CITY OF BELLINGHAM

WHATCOM COUNTY **WASHINGTON**

CITY OF BELLINGHAM WASHINGTON

WASTEWATER CONVEYANCE PLAN

VOLUME I

JULY 2016

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CHAPTER 1

INTRODUCTION

The City of Bellingham is located in Whatcom County, in the northwest portion of Washington State as shown in Figure 1-1. The City incorporated in 1904 and, as was typical of development during that time, created a system of combined storm and sanitary sewers for discharge to the surface waters of Bellingham Bay and Whatcom Creek. In 1947, the City built treatment facilities near the mouth of Whatcom Creek and then replaced the original treatment facilities in 1974 with the Post Point Wastewater Treatment Plant (WWTP). The WWTP has since been upgraded to improve capacity and treatment, including an upgrade just recently completed in 2014. The expansion of the collection and conveyance system that occurred after treatment facilities were constructed generally separated the sanitary from storm flows, and reduced the number of Combined Sewer Overflow (CSO) locations; however, portions of the legacy combined system still collect and convey flows to the WWTP. One CSO remains, and is used infrequently to relieve capacity during extreme weather events.

Because significant growth is projected for the Bellingham area over the next 20 years, planning for that growth will be essential to properly serve new customers within the intended sewer service area, including those areas within the City limits and the Urban Growth Area (UGA). It is also important to evaluate the *existing* wastewater collection infrastructure to determine its capability to serve the projected population and to determine required system improvements for the planning period.

This *2016 Wastewater Conveyance Plan* (*2016 Plan*) for the City of Bellingham (City) addresses the City's planning needs for wastewater collection and transmission for currently non-serviced areas within and on the periphery of the City's UGA while also assessing the impacts of flows from these areas on the downstream system. Development of the *2016 Plan* has been coordinated with the City of Bellingham's *Comprehensive Plan Update*, which is currently being prepared, and with the City of Bellingham *2009 Comprehensive Sewer Plan (2009 Plan)*.

1.1 PREVIOUS SEWER SYSTEM EVALUATION

Past evaluations of the City's sewer system include the *1998 Wastewater Conveyance System Plan* (*1998 Plan*) prepared by Earth Tech, Inc. and the *2009 Plan* developed by Carollo Engineers. The purpose of the *1998 Plan* was to guide development of the City's sewer system by reviewing the infrastructure at the time, proposing solutions to current problems and providing a prioritized, ranked set of projects. High priority projects included those aimed at reducing combined sewer overflows to one per year and to confine these overflows to one location.

The *2009 Plan* identifies collection system and treatment plant deficiencies, and proposes improvements to accommodate future growth. Deficiencies were identified in the *2009 Plan* based on a sewer model which evaluated the sewer system's capacity to convey present and projected wastewater flows as well as stormwater flows during particular high storm events. The *2009 Plan* focused on providing recommended system improvements, future expansion suggestions, and peak flow management solutions. For peak flow management, the *2009 Plan* recommended that peak flow reduction occur through inflow and infiltration (I/I) control and that additional storage be provided at the existing CSO outfall to contain more of the high flows associated with the Roeder Lift Station. Another major recommendation of the *2009 Plan* included an increase in the BOD capacity of the existing wastewater treatment plant, which led to the construction of improvements at the plant, completed in 2014. Since 2009, the City has completed the Lincoln Street project which involved upsizing the sanitary sewer mains in this region from 10-inch and 12-inch mains to 18-inch mains. The remaining capital improvement projects listed in the *2009 Plan* have yet to be addressed however, the City has been focusing on controlling inflow into the system. By reducing inflow, the City hopes to lessen the size of a future peak flow control facility as recommended in the 2009 Plan.

1.2 CURRENT EVALUATION

Since 2009, the City has re-examined growth projections for the outer city service area. The intent of the *2016 Plan* is to address these new growth projections by building on the information provided in the *2009 Plan* with a focus on providing service to currently non-serviced areas within the outer UGA limits. The *2016 Plan* also addresses the effect future growth has on the capacity of downstream infrastructure. It provides proposed conceptual designs for currently non-serviced areas in the outer UGA region, cost estimates, and a capital improvement schedule for recommended major facility improvements. All analyses are based upon flow criteria similar to the 2009 Plan as well as the Year 2026 hydraulic model.

In addition to this introductory chapter, this *2016 Plan* is presented in four chapters: Chapter 2 summarizes sewer service area characteristics; Chapter 3 summarizes existing wastewater collection and conveyance facilities; Chapter 4 provides projected flows, and a hydraulic analysis of the sewer system to convey the flows, Chapter 5 provides analysis of lift stations with capacity concerns; and Chapter 6 summarizes the Capital Improvement Plan.

The projects described in this Plan are consistent with Washington State regulations and guidance, including the Ecology's *Criteria for Sewage Works Design (Orange Book*). This Plan does not address the WWTP or any needed WWTP improvements.

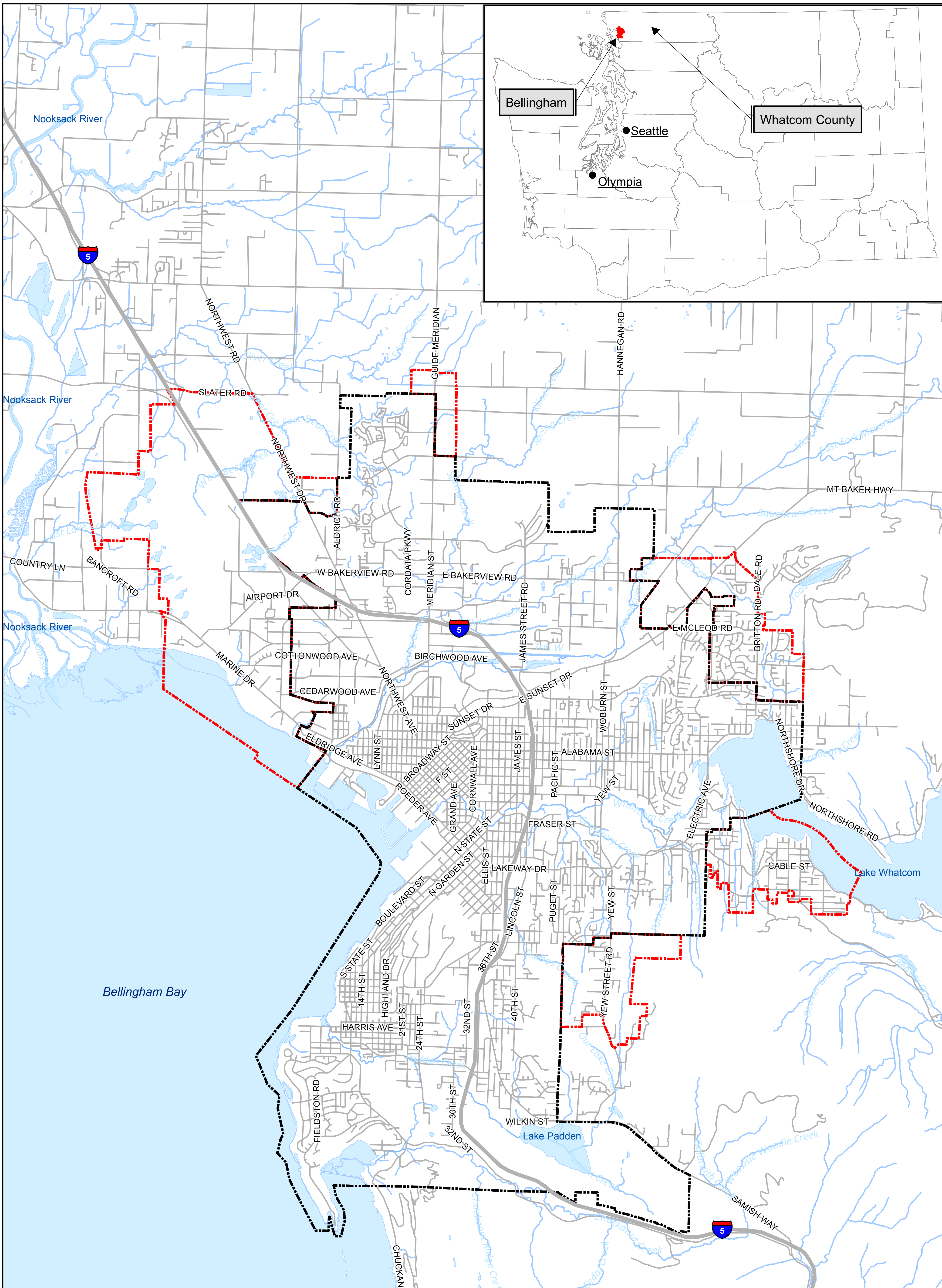

FIGURE 1-1

Legend

- City Limits
- UGA Boundary
- Streams
- Streets

N

0 2,500 5,000 10,000 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
VICINITY MAP

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L:\Bellingham\15448.00 WW Conveyance Plan\Report\Figure 1 Location MA 2.mxd

CHAPTER 2

SERVICE AREA CHARACTERISTICS

The City of Bellingham is located in Whatcom County, about 17 miles south of the Canadian border, and encompasses an area of 25.6 square miles. Located along Interstate 5 at the intersection with State Route 539, the City of Bellingham is bordered by Bellingham Bay to the west and abuts the northern portion of Lake Whatcom on the east. The nearest cities to Bellingham are Ferndale to the west, Lynden to the north, and Burlington to the south.

2.1 SEWER SERVICE AREAS

The City of Bellingham is served primarily by its own sanitary sewer collection and conveyance system, although the Lake Whatcom Water and Sewer District (LWWSD) serves some residences and businesses in the eastern area of the City. All wastewater is treated at the Post Point Wastewater Treatment Plant (WWTP), a 72 mgd facility located in the southwest portion of the City.

2.1.1 CITY OF BELLINGHAM

The City of Bellingham's current sewer service area includes approximately 39 square miles (25,161 acres) as shown in Figure 2-1, and serves approximately 93,000 customers. The City's sewer service boundary includes most of the City, as well as some areas outside of the City but within the UGA. The City's system is divided into eight basins: Birchwood, Broadway, Central, Cordata/Meridian, Lake Whatcom, Northwest, South Side, and Sunset/Mt. Baker. The collection system is primarily a conventional gravity sewer system, consisting of an estimated 323 miles of wastewater collection/conveyance pipes.

The potential future service area, as outlined in the City's *2006 Comprehensive Plan* and *2009 Sewer Plan*, includes the City limits, UGA, and other rural areas of Whatcom County that fall within the City's Urban Fringe Subarea (UFS). The UFS is approximately 30.8 square miles (20,000 acres), slightly smaller than the City's current service area. This *2016 Plan* evaluates potential additions to the City's service area, though these additions are all within the UGA. In total, 9.1 square miles (5,830 acres), both within the City limits and within the City's UGA, were evaluated for the potential of receiving sanitary service.

The City's sewer system in the downtown area is a combined sewer system (CSS), and conveys sewage along with stormwater. One combined sewer overflow (CSO) facility exists in the City at the C Street overflow structure and has been used occasionally when flows exceed capacity during extreme storm events. The City's existing National

Pollution Discharge Elimination System (NPDES) Permit allows only one CSO event per year.

2.1.2 LAKE WHATCOM WATER AND SEWER DISTRICT

LWWSD operates a sanitary sewer collection and conveyance system in the eastern portion of the City. LWWSD maintains approximately 330,000 feet of mainline gravity sewers, 65,000 feet of forcemain, and 24 lift stations to serve approximately 3,400 residences. LWWSD does not own a treatment plant, so sewage from its service area is transported, treated, and disposed of by the City, under a contract, at a maximum flow rate of 2,400 gallons per minute.

The *2009 Plan* indicates that LWWSD expects to serve an additional 1,500 to 1,550 residences by 2030. These residences would be located within the vicinity of Lake Whatcom.

2.2 NATURAL ENVIRONMENT

2.2.1 TOPOGRAPHY

The City of Bellingham is located in, and is the county seat of, Whatcom County. Elevations in Bellingham range from sea level at the shoreline in the western portion of the City, to approximately 1,000 feet in the south of the City. Topography of the City includes rolling hills, beaches, a high bank marine coastline, uplands, streams, and freshwater lakes. Slopes mostly range from flat up to 15 percent, with some steeper slopes in areas to the south, and along bluffs as noted below. These slopes naturally drain surface water toward Bellingham Bay.

Figure 2-2 is a topographic map based on United States Geologic Survey (USGS) showing the varying elevations within the City's sewer service area.

2.2.2 SOILS AND GEOLOGY

The primary soil types in the City of Bellingham consist of glacially derived surficial deposits underlain by bedrock and coal deposits. These soils are a combination of silt, clay, and sand according to the "Soil Survey of Whatcom County". Alluvial deposits exist along the floodplains of the Nooksack River and Squalicum Creek. Marine terrace deposits flank the alluvial deposits within the Nooksack River floodplain in the western area of the City. Glaciomarine drift deposits occupy most of the City's area. Glacial outwash deposits and peat deposits are found throughout the City. Slopes between 15 and 35 percent (within the County's CAO landslide hazard definition) exist along much of the Bellingham Bay bluffs, some of the incised creek and stream channels, and within the vicinity of Squalicum and Galbraith Mountains. Slopes in some of these locations may exceed 35 percent.

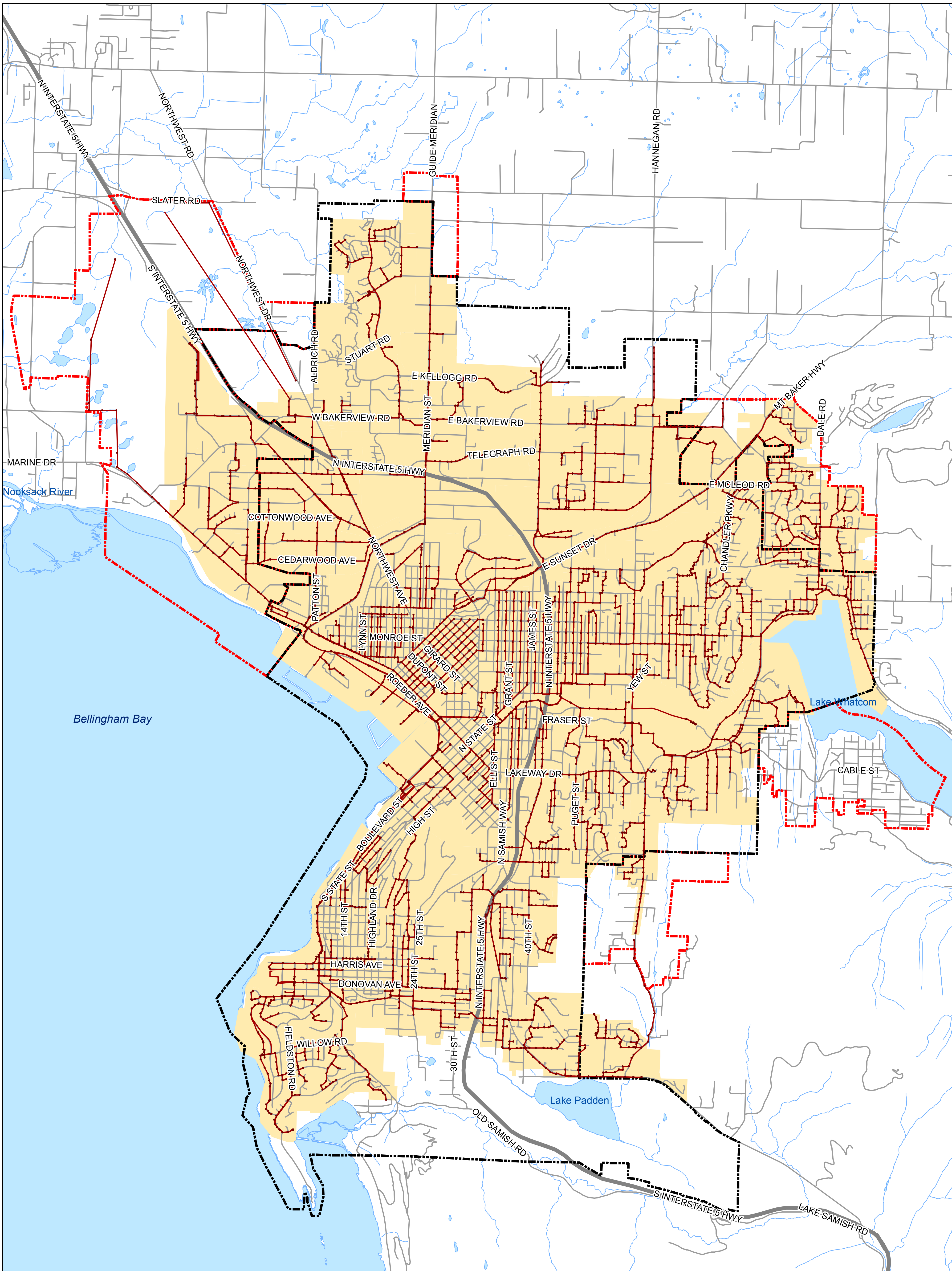

FIGURE 2-1

Legend

- City Limits
- UGA Boundary
- Area Served
- Sewer Pipes
- Manholes

N

0 2,500 5,000 10,000 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
SANITARY SEWER SERVICE AREA

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L:\Bellingham\15448.00 WW Conveyance Plan\Report\Figure 2 UGA limits 2.mxd

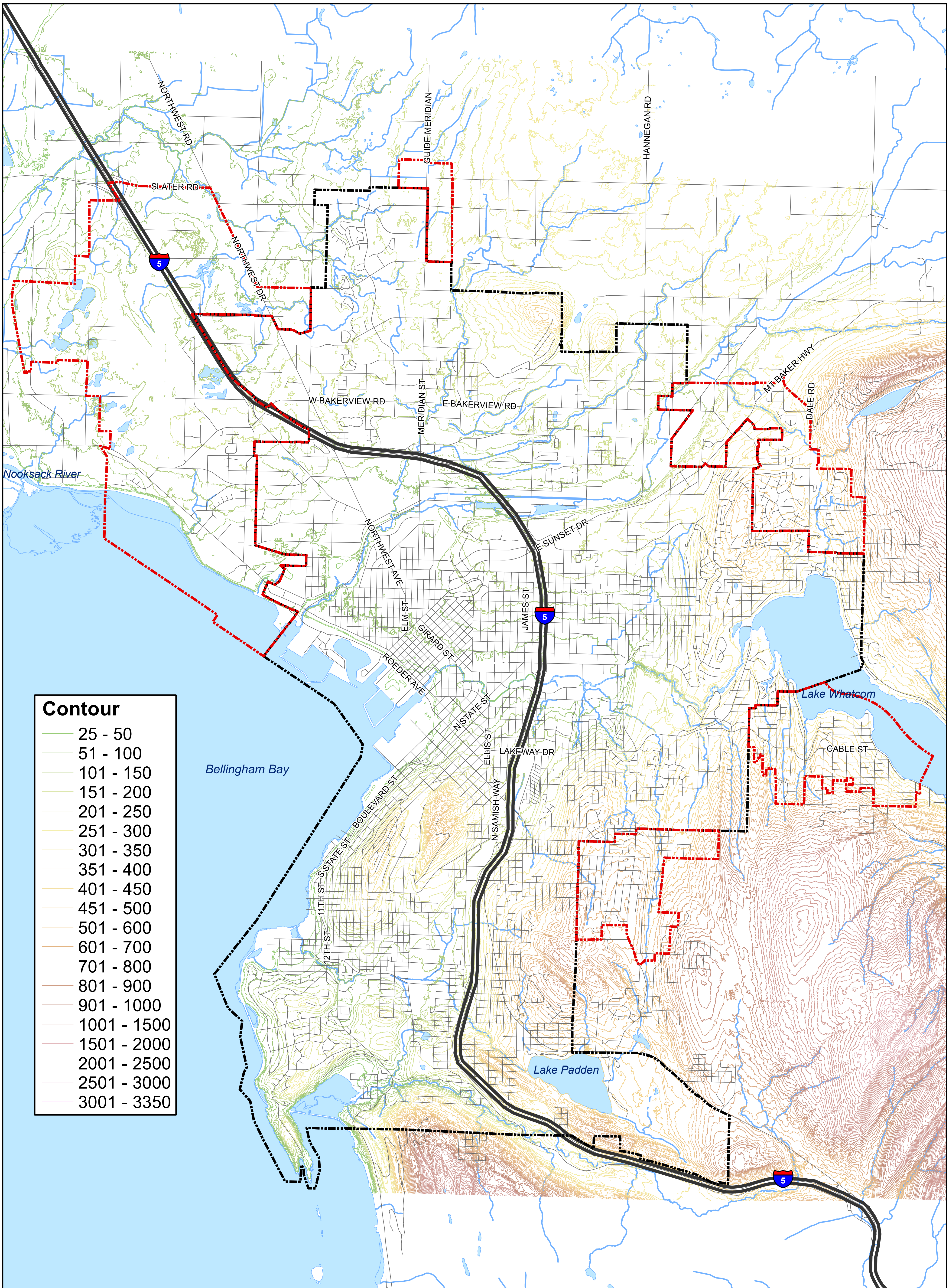

FIGURE 2-2

Legend

City Limits

UGA Boundary

Streams

N

0 2,500 5,000 10,000 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
TOPOGRAPHY

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L:\Bellingham\15448.00 WW Conveyance Plan\Report\Figure 3 topo.mxd

A more detailed definition of the types and locations of the soil classifications within the sewer service area is presented in Figure 2-3, based upon an NRCS Soil Survey of Whatcom County, Washington.

2.2.3 CLIMATE

A weather station is located in Bellingham, in the southwestern portion of the City. Table 2-1 provides precipitation data from this weather station.

TABLE 2-1

Bellingham Area Total Precipitation 2005 – 2014 (inches)

Year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Annual
2005	4.20	2.05	3.01	3.80	1.70	1.55	0.77	1.73	0.91	5.09	5.17	3.86	33.84
2006	8.12	3.23	1.42	2.81	1.76	1.75	0.62	0.59	1.68	1.58	12.06	4.35	39.97
2007	5.86	3.06	5.56	2.13	1.33	1.68	1.08	0.73	1.50	3.80	2.45	4.98	34.16
2008	2.25	3.17	3.96	1.45	2.28	2.37	0.26	2.77	0.78	2.38	5.77	2.49	29.93
2009	5.47	2.12	2.69	1.99	3.90	0.40	0.52	1.00	2.15	6.95	6.97	1.74	35.90
2010	4.07	2.08	3.27	2.09	4.10	2.69	0.09	0.68	4.22	2.21	4.64	5.82	35.96
2011	6.57	2.21	4.28	4.49	3.89	0.77	1.12	0.32	0.98	0.22	4.50	2.23	31.36
2012	3.74	4.71	6.04	4.07	1.88	3.36	2.26	0.13	0.13	6.80	4.26	5.04	42.42
2013	4.00	2.12	2.67	4.93	3.16	1.84	0.07	1.27	4.65	1.22	4.73	2.54	33.20
2014	3.06	3.62	5.48	2.85	3.61	0.88	1.72	0.84	2.97	6.53	4.79	5.95	42.30
Average	4.73	2.84	3.84	3.06	2.76	1.73	0.85	1.01	2.00	3.68	5.53	3.90	35.90

SOURCE: BELLINGHAM 3 SSW, WA gauge (450587)

The precipitation data from the Bellingham 2 SSW weather station is a reasonably accurate representation of the precipitation that much of Bellingham experiences, due to its location in the southwest portion of the City. The 10-year annual average precipitation at the station was 36 inches. The maximum monthly precipitation over the last 10 years was 12.06 inches, which occurred in November 2006. The 10-year minimum monthly precipitation was 0.07 inches, seen in July 2013.

2.2.4 SITE SENSITIVE AREAS

Critical areas within the sewer service area include those classified as streams and watercourses, wetlands, frequently flooded areas, critical aquifer recharge areas, geologically hazardous areas, and fish and wildlife habitat conservation areas.

Policies implemented by the City to protect and maintain sensitive areas include encouraging developments to be located away from environmentally sensitive areas, encouraging open space buffers, compliance with SEPA and the Shorelines Management Act, planning and development review, developer incentives for protecting critical areas on site, and coordination with other jurisdictions in the area.

The following plans and regulations address the protection of the natural environment within Bellingham:

- Bellingham Comprehensive Plan Update, 2016 (Draft currently being prepared).
- Bellingham Critical Areas Ordinance, BMC 16.55.

The site sensitive areas within the sewer service area are described further below. These areas are important to consider in expansion of the City's wastewater system, as sensitive areas can limit developable areas and can constrain where sanitary sewer infrastructure may be constructed.

2.2.4-A Surface Water

Lakes and streams are classified as sensitive areas due to the variety of plants and animals that they support. Bellingham's wet climate and sloping terrain provides flows to many streams and creeks, which drain into Bellingham Bay, at the west end of the City.

The sewer service area encompasses four watersheds, each of which includes multiple basins. Within the service area, 1,700 acres are located within the Nooksack Silver Watershed in the northwest region of the City; this area includes Silver Creek, Tennant Creek, Bear Creek, and Lost Lake. Approximately 4,700 acres of the sewer service area in the north area of the City is within the Squalicum Creek Watershed, which includes Baker Creek, Spring Creek, McCormick Creek, Toad Creek, Upper Squalicum Creek, and Squalicum Creek. Approximately 1,300 acres of the City's service area is within the Lake Whatcom Watershed. The majority of the sewer service area is within the Bellingham Bay Watershed, which includes Whatcom Creek, Padden Creek, Chuckanut Creek, Fragrance Lake, and Padden Lake.

2.2.4-B Wetlands

The Growth Management Act defines wetlands as areas that have surface or ground water that supports vegetation typically adapted in saturated soil conditions. Wetlands support valuable and complex ecosystems and consequently development is severely restricted if not prohibited in most wetlands and buffer areas around the wetland. There are approximately 600 acres of identified wetlands (BMC 16.55.280) within the Bellingham sewer service area. Figure 2-4 shows these areas.

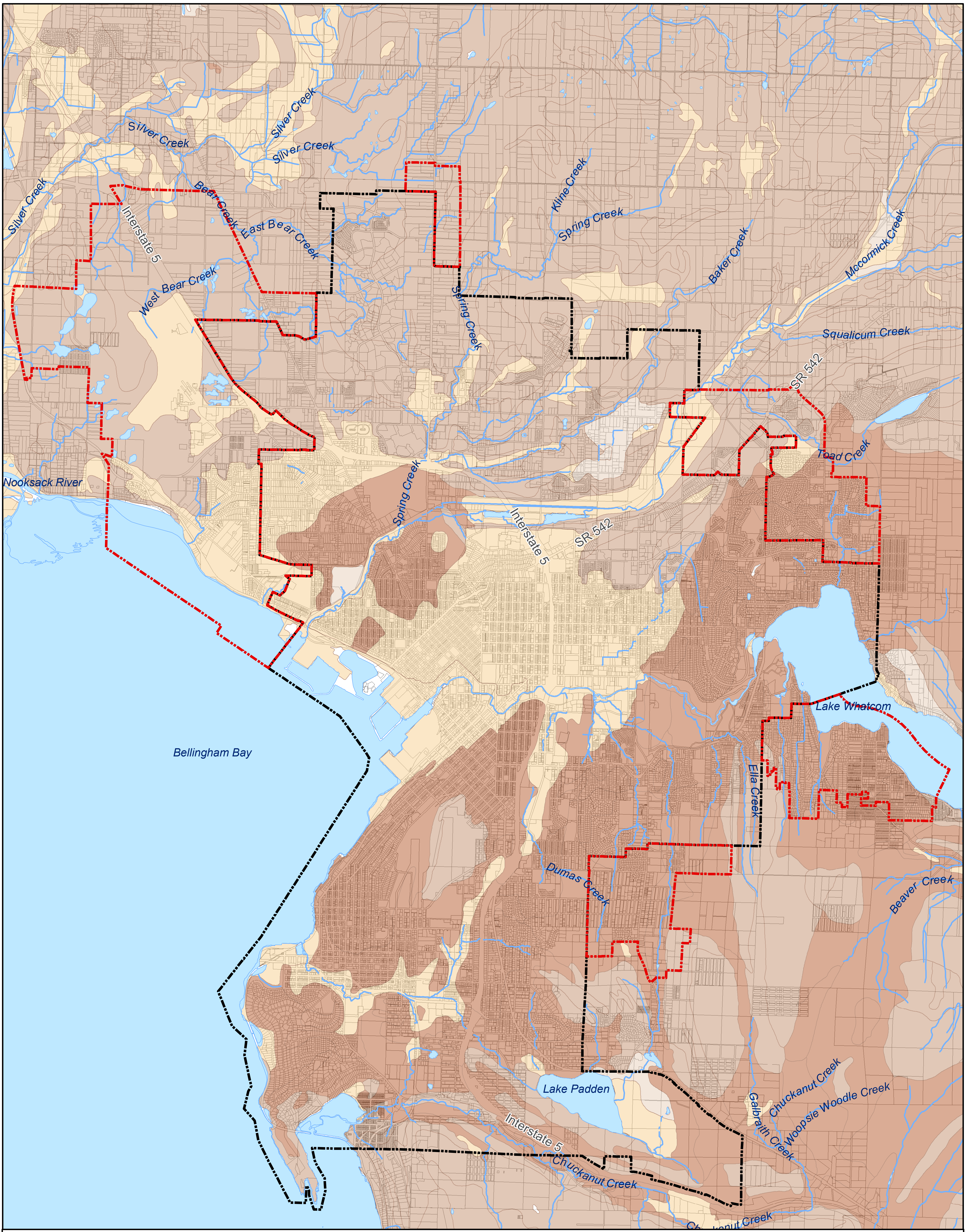

FIGURE 2-3

Legend

- City Limits
- UGA Boundary
- Streams
- Parcels

Soils

Source: National Resource Conservation Service

- Other
- Group A - Sand, Loamy Sand, Sandy Loam
- Group B - Silt Loam, Loam
- Group C - Sandy Clay Loam
- Group D - Clay Loam, Silty Clay Loam, Sandy Clay, Silty Clay, Clay

0 2,500 5,000 10,000 Feet

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WASTEWATER CONVEYANCE PLAN
SOILS

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L:\Bellingham\15448.00 WW Conveyance Plan\Report\Figure 4 soils 2.mxd

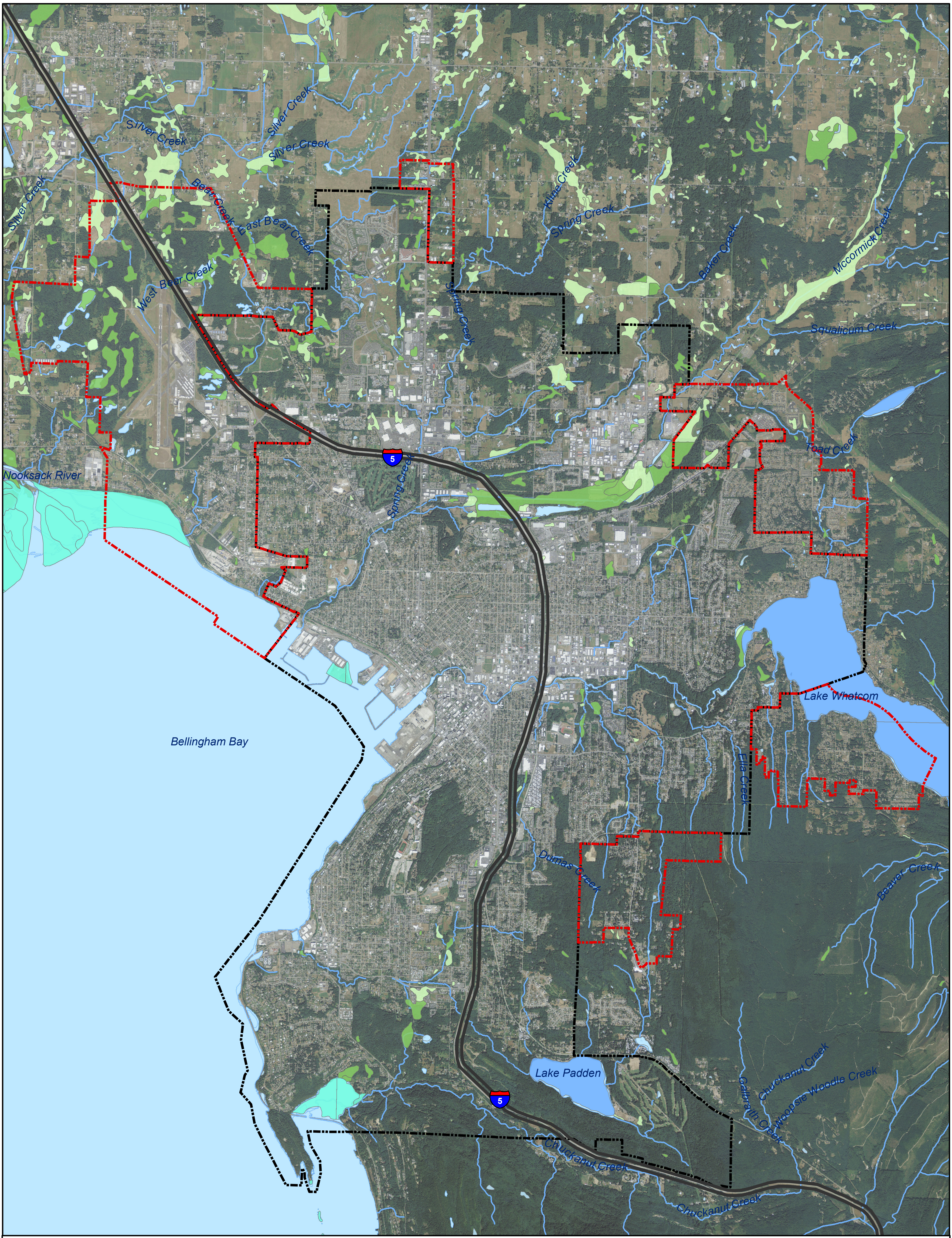

FIGURE 2-4

Legend

- City Limits
- UGA Boundary
- Streams
- Estuarine and Marine Wetland
- Freshwater Emergent Wetland
- Freshwater Forested/Shrub Wetland
- Freshwater Pond
- Lake
- Other
- Riverine

N

0 2,500 5,000 10,000 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
WETLANDS

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L:\Bellingham\15448.00 WW Conveyance Plan\Report\Figure 5 wetlands 2.mxd

2.2.4-C Frequently Flooded Areas

Flood hazard areas are regions adjacent to lakes, rivers, and streams that are prone to flooding during peak runoff periods. Construction of buildings and other development in these areas is regulated in accordance with Bellingham Municipal Code section 17.76. The City's 100-year flood zones (i.e., land that will have a flood event where there is a 1 percent chance of being equaled or exceeded in any given year) are depicted in Figure 2-5.

Because a portion of the City's sewer system is a CSS, it is possible that excess stormwater runoff and flooding could cause the system to exceed capacity and result in CSO events at the City's C Street CSO facility.

2.2.4-D Critical Aquifer Recharge Areas

Aquifer recharge areas are regions that have a critical effect on aquifers used for potable water as defined by WAC 365-190-030(2). The City of Bellingham draws most of its water from Lake Whatcom, which also supplies approximately half of Whatcom County, and additional water from the Middle Fork Nooksack River. However, in the case of private water purveyors within the City who use groundwater wells for their supply, critical aquifer recharge areas are important areas to protect.

2.2.4-E Geologically Hazardous Areas

Seismic hazard areas are those with low-density soils that are more likely to experience greater damage due to seismic-induced subsidence, liquefaction, or landslides. Seismic hazard areas are regulated mainly with respect to public safety, and, with the exception of a severe earthquake, these hazard areas do not typically impact wastewater facilities.

Erosion and landslide hazard areas are regulated under Section 16.55 of the Bellingham Municipal Code.

2.2.4-F Fish and Wildlife Habitat Conservation Areas

Sensitive fish and wildlife habitat areas are defined by WAC 365-190-080(5) and are essential for maintaining specifically listed species in suitable habitats. Any proposed activity within 300 feet of these areas requires that a habitat assessment be prepared. The waters in the area are home to a number of species of trout and salmon, both year-round and seasonally along with a variety of small mammals, Bellingham Municipal Code Section 16.55.470 designates fish and wildlife habitat conservation areas within the City.

2.3 BELLINGHAM WATER SYSTEM

The City of Bellingham owns and operates a water system consisting of intakes from Lake Whatcom and the Middle Fork Nooksack River, a water treatment plant, pump station, reservoirs, and a distribution system.

The water system serves almost the entire City limits and UGA. Service can be provided to any location once annexed into the City, provided the developer or potential customers agree to annexation prior to service being installed. Several community water associations exist within the City's sewer service area which own, operate, and maintain their own water system infrastructure.

The City of Bellingham currently has municipal water rights issued by the Department of Ecology (Ecology) that require the water level in Lake Whatcom to remain below elevation 314.94 feet, resulting in a maximum storage capacity of the reservoir of about 29,700 acre-feet. The City's water rights include a withdrawal of 82 mgd and an annual volume of 92,000 acre-feet from the Lake. The City also has two water rights for diversions from the Middle Fork Nooksack River for 81 mgd and three water rights for Lake Padden for a storage volume of 780 acre-feet.

2.4 PLANNING PERIOD

In order to provide wastewater services for future growth, the wastewater system is in need of continuous evaluation and improvement. A planning period for the evaluation of the wastewater utility should be long enough to be useful for an extended period of time, but not so long as to be impractical. The planning period for this *2016 Plan* includes a 20-year planning interval, up to Year 2036.

2.5 EXISTING LAND USE AND ZONING

The *2009 Sewer Comp Plan* outlined the City of Bellingham's zoning and land use at the time. Table 2-2 shows a summary of existing land use within the UGA. Residential land use makes up about 51 percent of Bellingham's total land area. Industrial use makes up approximately 15 percent of Bellingham's total land area. Figure 2-6 shows the zoning within the City and the UGA.

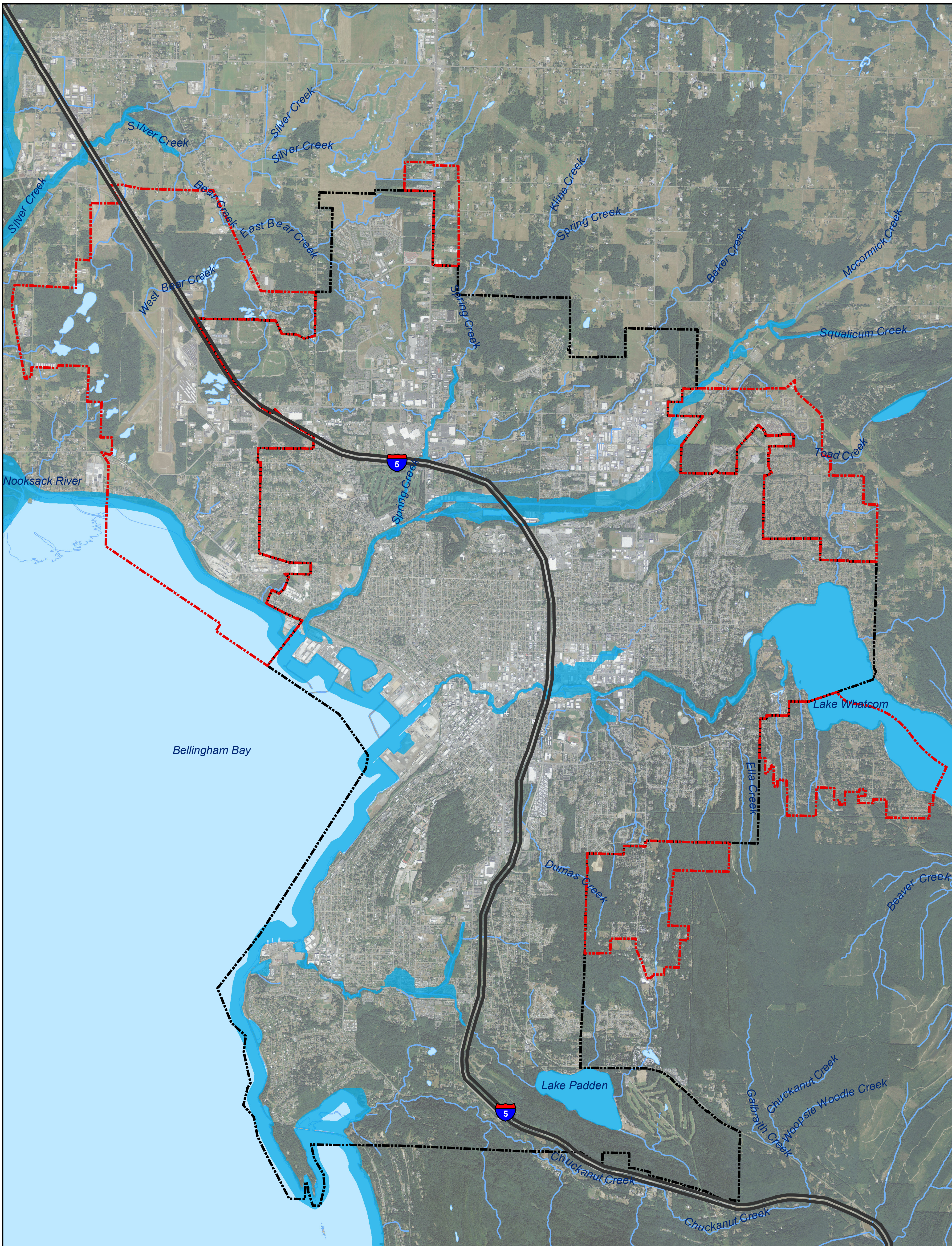

FIGURE 2-5

Legend

City Limits

UGA Boundary

100 Year Flood Zones

Streams

N

0 2,500 5,000 10,000 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
100-YEAR FLOODPLAIN

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L:\Bellingham\15448.00 WW Conveyance Plan\Report\Figure 7 flood.mxd

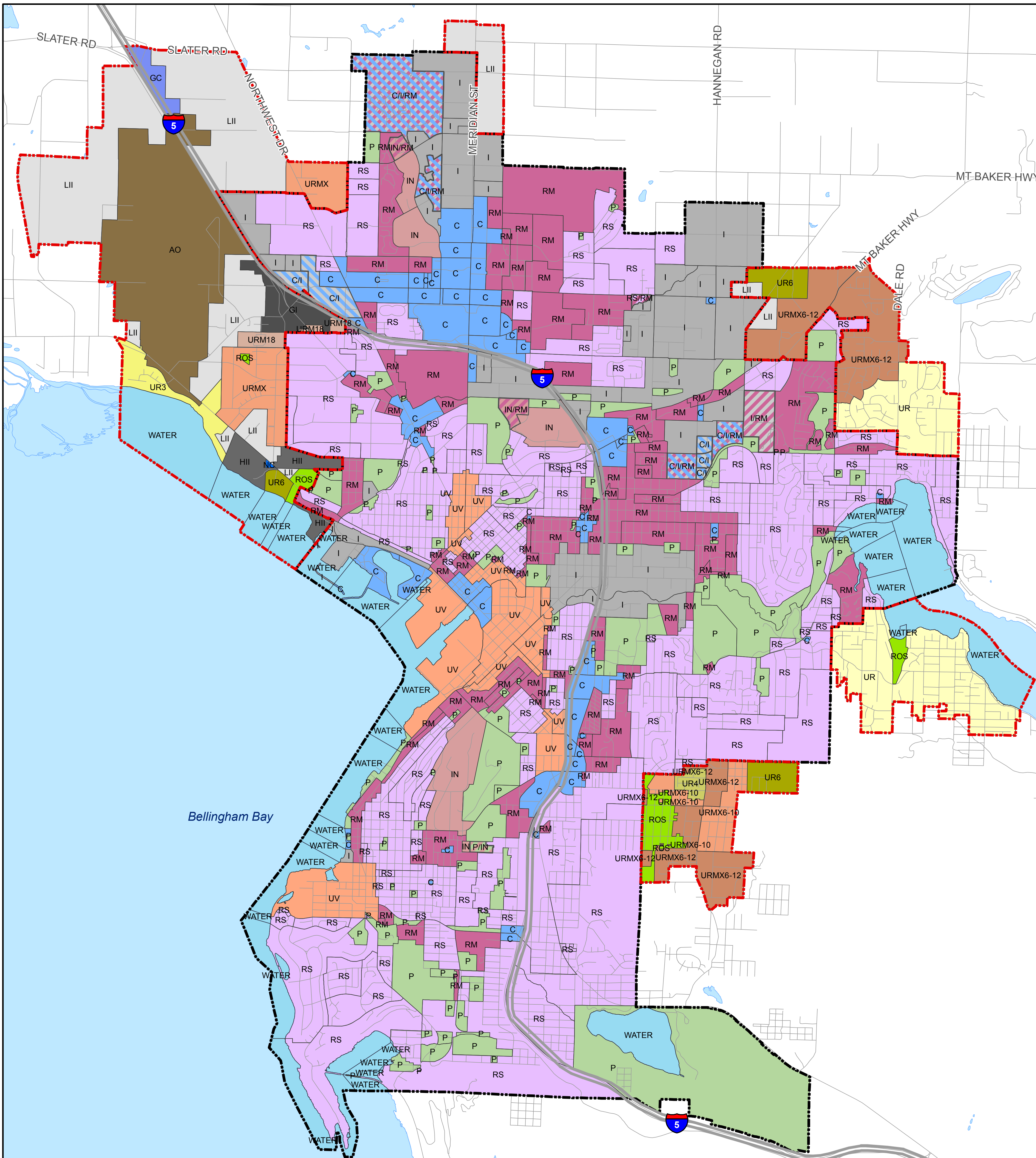

FIGURE 2-6

Legend

- AO - AIRPORT OPERATIONS
- C - COMMERCIAL
- C/I - COMMERCIAL/INDUSTRIAL
- C/I/RM - COMMERCIAL/INDUSTRIAL/RES MULTI
- GI - GATEWAY INDUSTRIAL
- GC - GENERAL COMMERCIAL
- HII - HEAVY IMPACT INDUSTRIAL
- I - INDUSTRIAL
- I/RM - INDUSTRIAL/RESIDENTIAL MULTII
- IN - INSTITUTIONAL
- IN/RM - INSTITUTIONAL/RESIDENTIAL MULTI
- LII - LIGHT IMPACT INDUSTRIAL
- NC - NEIGHBORHOOD COMMERCIAL
- P - PUBLIC
- P/N - PUBLIC/INSTITUTIONAL
- ROS - RECREATION OPEN SPACE
- RM - RESIDENTIAL MULTI
- RS - RESIDENTIAL SINGLE
- RS/RM - RESIDENTIAL SINGLE/RES MULTII
- UR1 - UR 1 UNIT PER 5 ACRES
- UR3 - UR 3 UNITS PER ACRE
- UR4 - UR 4 UNITS PER ACRE
- UR6 - UR 6 UNITS PER ACRE
- UV - URBAN VILLAGE
- URM18 - URM 18 UNITS PER ACRE
- URMX - URMX 6-10 UNITS PER ACRE
- URMX - URMX 6-12 UNITS PER ACRE
- WATER
- City Boundary
- UGA Boundary
- Streets

N

0 2,500 5,000 10,000
Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
ZONING

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L:\Bellingham\15448.00 WW Conveyance Plan\Report\Figure 6 zoning.mxd

TABLE 2-2

City of Bellingham UGA Land Use (2014)[1]

Land Use Designation	Area (Acres)	Percentage
Commercial	1,050	4.6%
Industrial	1,986	8.6%
Institutional	392	1.7%
Urban Village	865	3.8%
Single-Family Residential	7,797	33.9%
Multi-Family Residential	2,753	12.0%
Public	2,373	10.3%
Single/Multi-Family Residential	7	0.0%
Commercial/Industrial	112	0.5%
Comm/Indust/Multi-Family Residential	343	1.5%
Industrial/Multi-Family Residential	70	0.3%
Institutional/Multi-Family Residential	41	0.2%
Public/Indtitutional	9	0.0%
UGA Airport Ops	1,024	4.5%
UGA General Commercial	56	0.2%
UGA Neighborhood Comm	2	0.0%
UGA Gateway Industrial	125	0.5%
UGA Light Industrial	1,470	6.4%
UGA Heavy Industrial	122	0.5%
UGA Single-Family Resid	1,181	5.1%
UGA Multi-Family Resid	46	0.2%
UGA Urban Resid Mixed	1,025	4.5%
UGA Recreation Open Space	139	0.6%
Total	22,989 (36 sq mi)	100.0%

(1) Source: City of Bellingham

2.5.1 CURRENT POPULATION

Table 2-3 provides a history of population for Bellingham over a 15-year period, 2000 to 2014 based on data from the Washington State Office of Financial Management (OFM) and the US Census Bureau. Since the OFM data for housing units is currently available only to 2010, data since then has been projected using population data to calculate persons per household (pph). In 2010, OFM estimated the total number of housing units in Bellingham and in the unincorporated UGA to be 36,757 (with 2,088 unoccupied), and 4,237 (with 168 unoccupied), respectively. Bellingham's 2010 population within the city limits was estimated to be 80,885. This equates to an average population of 2.2 pph.

TABLE 2-3

City of Bellingham Historical Population 2000 to 2014

Year	Population	Growth
2000	67,171	
2001	69,487	3.45%
2002	70,823	1.92%
2003	70,880	0.08%
2004	72,489	2.27%
2005	73,963	2.03%
2006	75,562	2.16%
2007	77,948	3.16%
2008	78,739	1.01%
2009	79,383	0.82%
2010	80,885	1.89%
2011	81,070	0.23%
2012	81,360	0.36%
2013	82,310	1.17%
2014	82,810	0.61%
Average		**1.51%**

SOURCE: Washington State Office of Financial Management (OFM) and US Census Bureau. Population estimated as of April 1st of each year.

2.6 PROJECTED FUTURE CITY POPULATION

The City of Bellingham is currently in the process of completing a new comprehensive plan, which will include establishing a population growth rate for future growth in the City. However, based on discussions with the City, annual growth rates of 1.4 percent have been used for this *2016 Plan* for population within the City limits and within the remainder of the UGA. This growth rate is consistent with the City's existing *Water System Plan.* The City will evaluate water/sewer service to properties outside the UGA on a case-by-case basis, but in general does not serve new development outside the UGA. Therefore, it will be assumed for the purposes of this report that all service is within the UGA. These estimates are representative and reasonably conservative growth rates for the projection of future wastewater flows and loadings. Table 2-4 shows projected population totals for the service area through 2036, assuming a 1.4 percent growth rate.

TABLE 2-4

City of Bellingham Projected Population for the Sanitary Sewer Service Area

Year	Bellingham Population(1)
2013	93,107
2014	93,090
2015	93,940
2016	95,196
2017	96,469
2018	97,758
2019	99,065
2020	100,390
2021	101,732
2022	103,092
2023	104,470
2024	105,867
2025	107,282
2026	108,716
2027	110,170
2028	111,642
2029	113,135
2030	114,648
2031	116,180
2032	117,733
2033	119,307
2034	120,902
2035	122,519
2036	124,157

(1) Source: City of Bellingham

2.7 SEWER CONNECTIONS

Table 2-5 provides an estimate of the number of sewer connections to the City of Bellingham in 2014, based on billing data received from the City.

TABLE 2-5

City of Bellingham Sewer Service Connections by Customer Class

Customer Class	2014
Accessory Dwelling Unit	51
City	56
College	63
Commercial(1)	1805
County	11
Duplex	954
Duplex 2 (two services single lot)	61
Fireline	34
Hotel	42
Industrial	42
Institution	88
Irrigation	0
Multiple Building Condo Site (1 meter)	109
Multiple Building Condo Site (2 meters)	102
Multiple Building Condo Site (individual meters)	289
Multi-Family Residential(2)	1178
School	34
Single-Family Residential	18560
Water District	2
TOTAL	**23,481**

(1) Commercial connections also include churches, schools, City accounts, and motels

(2) Multi-family connections include apartments, RV parks, and mobile home parks.

CHAPTER 3

EXISTING FACILITIES

3.1 WASTEWATER COLLECTION SYSTEM

3.1.1 LIFT STATIONS

The City of Bellingham operates 29 lift stations and approximately 6 miles of force mains. Station locations and force main routes are shown on Figure 3-1 and shown in a simple schematic in Figure 3-2. The lift stations were installed between 1966 and 2010; some have been repaired or rehabilitated more recently. All of the lift stations include two pumps with the exception of the Roeder lift station (three pumps) and the Oak Street lift station (five pumps). Each lift station is also tied into a Supervisory Control and Data Acquisition (SCADA) system. The design capacities of the lift stations range from 75 gpm to 15,000 gpm; the majority have capacities below 2,000 gpm. Most of the City's wastewater is conveyed through the largest lift station, the Oak Street station, which is located near the waterfront along the Champion/Silverbeach Trunk, which conveys wastewater to the Post Point Wastewater Treatment Plant on McKenzie Avenue. Design and tested lift station capacities are provided in Table 3-1.

A technical memorandum prepared in December 2013 by Parametrix reviewed the City's lift stations for their characteristics and specific capacity or reliability issues (see Appendix E). The main issue for many of the lift stations in the City is limited pumping capacity. Ten of the lift stations were identified as being near or at their pumping capacities in 2013. Other issues with the City's lift stations include long residence times in force mains, low or high force main velocities, high pump run times, ragging, rust, and general aging. A number of the City's lift stations are approaching or have reached their design life. The oldest lift stations in the City are approximately 49 years old, while the newest is 5 years old.

TABLE 3-1

City of Bellingham Lift Stations and Force Mains

	Pump Station	Type of Pump	Sewer Area	Basin No.	Year Updated/ Installed	Pump Motor Size(1) (hp)	Tested Station Capacity(1) (gpm)	Design Station Capacity(1) (gpm)
1	48th Street (Lake Padden)	Dry/Wet Well	Separated	South Side	1973	15	606	600
2	Arbutus	Surface Mount Wet Well	Separated	South Side	1981	15	94	75
3	Bakerview Valley	Submersible	Separated	Sunset/Mt. Baker	1996	10	137	260
4	Britton Loop		Separated	Sunset/Mt. Baker	Post-1998	5	407	450
5	Briza Court	Submersible	Separated	South Side	1986	88	711	600
6	C Street	Dry/Wet Well	Combined	Broadway	1973	5	372	500
7	Fir	Dry/Wet Well	Separated	Lake Whatcom	1977	7.5	143	150
8	Flynn	Dry/Wet Well	Separated	Lake Whatcom	1966	20	521	500
9	Hilton	Dry/Wet Well	Combined	Central	1973	15	920	900
10	Horton	Submersible	Separated	Cordata/Meridian	1988	20	638	600
11	James Street	Submersible	Separated	Sunset/Mt. Baker	1985	88	1,728	1,200
12	June Road	Submersible	Separated	Cordata/Meridian	2010	7.5	238	440
13	Kline Road	Submersible	Separated	Cordata/Meridian	2014	5	--	463
14	Lakeside	Submersible	Separated	Lake Whatcom	2006/1968	7.5	88	137
15	Martin	Dry/Wet Well	Separated	Lake Whatcom	1973	10	615	700
16	Meadowbrook Court	Submersible	Separated	Cordata/Meridian	1997	7.5	198	150
17	Mitchell Way	Submersible	Separated	Northwest	1972	10	158	350
18	North Mitchell Way		Separated	Northwest	Post-1998	7.5	265	525
19	Northshore	Dry/Wet Well	Separated	Lake Whatcom	1980	75	1,580	1,500
20	Northern Meadows		Separated	Sunset/Mt. Baker	Post-1998	3	94	102
21	Oak Street	Dry/Wet Well	Combined	Central	2005/1974	300/600	--	50,000+
22	Edgemoor	Dry/Wet Well	Separated	South Side	1966	15	147	150
23	Pine Street	Submersible	Combined	Central	1977	3	?	?

TABLE 3-1 – (continued)

City of Bellingham Lift Stations and Force Mains

	Pump Station	Type of Pump	Sewer Area	Basin No.	Year Updated/ Installed	Pump Motor Size[(1)] (hp)	Tested Station Capacity[(1)] (gpm)	Design Station Capacity[(1)] (gpm)
24	Roeder	Dry/Wet Well	Combined	Birchwood	2004/1976	75/50	--	3,100/4,000
25	Shorewood	Submersible	Separated	South Side	2008/1979	5	58	350
26	Silver Beach	Dry/Wet Well	Separated	Lake Whatcom	1973	7.5	655	800
27	West Bakerview	Submersible	Separated	Northwest	1998	10	244	350
28	West Maplewood	Dry/Wet Well	Separated	Northwest	1973	5	441	350
29	Willow Road	Submersible	Separated	South Side	1985	47	346	500

(1) Data from Parametrix "Pump Station Evaluation" memorandum, December 2013. Lift station capacities are noted with one (redundant) pump out of service.

3.1.2 OVERFLOW STRUCTURES

The City has one CSO facility at C Street, in the downtown area. Previously, the City had four CSO locations, but upgrades to the sewer system and stormwater separation projects in the 1970s and 1980s reduced the number of CSO points to one. These projects also led to a reduction in the frequency of CSO events. The remaining C Street overflow is located at the downstream end of the Birchwood Trunk and is also connected to the Champion/Silverbeach Trunk. The structure was rehabilitated in 2005, when the weir elevation was raised to accommodate more storage.

The overflow structure consists of an 8.5-foot-wide by 14-foot-long precast vault installed in line with the 48-inch Birchwood Trunk. The sharp-crested, adjustable, side channel weir is set at an elevation of 16.4 feet and extends the entire 14-foot length of the vault. The inflow pipe invert is at an elevation of 9.06 feet. The two 36-inch-diameter overflow outlet pipes connect to an abandoned storm drainage tunnel at an elevation of 9.3 feet. The tunnel discharges CSO events to the southwest, downstream to Bellingham Bay. The last CSO event at this location occurred in 2009.

3.1.3 COLLECTION SYSTEM

Gravity sewer lines in downtown Bellingham were first constructed in the 1890s. Many of these original pipes are still in place and were mainly constructed of vitrified clay. As the City has expanded sewer service and replaced aging pipes, polyvinyl chloride (PVC) and high density polyethylene (HDPE) pipes have been installed to reduce infiltration and improve the condition of the sanitary sewer system. The City’s 318 miles of gravity sewer mains are composed of 6-inch to 60-inch-diameter pipes, the majority of which are between 8 inches and 12 inches in diameter.

Figure 3-1 shows the existing sewer system. Wastewater is discharged to the City’s Post Point Wastewater Treatment Plant, which has an outfall in Bellingham Bay. The sewer system service area is generally hilly and gradually slopes toward the southwestern portion of the City, where the treatment facility is located. Most of the collection system consists of gravity sewers, which drain to lift stations, and force mains.

A summary of the various pipe diameters within the City’s sewer system are provided in Table 3-2, based on data in the City’s *2009 Plan*.

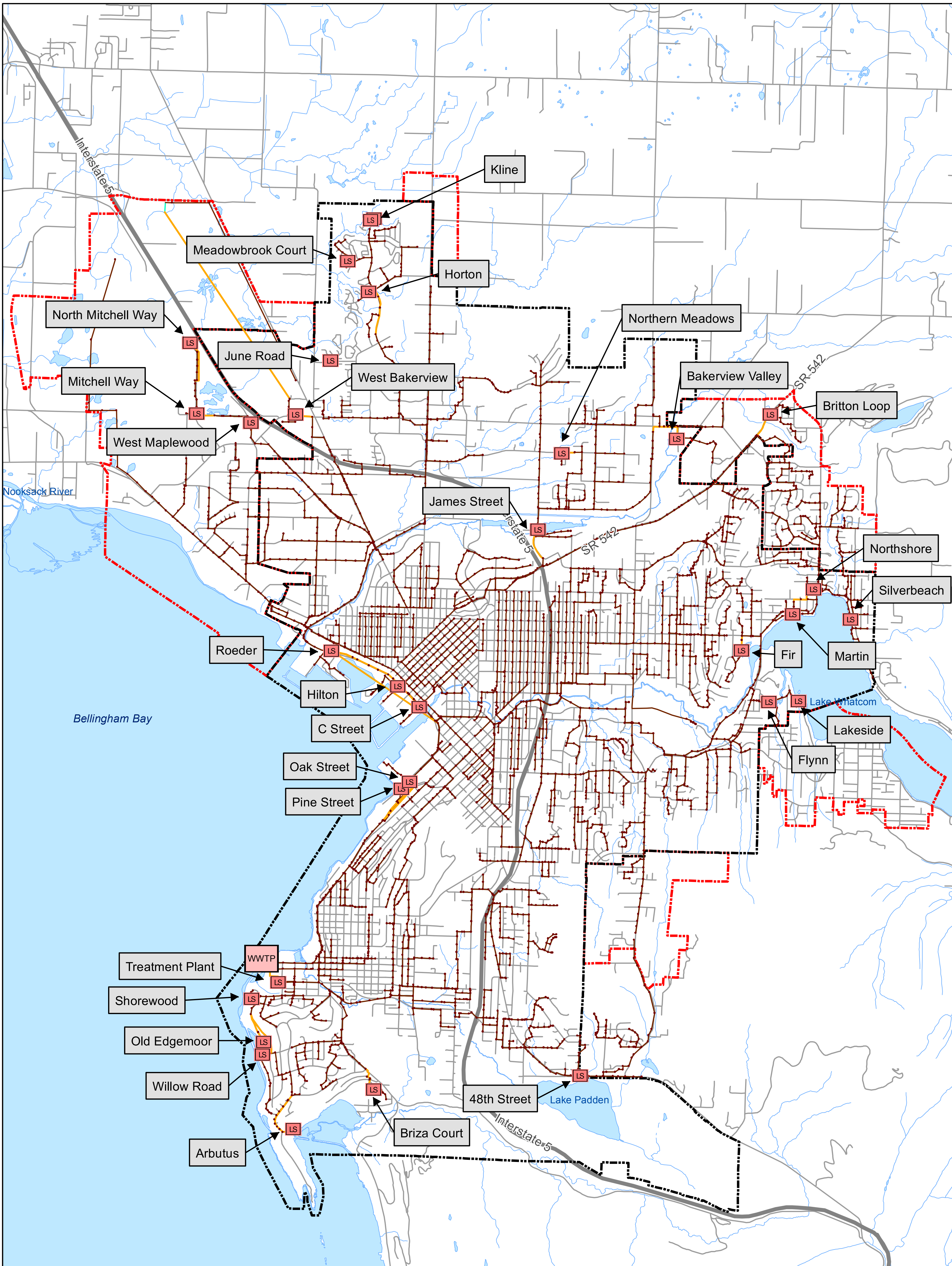


FIGURE 3-1

Legend

Existing Lift Station	City Limits
Manholes	UGA Boundary
Sewer Pipes	
Force Mains	

N

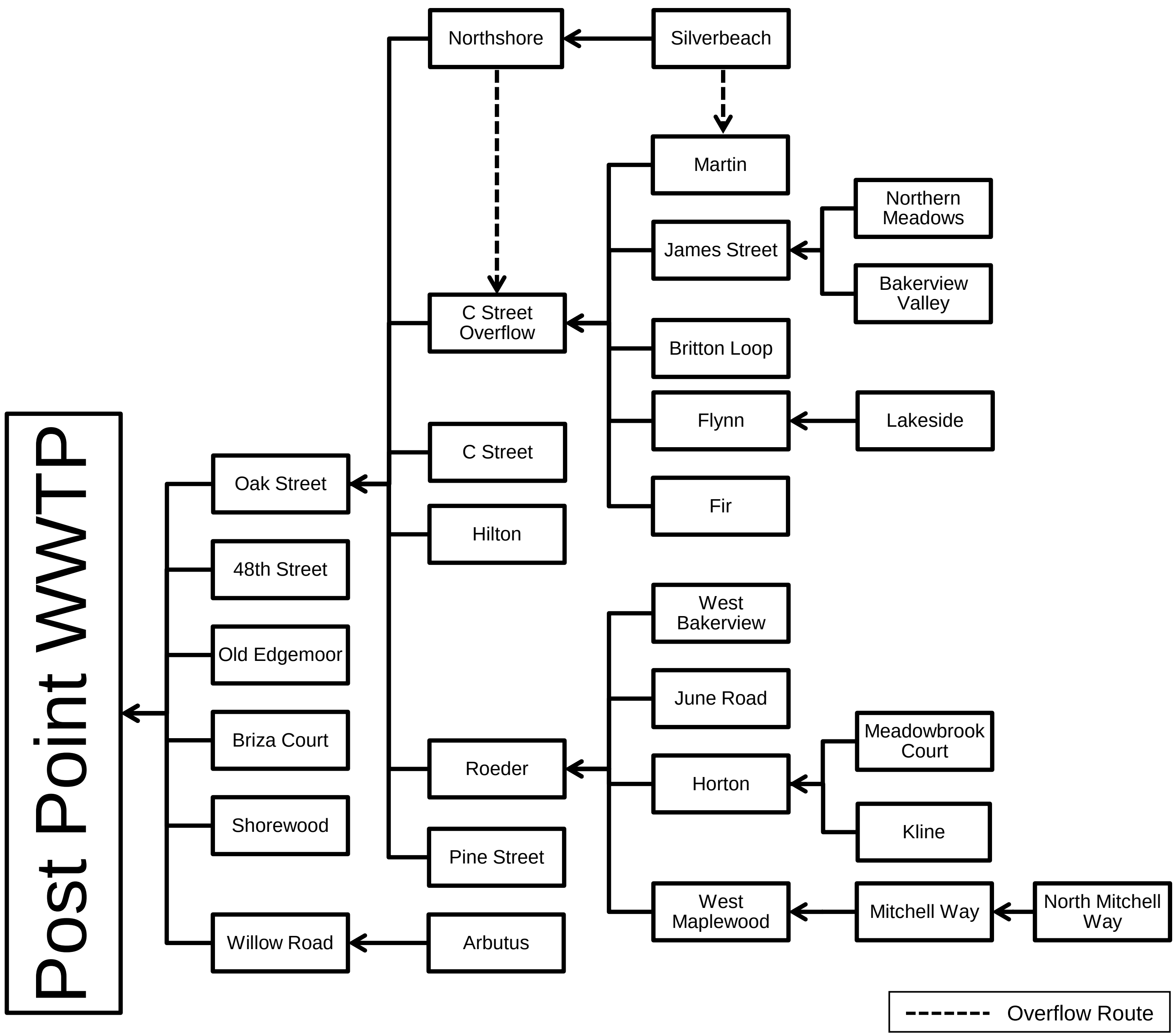
0 2,100 4,200 8,400 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
SANITARY SEWER CONVEYANCE SYSTEM

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\Figure 3. sewer map.mxd



CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
LIFT STATION FLOW
SCHEMATIC

Gray & Osborne, Inc.

FIGURE 3-2

TABLE 3-2

Sewer Pipe Summary, Bellingham Collection System

Pipe Diameter(1)	Length(1) (miles)	Number of Segments(1)
6 to 8 inches	238	5,659
10 to 18 inches	58	1,188
20 to 28 inches	11	198
30 inches +	8	161
Force Mains(2)	8	25
Total	**323**	**7,231**

(1) Data from City of Bellingham sewer system map, updated December 2014.
(2) Force main diameters range from 4 to 42 inches; one segment per force main.

The City's collection system contains four main sewer trunks and two interceptors that convey flows to the WWTP. The trunks include reinforced concrete (RCP) and PVC pipes from 8 inches to 60 inches in diameter and were built between 14 and 104 years ago. The Marine Drive Trunk is located in the northwest portion of the City and runs along Patton Street to Bancroft and C Street. The Squalicum Trunk is located in the north of the City and follows Meridian Street to Squalicum Parkway. The Champion/Silverbeach Trunk is located in the east portion of the City, starting at Lake Whatcom and following Silverbeach to Whatcom Creek and Champion Street. The Happy Valley Trunk is located in the south region of the City and follows 40th Street and Harrison Street, then 10th Street and West Fairhaven Parkway. The Waterfront Interceptor runs parallel to Bellingham Bay to the Donovan Avenue Interceptor, collecting flows from the Birchwood, Squalicum, and Champion/Silverbeach Trunks. The Donovan Avenue Interceptor follows Donovan Avenue to the WWTP, collecting flows from the Waterfront Interceptor and the Happy Valley Trunk.

The Bellingham sanitary sewer system contains a total of approximately 6,563 manholes. These manholes vary in construction material from all-brick to the newer precast concrete manholes. The older, all-brick manholes present a greater opportunity for infiltration to occur, due to the mortar joints between the bricks or concrete blocks, than the newer precast manholes.

TABLE 3-3

Sewer Trunks and Interceptors[3]

Interceptor/ Trunk	Diameter (in)	Length (ft)	Slope (avg)	Capacity[1] (gpm)	Manholes	Year Installed	Age (yr)	Material[2]
Marine Drive	12-48	13,695	0.48%	12,977	58	1924, 1958, 1959, 1961, 1974, 1999	13-88	CI, RC
Champion/ Silverbeach	15-48	21,030	1.46%	29,919	83	1908, 1909, 1980, 1991, 1997	15-104	RC, PVC
Donovan	60	1,757	0.4%	30,498	5	1974	38	RC
Happy Valley	8-30	13,392	3.2%	10,528	54	1948, 1949, 1966, 1973, 1975 – 1979, 1990, 1998	14-64	RC, PVC
Squalicum	186	12,712	0.8%	1,521	50	1970, 1983 – 1986, 1993	19-42	RC, PVC
Waterfront (upstream of Oak Street)	30-60	8,347	0.15%	47,336	27	1908, 1971, 1973, 1974, 1975, 1978, 1980, 1983	29-104	RC, DI, PVC
Waterfront (downstream of Oak Street)	48-60	8,717	0.6%	27,386	22			

(1) Limiting capacity calculated as d/D = 2/3 for the downstream segment of the trunk/interceptor; capacity decreases upstream with decreasing pipe size.
(2) CI = cast iron, RC = reinforced concrete, PVC = polyvinyl Chloride, DI = ductile iron.
(3) Data from City of Bellingham 2009 *Sewer Comprehensive Plan*.

3.1.4 COLLECTION AREAS

The Bellingham collection system is divided into eight different collection areas, or drainage basins. These collection areas predominantly follow the natural drainage patterns of the service area with each area containing lift stations to eventually transmit flow to the WWTP. The eight major sewer basins are summarized in Table 3-4 and are shown in Figure 3-3.

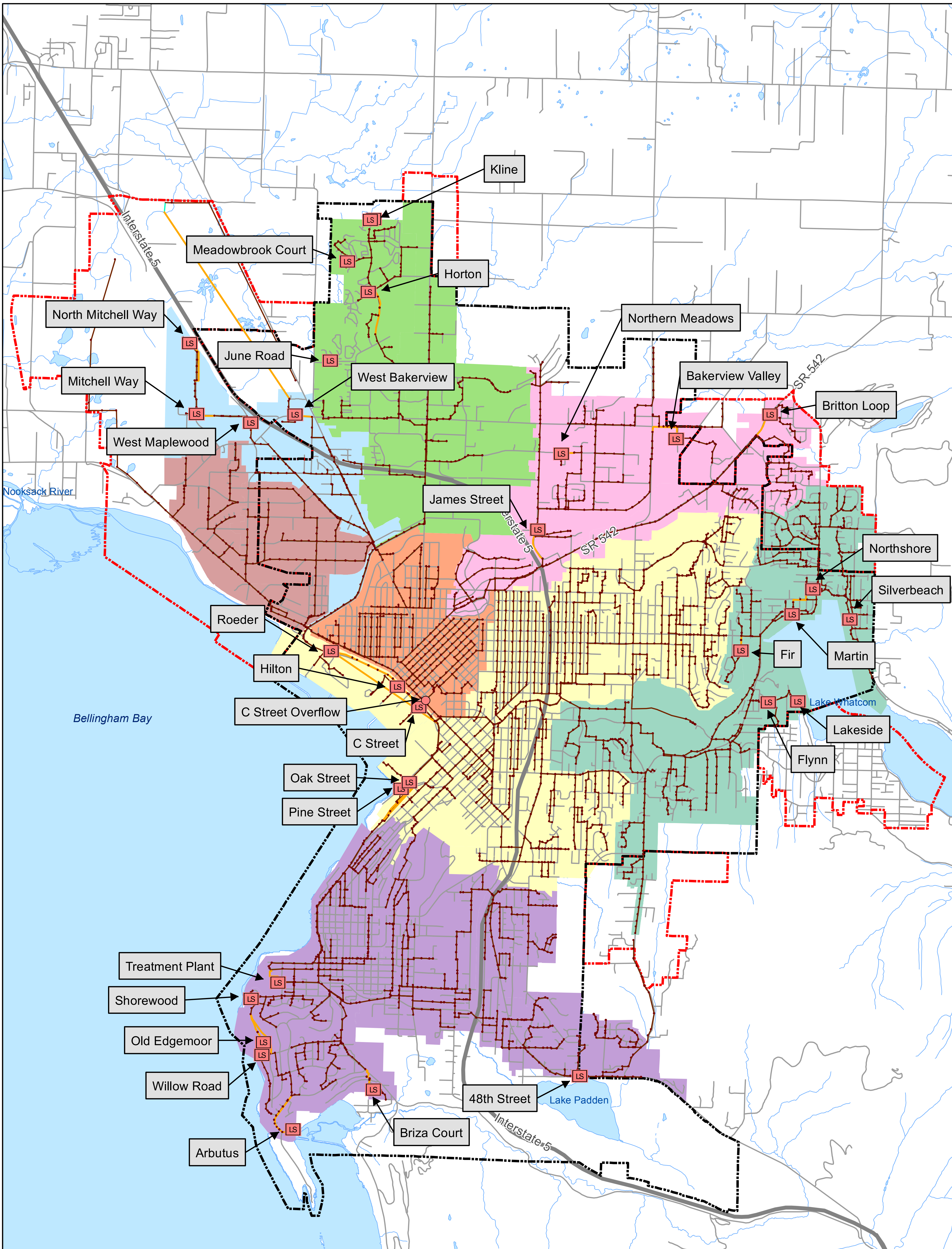


FIGURE 3-3

Legend

BIRCHWOOD
BROADWAY
CENTRAL
CORDATA/MERIDIAN
LAKE WHATCOM
NORTHWEST
SOUTH SIDE
SUNSET/MT.BAKER
Existing Lift Station
Manholes
Force Mains
Sewer Pipes
City Limits
UGA Boundary

N

0 2,500 5,000 10,000 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
SANITARY SEWER BASINS

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\Figure 3.3 basins.mxd

TABLE 3-4

Sewer Basins Summary

Basin	Area (ac)	Sewer Pipe Length (ft)
Birchwood	1,138	96,163
Broadway	865	145,610
Central	3,921	468,553
Cordata/Meridian	2,383	121,743
Lake Whatcom	2,165	229,751
Northwest	1,389	61,018
South Side	5,348	393,356
Sunset/Mt. Baker	3,002	166,343
Total	**20,211**	**1,682,537 (318.7 mi.)**[(1)]

(1) Some pipes are located outside of the City-defined basin areas, as seen on Figure 3-3.

The following section describes the boundaries and land use designations of each basin as well as information about the sewer lines within each basin.

3.1.4-A Birchwood

Birchwood consists of an area of about 1,138 acres in the northwest of Bellingham west of the I-5 corridor. Approximately half of the basin is within the City's limits, while the remainder is within the UGA. Birchwood includes areas primarily designated for single-family residential use and small areas for multi-family residential and public uses. Wastewater from this area flows to the southeast through Broadway and Central via the Birchwood Trunk and Waterfront Interceptor. Birchwood contains the Roeder Lift Station. This basin includes part of the City's combined sewer system.

3.1.4-B Broadway

Broadway is located to the southeast of Birchwood, separated by Squalicum Creek, and consists of about 865 acres. The basin is entirely within City limits. Broadway includes areas designated primarily for single-family residential and urban village use, as well as small areas of commercial and public use. Wastewater from this basin flows to the southeast via the Birchwood Trunk and Waterfront Interceptor through Central and South Side. Broadway contains the C Street Lift Station. The majority of this basin consists of combined sewers.

3.1.4-C Central

Central consists of an area of about 3,921 acres, spanning the central western and northeastern portions of the City, including Squalicum Harbor and Western Washington University. The basin is entirely within City limits. Central includes areas designated for single- and multi-family residential, urban village, institutional, commercial, and industrial uses. Wastewater from this basin flows west via the Champion/Silverbeach Trunk and south via the Waterfront Interceptor through South Side. Central contains the Hilton, Oak Street, and Pine Lift Stations. This basin includes part of the City's combined sewer system.

3.1.4-D Cordata/Meridian

Cordata/Meridian is located in the northernmost part of the City, north of the I-5 Corridor, consisting of an area of about 2,383 acres, including Whatcom Community College. Most of the basin is located within City limits, though a small portion is located outside, within the UGA. Cordata/Meridian includes areas primarily designated for commercial and industrial use, with areas of single and multi-family residential and institutional zones. Wastewater from this area flows to the south through Broadway Central, and South Side via the Squalicum Trunk and the Waterfront Interceptor. Cordata/Meridian contains the Horton, June Road, Kline Road, and Meadowbrook Court Lift Stations.

3.1.4-E Lake Whatcom

Lake Whatcom consists of an area of about 2,165 acres to the east of Central and borders the northwest shores of Lake Whatcom. The majority of the basin is located within the City limits. Lake Whatcom includes almost entirely single-family residential uses, with small areas of multi-family residential and public uses. Wastewater from this area flows to the west via the Champion/Silverbeach Trunk and south through the Waterfront Interceptor through Central and South Side. Lake Whatcom contains the Fir, Flynn, Lakeside, Martin, North Shore, and Silver Beach Lift Stations. This basin includes a small part of the City's combined sewer system.

3.1.4-F Northwest

Northwest consists of an area of about 1,389 acres and is located in the northwestern-most part of the City, north of Birchwood. Northwest includes primarily industrial and commercial zones, including Bellingham International Airport, with some single family residential area. Wastewater from this area flows south through Broadway via the Squalicum Trunk and through Central and South Side via the Waterfront Interceptor. Northwest contains the Mitchell Way, North Mitchell Way, West Bakerview, and West Maplewood Lift Stations

3.1.4-G South Side

South Side is the largest basin with an area of approximately 5,348 acres, spanning the entire southern portion of the City, south of Central. South Side is comprised of mostly single-family residential areas, though it includes some multi-family residential, urban village, and public use areas, as well as small areas of commercial and industrial use. Wastewater from this area flows south and west to the WWTP, also located in this basin, via the Waterfront Interceptor. South Side contains the 48th Street (Lake Padden), Arbutus, Briza Court, Edgemoor, Shorewood, and Willow Road Lift Stations.

3.1.4-H Sunset/Mt. Baker

Sunset/Mt. Baker consists of an area of about 3,002 acres in northeast Bellingham, to the north of Central. Sunset/Mt. Baker includes areas designated for industrial, single-family and multi-family residential, commercial, public, and institutional use. Wastewater from this area flows southwest through Broadway and Central. Sunset/Mt. Baker contains the Bakerview Valley, Britton Loop, James, and Northern Meadows Lift Stations. This basin includes a small part of the City's combined sewer system.

CHAPTER 4

COLLECTION SYSTEM HYDRAULIC EVALUATION

4.1 INTRODUCTION

This chapter presents an analysis of the City of Bellingham collection system and its ability to accommodate projected flows. The analysis includes a hydraulic/hydrologic analysis of the collection system. It was completed with hydraulic modeling software where specific consideration was given to anticipated flows outside of the City's current service area. Future population, land use and existing wastewater flows within the Urban Growth Areas (UGA) were utilized to develop data for use in the hydraulic model. Total land use area and resultant wastewater flows are allocated to individual subareas, or basins, to identify future deficiencies in the existing collection system as well as to analyze infrastructure needs for the currently unserved areas.

The components of the collection system are organized into three categories for the evaluation:

- Major Gravity Lines
- Force Mains
- Sewage Lift stations

4.2 HYDRAULIC MODEL

For the 2009 *Comprehensive Sewer Plan (2009 Plan)*, a model was developed using H2OMAP modeling software by Innovyze to evaluate the existing infrastructure under both Year 2009 and Year 2026 flow conditions. For this report, Version 7.6 of InfoSewer (formerly H2OMAP) was utilitzed; the software program was designed for steady-state and dynamic analysis of gravity flow and pressure flow pipe networks. Version 7.6 is capable of integration with GIS mapping and can do extended period simulation (EPS) modeling, which was used for this analysis. EPS allows flow attenuation to occur as flow travels downstream whereas steady state models carry a plug of flow throughout the model to the most downstream point without attenuation.

The current model builds upon the flows presented in the *2009 Plan* for the 2026 scenario by adding anticipated "buildout" flows from the UGA areas (see Figure 4-1). Therefore, it should be noted that the model prepared for this Plan does not represent a full buildout condition but rather a hybrid scenario incorporating buildout flows for the UGA regions combined with Year 2026 population estimates for the remainder of the City. For the purposes of this report, this hybrid planning basis is called "the future peak flow scenario." For the capacity analysis of the force mains and sewage lift stations, peak wet weather flows for the future condition were estimated and compared to existing pump and force main capacities. The output from the hydraulic model is used to evaluate the capacity of the existing collection system and to identify improvements that will be required to accommodate future wastewater flows. The model can be updated and maintained for use as a tool to aid in future planning and design.

4.2.1 Model Layers

The hydraulic model consists of numerous layers, each of which mimics a shapefile (or layer) utilized in GIS. Although the model layers are not specific .shp files, they can be exported as a .shp file which can be utilized in a GIS system. The layers consist of manholes, outlets, wetwells, pipes, force mains, and pumps. With the exception of the Kline Rd. and June Rd. lift stations, each of the City's 29 lift stations were included within the model with a pump curve (provided from the 2009 model) that appropriately represents the flow being discharged to the downstream manhole. Flows within each of the basins were calculated separately in an Excel spreadsheet (i.e., based on land use area, ERUs, flow estimates per ERU, and infiltration and inflow) and then input into the model at specific designated manholes.

Approximately 63 percent of the existing pipes within the entire collection system were modeled. The sections of the collection system that were modeled in the 2009 model were carried over and include all major sewer trunk lines and manholes that are expected to convey the majority of flows through the collection system. A schematic of the skeletonized system is shown in Figure 4-1. Pipe information required to construct the model was obtained from existing GIS data and/or construction/record drawings.

4.2.2 Basins

The collection system is organized around eight main sewer basins. These basins are described in Chapter 3 and are shown in Figure 3-3. In addition to these basins, 22 regions within the Urban Growth Area were specifically analyzed for this Plan. These areas are shown in Figure 4-1.

The model flow inputs for these areas of anticipated growth originated from parameters utilized within the 2009 model and from discussions with City staff. These flow parameters are shown in Table 4-1.

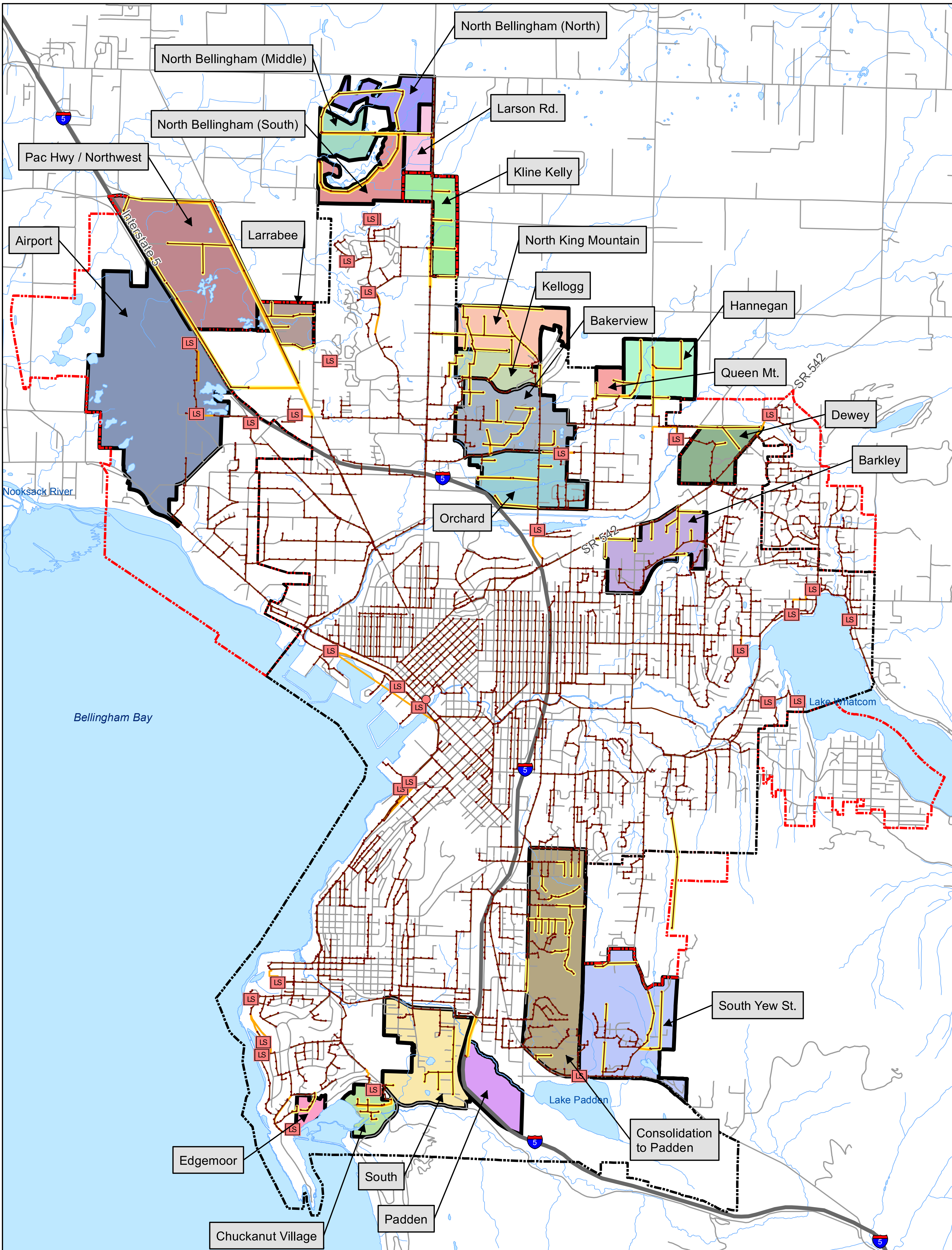


FIGURE 4-1

Legend

- Sewer MH (Modeled)
- Sewer Pipe (Modeled)
- Force Main (Modeled)
- Potential Infrastructure
- City Limits
- UGA Boundary
- LS Lift Station

N

0 2,100 4,200 8,400 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
UGA REGIONS

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\report figures\Figure 4-1 UGA areas.mxd

TABLE 4-1

2015 Model Flow Parameters

	Parameter Used
Base Flows	
Residential Flow	77 gpcd(1)
Commercial Flow	82 gpcd(2)
Base I/I *(Existing Pipes)*	1,460 gpad(3)
Base I/I *(Future Pipes)*	800 gpad(4)
Diurnal Curves	Same as 2009 Model(5)
Storm	10-Year Hypothetical(6)

(1) gpcd = gallons per capita per day; Within the 2009 model, this represented 91 percent of water meter usage minus commercial flows divided by the residential population.
(2) gpcd = gallons per capita per day; Within the 2009 model, this was based upon 91 percent of commercial water meter usage.
(3) gpad = gallons per acre day; Within the 2009 model, 1,460 gpad originated from [Measured ADWF at the treatment plant] -[91 percent water meter usage for residential, commercial and LWWSD flows].
(4) gpad = gallons per acre day; A lower value of 800 gpad was chosen to represent future new infrastructure since current construction methods and materials are less likely to be subject to infiltration.
(5) The 2009 model utilized nine different diurnal curves which were developed from flow meter data provided by the City for specific areas of the City's collection system. The 2015 model utilized these same diurnal curves for the UGA areas based upon the nearest downstream basin.
(6) The 10-year hypothetical storm from the 2009 model was chosen to be the representative storm for the 2015 model.

Using the parameters shown in Table 4-1, individual flow inputs were calculated for each of the new UGA areas as shown in Appendix A. The input into the model represented an average sanitary flow and base I/I flow. This combined flow was then peaked by a diurnal curve representative of the sewer basins downstream of the future growth areas. These diurnal curves (shown in Appendix D) were replicated from the 2009 model which had been calibrated to flow meter data available at that time. The InfoSewer software uses these diurnal curves to peak the sanitary flow at the model input point. From here, the flow is conveyed hydraulically downstream and eventually to the Post Point Wastewater Treatment Plant.

4.3 HYDRAULIC MODELING ANALYSIS

Once basin data and flow parameters were compiled, the flows were inserted into the major trunk lines of the existing sewer collection system in the hydraulic model as constructed for the *2009 Plan*. This approach was used to identify hydraulic deficiencies including surcharged pipes or pipes with low velocities. In addition, new infrastructure was added to the model to serve the UGA regions. This flow was then conveyed to the existing sanitary sewer system (see Figures 4-2 through 4-10).

Pipe deficiencies resulting from the model are included in tabular form within Appendix B. The model included a total of 4,232 nodes, or manholes, and approximately 233 miles of pipe representing 63 percent of the total collection system.

To support the development of the hydraulic model, an overall map was prepared to show the locations of the pipes and manhole identification numbers of the modeled system. This map is included in Appendix C.

Land use data and the collection area basins were used to distribute the wastewater flows from Appendix A into the hydraulic model. Individual subbasins were delineated within each of the urban growth areas to help distribute flow where necessary as shown in Figures 4-2 through 4-10. As seen in these figures, the subbasins were delineated around sensitive areas where development is not likely to occur. The average annual domestic and commercial/industrial flows based on the values of 77 gpcd and 82 gpcd; respectively, (per Table 4-1) were added into the InfoSewer model. In addition to the domestic and commercial flows, I/I was added as well. I/I is assumed to be constant throughout the areas of future growth so all future service areas were assumed to have the I/I value of 800 gpad (per Table 4-1).

Peak hour flows are used to evaluate capacity in the hydraulic model and to size new infrastructure. To obtain the peak hour flows, the average base flow was disbursed throughout the subbasins and then peaked within the model with basin-specific diurnal curves calibrated to flow meter data obtained at the time of the *2009 Plan*. The diurnal curves shown in Appendix D peak the flow from 1.2 to 1.6 times greater than the base flow to represent the peak hour flow depending on the specific basin.

4.4 MODELING RESULTS

Appendix B and Figure 4-11 present the modeling output results for the future peak hour flow scenario (i.e., Year 2026 with UGA buildout). Appendix B identifies each pipeline segment and manhole in a tabular format. The modeling indicates that the depth at the peak hour flow through the pipe will be equal to or exceed the maximum depth of pipe (i.e., the diameter). Essentially, this means that the pipe is running full at the modeled flows and that flow is surcharging within the manholes. However, the City of Bellingham defines deficiencies in the collection system as follows:

- A gravity main is deficient when InfoSewer reports that the upstream manhole is surcharged to within 2 feet of the manhole rim during peak hour flow conditions.

- In situations where shallow manholes do not provide at least 2 feet of separation between the rim and the crown of the pipe, then the peak hydraulic grade line (HGL) must be at or below the lowest pipe crown coming into or out of the manhole.

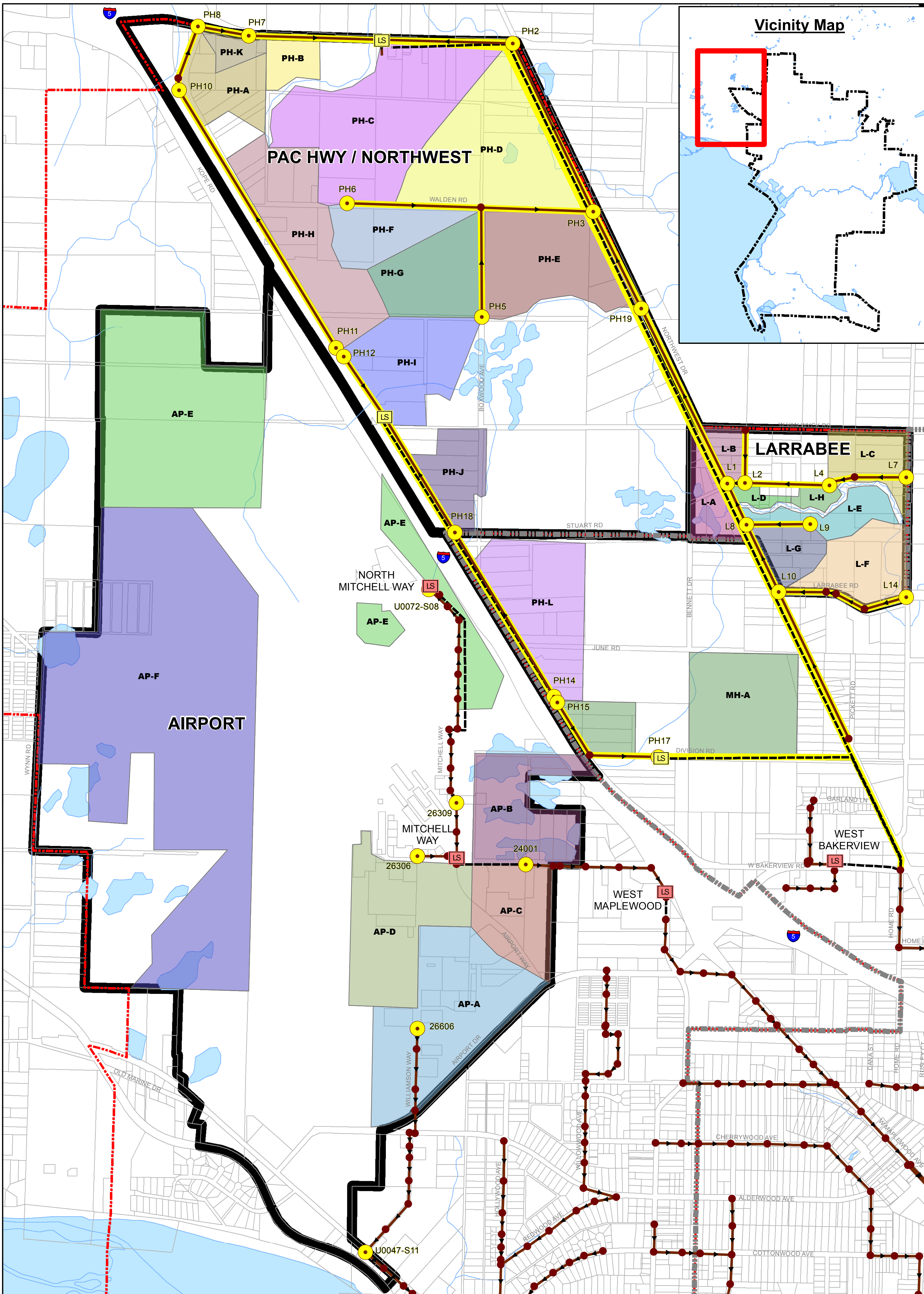


FIGURE 4-2

Legend

- LS Potential Lift Station
- LS Existing Lift Station
- UGA Flow Input MH
- Sewer MH (Modeled)
- Sewer Pipe (Modeled)
- Force Main (Modeled)
- Potential Infrastructure
- UGA Boundary
- City Limits
- UGA Area

N

0 500 1,000 2,000 Feet

CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
SEWER SUB-BASINS FOR UGA AREAS

(Airport/Pac Hwy-Northwest/Larrabee)

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\report figures\Figure 4-2 Airport.mxd

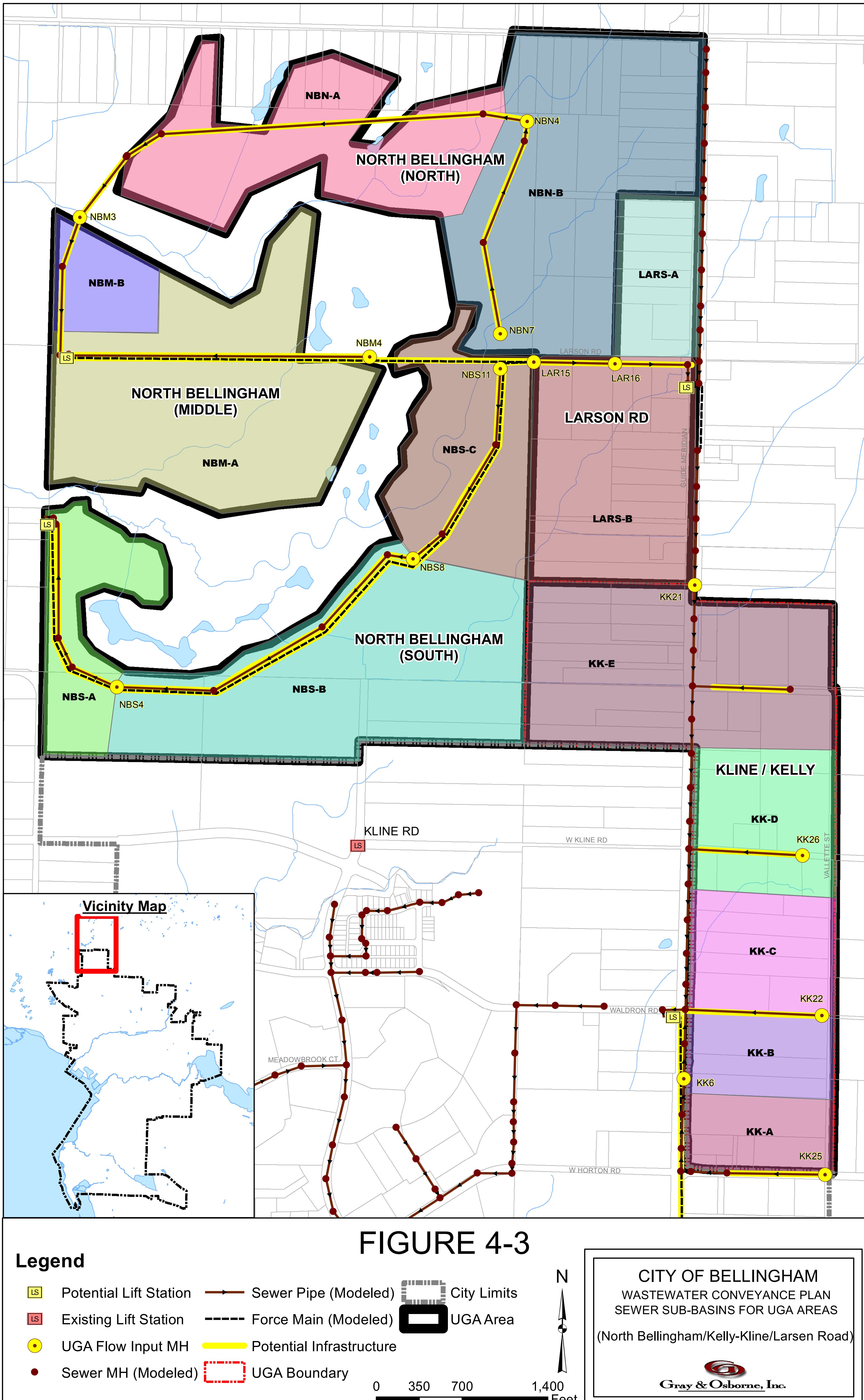


FIGURE 4-3

CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
SEWER SUB-BASINS FOR UGA AREAS
(North Bellingham/Kelly-Kline/Larsen Road)

Gray & Osborne, Inc.

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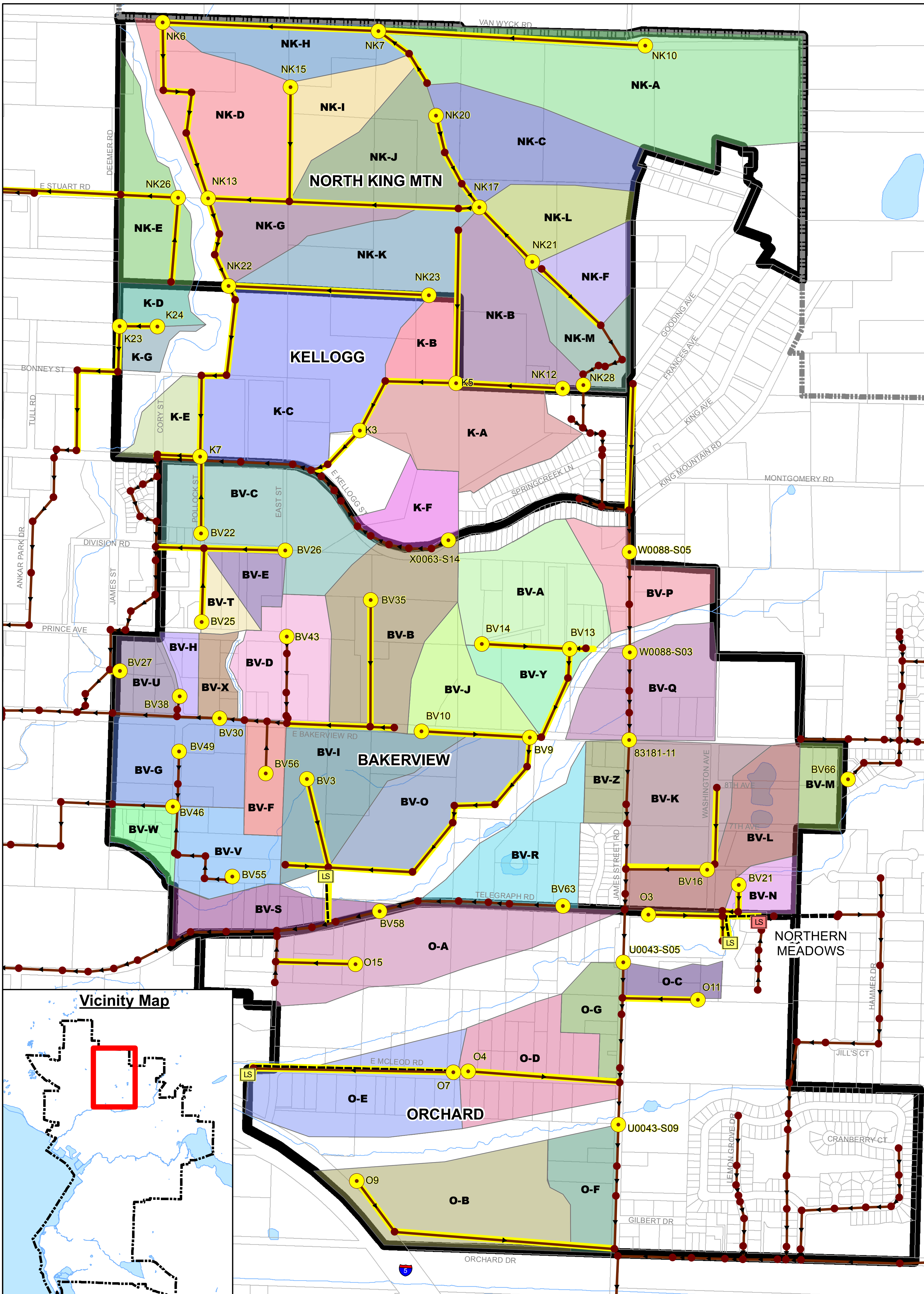

FIGURE 4-4

Legend

LS	Potential Lift Station		Sewer Pipe (Modeled)		City Limits
LS	Existing Lift Station		Force Main (Modeled)		UGA Area
	UGA Flow Input MH		Potential Infrastructure		
	Sewer MH (Modeled)		UGA Boundary		

N

0 350 700 1,400 Feet

CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
SEWER SUB-BASINS FOR UGA AREAS
(N. King Mt./Kellogg/Bakerview/Orchard)

Gray & Osborne, Inc.

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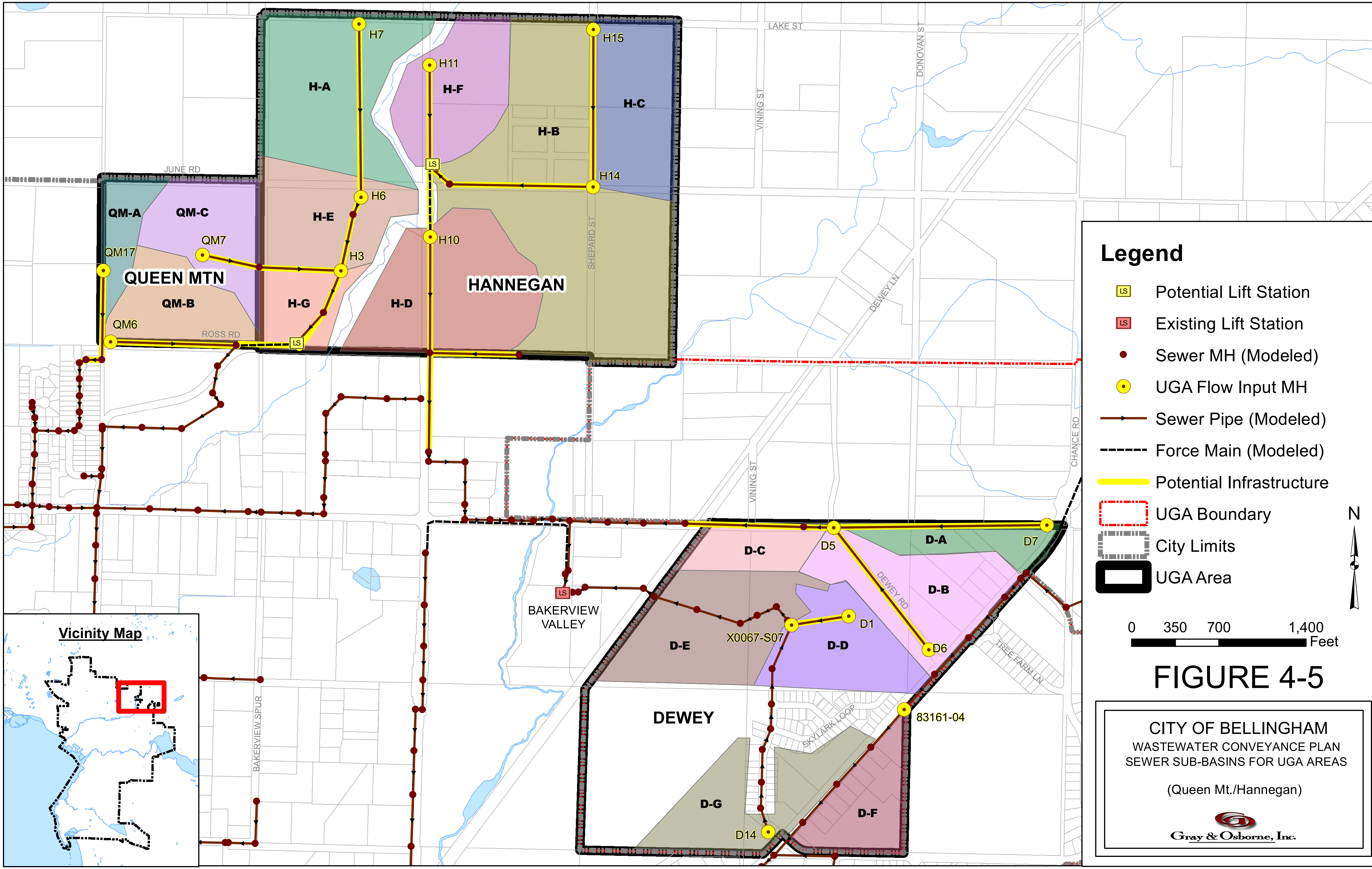

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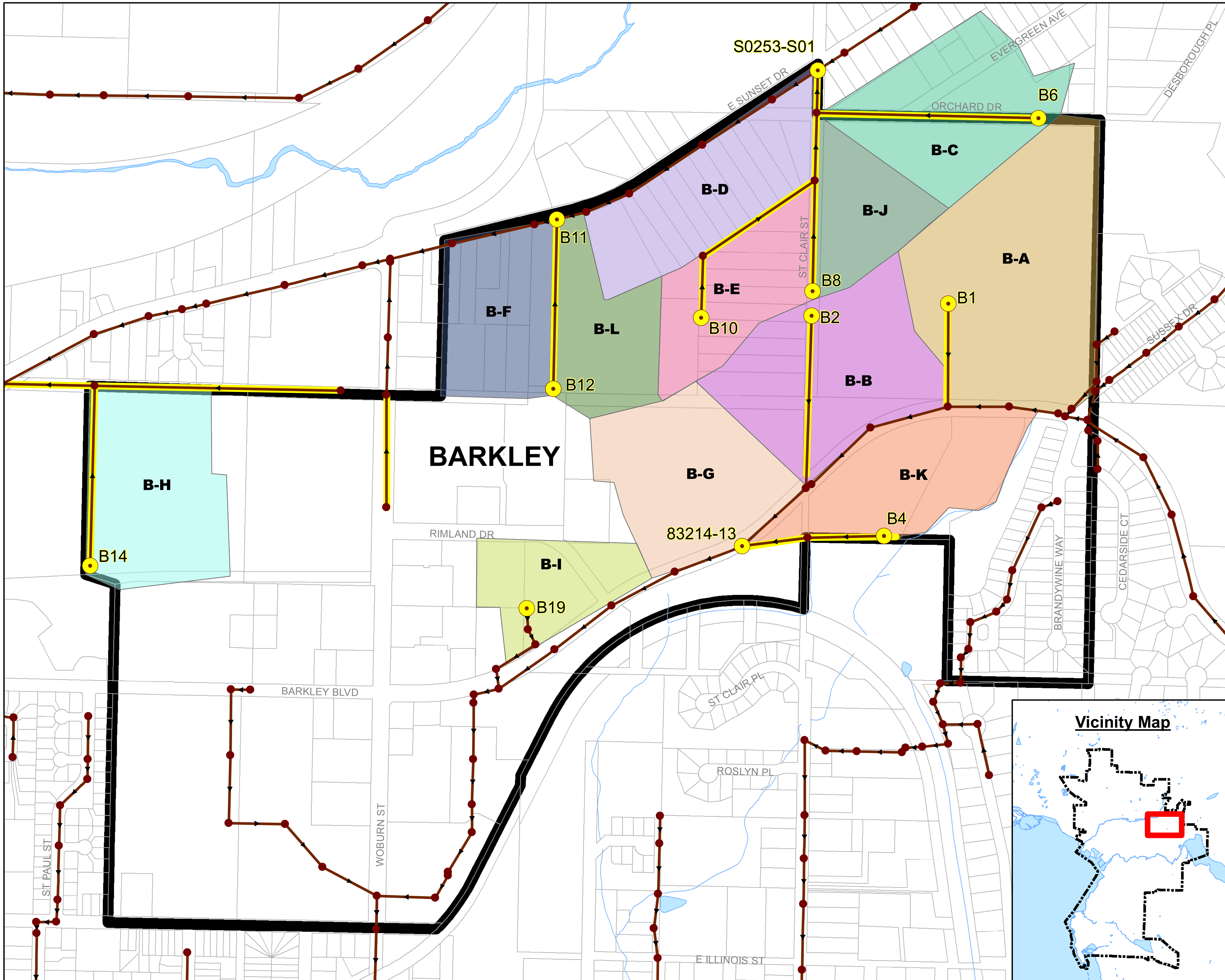

FIGURE 4-6

CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
SEWER SUB-BASINS FOR UGA AREAS

(Barkley)

Gray & Osborne, Inc.

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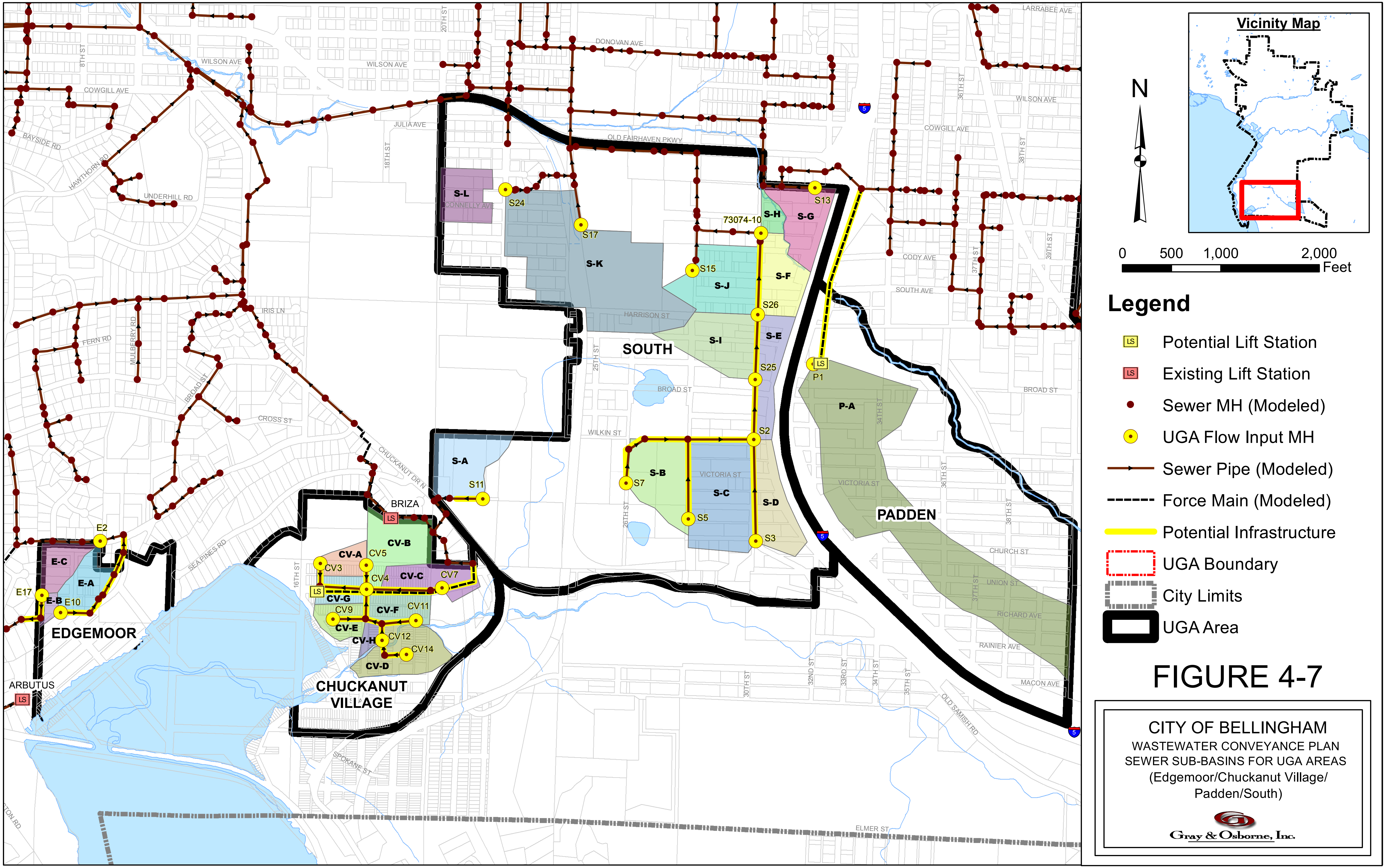

Vicinity Map
N
0
500
1,000
2,000
Feet
Legend
Potential Lift Station
Existing Lift Station
Sewer MH (Modeled)
UGA Flow Input MH
Sewer Pipe (Modeled)
Force Main (Modeled)
Potential Infrastructure
UGA Boundary
City Limits
UGA Area
FIGURE 4-7
CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
SEWER SUB-BASINS FOR UGA AREAS
(Edgemoor/Chuckanut Village/
Padden/South)
Gray & Osborne, Inc.
EDGEMOOR
CHUCKANUT
VILLAGE
SOUTH
PADDEN
BRIZA
ARBUTUS
E-A
E-B
E-C
E2
E10
E17
CV-A
CV-B
CV-C
CV-D
CV-E
CV-F
CV-G
CV-H
CV3
CV4
CV5
CV7
CV9
CV11
CV12
CV14
S-A
S-B
S-C
S-D
S-E
S-F
S-G
S-H
S-I
S-J
S-K
S-L
S2
S3
S5
S7
S11
S13
S15
S17
S24
S25
S26
73074-10
P-A
P1
LS
DONOVAN AVE
WILSON AVE
COWGILL AVE
JULIA AVE
BAYSIDE RD
HAWTHORN RD
UNDERHILL RD
IRIS LN
FERN RD
MULBERRY RD
BROAD ST
CROSS ST
CHUCKANUT DR N
SEA PINES RD
OLD FAIRHAVEN PKWY
CONNELLY AVE
HARRISON ST
WILKIN ST
VICTORIA ST
CODY AVE
SOUTH AVE
CHURCH ST
UNION ST
RICHARD AVE
RAINIER AVE
MACON AVE
OLD SAMISH RD
SPOKANE ST
ELMER ST
LARRABEE AVE
8TH ST
16TH ST
18TH ST
20TH ST
25TH ST
26TH ST
30TH ST
32ND ST
33RD ST
34TH ST
35TH ST
36TH ST
37TH ST
38TH ST
39TH ST
L:\Bellingham\15448.00 WW Conveyance Plan\Report\report figures\Figure 4-7 Chuckanut.mxd

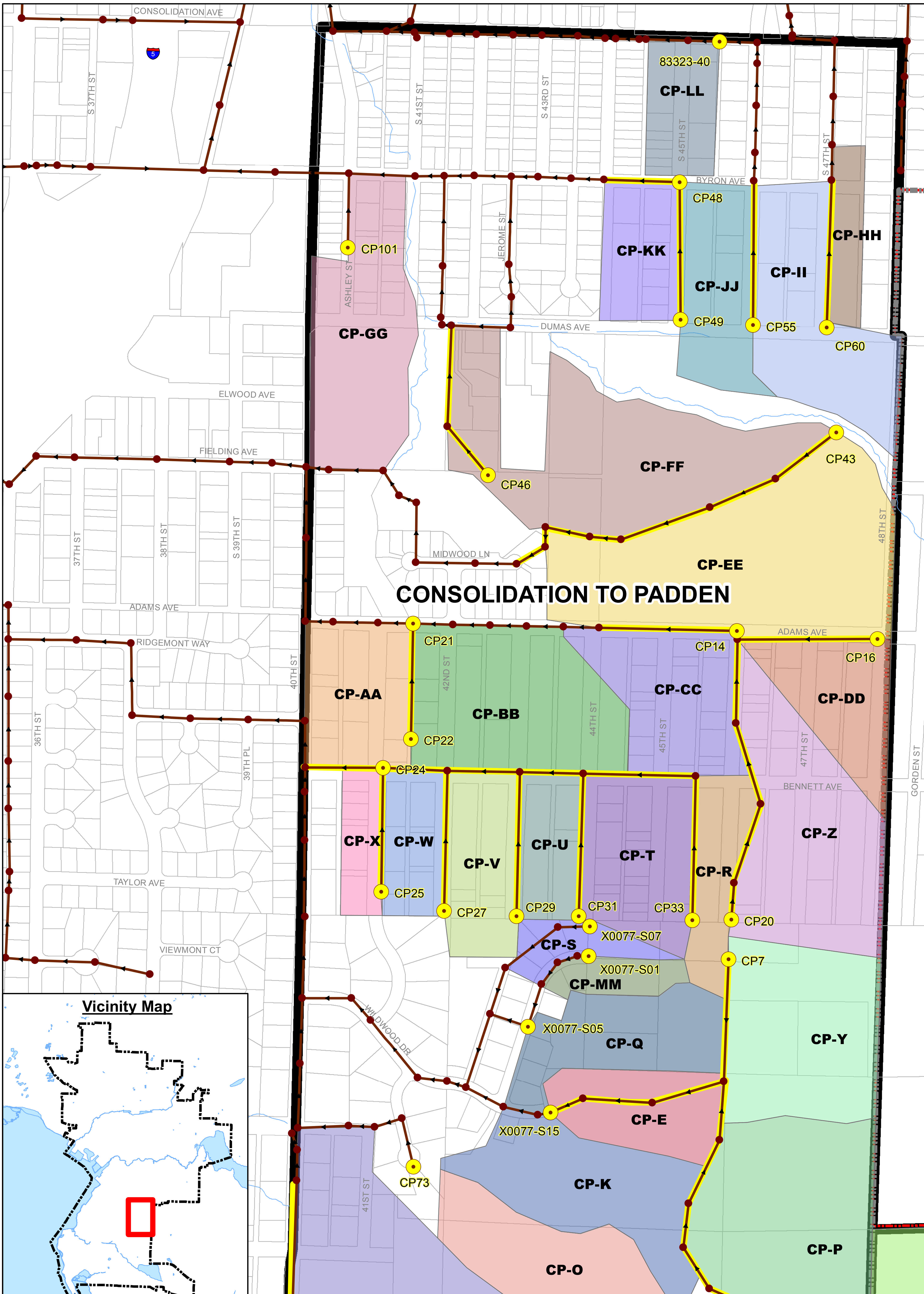

FIGURE 4-8

Legend

Potential Lift Station	Sewer Pipe (Modeled)	City Limits	
Existing Lift Station	Force Main (Modeled)	UGA Area	
UGA Flow Input MH	Potential Infrastructure		
Sewer MH (Modeled)	UGA Boundary		

N

0 200 400 800 Feet

CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
SEWER SUB-BASINS FOR UGA AREAS

(Consolidation to Padden - North End)

Gray & Osborne, Inc.

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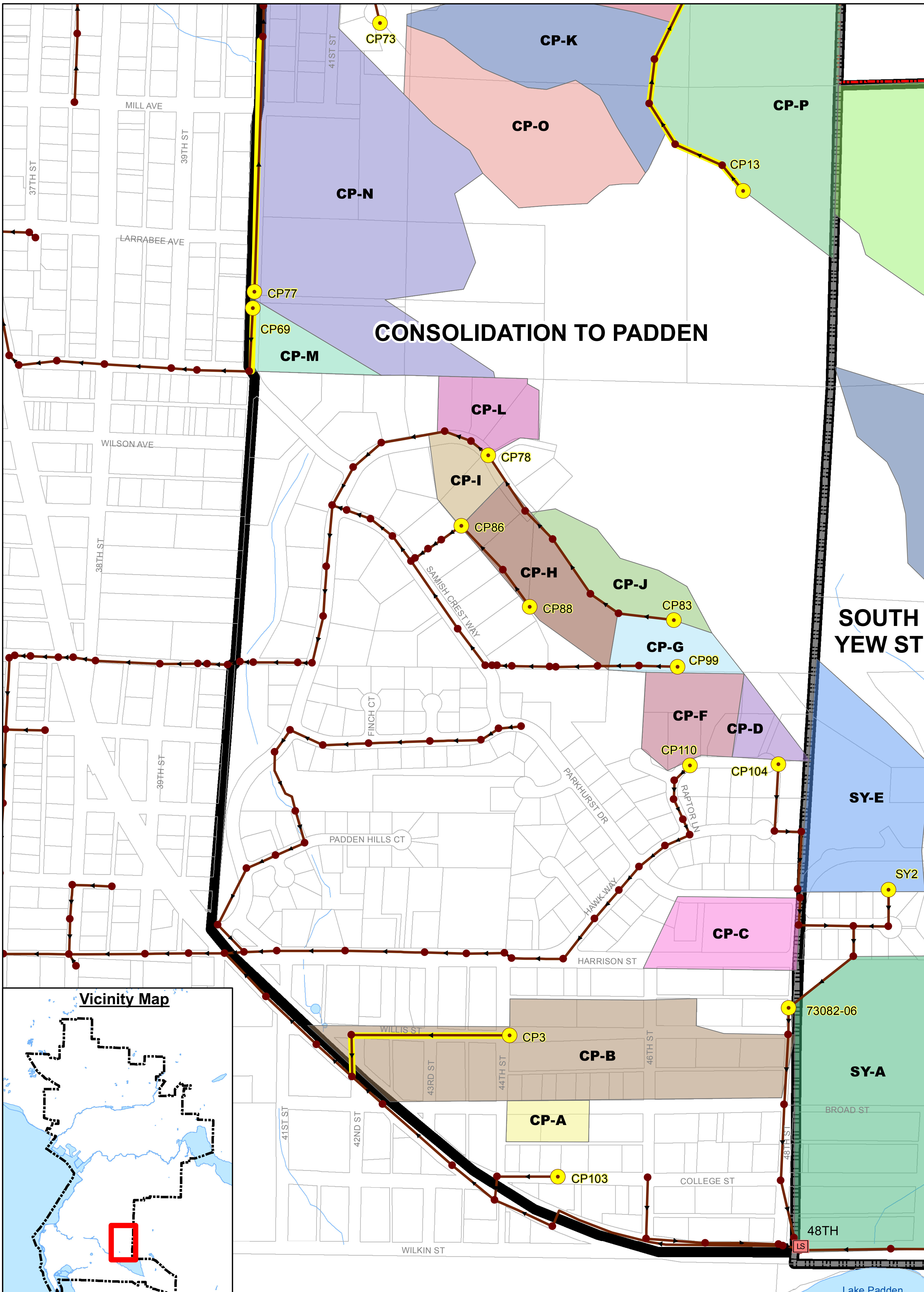

FIGURE 4-9

Legend

- LS Potential Lift Station
- LS Existing Lift Station
- UGA Flow Input MH
- Sewer MH (Modeled)
- Sewer Pipe (Modeled)
- Force Main (Modeled)
- Potential Infrastructure
- UGA Boundary
- City Limits
- UGA Area

N

0 200 400 800 Feet

CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
SEWER SUB-BASINS FOR UGA AREAS

(Consolidation to Padden - South End)

Gray & Osborne, Inc.

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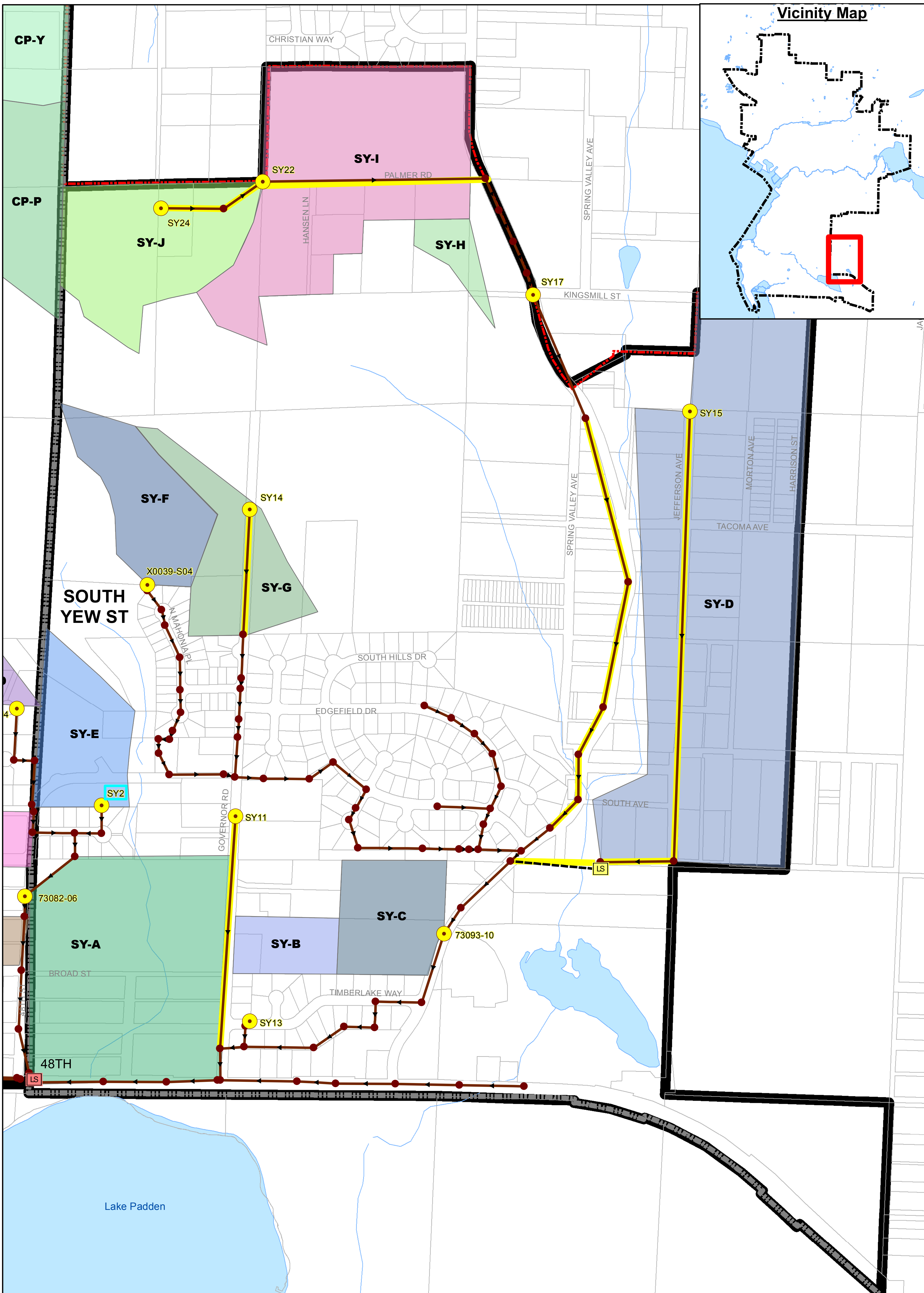

FIGURE 4-10

Legend

Potential Lift Station	Sewer Pipe (Modeled)
Existing Lift Station	Force Main (Modeled)
UGA Flow Input MH	Potential Infrastructure
Sewer MH (Modeled)	UGA Boundary
City Limits	UGA Area

N

0 250 500 1,000 Feet

CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
SEWER SUB-BASINS FOR UGA AREAS

(South Yew Street)

Gray & Osborne, Inc.

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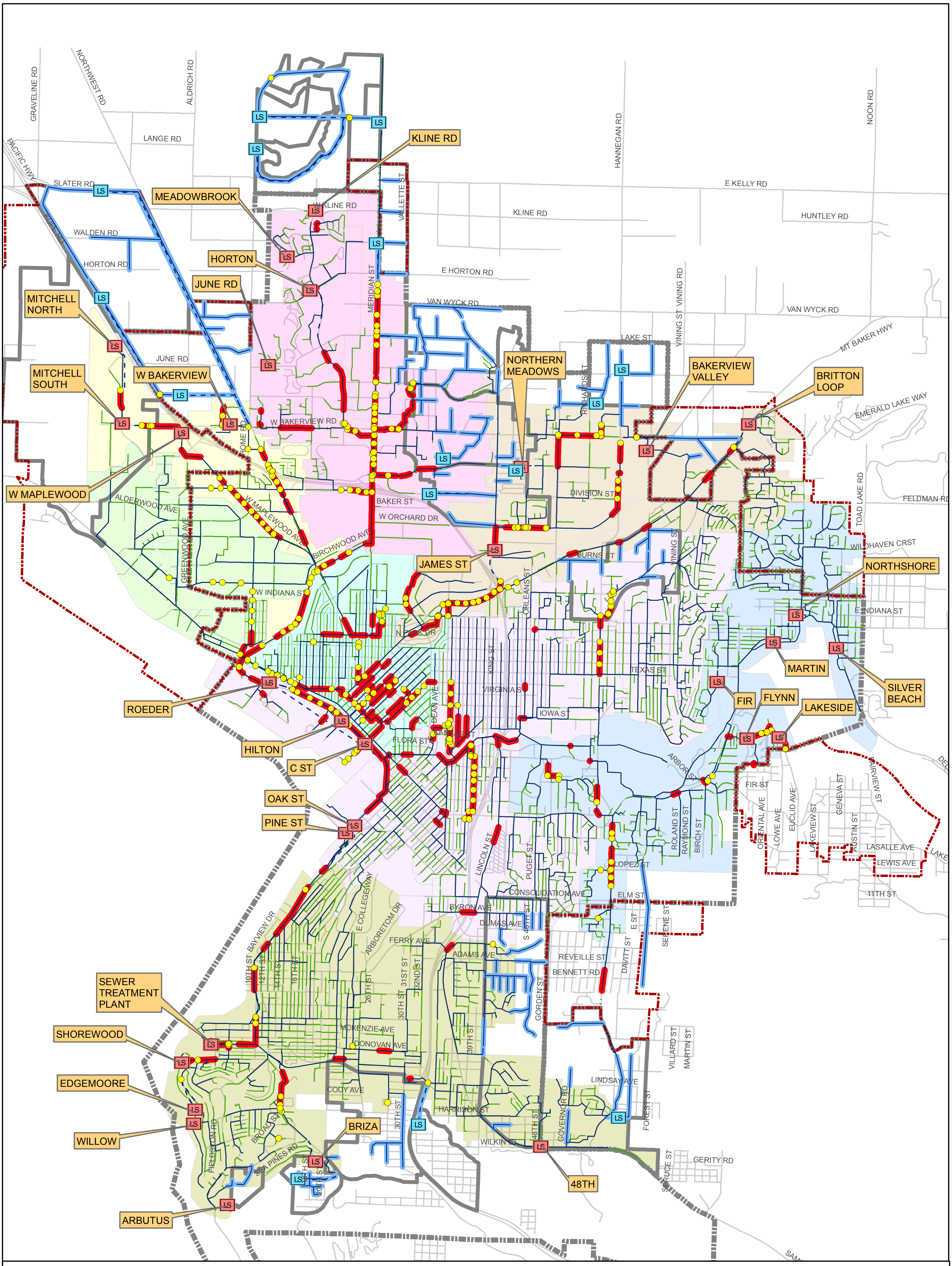

FIGURE 4-11

Legend

- Sewer MH (Surcharged w/in 2' of Rim)
- Sewer Pipe (Modeled)
- Sewer Pipe (Surcharged)
- Sewer Pipe (Existing)
- Force Main (Modeled)
- Potential Infrastructure
- LS Potential Lift Station
- LS Existing Lift Station
- UGA Boundary
- City Limits
- UGA Area

Sewer Basins

- Birchwood
- Broadway
- Central
- Cordata/Meridian
- Lake Whatcom
- Northwest
- Southside
- Sunset/MT.Baker

0 1,900 3,800 7,600 Feet

N

CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
MODEL RESULTS

Gray & Osborne, Inc.

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In the current modeling of the future flow scenario, a total of 432 pipe segments are shown to have insufficient pipeline capacity and 284 manholes are shown to be surcharged within 2 feet of the rim. Beyond the performance criteria set by the City, the model also reveals pipe segments whose velocity is less than 2 feet per second during peak hour flow conditions. Inadequate velocities may lead to lack of scouring and solids settlement in the pipe. The model output shows that approximately 20 percent of the modeled pipeline segments are characterized by low velocity (less than 2.0 feet per second). It is likely that additional pipe segments have low velocity during low flow periods. Although numerous pipelines have velocities less than 2 feet per second, many of them are capable of otherwise accommodating the projected flows, and therefore are not recommended for improvements. However, the pipelines should be monitored by the City as part of its ongoing operations and maintenance to identify, and as necessary, address, potential solids deposition in the pipelines.

4.5 GRAVITY COLLECTION SYSTEM DEFICIENCIES

After a review of deficiencies identified in the model, a number of improvements are recommended (see Tables 4-2 through 4-8). Some of these recommendations are carried over from the *2009 Plan*. The remaining projects are a result of downstream impacts from future anticipated flows in the UGA region. Figure 4-12 highlights the deficient pipes of concern while Chapter 6 will discuss the improvements in greater detail. With completion of the recommended improvements listed below, a revised model showed no deficiencies for the areas listed other than acceptable levels of surcharging described previously. It should be emphasized that the model was based on available limited GIS information and uncertain estimates and projections; thus, it is recommended that implementation of all projects related to this capacity assessment in Bellingham be further evaluated prior to design and construction.

Within the Northwest Basin, three areas are identified with insufficient hydraulic capacity at the future peak flow scenario as listed in Table 4-2.

TABLE 4-2

Hydraulic Deficiencies Identified in the Northwest Basin

CIP No.	Street Location	Manhole Location	Capacity Issue	Recommendation[1]
NW1	West Maplewood Avenue	MH 14904 to MH 15002	Surcharges; Undersized 8-inch pipes	Replace 1,327 LF of 8-inch pipe with 10-inch pipe
		MH 15002 to MH 82133-01	Surcharges; Undersized 8-inch pipes	Replace 1,694 LF of 8-inch with 12-inch pipe
		MH 15001 (S0420-S02) to 15701	No capacity issue; Issue maintaining parallel pipes when one pipe could take the total flow	Abandon and connect flow into new 12-inch pipes from MH 15002 to MH 82133-01

TABLE 4-2 – (continued)

Hydraulic Deficiencies Identified in the Northwest Basin

CIP No.	Street Location	Manhole Location	Capacity Issue	Recommendation[1]
NW2	Northwest Avenue	MH 82134-08 to MH 82134-07	Surcharges; 10-inch pipe at a slope that does not provide enough capacity	Replace 403 LF 10-inch pipe with steeper slope (0.0514)
		MH 82134-06 to MH 15120	Surcharges; Undersized 10-inch pipes	Replace 2,259 LF of 10-inch pipe with 12-inch pipe
NW3	Squalicum Way	MH 10507 to MH 17010	Surcharges; Undersized 18-inch/21-inch/24-inch/ 27-inch pipes	Install 7,040 LF of parallel 21-inch pipe

(1) Verification of inverts for these areas should be done prior to design or construction to confirm capacity issues.

Due to surcharging anticipated in the future flow scenario, five areas have been identified with insufficient hydraulic capacity within the Cordata/Meridian Basin as shown in Table 4-3.

TABLE 4-3

Hydraulic Deficiencies Identified in the Cordata/Meridian Basin

CIP No.	Street Location	Manhole Location	Capacity Issue	Recommendation[1]
CM1	Meridian Street	MH 83063-02 to MH 22403	Surcharges; Undersized 8-inch to 12-inch pipes	Replace 8,092 LF 8-inch to 12-inch pipe with 15-inch pipe
		MH 22403 to MH 21206	Surcharges; Undersized 8-inch, 18-inch, 21-inch pipes	Replace 1,081 LF 8-inch/18-inch/21-inch pipe with 24-inch pipe
CM2	East Bakerview Road	MH 83073-16 to MH 83184-02	Surcharges; Undersized 8-inch/10-inch pipes	Replace 916 LF 8-inch/10-inch pipe with 12-inch pipe
CM3	Deemer Road	MH STA85-S09 to MH STA85-S06	Surcharges due to shallow 10-inch pipes	Replace 576 LF 10-inch pipe with 18-inch pipe to contain flow in shallow area
CM4	East Bellis Fair Pkwy	MH 37405 to MH 23210	Surcharges; Undersized 12-inch/15-inch pipes	Replace 1,547 LF 12-inch/15-inch pipe with 18-inch pipe
CM5	Telegraph Road	MH 83184-13 to MH 22403	Surcharges; Undersized 8-inch pipes	Replace 964 LF 8-inch pipe with 10-inch pipe

(1) Verification of inverts for these areas should be done prior to design or construction to confirm capacity issues.

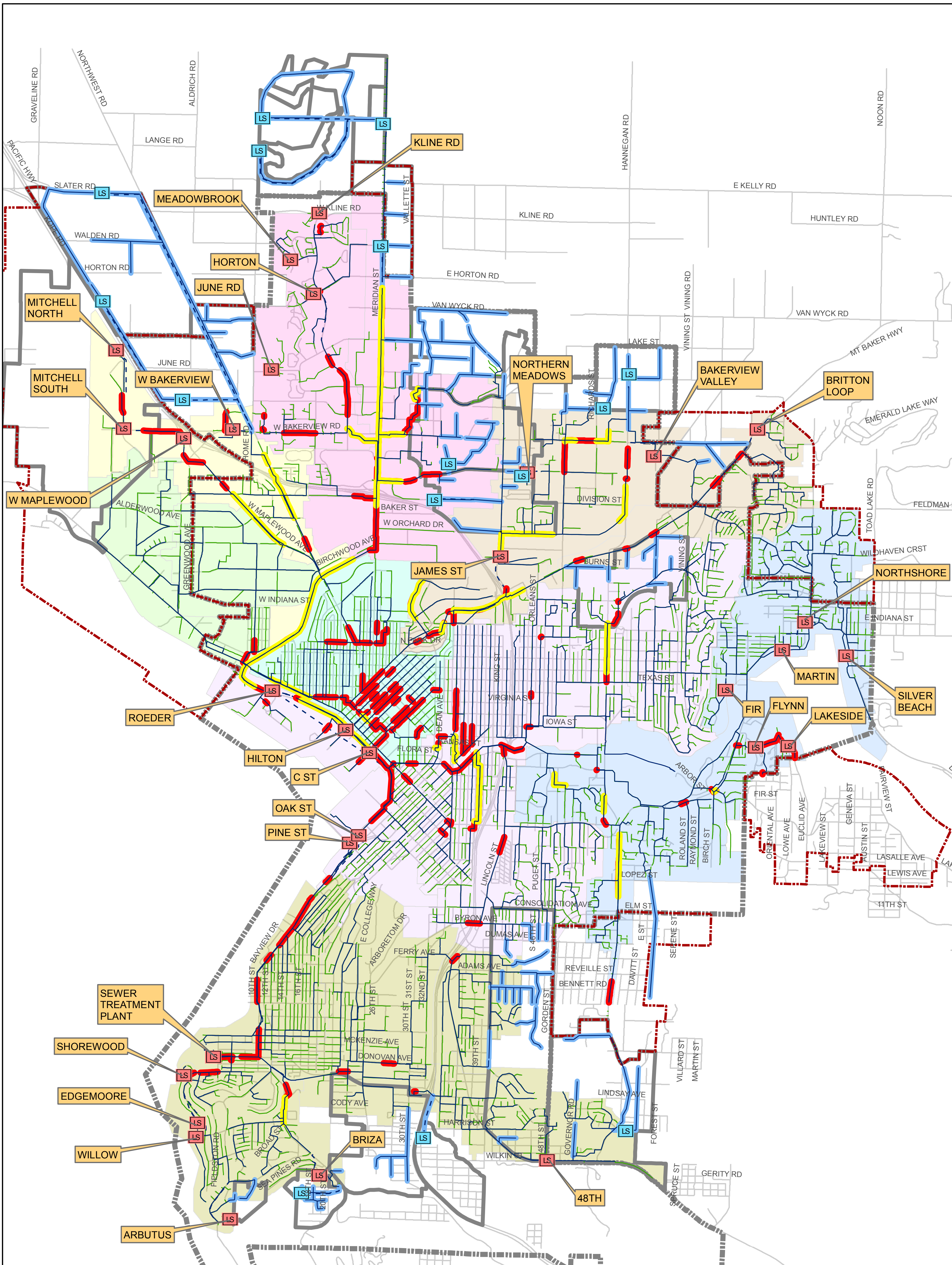

FIGURE 4-12

Legend

- Sewer Pipe (Modeled)
- Sewer Pipe (Surcharged - Prioritized Concern)
- Sewer Pipe (Surcharged)
- Sewer Pipe (Existing)
- Force Main (Modeled)
- Potential Infrastructure
- LS Potential Lift Station
- LS Existing Lift Station
- UGA Boundary
- City Limits
- UGA Area

Sewer Basins

- Birchwood
- Broadway
- Central
- Cordata/Meridian
- Lake Whatcom
- Northwest
- Southside
- Sunset/MT.Baker

0 1,900 3,800 7,600 Feet

N

CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
PRIORITIZED MODEL DEFICIENCIES

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\report figures\Figure 4-12 prioritized model deficiencies.mxd

Table 4-4 describes seven areas areas with insufficient hydraulic capacity at the future peak flow scenario for the Sunset/Mt. Baker Basin.

TABLE 4-4

Hydraulic Deficiencies Identified in the Sunset/Mt. Baker Basin

CIP No.	Street Location	Manhole Location	Capacity Issue	Recommendation(1)
SM1	Strider Loop/East Bakerview Road	MH U0099-S03 to MH S0271-S03	Surcharges; Undersized 8-inch pipes	Replace 928 LF 8-inch pipe with 24-inch pipe
SM2	East Bakerview Road/Irongate Road	MH 53903 to MH 53901	Surcharges; Flat 10-inch pipe and undersized 8-inch pipe	Replace 474 LF 8-inch/ 10-inch pipe with 18-inch at a new slope (0.0054)
SM3	Hannegan Road	MH 53106 to MH 53005	Surcharges; Undersized 10-inch Pipes	Replace 1,541 LF 10-inch pipe with 12-inch pipe
SM4	East Orchard Drive/ James Street	MH 52701 to MH W0073-S03	Surcharges; Undersized 18-inch pipes	Replace 3,114 LF 18-inch pipe with 24-inch pipe
SM5	East Sunset Drive	MH 11003 to MH 09910	Surcharges; Undersized 15-inch/18-inch pipes	Replace with 1,890 LF of 15-inch/18-inch with 21-inch pipe
		MH 09910 to MH 09804	Surcharges; Undersized 18-inch pipes	Install 1,332 LF parallel 24-inch pipe
SM6	East Illinois Street/ Broadway Street	MH 09713 to MH 08310	Surcharges; Undersized 15-inch pipes	Replace 373 LF 15-inch pipe with 18-inch pipe
SM7	Cornwall Avenue	MH 10703 to MH 09615	Surcharges; Undersized 12-inch pipes	Replace 1,811 LF 12-inch pipe with 18-inch pipe

(1) Verification of inverts for these areas should be done prior to design or construction to confirm capacity issues.

Within the Central Basin, seven areas are projected to have insufficient capacity at the future peak flow scenario as shown in Table 4-5.

TABLE 4-5

Hydraulic Deficiencies Identified in the Central Basin

CIP No.	Street Location	Manhole Location	Capacity Issue	Recommendation(1)
C1	Woburn Street	MH 83201-15 to MH 06806 (U0104-S07)	Surcharges; Undersized 8-inch pipes	Replace 2,048 LF 8-inch pipe with 12-inch pipe
C2	Humboldt Street/ Iron Street	MH 01912 to MH 03307	Surcharges; Undersized 8-inch pipes	Replace 3,213 LF 8-inch pipe with 18-inch pipe
C3	Franklin Street/ Ellis Street	MH 03209 to MH 83302-02	Surcharges; Undersized 8-inch/10-inch/12-inch pipes	Replace 1,043 LF 8-inch/10-inch/12-inch pipe with 12-inch pipe
		MH 83302-02 to MH 03211	Surcharges; Undersized 8-inch pipe	Replace 174 LF 8-inch pipe with 21-inch pipe

TABLE 4-5 (continued)

Hydraulic Deficiencies Identified in the Central Basin

CIP No.	Street Location	Manhole Location	Capacity Issue	Recommendation(1)
C4	Dean Avenue	MH 03114 to MH 03111	Surcharges; Undersized 12-inch pipe	Replace 139 LF 12-inch pipe with 18-inch pipe
C5	Roeder Avenue	MH 17009 to MH 05201	Surcharges; Undersized 30-inch pipe	Install 757 LF parallel 36-inch pipe
C6	Whatcom Creek Tunnel	MH 03509 to MH 02617	Structural issues (Built in 1909)	Line 6,550 LF with cured-in place pipe
C7	Roeder Avenue	MH 05104 to MH 24206	Surcharges; Undersized 36-inch pipe	Install 2,636 LF parallel 42-inch pipe
		MH 24206 to MH 02806	Surcharges; Undersized 36-inch pipe	Install 225 LF parallel 48-inch pipe
		MH 02806 to MH 02814	Surcharges; Undersized 36-inch pipe	Install 824 LF parallel 36-inch pipe
		MH 02814 to MH 02820	Surcharges; Undersized 30-inch pipe	Install 176 LF parallel 48-inch pipe

(1) Verification of inverts for these areas should be done prior to design or construction to confirm capacity isdsues.

Within the Lake Whatcom Basin, four regions are shown to have inufficient capacity at the future peak flow scenario as shown in Table 4-6.

TABLE 4-6

Hydraulic Deficiencies Identified in the Lake Whatcom Basin

CIP No.	Street Location	Manhole Location	Capacity Issue	Recommendation(1)
LW1	Yew Street	MH 20901 to MH 20805	Surcharges; Undersized 8-inch pipes	Replace 2,588 LF 8-inch pipe with 12-inch pipe
LW2	Old Woburn Street	MH 12503 to MH 12402	Surcharges; Undersized 10-inch pipes	Replace 654 LF 10-inch pipe with 18-inch pipe
LW3	North of Gladstone Street	MH 83292-01 to MH 83293-09	Surcharges; Undersized 8-inch pipes	Replace 947 LF 8-inch pipe with 12-inch pipe
LW4	Electric Avenue	MH 13214 to MH 13212	Surcharges; Undersized 8-inch pipes	Replace 129 LF 8-inch pipe with 24-inch pipe

(1) Verification of inverts for these areas should be done prior to design or construction to confirm capacity issues.

Within the South Side Basin, one region is shown to have insufficient capacity at the future flow scenario due to anticipated surcharging. This area is described in Table 4-7.

TABLE 4-7

Hydraulic Deficiencies Identified in the South Side Basin

CIP No.	Street Location	Manhole Location	Capacity Issue	Recommendation(1)
SS1	Chuckanut Drive North	MH 31912 to MH 30905	Surcharges; Undersized 12-inch pipes	Replace 1,101 LF 12-inch pipe with 21-inch pipe

(1) Verification of inverts for these areas should be done prior to design or construction to confirm capacity issues.

In the Broadway Basin within downtown Bellingham, two areas are shown to have insufficient capacity at the future flow due to anticipated surcharging. These areas are described in Table 4-8.

TABLE 4-8

Hydraulic Deficiencies Identified in the Broadway Basin

CIP No.	Street Location	Manhole Location	Capacity Issue	Recommendation(1)
B1	Bancroft Street	MH 04805 to MH 04823	Surcharges; Undersized 24-inch pipe	Replace 295 LF 24-inch pipe with 36-inch pipe
B2	Eldridge Avenue	MH 05218 to MH 05217	Surcharges; Undersized 12-inch pipe	Install 189 LF parallel 30-inch pipe
		MH 05217 to MH 05327	Surcharges; Undersized 12-inch/20-inch/24-inch pipe	Install 594 LF parallel 36-inch pipe
		MH 05327 to MH 05105	Surcharges; Undersized 20-inch pipe	Install 743 LF parallel 30-inch pipe

(1) Verification of inverts for these areas should be done prior to design or construction to confirm capacity issues.

A summary of the recommended improvements to the collection system based on the model results is presented in Chapter 6 (Capital Improvement Plan). A detailed cost analysis and a timeline for these improvements are also presented within Chapter 6. In addition, an evaluation of costs and potential modifications and repairs for the City's lift stations is provided within Chapter 5.

4.6 FORCE MAIN CAPACITY EVALUATION

The capacity evaluation for the force mains is tied directly to the lift station capacity evaluation. The capacity of each force main is based on a maximum design velocity of 8 feet per second (fps). Typical design parameters would include a design velocity less than 6 fps to reduce the friction losses and the increased power requirements. This capacity is compared to the existing lift station capacity and the predicted peak flow for the future peak flow scenario. The results of this evaluation are shown in Table 4-9.

TABLE 4-9

Force Main Capacity Analysis

Lift Station	Lift Station Design Capacity (gpm)	Force Main Diameter (in)	Existing Force Main Capacity (gpm)	Future Peak Flow Requirement (gpm)	Future Surplus (+) or Deficiency (-) (gpm)	Recommended Future Diameter (in)
48th Street (Lake Padden)	600	8	1,278	653	(+) 625	--
Arbutus	75	4	319	311	(+) 8	6
Bakerview Valley	260	4	319	596	(-) 277	8
Britton Loop	450	8	1,278	182	(+) 1,096	--
Briza Court	600	6	718	218	(+)500	--
C Street	500	6	718	1,329	(-) 611	10
Fir	150	4	319	94	(+)225	--
Flynn	500	6	718	997	(-) 279	8
Hilton	900	6	718	612	(+) 106	--
Horton	600	8	1,278	1,305	(-) 27	Parallel 8
James Street	1200	10	1,997	3,740	(-) 1,740	16
June Road[1]	440	6	718	--	--	--
Kline Road[1]	463	6	718	--	--	--
Lakeside	137	4	319	656	(-) 337	6
Martin	700	10	1,997	251	(+) 1,746	--
Meadowbrook Court	150	4	319	167	(+) 152	--
Mitchell Way	350	4	319	801	(-) 482	8
North Mitchell Way	525	6	718	668	(+) 50	--
Northshore	1,500	10	1,997	878	(+) 1,119	--
Northern Meadows	102	4	319	106	(+) 213	--
Oak Street	50,000+	36 (Dual FM)	51,700	44,005	(+) 7,695	--
Edgemoor	150	4	319	268	(+) 51	--
Pine Street		12	2,873	153	(+) 2,720	--
Roeder	3100/4000	18	6,470	12,507	(-) 6,037	30
Shorewood	350	4	319	82	(+) 237	--
Silver Beach	800	8	1,278	136	(+) 1,142	--
West Bakerview	350	4	319	1,190	(-) 871	8
West Maplewood	350	6	718	895	(-) 177	8
Willow Road	500	6	718	582	(+) 136	--

(1) The June Road and Kline Road lift stations were not included within the model.
(2) See Chapter 5, Evaluation of Lift Stations, for further analysis.

As shown in Table 4-9, numerous force mains have inadequate capacity to convey the flows anticipated under the future flow scenario. The recommended force main replacement sizes for the remaining hydraulically deficient pressure mains are shown in Table 4-9. They range in size from 6 to 30 inches in diameter.

4.7 LIFT STATION CAPACITY ANALYSIS

The City of Bellingham operates and maintains 29 lift stations. Table 4-10 summarizes the capacity analysis for each of the lift stations.

The peak flows developed for the future peak flow scenario for each lift station are compared to the lift station's existing capacity in Table 4-10. This table also lists the capacity surplus or deficiency of the station compared to the station's design capacity. Based on this comparison, 16 of Bellingham's lift stations have inadequate capacity for flows projected for the future flow scenario (2026 projected flows within the currently serviced area and buildout flows within the UGA areas). The deficits for the 48^{th} Street, Meadowbrook Court and Northern Meadows Lift Stations are minimal, and thus these lift stations should be monitored for capacity issues as flows increase. Based on the analysis of the remaining lift stations and in discussions with City staff, it is recommended that the City may want to address the Arbutus, James Street, Northshore, Edgemoor, Pine Street, Roeder, West Bakerview and West Maplewood lift stations due to capacity or other issues. The remaining lift stations with capacity issues are subject to private/developer funded upgrades including Bakerview Valley, C Street, Horton, Mitchell Way, and North Mitchell Way. All 13 of these lift stations ar analyzed within Chapter 5.

TABLE 4-10

Lift Station Capacity Analysis(1)

Lift Station	Basin	Tested Station Capacity (gpm)	Design Capacity (gpm)	2026 Flows w/UGA Buildout (gpm)	Future Surplus (+) or Deficiency (-) above Design Capacity (gpm)
48th Street (Lake Padden)	**South Side**	**606**	**600**	**653**	**(-) 53**
Arbutus	**South Side**	**94**	**75**	**311**	**(-) 236**
Bakerview Valley(2)	**Sunset/Mt. Baker**	**137**	**260**	**596**	**(-) 336**
Britton Loop	Sunset/Mt. Baker	407	450	182	(+) 268
Briza Court	South Side	711	600	218	(+) 382
C Street(2)	**Broadway**	**372**	**500**	**1,329**	**(-) 829**
Fir	Lake Whatcom	143	150	94	(+) 56
Flynn	**Lake Whatcom**	**521**	**500**	**997**	**(-) 497**
Hilton	Central	920	900	612	(+) 288
Horton(2)	**Cordata/Meridian**	**638**	**600**	**1,305**	**(-) 705**
James Street	**Sunset/Mt. Baker**	**1,728**	**1,200**	**3,737**	**(-) 2,537**
June Road(3)	Cordata/Meridian	238	440	--	--
Kline Road(3)	Cordata/Meridian	--	463	--	--
Lakeside	**Lake Whatcom**	**88**	**137**	**656**	**(-) 519**
Martin	Lake Whatcom	615	700	251	(+) 449
Meadowbrook Court	**Cordata/Meridian**	**198**	**150**	**167**	**(-) 17**
Mitchell Way(2)	**Northwest**	**158**	**350**	**801**	**(-) 451**
North Mitchell Way(2)	**Northwest**	**265**	**525**	**668**	**(-) 143**
Northshore	Lake Whatcom	1,580	1,500	878	(+) 622
Northern Meadows	**Sunset/Mt. Baker**	**94**	**102**	**106**	**(-) 4**
Oak Street	Central	--	50,000+	44,005	(+) 5,995
Edgemoor	South Side	147	150	268	
Pine Street	Central	?	?	153	
Roeder	**Birchwood**	**--**	**3,100/4,000**	**12,507**	**(-) 8,507**
Shorewood	South Side	58	350	82	(+) 268

TABLE 4-10 – (continued)

Lift Station Capacity Analysis[1]

Lift Station	Basin	Tested Station Capacity (gpm)	Design Capacity (gpm)	2026 Flows w/UGA Buildout (gpm)	Future Surplus (+) or Deficiency (-) above Design Capacity (gpm)
Silver Beach	Lake Whatcom	655	800	136	(+) 664
West Bakerview	**Northwest**	**244**	**350**	**1,190**	**(-) 840**
West Maplewood	**Northwest**	**441**	**350**	**895**	**(-) 545**
Willow Road	**South Side**	**346**	**500**	**582**	**(-) 82**

(1) Bold font represents lift stations shown to be undersized within the model and should be monitored for capacity issues in the future. Dark shading indicates a recommended capacity or lift station upgrade.

(2) Potentially developer funded upgrade.

(3) June Road and Kline Road lift stations were not included within the model.

4.8 FIELD OBSERVATIONS

Discussions with City maintenance staff indicate that the greatest concern in the field is with the 8-inch pipes along Guide Meridian Road between Horton Road and Bakerview Road due to the capacity of the pipes. Within the model, this area was confirmed to be a hydraulic issue. In addition, the City is concerned with the structural integrity of the 6-foot by 3-foot tunnel along Whatcom Creek in the vicinity between Champion Street and Nevada Street. It is recommended that this pipe be lined so as to increase the life of the 106-year old conveyance tunnel.

Additionally, the City is concerned with the downtown neighborhood between B Street and J Street located within the boundaries of West North Street and Girard Street. A lack of sloped topography in this area and the presence of connected roof drains lead to capacity concerns with the sanitary sewer system in this area. The City hopes to focus future I/I reduction efforts here so as to lessen the impact of stormwater into the combined sewer system within this region.

CHAPTER 5

EVALUATION OF LIFT STATIONS

5.1 INTRODUCTION

The City of Bellingham owns and operates a total of 29 lift stations throughout the sewer service area. In 2013, a preliminary evaluation of all 29 lift stations was completed. The results of this evaluation is summarized in a technical memorandum dated December 12, 2013, which is included in Appendix E (referenced as the "2013 Lift Station Evaluation" herein). Based on the 2013 Lift Station Evaluation, the recent modeling results, and field information provided by City personnel, 16 of these lift stations are in good condition and are capable of conveying the projected future flows to these lift stations. The remaining 13 lift stations need repair, upgrade or replacement to convey the projected future flows.

This chapter further evaluates the 13 lift stations requiring upgrade or replacement. This evaluation includes a review of the future flows to these stations based on the results of the hydraulic modeling that was completed and summarized in Chapter 4. Available pump station run time data was reviewed as needed to further evaluate current flows. The current analysis also included evaluation of the condition and capacity of the existing lift stations and associated force mains, pump and force main sizing calculations to convey the projected future flow, preliminary design and planning level cost estimates of preferred upgrades. The preferred alternatives were selected based on a variety of factors including cost, constructability, operational flexibility, and environmental considerations.

The lift stations evaluated in this chapter are summarized in Table 5-1 below.

TABLE 5-1

Summary of Lift Stations Evaluated

Lift Station	Basin	Current Flow(1) (gpm)	Tested Station Capacity (gpm)	Current Design Capacity (gpm)	Future Design Flow(4) (gpm)
Arbutus	South Side	22 – 38	94	75	311
Bakerview Valley(3)	Sunset/Mt. Baker	96 – 217	137	260	596
C Street(3)	Broadway	31 – 52	372	500	1,329
Horton(3)	Cordata/Meridian	505 – 704	638 (one pump)	600 (one Pump) 760(3) (two pumps)	1,305
James Street	Sunset/Mt. Baker	722 – 1,367	1,728	1,200	3,737
Mitchell Way(3)	Northwest	39 – 100	158	350	801
North Mitchell Way(3)	Northwest	4 – 5	265	525	668
Northshore	Lake Whatcom	889 – 1,215	1,580	1,500	878

TABLE 5-1 – (continued)

Summary of Lift Stations Evaluated

Lift Station	Basin	Current Flow(1) (gpm)	Tested Station Capacity (gpm)	Current Design Capacity (gpm)	Future Design Flow(4) (gpm)
Edgemoor	South Side	--	147	150	268
Pine Street	Central	--	--	--	153
Roeder(3)	Birchwood	(2)	--	3,100/4,000	12,507
West Bakerview	Northwest	92 – 165	244	350	1,190
West Maplewood	Northwest	98 – 198	441	350	895

(1) Estimated peak hour flow range presented in the 2013 Lift Station Evaluation.
(2) The current flow was not reported for the Roeder Lift Station. However, based on a review of run time data by Parametrix, the station is near or existing capacity exceeded at peak hour flows.
(3) Calculated capacity based on a comparison of the pump and system curves.
(4) Based upon Year 2026 flows with buildout occurring in the UGA regions (see Chapter 4).
-- Denotes data not available at the time of preparation of this report.

5.1.1 CONDITION ASSESSMENT

On March 2, 2016, Gray & Osborne staff inspected each of the lift stations included in Table 5-1. The purpose of these inspections was to evaluate the lift station structures and mechanical and electrical equipment for signs of deterioration. The findings of these inspections are summarized in the discussion of each lift station.

5.1.2 LIFT STATION CAPACITY EVALUATION

For each of the lift stations presented in Table 5-1, Gray & Osborne completed hydraulic calculations to evaluate the capability of the various components of the existing lift station to convey the projected future flow. The components of the lift station most significantly impacted by the increased flow are the pumps, wet well, force main, and electrical system.

5.1.2.1 Pump Evaluation

The existing pumps were evaluated by comparing the available pump curves for each of the pumps against the calculated system curve for the existing force main. The capacity of the station is determined to be the intersection of the pump curves for the maximum number of pumps that would run during normal operating conditions and the system curve. The tested station capacity was also compared with the calculated lift station capacity to verify the calculated capacity. As discussed further below, in some cases, where the velocity within the force main is high, the existing pump curves were also compared with a system curve for a new, larger diameter force main.

5.1.2.2 Wet Well Evaluation

Lift station wet wells are typically sized to avoid frequent starting and stopping of the pumps during fill and draw operation. Although the recommended time between pump starts varies with the particular pump, as a general rule, most wet wells are designed for pumps to start no more than once every 5 minutes, or 12 starts per hour, in worst case conditions. The worst case occurs when the influent flow is one-half of the capacity of the lift station with one pump out of service. For each lift station equipped with constant speed pumps, the maximum starts per hour was calculated using the following equation:

$$T = \left(\frac{4V}{Q}\right)(n-1)$$

where, *V = Wet Well Operating Volume (gallons)*
Q = Projected Peak Hour Flow (gallons per minute)
T = Time between Starts (minutes)
n = Number of Pumps

If it was determined that the time between pump starts for a particular lift station wet well is less than 5 minutes, replacement was considered. For stations with larger pumps, installation of VFDs was also considered to reduce starting frequency.

The calculations for evaluation of the wet well capacity at the Roeder Lift Station, which is equipped with two variable speed pumps and one constant speed pump, is discussed separately.

5.1.2.3 Force Main Evaluation

The force mains associated with each lift station were evaluated in Chapter 4 of this report. Table 4-9 summarizes the results of this evaluation.

5.1.2.4 Electrical System Evaluation

The electrical installations of each lift station were evaluated on the condition of the equipment, the size of the electrical service, and compliance with current codes.

When discussing the electrical loads, the connected loads are used in this report to evaluate the electrical capacity of the system rather than the alternative "bill demand" method allowed by The National Electrical Code (NEC) as it is the more conservative approach. Where necessary some ancillary loading, such as control panels and convenience receptacles, are estimated. Where necessary, the power factor (p.f.) is assumed to be 0.8.

The term "code compliance considerations" are based on the NEC 2014 and WAC requirements. The facilities are existing, and thus, are considered to be grandfathered in.

Any modifications to the systems will require the modified portions to be brought to current code standards.

The term “classified area(s)” or “hazardous area(s)” refer to those areas deemed as Class I, div 1 or Class I, div 2 under NFPA 820. These areas require special electrical methods such as seal-off fittings and intrinsically safe barriers to make the installation “explosion proof”.

5.1.2.5 Cost Estimates

The estimated cost of the preferred alternatives are presented in Chapter 6 and a detailed estimate presented in Appendix G.

5.1.3 ARBUTUS LIFT STATION

5.1.3.1 Existing Facilities

The existing Arbutus Lift Station is a top mounted vacuum-primed lift station equipped with two 15-hp Smith & Loveless pumps. The pumps are mounted at grade enclosed in a flip-top outdoor enclosure, which also encloses the controls and motor starters. The lift station is situated in the center of a cul-de-sac at the east end of Arbutus Place. The site is approximately 600 square feet and surrounded by a wood fence. The details of the lift station components are summarized in Table 5-2 below.

TABLE 5-2

Arbutus Lift Station

Year Built	1981
Type	Vacuum Prime
Pump Manufacturer	Smith & Loveless
Number of Pumps	2
Horsepower	15
Backup Power	30 kW diesel generator
Wet Well Diameter	4 feet
Force Main	4-inch ductile iron
Force Main Length	2,590 linear feet

5.1.3.2 Current Condition

Based on observations during the March 2, 2016 inspection of the lift station and discussions with City staff, the lift station is aging but is generally in good condition. The wet well has no visible evidence of deterioration or spalling and the piping in the wet well has limited corrosion. The pumps, valves and enclosure are operational and, aside from pumps occasionally losing prime, the operations staff has not reported any problems with this station. The flows to this station are currently quite low and the existing station has sufficient capacity to convey the current design flow. The lag pump is occasionally called to run but it is suspected that this only occurs when the lead pump loses prime and is unable to convey the flow. This station appears to be capable of remaining in service until flows increase beyond the capacity of the station.

5.1.3.3 Current Capacity Analysis

Per the 2013 Lift Station Evaluation, at current flows, the wet well and force main are oversized. As a result, this station experiences very short run times and long wet well and force main residence times during low flow periods. Additionally, at the tested pumping capacity of 94 gallons per minute reported in the 2013 Lift Station Evaluation, the velocity in the 4-inch-diameter force main is near the minimum recommended velocity of 2 feet per second.

5.1.3.4 Capacity Analysis at Projected Future Flow

5.1.3.4.1 Pumps

As presented in Table 5-1 above, the Arbutus Lift Station has a tested capacity of 94 gallons per minute. The current flow to the station is estimated between 22 and 38 gallons per minute and the projected future peak hour flow is estimated to be 311 gallons per minute. Based on a review of the pump and system curves, the existing pumps have sufficient capacity to convey the current peak hour flow but are not capable of conveying the projected peak hour flow, even if the 4-inch force main is replaced with a 6-inch pipe to reduce the velocity and total dynamic head. Replacing the existing force main with a new 6-inch force main would increase the capacity of the existing pumps to approximately 90 gallons per minute.

5.1.3.4.2 Force Main

With current lift station capacity, the velocity in the existing 4-inch-diameter ductile iron force main is near the minimum recommended velocity of 2 feet per second to maintain solids in suspension. At the projected future peak hour flow, the velocity would be at the maximum recommended velocity of 8 feet per second and the total dynamic head would be approximately 340 feet. This head requirement is greater than solids handling pumps are typically capable of providing without installing two pumps in series.

5.1.3.4.3 Wet Well

The existing 4-foot-diameter wet well has an operating volume of approximately 274 gallons. At the projected future peak hour flow, the pumps would have up to 17 starts per hour, which exceeds the maximum recommended 12 starts per hour for a duplex lift station. Based on a review of the available record drawings, the “pump on” elevation could be raised by approximately 10 inches, increasing the operating volume to approximately 352 gallons. This would reduce the pump starts at the future flow to 13 starts per hour.

5.1.3.5 Electrical System

5.1.3.5.1 General

The existing utility feed is a 100 A, 208/120 VAC, 3-phase service. The Smith & Loveless panel stated its full load amps (FLA) to be 109.4. This does not include the estimated 15 A of load from the telemetry panel. Based on the stamped nameplates, the existing utility feed is undersized for the connected loads.

5.1.3.5.2 Generator Capacity

The 30kW generator is capable of providing 104 A and as such is only capable of supporting one pump. It is in good physical condition.

5.1.3.5.3 Code Compliance Considerations

The entire system located above the wet well is in a classified area but is not constructed for such an environment. The PLC control panel and adjacent systems not owned by the City also fall inside the limits of the classified areas. The interior of the Smith & Loveless control panel is corroded, posing undo risk to electrical components and safety of personnel. Unless a specific automatic lockout exists, the utility service is undersized. While not an NEC violation, the spacing around the utility transformer is not compliant to current PSE standards.

5.1.3.6 Alternatives

The pumps, wet well and force main have more than adequate capacity for the current estimated peak hour flow but are all projected to be over capacity at the projected future peak hour flow. The station is also approximately 35 years old and near or beyond the end of its useful life. Additional future flows to the station are anticipated to come from a currently undeveloped area located east of the lift station. Three alternatives have been identified for serving the current and future service areas associated with this lift station and are summarized below. Figure 5-1, located at the end of this section, depicts the preferred alternative. Figures depicting all other alternatives are included in Appendix H.

5.1.3.6.1 Alternative No. 1 – Upgrade Lift Station on Current Site to Serve Entire Future Service Area.

Alternative No. 1 would serve the entire future service area with a single lift station located on the current lift station site. To serve the current service area and the area east of Arbutus Lift Station, when developed, it will be necessary to replace this lift station with a new station to handle the projected peak hour flows. This upgrade would construct a new 6-foot-diameter wet well, 6-inch-diameter force main and new pumps capable of conveying the projected peak hour design flows. Based on preliminary review of available pumps, a 23-hp pump would be capable of conveying the design flow through the proposed system. Because the existing station experiences long wet well detention times, low force main velocities and low pump run times at current flows, upgrades of this station should be coordinated to coincide with the development area east of Arbutus Lift Station to avoid increased odor and corrosion issues associated with increased wet well and force main residence times in the larger facilities. The new station would be constructed within the existing lift station site and bypass pumping would be provided throughout construction. Due to the significant site constraints, a wet pit-dry pit configuration would not be suitable for this site; and therefore, the City has indicated a preference for a submersible station. A submersible lift station with valve vault could be installed within the existing footprint with pump motor starters and controls installed in new outdoor control panels.

Electrical Aspects of Alternative No. 1

Any changes to the existing site will require all existing electrical code concerns to be remedied. If the generator is to support two running pumps it will need to be replaced with a larger generator. The existing footprint of the site may be insufficient if Puget Sound Energy (PSE) were to enforce current utility clearance requirements.

5.1.3.6.2 <u>Alternative No. 2 – Construct New Lift Station to Serve the Area East of Arbutus Lift Station and Upgrade Existing Station to Serve Current Service Area</u>

Alternative No. 2 would construct a new submersible or wet pit-dry pit lift station within the area east of Arbutus Lift Station that would serve only new development within that area. The station would likely pump to either the gravity sewer in Viewcrest Road, which drains to the Willow Lift Station, or to the gravity sewer in Sea Pines Road, which drains to the Briza Court Lift Station. If this alternative is selected, it will be necessary to evaluate the lift station that will receive this additional flow to confirm that it has sufficient spare capacity. The new lift station would likely be constructed through a developer extension.

The Arbutus Lift Station would continue to serve its current service area and would be upgraded to replace aging equipment and possibly convert the station to a submersible station. Based on inspection of the lift station facilities, the wet well does not show signs of deterioration and could be retained in the upgraded station. Based on City preference, the station would be converted to a submersible lift station. Bypass pumping would be required throughout construction of the Arbutus Lift Station improvements.

Electrical Aspects of Alternative No. 2

Any changes to the existing site will require all existing electrical code concerns to be remedied. If the generator is to support two running pumps it will need to be replaced with a larger generator. The existing footprint of the site may be insufficient if PSE were to enforce current utility clearance requirements.

5.1.3.6.3 <u>Alternative No. 3 – Construct a New Station on a Nearby Site to Serve Entire Future Service Area</u>

Alternative No. 3 would construct a new lift station on a new site and a new force main to collect and convey the sewage from the entire future service area.

5.1.3.6.4 Alternative No. 3A – Construct New Station on Adjacent Parcel Owned by Parks Department

One possible site is located east of the existing station. This site, which is designated as Natural Space, is owned by the City and managed by the Parks Department. This site is heavily wooded, has rock at shallow depth, and on a relatively steep slope. Significant clearing and grubbing and site grading would be required to construct this station. It is possible that a retaining wall would be required to provide a level site. Converting this site from a natural area to a sewage lift station may present permitting challenges as well. The proposed site is approximately 5-feet higher than the current site and the new wet well would likely need to be deeper to accommodate the existing gravity sewers.

5.1.3.6.5 Alternative No. 3B – Construct New Station in the Area East of Arbutus Lift Station

Alternative No. 3B would construct a new station within the area east of Arbutus Lift Station as proposed for Alternative No. 2; however the station would be sized to serve the entire projected Arbutus service area. The existing gravity sewer would be extended from the existing lift station to the new lift station site. The new station could be a developer constructed station with the City paying for the additional capacity to handle the existing service area or a City-constructed station with the developer paying for the capacity for the new development.

Electrical aspects of Alternatives No. 3A and No. 3B

Moving the station to a new location will likely reduce the difficulty of remediating electrical issues. If the generator is to support two running pumps, it will need to be replaced with a larger generator.

5.1.3.7 Preferred Alternative

Although the City has had some maintenance issues associated with the vacuum priming system, the existing lift station is currently operational and has sufficient capacity to convey the current flow and to accommodate some future development. Alternative No. 3B would maximize the remaining useful life of the existing components and presents an opportunity to construct a new station on a much less constrained site. There is also an opportunity to team with a developer to construct the new lift station with less cost incurred by the City. Therefore, Alternative No. 3B, as shown on Figure 5-1, is the preferred alternative.

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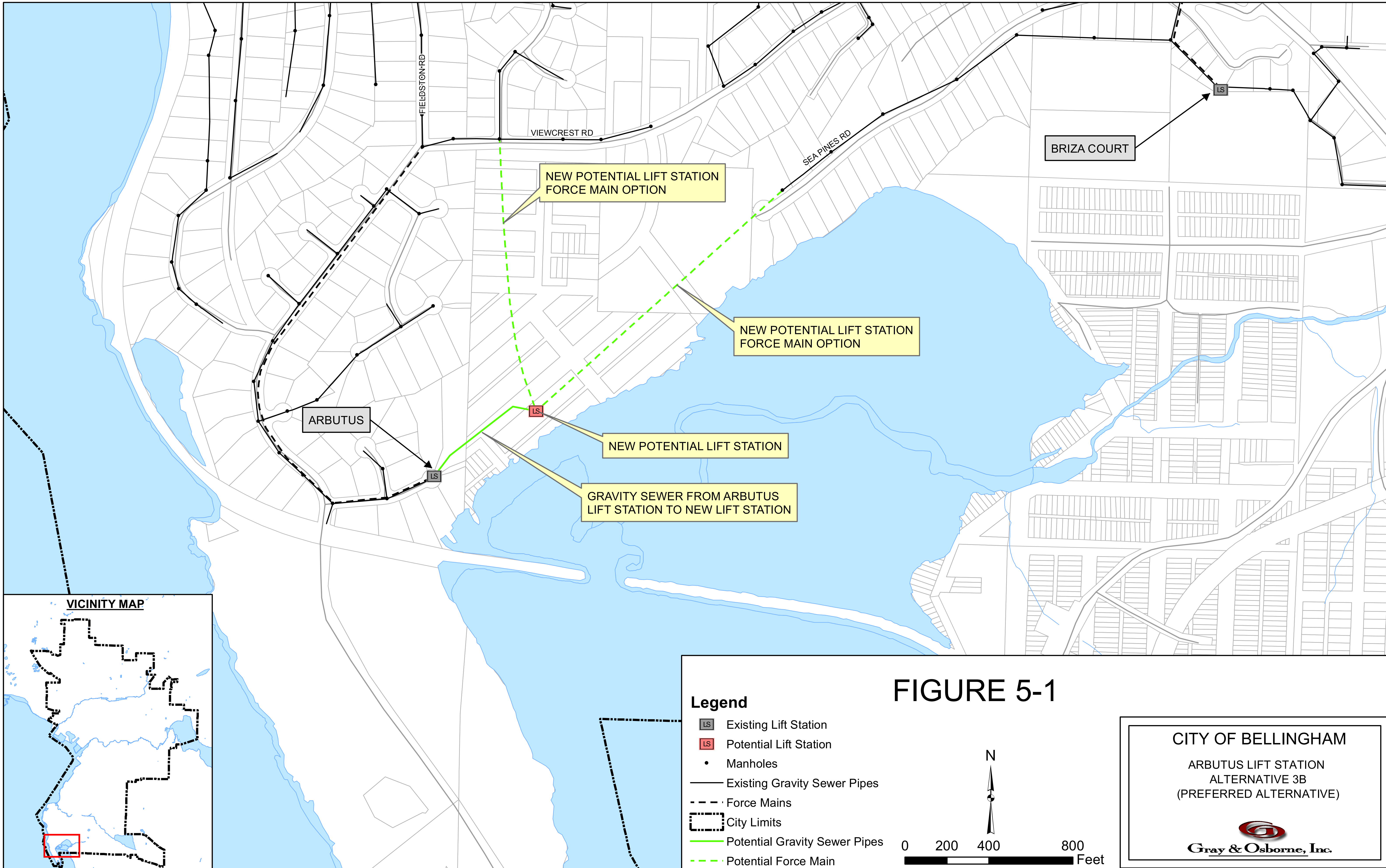

FIGURE 5-1

CITY OF BELLINGHAM
ARBUTUS LIFT STATION
ALTERNATIVE 3B
(PREFERRED ALTERNATIVE)

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\GIS\LiftStations\Arbutus5-1.mxd

5.1.4 BAKERVIEW VALLEY LIFT STATION

5.1.4.1 Existing Facilities

The existing Bakerview Valley Lift Station consists of two constant speed, 10-hp, Flygt submersible pumps mounted in an 8-foot-diameter concrete wet well that pumps through a 4-inch-diameter ductile iron force main. The lift station is situated in a sewer easement in the parking lot of a light industrial property owned by Ludtke Pacific Trucking. The details of the lift station components are summarized in Table 5-3 below.

TABLE 5-3

Bakerview Valley Lift Station

Year Built	1996
Type	Submersible
Pump Manufacturer	Flygt
Number of Pumps	2
Horsepower	10
Backup Power	30 A Outlet for Portable Generator
Wet Well Diameter	8 feet
Force Main	4-inch ductile iron
Force Main Length	1,905 linear feet

5.1.4.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this is a newer lift station and is generally in good condition. The wet well has no visible evidence of deterioration or spalling and the piping in the wet well has limited corrosion. The pumps, valves and enclosure are operational and the operations staff has not reported any problems with this station. Modeling suggests that this station is over capacity; however, the City has not reported any surcharging or overflows.

5.1.4.3 Capacity Analysis

5.1.4.3.1 Pumps

As presented in Table 5-1 above, the design flow for the existing lift station is approximately 260 gallons per minute but, based on the 2013 Lift Station Evaluation, the tested lift station capacity is significantly lower (137 gpm). As part of this evaluation, Gray & Osborne compared the system curve for this lift station and associated force main with the manufacturer’s pump curves (see Appendix F). Based on this comparison, the pumps appear to be pumping near their rated capacity of 137 gpm for the head conditions in this system. The original calculations used to size the pumps for a flow of 260 gallons per minute were not available at the time of this report and it is unknown why the existing pumps were selected for this application.

The current peak hour flow to this lift station is reported in the 2013 Lift Station Evaluation to be as high as 217 gallons per minute and the projected future peak hour flow to this station is 596 gallons per minute. Both the current and projected future peak hour flows exceed the capacity of the existing pumps. However, the City has not reported any surcharging or overflows at this station. In addition, per the available run time data, the station only runs a maximum of 5.4 hours in a single day, suggesting that the station has sufficient capacity to convey the current flows.

5.1.4.3.2 Force Main

At the current lift station capacity of 137 gpm, the velocity in the existing force main is approximately 3.5 feet per second with a total dynamic head of 96 feet. At the maximum reported current peak hour flow of 217 gpm into the lift station, the velocity would be 5.16 gallons per minute with a TDH of 145 feet. At the projected future peak hour flow of 596 gpm, the velocity in the existing force main would be approximately 15.4 feet per second, significantly exceeding the maximum recommended velocity of 8 feet per second.

5.1.4.3.3 Wet Well

The existing 8-foot-diameter wet well has an operating volume of approximately 1,165 gallons. At the current estimated peak hour flow, the maximum starts per hour is approximately three and at the projected flows, the maximum starts per hour would be approximately eight, which is less than the maximum recommended starts per hour of 12 for a duplex lift station. Therefore, the existing wet well has sufficient capacity to handle the current and projected future peak hour flows.

5.1.4.4 Electrical System

5.1.4.4.1 General

The existing utility feed is a 100 A, 480 VAC, 3-phase service. The estimated load of the two pumps and 6kVA 208/120 VAC transformer controls is 28.3kVA, 34A at 480V. The existing distribution equipment and control panels are in good condition. The PLC control panel is densely packed and it is unlikely any additional equipment could be added. The seal-off vault junction boxes and conduit seal-off fittings are rusted and, in general, poor condition.

5.1.4.4.2 Emergency Supply

The system has a 30 A, 480 VAC pin and sleeve receptacle for connection to an external generator via a 100 A rated manual transfer switch. Based on the estimated sizing above this connection is marginal in terms of being able to support both pumps and all ancillary loads.

5.1.4.4.3 Code Compliance Considerations

Labeling in the seal-off vault indicates there are intrinsically safe circuits however the equipment for these circuits is not obvious. At best, the conduits are misidentified as being intrinsically safe; at worst the PLC control panel is not compliant with the spacing and labeling requirements for intrinsically safe areas.

5.1.4.5 Alternatives

As summarized above, the pumps and force main are projected to be undersized at the future peak hour flow. The wet well has sufficient capacity for the projected peak hour flow and is in good condition. Two alternatives have been identified for serving the current and future service areas associated with this lift station and are summarized below. Figure 5-2, located at the end of this section, depicts the preferred alternative. Figures depicting all other alternatives are included in Appendix H.

5.1.4.5.1 Alternative No. 1 – Upgrade Lift Station on Current Site

Alternative No. 1 would upgrade the lift station and force main on the current lift station site. The existing force main would be replaced with an 8-inch diameter pipe and new 20-hp submersible pumps capable of conveying the projected future peak hour flow would be installed in the existing wet well. Electrical and control equipment would be replaced as needed to accommodate the new, larger pumps and to eliminate outdated components. Two subalternatives for completing these upgrades are summarized below.

5.1.4.5.2 Alternative No. 1A – Upgrade Station in Two Phases

Alternative No. 1A would replace the existing force main as a first phase of work, which will reduce the velocity and friction head in the system and increase the capacity of the existing pumps. The revised operating point would be within, but near the upper end of, the recommended operating range for the existing pumps. Once flows increase to near this revised lift station capacity, the second phase would replace the existing pumps with new larger submersible pumps capable of conveying the projected future peak hour flow. Electrical and controls equipment would be replaced as needed to accommodate the new, larger pumps and eliminate outdated components. Bypass pumping would be required for part or all of the second phase of work.

5.1.4.5.3 Alternative No. 1B – Upgrade Station in One Project

Alternative No. 1B would complete all of the proposed upgrades in a single project that would be completed in the near future.

Electrical Aspects of Alternatives No. 1A and No. 1B

The seal-off vault and all associated conduits/j-boxes/fittings should be removed and replaced with new, if any work at this site is done. Records should be checked to verify the presence (or not) of intrinsically safe circuits. If such circuits do enter the PLC control panel then the panel will need to be modified to become compliant with current codes. The existing service is acceptable for the proposed 20-hp pumps; however, the generator receptacle should be upsized to handle increased loads.

5.1.4.5.4 <u>Alternative No. 2 – Redirect Flows from the Hannegan Neighborhood to Gravity Sewers Upstream of the James Lift Station.</u>

Alternative No. 2 would replace the existing force main with an 8-inch pipe to increase the station capacity to approximately 400 gallons per minute. The existing lift station, which is in good condition, would be otherwise unmodified. The future flows from the new lift station proposed for the new Hannegan neighborhood would be directed to the gravity sewers that drain to the James Street Lift Station. However, diverting the projected peak hour flow of 96 gallons per minute from the Hannegan neighborhood would not sufficiently reduce the flow to the Bakerview Valley Lift Station to less than the revised capacity of 400 gallons per minute. <u>Therefore, this alternative is not recommended based on currently available information.</u>

5.1.4.6 Preferred Alternative

The existing lift station is currently operational, in good condition and the various components appear to have significant remaining useful life. Alternative No. 1A would keep the existing station in service while increasing the capacity through replacement of the existing force main. Once the force main is replaced, this station would have sufficient capacity to convey the current flow and provide approximately 200 gpm of additional capacity for future development. This alternative would maximize the useful life of the existing components and is, therefore, the preferred alternative. The preferred alternative is depicted on Figure 5-2.

(PHASE 1)
(PHASE 2)
4" DIA FORCE MAIN, PLUG AND ABANDON IN PLACE
CONTROL PANEL TO REMAIN
8 FT DIA WET WELL W/2 SUBMERSIBLE 10 HP PUMPS, WET WELL TO REMAIN, PUMPS TO BE REMOVED
VALVE VAULT TO REMAIN
10' 5' 0 10' 20'
SCALE: 1"=10'
EXISTING SITE AND DEMOLITION PLAN
8" DIA FORCE MAIN
ABANDONED 4" DIA FORCE MAIN
20 HP PUMPS IN EXISTING WET WELL
EXISTING CONTROL PANEL
EXISTING 8 FT DIA WET WELL
EXISTING VALVE VAULT
PROPOSED SITE PLAN
LEGEND:
EXISTING STRUCTURE
EXISTING STRUCTURE/PIPE/EQUIPMENT TO BE DEMOLISHED
EXISTING PIPE/EQUIPMENT TO BE ABANDONED
EXISTING PIPE/EQUIPMENT TO REMAIN
NEW PIPE/EQUIPMENT
NEW STRUCTURE
NEW ASPHALT PAVING
0 1"
ONE INCH AT FULL SCALE. IF NOT, SCALE ACCORDINGLY
CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
FIGURE 5-2
BAKERVIEW VALLEY LIFT STATION MODIFICATIONS
ALTERNATIVE NO. 1B
(PREFERRED ALTERNATIVE)
Gray & Osborne, Inc.
CONSULTING ENGINEERS

5.1.5 'C' STREET LIFT STATION

5.1.5.1 Existing Facilities

The existing 'C' Street Lift Station is a wet pit-dry pit lift station equipped with two constant speed, 5-hp Smith & Loveless pumps mounted in the dry well with an 8-foot-diameter concrete wet well. The station pumps through a 6-inch-diameter ductile iron force main. The lift station is situated in the right-of-way for C Street. The details of the lift station components are summarized in Table 5-4 below.

TABLE 5-4

'C' Street Lift Station

Year Built	1973
Type	Wet Pit – Dry Pit
Pump Manufacturer	Smith & Loveless
Number of Pumps	2
Horsepower	5
Backup Power	100 A Outlet for Portable Generator
Wet Well Diameter	8 feet
Force Main	6-inch ductile iron
Force Main Length	100 linear feet

5.1.5.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this lift station is generally in good condition. The wet well has no visible evidence of deterioration or spalling and the piping in the wet well has limited corrosion. The dry pit has no evidence of corrosion. The pumps and valves are operational and the operations staff has not reported any problems with this station.

5.1.5.3 Capacity Analysis

5.1.5.3.1 <u>Pumps</u>

As presented in Table 5-1 above, the design flow for the existing lift station is approximately 500 gallons per minute and the tested lift station capacity is approximately 372 gpm. Based on a comparison of the system curve for this lift station and associated force main with the manufacturer's pump curves (see Appendix F), the pumps should be capable of pumping approximately 475 gpm in this system. However, record drawings for this lift station were not available at the time of this report. Per the pump curve, if the total head in the system is 3 feet greater than current records would support, the capacity of this lift station would be reduced to the tested capacity of 372 gpm. At this time, it is suspected that the difference between the calculated and tested capacities is due to inaccuracy in available records and the pumps are likely operating at their rated capacities.

Per the 2013 Lift Station Evaluation, the current flow to the 'C' Street Lift Station is estimated to be between 31 and 52 gallons per minute. The station has a tested capacity of 372 gallons per minute, which is significantly greater than the current peak hour flow and based on a review of runtime data, this station runs less than one hour per day, even during wet weather. However, the *2009 Comprehensive Sewer Plan* (2009 Plan) projected the peak hour flow to increase to 1,329 gallons per minute by 2026. Based on a review of the sewer system model prepared as part of the 2009 Plan, it appears that a majority of the additional flow is anticipated to be stormwater, as this station serves a portion of the combined sewer system. It is unclear why the stormwater entering this station is projected to dramatically increase in the next 10 years and City staff does not know what assumptions were used that led to this projection. If the peak hour flow increases to 1,329 gallons per minute as projected in the 2009 Plan, the existing lift station would be significantly undersized. However, due to the uncertainty associated with the flow projections, additional monitoring and ongoing review of future flows is recommended.

5.1.5.3.2 Force Main

At the current lift station capacity of 372 gpm, the velocity in the existing force main is approximately 4.2 feet per second with a total dynamic head of 21 feet. If the flows reach the projected future peak hour flow presented in the 2009 Plan, the velocity in the existing force main would exceed 9 feet per second.

5.1.5.3.3 Wet Well

The existing 8-foot-diameter wet well has an operating volume of approximately 1,125 gallons. At the current estimated peak hour flow, the maximum starts per hour is less than one start per hour. At the projected flows, the maximum starts per hour would be more than 17, which is greater than the maximum recommended starts per hour of 12 for a duplex lift station.

5.1.5.4 Electrical System

5.1.5.4.1 General

The existing utility feed is a 100 A, 240 VAC, 3-phase service. All loads are serviced from the Smith & Loveless panel, supplied by a 60 A disconnect for a possible maximum of 25 kVA of load. While the electrical feed is appropriately sized, the Smith & Loveless panel is in poor condition. The PLC control panel is densely packed and it is unlikely any significant modifications could be added. The electrical distribution and control stand at street level are in good condition.

5.1.5.4.2 Emergency Supply

The system has a 100A, 240 VAC pin and sleeve receptacle for connection to an external generator via a 100 A rated manual transfer switch. It is sufficiently sized for existing loads.

5.1.5.4.3 Code Compliance Considerations

The interior and exterior of the Smith & Loveless control panel is corroded, posing undo risk to electrical components and safety of personnel. While not a code issue, a 240 VAC, 3-phase system, carries a "high leg" which requires extra attention when working on the electrical system.

5.1.5.5 Alternatives

Due to the uncertainty associated with the projected flows to this station and the understanding that most of the projected additional flow would be stormwater, no upgrades to the existing lift station are recommended at this time. Flows to this station are currently well below the capacity of the existing facility and the City has not reported any operation and maintenance concerns. Therefore, it is recommended that the City continue to monitor flow to this station. If stormwater flows to the station increase as projected by the 2009 Plan, alternatives for separating these flows from the sanitary sewer system should be considered.

5.1.6 HORTON LIFT STATION

5.1.6.1 Existing Facilities

The existing Horton Lift Station consists of three constant speed, 20-hp submersible pumps mounted in an 8-foot-diameter concrete wet well that pump through an 8-inch-diameter ductile iron force main. The lift station is situated on an approximately 6,200 square foot parcel owned by the City. The existing lift station site covers approximately 400 square feet and the remainder of the parcel is undeveloped. The details of the lift station components are summarized in Table 5-5 below.

TABLE 5-5

Horton Lift Station

Year Built	1988
Type	Submersible
Pump Manufacturer	Flygt
Number of Pumps	3
Horsepower	20
Backup Power	200 A Outlet for Portable Generator
Wet Well Diameter	8 feet
Force Main	8-inch ductile iron
Force Main Length	2,400 linear feet

5.1.6.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this lift station is generally in good condition and reliable, although the station experiences frequent pump cycling. The wet well has no visible evidence of deterioration or spalling and the piping in the wet well has limited corrosion. The third pump was installed in this station approximately 3 years ago and the two original pumps were replaced approximately 10 years ago. All three pumps, valves and vaults are operational and the existing structures are in reusable condition.

5.1.6.3 Capacity Analysis

5.1.6.3.1 Pumps

As presented in Table 5-1 above, the design flow for one pump running is approximately 600 gallons per minute and the tested lift station capacity with one pump running is approximately 638 gpm. Based on a comparison of the system curve for this lift station and associated force main with the manufacturer's pump curves (see Appendix F), one pump should be capable of pumping approximately 645 gpm in this system. With two pumps running, the station should be capable of pumping 760 gpm.

Per the 2013 Lift Station Evaluation, the current flow to the Horton Lift Station is estimated to be as high as 704 gallons per minute, which exceeds the current capacity of one pump but is within the calculated lift station capacity (two pumps running) of 760 gpm. The projected future peak hour flow is estimated to increase to 1,305 gallons per minute.

5.1.6.3.2 Force Main

At the current estimated lift station capacity of 760 gpm, the velocity in the existing force main is approximately 4.7 feet per second with a total dynamic head of 76 feet. At the projected future peak hour flow of 1,305 gpm, the velocity in the existing force main would be slightly above the maximum recommended velocity of 8 feet per second with a total dynamic head of 81 feet.

5.1.6.3.3 Wet Well

The existing 8-foot-diameter wet well has an operating volume of approximately 1,354 gallons. At the current estimated peak hour flow, the maximum starts per hour is approximately seven starts per hour. At the projected future flows, the maximum starts per hour would be more than 14, which is less than the maximum recommended starts per hour of 18 for a triplex lift station. Therefore, the existing wet well has sufficient capacity to handle the current flows and projected future peak hour flows.

5.1.6.4 Electrical System

5.1.6.4.1 General

The existing utility feed is a 125 A, 480 VAC, 3-phase service. The estimated load of the three pumps and 5kVA 240/120 VAC transformer controls is 71.5 kVA, 90 A at 480V. The existing distribution equipment and control panels are in good condition. The PLC control panel and motor starter panel have ample room for modification.

5.1.6.4.2 Emergency Supply

The system has a 200 A, 480 VAC, 3-phase pin and sleeve receptacle for connection to an external generator via a 200 A rated manual transfer switch. It is sufficiently sized for existing loads.

5.1.6.4.3 Code Compliance Considerations

The circuit breakers in the load center for ancillary loads are not well seated and the load center is starting to show signs of rust.

5.1.6.5 Alternatives

This station is generally in good condition but the lift station capacity will be exceeded at the projected design flow. It will be necessary to replace either the pumps or the force main to convey the projected flow. Figure 5-3, located at the end of this section, depicts the preferred alternative. Figures depicting all other alternatives are included in Appendix H.

5.1.6.5.1 Alternative No. 1 – Replace Force Main.

Alternative No. 1A would replace the existing 8-inch diameter ductile iron force main with a new 12-inch-diameter pipe. The larger pipe will have lower flow velocity and friction head and, with two of the three existing pumps pumping, the lift station will have sufficient capacity to convey the projected design flow. With one pump pumping, the velocity in the force main will be slightly lower than 4 feet per second, which is high enough to maintain solids in suspension. With two pumps running, the force main velocity would be slightly above 5 feet per second. The revised station capacity would be approximately 1,600 gallons per minute. The pumps can operate on the recommended portion of the pump curve for all pumping conditions.

Alternative 1B would retain the existing 8-inch-diameter force main and a second parallel 8-inch diameter force main would be installed. This would provide greater operational flexibility with a lower capital cost.

Electrical Aspects of Alternative No. 1

The City should consider replacing the load center for something more suited for commercial/industrial use over the current residential style panel.

5.1.6.5.2 Alternative No. 2 – Replace Pumps.

Alternative No. 2 would retain the existing force main and install three new 34-hp pumps capable of conveying the future design flow. The pump motor starters and other associated appurtenances would also be replaced to accommodate the larger motors. With two pumps pumping through the 8-inch force main, the velocity will be approximately 8 feet per second, which is the maximum recommended flow velocity.

Electrical Aspects of Alternative No. 2

The City should consider replacing the load center for something more suited for commercial/industrial use over the current residential style panel. The incoming utility feed should be further evaluated for capacity. The main disconnect breaker and all wire and conduit to the motor starter panel will need to be upsized. The motor starter panel will require new motor starters, recommended to be RVSS.

5.1.6.6 Preferred Alternative

Although replacing the existing force main (Alternative No. 1) may have a higher capital cost than replacing the pumps (Alternative No. 2), installing a new, larger diameter or parallel force main will reduce the headloss through the system. Under Alternative No. 2, the velocity in the existing force main would be at the maximum recommended velocity, resulting in significant headloss. Therefore, the operational costs associated with Alternative No. 1 are expected to be lower. Alternative No. 1B would have lower operating costs and provide greater operational flexibility and is, therefore, the preferred alternative. The preferred alternative is shown on Figure 5-3.

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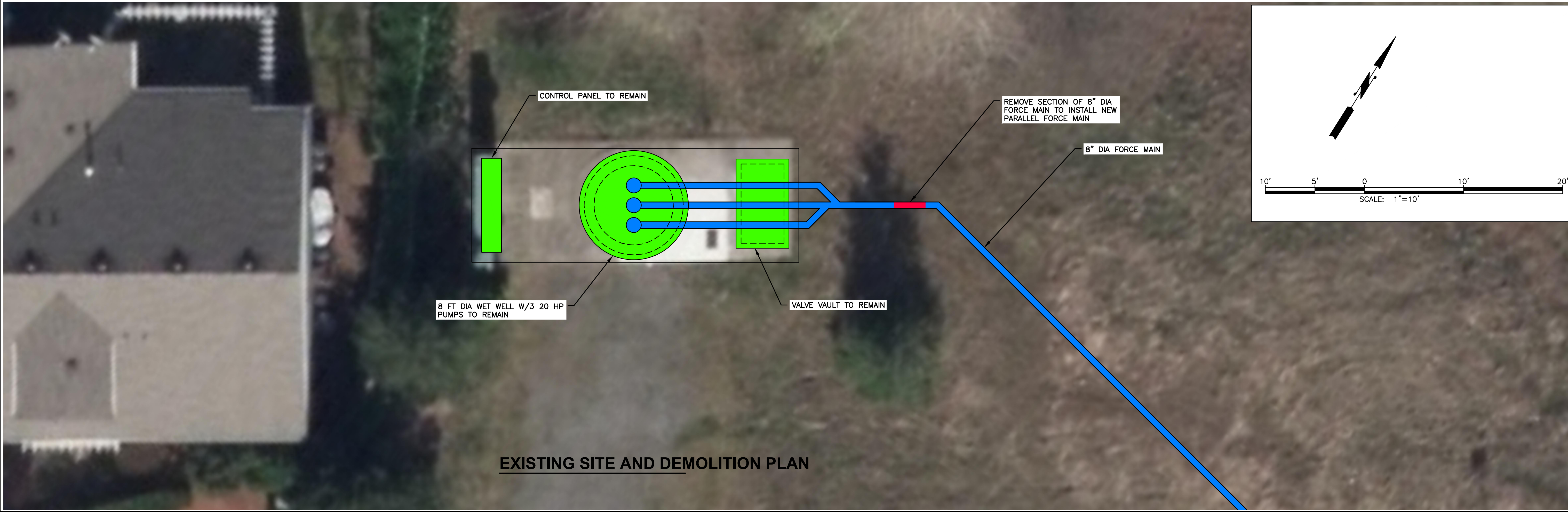
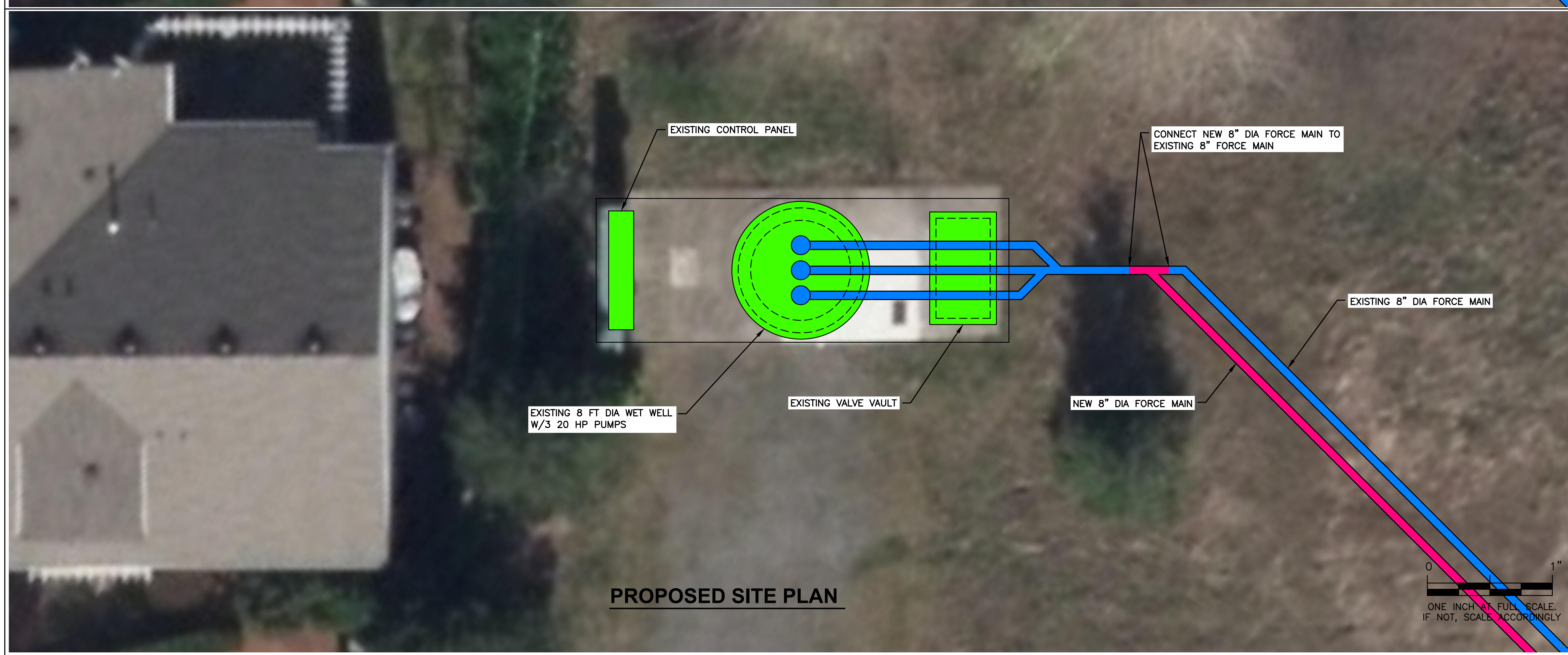


EXISTING SITE AND DEMOLITION PLAN



PROPOSED SITE PLAN

LEGEND:

- EXISTING STRUCTURE
- EXISTING STRUCTURE/PIPE/EQUIPMENT TO BE DEMOLISHED
- EXISTING PIPE/EQUIPMENT TO BE ABANDONED
- EXISTING PIPE/EQUIPMENT TO REMAIN
- NEW PIPE/EQUIPMENT
- NEW STRUCTURE
- NEW ASPHALT PAVING

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN

FIGURE 5-3

HORTON LIFT STATION MODIFICATIONS
ALTERNATIVE NO. 1B
(PREFERRED ALTERNATIVE)

Gray & Osborne, Inc.
CONSULTING ENGINEERS

L:\Bellingham\15448.00 WW Conveyance Plan\Lift Station Amendment\FIGURES\HORTON_ALT-1B.dwg, 4/26/2016 11:05:38 AM, DWG To PDF.pc3

5.1.7 JAMES STREET LIFT STATION

5.1.7.1 Existing Facilities

The existing James Street Lift Station consists of two constant speed, 88-hp Flygt submersible pumps mounted in a 10-foot-diameter concrete wet well that pumps through a 10-inch-diameter force main that has been relined using cured-in-place pipe (CIPP). The lift station is situated within the right-of-way for James Street. The existing lift station site covers roughly 1,200 square feet. The area to the north of the lift station is also within the right-of-way and is undeveloped. The site to the east of the lift station is a park/open space area owned by the City of Bellingham. The details of the lift station components are summarized in Table 5-6 below.

TABLE 5-6

James Street Lift Station

Year Built	1985
Type	Submersible
Pump Manufacturer	Flygt
Number of Pumps	2
Horsepower	88
Backup Power	225 kW generator
Wet Well Diameter	10 feet
Force Main	10-inch cured-in-place
Force Main Length	1,550 linear feet

5.1.7.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this lift station is in generally in good condition but has become unreliable. This station experiences frequent cycling and pumps have failed on multiple occasions. The City replaced the two pumps recently and they maintain two replacement pumps on the shelf in the event that a pump fails. VFDs were added to this station recently in an effort to mitigate this problem but it has not been successful in protecting the pumps. All of the existing structures are in reusable condition. The wet well has no visible evidence of deterioration or spalling and the piping in the wet well has limited corrosion.

5.1.7.3 Capacity Analysis

5.1.7.3.1 <u>Pumps</u>

As presented in Table 5-1 above, the design flow for the existing lift station is approximately 1,200 gallons per minute and the tested lift station capacity is approximately 1,728 gpm. The existing force main is cured in place pipe (CIPP) installed inside an existing 10-inch diameter host pipe and the inside diameter of the pipe is not known. For the purposes of this evaluation, the inside diameter was estimated based on the assumed inside diameter of the host pipe and a typical wall thickness for CIPP rated for 150 psi. A comparison of the system curve for this lift station, which is based on the estimated inside diameter of the force main, with the manufacturer's pump curves (see Appendix F), the pumps should be capable of pumping approximately 1,600 gpm in this system.

Per the 2013 Lift Station Evaluation, the current flow to the James Street Lift Station is estimated to be as high as 1,367 gallons per minute, which exceeds the design capacity of this lift station but does not exceed the tested or calculated capacity of the lift station. The projected future peak hour flow is estimated to increase to 3,737 gallons per minute, which will significantly exceed the capacity of the existing pumps.

5.1.7.3.2 Force Main

At the current tested lift station capacity of 1,728 gpm, the velocity in the existing force main is approximately 9 feet per second with a total dynamic head of 127 feet. At the projected future peak hour flow of 3,737 gpm, the velocity in the existing force main would exceed 18 feet per second with a total dynamic head greater than 280 feet.

5.1.7.3.3 Wet Well

The existing 10-foot-diameter wet well has an operating volume of approximately 1,762 gallons. Assuming constant speed operation, the current the maximum starts per hour is approximately 12 start per hour. At the projected flows, the maximum starts per hour would be more than 30 with constant speed operation, which is greater than the maximum recommended starts per hour of 12 for a duplex lift station. Although these pumps are equipped with VFDs, which should reduce the number of pump starts, the City has reported frequent cycling of these pumps which has burned out multiple pumps. At the projected future flows, the existing wet well would be significantly undersized for constant speed operation and it is unlikely that VFD operation could sufficiently reduce the starting frequency to protect the pump motors.

5.1.7.4 Electrical System

5.1.7.4.1 General

The existing utility feed is a 200 A, 480 VAC, 3-phase service. The estimated load of the two pumps, 3-hp digester (unused) and 5kVA 240/120 VAC transformer controls is 193 kVA, 232 A at 480V. The existing distribution equipment and control panels are in good condition. The PLC control panel is densely packed and it is unlikely any significant modifications could be added. The Owner has stated that the VFD starters that are installed would burn up the motors and as such they have been disabled, only the FVNR bypass portion of the motor starters are used.

5.1.7.4.2 Emergency Supply

The system has an onsite 225 kW generator capable of 338 A of 480 VAC, 3-phase power.

5.1.7.4.3 Code Compliance Considerations

Unless a specific automatic lockout exists to limit only one running pump, the utility service is undersized. The 5 kVA transformer for ancillary loads does not have the clearance space required by the NEC. The control panel is mounted on unsecured wooden blocks. The insulation on one of the conductors inside the ATS is damaged.

5.1.7.5 Alternatives

The James Street Lift Station is currently near or at capacity and the City has had numerous issues with the pumps at this station. This station will be well over capacity at the projected design flow and the wet well, pumps and force main will all be over capacity. Two alternatives were evaluated for replacing this station. Figure 5-4, located at the end of this section, depicts the preferred alternative. Figures depicting all other alternatives are included in Appendix H.

5.1.7.5.1 Alternative No. 1 – Upgrade Lift Station and Force Main in Two Phases

Alternative No. 1 would upgrade the lift station and force main in two phases. The force main capacity would be increased as part of the first phase of work and the lift station would be replaced as part of the second phase of work. In the first phase of work, the existing 10-inch-diameter ductile iron force main would be replaced with a new 16-inch ductile iron pipe (Alternative 1A) or the existing force main would be retained and a parallel 10-inch force main would be installed (Alternative 1B) to increase the capacity of the existing pumps to approximately 2,400 gpm. The entire lift station would be replaced at a later date as flows increase. This option is not recommended, however, because the wet well is undersized to handle 2,400 gallons per minute. At 2,400 gallons per minute, the pumps would experience upwards of 20 starts per hour, which is significantly greater than the typical recommended design maximum starts of 12 per hour.

5.1.7.5.2 Alternative No. 2 – Replace Lift Station and Force Main in Single Project

Since the capacity of all major components will be exceeded at the projected future flow rate, Alternative No. 2 would replace the entire lift station with a new lift station. It appears that a new wet well and valve vault could be constructed immediately north of the existing wet well. Based on the projected flow and head conditions, the new lift station would likely be a triplex lift station sized to convey the design peak hour flow with two 70-hp pumps running. Given the constrained site, a lift station equipped with three submersible pumps may be the most suitable configuration for this site. Bypass pumping may be required during replacement of the control panels and motor starters. Additional force main capacity would also be constructed. The force main could be replaced with a new 16-inch-diameter force main (Alternative 2A), or the existing force main could be retained and a parallel 10-inch-diameter force main could be installed (Alternative 2B). If parallel force mains are installed, one pipe could be taken out of service during low flow periods to maintain the recommended minimum velocity.

Electrical Aspects of Alternative No. 2

All items should be addressed if any changes are made to this facility. This includes replacement of the VFDs.

5.1.7.5.3 <u>Alternative No. 3 – Replace Pumps, Construct Second Wet Well and Increase Force Main Capacity</u>

Based on visual inspection, all of the existing structures are in reusable condition. Alternative No. 3 would retain the existing lift station structures and the station would be upgraded to convey the design flow. The station would remain a duplex lift station that will be equipped with two 140-horsepower submersible pumps. A second 10-foot-diameter wet well would be constructed adjacent to the south side of the existing wet well to provide additional operating volume. Discharge piping and valves will be replaced in the existing valve vault and the electrical equipment will be upgraded for the larger pumps and to replace outdated equipment. Bypass pumping will be required during most of the construction. Additional force main capacity would also be constructed. The force main could be replaced with a new 16-inch diameter force main (Alternative 3A) or the existing force main could be retained and a parallel 10-inch-diameter force main could be installed (Alternative 3B). If parallel force mains are installed, one pipe could be taken out of service during low flow periods to maintain the recommended velocity.

Electrical Aspects of Alternative No. 3

The entire electrical distribution, including the ATS would have to be upsized. The generator would only be capable of supporting one pump. It is unlikely that the process control panel would meet the modified system requirements and has no room for modification.

5.1.7.6 Preferred Alternative

Alternative No. 1 was determined to be infeasible and discarded from consideration. The existing force main has been restored by cured in place pipe and likely has significant remaining life. Alternative 2 would construct a new wet well that would be large enough to accommodate three pumps, which will provide greater operational flexibility and is therefore, preferred over Alternative No. 3, which would install a second smaller wet well. Installing a second parallel force main, as proposed as part of Alternative No. 2B, would be lower in cost and would provide additional operational flexibility. Therefore, Alternative No. 2B, which would construct a new station and install a second parallel force main, is the preferred alternative and is shown on Figure 5-4.

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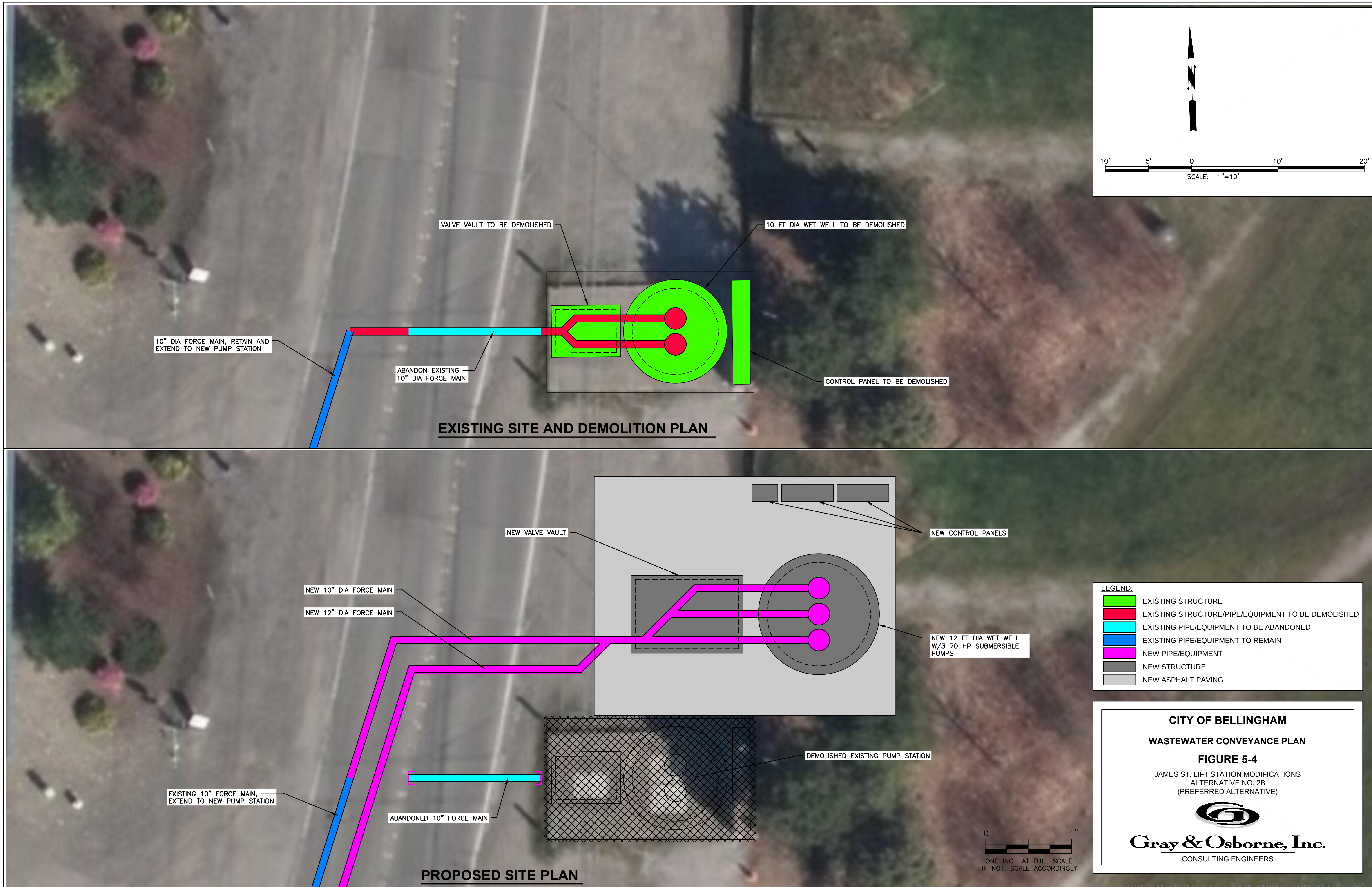

10' 5' 0 10' 20'
SCALE: 1"=10'
VALVE VAULT TO BE DEMOLISHED
10 FT DIA WET WELL TO BE DEMOLISHED
10" DIA FORCE MAIN, RETAIN AND EXTEND TO NEW PUMP STATION
ABANDON EXISTING 10" DIA FORCE MAIN
CONTROL PANEL TO BE DEMOLISHED
EXISTING SITE AND DEMOLITION PLAN
NEW CONTROL PANELS
NEW VALVE VAULT
NEW 10" DIA FORCE MAIN
NEW 12" DIA FORCE MAIN
NEW 12 FT DIA WET WELL W/3 70 HP SUBMERSIBLE PUMPS
DEMOLISHED EXISTING PUMP STATION
EXISTING 10" FORCE MAIN, EXTEND TO NEW PUMP STATION
ABANDONED 10" FORCE MAIN
PROPOSED SITE PLAN
LEGEND:
EXISTING STRUCTURE
EXISTING STRUCTURE/PIPE/EQUIPMENT TO BE DEMOLISHED
EXISTING PIPE/EQUIPMENT TO BE ABANDONED
EXISTING PIPE/EQUIPMENT TO REMAIN
NEW PIPE/EQUIPMENT
NEW STRUCTURE
NEW ASPHALT PAVING
0 1"
ONE INCH AT FULL SCALE. IF NOT, SCALE ACCORDINGLY
CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
FIGURE 5-4
JAMES ST. LIFT STATION MODIFICATIONS
ALTERNATIVE NO. 2B
(PREFERRED ALTERNATIVE)
Gray & Osborne, Inc.
CONSULTING ENGINEERS

5.1.8 MITCHELL WAY LIFT STATION

5.1.8.1 Existing Facilities

The existing Mitchell Way Lift Station consists of two constant speed, 10-hp Myers submersible pumps mounted in an 8-1/2-foot-diameter concrete wet well that pump through a 4-inch-diameter ductile iron force main. A second 8-1/2-foot-diameter wet well is located adjacent to the wet well containing the pumps. The lift station is situated near the intersection of Mitchell Way and West Bakerview Road on a parcel owned by the Port of Bellingham. The nearby area is developed with the Port-owned Bellingham Airport. The existing lift station covers roughly 875 square feet and is located at the southwest corner of an undeveloped lot. The details of the lift station components are summarized in Table 5-7 below.

TABLE 5-7

Mitchell Way Lift Station

Year Built	1972
Type	Submersible
Pump Manufacturer	Myers
Number of Pumps	2
Horsepower	10
Backup Power	80 kW generator
Wet Well Diameter	(2) 8-1/2 ft dia. wet wells
Force Main	4-inch Ductile Iron
Force Main Length	920 linear feet

5.1.8.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this lift station is in generally good condition for its age and is reliable. Modifications have been made to this station, including installation of a new wet well hatch and relocation of the valves to a new concrete valve vault. The two wet wells have evidence of minor deterioration at the surface. The piping in the wet well has moderate corrosion. The existing structures have evidence of minor deterioration but are in reusable condition.

5.1.8.3 Capacity Analysis

5.1.8.3.1 <u>Pumps</u>

As presented in Table 5-1 above, the design flow for the existing lift station is approximately 350 gallons per minute. However, the tested lift station capacity is significantly lower than the design flow at approximately 158 gpm. Based on a comparison of the system curve for this lift station and associated force main with the manufacturer's pump curves (see Appendix F), the pumps should be capable of pumping approximately 160 gpm in this system. It is not known why the current capacity of the lift station is approximately half of the design capacity. Per the record drawings, the original pumps were equipped with 7.5-horsepower pumps. These pumps were recently replaced with the existing 10-horsepower pumps and it is unlikely that the original pumps had higher capacity than the new, larger pumps.

Per the 2013 Lift Station Evaluation, the current flow to the Mitchell Way Lift Station is estimated to be as high as 100 gallons per minute, which is less than the tested capacity of the lift station. The projected future peak hour flow is estimated to increase to 801 gallons per minute, which will significantly exceed the capacity of the existing pumps.

5.1.8.3.2 Force Main

At the current tested lift station capacity of 158 gpm, the velocity in the existing force main is approximately 4 feet per second with a total dynamic head of 58 feet. At the projected future peak hour flow 801 gpm, the velocity in the existing force main would exceed 20 feet per second with a total dynamic head greater than 500 feet.

5.1.8.3.3 Wet Well

The two existing 8-1/2-foot-diameter wet wells have a combined operating volume of approximately 2,547 gallons. At the maximum estimated current peak hour flow of 158 gpm, the maximum starts per hour is approximately one start per hour. At the projected flows, the maximum starts per hour would be approximately five. Therefore, the two existing wet wells are somewhat oversized for the current flows and have sufficient capacity to handle the projected future peak hour flows. Despite the long wet well detention times and infrequent pump starts, the City has not reported odor or corrosion concerns associated with this station.

5.1.8.4 Electrical System

5.1.8.4.1 General

The existing utility feed is a 100 A, 480 VAC, 3-phase service. The estimated load of the two pumps and 5kVA 240/120 VAC transformer controls is 29.3 kVA, 36 A at 480V. The existing distribution equipment and control panels are in good condition as the majority of it was replaced with the addition of the generator. The PLC control panel and motor starter panel have ample room for modification.

5.1.8.4.2 Emergency Supply

The City is just completing a project which replaces the generator receptacle and manual transfer switch with a new fully automatic transfer switch and 80 kW generator. The new generator has a maximum output of 120 A making it sufficiently sized for existing loads.

5.1.8.5 Alternatives

This station is approximately 45 years old but appears to be in generally good condition. The City recently moved the valves on the discharge piping from the wet well to a new valve vault. The station has twin wet wells which have capacity to handle the projected design flow. The pumps and force main need to be upgraded to convey the future design flow. Figure 5-5, located at the end of this section, depicts the preferred alternative. Figures depicting all other alternatives are included in Appendix H.

5.1.8.5.1 Alternative No. 1 – Replace Pumps and Force Main

Alternative No. 1 would retain the existing twin wet wells, which have capacity for the future design flows. Based on visual inspection of the wet wells, despite being over 40 years old, the structures are in good condition and suitable for continued service. Recoating the interior of both structures would be recommended to extend the life of these wet wells. New 20-hp pumps, piping and electrical controls would be installed. The existing 4-inch-diameter discharge piping would be replaced with 6-inch-diameter ductile iron pipe and the 4-inch-diameter force main would be replaced with a new 8-inch force main. The discharge piping would be re-oriented from the current vertical orientation to a more typical side-by-side arrangement, which will require installation of a new, larger valve vault. Bypassing flow during most of construction will be required.

Electrical Aspects of Alternative No. 1

Increasing the horsepower of the motors for additional flow would result in only minor cable and conduit work, the existing distribution can supply the proposed 20-hp motors.

5.1.8.5.2 Alternative No. 2 – Replace Lift Station and Force Main

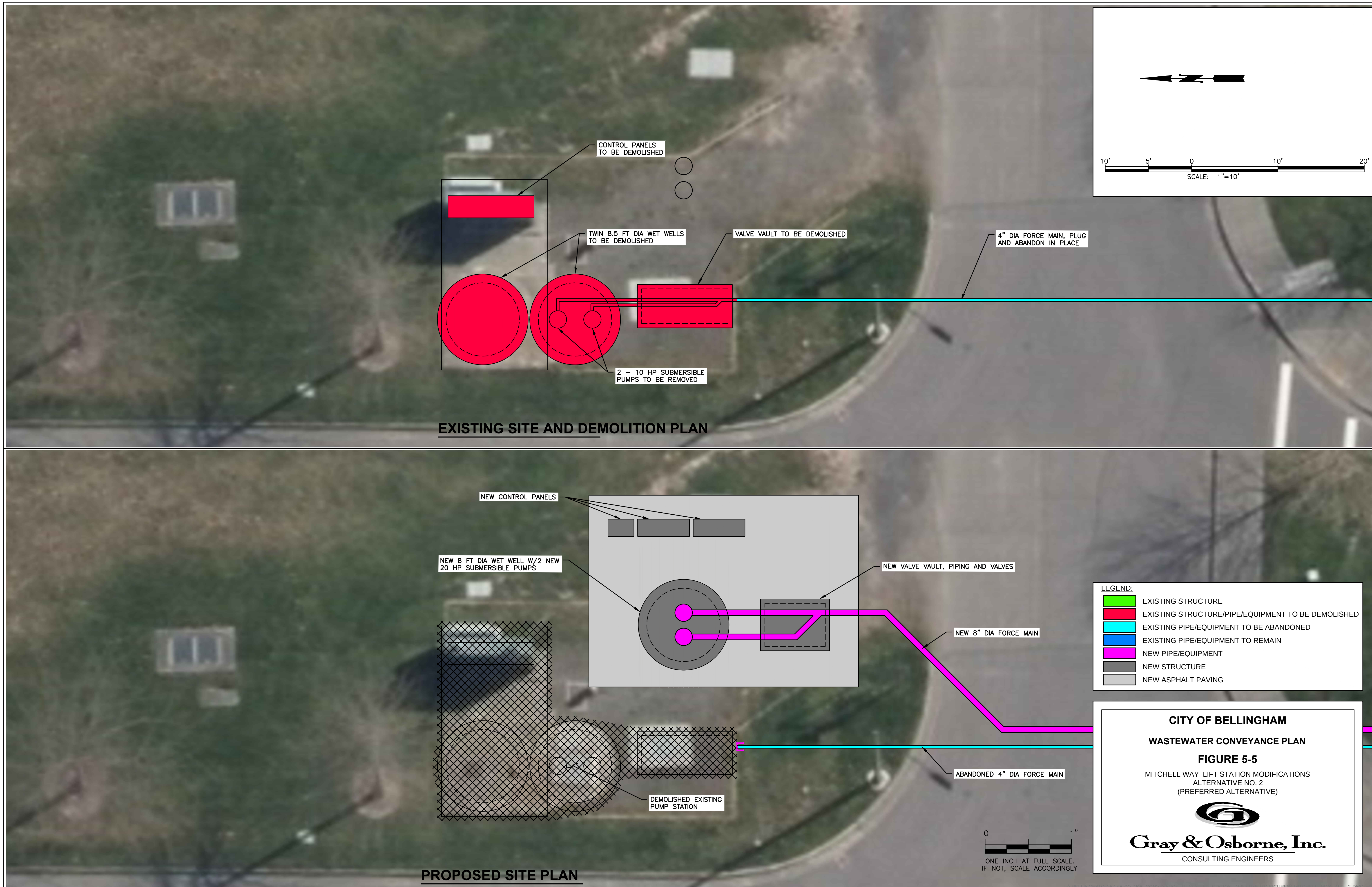
Alternative No. 2 would construct a new lift station, including 20-hp pumps, wet well, controls and force main, adjacent to the east side of the existing lift station site. The station would include a new 6-foot-diameter wet well and 8-inch force main. New pumps, control panels, and motor starters would be provided with sufficient capacity to convey the projected future design flow. The existing lift station is on and surrounded by Port-owned property; thus, it would likely be necessary to negotiate this relocation with the Port. Minimal bypass pumping would be required during construction.

Electrical Aspects of Alternative No. 2

As much of the electrical was recently redone, some of the equipment could be reused for a new station.

5.1.8.6 Preferred Alternative

The existing pump discharge piping is oriented in a non-standard vertical orientation, which makes accessing the valves and piping somewhat challenging. If Alternative No. 1 is constructed, the existing wet wells, which are over 40 years old, are retained, The piping penetrations would need to be modified and the structures would need to be rehabilitated prior to installing the new pipes and pumps. Bypass pumping would be required throughout construction. Due to the age of the wet wells and the challenges during construction, Alternative No. 2, which will replace the lift station with a new station, is the preferred alternative and is shown on Figure 5-5.


CONTROL PANELS TO BE DEMOLISHED
TWIN 8.5 FT DIA WET WELLS TO BE DEMOLISHED
VALVE VAULT TO BE DEMOLISHED
4" DIA FORCE MAIN, PLUG AND ABANDON IN PLACE
2 – 10 HP SUBMERSIBLE PUMPS TO BE REMOVED
EXISTING SITE AND DEMOLITION PLAN
10' 5' 0 10' 20'
SCALE: 1"=10'
NEW CONTROL PANELS
NEW 8 FT DIA WET WELL W/2 NEW 20 HP SUBMERSIBLE PUMPS
NEW VALVE VAULT, PIPING AND VALVES
NEW 8" DIA FORCE MAIN
ABANDONED 4" DIA FORCE MAIN
DEMOLISHED EXISTING PUMP STATION
PROPOSED SITE PLAN
LEGEND:
EXISTING STRUCTURE
EXISTING STRUCTURE/PIPE/EQUIPMENT TO BE DEMOLISHED
EXISTING PIPE/EQUIPMENT TO BE ABANDONED
EXISTING PIPE/EQUIPMENT TO REMAIN
NEW PIPE/EQUIPMENT
NEW STRUCTURE
NEW ASPHALT PAVING
0 1"
ONE INCH AT FULL SCALE. IF NOT, SCALE ACCORDINGLY
CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
FIGURE 5-5
MITCHELL WAY LIFT STATION MODIFICATIONS
ALTERNATIVE NO. 2
(PREFERRED ALTERNATIVE)
Gray & Osborne, Inc.
CONSULTING ENGINEERS

5.1.9 NORTH MITCHELL WAY LIFT STATION

5.1.9.1 Existing Facilities

The existing North Mitchell Way Lift Station consists of two constant speed, 7.5-hp submersible pumps mounted in an 8-foot-diameter concrete wet well that pump through a 6-inch-diameter HDPE force main. The lift station is situated along Mitchell Way on a parcel owned by the Port of Bellingham. The nearby area to the southwest is developed with the Port-owned Bellingham Airport. The existing lift station covers roughly 875 square feet. The adjacent land to the north and east is also owned by the Port and appears to be undeveloped. The details of the lift station components are summarized in Table 5-8 below.

TABLE 5-8

North Mitchell Way Lift Station

Year Built	2001
Type	Submersible
Pump Manufacturer	Flygt
Number of Pumps	2
Horsepower	7.5
Backup Power	80 kW generator
Wet Well Diameter	8 feet
Force Main	6-inch HDPE
Force Main Length	1,965 linear feet

5.1.9.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this lift station is relatively new and in good condition. This station is reportedly reliable and all of the existing equipment and structures are in reusable condition. A generator was added to this station in the past year to provide additional reliability.

5.1.9.3 Capacity Analysis

5.1.9.3.1 <u>Pumps</u>

As presented in Table 5-1, the design capacity is reportedly 525 gallons per minute. Per drawdown testing performed by CH2M Hill in 2014 (see Appendix F), the tested capacity for the existing lift station is approximately 265 gallons per minute. Based on a comparison of the system curve for this lift station and associated force main with the manufacturer's pump curves (see Appendix F), the pumps should be capable of pumping approximately 260 gpm in this system. It is not know why the tested and calculated capacities of the lift station are approximately 1/2 of the design capacity.

Per the 2013 Lift Station Evaluation, the current flow to the North Mitchell Way Lift Station is estimated to be approximately 5 gallons per minute, which is significantly less than the tested capacity of the lift station. This station is reported to have very low run times. The projected future peak hour flow is estimated to increase to 668 gallons per minute, which exceeds the capacity of the existing pumps.

5.1.9.3.2 Force Main

At the current lift station capacity of 265 gpm, the velocity in the existing force main is approximately 3 feet per second with a total dynamic head of 36 feet. At the projected future peak hour flow of 668 gpm, the velocity in the existing force main would be approximately 7.5 feet per second with a total dynamic head greater than 100 feet.

5.1.9.3.3 Wet Well

The existing 8-foot-diameter wet well has an operating volume of approximately 752 gallons. At the maximum estimated current peak hour flow of 5 gpm, the maximum starts per hour is approximately one start every 10 hours. At the projected future flows, the maximum starts per hour would be approximately 13, which slightly exceeds the maximum recommended starts per hour. However, the lowest influent pipe to the wet well is approximately 6-feet higher than the "Pump On" elevation and the current operating depth of 2 feet could be increased without impacting the gravity sewers that drain to this station. If the operating depth is increased to 3 feet, the starts per hour would be reduced to nine starts per hour, which is within the recommended range.

5.1.9.4 Electrical System

5.1.9.4.1 General

The existing utility feed is a 100 A, 480 VAC, 3-phase service. The estimated load of the two pumps and 5kVA 240/120 VAC transformer controls is 24.5 kVA, 30 A at 480V. The existing distribution equipment and control panels are in good condition. The PLC control panel is densely packed and it is unlikely any additional equipment could be added.

5.1.9.4.2 Emergency Supply

The City is just completing a project which replaces the generator receptacle and manual transfer switch with a new fully automatic transfer switch and 80 kW generator. The new generator has a maximum output of 120 A making it sufficiently oversized for existing loads.

5.1.9.5 Alternatives

This station is approximately 15 years old and has significant remaining useful life. However, projected future flows will exceed the capacity of the existing pumps. The wet well and force main have sufficient capacity to handle the projected flows. One alternative was considered for increasing the capacity of this station. Figure 5-6, located at the end of this section, depicts the preferred alternative.

5.1.9.5.1 Alternative No. 1 – Replace Pumps

Alternative No. 1 would retain the existing wet well, force main and any controls that can be used with the larger pumps, as shown on Figure 5-14. The existing pumps would be replaced with 35-hp pumps capable of conveying the projected design flows. The “Pump On” elevation would be raised one foot to reduce pump cycling. Otherwise, no additional modifications would be recommended.

Electrical Aspects of Alternative No. 1

As much of the electrical was recently redone, some of the equipment could be reused for a new station but the incoming utility service would need to be upsized. The generator should be able to run both pumps at the larger horsepower.

5.1.9.6 Preferred Alternative

The North Mitchell Way Lift Station is relatively new and in very good condition. With the exception of the pumps, the existing facilities have sufficient capacity to handle the projected future flows with only minor modification of the control points. Therefore, Alternative No. 1, depicted on Figure 5-6, is the preferred alternative.

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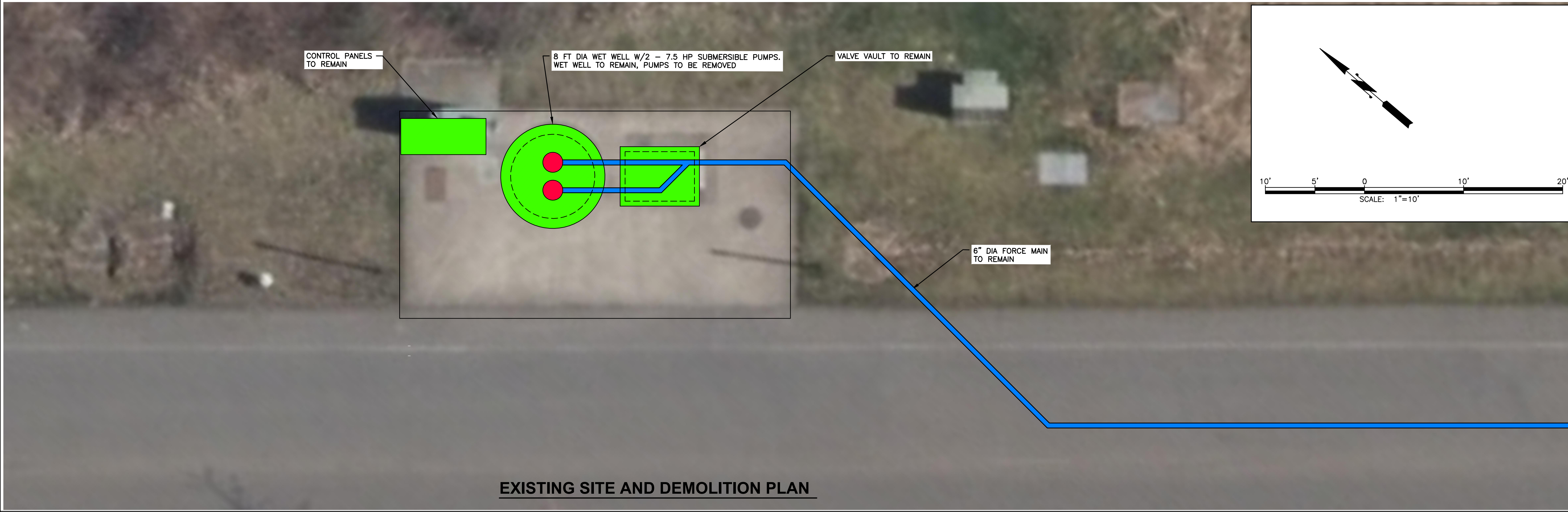
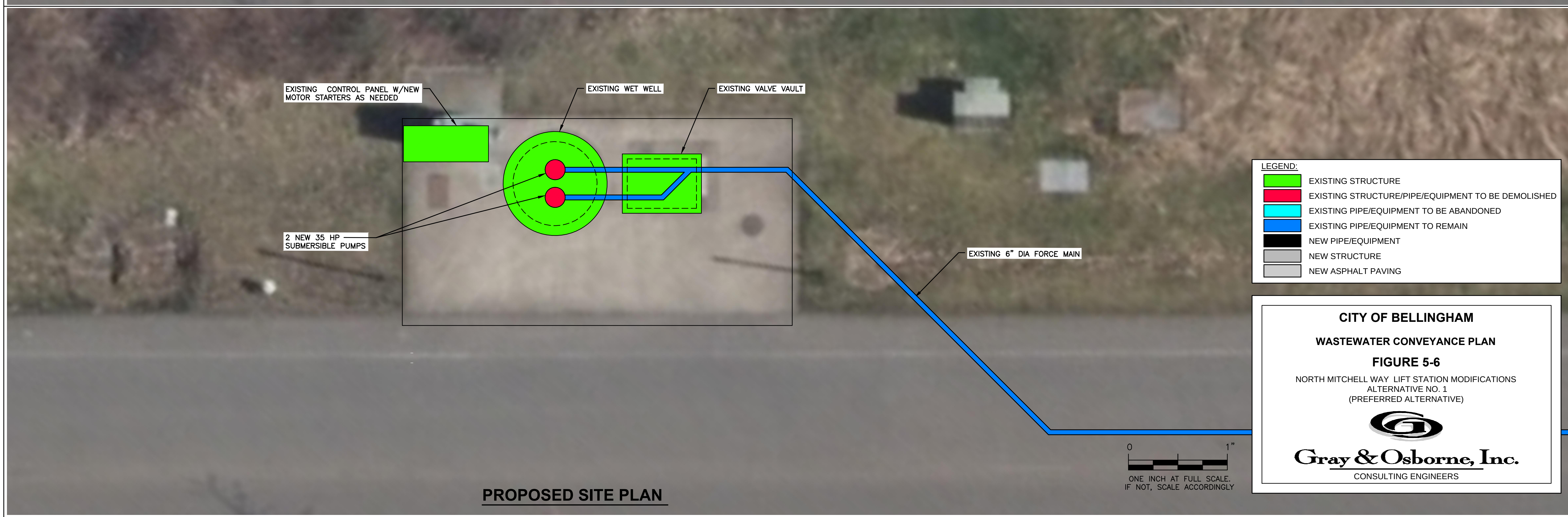


EXISTING SITE AND DEMOLITION PLAN



PROPOSED SITE PLAN

LEGEND:

- EXISTING STRUCTURE
- EXISTING STRUCTURE/PIPE/EQUIPMENT TO BE DEMOLISHED
- EXISTING PIPE/EQUIPMENT TO BE ABANDONED
- EXISTING PIPE/EQUIPMENT TO REMAIN
- NEW PIPE/EQUIPMENT
- NEW STRUCTURE
- NEW ASPHALT PAVING

0 1"

ONE INCH AT FULL SCALE. IF NOT, SCALE ACCORDINGLY

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN

FIGURE 5-6

NORTH MITCHELL WAY LIFT STATION MODIFICATIONS
ALTERNATIVE NO. 1
(PREFERRED ALTERNATIVE)

Gray & Osborne, Inc.
CONSULTING ENGINEERS

L:\Bellingham\15448.00 WW Conveyance Plan\Lift Station Amendment\FIGURES\N MITCHELL.dwg, 4/26/2016 11:10:35 AM, DWG To PDF.pc3

5.1.10 NORTHSHORE LIFT STATION

5.1.10.1 Existing Facilities

The existing Northshore Lift Station consists of two constant speed, 75-hp Smith & Loveless pumps mounted in a dry pit that pump through a 10-inch-diameter ductile iron force main. The lift station is located at the north end of Lake Whatcom situated within the right-of-way for Northshore Drive. The existing lift station covers roughly 875 square feet. An alley borders the site to the east. Limited information was available regarding this lift station at the time of this report. The available details of the lift station components are summarized in Table 5-9 below.

TABLE 5-9

Northshore Lift Station

Year Built	1980
Type	Wet Well-Dry Well
Pump Manufacturer	Smith & Loveless
Number of Pumps	2
Horsepower	75
Backup Power	125 kW generator
Wet Well Diameter	8 feet
Force Main	10-inch ductile iron
Force Main Length	1,510 linear feet

5.1.10.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this lift station is aging and is beginning to deteriorate. The existing dry pit has evidence of minor corrosion and the City has reported some leakage into the dry pit. The City has reported frequent cycling of the pumps and the motors and starters have been replaced in the past 5 years. The existing wet well, which is likely undersized, is in reusable condition. The standby generator is located in a below grade concrete vault that was installed within the past 10 years and is in good condition.

5.1.10.3 Capacity Analysis

Modeling completed for the *2009 Comprehensive Sewer Plan* projects flows to this station to decrease from 1,215 gallons per minute to 878 gallons per minute by 2026. It is unclear from the model and its documentation why flows are projected to decrease. For conservatism, evaluation of the existing lift station and sizing for proposed upgrades were completed using the current peak hour flow of 1,215 gallons per minute. If it is determined that the flows to this station will, in fact, decrease, sizing for proposed upgrades should be revised accordingly.

5.1.10.3.1 Pumps

As presented in Table 5-1 above, the design capacity of the lift station is 1,500 gpm and the tested capacity is approximately 1,580 gallons per minute. Pump curves for the pumps installed in this lift station were not available at the time of this report.

5.1.10.3.2 Force Main

At the current tested lift station capacity of 1,580 gpm, the velocity in the existing force main is approximately 6 feet per second with a total dynamic head of 108 feet. Flows to this station are not expected to increase above the current lift station capacity in the future.

5.1.10.3.3 Wet Well

The existing wet well is 8 feet in diameter. The operating depth for this station was not available at the time of this report. Assuming that the operating depth is 3 feet, this station has an operating volume of approximately 1,128 gallons. Based on the maximum estimated current peak hour flow of 1,215 gpm, the maximum starts per hour is approximately 16, which exceeds the maximum recommended starts of 12 per hour, which is particularly important given the large size of these pumps.

5.1.10.4 Electrical System

5.1.10.4.1 General

The existing utility feed is a 200A, 480 VAC, 3-phase service. The estimated load of the two pumps and 9kVA 208/120 VAC transformer is 162 kVA, 195A at 480V. The existing distribution equipment and control panels in the generator vault are in good condition. The PLC control panel has ample room for modification. In the Smith & Loveless dry well, all components are old and in poor physical condition. Most of the functionality for process control and power distribution has been relocated to the generator vault.

5.1.10.4.2 Generator Capacity

The 125kW generator is capable of providing 188 A and as such may not be capable of supporting the entire facility if the 208/120 VAC transformer is heavily loaded. The generator is in good physical condition. While fused at 200 A, the ATS is rated for up to 400 A.

5.1.10.4.3 Code Compliance Considerations

In the Smith & Loveless dry well, neither the motor starter panel nor the control panel have the clearances required by the NEC. The interior and exterior of the Smith & Loveless panel is corroded, posing undo risk to electrical components and safety of personnel.

5.1.10.5 Alternatives

Upgrade of the Northshore Lift Station is a high priority for the City. The current flows to this station exceed the capacity of the existing wet well causing frequent cycling of the pumps. Evidence of deterioration was observed in the dry pit and control panel and the station is likely beyond its useful life. As discussed under the "Current Condition" section above, modeling suggests that the flow to this station will decrease in the future, but the station evaluation was based on the current peak hour flow to be conservative. Figure 5-7, located at the end of this section, depicts the preferred alternative. Figures depicting all other alternatives are included in Appendix H.

5.1.10.5.1 Alternative No. 1 – Convert Lift Station to a Submersible Station and Add a Second Wet Well

Alternative No. 1 would convert the existing wet pit-dry pit station to a submersible station. Two 85-horsepower submersible pumps capable of conveying the current peak hour flow would be installed in the existing wet well and a second 8-foot diameter wet well would be constructed immediately east of the existing wet well to provide additional operating volume. A new valve vault would be installed and the discharge piping would be extended to the existing 10-inch diameter force main. The existing generator and below grade vault would be retained to provide emergency backup power. Bypass pumping would be required during a majority of the construction.

Electrical Aspects of Alternative No. 1

The incoming utility service would need to be upsized as well as all electrical distribution equipment except the ATS which would only need to have larger fuses installed. The generator will only run one pump and ancillary loads at the increased size.

5.1.10.5.2 Alternative No. 2 – Replace Lift Station

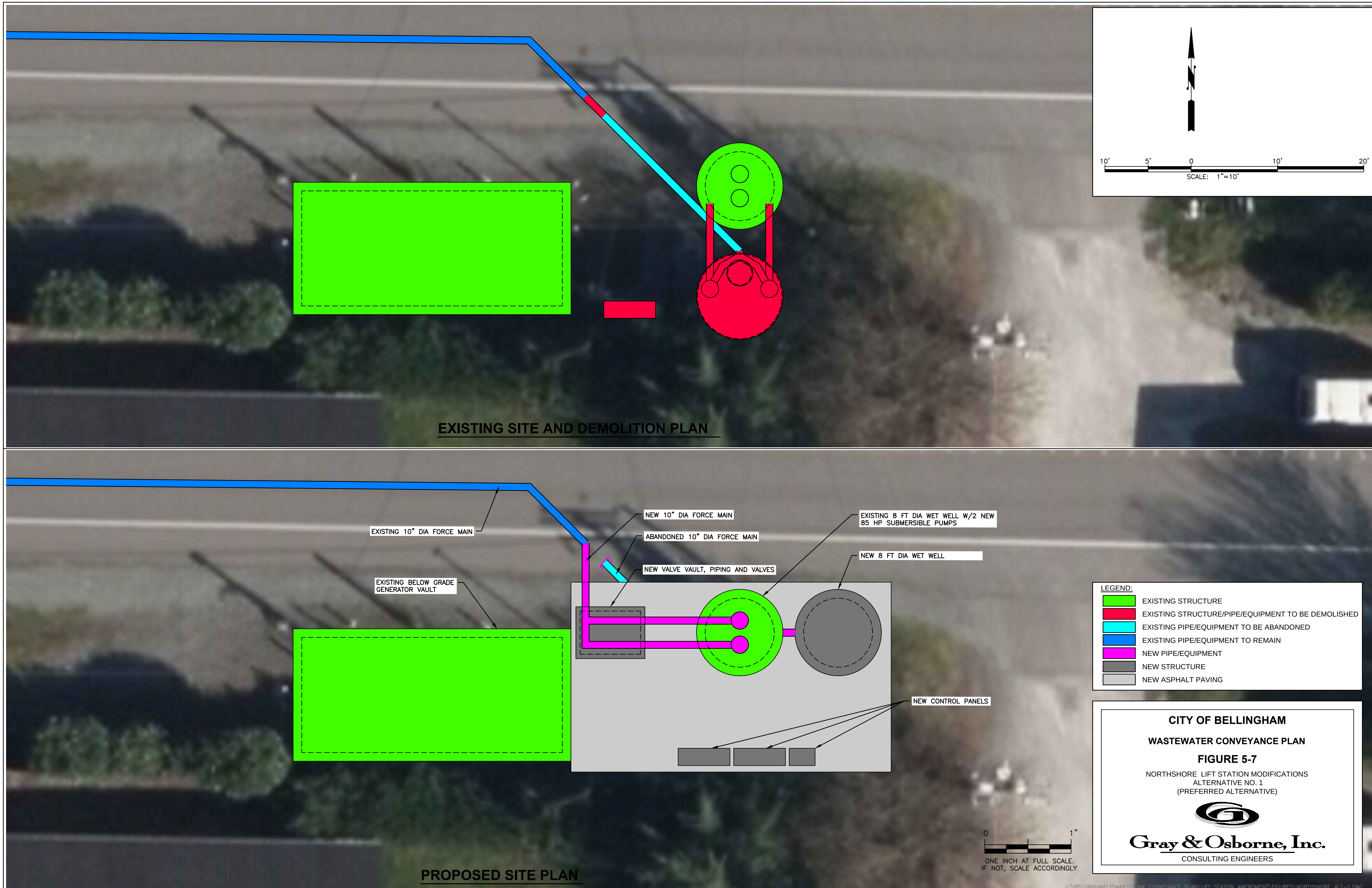
Alternative No. 2 would replace the existing lift station with a new lift station including pumps, wet well, controls and force main stub, adjacent to the north side of the existing lift station site. The station would include a new 10-foot diameter wet well and 10-inch force main that would connect to the existing 10-inch force main. New 85-hp pumps, control panels, and motor starters would be provided with sufficient capacity to convey the projected current and future design flow. Due to the very constrained site it would be necessary to construct the new wet well within the footprint of the existing dry well. As a result, bypass pumping would be required throughout the majority of construction.

Electrical Aspects of Alternative No. 2

The incoming utility service would need to be upsized as well as all electrical distribution equipment except the ATS which would only need to have larger fuses installed. The generator will only run one pump and ancillary loads at the increased size.

5.1.10.6 Preferred Alternative

The Northshore Lift Station is over 30 years old and the projected design flows exceed the capacity of all components. Therefore, replacement of the station is recommended. Both alternatives presented above would construct a new station capable of conveying the design flow. However, the station is located on a very constrained site and replacement of the wet well, as proposed for Alternative No. 2, would require bypass pumping throughout construction. Based on visual inspection, the existing wet well appears to be in good condition and could be reused. By constructing a second wet well, as proposed for Alternative No. 1, the existing station could be kept in service during a majority of construction of the new station. Therefore, Alternative No. 1, which is shown on Figure 5-7, is the preferred alternative. However, a thorough evaluation of the condition of the existing wet well should be completed at the start of design for this project and recoating of the interior of the structure would be recommended. In addition, due to the close proximity of the lift station to Lake Whatcom, there may be permitting challenges associated with relocating the lift station. A thorough review of permitting requirements should be reviewed early on in the project.


EXISTING SITE AND DEMOLITION PLAN
10' 5' 0 10' 20'
SCALE: 1"=10'
EXISTING 10" DIA FORCE MAIN
NEW 10" DIA FORCE MAIN
ABANDONED 10" DIA FORCE MAIN
NEW VALVE VAULT, PIPING AND VALVES
EXISTING 8 FT DIA WET WELL W/2 NEW 85 HP SUBMERSIBLE PUMPS
NEW 8 FT DIA WET WELL
EXISTING BELOW GRADE GENERATOR VAULT
NEW CONTROL PANELS
LEGEND:
EXISTING STRUCTURE
EXISTING STRUCTURE/PIPE/EQUIPMENT TO BE DEMOLISHED
EXISTING PIPE/EQUIPMENT TO BE ABANDONED
EXISTING PIPE/EQUIPMENT TO REMAIN
NEW PIPE/EQUIPMENT
NEW STRUCTURE
NEW ASPHALT PAVING
CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
FIGURE 5-7
NORTHSHORE LIFT STATION MODIFICATIONS
ALTERNATIVE NO. 1
(PREFERRED ALTERNATIVE)
Gray & Osborne, Inc.
CONSULTING ENGINEERS
0 1"
ONE INCH AT FULL SCALE.
IF NOT, SCALE ACCORDINGLY
PROPOSED SITE PLAN
L:\BELLINGHAM\15448.00 WW CONVEYANCE PLAN\LIFT STATION AMENDMENT\FIGURES\NORTHSHORE_ALT-1.DWG

5.1.11 EDGEMOOR LIFT STATION

5.1.11.1 Existing Facilities

The existing Edgemoor Lift Station consists of two constant speed, 15-horsepower Smith & Loveless pumps mounted in a 7-foot-diameter steel dry well with an adjacent 4-foot-diameter concrete wet well. This station pumps through a 4-inch-diameter ductile iron force main. The lift station is on privately owned residential lots within a 20-foot-wide easement located west of Bayside Road. The existing lift station covers roughly 150 square feet. The land surrounding the station is all residential with a 20-foot-wide easement extending to the north and south of the station site. The details of the lift station components are summarized in Table 5-10 below.

TABLE 5-10

Edgemoor Lift Station

Year Built	1965
Type	Wet Pit-Dry Pit
Pump Manufacturer	Smith & Loveless
Number of Pumps	2
Horsepower	15
Backup Power	Off-site generator shared with Willows LS
Wet Well Diameter	4 feet
Force Main	4-inch ductile iron
Force Main Length	1,600 linear feet

5.1.11.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this lift station is aging and the City desires to eliminate this station if possible. The station is located adjacent to a steep slope that is reportedly regulated as a critical area, making upgrades to this station very challenging. In addition, the station is located adjacent to a residence and does not have vehicle access. The wet well could not be inspected at the time of the site visit; however, given its age, recoating or replacement of this structure is likely needed if the station is retained. The dry well and associated equipment appear to be in good condition with no evidence of significant deterioration. The dry well could likely be reused if the station is upgraded.

5.1.11.3 Capacity Analysis

5.1.11.3.1 Pumps

As presented in Table 5-1 above, the design capacity for the existing lift station is approximately 150 gallons per minute. Based on a comparison of the system curve for this lift station and associated force main with the manufacturer's pump curves (see Appendix F), the pumps should be capable of pumping approximately 145 gpm in this system. The tested capacity of this station is 147 gpm.

The projected future peak hour flow is estimated to be 268 gallons per minute, which exceeds the capacity of the existing pumps.

5.1.11.3.2 Force Main

At the current lift station capacity of 150 gpm, the velocity in the existing force main is slightly less than 4 feet per second with a total dynamic head of 107 feet. At the projected future peak hour flow of 268 gpm, the velocity in the existing force main would be approximately 7 feet per second with a total dynamic head of approximately 176 feet.

5.1.11.3.3 Wet Well

The existing 4-foot-diameter wet well has an operating volume of approximately 188 gallons. At the current design peak hour flow of 150 gpm, the maximum starts per hour is approximately 12. At the projected future flow, the maximum starts per hour would be approximately 22, which exceeds the maximum recommended starts per hour. Based on record drawings the invert elevation of the lowest influent pipe to the wet well is only 1.5 feet above the current pump on elevation, which is not sufficient to increase the operating volume of the wet well to reduce the pump starts to below 12 per hour.

5.1.11.4 Electrical System

5.1.11.4.1 General

The existing utility feed is a 100A, 480 VAC, 3-phase service. The estimated load of the two pumps and 3kVA transformer is 38 kVA, 46A at 480V. While the electrical feed is appropriately sized, the Smith & Loveless panel is in poor condition. There is no telemetry panel.

5.1.11.4.2 Emergency Supply

The system shares a generator with the Willows Lift Station. The generator and Willows Lift Station were not evaluated.

5.1.11.4.3 Code Compliance Considerations

The interior and exterior of the Smith & Loveless control panel is corroded, posing undo risk to electrical components and safety of personnel. The Smith & Loveless panel lacks the clearance required by the NEC. The outside of the panels, at grade, show corrosion and signs of aging.

5.1.11.5 Alternatives

Based on conversations with City staff, due to accessibility, capacity and age concerns, the City would prefer to eliminate this lift station if possible and direct flows to other portions of the collection system. Therefore, the alternatives presented include alternatives that would redirect flows and eliminate this station. Alternatives for upgrading the existing station were also considered and summarized below; however, due to its proximity to critical areas, permitting for an upgraded station may be difficult. Figure 5-8, located at the end of this section, depicts the preferred alternative. Figures depicting all other alternatives are included in Appendix H.

5.1.11.5.1 Alternative No. 1 – Decommission Existing Lift Station and Install Individual Grinder Pumps

Alternative No. 1 would install individual grinder pumps to serve each of the 17 homes that are served by the Edgemoor Lift Station. The grinder pumps would pump through individual force mains and discharge to the existing 8-inch-diameter gravity sewer within Bayside Road, which conveys flow to the Willow Lift Station. The Edgemoor Lift Station would be demolished below grade and the force main and gravity sewers plugged and abandoned in place. To implement this alternative, it would be necessary to receive approval from all 17 property owners to convert their homes to grinder pump systems.

Electrical Aspects of Alternative No. 1

Grinder pumps systems are typically small and powered directly from the property owners' home electrical service.

5.1.11.5.2 Alternative No. 2 – Decommission Existing Lift Station and Install New Gravity Sewer

Alternative No. 2 would install approximately 600 feet of 8-inch-diameter gravity sewer pipe between the Edgemoor Lift Station and the Willow Lift Station. The Edgemoor Lift Station would be decommissioned and all of the flows from the Edgemoor Lift Station service area would be conveyed by gravity to the Willow Lift Station. The new gravity sewer would run along the toe of the slope near Bellingham Bay at the minimum recommended slope of 0.5 percent. The new sewer would be up to 20 feet deep and it would be necessary to install a new, deeper wet well for the Willow Lift Station to accommodate this new sewer. Based on conversations with City staff, the slopes that are adjacent to this proposed alignment are very steep and are considered critical areas. Due to this classification and the close proximity to Bellingham Bay (within 200 feet), obtaining permits to construct this alternative could be challenging.

5.1.11.5.3 Alternative No. 3 – Replace Pumps and Force Main and Construct a Second Wet Well

Alternative No. 3 would retain the wet pit-dry pit configuration and would replace the existing pumps with new 25-hp pumps capable of conveying the projected design flow. The existing force main would be replaced with a new 8-inch-diameter force main. A second 4-foot-diameter wet well would be constructed to provide additional operating capacity. New controls, motor starters, discharge piping, and valves sized for the projected design flow would also be installed. Although the station is over 40 years old, the existing structures do not show signs of significant deterioration. However, to extend their useful life, the interior of both structures would be recoated as part of the upgrade. Bypass pumping during a majority of the construction would be required.

Electrical Aspects of Alternative No. 3

All existing electrical will be demolished and replaced. Utility and backup services would require an evaluation of the Willows Lift Station as well.

5.1.11.6 Preferred Alternative

The Edgemoor Lift Station is over 40 years old and the projected design flows will exceed the current capacity of the wet well and pumps. Due to the site constraints and nearby critical areas, any work to upgrade this station outside of the existing structures may present significant permitting challenges. Prior to final selection of the preferred alternative, the City should closely coordinate with the various permitting agencies to verify that the preferred alternative can be permitted and constructed.

Because the City would prefer to eliminate this station, Alternative No. 3, which would replace the existing station, has been discarded from consideration. Although Alternative No. 1 would likely be the easiest from a permitting standpoint, it would require all 17 affected property owners to agree to having grinder pumps and new piping installed on their property which can be a very difficult process. Therefore, assuming that the project can be permitted, Alternative No. 2, shown on Figure 5-8, which would install a new gravity sewer to convey all of the flows from the Edgemoor Lift Station to the Willow Lift Station, is the preferred alternative. However, this recommendation is based on limited topographical information and, therefore, careful evaluation during preliminary design should be completed to verify that this alternative is feasible.

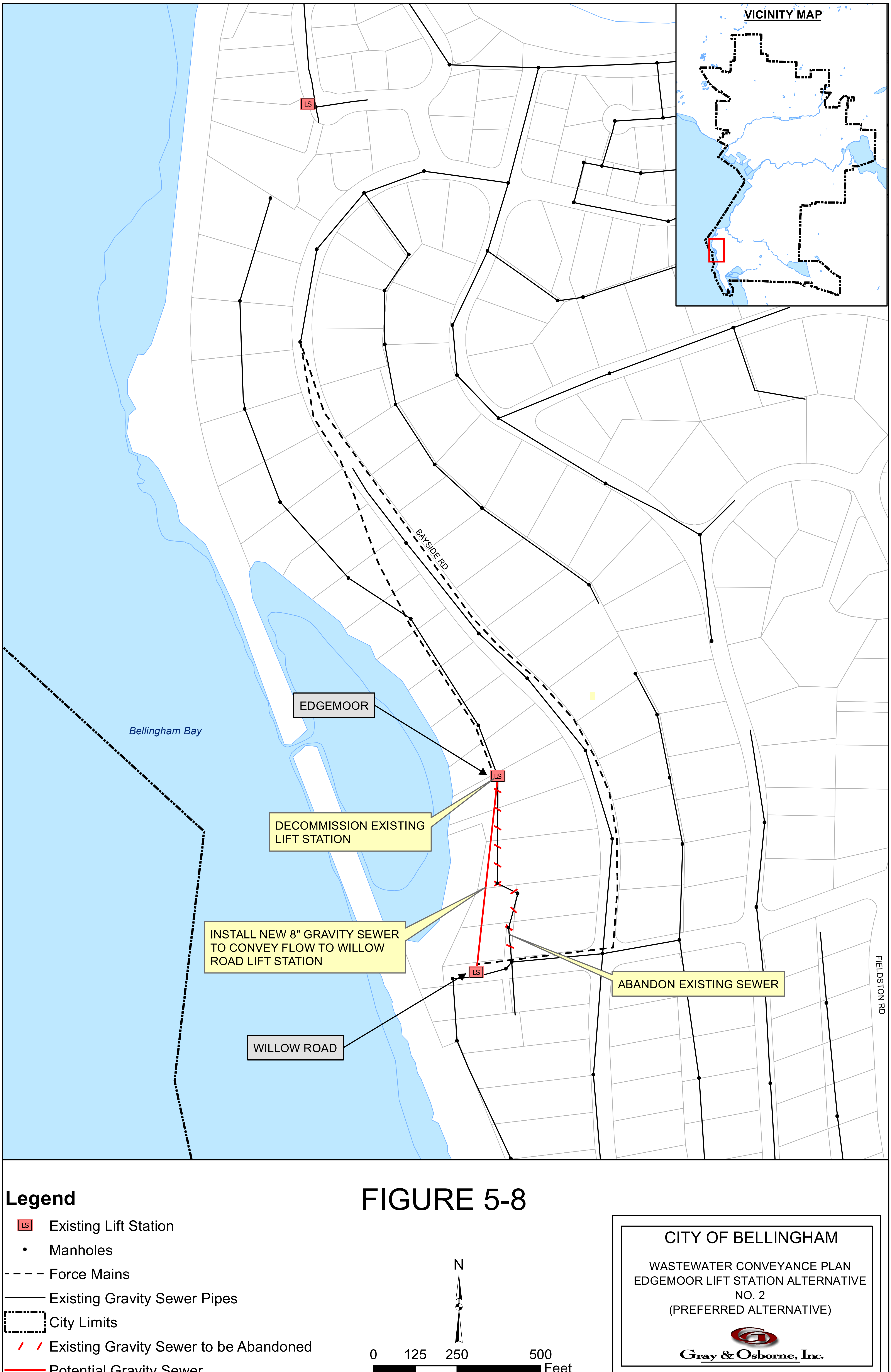


FIGURE 5-8

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
EDGEMOOR LIFT STATION ALTERNATIVE NO. 2
(PREFERRED ALTERNATIVE)

Gray & Osborne, Inc.

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5.1.12 PINE STREET LIFT STATION

5.1.12.1 Existing Facilities

The existing Pine Lift Station was originally equipped with two vacuum-primed pumps mounted on top of a 4-foot-diameter concrete wet well. The station has since been converted to a submersible lift station. The existing station consists of two constant speed, 3-hp pumps mounted in the original 4-foot-diameter wet well. This station does not have a force main and the pumps discharge directly to a gravity sewer that enters the wet well. The lift station is situated within the right-of-way for Pine Street. The land adjacent to the north side of the lift station site is owned by the City of Bellingham and is currently undeveloped. The details of the lift station components are summarized in Table 5-11 below.

TABLE 5-11

Pine Street Lift Station

Year Built	1969
Type	Submersible
Pump Manufacturer	Flygt
Number of Pumps	2
Horsepower	3
Backup Power	None
Wet Well Diameter	4 feet
Force Main	None

5.1.12.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this lift station is in serviceable condition but may not be needed with minor collection system modifications. This station does not have an associated force main and pumps directly into a gravity sewer that connects to the wet well. It appears that the total lift in this station is approximately three feet. Per City staff, this station is needed to serve a basement of a building located immediately south of the station. The existing wet well has no evidence of deterioration or spalling and is in reusable condition.

5.1.12.3 Capacity Analysis

5.1.12.3.1 <u>Pumps</u>

As presented in Table 5-1 above, the design capacity for the existing lift station is not available and no drawdown testing has been done. Based on a comparison of the system curve for this lift station and associated force main with the manufacturer's pump curves (see Appendix F), the pumps are running off the end of their curves due to the extremely low head on the pumps. These pumps are estimated to be pumping at approximately 300 gallons per minute.

The current flow to the Pine Lift Station is estimated to be approximately 5 gallons per minute, which is significantly less than the tested capacity of the lift station. This station is reported to have very low run times. The projected future peak hour flow is estimated to be 153 gallons per minute, which is within the capacity of the existing pumps.

5.1.12.3.2 Force Main

This station does not have a force main. The pumps discharge directly to a gravity sewer that connects to the wet well. This station has approximately 3 feet of total lift.

5.1.12.3.3 Wet Well

The existing 4-foot-diameter wet well has an operating volume of approximately 291 gallons. At the projected future flow, the maximum starts per hour would be approximately eight, which is within the recommended range.

5.1.12.4 Electrical System

5.1.12.4.1 General

The existing utility feed is a 60A, 480 VAC, 3-phase service. The estimated load of the two pumps and control transformer is 38 kVA, 46A at 480V. While the electrical feed is appropriately sized, the Flygt control panel and all other electrical panels are in poor condition. The radio telemetry is unused and the fiber optic telemetry installation is incomplete.

5.1.12.4.2 Code Compliance Considerations

The exterior of all panels are corroded, posing undo risk to electrical components and safety of personnel. The interior of the Flygt panel shows scorch marks on the backplane indicating equipment failure at some point.

5.1.12.5 Alternatives

The Pine Lift Station is a very shallow lift station with no force main and very little static lift. Based on discussions with City staff, this station is only needed to prevent sewage from backing up into one adjacent building with a basement. This lift station could potentially be eliminated if this connection is addressed. Three alternatives were considered for modifying this station. Figures depicting these alternatives (excluding Alternative No. 3) are included in Appendix H.

5.1.12.5.1 Alternative No. 1 – Decommission Existing Lift Station and Install One Grinder Pump

Alternative No. 1 would remove the existing pumps and controls from the existing wet well and the wet well would be converted to a gravity manhole. An individual grinder pump would be installed to convey sewage from the basement of the neighboring building. Because the outlet gravity sewer from the wet well is slightly higher than the inlet pipes, the inlet sewers would be surcharged at all times. This condition would likely require additional maintenance to remove solids that may settle out in the pipes during low flow periods.

Electrical Aspects of Alternative No. 1

Grinder pumps systems are typically small and would be powered from the buildings they served.

5.1.12.5.2 Alternative No. 2 – Decommission Existing Lift Station and Install New Gravity Sewers

Alternative No. 2 would decommission the existing lift station and construct a new gravity sewer to convey the flow by gravity to the existing Oak Street Lift Station. In 2007, HDR Engineering, Inc. prepared preliminary plans to construct the improvements proposed as part of this alternative. These improvements, which have not been constructed, included installation of approximately 600 lineal feet of 12-inch-diameter gravity sewer pipe from an existing manhole located east of the existing station through the existing station and to the Oak Street Lift Station. The grades for the new sewer would be selected to continue to serve all existing connections by gravity.

5.1.12.5.3 Alternative No. 3 – Status Quo

Alternative No. 3 would leave the existing lift station as is. Per City staff, this station has not been a significant maintenance problem and could continue to operate with little cost or attention. Based on hydraulic calculations, the existing pumps have sufficient capacity to convey the projected design flow to this station.

5.1.12.6 Preferred Alternative

Alternative No. 1 would result in the sewers upstream of the station to be surcharged at all times, which would present a maintenance concern. Therefore, this alternative is not recommended. The existing lift station is quite old but appears to be in serviceable condition. Per City staff, this station has not presented significant maintenance concerns and the station is capable of pumping the current and projected peak hour flow. Although it is possible to eliminate this station and convey the flow by gravity to the Oak Lift Station, there would be significant cost to install the new sewers proposed for Alternative

No. 2. Therefore, Alternative No. 3, which would retain the existing station, is the preferred alternative.

5.1.13 ROEDER LIFT STATION

5.1.13.1 Existing Facilities

The existing Roeder Lift Station consists of a 24-foot-diameter concrete caisson that is divided into a wet pit and a dry pit by a concrete wall. A CMU building that houses the electrical equipment and standby generator sits on top of the dry pit. The dry pit is equipped with two 75-hp Yeoman pumps and one 50-hp pump that pump through an 18-inch-diameter ductile iron force main. The two 75-hp pumps are equipped with variable frequency drives and the 50-hp pump is constant speed. The existing lift station site covers approximately 1,000 square feet. The details of the lift station components are summarized in Table 5-12 below.

TABLE 5-12

Roeder Lift Station

Year Built	1973
Type	Submersible
Pump Manufacturer	Yeoman
Number of Pumps	3
Horsepower	2-75 hp, 1-50 hp
Backup Power	200 kW generator
Wet Well Diameter	See below
Force Main	18-inch ductile iron
Force Main Length	1,045 linear feet

5.1.13.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this lift station is in generally good condition. However, the City is concerned with the small size of the wet well. The wet well structure does not appear to be deteriorated; however, the piping, valves and slide gate in the wet well are severely corroded and inoperable. The dry well and building are in good condition and could remain in service in the future. Per City staff, all three pumps are required to convey the peak flows during storm events. The station is reportedly located on private property owned by the railroad.

5.1.13.3 Capacity Analysis

5.1.13.3.1 <u>Pumps</u>

Per record drawings, the original lift station was constructed in 1973 with two 75-hp pumps and space for a third pump. The 50-hp pumps was installed as part of an upgrade project in 1983.

As presented in Table 5-1 above, the design flow for the existing lift station is approximately 4,000 gallons per minute. Based on a comparison of the system curve for this lift station and associated force main with the manufacturer's pump curves for the two 75-hp pumps (see Appendix F), one 75-horsepower pump should be capable of pumping approximately 5,350 gpm in this system and two 75-hp pumps running in parallel should be able to convey approximately 7,700 gpm. A pump curve for the 50-hp pump was not available at the time of this report.

Per the 2013 Lift Station Evaluation, current flow data for the Roeder Lift Station was not available at the time of the report. However, based on pump run-times, two of the three pumps have operated for an entire 24-hour period on one occasion. This could suggest that the influent flow is nearing the lift station capacity or it could be due to ragging, which is a significant concern at this station. The projected future peak hour flow is estimated to increase to 12,507 gallons per minute.

5.1.13.3.2 Force Main

At the current design lift station capacity of 4,000 gpm, the velocity in the existing 18-inch force main is approximately 5 feet per second with a total dynamic head of 23 feet. At the projected future peak hour flow of 12,500 gpm, the velocity in the existing force main would exceed 15 feet per second with a total dynamic head of 82 feet.

5.1.13.3.3 Wet Well

The existing wet well has an estimated operating volume of approximately 1,136 gallons. Two of the three pumps in this station are equipped with variable frequency drives and the control scheme is programmed to maintain level in this wet well. Therefore, the wet well is significantly smaller than typically selected for a station with fill and draw operation. The City has expressed concern over the small size of this wet well and the limited response time in the event that there is a problem with this station. Additionally, the VFD-controlled pumps regularly run at low speed in an effort to match the incoming flow, which causes frequent clogging of the pumps.

5.1.13.4 Electrical System

5.1.13.4.1 General

The existing utility feed is a 300 A, 480 VAC, 3-phase service. The estimated load of the three pumps and 10kVA 240/120 VAC transformer controls is 219 kVA, 264 A at 480V. The existing distribution equipment is in an MCC and is in good condition. The exterior of the control panel is a bit worn but the interior is in good condition and there is some room for modification/ expansion.

5.1.13.4.2 Emergency Supply

The system has an onsite 200 kW generator capable of 300 A of 480 VAC, 3-phase power. This is almost exactly sized to the requirement of the system at 100 percent loading.

5.1.13.5 Alternatives

Upgrade of the Roeder Lift Station is the City's top priority. This station has multiple variable speed pumps designed to match the incoming flow and maintain a constant level in the wet well. As a result, the wet well was originally designed with a much smaller capacity than would be required for a lift station with typical fill and draw operation. However, this small wet well provides very little storage capacity in the event that any component of the station fails. Per City staff, during high flow events, all three pumps are needed to convey the design flow. As a result, this station does not have the redundancy required for sewage lift stations. The existing building, dry well and wet well are in very good condition.

Four alternatives have been developed for upgrade of this station and a preferred alternative has been identified as discussed further below. However, given the size and complexity of this station, a preliminary engineering report evaluating these, and potentially other, alternatives should be completed, including a thorough hydraulic analysis of each alternative. Figure 5-9, located at the end of this section, depicts the preferred alternative. Figures depicting all other alternatives are included in Appendix H.

5.1.13.5.1 Alternative No. 1 – Replace Pumps and Retain Existing Wet Well

Alternative No. 1 would retain the existing lift station building, dry well and wet well and replace the three existing pumps with pumps capable of conveying the future projected design flow. The existing force main would be replaced with a new 30-inch force main. The controls, motor starters and generator would also be replaced as part of this project. The station would continue to operate to match the incoming flow. This alternative would not address the City's concerns regarding storage capacity. Two sub-alternatives include the following:

5.1.13.5.2 Alternative No. 1A – Install Three New Pumps

Alternative No. 1A would replace the three existing pumps with three new pumps that would be sized to pump the design flow with two pumps in service. This alternative would reuse the existing suction piping and would require minimal modification to the structure. However, this alternative likely would not address the ragging issues that have been reported at this station. Alternative pumps, such as screw centrifugal pumps, that are less susceptible to ragging could be installed to mitigate the ragging issue.

Electrical Aspects of Alternative No. 1A

Pump sizes would need to be known to evaluate the full impact. The NEC allows actual utility bill demand to be used when modifying an existing station. Under this method it is possible that the existing incoming electrical feed, transfer switch, and generator would be sufficient.

5.1.13.5.3 Alternative No. 1B – Install Four New Pumps

Based on a preliminary review of the dry pit, it appears to be large enough to accommodate up to four pumps that are similar size to the existing pumps. Alternative No. 1B would demolish the existing pumps and suction piping and install four new pumps sized to convey the future projected design flow with three pumps in operation. Based on preliminary analysis, three 60 horsepower pumps would be capable of conveying the peak design flow. All four pumps would be equipped with VFDs and the pumps would be operated to match the incoming flow. Operation of the pumps would be controlled to ensure that the minimum velocity in the force main is always above 2 feet per second.

By installing four pumps, each pump would individually have lower capacity and the station would be capable of meeting the low flow condition at a higher pump speed. Based on pump curves for a preliminary pump selection (see Appendix F), one 60-horsepower pump running at or near full speed, would be capable of providing the minimum velocity of 2 feet per second (5,000 gallons per minute). Operating the pumps at higher speed will reduce ragging associated with significant VFD turndown. The station would be programmed to rotate lead pump operation and, with an additional pump in the rotation, the starting frequency for each pump would be reduced.

Electrical aspects of Alternative No. 1B

A four pump solution will require complete electrical upgrade and replacement.

5.1.13.5.4 Alternative No. 2 – Replace Pumps and Increase Operating Volume to allow Fill-Draw Operation

Similar to Alternative No. 1, Alternative No. 2 would retain the existing lift station building, dry well and wet well and replace the three existing pumps with three 50-hp pumps capable of conveying the design flow. The existing force main would be replaced with a new 30-inch force main. The controls, motor starters and generator would also be replaced as part of this project. However, Alternative No. 2 would provide additional operating volume for the station. For the purposes of sizing and cost estimating, the operating volume would be sized for fill/draw operation with a total of 18 starts per hour, alternating between all three pumps.

5.1.13.5.5 Alternative No. 2A – Construct Second Wet Well

For Alternative No. 2A, a second wet well would be constructed adjacent to the existing wet well to provide additional operating volume. At the current operating level, to provide sufficient volume to support fill and draw operation, the wet well would need to be approximately 22 feet in diameter and approximately 22 feet deep.

5.1.13.5.6 Alternative No. 2B – Provide Additional Operating Volume with Pipe Manifold System

For Alternative No. 2B, a series of interconnected 36-inch-diameter pipes would be installed adjacent to the existing wet well to provide additional operating volume. Based on preliminary calculations, sufficient volume could be provided by installing three 75-foot-long, 36-inch-diameter pipes connected by a manifold. Each pipe would be sloped at approximately 0.5 percent and the downstream invert elevation would be set at the "Pumps Off" elevation to allow the pipe system to drain out at the end of each pumping cycle. A vault would be constructed at the upstream end of the pipe system to provide access. Based on current operating levels, it is assumed that the pipes will be approximately half full at the "Second Pump On" elevation.

Electrical Aspects of Alternatives No. 2A and No. 2B

The proposed plan for three 50-hp pumps would result in a net decrease in the electrical loads. Motor starters would be the only thing that would have to be replaced. Replacement of other electrical equipment would be at the Owner's request.

5.1.13.5.7 Alternative No. 3 – Convert Station to a Submersible Lift Station

Alternative No. 3 would demolish all of the existing pumps, piping and electrical equipment. The wet well and dry well would then be hydraulically connected to create a single large wet well. The station would be equipped with three or four submersible pumps capable of conveying the design flow. Preliminary review of available pump curves suggests that 60-hp pumps would be required if three pumps are installed and 45-hp pumps would be required if four pumps are installed. Due to electrical classification requirements, the motor starters and controls would have to be relocated from the existing building, which is directly above the existing dry well that will be converted to a wet well. The building would be demolished and a new building would be constructed adjacent to the converted wet well to house all of the new electrical and control equipment. Bypass pumping from the manhole immediately upstream of the station would be required throughout construction.

5.1.13.5.8 Electrical Aspects of Alternative No. 3

The incoming utility needs to be verified but it appears that the system could accept either the three 60-hp or the four 45-hp configurations. The generator and transfer switch could support this pump configuration but at the very least the transfer switch should be replaced due to age.

5.1.13.5.9 Alternative No. 4 – Construct a New Lift Station on an Adjacent Site

Alternative No. 4 would demolish the existing lift station and construct a new station on an adjacent site. Two sub-alternatives include:

5.1.13.5.10 Alternative No. 4A – Construct a New Submersible Lift Station

For Alternative No. 4A, three or four submersible pumps would be installed in a new 16-foot-diameter, 23-foot-deep wet well. The pumps would be similar to the pumps proposed under Alternative No. 3 (three 60 hp). A new building that would house the electrical equipment and standby generator would be constructed near the new wet well or the existing building could be reused. However, additional bypass pumping would be required if the existing building is reused.

5.1.13.5.11 Alternative No. 4B – Construct a New Wet Pit-Dry Pit Lift Station

For Alternative No. 4B, three or four dry-mounted pumps would be installed in a new dry pit sized to accommodate the new pumps. A new 23-foot wet well with sufficient capacity for fill and draw operation would be installed. The two pits could be part of the same structure or two separate structures. The pumps would be similar to the pumps proposed under Alternative No. 1. A new building that would house the electrical equipment and standby generator would be constructed above the new dry well.

5.1.13.6 Preferred Alternative

As discussed above, this is a complex station and numerous alternatives are available for upgrading this station. Therefore, a preliminary engineering analysis dedicated to this station should be completed prior to final selection of the preferred alternative.

Based on past experience, alternatives that would modify the existing structures (Alternatives No. 1B and No. 3) would require extensive bypass pumping of substantial flows, which would result in significant additional project costs. In addition, modifying existing structures frequently exposes latent defects that would need to be remedied during construction through change orders, presenting a significant risk of cost overruns. Alternative No. 1A would increase the capacity of the lift station to be able to convey the projected design flow. However, it would not address any of the City's concerns with the undersized wet well and the ragging issues associated with substantial VFD turn down. Alternative No. 4 would likely address all of the capacity and operational concerns associated with this station but would be the most expensive alternative. Alternative No. 2 could be constructed while maintaining the station in service during a majority of construction and retain the existing structure, which is in good condition. It would increase the wet well capacity so that the pumps could be operated at a higher minimum speed, operating as a fill-and-draw station during low flow periods. This would address the issues with pump ragging during low flow periods without increasing the number of the pumps in the station. Because Alternative No. 2B, which would construct a pipe manifold system to increase storage volume, could likely be constructed at a lower cost, it is the preferred alternative. The preferred alternative is depicted on Figure 5-9.

This station is located in tide flats and it is possible that the soils in the region may require additional geotechnical considerations to support the proposed improvements and reuse of excavated soils will likely not be possible. A thorough geotechnical investigation should be completed early on in this project to determine if any additional improvements may be needed. In addition, given the industrial nature of the land surrounding the lift station, there is potential for contaminated soils to be present within the project area. Based on a review of the City's GIS data, the proposed improvements do not appear to cross any known Model Toxics Control Act listed sites. However, thorough environmental review should be included in the project design scope. The estimate for the Roeder Lift Station in Appendix G does not include any geotechnical improvements that may be required or removal of any contaminated soils that may be present within the project area.

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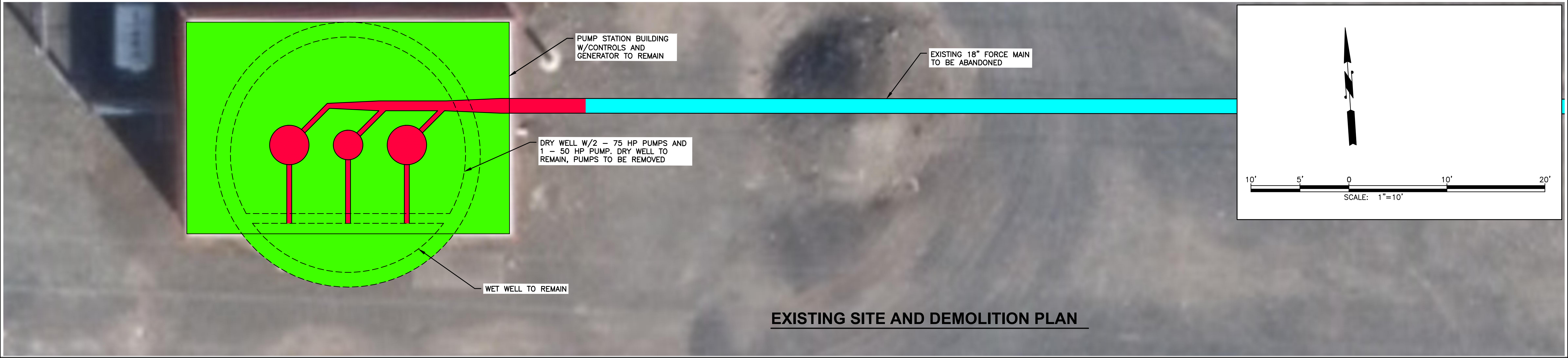
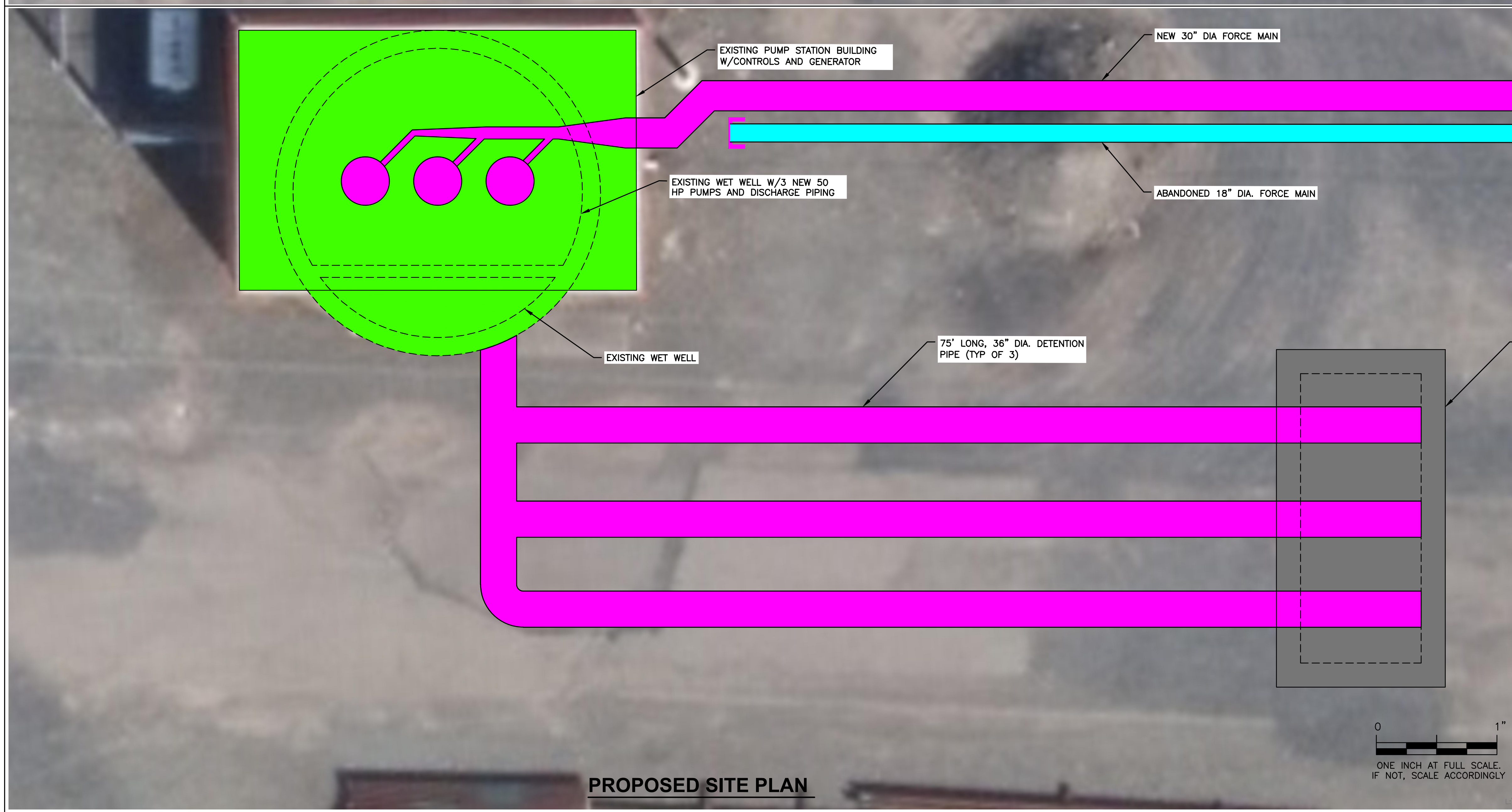


EXISTING SITE AND DEMOLITION PLAN



PROPOSED SITE PLAN

LEGEND:

- EXISTING STRUCTURE
- EXISTING STRUCTURE/PIPE/EQUIPMENT TO BE DEMOLISHED
- EXISTING PIPE/EQUIPMENT TO BE ABANDONED
- EXISTING PIPE/EQUIPMENT TO REMAIN
- NEW PIPE/EQUIPMENT
- NEW STRUCTURE
- NEW ASPHALT PAVING

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN

FIGURE 5-9

ROEDER LIFT STATION MODIFICATIONS
ALTERNATIVE NO. 2B
(PREFERRED ALTERNATIVE)

Gray & Osborne, Inc.
CONSULTING ENGINEERS

5.1.14 WEST BAKERVIEW LIFT STATION

5.1.14.1 Existing Facilities

The existing West Bakerview Lift Station consists of two constant speed, 10-horsepower submersible pumps mounted in an 8-foot diameter concrete wet well that pump through a 4-inch diameter PVC force main. The lift station is located within an extension of the right-of-way for West Bakerview Road. The limits of the right-of-way surrounding the lift station site are coincident with the edge of the lift station on three sides with privately owned residential lots beyond. West Bakerview Road borders the site to the south. The existing lift station covers roughly 1,000 square feet. The details of the lift station components are summarized in Table 5-13 below.

TABLE 5-13

West Bakerview Lift Station

Year Built	1998
Type	Submersible
Pump Manufacturer	Morris
Number of Pumps	2
Horsepower	10
Backup Power	30 A Outlet for Portable Generator
Wet Well Diameter	8 feet
Force Main	4-inch PVC
Force Main Length	830 linear feet

5.1.14.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this lift station is in generally in good condition and is reportedly reliable, though grease accumulation is a major issue at this station. Grease accumulation causes false level readings in the wet well, causing the pumps to run dry on occasion. Pumps in this station have failed in the past as a result of running dry. In an effort to mitigate the grease accumulation, the City has installed a "Little John Digester" in the wet well with limited success.

The wet well has no visible evidence of deterioration or spalling and the piping in the wet well has limited corrosion. The pumps and valves are operational and the existing structures are in reusable condition. Minor seepage was observed at pipe penetrations in the valve vault.

5.1.14.3 Capacity Analysis

5.1.14.3.1 Pumps

As presented in Table 5-1, the reported design flow for the existing lift station is approximately 350 gallons per minute. The tested lift station capacity is significantly lower than the design flow at approximately 244 gpm. Based on a comparison of the system curve for this lift station and associated force main with the manufacturer's pump curves (see Appendix F), the pumps should be capable of pumping approximately 220 gpm in this system. It is not known why the current capacity of the lift station is approximately two-thirds of the design capacity. The projected future peak hour flow is estimated to be 1,190 gallons per minute, which exceeds the capacity of the existing pumps.

5.1.14.3.2 Force Main

At the current tested lift station capacity of 244 gpm, the velocity in the existing force main is approximately 5 feet per second with a total dynamic head of 107 feet. At the projected future peak hour flow of 1,100 gpm, the velocity in the existing force main would be approximately 23 feet per second with a total dynamic head of approximately 446 feet.

5.1.14.3.3 Wet Well

The existing 8-foot-diameter wet well has an operating volume of approximately 1,354 gallons. At the current tested peak hour flow of 244 gpm, the maximum starts per hour is approximately three. At the projected future flow, the maximum starts per hour would be approximately 13, which slightly exceeds the maximum recommended starts per hour. Based on record drawings the invert elevation of the lowest influent pipe to the wet well is approximately 5 inches above the current pump-on elevation. If the pump-on elevation is raised 6 inches, the starts could be reduced to just below the maximum recommended 12 starts per hour. However, this would result in minor surcharging of the incoming gravity sewer.

5.1.14.4 Electrical System

5.1.14.4.1 General

The existing utility feed is a 100 A, 480 VAC, 3-phase service. The estimated load of the two pumps, a blower motor (3hp est.) and 3kVA 240/120 VAC transformer controls is 30.3kVA, 37A at 480V. The existing distribution equipment and control panels are in good condition. The PLC control panel and telemetry control panel have ample room for modification, but the motor starter panel does not.

5.1.14.4.2 Emergency Supply

The system has a 30 A, 480 VAC pin and sleeve receptacle for connection to an external generator via a 100 A rated manual transfer switch. Based on the estimated sizing above, this connection is marginal in terms of being able to support both pumps and all ancillary loads.

5.1.14.5 Alternatives

This station is less than 20 years old and, based on visual inspection, appears to have significant remaining useful life. At the projected design flow, the wet well will be slightly undersized and larger pumps and force main will be required. Two alternatives were developed for upgrading this station. Figure 5-10, located at the end of this section, depicts the preferred alternative. Figures depicting all other alternatives are included in Appendix H.

5.1.14.5.1 Alternative No. 1 – Replace Pumps and Force Main and Construct a Second Wet Well

Alternative No. 1 would replace the existing pumps with new 30-hp submersible pumps capable of conveying the projected design flow and replace the existing force main with a new 8-inch-diameter force main. A second 4-foot-diameter wet well would be constructed immediately south of the existing wet well to provide additional operating volume. New wet well controls, motor starters, discharge piping, and valves sized for the projected design flow would also be installed. Bypass pumping during a majority of construction would be required.

5.1.14.5.2 Alternative No. 2 – Replace Lift Station

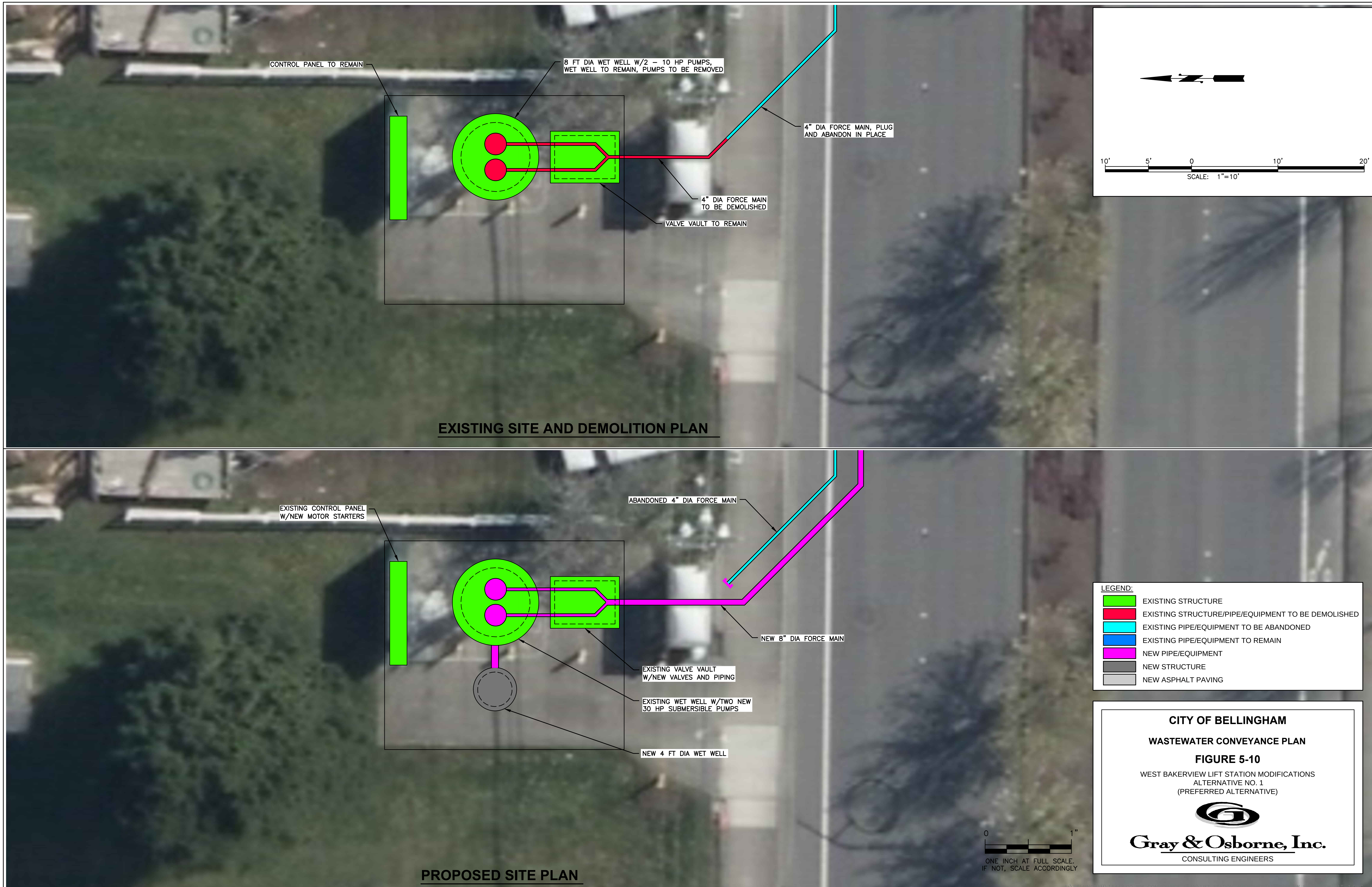
Alternative No. 2 would replace the existing lift station with a new lift station including pumps, wet well, wet well controls and force main, adjacent to the west east side of the existing lift station site. The station would include a new 8-foot diameter wet well and 8-inch force main. New 30-hp pumps, control panels, and motor starters would be provided with sufficient capacity to convey the projected future design flow. Minimal bypass pumping would be required during construction.

Electrical Aspects of Alternatives No. 1 and No. 2

The existing service and distribution will support the proposed 30-hp pumps; however, the generator receptacle should be upsized to handle increased loads. The motor starter panel will need to be replaced to accommodate larger pumps. At 30 hp, it is not uncommon for utilities to require RVSS starters. The existing control panel should be able to accommodate any modifications required,

5.1.14.6 Preferred Alternative

The existing lift station is less than 20 years old and appears to have significant remaining useful life. Alternative No. 1 would increase the wet well capacity while retaining many of the reusable portions of the existing station. A majority of construction could be completed while maintaining the existing station in service. Therefore, Alternative No. 1, which is depicted on Figure 5-10, is the preferred alternative.



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5.1.15 WEST MAPLEWOOD LIFT STATION

5.1.15.1 Existing Facilities

The existing West Maplewood Lift Station consists of two constant speed, 5-hp Smith & Loveless pumps mounted in an 8-foot-diameter steel dry well with an adjacent 8-foot-diameter concrete wet well. This station pumps through a 6-inch-diameter cast iron force main. The lift station is situated within the right-of-way for West Maplewood Avenue. The site is bordered on all sides by West Maplewood Avenue and a privately owned parcel developed with a hotel. The existing lift station covers roughly 190 square feet. The details of the lift station components are summarized in Table 5-14 below.

TABLE 5-14

West Maplewood Lift Station

Year Built	1973
Type	Wet Pit-Dry Pit
Pump Manufacturer	Smith & Loveless
Number of Pumps	2
Horsepower	5
Backup Power	None
Wet Well Diameter	8 feet
Force Main	6-inch cast iron
Force Main Length	100 linear feet

5.1.15.2 Current Condition

Based on observations during the March 2, 2016 inspection and discussions with City staff, this lift station is generally in fair condition but the station is rather old. No evidence of corrosion or deterioration was observed in the dry pit. The wet well has no visible evidence of deterioration or spalling and the piping in the wet well has limited corrosion. If the wet well is reused, coating the interior of the structure may be advisable.

5.1.15.3 Capacity Analysis

5.1.15.3.1 <u>Pumps</u>

As presented in Table 5-1 above, the design capacity for the existing lift station is approximately 350 gallons per minute. Based on a comparison of the system curve for this lift station and associated force main with the manufacturer's pump curves (see Appendix F), the pumps should be capable of pumping approximately 390 gpm in this system. The tested capacity of this station is 441 gpm.

The projected future peak hour flow is estimated to be 895 gallons per minute, which exceeds the capacity of the existing pumps.

5.1.15.3.2 Force Main

At the current tested lift station capacity of 441 gpm, the velocity in the existing force main is approximately 5 feet per second with a total dynamic head of 16 feet. At the projected future peak hour flow of 895 gpm, the velocity in the existing force main would exceed 10 feet per second with a total dynamic head of approximately 24 feet.

5.1.15.3.3 Wet Well

The existing 8-foot-diameter wet well has an operating volume of approximately 1,128 gallons. At the current design peak hour flow, the maximum starts per hour is approximately five. At the projected future flow, the maximum starts per hour would be approximately equivalent to the maximum recommended 12 starts per hour.

5.1.15.4 Electrical System

5.1.15.4.1 General

The existing utility feed is 480 VAC, 3-phase service, with unknown ampacity. The estimated load of the two pumps and 5kVA 240/120 VAC transformer is 19 kVA, 23A at 480V. The motor starter panel and telemetry panel are in poor condition.

5.1.15.4.2 Code Compliance Considerations

The interior and exterior of the panels are corroded, posing undo risk to electrical components and safety of personnel. The panels lack the clear space below the equipment required by the NEC. The motor starter panel has a hole in the front door covered by tape. The ampacity of the main disconnect cannot be read so it is not known if it is appropriately sized; however, it is marked as being rated for 240 VAC and not 480 VAC as required.

5.1.15.5 Alternatives

This station is over 40 years old and constructed on a very constrained site. However, based on visual inspection, it appears that the wet well and dry well are in good condition. The existing wet well would have sufficient capacity to handle the projected design flow and could be retained. The projected peak hour flow will exceed the capacity of the pumps and force main . Given the age and location of this station, it may be desirable to construct a new station in a more easily accessible location. Two alternatives were developed for upgrading this station. Figure 5-11, located at the end of this section, depicts the preferred alternative. Figures depicting all other alternatives are included in Appendix H.

5.1.15.5.1 Alternative No. 1 – Replace Pumps and Force Main and Construct a Second Wet Well

Alternative No. 1 would replace the existing pumps with new 10-hp pumps capable of conveying the projected design flow and would replace the existing force main with a new 8-inch diameter force main. New controls, motor starters, discharge piping, and valves sized for the projected design flow would also be installed. Although the station is over 40 years old, the existing structures do not show signs of significant deterioration. However, to extend their useful life, the interior of both structures would be recoated as part of the upgrade. Bypass pumping during a majority of construction would be required.

5.1.15.5.2 Alternative No. 2 – Construct a New Lift Station

Alternative No. 2 would replace the existing lift station with a new station constructed northwest of the existing station in a planting strip associated with the adjacent hotel. This site is located on private property and it would be necessary to obtain an easement from the property owner for this new site. A wet pit-dry pit lift station would be recommended for this alternative because it would have the fewest above-grade impacts. The new station would include two new 10-hp pumps in a new dry well and an 8-foot diameter wet well. A new 8-inch-diameter force main would be installed and the existing 4-inch force main would be plugged and abandoned in place. A submersible station could also be considered for this site, though the control panels would likely be mounted above grade, presenting greater visual impact.

5.1.15.5.3 Electrical Aspects of Alternatives No. 1 and No. 2

All electrical equipment should be replaced. All new electrical equipment should be located outside of any drywell.

5.1.15.6 Preferred Alternative

The existing station is over 40 years old and likely beyond its useful life and is located on an extremely constrained site. Therefore, if the City can negotiate a new easement on the neighboring property to relocate the station (as shown on Figure 5-11), Alternative No. 2 is the preferred alternative.

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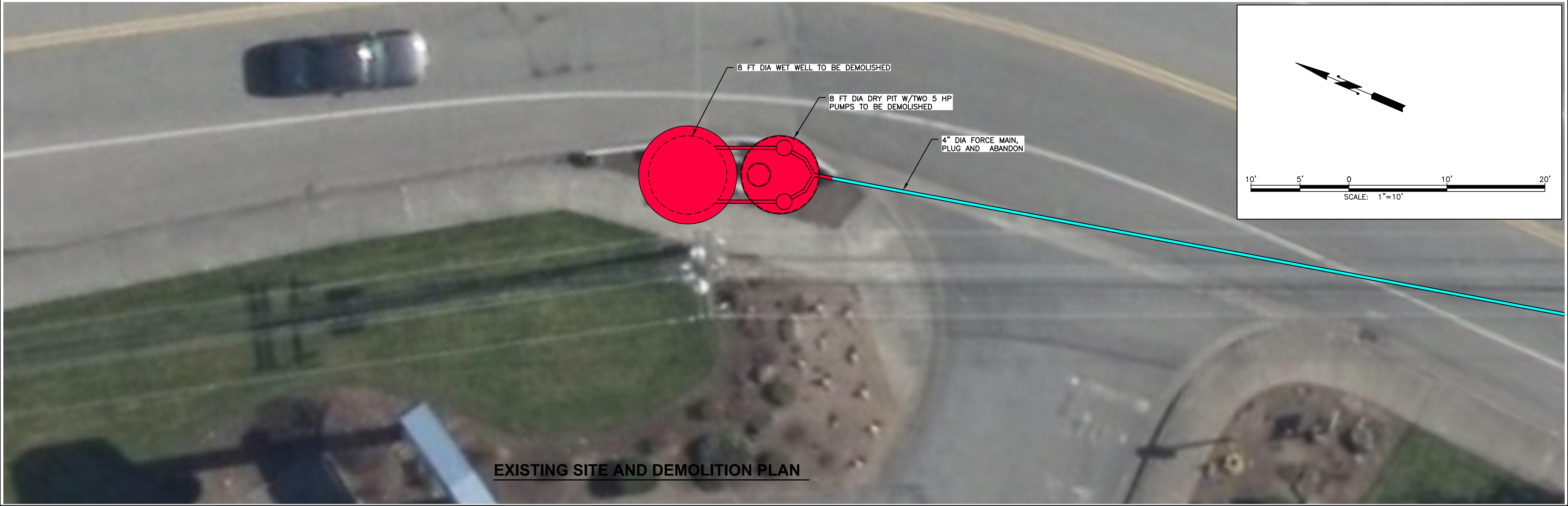

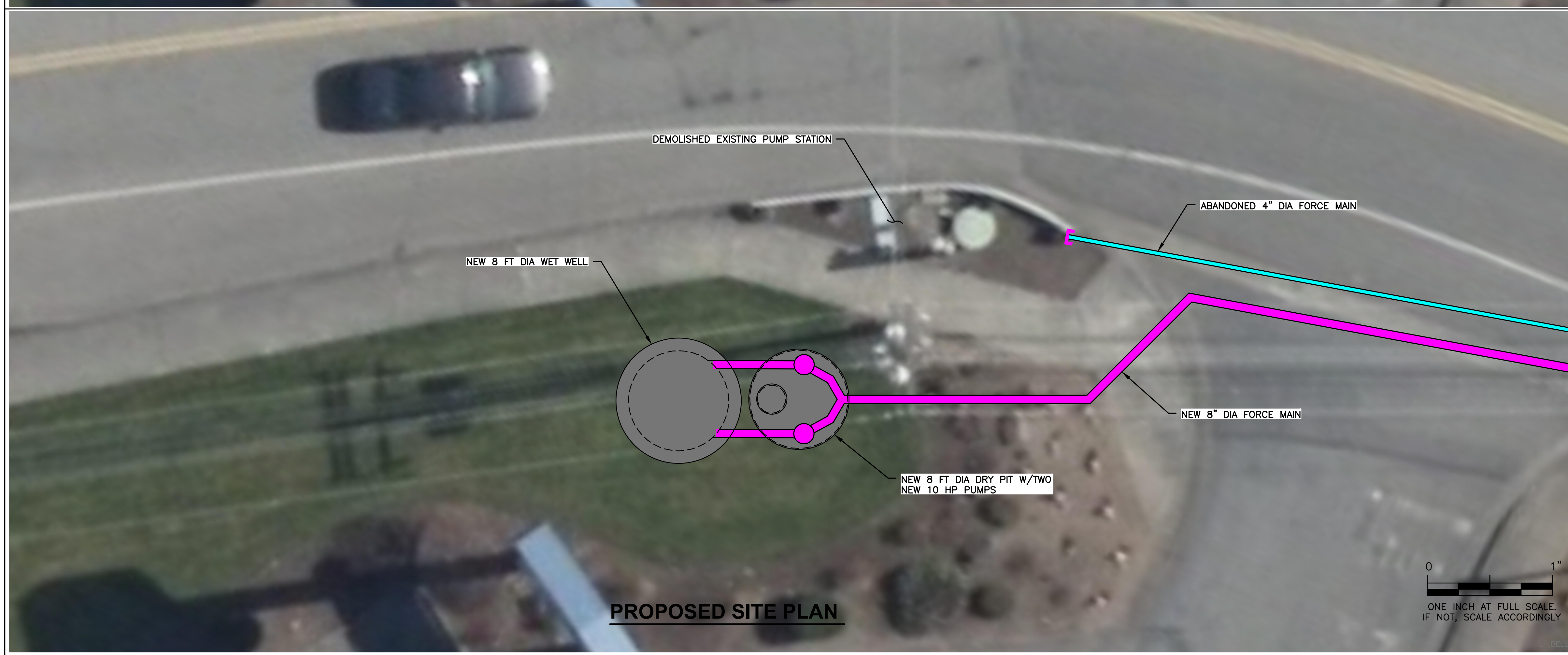

LEGEND:

- EXISTING STRUCTURE
- EXISTING STRUCTURE/PIPE/EQUIPMENT TO BE DEMOLISHED
- EXISTING PIPE/EQUIPMENT TO BE ABANDONED
- EXISTING PIPE/EQUIPMENT TO REMAIN
- NEW PIPE/EQUIPMENT
- NEW STRUCTURE
- NEW ASPHALT PAVING

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN

FIGURE 5-11

WEST MAPLEWOOD LIFT STATION MODIFICATIONS
ALTERNATIVE NO. 2
(PREFERRED ALTERNATIVE)

Gray & Osborne, Inc.
CONSULTING ENGINEERS

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CHAPTER 6

CAPITAL IMPROVEMENT PLAN

6.1 INTRODUCTION

This chapter summarizes the Capital Improvement Plan (CIP) for the Wastewater Conveyance Plan Update. Wastewater capital improvements have been identified and prioritized based on the system hydraulic analysis (Chapter 4), pump station evaluation (Chapter 5), regulatory requirements, component reliability, system benefit, and cost. For all proposed projects identified in this chapter, detailed preliminary project cost estimates are presented in Appendix G. Basin maps illustrating the conceptual locations of the proposed improvement projects are included in this chapter (see Figures 6-1 through 6-7).

Other collection/conveyance capital improvement projects may arise in the future that are not identified as part of the City's CIP presented in this chapter. Such projects may be deemed necessary for remedying an emergency situation, assessing growth in other areas, accommodating improvements proposed by other agencies or land development, or addressing unforeseen problems with the City's wastewater system. Due to budgetary constraints and/or addressing growth scenarios that differ from those modeled in this report, the construction of these projects may require changes in the proposed completion date for projects in the CIP. When new information becomes available, the City retains the flexibility to reschedule, add to, or delete proposed projects and to expand or reduce the scope of the projects, as best determined by the City. Additionally, future planning efforts may affect land use zoning and service requirements within the City. Developments may create streets or provide alignments and locations of facilities that are different than shown on the Plan. Each capital improvement project should be re-evaluated to consider the most recent planning efforts as the proposed completion date for the project approaches.

6.2 PROPOSED SYSTEM IMPROVEMENTS

The proposed system improvements in the CIP are based on projects identified in the collection system hydraulic model, problematic gravity mains and lift station issues identified by City operational staff, on-going programs intended to reduce infiltration and inflow, and projects previously scheduled by the City. Each project cost estimate includes an additional 20 percent construction contingency, 25 percent for design, engineering, and permitting, and an 8.7 percent sales tax. The asphalt restoration costs within the estimates include trench repair so it should be noted that in some cases, an overlay may be required and the cost should be adjusted. All project costs are based on 2016 dollars with no adjustments made for inflation in future years.

Due to changing development needs, project costs were not estimated for the proposed infrastructure within the UGA regions. Although a series of gravity mains and lift stations are shown to serve these regions throughout Figures 4-2 to 4-10, it should be noted that the City may want to investigate the use of vacuum systems to service areas with difficult and/or rocky terrain such as in the “Orchard” and “South” UGA areas (shown in Figures 4-4 and 4-7, respectfully) where a shallower system is necessitated.

The recommended CIP projects are summarized below using the initials of the basin name within the CIP project number.

6.3 GRAVITY SYSTEM CAPITAL IMPROVEMENTS

The gravity system improvements identified are generally designed to increase capacity to accommodate future flows, and in some cases repair/replace failing mains. For budgeting purposes, the cost estimates shown here assume 100 percent import of gravel within the trench and open trench construction in existing right-of-way, with the exception of projects CM2, CM3, and CM4, which require trenchless technologies due to their proximity to streams. In addition, portions of projects NW2, SM5, C5, and B3 include trenchless technologies due to their intersection with Interstate 5, Burlington Northern Sante Fe Railroad, or waterways. The City may be able to realize cost savings by using trenchless technologies in other projects as well. The City should investigate these alternative construction methods during the pre-design or design phase of these projects.

6.3.1 NORTHWEST BASIN GRAVITY SYSTEM PROJECTS (FIGURE 6-1)

6.3.1.1 NW1: West Maplewood Avenue Gravity Sewer Replacement

Replace approximately 1,327 LF of existing 8-inch-diameter sewer pipe along West Maplewood Avenue (west of Firwood Avenue) with 10-inch-diameter pipe, and approximately 1,694 LF of existing 8-inch-diameter sewer pipe with 12-inch-diameter pipe. This project includes abandonment of the parallel pipe system between MH 15001 and MH 15701.

Estimated Project Cost: $1,115,000

6.3.1.2 NW2: Northwest Avenue Gravity Sewer Main Replacement

Replace approximately 403 LF of existing 10-inch-diameter gravity sewer main along Northwest Avenue, north of Alderwood Avenue, with 10-inch-diameter pipe at a new slope of 0.0514. Replace approximately 2,259 LF of existing 10-inch-diameter pipe with 12-inch-diameter pipe. Approximately 185 LF of boring is necessary where this project intersects WSDOT right-of-way under Interstate 5.

Estimated Project Cost: $1,581,000

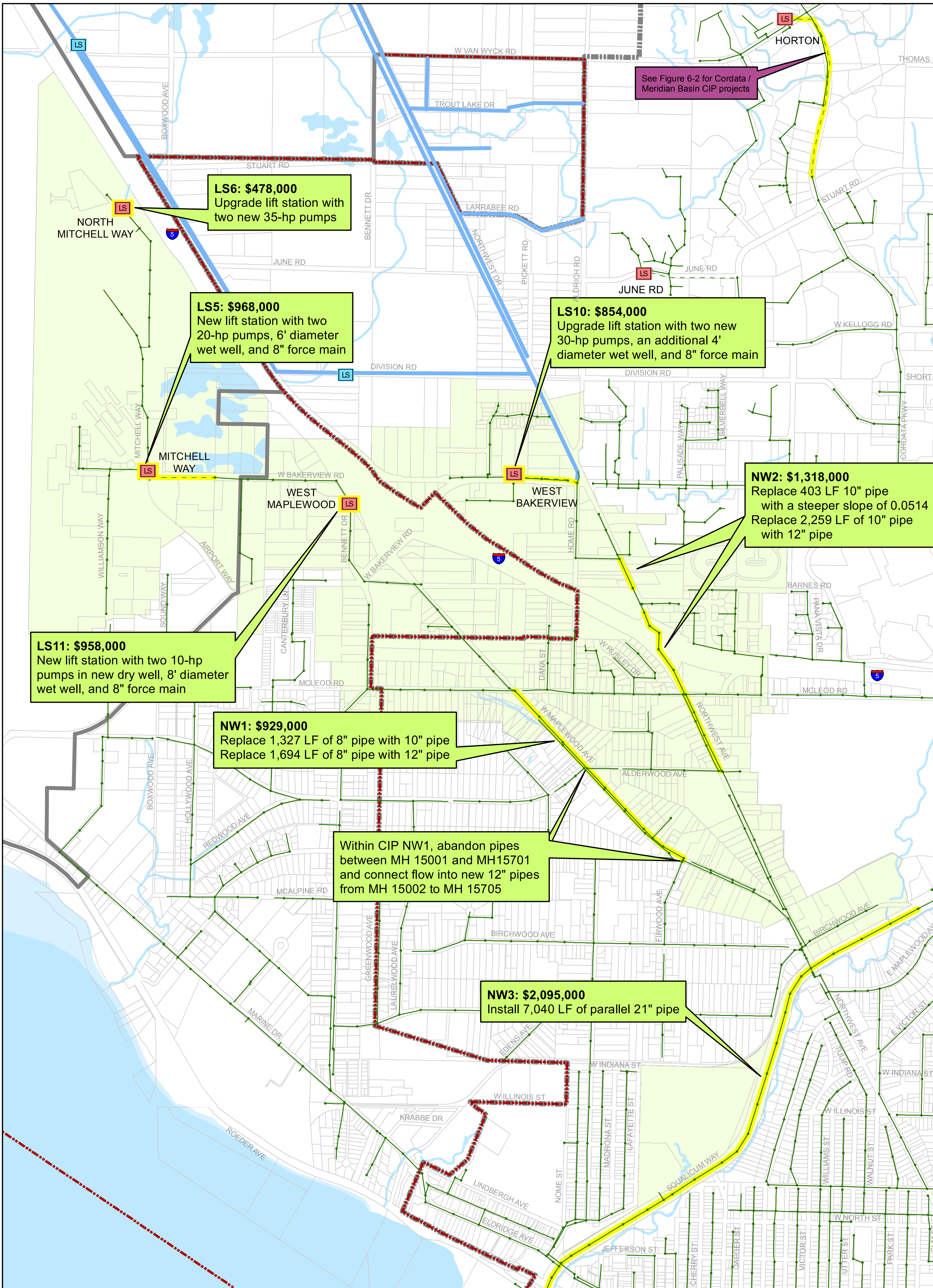

FIGURE 6-1

Legend

- Capital Improvement Project
- Future Development Infrastructure
- Sewer Pipe (Existing)
- Force Main (Existing)
- Manhole (Existing)
- LS Existing Lift Station
- LS Future Development Lift Station
- Northwest Sewer Basin
- UGA Boundary
- City Limits
- UGA Areas

N

0 1,000 2,000 4,000 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
NORTHWEST BASIN CIP PROJECTS

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\report figures\Figure 6-1 Northwest Basin CIPs.mxd

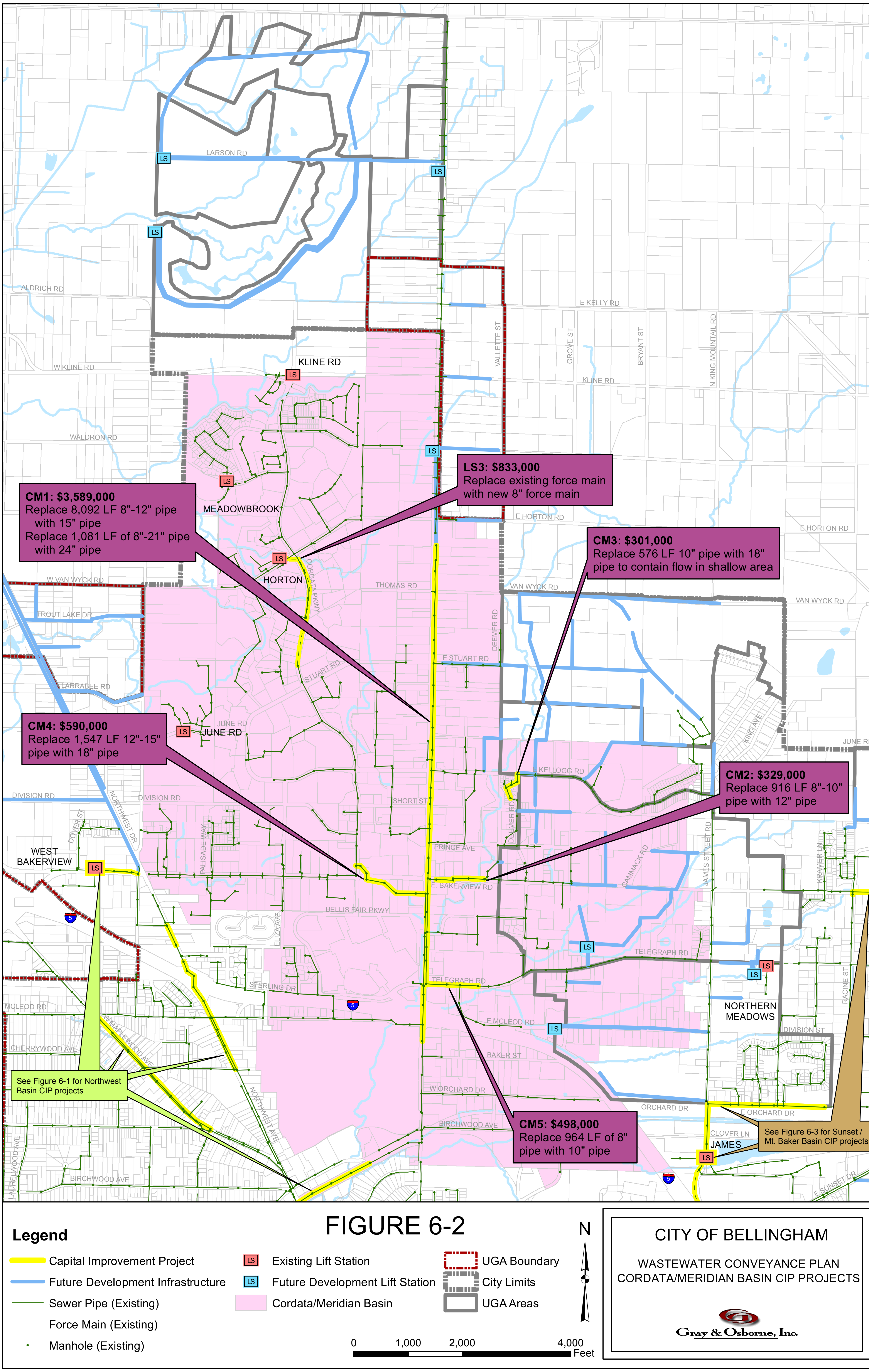

FIGURE 6-2

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
CORDATA/MERIDIAN BASIN CIP PROJECTS

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\report figures\Figure 6-2 Cordata-Meridian Basin CIPs.mxd

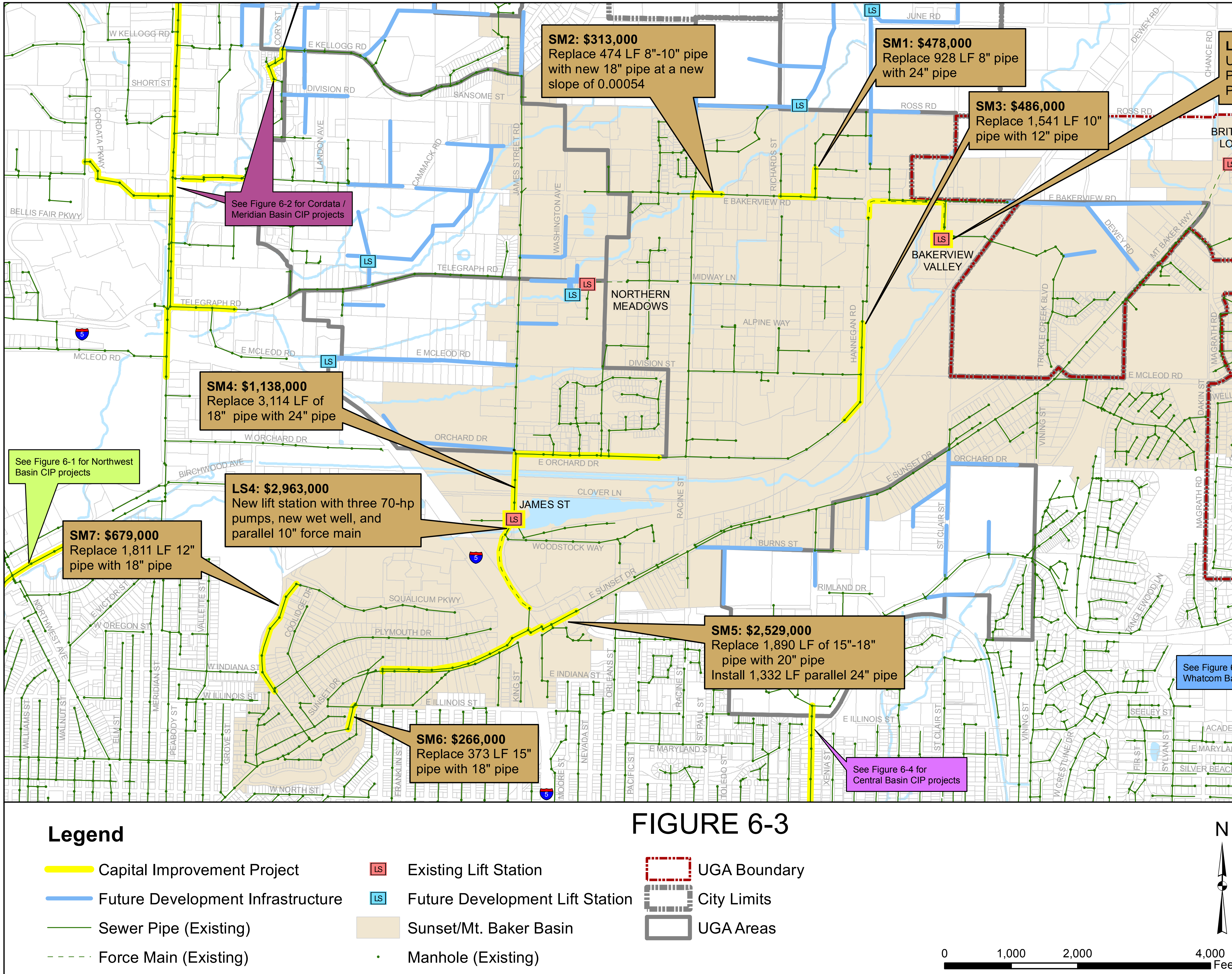

FIGURE 6-3

CITY OF BELLINGHAM
WASTEWATER CONVEYANCE PLAN
SUNSET / MT. BAKER BASIN
CIP PROJECTS

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\report figures\Figure 6-3 Sunset-Mt.Baker Basin CIPs.mxd

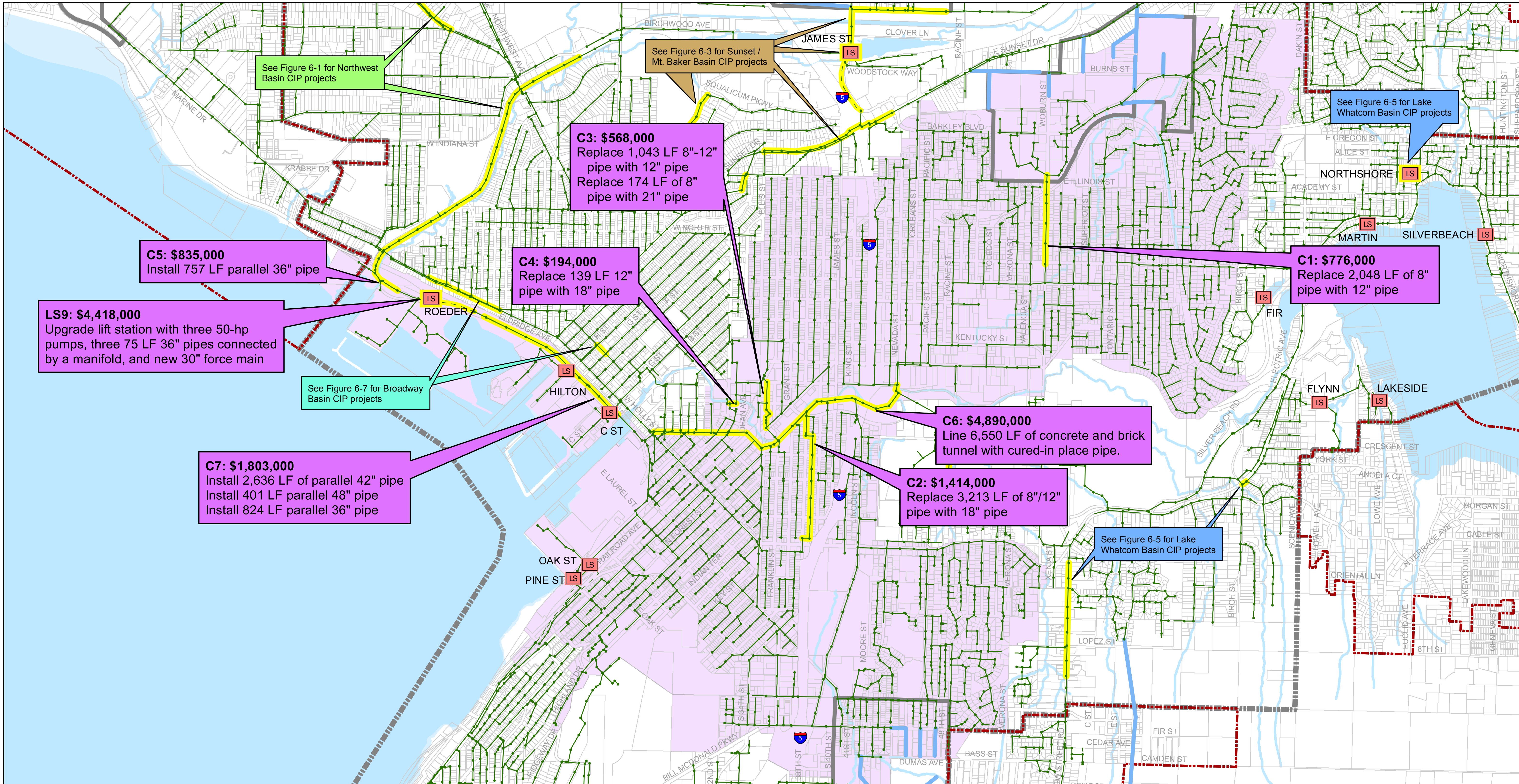

FIGURE 6-4

Legend

- Capitol Improvement Project
- Future Development Infrastructure
- Sewer Pipe (Existing)
- Force Main (Existing)
- Existing Lift Station
- Future Development Lift Station
- Central Basin
- Manhole (Existing)
- UGA Boundary
- City Limits
- UGA Areas

N

0 1,500 3,000 6,000 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
CENTRAL BASIN CIP PROJECTS

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\report figures\Figure 6-4 Central Basin CIPs.mxd

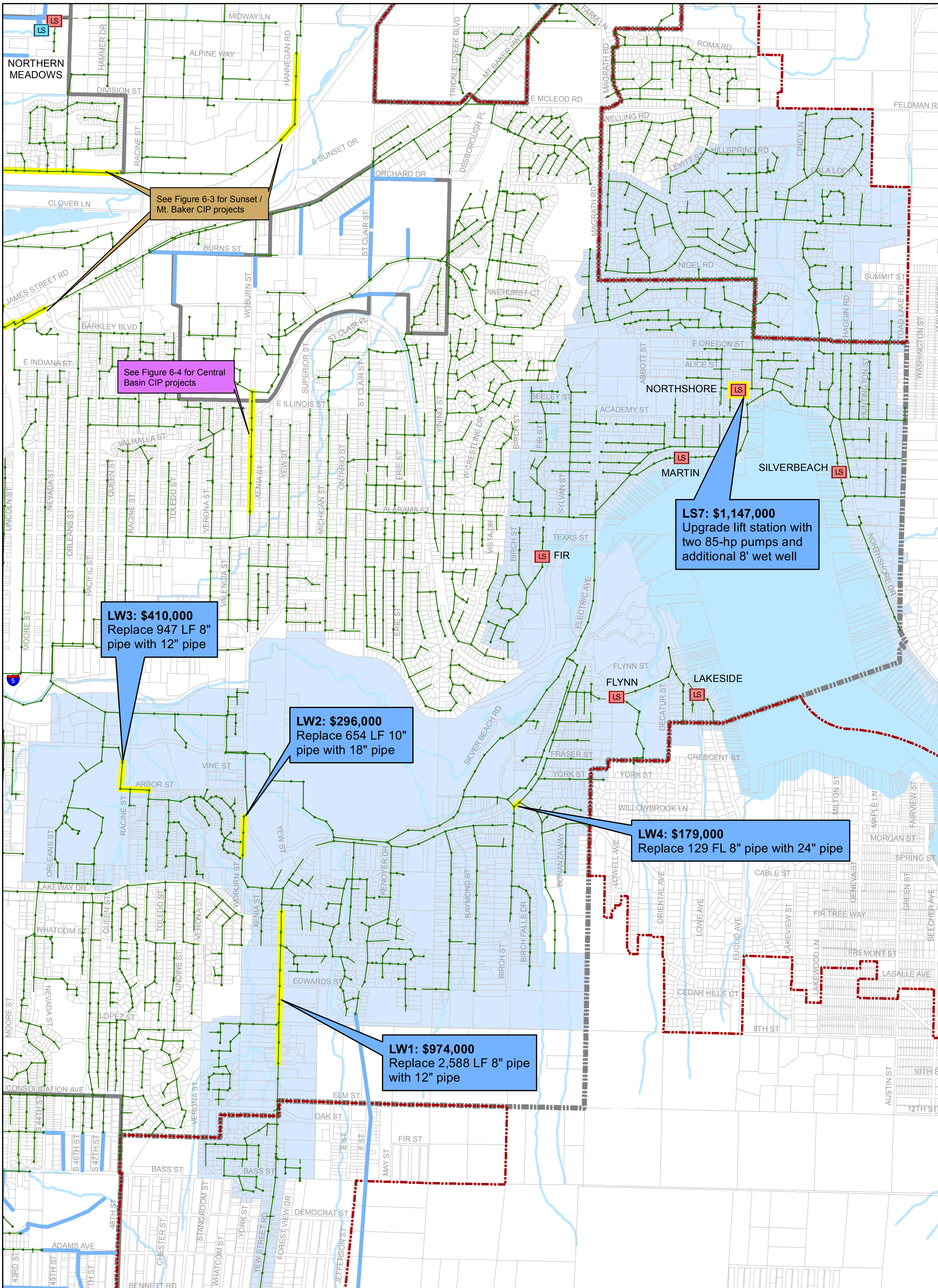

FIGURE 6-5

Legend

- Capital Improvement Project
- Future Development Infrastructure
- Sewer Pipe (Existing)
- Force Main (Existing)
- Manhole (Existing)
- LS Existing Lift Station
- LS Future Development Lift Station
- Lake Whatcom Basin
- UGA Boundary
- City Limits
- UGA Areas

N

0 1,000 2,000 4,000 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
LAKE WHATCOM BASIN CIP PROJECTS

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\report figures\Figure 6-5 Lake Whatcom Basin CIPs.mxd

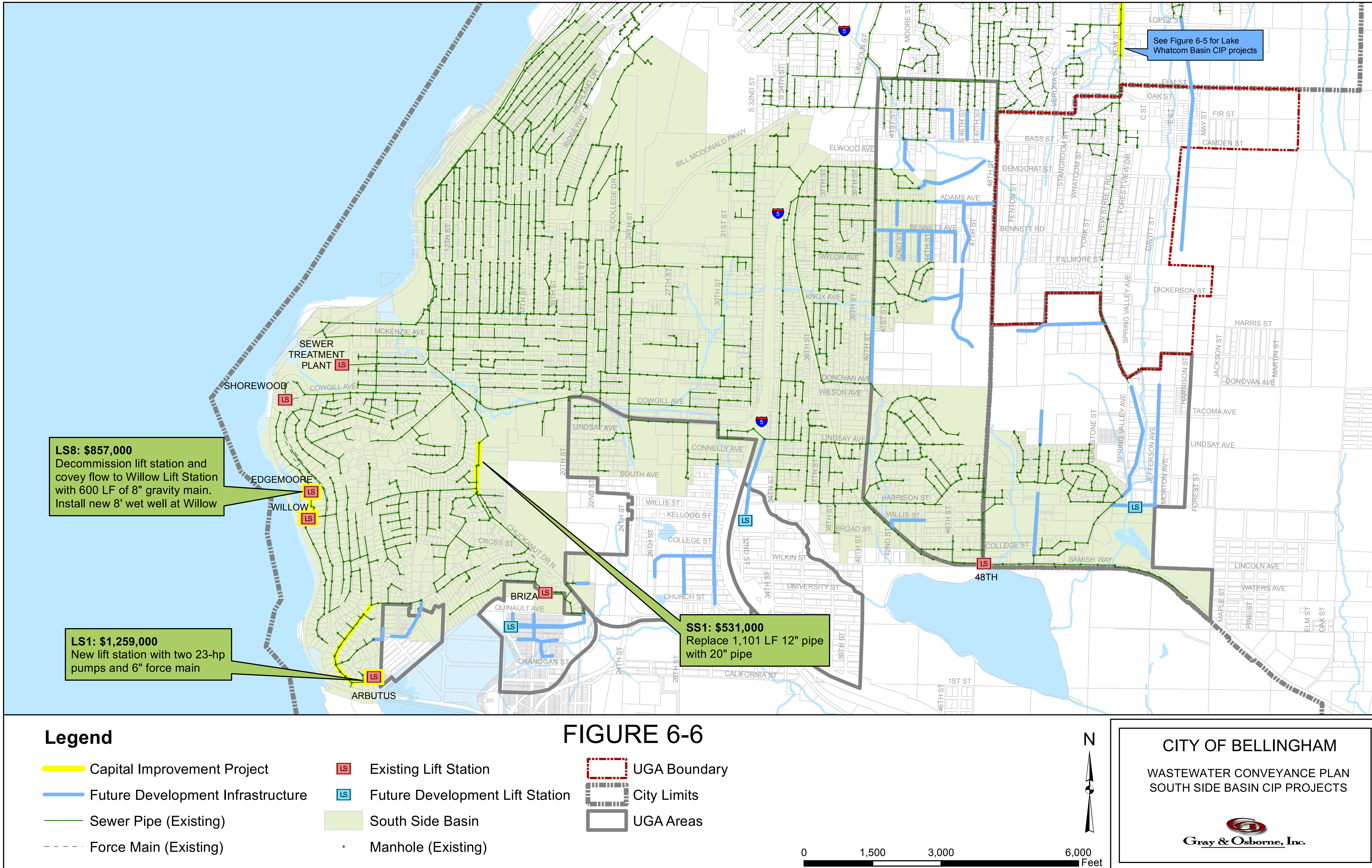

FIGURE 6-6

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
SOUTH SIDE BASIN CIP PROJECTS

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\report figures\Figure 6-6 South Side Basin CIPs.mxd

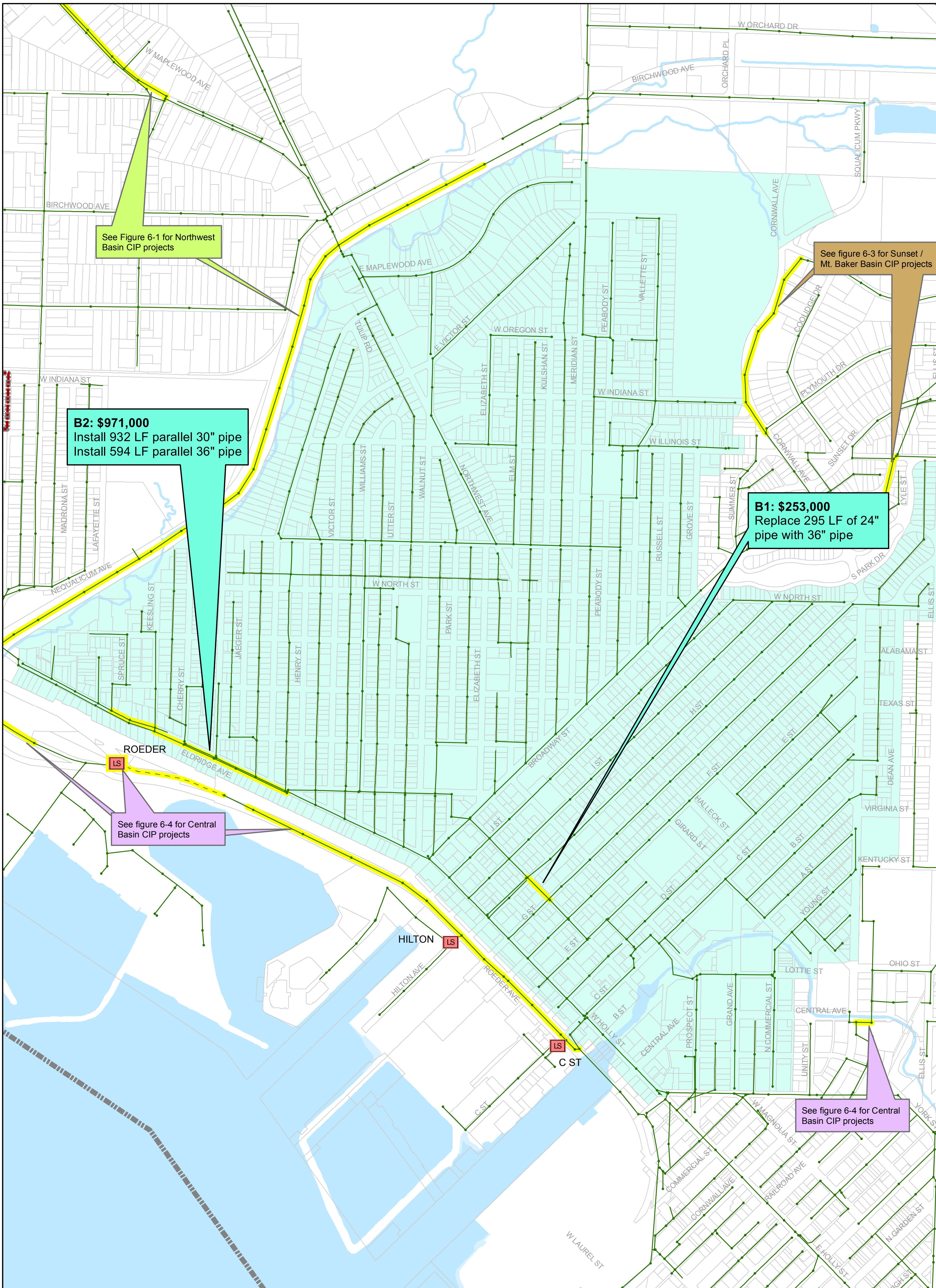

FIGURE 6-7

Legend

- Capital Improvement Project
- Future Development Infrastructure
- Sewer Pipe (Existing)
- Force Main (Existing)
- Manhole (Existing)
- LS Existing Lift Station
- LS Future Development Lift Station
- Broadway Basin
- UGA Boundary
- City Limits
- UGA Areas

N

0 500 1,000 2,000 Feet

CITY OF BELLINGHAM

WASTEWATER CONVEYANCE PLAN
BROADWAY BASIN CIP PROJECTS

Gray & Osborne, Inc.

L:\Bellingham\15448.00 WW Conveyance Plan\Report\report figures\Figure 6-7 Broadway Basin CIPs.mxd

6.3.1.3 NW3: Squalicum Way Parallel Gravity Sewer Main Installation

Install approximately 7,040 LF of 21-inch parallel gravity sewer main along Squalicum Way, north of Roeder Avenue.

Estimated Project Cost: $2,514,000

6.3.2 CORDATA/MERIDIAN BASIN GRAVITY SYSTEM PROJECTS (FIGURE 6-2)

6.3.2.1 CM1: Meridian Street Gravity Sewer Main Replacement

Replace approximately 8,092 LF of existing 8-inch to 12-inch gravity sewer main along Guide Meridian, north of Telegraph Road, with 15-inch-diameter sewer pipe. Along Guide Meridian (south of Telegraph Road), this project would replace approximately 1,081 LF of existing 8-inch, 18-inch, and 21-inch gravity sewer main with 24-inch-diameter pipe.

Estimated Project Cost: $4,307,000

6.3.2.2 CM2: East Bakerview Road Gravity Sewer Main Replacement

Replace approximately 916 LF of existing 8-inch and 10-inch gravity sewer main with 12-inch-diameter pipe just east of Guide Meridian. This project requires the utilization of pipe bursting methods to avoid disruption of nearby Spring Creek.

Estimated Project Cost: $395,000

6.3.2.3 CM3: Deemer Road Gravity Sewer Main Replacement

Replace approximately 576 LF of existing 10-inch gravity sewer main along Deemer Road and within the adjacent housing development with 18-inch-diameter pipe to contain sanitary sewer flows within the existing shallow profile. This project requires the utilization of pipe bursting methods to avoid disruption of nearby Spring Creek.

Estimated Project Cost: $362,000

6.3.2.4 CM4: West Bakerview Road Gravity Sewer Main Replacement

Replace approximately 1,547 LF of existing 12-inch to 15-inch gravity sewer main in parcels along West Bakerview Road, west of Guide Meridian, with 18-inch-diameter pipe. This project requires the utilization of pipe bursting methods to avoid disruption of nearby West Fork Spring Creek.

Estimated Project Cost: $708,000

6.3.2.5 CM5: Telegraph Road Gravity Sewer Main Replacement

Replace approximately 964 LF of existing 8-inch gravity sewer main with 10-inch-diameter pipe along Telegraph Road, east of Guide Meridian.

Estimated Project Cost: $597,000

6.3.3 SUNSET/MT. BAKER BASIN GRAVITY SYSTEM PROJECTS (FIGURE 6-3)

6.3.3.1 SM1: Strider Loop and East Bakerview Road Gravity Sewer Main Replacement

Replace approximately 928 LF of existing 8-inch gravity sewer main with 24-inch-diameter pipe along Strider Road and East Bakerview Road.

Estimated Project Cost: $573,000

6.3.3.2 SM2: East Bakerview Road and Irongate Road Gravity Sewer Main Replacement

Replace approximately 474 LF of existing 8-inch and 10-inch gravity sewer main with 18-inch-diameter pipe along East Bakerview Road at a new slope of 0.0054.

Estimated Project Cost: $375,000

6.3.3.3 SM3: Hannegan Road Gravity Sewer Main Replacement

Replace approximately 1,541 LF of existing 10-inch gravity sewer main with 12-inch-diameter pipe along Hannegan Road and adjacent parcels.

Estimated Project Cost: $584,000

6.3.3.4 SM4: East Orchard Drive and James Street Gravity Sewer Main Replacement

Replace approximately 3,114 LF of existing 18-inch gravity sewer main with 24-inch-diameter pipe along East Orchard Drive and James Street.

Estimated Project Cost: $1,365,000

6.3.3.5 SM5: East Sunset Drive Gravity Sewer Main Replacement and Parallel Gravity Sewer Main Installation

Replace approximately 1,890 LF of existing 15-inch and 18-inch gravity sewer main with 21-inch-diameter pipe and install 1,332 LF of parallel 24-inch pipe along East Sunset Drive, east of Ellis Street. Approximately 445 LF of boring is necessary where this project intersects the WSDOT right-of-way under Interstate 5.

Estimated Project Cost: $3,035,000

6.3.3.6 SM6: East Illinois Street and Broadway Street Gravity Sewer Main Replacement

Replace approximately 373 LF of existing 15-inch gravity sewer main with 18-inch-diameter pipe along Broadway, south of East Illinois Street.

Estimated Project Cost: $320,000

6.3.3.7 SM7: Cornwall Avenue Gravity Sewer Main Replacement

Replace approximately 1,811 LF of existing 12-inch gravity sewer main with 18-inch-diameter pipe behind Parkview Elementary School and along Cornwall Avenue.

Estimated Project Cost: $815,000

6.3.4 CENTRAL BASIN GRAVITY SYSTEM PROJECTS (FIGURE 6-4)

6.3.4.1 C1: Woburn Street Gravity Sewer Main Replacement

Replace approximately 2,048 LF of existing 8-inch gravity sewer main with 12-inch-diameter pipe along Woburn Street, north of Alabama Street.

Estimated Project Cost: $932,000

6.3.4.2 C2: Humboldt Street and Adjacent Alley Gravity Sewer Main Replacement

Replace approximately 3,213 LF of existing 8-inch and 12-inch gravity sewer main with 18-inch-diameter pipe south of Meador Avenue, along one block of Humboldt Street, half of a block along Fraser Street, and then along the alley between Iron Street and Humboldt Street.

Estimated Project Cost: $1,697,000

6.3.4.3 C3: Franklin Street and Ellis Street Gravity Sewer Main Replacement

Replace approximately 1,043 LF of existing 8-inch to 12-inch gravity sewer main with 12-inch-diameter pipe. In addition, this project would entail replacing approximately 174 LF of 8-inch-diameter pipe with 21-inch-diameter pipe within the parcels West of Franklin Street and east of Ellis Street. This estimate assumes that DI pipe which currently runs under buildings just north of North State Street will be abandoned and that new pipe will be placed around the perimeter of the building.

Estimated Project Cost: $681,000

6.3.4.4 C4: Dean Avenue Gravity Sewer Main Replacement

Replace approximately 139 LF of existing 12-inch gravity sewer main with 18-inch-diameter pipe within parcels just east of Cornwall Avenue, south of Whatcom Creek.

Estimated Project Cost: $233,000

6.3.4.5 C5: Roeder Avenue (Near Squalicum Way) Parallel Gravity Sewer Main Installation

Install approximately 757 LF of parallel 36-inch gravity sewer main on Roeder Avenue, just south of the Squalicum Way intersection. Approximately 180 LF of boring is necessary where this project intersects Squalicum Creek.

Estimated Project Cost: $1,002,000

6.3.4.6 C6: Whatcom Creek Tunnel restoration

Line 6,550 feet of existing concrete and brick tunnel with Insituform cured-in place pipe. The project is located between the Champion Street Tunnel and the intersection of Ohio Street and Nevada Street.

Estimated Project Cost: $4,890,000

6.3.4.7 C7: Roeder Avenue (to C Street) Parallel Gravity Sewer Main Installation

Install approximately 2,636 LF of 42-inch parallel gravity sewer main, 401 LF of parallel 48-inch gravity sewer main, and 824 LF of parallel 36-inch gravity sewer main along Roeder Avenue to C Street (west of Whatcom Creek). Approximately 40 LF of boring is necessary where this project intersects the Burlington Northern Santa Fe Railroad right-of-way.

Estimated Project Cost: $2,163,000

6.3.5 LAKE WHATCOM BASIN GRAVITY SYSTEM PROJECTS (FIGURE 6-5)

6.3.5.1 LW1: Yew Street Gravity Sewer Main Replacement

Replace approximately 2,588 LF of existing 8-inch gravity sewer main with 12-inch-diameter pipe along Yew Street, south of Old Lakeway Drive.

Estimated Project Cost: $1,169,000

6.3.5.2 LW2: Old Woburn Street Gravity Sewer Main Replacement

Replace approximately 654 LF of existing 10-inch gravity sewer main with 18-inch-diameter pipe along Old Woburn Street, north of Lakeway Drive.

Estimated Project Cost: $356,000

6.3.5.3 LW3: North of Gladstone Street Gravity Sewer Main Replacement

Replace approximately 947 LF of existing 8-inch gravity sewer main with 12-inch-diameter pipe within the parcels east of Puget Street and north of Gladstone Street.

Estimated Project Cost: $492,000

6.3.5.4 LW4: Electric Avenue Gravity Sewer Main Replacement

Replace approximately 129 LF of existing 8-inch gravity sewer main with 24-inch-diameter pipe along Electric Avenue between Birch Street and Portal Drive.

Estimated Project Cost: $215,000

6.3.6 SOUTH SIDE BASIN GRAVITY SYSTEM PROJECTS (FIGURE 6-6)

6.3.6.1 SS1: Chuckanut Drive North Gravity Sewer Main Replacement

Replace approximately 1,101 LF of existing 12-inch gravity sewer main with 21-inch-diameter pipe along Chuckanut Drive North north of Iris Lane.

Estimated Project Cost: $638,000

6.3.7 BROADWAY BASIN GRAVITY SYSTEM PROJECTS (FIGURE 6-7)

6.3.7.1 B1: Bancroft Street Gravity Sewer Main Replacement

Replace approximately 295 LF of existing 24-inch gravity sewer main with 36-inch-diameter pipe along Bancroft Street between H Street and G Street.

Estimated Project Cost: $303,000

6.3.7.2 B2: Eldridge Avenue Parallel Gravity Sewer Main Installation

Install approximately 189 LF of 30-inch parallel gravity sewer main, 594 LF of parallel 36-inch gravity sewer main, and 743 LF of parallel 30-inch gravity sewer main along Eldridge Avenue west of Henry Street.

Estimated Project Cost: $1,166,000

Although not listed as a specific capital improvement project, it should be noted that the City may eventually want to remove the overflow structure at C Street. To accommodate the anticipated overflow coming to this region, approximately 3,052 LF of parallel 48-inch and 655 LF of 60-inch parallel pipe would need to be installed between the C Street Overflow structure (MH 02825) and the Oak St. lift station.

Table 6-1 provides a summary of each gravity system capital improvement project and the proposed schedule for implementation.

TABLE 6-1

Gravity System Capital Improvement Project Summary

Project No.	Street Location	Total Estimated Cost[(1)]
NW1	West Maplewood Avenue	$1,115,000
NW2	Northwest Avenue	$1,581,000
NW3	Squalicum Way	$2,514,000
CM1	Meridian Street	$4,307,000
CM2	East Bakerview Road	$395,000
CM3	Deemer Road	$362,000
CM4	West Bakerview Road	$708,000
CM5	Telegraph Road	$597,000
SM1	Strider Loop Road/East Bakerview Road	$573,000
SM2	East Bakerview Road/Irongate Road	$375,000
SM3	Hannegan Road	$584,000
SM4	East Orchard Drive/James Street	$1,365,000
SM5	East Sunset Drive	$3,035,000

TABLE 6-1 – (continued)

Gravity System Capital Improvement Project Summary

Project No.	Street Location	Total Estimated Cost(1)
SM6	East Illinois Street/Broadway Street	$320,000
SM7	Cornwall Avenue	$815,000
C1	Woburn Street	$932,000
C2	Humboldt Street/Iron Street	$1,697,000
C3	Franklin Street/Ellis Street	$681,000
C4	Dean Avenue	$233,000
C5	Roeder Avenue (Squalicum Way)	$1,002,000
C6	Whatcom Creek	$4,890,000
C7	Roeder Avenue (to C Street)	$2,163,000
LW1	Yew Street	$1,169,000
LW2	Old Woburn Street	$356,000
LW3	North of Gladstone Street	$492,000
LW4	Electric Avenue	$215,000
SS1	Chuckanut Drive North	$638,000
B1	Bancroft Street	$303,000
B2	Eldridge Avenue	$1,166,000

(1) All project costs in 2016 dollars.

6.4 LIFT STATION CAPITAL IMPROVEMENTS

The following lift station projects are designed to improve operation and/or address the future capacity limitations of the existing lift station.

6.4.1 LS1: Arbutus Lift Station

Construct a new submersible station with two 23-hp pumps east of Arbutus. The station would be sized to serve the entire projected Arbutus service area (equivalent to 311 gpm capacity). The existing gravity sewer would be extended from the existing lift station to the new lift station site and a new 6-inch diameter force main would be installed.

Estimated Project Cost: $1,259,000

6.4.2 LS2: Bakerview Valley Lift Station

This project would consist of replacing the existing force main with an 8-inch force main as a first phase of work, which will reduce the velocity and friction head in the system and increase the capacity of the existing pumps to be capable of conveying the current peak hour flow. Once flows increase to near this revised lift station capacity, the second phase would replace the existing pumps with new larger submersible pumps (20 hp) capable of conveying the projected future peak hour flow of 596 gpm. Electrical and controls equipment would be replaced as needed to accommodate the new, larger pumps and eliminate outdated components.

Estimated Project Cost: $881,000

6.4.3 LS3: Horton Lift Station

Upgrades to this lift station include installing a parallel 8-inch-diameter ductile iron force main. The larger pipe will have lower flow velocity and friction head and with two of the three existing pumps pumping, the lift station will have sufficient capacity to convey the projected potential design flow of 1,305 gpm. The revised station capacity would be approximately 1,600 gpm.

Estimated Project Cost: $833,000

6.4.4 LS4: James Street Lift Station

Replace the entire lift station with a new lift station. The project would entail installing a new wet well and valve vault immediately north of the existing wet well. Based on the projected flow and head conditions, the new lift station would be a triplex submersible lift station with three 70-hp pumps, which will be sized to convey the design peak hour flow (3,737 gpm) with two pumps running. Replacement of the control panels and motor starters would be necessary and an additional parallel 10-inch-diameter force main would be constructed as well.

Estimated Project Cost: $2,963,000

6.4.5 LS5: Mitchell Way Lift Station

Construct a new lift station, including two 20-hp submersible pumps, wet well, controls and force main, adjacent to the east side of the existing lift station site. The station would include a new 6-foot-diameter wet well and 8-inch force main. New pumps, control panels, and motor starters would be provided with sufficient capacity to convey the projected future design flow of 801 gpm. The existing lift station is on and surrounded by Port-owned property; thus, it would likely be necessary to negotiate this relocation with the Port.

Estimated Project Cost: $968,000

6.4.6 LS6: North Mitchell Way Lift Station

This project would entail retaining the existing wet well, force main and any controls that can be used with two new, larger (35-hp) pumps. The new pumps would be capable of conveying the projected design flow of 668 gpm.

Estimated Project Cost: $478,000

6.4.7 LS7: Northshore Lift Station

This project entails converting the existing wet pit-dry pit station to a submersible station. Two 85-horsepower submersible pumps capable of conveying the current peak hour flow (1,215 gpm) would be installed in the existing wet well and a second 8-foot-diameter wet well would be constructed immediately east of the existing wet well to provide additional operating volume. A new valve vault would be installed and the discharge piping would be extended to the existing 10-inch-diameter force main. The existing generator and below grade vault would be retained to provide emergency backup power. This project assumes the station will need to handle the current peak flow of 1,215 gpm. Projections of future flows show a decreased peak value of 878 gpm so this area shall be monitored and proposed upgrades should be adjusted accordingly.

Estimated Project Cost: $1,147,000

6.4.8 LS8: Edgemoor Lift Station

The Edgemoor Lift Station would be decommissioned and all of the flows from the Edgemoor Lift Station service area would be conveyed by 600 feet of 8-inch-diameter gravity to the Willow Lift Station. The new gravity sewer would run along the toe of the slope near Bellingham Bay at the minimum recommended slope of 0.5 percent. The project would involve installing a new, deeper 8-foot-diameter wet well for the Willow Lift Station to accommodate this new sewer. Permitting and topographical concerns are anticipated so it is recommended that a further, more detailed investigation be conducted related to the design of this project.

Estimated Project Cost: $857,000

6.4.9 LS9: Roeder Lift Station

This project would retain the existing lift station building, dry well and wet well and replace the three existing pumps with three 50-hp Fairbanks Morse pumps capable of conveying the future design flow of 12,507 gpm. This project also involves installing three 75-foot-long, 36-inch-diameter pipes connected by a manifold to provide additional operating volume. Each pipe would be sloped at approximately 0.5 percent and the downstream invert elevation would be set at the “Pumps Off” elevation to allow the pipe system to drain out at the end of each pumping cycle. A vault would be constructed at the upstream end of the pipe system as well to provide access. In addition, the existing force main would be replaced with a 30-inch-diameter pipe.

Estimated Project Cost: $4,418,000

6.4.10 LS10: West Bakerview Lift Station

This project includes replacing the existing pumps with new 30 horsepower submersible pumps capable of conveying the projected design flow of 1,190 gpm and replacing the existing force main with a new 8-inch-diameter force main. A second 4-foot-diameter wet well would be constructed immediately south of the existing wet well to provide additional operating volume. New controls, motor starters, discharge piping, and valves sized for the projected design flow would also be installed.

Estimated Project Cost: $854,000

6.4.11 LS11: West Maplewood Lift Station

Replace the existing pump station with a new wet pit-dry pit station constructed northwest of the existing station in a planter strip associated with the adjacent hotel. This site is located on private property and it would be necessary to obtain an easement from the property owner for this new site. The new station would include two new 10-hp pumps in a new dry well and an 8-foot-diameter wet well. A new 8-inch-diameter force main would be installed and the existing 4-inch force main would be plugged and abandoned in place.

Estimated Project Cost: $958,000

Table 6-2 provides a summary of each lift station capital improvement project and the proposed schedule for implementation.

TABLE 6-2

Lift Station Capital Improvement Project Summary

Project Number	Project Title	Year to be Completed	Total Estimated Cost(1)
LS1	Arbutus Lift Station	--(2)	$1,259,000
LS2	Bakerview Valley Lift Station	--(2)	$881,000
LS3	Horton Lift Station	2017	$833,000
LS4	James St. Lift Station	2022	$2,963,000
LS5	Mitchell Way Lift Station	--(2)	$968,000
LS6	North Mitchell Way Lift Station	--(2)	$478,000
LS7	Northshore Lift Station	--(2)	$1,147,000
LS8	Edgemoor Lift Station	--(2)	$857,000
LS9	Roeder Lift Station	2018	$4,418,000
LS10	West Bakerview Lift Station	2017	$854,000
LS11	West Maplewood Lift Station	--(2)	$958,000

(1) All project costs in 2016 dollars.
(2) Project to occur beyond Year 2022.

6.5 CAPITAL IMPROVEMENTS PLAN SCHEDULE

Table 6-3 provides a summary of each capital improvement project and the proposed schedule for implementation over the next 6 years whereas Table 6-4 lists the remaining projects scheduled within the next 20 years. The City can reschedule its capital improvement projects as needed to accommodate unanticipated projects or financial constraints in the future.

TABLE 6-3

6-Year Capital Improvement Project Summary

Project Number	Location	2017(1)	2018(1)	2019(1)	2020(1)	2021(1)	2022(1)
NW3	Squalicum Way					$2,514,000	
CM1	Meridian Street			$4,307,000			
SM5	East Sunset Drive				$3,035,000		
C5	Roeder Avenue (South of Squalicum)					$1,002,000	
C6	Whatcom Creek Tunnel		$4,890,000				
LS10	West Bakerview LS	$854,000					
LS3	Horton LS	$833,000					
LS4	James Street LS						$2,963,000
LS9	Roeder LS		$4,418,000				
Project Total Cost		$1,687,000	$9,308,000	$4,307,000	$3,035,000	$3,516,000	$2,963,000
Estimated Budget		$2,000,000	$15,000,000	$2,000,000	$2,000,000	$2,000,000	$2,000,000
Balance		$313,000	$6,005,000	$3,698,000	$2,663,000	$1,147,000	$184,000

(1) All project costs in 2016 dollars.

Projects to be considered next

NW2	Northwest Avenue	$1,581,000
LS7	Northshore LS	$1,147,000
NW1	West Maplewood Avenue	$1,115,000
C7	Roeder Avenue (West of Whatcom Creek)	$2,163,000

TABLE 6-4

20-Year Capital Improvement Project Summary

Project No.	Street/Location	Total Estimated Cost[(1)]
Gravity Mains		
CM2	East Bakerview Road	$395,000
CM3	Deemer Road	$362,000
CM4	West Bakerview Road	$708,000
CM5	Telegraph Road	$597,000
SM1	Strider Loop Road/East Bakerview Road	$573,000
SM2	East Bakerview Road/Irongate Road	$375,000
SM3	Hannegan Road	$584,000
SM4	East Orchard Drive/James Street	$1,365,000
SM6	East Illinois Street/Broadway Street	$320,000
SM7	Cornwall Avenue	$815,000
C1	Woburn Street	$932,000
C2	Humboldt Street/Iron Street	$1,697,000
C3	Franklin Street/Ellis Street	$681,000
C4	Dean Avenue	$233,000
C7	Roeder Avenue (West of Whatcom Creek)	$2,163,000
LW1	Yew Street	$1,169,000
LW2	Old Woburn Street	$356,000
LW3	North of Gladstone Street	$492,000
LW4	Electric Avenue	$215,000
SS1	Chuckanut Drive North	$638,000
B1	Bancroft Street	$303,000
B2	Eldridge Avenue	$1,166,000
NW1	West Maplewood Avenue	$1,115,000
NW2	Northwest Avenue	$1,581,000
Lift Stations		
LS1	Arbutus LS	$1,259,000
LS2	Bakerview Valley LS	$881,000
LS5	Mitchell Way LS	$968,000
LS6	North Mitchell Way LS	$478,000
LS7	Northshore LS	$1,147,000
LS8	Edgemoor Lift Station	$857,000
LS11	West Maplewood Lift Station	$958,000

(1) All project costs in 2016 dollars.