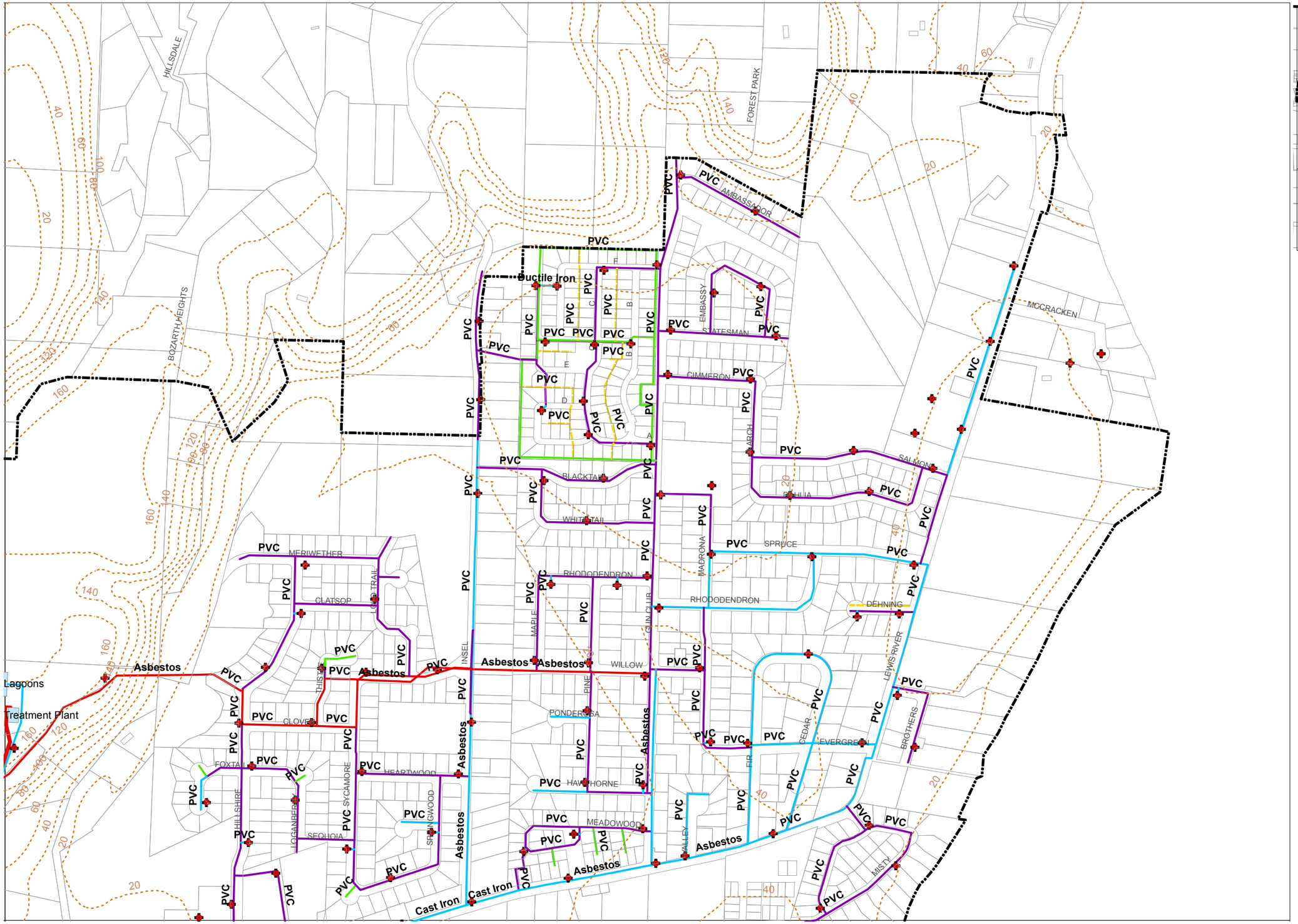
Water Line Diameter

- All Other Sizes
- 4"
- 6"
- 8"
- 10"
- 12"
- 16"

N W E S

0 200 400 800 Feet

SHEET 1 OF 6



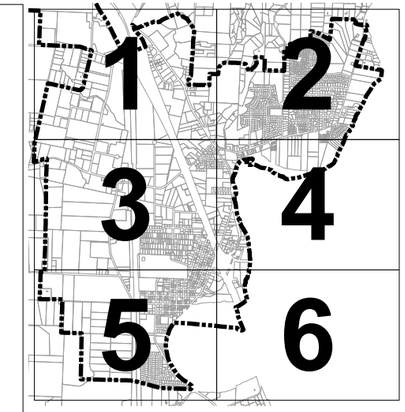


Figure III-3.2
WATER DISTIBUTION MAP
November 2012

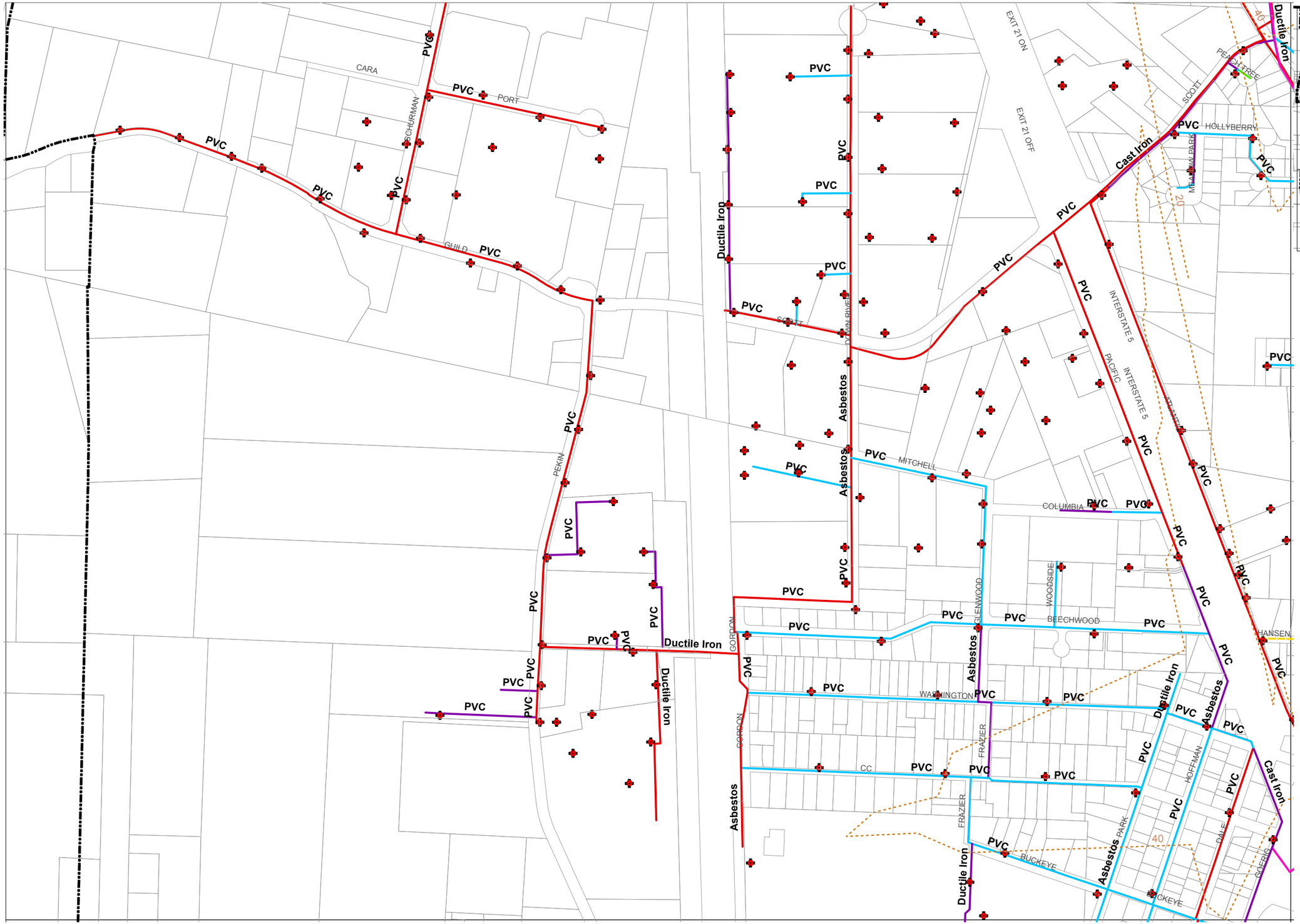
Water Line Diameter

- All Other Sizes
- 4"
- 6"
- 8"
- 10"
- 12"
- 16"

N
W E
S

0 200 400 800 Feet

SHEET 2 OF 6



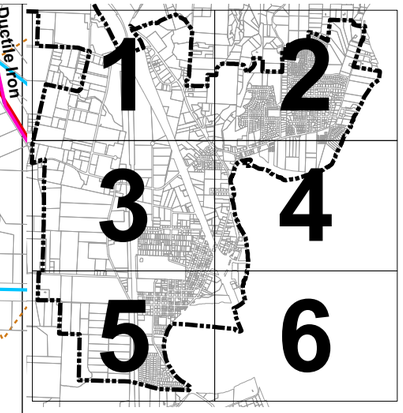


Figure III-3.3
WATER DISTIBUTION MAP
November 2012

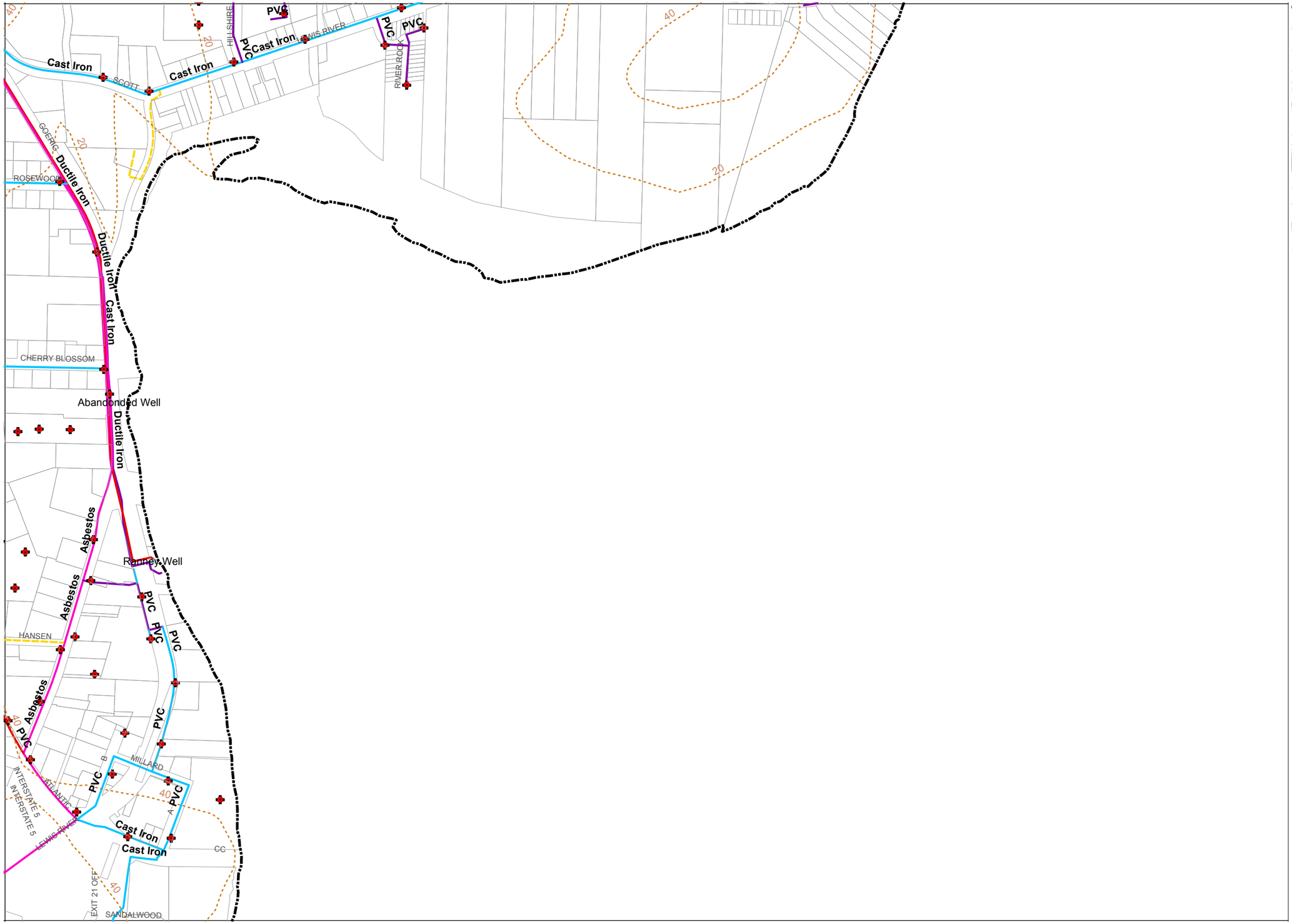
Water Line Diameter

- All Other Sizes
- 4"
- 6"
- 8"
- 10"
- 12"
- 16"

N
W E
S

0 200 400 800 Feet

SHEET 3 OF 6



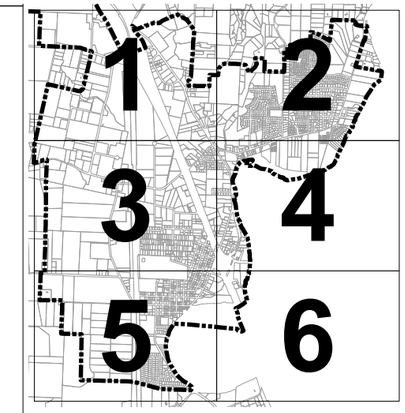


Figure III-3.4
WATER DISTIBUTION MAP
November 2012

Water Line Diameter

- All Other Sizes
- 4"
- 6"
- 8"
- 10"
- 12"
- 16"

N
W E
S

0 200 400 800 Feet

SHEET 4 OF 6

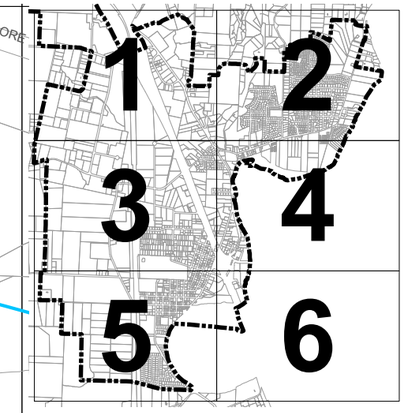


Figure III-3.5
WATER DISTIBUTION MAP
November 2012

Water Line Diameter

- All Other Sizes
- 4"
- 6"
- 8"
- 10"
- 12"
- 16"

N
W E
S

0 200 400 800 Feet

SHEET 5 OF 6

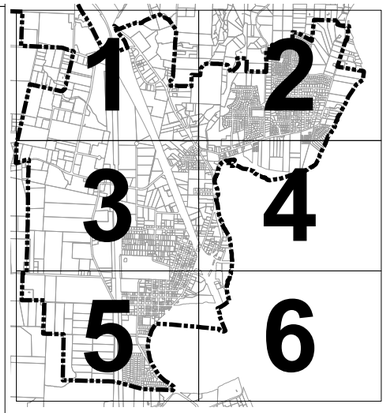


Figure III-3.6
WATER DISTIBUTION MAP
November 2012

Water Line Diameter

- All Other Sizes
- 4"
- 6"
- 8"
- 10"
- 12"
- 16"

N
W E
S

0 200 400 800 Feet

SHEET 6 OF 6