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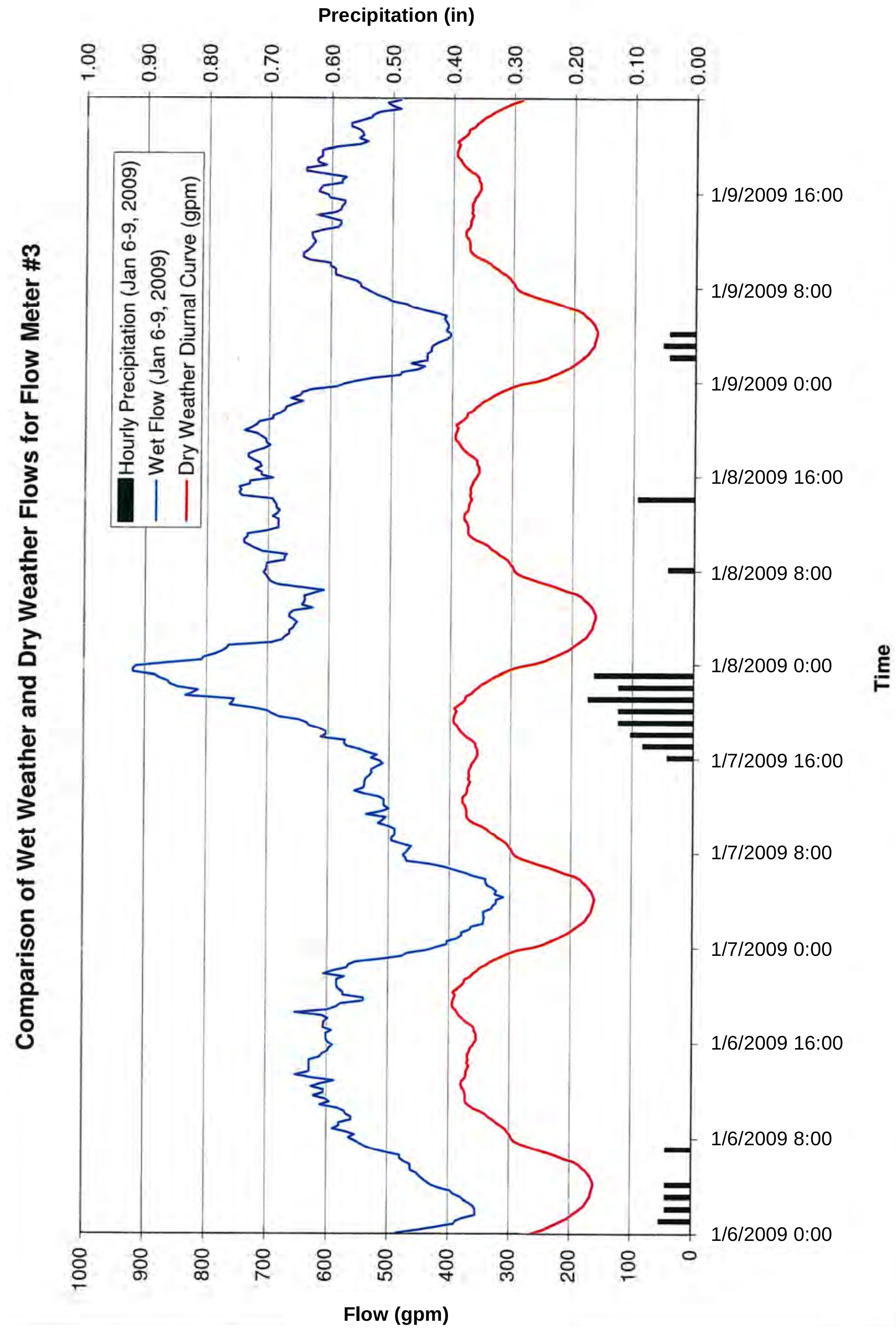

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LYNNWOOD
WASHINGTON

TYPICAL WET WEATHER & DRY WEATHER FLOWS

City of Lynnwood
2012 Wastewater
Comprehensive Plan Update
August 2012

Figure

5-5

Chapter 6 Conveyance System

6.1 Existing City Sewer Pipe System

The Lynnwood wastewater collection system is comprised of approximately 100 miles of gravity pipe varying in size from 6-inch collectors to 36-inch interceptors as shown in Figure 3-8. Many types of pipe material have been used in the construction of the system including concrete, polyvinyl chloride (PVC), high-density polyethylene (HDPE), and ductile iron. The majority of the system as constructed in the 1960s and 1970s is concrete pipe. Eight-inch pipe comprises approximately 80 percent of the collection system as summarized in Table 6-1.

Table 6-1 Gravity Sewer Inventory

Diameter (inch)	Total Length (ft)	Number of Pipes	% of System
6	3,390	27	0.7%
8	415,905	1,824	81.3%
10	24,690	105	4.8%
12	17,640	73	3.4%
15	8,595	47	1.7%
18	18,080	72	3.5%
21	7,880	29	1.5%
24	11,160	37	2.2%
36	4,220	14	0.8%
Total	**511,560**	**2,228**	**100%**

The wastewater collection system is a “separate” system. There are no intentional combined sewers carrying storm water and sewage. Some storm and groundwater does enter the sewage collection system as infiltration and inflow, however.

Generally, wastewater collected by the system flows from north to south in gravity pipes into lift stations, then west in pressure force mains to gravity interceptors, and lastly north to the WWTP. The system is operated from five major collection basins as described below:

<u>Lift Station No. 10</u> serves the largest City basin and includes the City Center area. It receives flow from all of the Scriber Creek Drainage Basin and a portion of the Swamp Creek Drainage Basin. The gravity trunk sewer that services the majority of the basin and discharges to the lift station runs parallel to Scriber Creek and is constructed of 18-inch and 21-inch-diameter concrete pipe. Three minor lift stations with primarily 8-inch collector sewers are tributary to Lift Station No. 10:

- Lift Station No. 4 serve portions of the Swamp Creek Drainage Basin and the Alderwood Mall area discharging into a 6-inch pipe in Alderwood Mall Boulevard and into Lift Station No. 8.
- Lift Station No. 8 serves the remainder of the Alderwood Mall and discharges into an 8-inch line and then into Lift Station No. 10.

- Lift Station No. 14 collects flows from the relatively small basin associated with the Embassy Suites complex south of Interstate 5 and west of 44th Avenue West and thence into Lift Station No. 10.
- Lift Station No. 10 discharges through a 24-inch force main into a 36-inch interceptor in 204th Street near the intersection with 68th Avenue West. From there, the interceptor conveys sewage west to 76th Avenue West Interceptor, and then north to the WWTP.

Lift Station No. 12, located at the southwest corner of the City, services an area of approximately 880 acres. The collection system is comprised primarily of 8-inch pipe, but increases to 21-inch as it nears the lift station. The collection system includes two north-to-south 8-inch collection lines and an 18-inch to 21-inch east-to-west interceptor near the southern City limits. Lift Station No. 12 pumps flows through a 24-inch force main to the interceptor line in 76th Avenue West, where it flows north to the WWTP.

The Browns Bay Trunk Basin, located in the north of the City, drains primarily west in Olympic View Drive through a series of 12-inch and 15-inch pipes. Flow continues south along Olympic View Drive to the 76th Avenue West interceptor and is conveyed north to the WWTP. Lift Station No. 7 serves a small area in the northwest corner of the basin by pumping to the Olympic View Drive trunk sewer.

Gravity Sewer Areas from the remainder of the system drain through several local collector sewers directly into the 36-inch interceptor line in 76th Avenue West which flows north to the wastewater treatment plant.

Edmonds Area served by the City of Lynnwood in part drains into the 76th Avenue West Interceptor. The remaining area is collected by sewer lines in or near the beach into two pump stations immediately north and south of the WWTP.

6.2 Existing Sewer Lift Stations

The City currently operates six stations as shown on Figure 6-1. An inventory of the City's sewage lift stations and force mains is shown in Table 6-2.

Table 6-2 Sewage Lift Station and Force Main Inventory

Station	Location	Type	Pumps	Size HP	Capacity	Force Main	Velocity FPS
4	18150 26th Avenue West	Package Suction Lift	2	10	300 GPM at 77 feet TDH each pump	6-in/650-ft	3.4 fps
7	17131 Meadowdale Dr	Package Suction Lift	2	3	125 GPM at 30 feet TDH each pump	4-in/NA	NA
8	3015 Alderwood Mall Boulevard	Package Wet Well/Dry Well	2	20	600 GPM at 74 feet TDH each pump	8-in/1,500-ft	3.8 fps
10	20329 46th Avenue West	Custom Wet/Dry Well	4	125	1 pump of 2,300 GPM at 150 feet TDH	24-in/9,490-ft	1.6 fps
					2 pumps of 4,400 GPM at 150 feet TDH		3.1 fps
					3 pumps of 6,000 GPM at 150 feet TDH		4.3 fps
					4 pumps of 7,200 GPM at 150 feet TDH		5.1 fps
12	7000 216th Avenue West	Custom Wet/Dry Well	4	75	1 pump of 1,400 GPM at 125 feet TDH	18-in/4,740-ft	1.8 fps
					2 pumps of 2,500 GPM at 130 feet TDH		3.2 fps
					3 pumps of 3,450 GPM at 140 feet TDH		4.4 fps
					4 pumps of 4,200 GPM at 140 feet TDH		5.3 fps
14	20899 44th Avenue West	Package Suction Lift	2	3	300 GPM at 37 feet TDH each pump	6-in/NA	3.4 fps

All pumps in the lift stations are constant speed drives. Wet well parameters for the main lift stations are summarized in Table 6-3.

Table 6-3 Lift Station Wet Well Parameters

Bottom Elevation	Feet From Bottom	Cross Sect Area (ft^2)	Surface Area (ft^2)
Lift Station No. 4			
336.20	0.00	0.0000	0.7854
340.53	4.33	15.1550	28.2743
356.50	20.30	46.0250	28.2743
Lift Station No. 8			
359.80	0.00	0.0000	3.1416
362.00	2.20	5.7829	14.9301
365.05	5.25	26.2500	50.2655
368.00	8.20	49.8500	50.2655
371.66	11.86	79.1300	50.2655
374.37	14.57	100.8100	50.2655
Lift Station No. 10 Wet Well and Stilling Basin			
305.53	0.00	0.0000	125.6600
306.53	1.00	5.5830	256.7200
308.30	2.77	28.4008	305.2200
319.27	13.74	140.8762	305.2200
319.80	14.27	146.3103	256.7200
Lift Station No. 12			
294.50	0.00	0.0000	39.1389
298.90	4.40	86.5480	173.6861
300.68	6.18	121.5606	173.6861
302.50	8.00	157.3600	173.6861

Actual pump curves are not available for all pumps. Draw-down tests have not been conducted to verify pump operating conditions. The pumps within the lift stations described in Table 6-2 are assumed to operate along the approximate pump curves summarized in Table 6-4, which are the operating parameters entered for the hydraulic model.

Table 6-4 Pump Operating Points

Pump Rate in GPM	Dynamic Head
Lift Station No. 4	
400 GPM	50 feet = 22 PSI
300 GPM	77 feet =33 PSI
150 GPM	150 feet = 65 PSI
Lift Station No. 8	
1818 GPM	56 feet = 24 PSI
1560 GPM	84 feet = 36 PSI
1456 GPM	94 feet = 40 PSI
1300 GPM	105 feet = 45 PSI
1040 GPM	121 feet = 52 PSI
780 GPM	157 feet = 68 PSI
Pump Shutoff	161 feet = 69 PSI
Lift Station No. 10	
2250 GPM	188 feet = 81 PSI
1500 GPM	152 feet = 65 PSI
Pump Shutoff	235 feet = 83 PSI
Lift Station No. 12	
1300 GPM	140 feet = 60 PSI
500 GPM	166 feet = 71 PSI
Pump Shutoff	192 feet = 83 PSI

Long detention times in the force mains for Lift Stations Nos. 10 and 12 encourages anaerobic conditions to develop within the pipes resulting in the formation of hydrogen sulfide. Liquid oxygen is injected into each at the discharge points to reduce odors by maintaining aerobic conditions in the force mains, thereby reducing the formation of hydrogen sulfide.
Lift Station No. 14 was upgraded in March 2004 to provide larger motors and pumps, new motor starters, and other electrical upgrades. Pump capacity was increased from 92 to over 300 GPM.

Pump No. 1 in Lift Station No. 8 was upgraded in 1994 with a new pump motor, impeller, and volute, and in 1998 with a new impeller and backplate. Pump No. 2 in Lift Station No. 8 was upgraded in 1995 with a new pump motor, impeller, and volute, and in 2001 with a new impeller. The new station capacity is 600 GPM.

Of the six City pump stations, the only four lift stations inserted into the model are LS No. 4, 8, 10, and 12. Flow areas collected and pumped by Lift Stations 7 and 14 are relatively small and have been loaded into the nearest manholes within the truncated system.

6.3 Hydraulic Model

MIKE URBAN is the software used to simulate the sewer system hydraulics. It is a urban water modeling software package that integrates GIS with water modeling for sewers, water distribution and/or storm drainage systems. The packages are available in modules of different sizes to be relevant to the application envisioned. The Pipeflow module includes the 1D engine which simulates unsteady flow in pipe and channel networks. It has a wide range of network components and flow processes such as:

- Models standard and flexible cross-sections , circular manholes, detention basins, overflow weirs, orifices, pump curves and flow regulators
- CS-Pipeflow simulates subcritical as well as supercritical flow conditions in partially full, full and pressurized pipes
- CS-Pipeflow includes a long term simulation tool for continuous simulations of long time series and an automatic pipe design tool, which finds optimal pipe dimensions based on dynamic simulations.

The Control module features advanced real-time control capabilities for weir crest levels, gate openings, and pump discharges. It permits description of the controllable devices and makes the definition of complex operational logic for interdependent regulators fully transparent.

Other modules can address pollution transport as well as simulate chemical and biological processes in wastewater facilities.

The actual model used for the City sewer system is a truncated model including those portions of the existing City trunk sewer system shown in Figure 6-2. This truncated model is based upon the hydraulic model from the 2006 Comprehensive Sewer Plan, and modified to reflect the current System as shown in the City GIS mapping. The truncated model is comprised of 664 pipes, 659 nodes, four lift stations, and 17 catch basins.

As shown on Figure 6-3, for purposes of the hydraulic model the City sewer collection system was organized into 18 basins or catchment areas, each discharging through a single point.

MIKE URBAN can simulate the dynamic operations of pumps to describe diurnal flows, such as Lift Station No. 10 as shown in Figure 6-4

Figure 6-4 Lift Station No. 10

The sewer modeling basins are described and summarized in Table 6-5.

Table 6-5 Sewer Model Basins

Basin Name	Routed To	Discharge Point	Sewer Acres	Total Acres
Pump Station 4	LS No. 8	Pump Station 4	41	44
Pump Station 8	LS No. 10-B	Pump Station 8	276	279
Pump Station 10A	LS No. 10 - E	SR 99 & 286th Pl	424	433
Pump Station 10B	Western Gravity	PS 10	216	270
Pump Station 10C	LS No. 10 - B	48th Ave & Scriber	324	331
Pump Station 10D	LS No. 10 - B	48th Av & Scriber	216	223
Pump Station 10E	LS No. 10 - D	52nd Pl & 200th	253	264
Pump Station 10F	LS No. 10 - F	SR 99 & 296th St	296	312
Pump Station10G	LS No. 10 – E	SR 99 & 286th Pl	228	250
Pump Station 12A	Western Gravity	LS No. 12	163	163
Pump Station 12B	LS No. 12 – A	SR 99 & 204th St	217	221
Pump Station 12C	LS No. 12 – B	63rd Ave & 212th	292	304
Western Gravity	WWTP	76th Av & Olympic	406	406
Brown Bay A	Browns Bay – B	172nd Pl & 60th	205	315

Table 6-5 Sewer Model Basins

Basin Name	Routed To	Discharge Point	Sewer Acres	Total Acres
Brown Bay B	Browns Bay - C	173rd St & Meadow	137	144
Brown Bay C	Western Gravity	73rd Av & Olympic	171	171
Brown Bay D	Browns Bay - A	SR 99 & 168th St	168	187
Pump Station 14	LS No. 10 - B	I-5 & 44th Ave W	52	60

The truncated sewer model as developed operates in the protocol summarized below:

- 18 catchment areas were created in GIS format, or model basins based upon the physical system's designed drainage
- MIKE URBAN was used to create catchment areas or Theissen polygons for the entire system
- Each polygon has a flow load point located at the centroid of the polygon
- Flow load points are connected to manholes within each of the 18 model basin
- Sum of flows within each 18 catchment areas calculated and distributed equally among the number of Theissen polygon flow load points contributing to each manhole within the 18 catchment areas
- Key manholes are the points of flow origin for the MIKE Urban hydraulic model
- Theissen polygon load points insert a travel time for flow to reach the key manhole and thus attenuate the basin flow to more accurately simulate actual flow conditions

Once the model was created it was loaded with 2010 population and employment data distributed among the catchment areas in the approximate relationship indicated by overlaying the 2010 census tract. Table 6-6 shows 2010 population distributed among the Sub-Basins, plus the projected populations distributed by target year.

Table 6-6 Total Estimated Population by Sub-Basin per Target Year

Sub-Basin	2010	2012	2018	2025	2032
Brown Bay Trunk A	2,972	2,987	3,034	3,088	3,141
Brown Bay Trunk B	776	780	792	806	820
Brown Bay Trunk C	1,443	1,450	1,473	1,499	1,525
Brown Bay Trunk D	1,397	1,409	1,446	1,488	1,530
Pump Station No. 10-A	5,189	5,264	5,488	5,749	6,010
Pump Station No. 10-B	1,177	1,650	3,069	4,725	6,380
Pump Station No. 10-C	2,463	2,787	3,759	4,893	6,028
Pump Station No. 10-D	2,575	2,588	2,628	2,675	2,722
Pump Station No. 10-E	2,087	2,195	2,517	2,894	3,270
Pump Station No. 10-F	2,191	2,284	2,564	2,890	3,217
Pump Station No. 10-G	1,404	1,479	1,703	1,965	2,226
Pump Station No. 12-A	1,061	1,066	1,083	1,102	1,121
Pump Station No. 12-B	2,516	2,536	2,596	2,667	2,737
Pump Station No. 12-C	2,559	2,579	2,640	2,711	2,782
Pump Station No. 8	703	748	881	1,037	1,193
Pump Station No. 4	1	42	165	308	451
Served by Western Gravity	2,344	2,356	2,393	2,435	2,478
Pump Station No. 14	295	297	301	306	312
Total City Served	33,153	34,498	38,532	43,238	47,945
Percent above 2010	-0-	4.1	16.2	30.4	44.6
Served by AWWD	1,562	1,570	1,594	1,623	1,651
Served by Edmonds	442	444	451	459	467
Served by Montlake Terrace	60	60	61	62	63
Total Served by Others	2,064	2,075	2,107	2,144	2,182
City Total	35,217	36,572	40,639	45,382	50,126

Totals shown in Table 6-6 differ slightly from the projection shown in Table 4-2. These differences arise from reconciliation of the City planning assumptions developed for the City Center, the High School Site and the nodes along SR-99. All numbers shown are only projections. The differences are not significant in terms of modeling results and sewer capacities.

A similar method was used to distribute projected employment among the Sub-Basins, with similar minor differences from Table 4-2 for the same reasons. These results are as summarized in Table 6-7.

Table 6-7 Total Estimated Employment by Sub-Basin per Target Year					
Sub-Basin	**2010**	**2012**	**2018**	**2025**	**2032**
Brown Bay Trunk A	572	586	590	637	672
Brown Bay Trunk B	114	117	118	127	134
Brown Bay Trunk C	0	0	0	0	0
Brown Bay Trunk D	603	618	628	683	725
Pump Station No. 10-A	768	799	863	971	1,073
Pump Station No. 10-B	4,695	5,404	7,472	9,955	11,837
Pump Station No. 10-C	1,957	2,426	3,803	5,443	6,687
Pump Station No. 10-D	1,144	1,171	1,179	1,275	1,344
Pump Station No. 10-E	1,399	1,471	1,658	1,909	2,099
Pump Station No. 10-F	1,023	1,080	1,237	1,436	1,587
Pump Station No. 10-G	831	871	968	1,107	1,235
Pump Station No. 12-A	687	703	708	765	807
Pump Station No. 12-B	754	775	793	866	919
Pump Station No. 12-C	640	658	675	738	785
Pump Station No. 14	343	351	354	382	403
Pump Station No. 4	343	351	844	872	893
Pump Station No. 8	7,313	7,422	7,944	8,325	8,603
Served by Western Gravity	1,030	1,054	1,061	1,147	1,210
Total City Served	26,227	27,870	32,911	38,663	43,047
Percent above 2010	-0-	6.3	25.5	47.4	64.1
Served by AWWD	1,373	1,406	1,415	1,530	1,613
Served by Edmonds	229	234	236	255	269
Served by Montlake Terrace	114	117	118	127	134
Total Served by Others	1,716	1,757	1,769	1,912	2,016
City Total	27,943	29,627	34,680	40,575	45,063

The population, employment and area for each catchment basin were then organized by the model software into Theissen polygons. The total flow from the residential, employment and area input were divided equally among the resulting polygons comprising each basin.

Table 6-8 shows the 18 model basins with the number of Theissen polygon load points for each basin and the resulting flow per load point.

Table 6-8 Model Basins Setup for 2010 Flow Modeling

		2010 Average Day Flow in GPD				Load	GPD/
Basin	**Name**	**Resident**	**Non-resident**	**Infiltration**	**Total**	**Points**	**Point**
1	Brown Bay A	135,523	12,500	55,282	203,306	40	5,083
2	Brown Bay B	44,698	12,500	36,936	94,134	27	3,486
3	Brown Bay C	86,580	12,500	46,170	145,250	28	5,188
4	Brown Bay D	80,467	-0-	45,441	125,908	33	3,815
	Subtotal	347,268	37,500	183,829	568,598	128	568,598
5	Lift Station 10 A	308,227	50,000	114,572	472,798	45	10,507
6	Lift Station 10 B	64,264	55,000	58,320	177,584	56	3,171
7	Lift Station 10 C	147,780	25,000	87,583	260,363	63	4,133
8	Lift Station 10 D	154,500	50,000	58,404	262,904	24	10,954
9	Lift Station 10 E	123,968	25,000	68,429	217,397	50	4,348
10	Lift Station 10 F	118,314	25,000	80,028	223,342	31	7,205
11	Lift Station 10 G	80,870	25,000	61,425	167,295	29	5,769
	Subtotal	997,923	255,000	528,760	1,781,683	298	1,781,683
12	Lift Station 12 A	63,660	50,000	44,010	157,670	34	4,637
13	Lift Station 12 B	150,960	25,000	58,477	234,437	12	19,536
14	Lift Station 12 C	153,540	25,000	78,797	257,337	45	5,719
	Subtotal	368,160	100,000	181,283	649,443	91	649,443
15	Lift Station 8	42,180	125,000	74,577	241,757	58	4,168
16	Lift Station 4	60	25,000	11,048	36,108	13	2,778
17	West Gravity	140,640	12,500	109,620	262,760	48	5,474
18	Lift Station 14	15,930	12,500	14,094	16,200	2	8,100
	Subtotal	198,810	175,000	209,339	556,825	121	556,825
	Model Total	1,912,161	567,500	1,103,211	3,556.549	638	

The last line of Table 6-5 shows about 3.56 MGD has been loaded into the model through 638 Theissen polygons, each with a load point. This total flow approximates Lynnwood portion of the 2010 average day flow shown in Table 5-1 with 14 percent excluded as flow from Edmonds, which is computed as follows:

City 2010 Average Day Flow without Edmonds = 4.21 MGD x 0.86 = 3.62 MGD

The model inputs summarized in Table 6-5 are 98 percent of the computed value to three decimal places. Two-decimal accuracy is 100 percent, which is a more realistic view.

6.4 Model Calibration

Calibration of the model required assigning flow from Table 6-8 to each model basin based on population and employment plus allowance for infiltration and rain-induced inflow to match the flows recorded at the wastewater treatment facility. Model Peaking Factors were defined in Table 5-5. Resulting model flows for 2010 are shown by Sub-Basin in Table 6-9.

Table 6-9 Existing System 2010 Flows Modeled by Basin
Flows Shown as GPD

	Model Factor	1.0	1.28	3.03
Basin Number	**Basin Name**	**Average Day**	**Av Day Max Month**	**Peak Day**
1	Brown Bay A	192,093	245,879	582,042
2	Brown Bay B	81,822	104,732	247,921
3	Brown Bay C	123,450	58,016	374,054
4	Brown Bay D	136,239	174,386	412,804
	Subtotal	533,604	683,013	1,616,820
5	Lift Station 10 A	407,966	522,196	1,236,135
6	Lift Station 10 B	263,899	337,790	799,612
7	Lift Station 10 C	281,131	359,848	851,827
8	Lift Station 10 D	229,107	293,257	694,194
9	Lift Station 10 E	222,708	385,066	674,804
10	Lift Station 10 F	219,228	280,612	664,261
11	Lift Station 10 G	161,403	206,596	489,051
	Subtotal	1,785,441	2,285,364	5,409,885
12	Lift Station 12 A	123,247	157,756	373,438
13	Lift Station 12 B	214,148	274,109	648,868
14	Lift Station 12 C	235,342	301,238	713,086
	Subtotal	572,737	733,103	1,735,393
15	Lift Station 8	344,716	441,236	1044,489
16	Lift Station 4	22,959	29,388	69,566
17	West Gravity	270,930	346,790	820,918
18	Lift Station 14	39,568	50,647	119,891
	Subtotal	678,173	868,061	2,054,864
	Model Total	3,569,955	4,569,542	10,816,962

Results of the modeling runs tabulated in Table 6-7 for the pipes and pump stations comprising the truncated model under 2010 average day conditions were used for calibration. Actual metered flow from any of the Sub-Basins is not available for numerical calibration. Total flows produced through the model can be compared with flows recorded at the treatment facility in Table 5-1 as summarized in Table 6-10.

Table 6-10 Model Calibration Comparison
Flow Shown as MGD

Description	Average Day	Average Day Max Month	Peak Day
Flow Recorded at WWTP	4.21	5.37	12.75
Hydraulic Model Output	3.57	4.57	10.82
Estimated Edmonds Flow	0.58	0.74	1.76
Modeled Total	4.15	5.31	12.58
Difference from Recorded	-0.06	-0.26	-0.17
Percent	1.4	4.8	1.3

Differences shown in Table 6-10 represent an effort at defining model accuracy in relation to actual system performance. The flow difference between model results and the WWTP record are reasonably close for the data available. Edmonds contribution is estimated as 14 percent of the WWTP total flow. Flow monitoring conducted for the I/I Study has been incorporated but is not sufficiently accurate for more than generalized relationships among the model sub-basins.

Generally speaking, hydraulic profiles plotted for the 2010 modeled conditions reflect the antidotal understanding of the actual pipe system performance as shown in Figure 6-4.

6.5 Projected Wastewater Flows

The City of Lynnwood 2032 population is projected to be about 50,127 people. The sewer service area relationships shown in Table 6-3 for acreage are not expected change, and the Edmonds population served by Lynnwood may remain about the same as shown. However, the City population served by Alderwood may increase due to increased densities. Therefore, the City sewer system may collect wastewater from about 47,945 people and the treatment facility may serve about 53, 000 people, including the Edmonds area.

Assuming household size remains similar to the 2000 Census at about 2.5 persons, the City collection system will serve about 19,200 ERU. The treatment facility will serve about 21,200 ERU. Projected flows are shown in Table 6-11 for the milestone dates of 2018 (six-year projection) and 2032 (20-year projection).

Table 6-11 Projected Flows for the Existing Sewer System Model Flows Shown as MGD				
Model Factor	**1.0**	**1.28**	**3.03**	**4.28**
Milestone Date	**Average Day**	**Av Day Max Month**	**Peak Day**	**Peak Hour**
2010	3.569	4.570	10.816	15.577
2018	4.104	5.252	12.434	17.563
2032	4.867	6.227	14.742	20.824

Wastewater flows summarized in Table 6-11 models only the City sewer system. Flow into the WWTP also includes acreage from the City of Edmonds as shown in Table 4-8 producing a 2010 average of about 14.3 percent of the WWTP average day flow, or about 0.59 MGD. The Edmunds tributary area is mostly single family homes on largely developed lots. The Edmonds 2032 population and wastewater flow are unlikely to increase as significantly.

However, projections for Lynnwood shown in Table 6-6 project population will increase 45 percent and Table 6-7 shows employment increasing 64 percent by 2032. The Edmonds tributary area may experience some growth as the few remaining vacant lots that exist are developed and some other lots are redeveloped at increased densities. Perhaps the Edmonds wastewater flow will increase by 10 percent in 2032 with the total flow reaching the Lynnwood WWTP as shown in Figure 6-5 and summarized in Table 6-12.

Table 6-12 Total 2032 Wastewater Flows to the Lynnwood Treatment Facility				
Flow Component	**Annual Av Dy**	**Av Day Max Mo**	**Peak Day**	**Peak Hour**
Peak Factor	1.0	1.28	3.03	4.28
Lynnwood MGD	4.87	6.22	14.7	20.8
Edmonds MGD	0.65	0.83	1.97	2.8
Total MGD	5.52	7.05	16.7	23.6
Total GPM	3,830	4,900	11,600	16,390
Projection to 2040 Flows				
Total MGD	6.1	7.7	18.3	26.0
Total GPM	4,200	5,400	12,700	18,000

Assuming a straight-line flow increase, average day maximum month flow may reach 85 percent of the existing NPDES permit condition of 7.4 MGD, 6.29 MGD, about the year 2028. The permit requires that when flow reaches the 85 percent threshold the City must prepare plans to maintain capacity.

Analysis of the WWTP is based on projected 2040 flows however. Available population and employment data is insufficient to distribute among the Sub-Basins to populate the hydraulic model. Table 6-11 indicates population in the Lynnwood sewer service area will be increasing at about 1.5 percent annually with employment growing at about 1.4 percent annually. Growth rates usually decline as population and employment grow. Accordingly, the 2040 flow projections shown in Table 6-12 reflect 10 percent above the 2032 flows.

There are no significant industrial users in the City of Lynnwood, and there are not expected to be any more to the City in the future, as there is no heavy industrial zoning. The industry that has the largest impact on wastewater is probably the food service industry which generates a large quantity of grease. City staff routinely contacts all restaurants to inform them of the City's Fats, Oils, and Grease (FOG) program. The FOG program requires all restaurants to have grease trap, and/or a grease separator, and to keep them maintained.

Maintenance is always an issue with these devices. In 2013, staff will propose an ordinance to the City Council to give staff the ability to fine restaurants who do not keep their grease separators in working order.

In this same request to Council, there will also be a request to require dental offices to have and maintain amalgam separators. Dental offices are considered to be primary source of mercury, which is an issue for the exhaust from the treatment plant incinerator.

6.6 Collection System Deficiencies - 2032

The hydraulic model was updated to include the three lift station improvements envisioned in the 2006 Plan as reflecting the minimum 2018 collection system configuration and run with the flow inputs summarized in Table 6-11. Figure 6-6 shows the pipes identified by the model as flowing full under 2032 peak hour flow conditions.

As noted earlier in Section 2.7 Operations and Maintenance, the City intends that the sewer system will pipe capacity for wastewater flows resulting from a 20-year storm event, which means a storm with a 5 percent likelihood of occurring in any given year. Some level of pipe surcharge is acceptable under the peak hour flow conditions for this once in 20-year circumstance when water levels will rise in certain manholes. Maintaining at least 5 feet of pipe and manhole freeboard to the ground above the modeled hydraulic gradient is deemed sufficient that wastewater would not overflow onto the ground or backup into any home or business.

The areas where such conditions are predicted by the hydraulic model to occur under 2032 peak hours flow conditions are indicated on Figure 6-6. Figure 6-7 shows the Pipe Segment G-1 in plan view on an aerial photograph. Figure 6-8 illustrates the modeled hydraulic profile of of the existing pipe in this Segment under 3032 flows. The blue color shows the water level within the pipe system at midnight for peak hour conditions. The red line shows the maximum water level reached under peak hour conditions during the modeled 2032 day. In places the red line is above the ground line, indicating the existing pipe system would overflow unless capacity is improved.

Iterative adjustments were evaluated using the hydraulic model to determine what pipe diameter would provide adequate capacity. Figure 6-9 shows the same pipe Segment with pipe of sufficient diameter under 2032 peak day flow conditions. The red line now remains well below the ground line and minor surcharging occurs only in limited places.

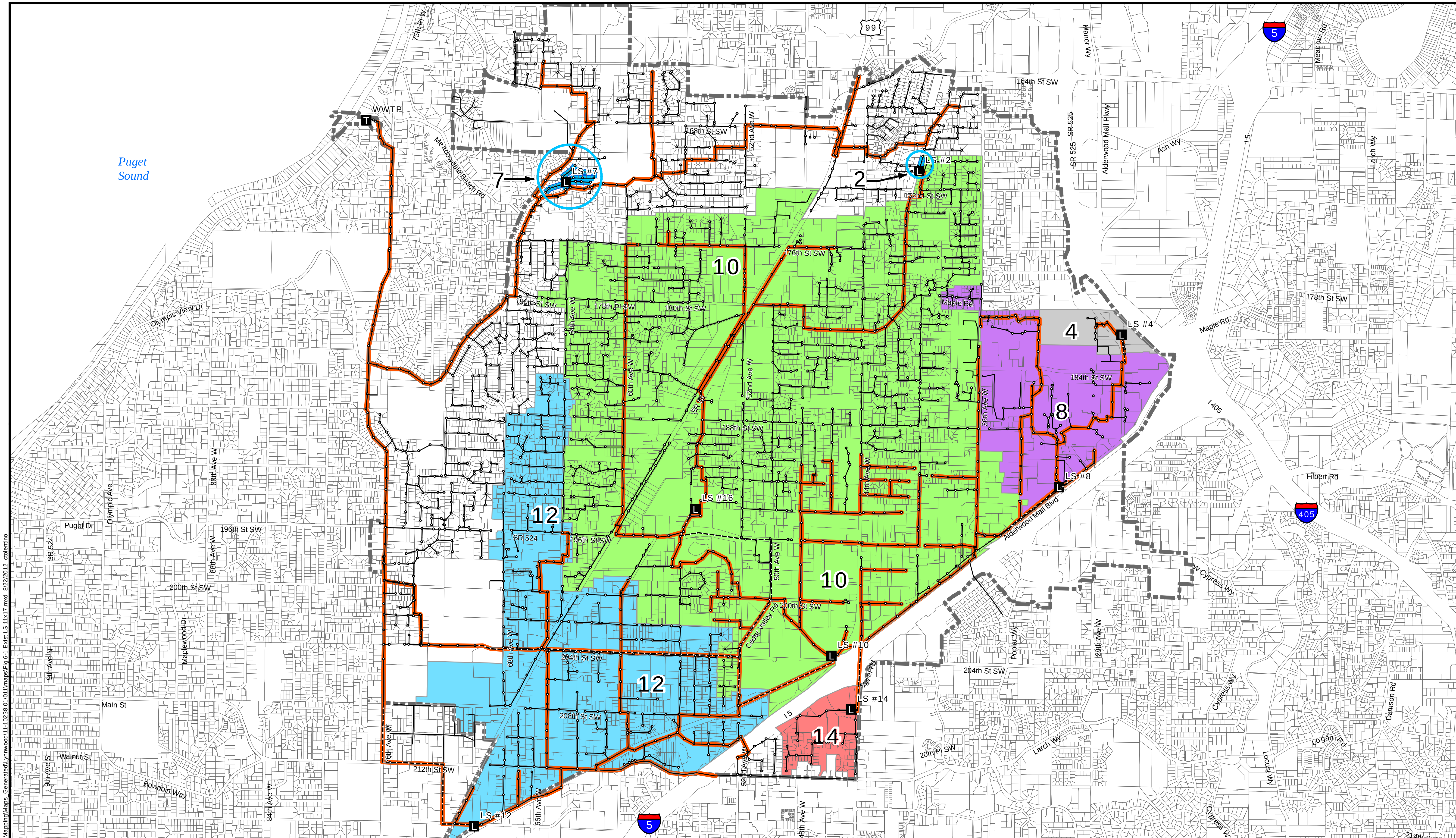
Similar model evaluations were conducted for the other pipe segments identified in Figure 6-6 as providing inadequate capacity for 2032 peak hour flow conditions. Plans and hydraulic profiles for these Segments are included as Appendix A.

Perhaps because of revised growth forecasts and differences in hydraulic modeling, the deficiencies identified herein are significantly different than are shown in previous sewer plans.

6.7 Collection System Deficiencies - 2018

Wastewater flows projected for 2018 development as summarized in Table 6-11 were input to the minimal 2018 hydraulic model, which was then run. Figure 6-10 identifies the pipes projected by this model to flow full under these conditions, and those pipe reaches where surcharging poses the risk of overflow or backup are indicated.

The deficiencies identified through the 2018 hydraulic model of the collection system as indicated on Figure 6-10, all of which are also shown as deficient on Figure 6-6,were used to formulate the 6-year capital improvement plan (CIP).



P:\MappingMaps Generated\Lynnwood\11-10238.01\0011\maps\Fig 6-1 Exist LS 11x17.mxd 8/22/2012 ctolentino

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Sewer System: City of Lynnwood March 2011
GIS Base Data: Snohomish County
This map is a geographic representation based on information available. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.

0 1,000 2,000 Feet
Scale in Feet

NORTH

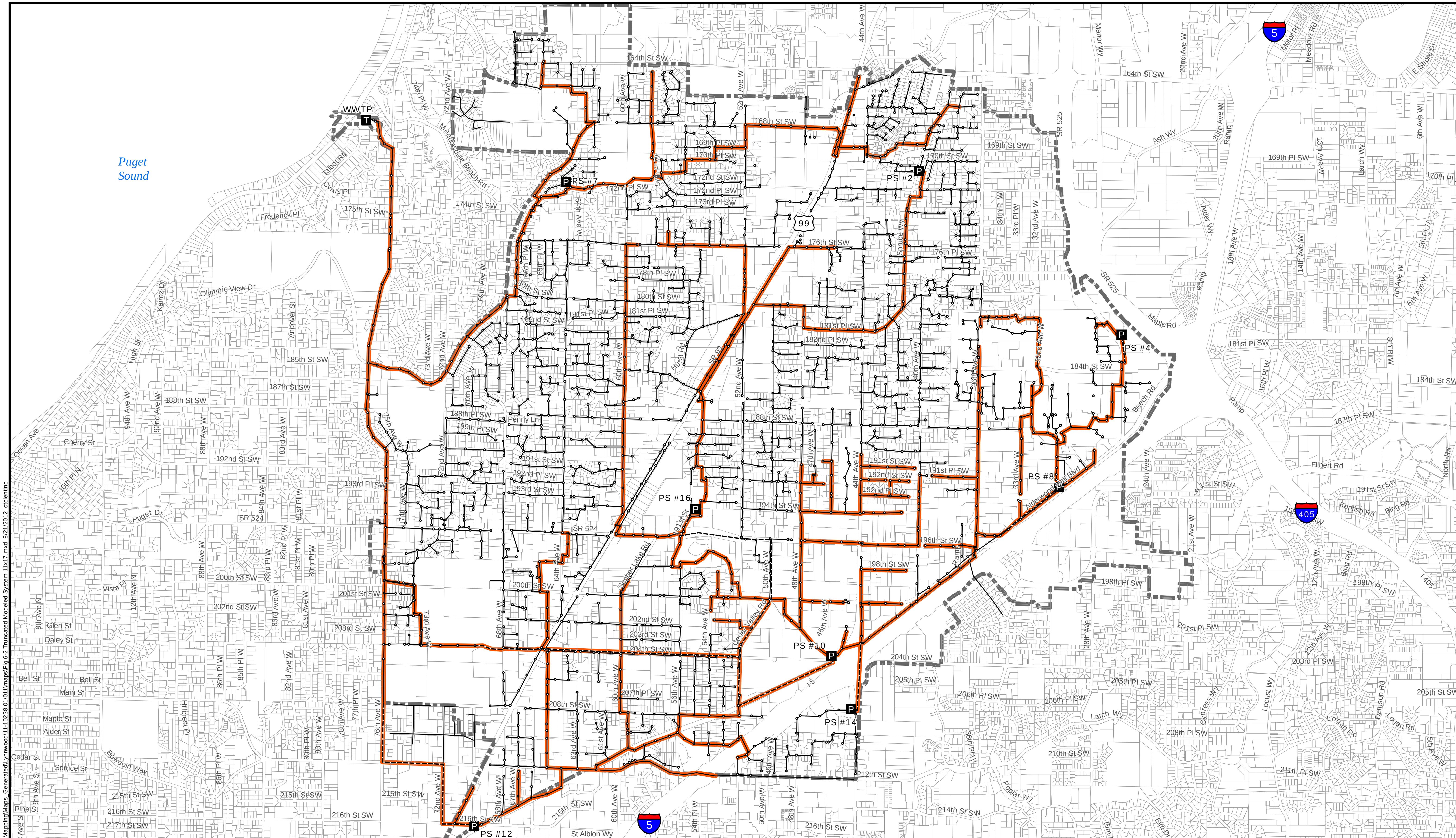
- Manholes
- Septic Tanks
- T WWTP
- L Lift Stations
- Force Main
- Sewer Lines
- Sewer System Truncated
- City of Lynnwood
- Parcels

LYNNWOOD WASHINGTON

EXISTING LIFT STATIONS

City of Lynnwood
2012 Wastewater Comprehensive Plan Update
August 2012

Figure
6-1



P:\Mapping\Maps_Generated\Lynnwood\11-10238.01\0011\maps\Fig 6-2 Truncated Modeled System 11x17.mxd 8/21/2012 ctolentino

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0 1,000 2,000 Feet
Scale in Feet

NORTH

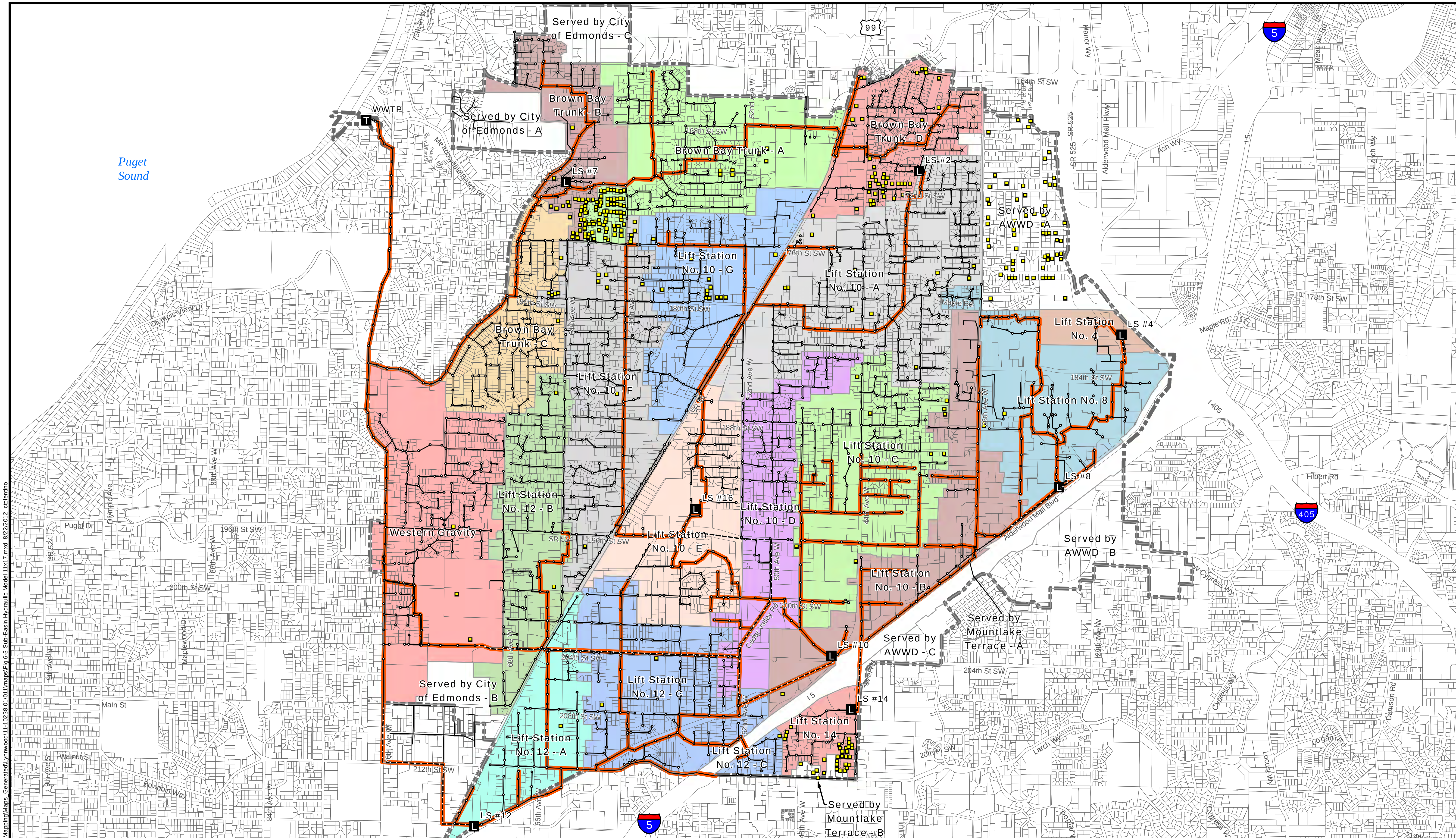
Manholes
Septic Tanks
T WWTP
P Pump Stations
Force Main
Sewer Lines
Sewer System Truncated
City of Lynnwood

LYNNWOOD WASHINGTON

TRUNCATED MODELED SYSTEM

City of Lynnwood
2012 Wastewater Comprehensive Plan Update
August 2012

Figure
6-2



P:\Mapping\Maps_Generated\Lynnwood\11-10238.01\011\maps\Fig 6-3 Sub-Basin Hydraulic Model 11x17.mxd 8/22/2012 ctolentino

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GIS Base Data: Snohomish County
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0 1,000 2,000 Feet
Scale in Feet

NORTH

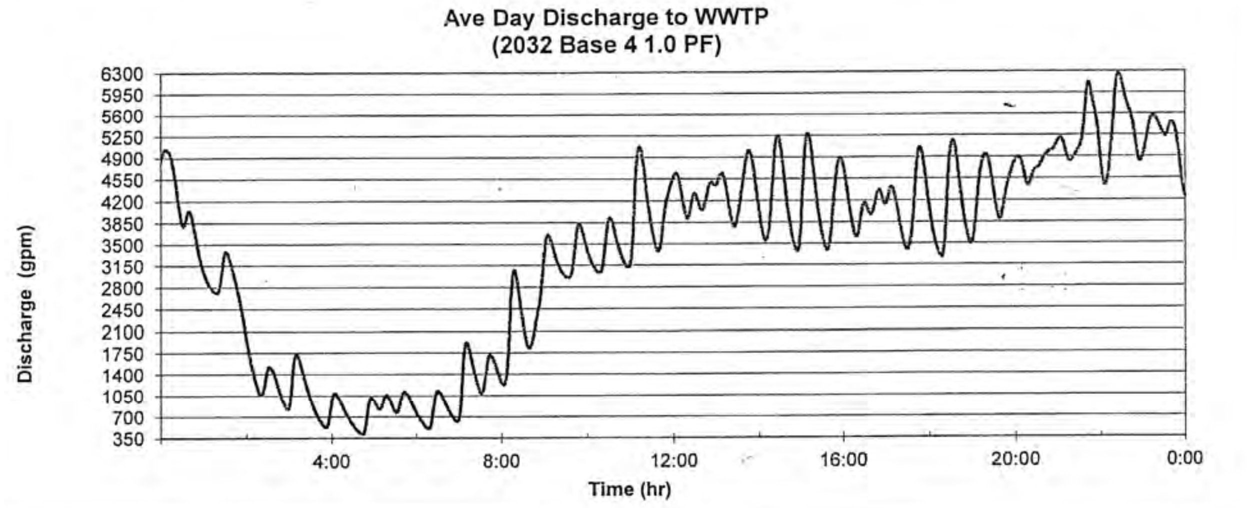
Manholes
Septic Tanks
WWTP
Lift Stations
Force Main
Sewer Lines
Sewer System Truncated
City of Lynnwood
Parcels

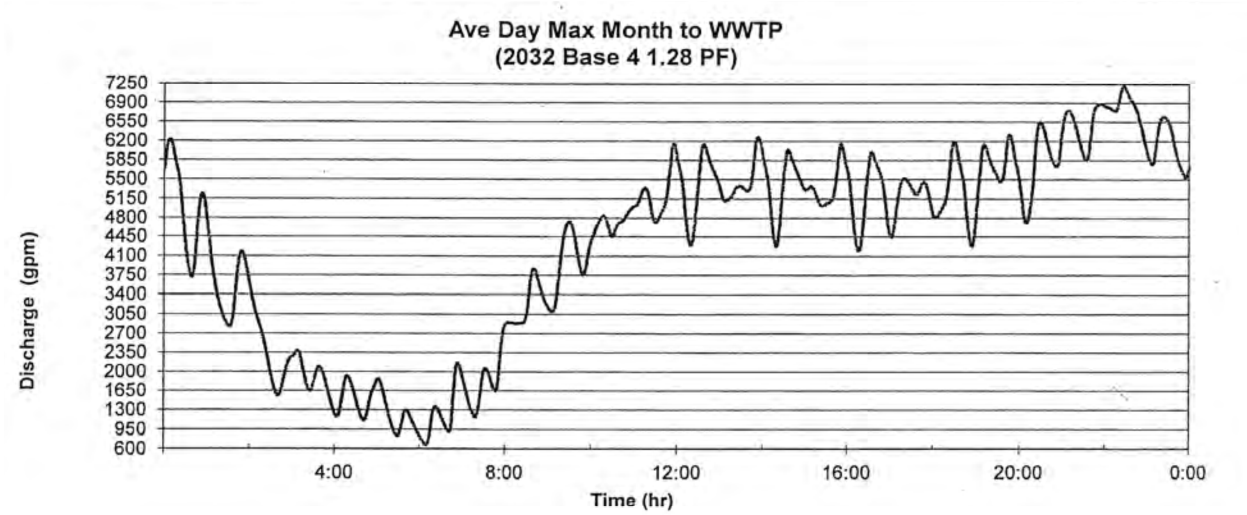
LYNNWOOD WASHINGTON

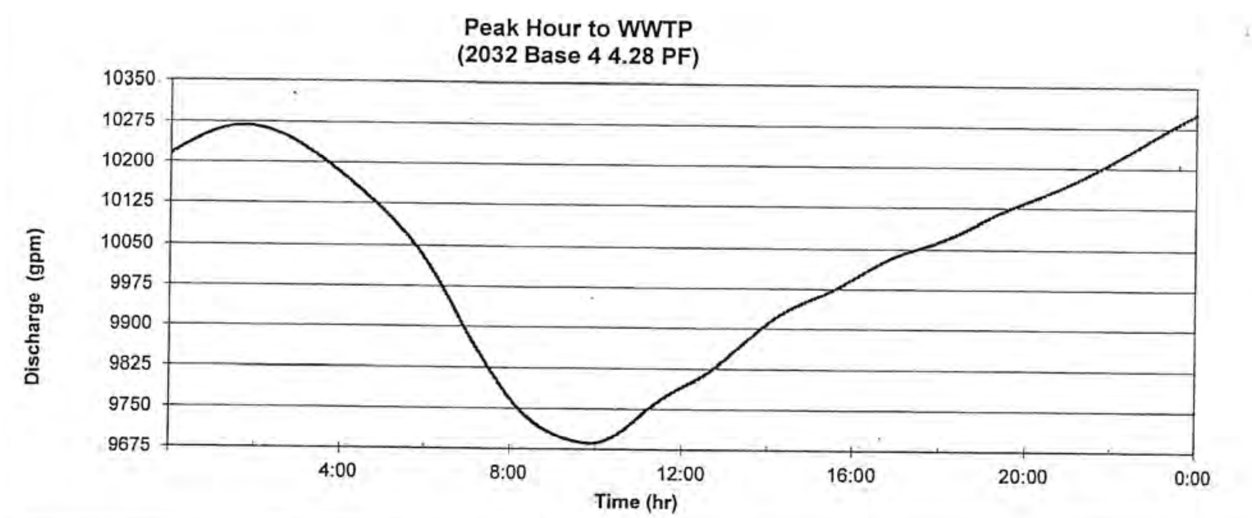
SUB-BASINS FOR HYDRAULIC MODEL

City of Lynnwood
2012 Wastewater Comprehensive Plan Update
August 2012

Figure
6-3







P:\Mapping\Maps_Generated\Lynnwood\11-10238.01\011\maps\Fig 6-5 Diurnal Curves Hydraulic WWTP 8.5x11.mxd 8/17/2012 ctolentino

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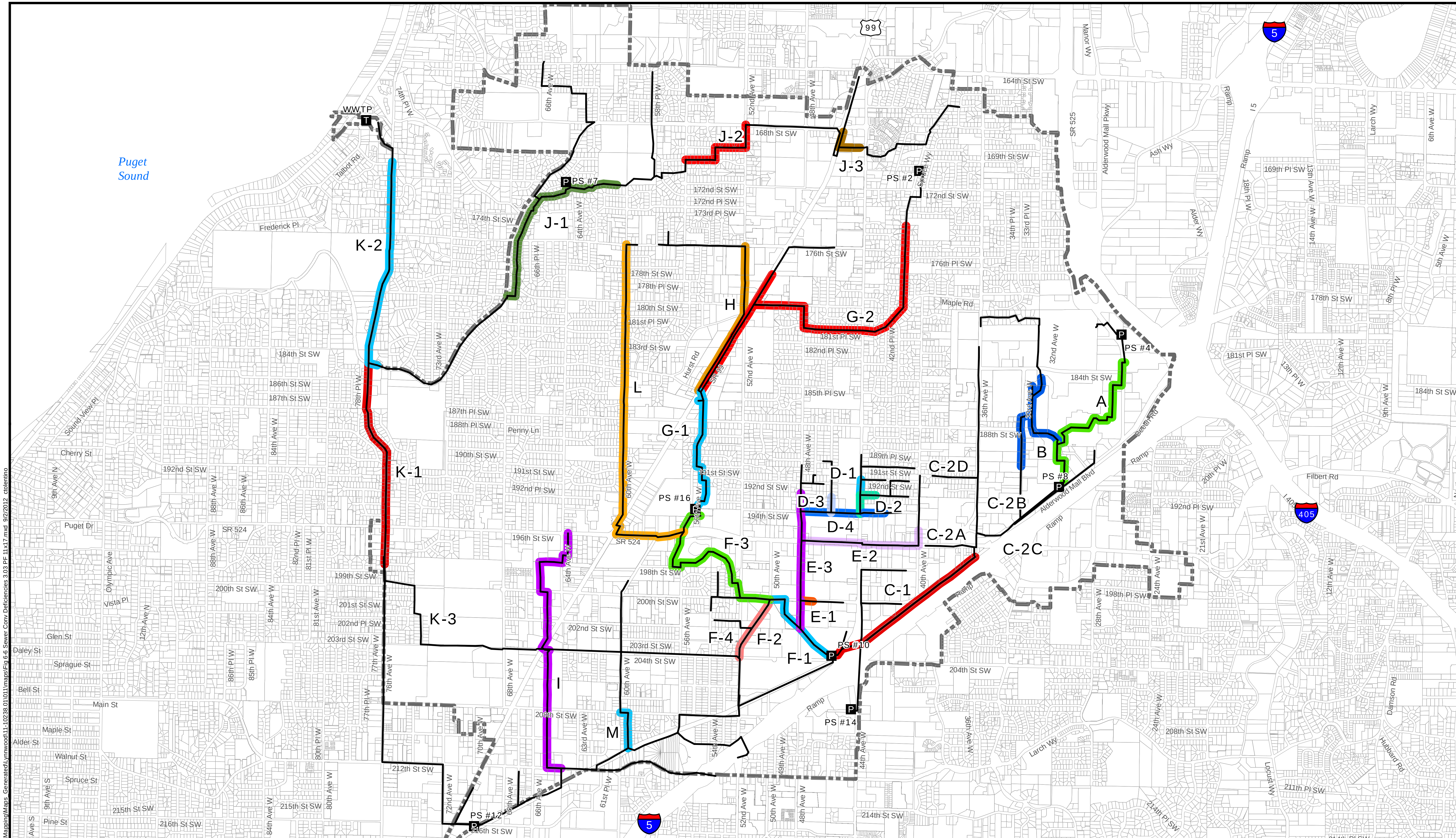
DIURNAL CURVES FROM HYDRAULIC MODEL TO WWTP

City of Lynnwood
2012 Wastewater
Comprehensive Plan Update
August 2012

Figure

6-5



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Sewer System: City of Lynnwood March 2011
GIS Base Data: Snohomish County
This map is a geographic representation based on information available. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.

0 1,000 2,000 Feet
Scale in Feet

NORTH

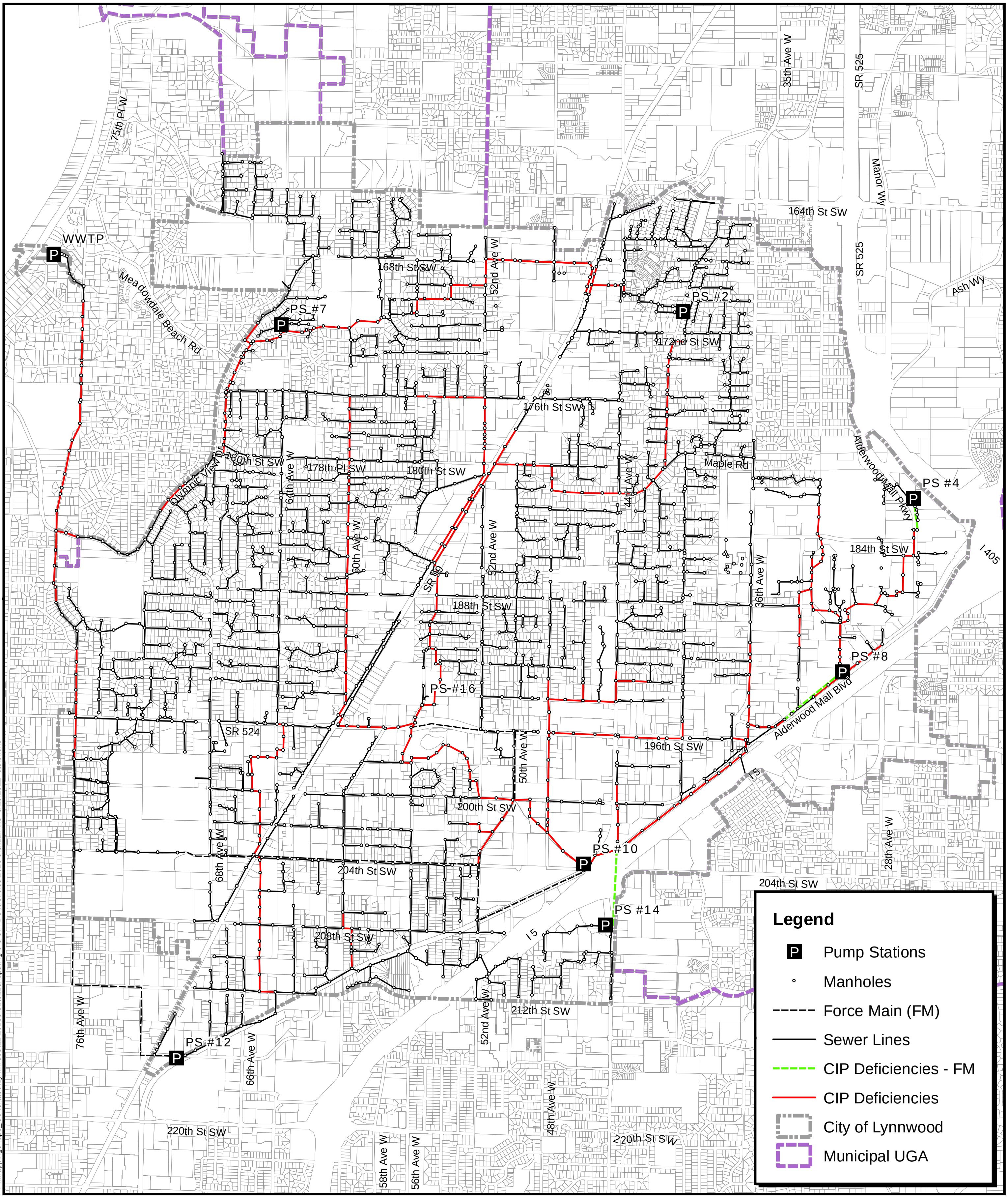
T WWTP
P Pump Stations
City of Lynnwood
Sewer System Truncated

LYNNWOOD WASHINGTON

SEWER CONVEYANCE DEFICIENCIES 3.03 PEAKING FACTOR

City of Lynnwood
2012 Wastewater Comprehensive Plan Update
September 2012

Figure
6-6



P:\Mapping\Maps_Generated\Lynnwood\11-10238.01\011\maps\Fig 6-8 CIP Deficiencies 2018 8.5x11.mxd 8/16/2012 ctolentino

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NORTH

Feet
0 1,250 2,500

CIP IDENTIFIED DEFICIENCIES (2018, 3.03 PF)

City of Lynnwood
2012 Wastewater
Comprehensive Plan Update
August 2012

Figure
6-8

Chapter 7 Treatment Facilities

7.1 Existing Treatment Permit Requirements

The Lynwood wastewater treatment plant operates under the terms of NPDES Permit No. WA-002403. The treatment plant effluent requirements established by the permit are a maximum monthly concentration of 25 mg/L. for carbonaceous biochemical oxygen demand ($CBOD_5$), 30 mg/L for total suspended solids (TSS) and 200/ 100 mL for fecal coliform bacteria density. The permit also limits the chlorine residual concentration to 318 µg/L.

Table 7-1 shows the reported plant effluent quality for the period from January, 2010 through September, 2011. As indicated, the effluent quality is well below the permit limits.

Table 7-1 Effluent Water Quality

Condition	Flow	CBOD		TSS	
	MGD	Mg/L	Pounds/Day	Mg/L	Pounds/Day
Annual Average Day	4.16	8	284	13	437
Maximum Month	5.65	13.4	442	19.3	615
Minimum Month	3.32	6	204	10	322

The NPDES permit also specifies the permitted capacity of the plant. The Lynnwood WWTP is permitted to treat an average day of maximum month flow of 7.4 MGD. The “maximum month” criterion is the highest monthly average loading in one calendar year. The NPDES section S4.A establishes the BOD and TSS loading design criteria for the plant is 15,120 pounds per day for the average day of the maximum month.

In the permit under S4.B: *Plans for Maintaining Adequate Capacity*, section one states that when the influent flow reaches 85 percent of the design flow criteria for three consecutive months the City of Lynnwood must submit a plan and schedule to Ecology showing how capacity will be maintained. This requirement will apply when flows reach 6.29 MGD for three consecutive months. The requirement would also apply when either the BOD or the TSS loads reach 85 percent of permitted capacity or 15,120 pounds per day for each.

Table 7-2 shows the recorded influent from the plants Daily Monitoring Reports (DMRs) from April of 2008 through September of 2011.

Table 7-2 Recorded Influent Data

Condition	Flow	BOD		TSS	
	MGD	Mg/L	Pounds/Day	Mg/L	Pounds/Day
Annual Average Day	4.13	240	7,974	231	7,646
Maximum Month	5.65	299	8,976	284	9,190
Minimum Month	3.32	173	7259	158	6,635

Table 7-2 shows the recent maximum monthly flow and BOD/TSS loads as recorded are below the NPDES level that requires planning for expansion.

Table 7-3 shows the projected influent loads for the flows projected in Table 6-12 as developed for the hydraulic model

Table 7-3 Projected Influent Data

Condition	Flow	BOD		TSS	
	MGD	Mg/L	Pounds/Day	Mg/L	Pounds/Day
2032					
Annual Average Day	5.52	250	11,510	240	11,050
Average Day Max Month	7.05	300	17,640	290	17,050
2040					
Annual Average Day	6.1	250	12,720	240	12,210
Average Day Max Month	7.7	300	19,270	290	18,620

Table 7-3 show projections for 2032 flows for the average day of the maximum month will be less than the 7.4 MGD allowed under the current NPDES permit; however, BOD and TSS loads are projected to exceed the 15,120 pounds per day allowed by that permit for the average day of the maximum month.

Projection for the year 2040 average day of maximum month will exceed all three permit parameters.

7.2 Description of Treatment Facilities

Figure 7-1 shows the site plan of the existing wastewater treatment facilities. The site is also illustrated by aerial photography in Figure 7-2. Table 7-4 summarizes the recorded dimensions and capacities of the existing treatment components in relation to established criteria projected for 2032 conditions.

Table 7-4 Treatment Facilities Projected for 2032 Conditions

WWTP Plant Design Record	Design	Metcalf and Eddy		DOE Orange Book
Component		Typic al	Range	Range
Flow, MGD				
Average Annual	5.52			
Maximum Month	7.05			
BOD_5, lbs/day				
Average Annual	11,510			
Maximum Month Average Day	17,640			

Table 7-4 Treatment Facilities Projected for 2032 Conditions				
WWTP Plant Design Record	**Design**	**Metcalf and Eddy**		**DOE Orange Book**
Component		**Typic al**	**Range**	**Range**
TSS, lbs/day				
Average Annual	11,050			
Maximum Month Average Day	17,050			
Rectangular Primary Clarifiers				
Number, each	3			
Straight Length, feet	105			
width, feet	16			
Side water depth, feet	8.5			
Settling Area each, square feet	1,680			
Volume/unit, gallon	106,822			
Weir length/unit	179			
sludge collector size				
length, feet	110			
width, feet	15.67			
screw conveyor size				
length, feet	16			
diameter, inches	18			
Hydraulic Loading/unit, MGD				
@ design average flow	1.47			
@ design maximum month	1.88			
@ peak day flow	4.44			
Surface loading rate/unit,		Table 5-20		
@ design average flow	874	1200	800-1200	800-1200
@ design max month flow	1,116			
@ peak day flow	2,145	2500	2000-3000	2000-3000
Detention Time/unit, hour				
@ design average flow	1.75			
@ design max month flow	1.37	2	1.5-2.5	
@ peak day flow	0.58			
Weir loading rate/unit, GPD/LF				
@ design average flow	8,204			
@ design max month flow	10,478	20,00	10,000-	20,000
@ peak day flow	24,821			
Circular Primary Clarifiers				
Number, each	1			
diameter, feet	45	40 -	10 - 200	

Table 7-4 Treatment Facilities Projected for 2032 Conditions				
WWTP Plant Design Record	**Design**	**Metcalf and Eddy**		**DOE Orange Book**
Component		**Typic al**	**Range**	**Range**
Side water depth, feet	12.5	14	10 - 16	
Settling Area, square feet	1,590			
Volume/unit, gallons	148,716			
sludge collector size diameter,	45			
Weir length/unit, feet	134			
Hydraulic Loading/unit, MGD				
@ design average flow	1.39			
@ design max month flow	1.86			
@ peak day flow	4.42			
Surface loading rate, GPD/SF		Table 5-20		
@ design average flow	874	1200	800-1200	800-1200
@ design max month flow	1,172			
@ peak flow	2,777	2500	2000-3000	2000-3000
Detention Time, hour				
@ design average flow	2.6			
@ design max month flow	1.9			
@ peak day flow	0.8	2	1.5-2.5	
Weir loading rate, GPD/SF				
@ design average flow	10,375			
@ design max month flow	13,913	20,00	10,000-	
@ peak day flow	32,958			
Aeration Basins				
Number, each	3			
Side water depth, feet	24			
Volume each, gallon	309,000			
Number of cells per Basin	4			
Volume of cells, gallon				
Cell No. 1	19,500			
Cell No. 2	19,500			
Cell No. 3	212,500			
Cell No. 4	57,500			
Total Detention Time, hour				
@ design average flow	4.0	Table 8-16		
@ design max month flow	3.2		3 - 5	
@ peak day flow	1.3			
SRT, days				
@ design average flow	4.2			

Table 7-4 Treatment Facilities Projected for 2032 Conditions				
WWTP Plant Design Record	**Design**	**Metcalf and Eddy**		**DOE Orange Book**
Component		**Typic al**	**Range**	**Range**
@ design max month flow	2.5		3-15 days	
@ peak day flow	1.8			
F:M Ratio			0.2 - 0.6	
Average Annual	0.42			
Maximum Month	0.64			
Peak Day	0.71			
MLSS Concentration, mg/L		2500	1,500-	
Secondary Clarifiers				
Number, each	4			
Length, feet	120			
Width, feet	24			
Side water depth, feet	14			
Area each, square feet	2,880			
Volume/unit, gallons	301,615			
Weir length / unit, feet	320			
Surface loading rate/unit,			**Table 8-7**	
@ design average flow	479		400-700	
@ design max month flow	612			
@ peak day flow	1,450		1,000-	
Solids loading rate/unit,				
@ design average flow	1.7		0.8 - 1.2	
@ design max month flow	2.1			
@ peak day flow	5.0		1.6	
Chlorine Contact Tank				
Length, feet	56			
Width, feet	42			
Side water depth, feet	20			
Volume, gallons	326,000			
Detention Time, minutes				
@ design average flow	85		30 to 120	60-120
@ design max month flow	67			
@ peak day flow	28		15 to 90	> 20
Number of chlorine containers	6			
Chlorine container size, ton	1			
Chlorine dosage, mg/L	5		5 - 15	

Table 7-4 Treatment Facilities Projected for 2032 Conditions				
WWTP Plant Design Record	Design	Metcalf and Eddy		DOE Orange Book
Component		Typic al	Range	Range
Gravity Thickener				
Number, each	1			
Settling area, sq feet	450			
Side water depth, feet	11			
Volume, gallons	37,000			
Surface loading rate,				
@ design average flow	186			
@ design max month flow	270			
@ peak day flow	204		380 - 760	
Solids loading rate,				
@ design average flow	14		Table 14-	
@ design max month flow	21		20-30	
@ peak day flow	16			
Underflow solids, concentration	3.95%		5 - 10%	
WAS Pre-Concentration Tank				
Number, each	1			
Settling Area, square feet	400			
Side water depth	15			
Volume, gallon	44,883			
Surface loading rate,				
@ design average flow	210			
@ design max month flow	339			
@ peak day flow	552		100 - 200	
Solids loading rate,				
@ design average flow	14		Table 14-	
@ design max month flow	22		4 - 8	
@ peak day flow	36			
Underflow Solids, Concentration	1.75%		2 - 3	
Centrifuge				
Number, each	2			
Capacity per unit, pounds/hour	940			
Hydraulic loading, GPM (24-hr				
@ design average flow	38			
@ design max month flow	60			
@ peak day flow	80			
Solids loading, pounds/hr (24-hr				

Table 7-4 Treatment Facilities Projected for 2032 Conditions				
WWTP Plant Design Record	**Design**	**Metcalf and Eddy**		**DOE Orange Book**
Component		**Typic al**	**Range**	**Range**
Solids Loading, pounds/hour				
@ design average flow	495			
@ design max month flow	755			
@ peak day flow	888			
Polymer System				
Number, each	2			
Tank size	5' DIA. X 6' H			
Tank volume, gallon	1,200			
Aging concentration	0.50%			
Feed concentration	0.25%			
Dry polymer feeder Capacity,	200			
Polymer metering pump rating	3 GPM @ 25' TDH			
Fluid Bed Incineration System				
Number, each	1			
Capacity, pounds/hour D.S.	860			
Diameter, feet	9.5			
Solids loading, pounds/day				
@ design average flow	11,277			
@ design max month flow	18,116			
@ peak day flow	21,321			
Feed solids conc., % TS	28			
Feed VS content, BTU/pound	10,000			
Feed sludge fuel value, 10^6				
@ design average flow	71.7			
@ design max month flow	97.9			
@ peak flow				
Fluidizing air blower rating	1,900 SCFM @ 5			
Atomizing blower rating	700 SCFM @ 1.5			
Preheat blower rating, SCFM	2,000			
Preheater (heat exchanger)	1.9×10^6			
Preheater Temperature, ^oF	1,000			
Venture pressure drop, in W.C.	30			
Cooling scrubber size	4 TRAY			
Ash thickener diameter, feet	10			
Ash vacuum filter size, square	3			

A more complete descriptive model of the treatment facilities showing design data contrasting with 2017, 2025 and 2032 projections is contained in Appendix F.

Figure 7-3 shows a schematic of the treatment process. The hydraulic profile of the treatment processes is shown in Figure 7-4. A narrative description of the existing treatment components included in Table 7-4 is summarized below:

<u>Headworks:</u> Wastewater flow enters the facility at the headworks. The influent flow passes through a Parshall flume, then continues through a mechanically cleaned bar screen followed by grit removal equipment. Grit and screenings are removed, washed, compacted, and then disposed of in a dumpster.

The 24-inch throat Parshall flume has a capacity of 21.4 MGD. Because of the high velocity in the influent sewer pipe, there is a high level of turbulence at the flume and the readings are not believed to be accurate. For this reason, the influent flume is not currently being used for flow measurement

<u>Primary Clarifiers</u>: Leaving the headworks, the wastewater flows by gravity to the primary clarifiers. The flow will enter either one of the three rectangular primary clarifiers or a circular primary clarifier. The rectangular primary clarifiers are each 105 feet long by 16 feet wide and have a side water depth of 16 feet. The surface area of each rectangular clarifier is 1,680 SF, for a total of 5,040 SF.

The one circular primary clarifier has a diameter of 45 feet and a side water depth of 12 ½ feet. The total surface area of the circular clarifier is 1,590 SF, which is about 24 percent of the total primary clarifier surface area.
The primary clarifiers remove settleable solids (and the associated BOD) by slowing the flow in the long tanks. The flow velocity dissipates here, allowing solids to settle out of suspension and scum to rise to the surface. All primary sludge and scum is collected for separate processing. The effluent discharges from the clarifiers and flows by gravity to the main plant lift station. Currently the primary clarifiers are performing very well and remove on average 60 percent of the suspended solids and 55 percent of the influent BOD. A typical assumption for primary removal efficiencies is 50 percent for suspended solids and 30 percent for BOD.

Design loading for primary clarifiers is primarily based on the surface overflow rate (flow rate divided by the settling area; or on the vertical upward flow velocity which opposes downward vertical settling velocity). The primary clarifiers flow consists of about 95 percent plant influent and 5 percent return flows from the thickeners and centrifuge processes. The typical design parameter for a primary clarifier is between 800 to 1,200 gallons per day per square foot for average annual flows.

Lynnwood's primary clarifiers were originally designed for an average annual flow of 815 gallons per day per square foot, not including return flows. Assuming 5 percent increase due to return flows would make Lynnwood's clarifier's design capacity 855 gallons per day per square foot, which is relatively low and likely contributes to their excellent performance.

<u>Main Plant Lift Station:</u> The plant was originally constructed to provide only primary wastewater treatment. When the secondary treatment was added there were limited site options, so the secondary components were built hydraulically higher than the existing primary treatment. For this reason, the main plant lift station pumps the primary effluent and gravity fed

return activated sludge into the aeration basins at the beginning of the secondary treatment. There are four 75-hp dry-pit, non-clog centrifugal pumps with a capacity of 4,600 GPM each at a TDH of 48 feet.

Aeration Basins: There are three 309,000 gallon aeration basins for the secondary biological treatment process. Each of the basins is comprises four cells. The first two cells (No. 1 & 2) are small anoxic cells known as "selectors" because they encourage the growth of particular bacteria over undesired filamentous bacteria that would cause "bulking". By keeping the cells anoxic (no free oxygen) non-filamentous bacteria predominate over filamentous bacteria. Non-filamentous bacteria can then settle and compact properly in the secondary clarifiers.

The second two cells (No. 3 & 4) are larger and aerobic (dissolved oxygen ≥ 1.0 mg/L.) for carbonaceous oxidation. Air is blown through submerged piping and through air diffusers to mix and provide oxygen to the activated sludge. Three 200-hp multi stage centrifugal aeration blowers, with a capacity of 2,500 CFM at 8 psig, supply air to all thee basins.

The solids retention time (SRT – or sludge age expressed in days) in the aeration basins is the average time that the solids remain in the basin, which is computed as the total mass divided by the amount grown (and wasted) per day. The aeration basins at Lynnwood are operated within the normal range for conventional activated sludge plants of 3 to 5 days.

Currently there are no nitrogen effluent limits (ammonia nitrogen or total nitrogen) for Lynnwood and the aeration basins are being operated to prevent nitrification, which can begin at an SRT of about 8 days in warmer weather or at about 12 days in winter.

The aerated mixed liquor has the suspended solids concentration controlled to maximize plant performance through seasonal variations in the wastewater and climate. The mixed liquor suspended solids (MLSS) concentration averages about 2,000 mg/L in winter and 2,500 mg/L in summer. The MLSS concentration is limited by the solids settling ability in the subsequent secondary clarifiers.

If ammonia-nitrogen removal becomes a requirement in the future, the volume of the aeration basins, or the solids concentration in the basins, will need to be increased to about three times their current values. Since expanding the aeration basin size is not practical at the limited Lynnwood site, increasing the solids concentration is the only practical choice.

Converting the plant to a membrane bioreactor (MBR) plant, which can be operated at a MLSS concentration of 8,000 to 12,000 mg/L., is the logical choice. With the MBR process the liquid and solids are separated by a membrane, not by gravity settling, so the need for secondary clarifiers is eliminated. In addition to allowing for higher MLSS concentrations in the aeration basins, the elimination of the secondary clarifiers would free up significant space for future expansions.

Secondary Clarifiers: The aeration basin effluent flows by gravity to one of the four rectangular secondary clarifiers. Each clarifier is 120 feet long by 24 feet wide with a side water depth of 14 feet. Activated sludge settles and collects along the bottom where it is either returned to the aeration basins or wasted to the incinerator. The effluent that leaves the clarifier effluent weirs and continues by gravity to the chlorine contact tanks.

Chlorine Gas Disinfection System: There is a 30-inch diameter ultrasonic flow meter in the pipe between the secondary clarifiers and the chlorine contact tank. This meter is used to control the chlorine dose rate and record the effluent flow.

The Lynnwood plant disinfects with gaseous chlorine, which is delivered to the plant in one-ton gas cylinders. The gas is stored on site in three one-ton cylinders hooked up to a single common header that feeds the system. There are three other containers on site but those are generally empty and swapped out when the new chlorine delivery arrives. The chlorine gas is converted to a chlorine solution using chlorinators, and is then mixed with the secondary clarifier effluent in a flash mix compartment where the chlorine is rapidly mixed with the effluent.

The chlorine contact tank is divided into two halves. Each tank half is 56 feet long x 24 feet wide x 20 feet water depth. The total volume is 326,000 gallons, which provides a current detention time of 77 minutes at maximum month flow. A minimum contact time of about 60 minutes at average daily flow rates is required to achieve the necessary fecal coliform kills. The effluent from the chlorine contact tank is de-chlorinated, using an ORP-probe controlled sodium bisulfite feed, before being discharged to the 36-inch diameter outfall.

Outfall: Treated effluent is discharged through a 36-inch diameter 240 foot long x 130 foot deep diffuser that disperses the flow through multiple 3 to 4 inch ports into Browns Bay of Puget Sound.

7.3 Liquid Stream Evaluation

This section evaluates the liquid stream of the wastewater treatment plant for ability to meet treatment objectives over the planning period. The Lynnwood treatment plant liquid stream consists of the following major components:

- headworks
- four primary clarifiers (1 circular + 3 rectangular)
- three conventional activated sludge aeration basins
- four rectangular secondary clarifiers
- chlorine gas disinfection with detention in a chlorine contact tank prior to discharge.

A capacity analysis has been conducted as part of this plan to predict what equipment or processes will be exceeded in their ability to handle plant loading during the planning period. The capacity analysis is presented in Table 7-4.

Capacity of the plant's unit processes was evaluated against common design values, as shown in literature that is widely consulted in the wastewater engineering field, including the following:

- "*Wastewater Engineering: Treatment and Reuse",* by Metcalf & Eddy
- "*Design of Municipal Wastewater Treatment Plants",* prepared jointly by the Water Environment Federation (WEF) and American Society of Civil Engineers (ASCE)
- "*Criteria for Sewage Works Design*" by the Department of Ecology

The plant's flow process was modeled using these sources and calibrated with plant data to reflect the specific design needed for the Lynnwood Wastewater Treatment Plant. Once the model was calibrated, projections for future flows and loads were made based on predicted population growth for the City of Lynnwood. With these projections, capacities can be evaluated. The recommended capacity for the mechanical equipment was taken from the manufacturers' and the plant design data.

The flow projections calculated for Lynnwood gave a maximum monthly flow of 6.35 MGD in the year 2025 and 7.05 MGD in year 2032. These projected flows are less than the overall rated capacity of the plant (7.4 MGD) and none of the unit processes or equipment capacities are projected to be exceeded during this planning period. Although the projected flows are less than the rated capacity of the plant, they are greater than the 85 percent criteria (6.29 MGD) that would trigger an expansion study. The year 2024 is the likely date when the City of Lynnwood will have to submit a plan and schedule for continuing to maintain capacity.

The projected loadings for the plant do not exceed 85 percent (12,850 pounds/day) of the NPDES permit within the planning period. The projected max month BOD and TSS lbs/day loading for the Lynnwood plant are 17,639 and 17,051 pounds/day, respectively, by the year 2032.

Although the existing liquid stream facilities are performing very well, and have sufficient capacity for future growth, there are two issues that warrant further analysis. These are:

- The impact on aeration capacity of removing the existing circular primary clarifier to make space for additional sludge handling facilities
- Chlorine gas safety issues

Chlorine gas is an effective and cheap disinfectant but the storage of large quantities is heavily regulated because of the potential for devastating hazards due to leaks. Over the years several regulations have been developed to govern the handling of toxic materials, such as gaseous chlorine, including:

o Section 112 (r) of the 1990 Clean Air Act Amendments requires that wastewater facilities that are storing over 2,500 pounds of chlorine gas prepare a risk management plan that lays out accident prevention and emergency response activities. The rule requires that the facility develop a hazard assessment to determine the consequences of a worst case accidental release scenario on the public and the environment.

The Clean Air Act and the Occupational Safety and Health Administration (OSHA), regulation 29 CFR 1910.120, requires that any facility with extremely hazardous substances on site (such as chlorine gas) develop an emergency response plan, and to adequately train their employees. The emergency response plan should address issues such as the following:

> Pre-emergency planning and coordination with first responders
> Personnel roles and lines of authority
> Emergency alerting and response procedures
> Site security and control
> Emergency first aid
> Decontamination
> Evacuation routes and safe distances
> Training on use of personal protective equipment

o The Emergency Planning and Community Right-to-Know Act, which is also known as Title III, Section 302/304 of the Superfund Amendments and Reauthorization Act (SARA) of 1986, requires that any facility that has on its premises more than 100 pounds of chlorine, must report that to state and local emergency planners, along with specific information listing the facility emergency coordinator and spokesperson, a telephone roster to be used in case of an actual emergency, and designating specific evacuation routes and zones.

o Article 80 of the Uniform Fire Code contains provisions for the storage, spill control, containment, ventilation and emergency scrubbing required to neutralize leaking toxic gases, including chlorine. The code established relatively small exempt amounts that may be stored without compliance with these provisions. The interpretation and enforcement of this code varies for one jurisdiction to another, so it is important to check with local fire authorities. In most jurisdictions, no more than four-150 pound cylinders may be stored in the same room or gas enclosure, without the use of a scrubber.

o Lynnwood does not have a chlorine gas scrubber, as required by the Fire Code. Alternatives to correct this situation include providing dual containment enclosures for each active ton cylinder, providing emergency ventilation of the chlorine storage room to a new gas scrubber, or switching to another form of disinfection. Other commonly used forms of disinfection include liquid sodium hypochlorite (bleach) that is purchased as a liquid or generated on site; or ultraviolet light (UV) disinfection.

7.4 Sludge Management

The Lynnwood treatment plant dewaters sludge using centrifuges and incinerates the dewatered sludge using a fluidized bed incinerator. The inert ash from the incinerator, which is less than one cubic yard per day, is hauled by truck to a landfill.

Although the existing incineration system has sufficient capacity for projected future sludge quantities, there are several concerns with the current operation as discussed below:

- New air quality standards and the ability to meet them (year 2016 compliance required)
- Energy utilization, efficiency and sustainability
- Greenhouse gas emissions
- Equipment condition and remaining life of the facilities
- Lack of a viable redundant sludge handling scheme

The existing sludge handling facilities are described below together with an evaluation of their current performance.

<u>Primary Sludge Handling</u>: The sludge removed from the primary clarifiers is pumped to the sludge manhole by four air-operated diaphragm primary sludge pumps that have a capacity of 50 GPM at 45 feet total dynamic head (TDH). In the manhole the sludge is diluted to about 0.5 percent total solids concentration (TS) and pumped by two submersible pumps with a capacity of 206 GPM at 30 feet TDH. The diluted sludge passes through a cyclone grit remover before entering the gravity thickener.

The gravity thickener is a 24 feet diameter tank with a settling area of 450 square feet x about 11 feet deep for a volume of 37,000 gallons. The sludge is pumped out by two progressive cavity pumps with a capacity of 35 GPM at 25 feet TDH. The primary sludge flow is combined with the thickened waste activated sludge in the sludge blend tank.

Typical primary sludge thickeners are designed for a solids loading of 20 to 30 pounds per day per square and a hydraulic load foot of 380 to 760 gallons per day per square foot. Lynnwood’s current maximum month loading for the gravity thickener is 14 pounds/ day/SF and 150 gallons per day per square foot. The thickener is being loaded far below typical design values.

Typically primary sludge in gravity thickeners enters at 2 to 4 percent solids and is thickened to 5 to 10 percent solids. Lynnwood’s primary sludge is entering at about 0.9 percent and is being thickened to less than 4 percent. The gravity thickener was originally designed to take 0.5 percent solids and thicken them to 8 percent solids.

The fact that the thickener is only achieving one-half the design solids concentration means that the primary sludge flow rate to the centrifuge is over twice the original design rate. The poor performance of the gravity thickener may be due to the fact that the influent sludge to the thickener is being diluted for grit removal.

<u>Waste Activated Sludge (WAS):</u> The mixed liquor that leaves the aeration basin settles in the secondary clarifiers. A portion of this is recycled back into the aeration basins as return activated sludge (RAS) and the rest becomes waste activated sludge (WAS) that flows to a pre-concentration tank for thickening. The WAS 45,000 gallon pre-concentration tank has a settling area of 400 square feet and a design underflow solids concentration of about 1.5 percent TS. The thickened WAS flows to a holding tank. From the holding tank the thickened sludge is ground before two progressive cavity pumps with a capacity of 65 GPM and 5 feet TDH to move the WAS to join the thickened primary sludge.

Typical design loading for gravity thickening WAS is 4 to 8 pounds per day per square foot and between 100 to 200 gallons per day per square foot. Lynnwood's tank is currently being loaded to about 8 pounds and 170 gallons per day per square foot. Gravity thickening for activated sludge is not as effective for primary sludge but generally an activated sludge stream of 0.5 to 1.5 percent can be thickened to 2 to 3 percent solids. Lynnwood's WAS, on the average, enters the tank at 0.75 percent and leaves thickened to 1.9 percent TS. The gravity thickener appears to be performing well and captures 93 percent of the solids but is expected to exceed typical loading before 2025.

<u>Centrifuge:</u> Thickened primary and waste activated sludge flows are combined and mixed in an in-line static mixer before entering the centrifuges. Clay and polymer is added to the combined thickened sludge. The polymer is used as a coagulant and the clay is added to react with sodium and potassium chlorides to form higher melting silicates, which prevents the formation of clinkers in the subsequent incinerator.

The facility has two centrifuges. One runs 24 hours per day and each has a solids capacity of 940 pounds per hour and a hydraulic capacity of 29 GPM. The sludge is dewatered in the centrifuges to a sludge cake which is conveyed to the incinerator for final processing.

The centrifuges produce a dewatered sludge with an average of about 26 percent solids concentration and a range of 20 to 32 percent. The need for supplemental fuel at the downstream incinerator is very sensitive to the fuel value of the volatile solids and the solids concentration. Lower solids concentrations result in higher concentrations of water which increases the demand of diesel.

The poor solids thickening may be having an adverse effect on the centrifuges' ability to dewater. The centrifuges are currently being loaded at less than half their solids capacity in terms of pounds per day, but in excess of their hydraulic capacity. The design for the centrifuges was 29 GPM but the centrifuges are actually being loaded at 39 GPM on an average annual basis.

The solids capture is only 86 percent, which is far less than typical values of 95 percent capture. The 9 percent decrease in efficiency means solids lost in the return flow back to the headworks increases from 500 pounds per day to about 1,300 pounds per day. This low solids capture rate increases loading throughout the liquid stream processes.

Fluid Bed Incinerator: From the centrifuge, the dewatered sludge cake is deposited by a screw conveyor into a cake pump with an auger that delivers the dewatered sludge to the fluid bed incinerator. The fats, oils, and greases (FOG) that were collected separately at the primary clarifiers, are fed with the cake into the incinerator at a controlled rate. Large influxes of FOG are not desirable for incineration because their high fuel value makes temperature control more difficult.

Internally the incinerator has a reactor bed of sand that is fluidized by blowing a controlled, pre-heated air supply through it. Here the dewatered sludge cake is fed into the fluidized sand bed where it is burned. In this bed the dewatered sludge is broken up by the sands fluidizing motion and is incinerated by the heat. Diesel fuel is pumped into the reactor as required to aid combustion. The amount of supplemental fuel required depends on the heat content of the combustion air (which is pre-heated by the primary flue gas heat exchanger), and the heat content of the sludge, which is dependent on the solids content of the dewatered sludge (percent volatile solids). When the incinerator is operating at its' design capacity with 27 percent dry solids (75 percent volatile), or greater, the incinerator should theoretically operate autogenously (thermally self-supporting), and supplemental fuel should only be required for start-up.

The incinerated sludge byproducts are gases and ash. The gases and ash exit the top of the incinerator and pass through two heat exchangers. The primary heat exchanger pre-heats the incinerator fluidizing air and the secondary exchanger re-heats the exhaust stack. The gas and ash then passes to a wet Venturi cyclone scrubber for particulate removal, followed by a four-tray wet scrubber where the finer particulate is removed and the gas is cooled by passing it counter-current to a large quantity of water. The scrubbed gas is re-heated (to prevent a visible steam plume) and released through a stack to the atmosphere.

Figure 7-4 shows a schematic flow diagram for the existing incineration system.
The captured ash is thickened in a gravity thickener and dewatered in a vacuum filter. Current operations produce, on average, about 5 cubic yards of ash a week.

Incinerator History / Capacity / Condition: According to WEF, Manual of Practice 30, "Wastewater Solids Incineration Systems" (page 47), Lynnwood installed the first municipal fluidized bed sludge incinerator in North America, in 1962. That small 4.0 foot diameter initial unit was replaced in 1989 with the current 9.5 foot diameter unit, which has a rated capacity of 860 pounds per hour (20,640 pound per day).

The 2006 Wastewater Comprehensive Plan states on page 8-8 that the incinerator was upgraded in 1994 to a capacity of 750 to 800 pounds per hour (18,000 to 19,200 pounds per day). Based on their experience the operators believe that the incinerator now has a maximum sustained capacity of only about 80 percent of its' rated capacity, or about 688 pounds per hour (16,500 lbs/day).

Based on operating records from October 1, 2010 thru September 31, 2011, the incinerator production averaged 7,770 dry pounds of sludge solids per day (324 pounds per hour), so it is being used at less than one-half of its' rated capacity, on average. The highest reported daily production during this period was 16,750 pounds (698 lbs/hr) on October 31, 2010.

As shown on Table 7-5, the projected maximum monthly sludge production, in year 2032, is 12,160 pounds per day (507 lbs/hr), so the incinerator has sufficient capacity under future projected conditions, even at the reduced capacity.

Table 7-5 Projected 2032 Sludge Production
Dry Pounds per Day

Parameter	2010	2017	2012	2032
Annual Average Day	7,703	8,434	9,350	10,417
Average Day Maximum Month	9,715	9,889	10,941	12,160
Peak Day	16,033	17,137	19,934	21.007

Even though the incinerator has sufficient capacity, the question of its' useful life remains. The existing unit is 22 years old, which is about the normal life expected for mechanical components (pumps, blowers, etc.) and well beyond the normal life of instruments and controls. The incinerator steel shell, insulation and fire brick are expected to have a life of at least 30 years if properly maintained and if the temperatures are controlled within acceptable ranges.
The incinerator is taken out of service annually for inspection and is regularly maintained by a company which specializes in servicing incinerators (CH Murphy Co.) during the last four years (2008, 2009, 2010 and 2011).

This level of service is normal and should be expected in the future. Given enough time, all of the incinerator components are replaceable. However, certain components, such as the fire brick, may be difficult to obtain in a timely manner. It is recommended that spares be kept on site for all components that cannot be readily obtained locally.

<u>Air Quality Requirements</u>: Previous regulations for sewage sludge incinerators (SSI) were based on emission limits established in Title 40 Part 503 of the Clean Water Act (CWA). The previous air quality emission limits were as follows:

- Particulate matter emission limit of 0.18 g/m^3 (0.08 grams/cu ft) dry gas at standard temperature and pressure corrected to 12 percent carbon dioxide.
- Beryllium emission of 10 g per 24-hour period.
- Mercury emission limit of 3200 g per 24-hour period.
- Lead, arsenic, cadmium, chromium, and nickel feed cake limits based on ambient air quality and health risk specific concentrations.
- Total Hydrocarbons monthly average concentration of 100 ppm by volume, corrected to 0 percent moisture and 7 percent oxygen.
- Carbon monoxide monthly average concentration of 100 ppm by volume, corrected to 0 percent moisture and 7 percent oxygen.

On February 21, 2011, the EPA published their newest final SSI emission regulations in the Federal Register Title 40 Part 60, under the provisions of the Clean Air Act (CAA), Section 129 "Solid Waste Incineration Units". These new standards supplement the older CWA regulations. The new regulations were declared to be effective on May 20, 2011. The regulations dictate different emission limits for new (or substantially modified) incinerators and for existing sewage sludge incinerators; as well as different limits for multiple hearth incinerators and fluidized bed incinerators.

Existing SSI units will have 5 years to comply from the effective date of the regulations, or until May 19, 2016. The new CAA regulations will require a Title V air permit.

The new emission limits that apply to the Lynnwood Wastewater Treatment Plant are given in Table 7-6 below. The existing SSI regulation is for incinerators that were in operation before the new regulations. New SSI's built, or substantially modified, will be held to more stringent emission limits. Substantial modification includes any revisions to the feed system, reactor, ash system and energy recovery systems; but not the emission control equipment or routine maintenance.

Table 7-6 EPA Emission Limits for Fluidized Bed Incinerators

Pollutant	Units	Existing SSI	New SSI	2005
Cd	mg/dscm @ 7% O_2	0.0016	0.0011	**0.0029**
CO	ppmvd @ 7% O_2	64	27	8.6
HCl	ppmvd @ 7% O_2	0.51	0.24	No test
Hg	mg/dscm @ 7% O_2	0.037	0.0010	**0.0109**
NO_X	ppmvd @ 7% O_2	150	30	No test
Pb	mg/dscm @ 7% O_2	0.0074	0.00062	**0.0028**
PCDD/PCDF, TEQ	ng/dscm @ 7% O_2	0.10	0.0044	No test
PCDD/PCDF, TMB	ng/dscm @ 7% O_2	1.2	0.013	No test
PM	mg/dscm @ 7% O_2	18	9.6	**22**
SO_2	ppmvd @ 7% O_2	15	5.3	-
Where:				
Cd	- Cadmium			
CO	- Carbon Monoxide			
HCl	- Hydrogen Chloride			
Hg	- Mercury			
NO_X	- Nitrogen Oxides			
Pb	- Lead			
PCDD/PCDF	- Polychlorinated Dibenzo -P-Dioxins and Polychlorinated Dibenzofurans			
PM	- Particulate Matter			
SO_2	- Sulfur Dioxide			
TEQ	- Toxic Equivalency			
TMB	- Total Mass Balance			

The "2005" column in Table 7-6 summarizes data received from the plant for their source emissions evaluation by Am Test-Air Quality, LLC, which was conducted in October 12, 2005 and finalized into a report in January 19, 2006. The 2005 evaluation was based on the old regulations, so there are several pollutants that are currently regulated that were not sampled or tested.

In Table 7-6, two pollutants from the 2005 exceed the new regulations for existing SSI's: Cadmium (Cd) and Particulate Matter (PM). Two other pollutants, Mercury and Lead exceed the regulations for new or modified incinerators.

Assuming that this one test is representative of Lynnwood's sludge, improvements to the existing emission control equipment will be required. However, it should be noted that sludge quality can vary two to three fold between samples, and one sample is not representative. A significant sampling and testing program is recommended before any modifications are made to the emission control system.

Particulate matter and most metals (except mercury) are removed as solids by the plants existing Venturi scrubber and impingement tray scrubber. As discussed in WEF, Manual of Practice 30, "Wastewater Solids Incineration Systems" (page 159), "Because this type of scrubber is inefficient in removing small diameter particles, it cannot be used as the sole particulate control devise."

Energy Utilization: From October, 2010 through September, 2011, the incinerator used 30,779 gallons of diesel fuel for an average of 87 gallons per operating day. At $3.50 per gallon, the diesel fuel costs about $108,000 per year. Based on a cursory review of the operating record, the use of supplemental fuel is very sensitive to the solids concentration of the feed sludge. The solids concentration from the centrifuges ranges from 20 to 32 percent, and averages about 26 percent. When the solids concentration exceeds about 30 percent, little or no supplemental fuel is required.

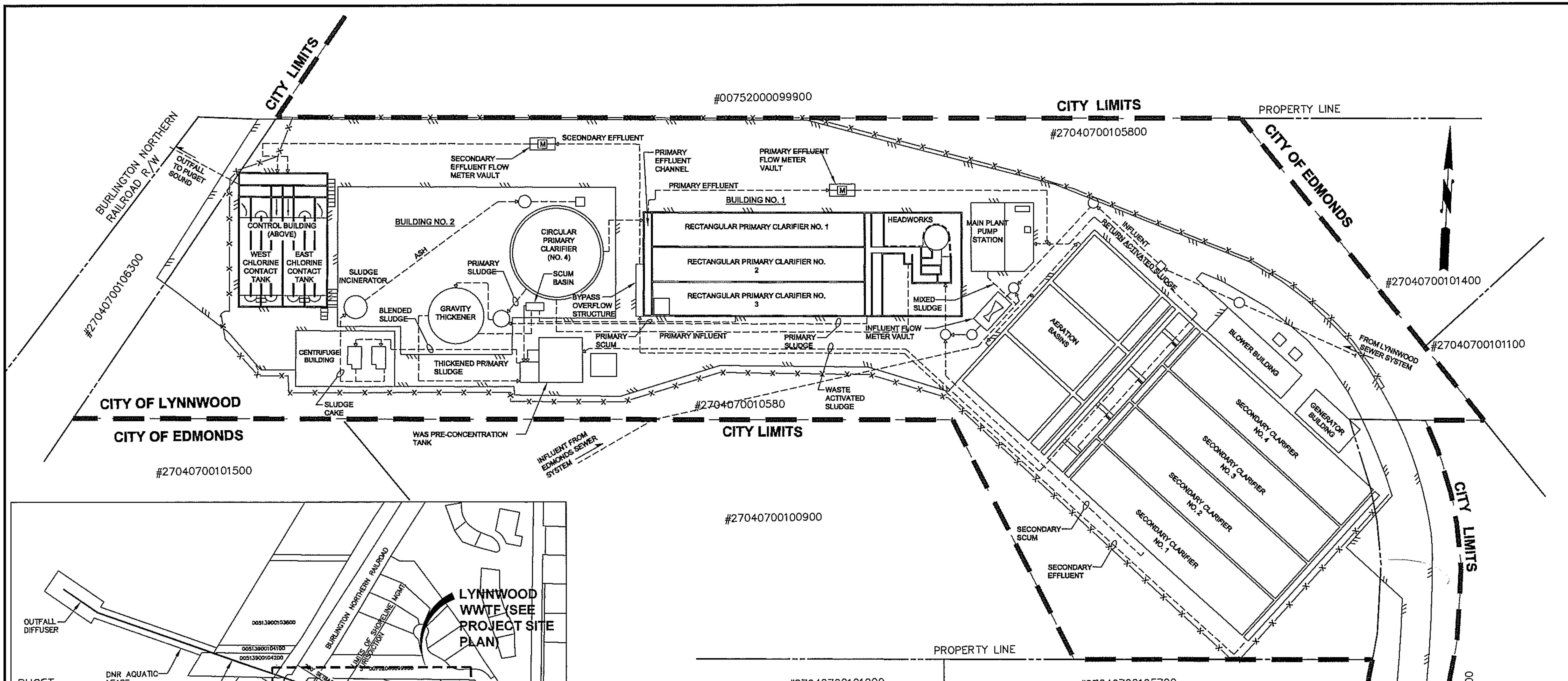
Additional fuel is also likely being consumed because the incinerator is normally operated continuously at substantially less than its' design capacity. Since the air flow and temperature of combustion are fixed by the incinerator design, there is a fixed minimum energy consumption requirement, regardless of the quantity of sludge being fed and there is less fuel value in the sludge solids. Therefore the supplemental fuel use is higher per pound of sludge solids burned when the incinerator is under-loaded.

Greenhouse Gas Emissions: Another emerging issue with respect to incineration is the sustainability of the practice with respect to greenhouse gas (GHG) emissions. Although not currently regulated, the possibility exists that GHG reductions will be required at some point in the future.

Lack of Alternative Sludge Facilities

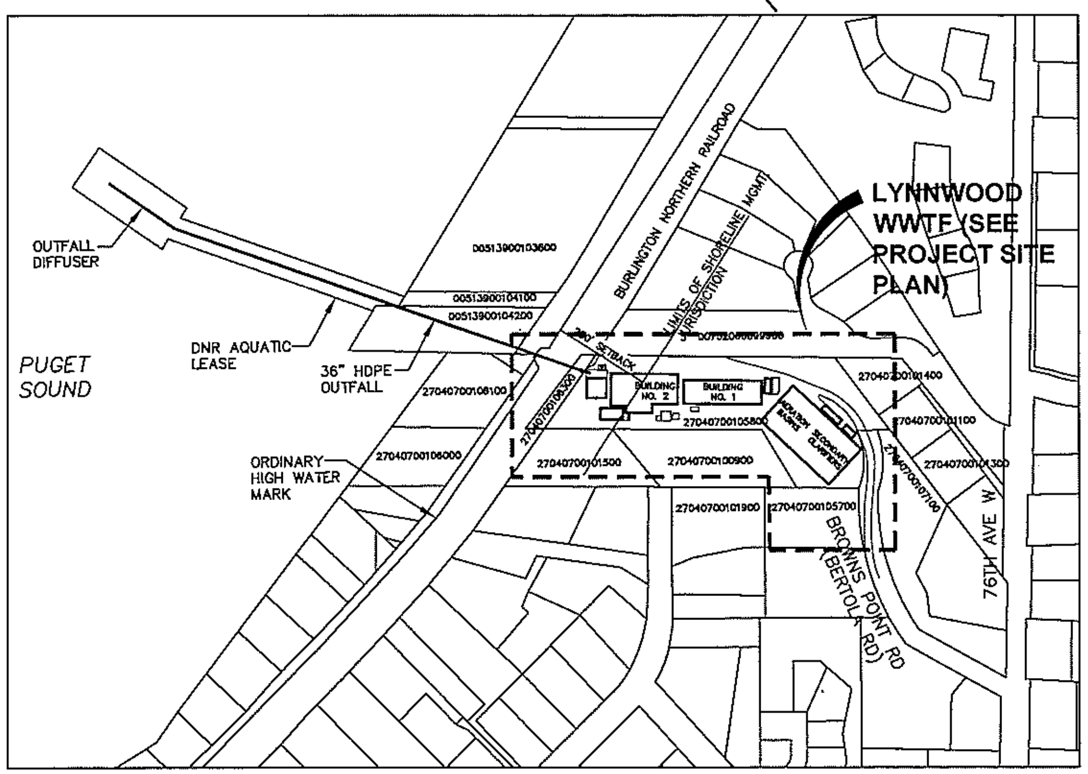
One of the major concerns at the Lynnwood treatment facility is the lack of redundant sludge handling alternatives. In the event of an incinerator system failure (or the end of its functional life) sludge handling alternatives are limited to truck hauling of undigested, liquid sludge. Because the only access road is steep, narrow and winding; and there is no truck turn-around at the site, tanker truck access is very difficult and may be impossible during freezing or snowy conditions.

During annual incinerator maintenance shut-downs (which can be scheduled in advance and normally last about one week during the summer) 6,000 gallon tanker trucks have to back down the access road and extensive truck flagging and traffic control is required. At 3.0 percent solids concentration, each 6,000 gallon tanker truck can haul about 1,500 dry pounds of sludge solids per trip. At a projected future average annual sludge production of over 9,000 pounds per day, about 6 trips per day would be required. This would be very disruptive to the surrounding residential neighborhood over an extended period of time.



EXISTING WWTF SITE PLAN

SCALE: 1"=25'



WWTF VICINITY SITE PLAN

SCALE: 1"=200'

FILE NAME (UPDATED BY) S:\CAD\LYNNWOOD\11-10238 SEWER COMP PLAN\DWGS\F11-10238 FIG SITE PLAN.DWG (GOC)
PLOT DATE & TIME AUG 10 2012 08:35:18
XREFS: Parcels

BHC CONSULTANTS

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1601 Fifth Avenue, Suite 500
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206.505.3400
206.505.3406 (fax)
www.bhcconsultants.com

LYNNWOOD WASHINGTON

Existing Site Plan

2012 Wastewater Comprehensive Plan Update
August 2012

Figure
7-1

BHC Consultants, LLC
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Seattle, Washington 98101

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Water System: Lynnwood March 2011
Intertie Locations: G&O Comp Plan November 2005
This map is a geographic representation based on information available. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.

0 27.5 55 Feet
Scale in Feet

NORTH

Legend

- Parcel
- Railroad
- 2ft Contour

LYNNWOOD
WASHINGTON

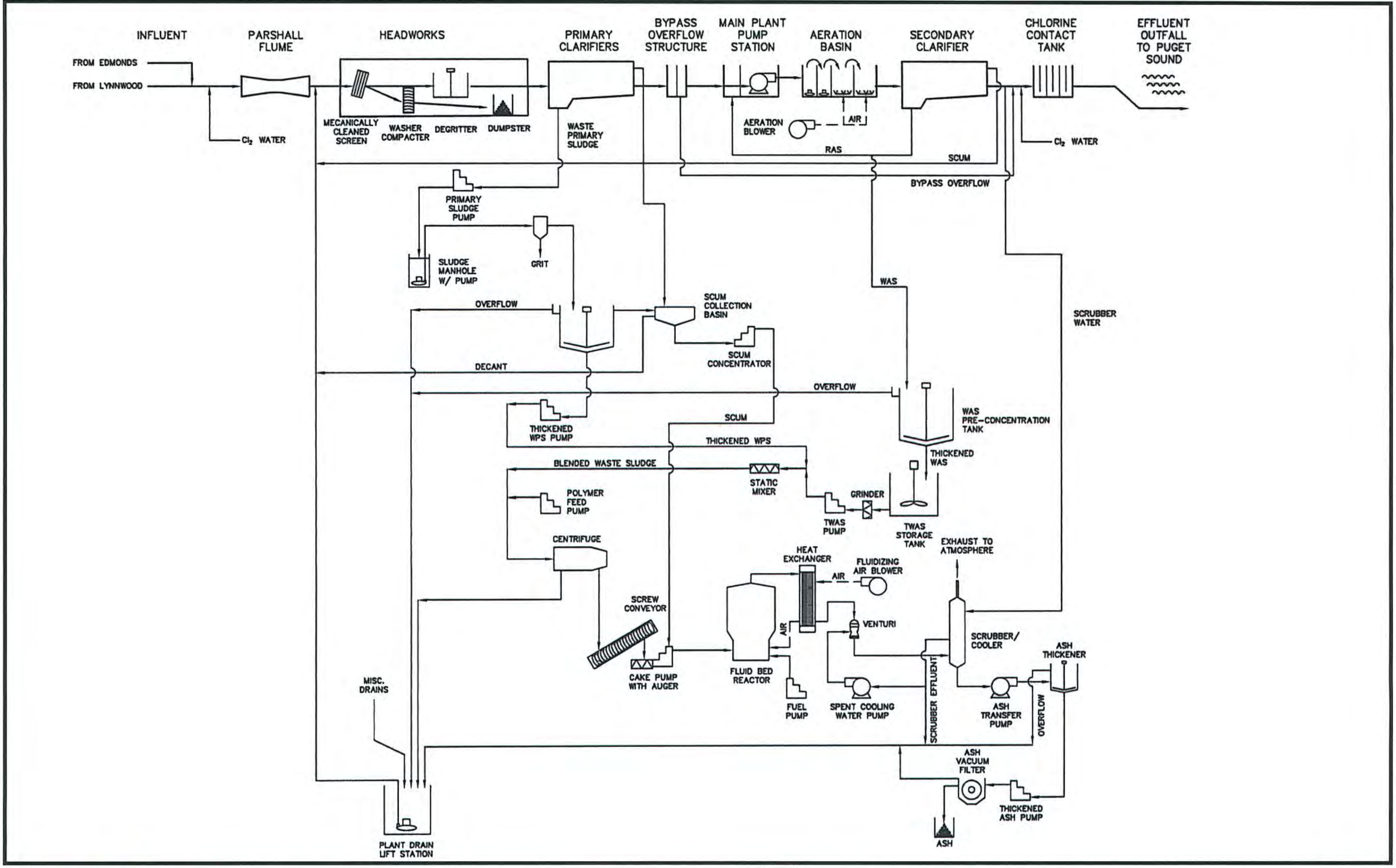
AERIAL PHOTOGRAPHY WWTP

City of Lynnwood
2012 Wastewater
Comprehensive Plan Update
August 2012

Figure
7-2

FILE NAME (UPDATED BY)	PLOT DATE & TIME
S:\CAD\LYNNWOOD\11-10227 WATER COMP PLAN\DWGS\F11-10227 PROCESS SCH.DWG (MTC)	AUG 17 2012 13:19:46

XREFS: NONE



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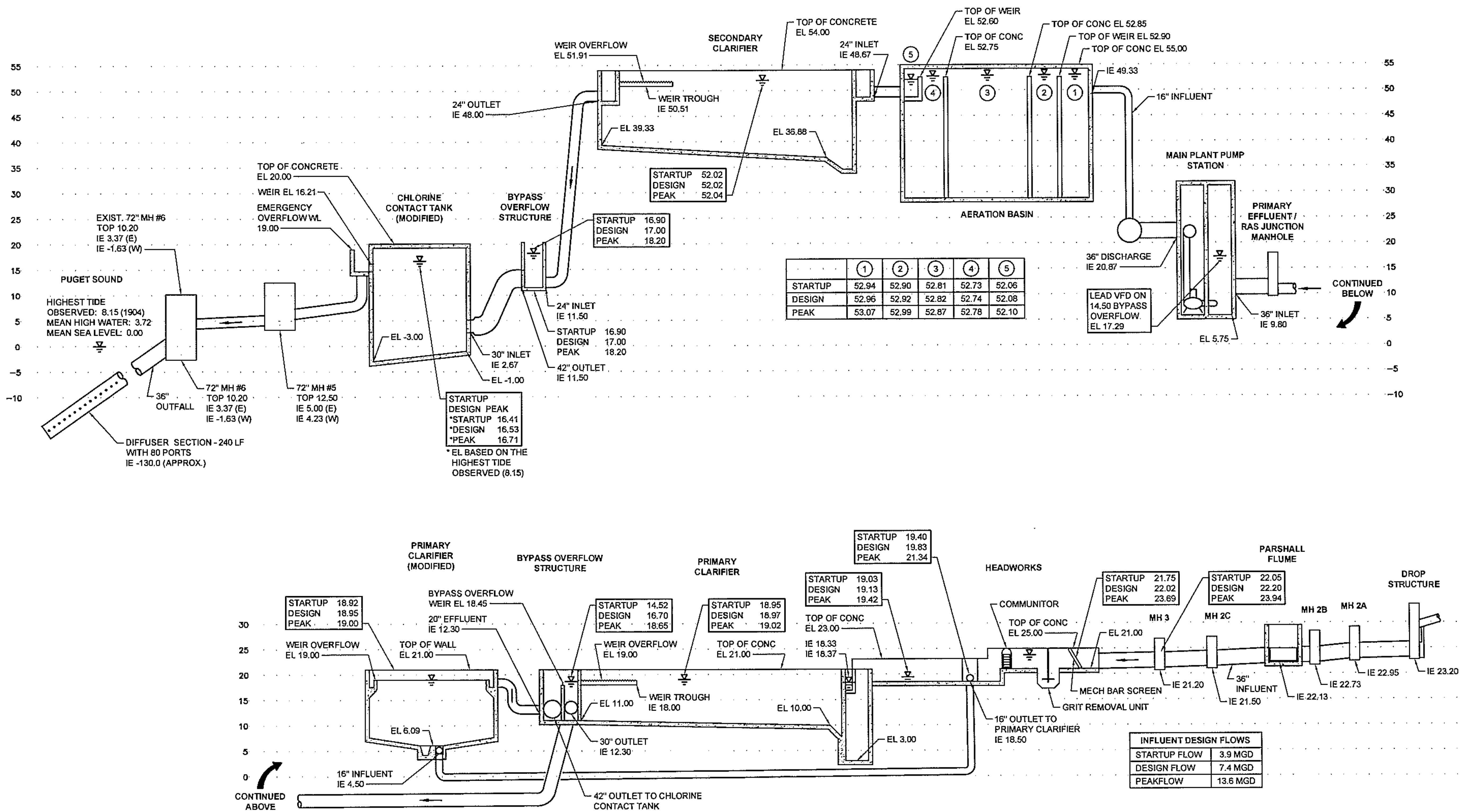
LYNNWOOD WASHINGTON

Process Schematic
City of Lynnwood
2012 Water Comprehensive Plan Update
August 2012

Figure **7-3**



	1	2	3	4	5
STARTUP	52.94	52.90	52.81	52.73	52.06
DESIGN	52.96	52.92	52.82	52.74	52.08
PEAK	53.07	52.99	52.87	52.78	52.10

INFLUENT DESIGN FLOWS	
STARTUP FLOW	3.9 MGD
DESIGN FLOW	7.4 MGD
PEAKFLOW	13.6 MGD

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LYNNWOOD WASHINGTON

Hydraulic Profile

2012 Wastewater Comprehensive Plan Update
August 2012

Figure 7-4

PLOT DATE & TIME AUG 14 2012 13:24:51
FILE NAME (UPDATED BY) S:\CAD\LYNNWOOD\11-10238 SEWER COMP PLAN\DWGS\F11-10238 FIG 7-4 HYDRAULIC PROF.DWG (GDC)
XREFS: NONE

Chapter 8 Alternatives Considered

8.1 Sewer Interception Alternatives

Alternative sewer service facilities need to be compared with the existing sewer facilities and the capacities available in relation to projected future conditions. For sewer interception alternatives, these comparisons start with projections made for the foreseeable future in previous chapters.

Sewer pipes are often considered to have a useful life of about 100 years. Concrete structures like lift stations may be expected to function for about 50 years. However, economic conditions prevalent as this sewer plan was prepared in 2012 suggest population and employment projections beyond about 30 years, or about the year 2040, may be problematic.

Flow diversions would reduce both the flow and the pollutant loadings at the WTTP. However, the City WWTP capacity appears adequate at least through 2032.

The four principal diversion alternatives appear technically feasible are shown conceptually in Figure 8-1 without identifying specific potential pipe alignments:

1. Edmonds – The City of Edmonds has capacity available in their WWTP and would like to capture that portion of their system discharging to the Lynnwood sewer system or divert an equivalent Lynnwood flow into Edmonds. Edmonds LS 4, 12 and 15 are in the vicinity of Lynnwood WWTP. Interception of these stations at the Lynnwood WWTP site is technically feasible.

   To intercept that portion of Edmonds flowing to the City of Lynnwood for treatment requires a new sewer force main routed south in 76th Avenue about 13,000 feet to the vicinity of Edmonds sewers. However the Edmonds Sewer Plan shows only a 10-inch existing interceptor available. Therefore, about 12,000 feet of additional pipe to the west would be needed to reach the Edmonds WWTP vicinity. The static lift would be about 450 feet and the total dynamic head (TDH) would depend of the force mains size, or sizes, depending on how much of the Edmonds flow is intercepted.

   It is also possible to divert part of the Lynnwood wastewater flow in the 76th Avenue sewer trunk to Edmonds for treatment. This variation would still require a new pipeline to the Edmonds WWTP unless the diversion was limited to just the capacity available in the Edmonds piping. And while adequate capacity exists in the 76th Avenue trunk, sewer improvements in Lynnwood east of the trunk would still be required.

2. King County Metro – The Swamp Creek Trunk of the Metro interceptor system flows south just east of Lynnwood near SR-525 and I-405. The new piping to reach the Metro system has been estimated as follows:
   Lift Station No. 4 is less than 500 feet to the west
   Lift Station No. 8 is about 3,000 feet to the west
   Lift Station No. 10 is about 8,000 feet to the west

   Any connection to the Metro wastewater system commits Lynnwood to the Metro rate structure however, which currently charges about $38 per ERU and has been documented by others as one of the highest cost systems in the US.

3. Alderwood – That portion of the District service area near Lynnwood LS 4, 8, 10, or 14 is tributary to Metro. Discharge from any of these stations could be diverted to Alderwood. Connection to Alderwood in this vicinity offers no apparent advantage to Lynnwood over a direct connection by Lynnwood to Metro.

   Diversion of a portion of the northern Lynnwood sewer service area to Alderwood for treatment at the District WWTP would require at least one new City lift station and would not directly address capacity concerns for the existing lift stations.

4. Mountlake Terrace – Lynnwood LS 12 is across I-5 from Mountlake Terrace and presently has an overflow connection to the existing Mountlake Terrace 12-inch interceptor. This interceptor discharges to the Metro system and permanent connection involves the Metro cost issues mentioned above plus a wheeling cost to Mountlake Terrace.

None of the sewer diversion alternatives appears sufficiently attractive to justify further analysis.

8.2 Satellite Treatment Alternative

An intercepting satellite WWTP could be built at the City Golf Course as shown on Figure 8-2. The distance is about 12,000 feet from the WWTP to the Golf Course, where the highest elevation is about 440 feet above sea level. The force main from Lift Station No. 10 runs west in 204th Street SW (the southern edge of the golf course); discharges into the 204th gravity trunk and thence to the 76th Avenue West trunk flowing north to the WWTP. This discharge could be diverted into a membrane bioreactor treatment facility at the Golf Course which could produce Class A reclaimed water. The water reclaimed could irrigate the Course, either by surface spray systems or by subsurface drip irrigation. Liquid sludge produced from the membrane treatment process could be discharged to the 204th trunk for conveyance via the 76th trunk to the WWTP and the sludge management process.

Subsurface irrigation appears technically feasible year-round, though permitting could be challenging. The potential quantity that can be applied is unknown, and would depend on adequate geotechnical investigations. However, summer irrigation of 18-hole golf course can require about 1,000,000 gallons daily. If soils permit irrigation to continue year-round, then a significant portion of the average day, maximum month flow during winter wet weather could be removed from the existing treatment process. Reclaimed water treatment facilities do require redundancy and reliability components. In the event of failure of the membrane process, these requirements could probably be satisfied by resuming discharge to the 2014th trunk.

Lacking a significant year-round demand for reclaimed water, reclaimed water facilities are only seasonal and are not usually economically viable.

8.3 Improvements in Process

Several lift station improvements were identified through previous planning efforts and are now in the process of being implemented for completion prior to 2018 as summarized in Table 8-1.

Table 8-1 Lift Station Improvements to be Completed Before 2018		
Lift Station	**Improvement Description**	**Pumps to be Installed**
Lift Station No. 4	Upgrade by developer	3 x 40 horsepower
Lift Station No. 8	Upgrade by City	2 x 75 horsepower
Lift Station No. 16	New station	3 x 100 horsepower

These improvements were included in the hydraulic models of 2032 and 2018 conditions.

8.4 Conveyance Facilities Required

Hydraulic model results for the year 2010 identified several pipes flowing full and surcharging under peak hour conditions. Those pipe reaches with sufficient surcharge to qualify as deficiencies are shown in Figure 8-3. Improving capacity in these pipe trunks is the highest conveyance priority for the six year capital improvement program. The added capacity needed for these pipe reaches as projected through 2032 are summarized in Table 8-2.

Table 8-2 Conveyance Improvement Alternatives for 2032					
Trunk	**Exist Inches**	**Pipe Footage**	**Inches for 2032**	**Est. Construct Cost**	**Est. Project Cost**
B-1	12	622	15	$ 239,000	$ 311,000
C-1	12	846	18	$ 328,000	$ 426,000
C-2C	8 to 12	1,357	15 to 18	$ 578,000	$ 751,000
G-1A	10 to 12	2,982	18	$ 1,156,000	$ 1,502,000
G-2A	8 to 10	5,439	10 to 12	$ 1,221,000	$ 1,587,000
I	8	5,762	10 to 12	$ 1,009,000	$ 1,311,000
J	15	2,588	18	$ 1,003,000	$ 1,304,000
K-1	24	1,074	27	$ 582,000	$ 758,000
K-2	24	3,419	30 to 33	$ 2,138,000	$ 2,779,000
L	8	1,885	10	$ 88,000	$ 115,000
Totals		23,149	-----------	$ 7,197,000	$ 9,356,000

Figure 8-4 shows the plan view of Trunk G-1A, which is tributary to the new Lift Station No.16. Results from computer simulation of wastewater hydraulic conditions projected for 2032 on the existing G-1A sewers is shown in Figure 8-5. Surcharging would occur under peak hour conditions for much of the trunk and overflows would result in the upper segments. Figure 8-6 shows the same Trunk G-1A alignment when modeled with the added pipe diameters shown in Table 8-2, which eliminates the overflows though some pipe reaches still would surcharge.

Similar results were produced for the other trunks shown in Appendix A. The hydraulic grade line profile shows where projected flow is contained within the existing pipe, where it surcharges and rises up manholes, and where it rises above the existing ground line to produce an overflow.

Table 8-2 shows the capacity needed in MGD for the stressed pipe reaches under the 2032 peak hour flow condition; the equivalent pipe diameter required to convey that peak hour flow; and the approximate feet of pipe length that should be upgraded to provide the capacity needed.

The equivalent diameter could be achieved by several alternative methods:

- Pipe bursting to achieve the equivalent inches in diameter
- Installation of parallel piping to supplement the existing pipe and achieve the capacity
- Replacing the existing pipe with a new pipe of the equivalent diameter needed.
- Diverting flow through a new pipe in a different direction

One or more of these methodologies may be infeasible for some trunks requiring upgrade. The implementation methodology should be selected through site-specific Project Report evaluating surface and subsurface conditions for potential conflicts and limitations, as well as the non-cost, non-technical issues that may produce unacceptable impacts. For planning purposes, one methodology is proposed for each trunk upgrade as a cost baseline for capital improvement planning.

8.5 Liquid Stream Improvement Alternatives

Aeration Basin Capacity Alternatives: The circular clarifier takes up what could be very useful space that could be used for disinfection and/or sludge handling improvements. Removing the circular clarifier would reduce the total primary clarifier surface area from 6,330 square feet to 5,040 square feet (26 percent decrease), and would increase the future average day surface overflow rate from 751 GPD/SF to 989 GPD/SF, in year 2040. This is still within the commonly accepted 800 to 1,200 GPD/SF design range but would result in some loss of BOD and TSS removal efficiency.

The theoretical maximum month BOD removal efficiency would drop from 42 to 37 percent, which would add additional BOD load to the aeration basins. Based on a mixed liquor concentration of 2,500 mg/L, the maximum month SRT would drop from 3.3 days to 2.7 days in 2040. This is less than the 3.0 days commonly recommended. By about year 2030 there would be less than 3.0 days SRT and no room for error or buffering in the operation of the aeration basins.

If the circular clarifier were removed and operation of the aeration basins became problematic Lynnwood could implement chemically enhanced primary treatment (CEPT) to restore, or even improve, the removals in the remaining primary clarifiers, so as to not overload the secondary process. CEPT can be achieved by adding chemical coagulants such as alum or polyaluminum chloride (PAC). BOD removal in excess of 50 percent can be achieved.

Disinfection Alternatives: If the City of Lynnwood wishes to continue using chlorine gas, providing a containment system for each active chlorine cylinder is possible. As shown in Figure 8-6, there are commercial containment vessels available that can hold a one ton-cylinder of chlorine gas. If the chlorine tank leaks, the containment vessel contains the spill and continues to deliver the gas to the system until the remaining chlorine is spent. The budget price for Chlortainer's™ 2-bolt Ton Container is about $112,000 each. The total installed cost for three ton cylinders, with the load system, is estimated to be about $0.5 to $0.6 million.

Figure 8-6 Chlorine Containment Vessel

Another disinfection alternative is to contain the contents of a leak in the existing chlorine container room and then scrub (neutralize) the chlorine gas prior to discharge to the atmosphere. Wet and dry scrubbers are available (Purafil™) that resemble odor scrubbers and can be located outdoors. This alternative would also require additional room ventilation and ducting. The cost of the 3-ton scrubber alone would be about $ 330,000. The total installed cost of a new ventilation system and scrubber is estimated to be about $0.5 to $0.6 million.

Containment and scrubbing of the chlorine gas would meet the Fire Code requirements, but still requires safety training and emergency response planning in case of an accident during the container loading operation. Emergency response planning will undoubtedly require public involvement at some point in the future, and containment of chlorine gas may not be acceptable to the public because of the close proximity of residences to the plant.

Feeding liquid sodium hypochlorite (bleach) would eliminate the concern about chlorine gas drifting off-site, but not about the potential water quality impacts of a spill. Bleach can be purchased for delivery in tank trucks at 12.5 percent strength. The only installation requirements would two FRP storage tanks of about 8,000 gallons each for storing the liquid bleach and chemical metering pumps for feeding the bleach at a flow-paced rate. Aside from spill containment, the major concern with storing and feeding bleach is that it is extremely corrosive and loses strength with time, which produces chlorine gas. Leaks and gas binding of pump and piping systems have been an issue in the past, but these issues can be dealt with through good design. Another concern with feeding liquid bleach is the possibility of liquid spills with the need to deliver with tanker trucks on the plant's steep and narrow access road. The installed cost of a liquid bleach storage and feed system is estimated to be about $0.3 to $0.4 million.

Onsite sodium hypochlorite generation using salt and DC power is another alternative. This eliminates the problem of strength loss with time, because the bleach can be generated as needed. It also eliminated the possibility of liquid spills, because only salt must be delivered. Several manufacturers make on-site generators which have relatively small footprint of about 4 x 16 feet. In addition, space for salt storage and the FRP storage tanks will be required. The generator would cost about $275,000 alone, and the total installed cost is estimated to be about $0.6 to $0.8 million.

Ultraviolet disinfection (UV) implementation has been attempted for the Lynnwood wastewater facility in the past using a low- pressure UV lamp system located in a new open channel located near the discharge end of the existing secondary clarifiers. Based on a nearly complete design and engineer's estimate, the project was abandoned due to excessive estimated costs.

It is believed that a major factor with the cost of the previously planned installation was the large number of low-pressure UV lamps required and the large open-channel flow structure required to house the lamps. It is possible that considerable capital cost savings can be achieved by using a medium-pressure, in-line UV system located in one-half of the existing chlorine contact tank.

Figure 8-7 shows a concept design for an in-line UV installation using three, 20-inch diameter units. Each unit would have a rated capacity of 6.0 MGD, for a peak flow capacity of 18 MGD and a firm (one standby) capacity of 12 MGD. The units would be designed to meet the discharge requirement of less than 200 fecal coliforms/ 100ml while treating an effluent with less than 15 mg/L of TSS and 65 percent light transmittance.

Each of the three units would contain 12-60 KW medium-pressure lamps, for a total of 36 lamps. The power requirement for these lamps is greater than for low-pressure units, but the number of lamps and space requirements are much less. Based on operating at 6.0 MGD continuously at 7¢ per KWH, the power cost for the medium pressure lamps is estimated to be $36,792 per year, compared with $15,943 per year for the low-pressure units. However, the low-pressure units would require 40 to 26 KW lamps per unit, for a total of 120 lamps. The savings in lamp and quartz sleeve replacement costs is estimated to be about $12,000 per year for the medium-pressure units as compared with the low-pressure units, so the total operating costs are not significantly more.

In the past, concern has been expressed about access to the chlorine contact tank, which is located beneath the existing lab/ office building. However, as shown on Figure 8-7, there is sufficient open area on the north end of the tank to allow for a 4.5 feet wide metal stairway, down to a landing at the level of the UV units. By removing the existing non-bearing baffle walls there is sufficient room to install and access the UV units, which would be piped with valves to allow for either parallel or series operation. Adequate lighting and ventilation would be provided to make this a "non-confined space" working environment.

The budgetary price for three in-line UV units alone is $450,000, including the control system. The installed cost, including demo and modifications to the existing contact tank, piping, valves, new metal access stairs, etc., is estimated to be about $1.4 to $1.7 million.

8.6 Incinerator Upgrades

For planning purposes, it is recommended "that the City plan and budget for adding a wet electrostatic precipitator for added particulate removal". This should insure compliance with the PM and metals standards, except mercury. The estimated cost of an electrostatic precipitator of is about $0.8 to $1.0 million, installed (excluding any required building expansion).

Because of its extremely high vapor pressure, mercury is 100 percent vaporized during the incineration process. With only a wet scrubbing system, mercury is 100 percent emitted in the flue gas. If required, mercury (along with dioxins and furans) can be removed with a static bed carbon adsorber. However, before mercury removal emission control equipment is recommended, a source control program should be implemented (dental amalgam bans).

Total hydrocarbons, carbon monoxide, nitrogen oxides, dioxins and furans can be controlled by closely monitoring the combustion process. Because of the obsolescence of the existing control system and the need to more closely control the combustion process, a new incinerator control system is recommended.

It is estimated that complete replacement of the incineration system (including new air quality equipment) would take 10 to 12 months. The haul costs during a complete replacement would add over $1.0 million to the replacement cost. Based on the potential financial liabilities of a major incinerator failure, it is recommended that Lynnwood develop a second redundant long-term sludge handling alternative. Alternatives that could be considered (in order of decreasing sludge/ ash volume to be trucked from the site) are anaerobic digestion and hauling digested dewatered sludge cake for land application, drying and hauling dry sludge pellets for fertilizer and adding a second incinerator.

A redundant sludge management alternative (beyond hauling undigested liquid sludge) is recommended because of the potential impacts of a major incinerator failure. Alternatives to be considered are anaerobic digestion and hauling digested dewatered sludge cake for land application, drying and hauling dry sludge pellets for fertilizer and adding a second incinerator.

8.7 Liquid Sludge Management

The treatment facilities capable of handling undigested liquid sludge are larger municipalities with excess sludge handling capacity, such as the City of Edmond and King County Metro's Renton plant. Each round trip to Renton would be about 60 miles. At $2.00 per mile for the truck and driver the haul cost would be about $120 per trip. In addition, a tipping fee of about 8 cents per gallon would add another $480 per trip for a total cost of about $600 per trip. Six trips per day would cost about $3,600 per day, or about $108,000 per month.

Anaerobic Digestion: Class B sludge produced through anerobic digestion for land application is the most common sludge handling alternative used for treatment plants of Lynnwood's size. However, digesters take considerable space, which is very limited at Lynnwood. Based on the projected future ultimate sludge production and a solids retention time of 15 days, a digester of about 70 feet diameter x 30 feet deep would be required. There is no space of this size available on the existing site, but sufficient space might be available immediately south of the existing aeration basins on an adjacent land parcel in a small ravine.

One of the advantages of anaerobic digestion is the volatile solids reduction and digester gas produced that can be beneficially used. If the digested sludge is dewatered to 25 percent solids for truck hauling, the volumes would be about 13 cubic yards per day initially and 22 CY/day ultimately. This would require 2 to 3 loads with an 8 CY truck per day.

The estimated cost of a new anaerobic digester, with adjacent support building and support equipment for mixing, gas handling, heating, etc., is $4 to $6 million.

Sludge Drying: A second alternative for providing a redundant sludge handling system would be to add a sludge heat dryer in parallel with the existing incineration system. The dryer could accept undigested dewatered sludge from the existing centrifuge system. Heat dryers produce Class A sludge pellets that have over 90 percent solids content. The pellets have the appearance of dry fertilizer and can be used in the same way for fertilization of landscaping and agricultural land. In some cases the pellets have been sold for about $10 to $20 per ton, and at other Cities the pellets are used for fertilization of parks and golf courses.

One of the primary advantages of the heat drying system is the significant reduction in volume compared with digested, dewatered sludge cake. At 90% solids, the volumes would be about 8 cubic yards per day initially and 13 CY/day ultimately. This would require one or two- 8 CY truck loads per day. While this is more than the existing ash volumes currently being hauled (about 5 CY/week), it is not unreasonable if the truck loading and hauling conditions can be improved.

One of the disadvantages of the dryer is the space required, which must be in close proximity to the existing centrifuges. The projected ultimate sludge production is 13,444 dry pounds per day during the maximum month in year 2040. If dewatered to 25% solids, a dryer with a capacity of 26.9 wet tons per day is required. A twin-screw conductive dryer with a capacity of 30 wet tons per day (such as the Therma-Flite IC 1800 or Komline-Sanderson) would require a space which is about 1,600 sf (40’ x 40’). Space of this size is not currently available near the centrifuges.

A second disadvantage of the dryer is the energy required to operate. Unlike the current incineration process, the dryer does not combust the sludge solids, so it does not gain the heat of combustion. The projected ultimate average sludge production is 9,374 dry pounds per day, or about 18.7 wet tons per day, so the 30 wet ton dryer would operate about 62 percent of the time, or about 5,474 hours per year. A 30 wet ton dryer requires about 2.3 million BTUs per hour of fuel input. Using natural gas at $7.00 per million BTU would cost about $16.10 per hour, or about $88,000 per year.

The cost of a 30 wet ton per day dryer is about $0.75 million. The total estimated cost of the installation, including a new building and materials handling equipment is estimated at $2 to $3 million.

Second Incinerator: A third alternative for providing a redundant sludge handling system would be to add a completely new incineration system in parallel with the existing system. Based on recent quotes by the existing incinerator manufacturer (Infilco Degremont) it is estimated that complete replacement of the incineration system (including new air quality equipment) would cost about $16 to $18 million installed.

8.8 Sludge Management Recommendations

Based on this brief analysis, it is recommended that Lynnwood consider adding a sludge dryer in parallel, or in series, with the existing incinerators, as a redundant sludge management alternative. The dryer would be sized at 30 wet tons per day, which would be sufficient for all of Lynnwood’s future processing needs. The dryer could be operated in parallel as a backup to the incinerators, to be used when the incinerator is shut down for maintenance or repairs.

Alternatively it could be used ahead of and in series with the incinerator, for the mutual benefit of both systems.

If both the dryer and incinerator were operated simultaneously in series, the waste heat from the incinerators could be used to fuel the dryer, and the dry sludge pellets could be fed to the incinerator, to eliminate the need for supplemental fuel at either.

Figure 7-3 shows a schematic flow diagram of the existing sludge thickening and incinerator operation. If the incinerator gas is intercepted between the two existing heat exchangers, about 600 degrees F. of excess heat is available to heat thermal fluid for use at the dryer. This is in excess of the 2.3 million BTUs required.

If the dry sludge pellets are fed to the incinerator, the fuel value will be in excess of the energy needed to evaporate the remaining water, and the end product would be ash, with the volume substantially reduced.

With respect to the space required for installation of the dryer, it is recommended that Lynnwood consider abandoning existing Primary Clarifier No. 4, the existing 45 foot diameter circular clarifier, to make room for the dryer. It is also proposed that the existing primary sludge thickener be replaced with a mechanical thickener to save space and for better performance. Figure 8-8 shows a proposed site plan with the mechanical thickener and dryer located in existing Building No. 2.

Eliminating Primary Clarifier No. 4 would reduce the total primary clarifier surface area from 6,630 to 5,040 SF, and increase the future peak flow surface loading rate to 3,200 GPD/SF. This is in excess of the typical 2,000 to 3,000 GPD/SF range recommended by common design guidelines (Metcalf and Eddy, Table 5-20), and would result in less removal of total suspended (TSS) solids and BOD in the remaining primary clarifiers. It is estimated that the BOD and TSS removals would each be decreased by about 4 percent.

To avoid overloading the downstream secondary aeration facilities, consideration should be given to adding chemical enhancement to the remaining rectangular primary clarifiers to improve their performance. Converting the primaries to chemically enhanced primary treatment (CEPT) would improve the removals in the primaries to the extent required to avoid the need to expand the aeration basins.

8.9 Energy Conservation

It is recommended that the City conduct a performance and energy study of the thickening, dewatering and incineration process to determine if improvements can be made to reduce supplemental fuel use. Improvements could include better thickening, dewatering or more efficient operation of the incinerator. Advances in centrifuge design and control systems, have resulted in improved dewatering over the last 10 years. Operating the incinerator less frequently at full design capacity should also be considered.

Another energy related opportunity that should be explored is heat recovery from the incinerator flue gas. The incinerator operates at a temperature of about 1,500 degrees F. Part of the heat is used to pre-heat the fluidizing air and part is used to re-heat the stack gas in order to avoid a visible steam plume, but the exhaust gas going to the scrubber still has a temperature in excess of 800 degrees F. The temperature is reduced at the Venturi and tray scrubber by the spray water, so there is considerable heat energy being discharged with the drain water and out the exhaust stack.

Alternatives for capturing this lost energy could include use for pre-drying the sludge or for generating electricity using a waste heat generator (organic Rankine-cycle heat-to-power generator). It is recommended that a detailed energy study be conducted to explore if waste heat recovery would be economically beneficial.

8.10 Greenhouse Gas Emissions

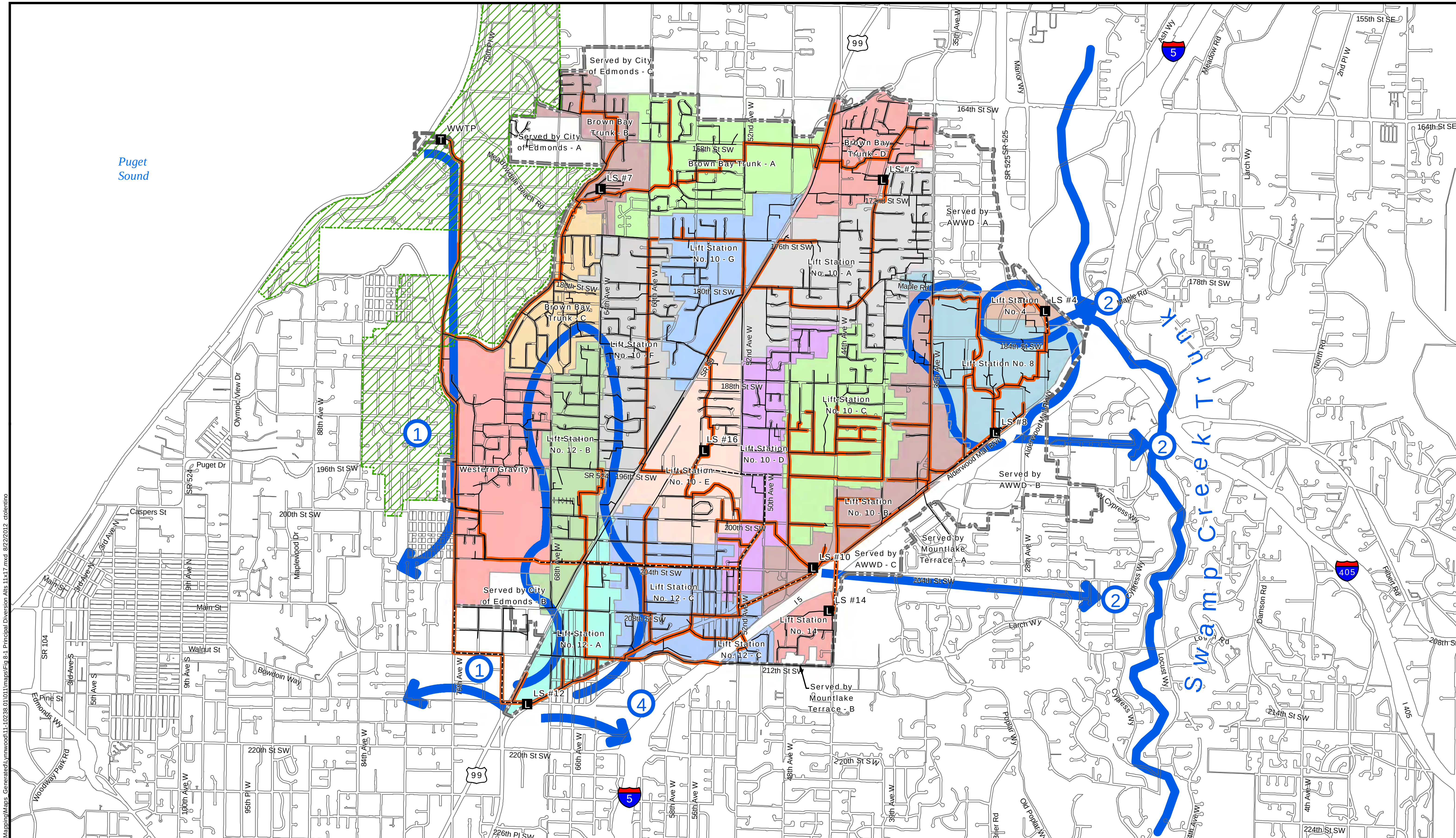
To compare the GHG impacts of sludge handling alternatives, a Biosolids Emission Assessment Model (BEAM) was used to determine the carbon footprint of incineration versus heat drying. Table 8-3, below, summarizes the results from the model. The results are expressed in terms of metric tons of carbon dioxide equivalents for the existing and projected future sludge production.

Table 8-3 CO_2 Equivalents for Sludge Handling Alternatives
Biosolids Emission Assessment Model (BEAM)

Bio-solids Handling Method	Current 8,500 lbs/day TS		Future 14,000 lbs/day TS	
	CO_2 Equivalents	Biomass*	CO_2 Equivalents	Biomass*
	tons/yr	tons/yr	tons/yr	tons/yr
Heat dryer	728	0	1198	0
Incineration	2,335	2,222	3,940	3,694
*Biomass combustion emissions are not included in total CO_2 equivalents				

The model used was developed for the Canadian Council of Ministers of the Environment (CCME) by Sylvis Environmental. The CO_2 equivalents in the table are almost entirely due to diesel fuel usage and N_2O emissions from the incinerator. N_2O emissions would account for over 80 percent of the emissions for the incinerator. This is because a ton of N_2O emitted is equivalent to 320 tons of CO_2.

Table 8-3 gives a good approximation of CO_2 equivalent emissions for each handling method but better serves to show that the incinerator produces far more CO_2 equivalents than the other method. By choosing to dry sludge, Lynnwood could significantly reduce GHG emissions.



P:\Mapping\Maps_Generated\Lynnwood\11-10238.01\0011\maps\Fig 8-1 Principal Diversion Alts 11x17.mxd 8/22/2012 ctolentino

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Sewer System: City of Lynnwood March 2011
GIS Base Data: Snohomish County
This map is a geographic representation based on information available. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.

0 1,250 2,500 Feet
Scale in Feet

NORTH

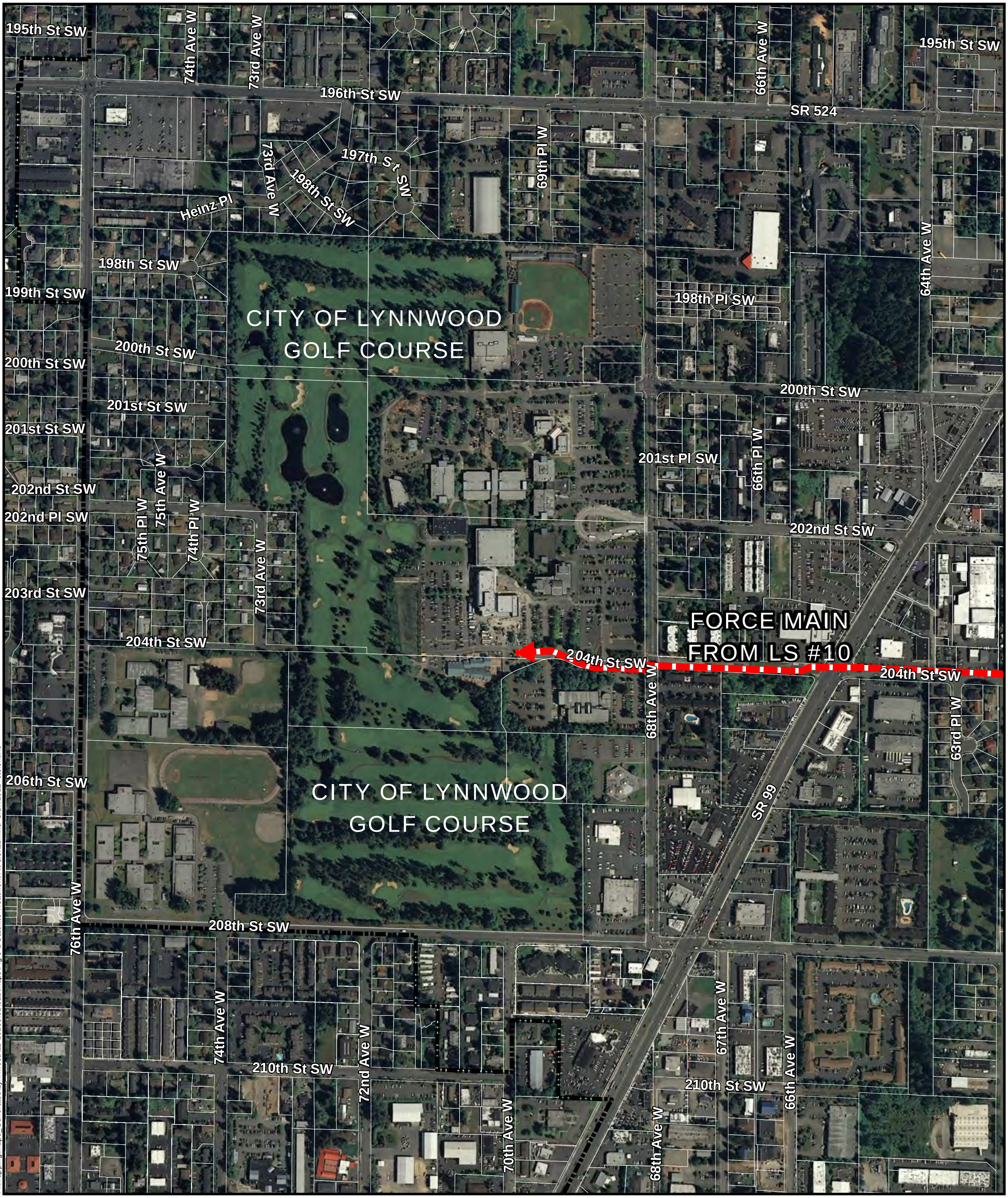
T WWTP
L Lift Stations
Force Main
Sewer Lines
Sewer System Truncated
City of Lynnwood

LYNNWOOD WASHINGTON

PRINCIPAL/DIVERSION ALTERNATIVES

City of Lynnwood
2012 Wastewater Comprehensive Plan Update
August 2012

Figure
8-1



P:\Mapping\Maps_Generated\Lynnwood\11-10238.01\011\maps\Fig 8-2 Satellite Treatment Alt 8.5x11.mxd 8/17/2012 ctolentino

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NORTH

Feet
0 300 600

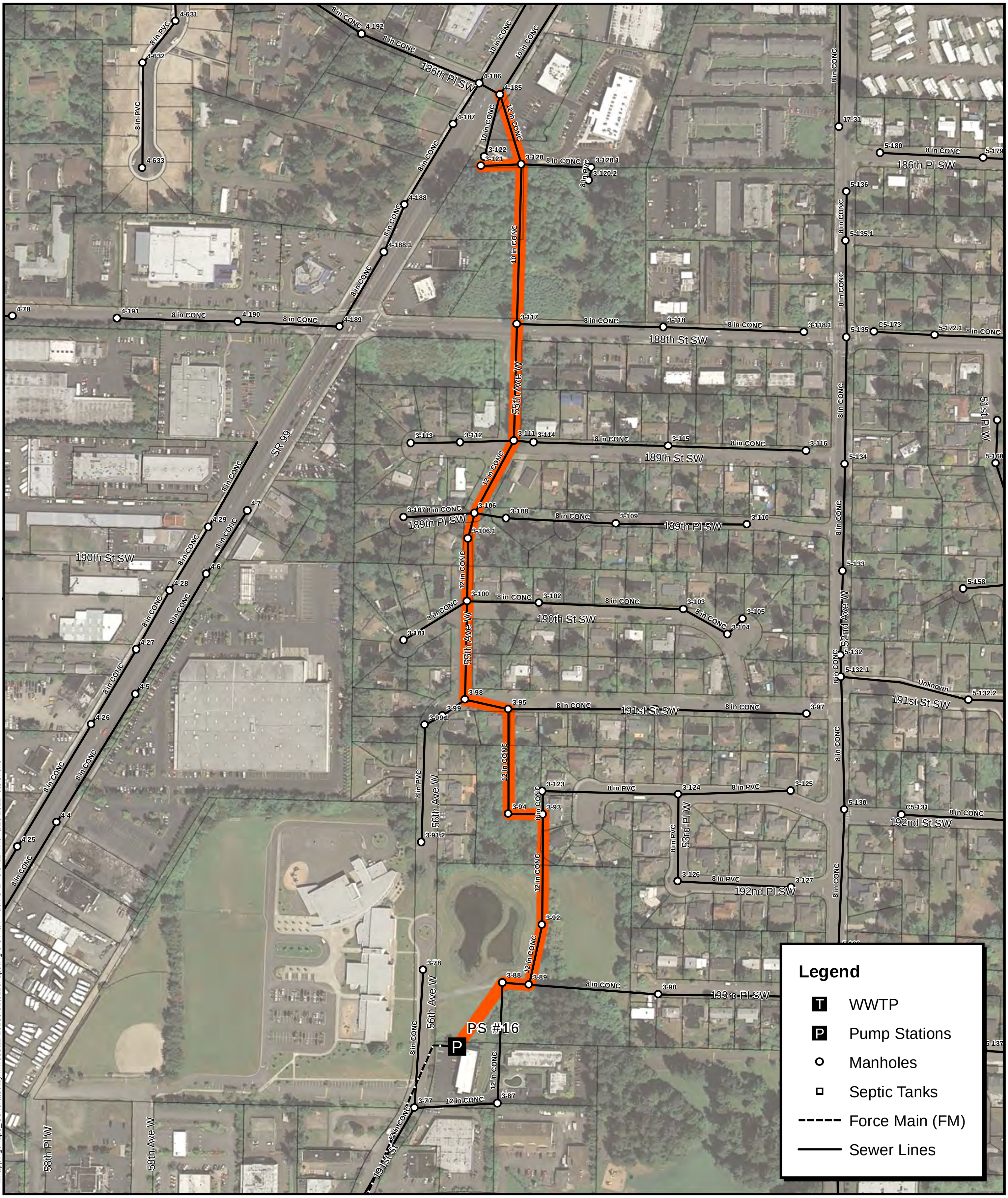
LYNNWOOD WASHINGTON

SATELLITE TREATMENT ALTERNATIVE

City of Lynnwood
2012 Wastewater
Comprehensive Plan Update
August 2012

Figure

8-2



P:\Mapping\Maps_Generated\Lynnwood\11-10238.01\011\maps\Fig 8-3 Plan Truck G-1A 8.5x11.mxd 8/23/2012 ctolentino

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NORTH

Feet
0 175 350

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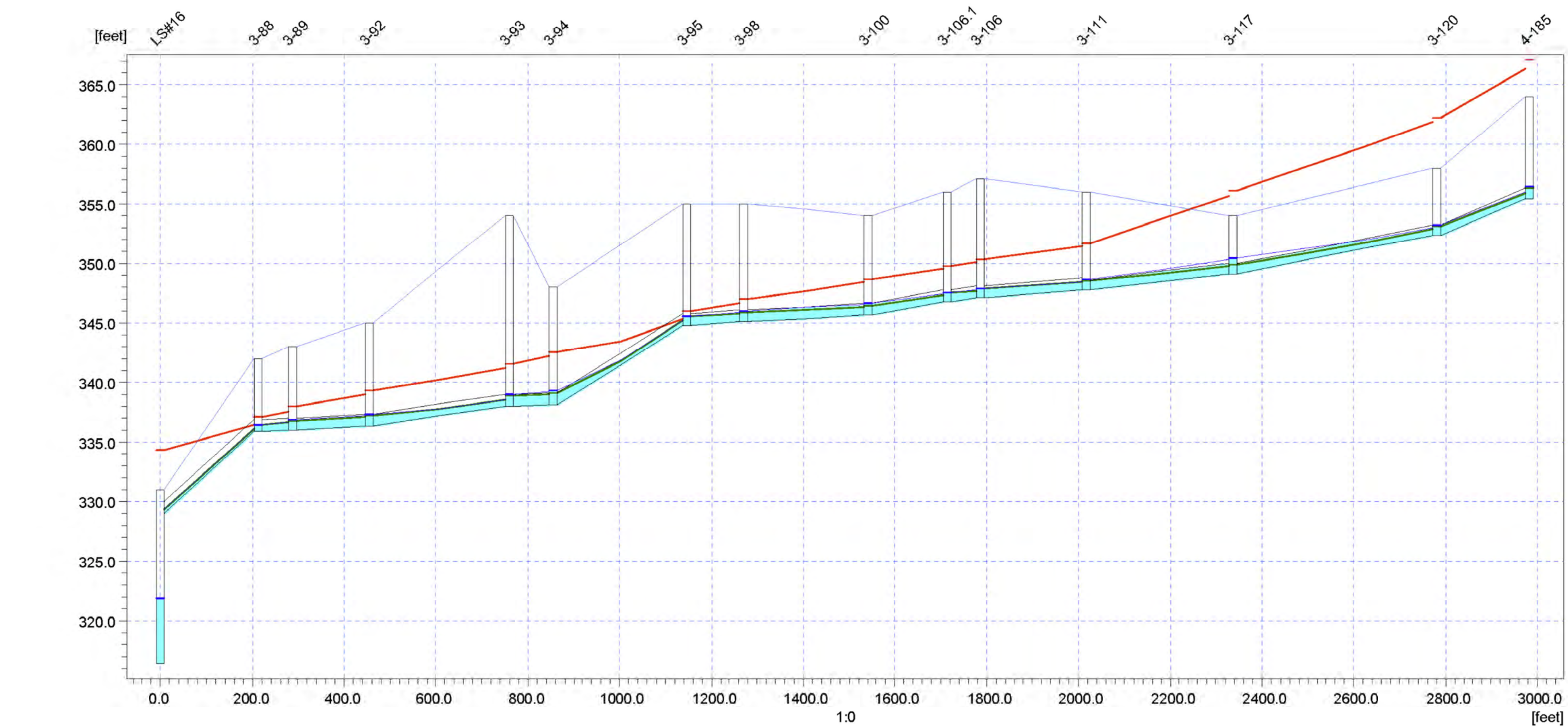
PLAN TRUNK G-1A
LS 16 TO 4-185

City of Lynnwood
2012 Wastewater
Comprehensive Plan Update
August 2012

Figure
8-3



Manhole	LS#16	3-88	3-89	3-92	3-93	3-94	3-95	3-98	3-100	3-106.1	3-106	3-111	3-117	3-120	4-185
Ground Lev.	331.00	342.00	343.00	345.00	354.00	348.00	355.00	355.00	354.00	356.00	357.09	356.00	354.00	358.00	
Invert lev.	316.43	335.85	335.95	336.34	338.01	338.09	344.82	345.07	345.67	346.81	347.09	347.75	349.14	352.34	

Segment	Length	Diameter	Slope o/oo
LS#16 – 3-88	212.45	1.00	32.24
3-88 – 3-89		1.00	1.36
3-89 – 3-92	169.03	1.00	2.31
3-92 – 3-93	305.91	1.00	5.46
3-93 – 3-94	95.41	1.00	0.84
3-94 – 3-95	289.64	1.00	23.24
3-95 – 3-98	122.97	1.00	2.03
3-98 – 3-100	272.22	1.00	2.20
3-100 – 3-106.1	174.05	1.00	6.54
3-106.1 – 3-106		1.00	3.87
3-106 – 3-111	228.00	1.00	2.87
3-111 – 3-117	321.89	0.83	4.33
3-117 – 3-120	442.97	0.83	7.22
3-120 – 4-185	201.16	0.83	15.56

P:\Mapping\Maps_Generated\Lynnwood\11-10238.01\011\maps\Fig 8-4 Exist Trunk 2032 Peak Hour CIP G-1A Profile 11x17.mxd 8/23/2012 cblentino

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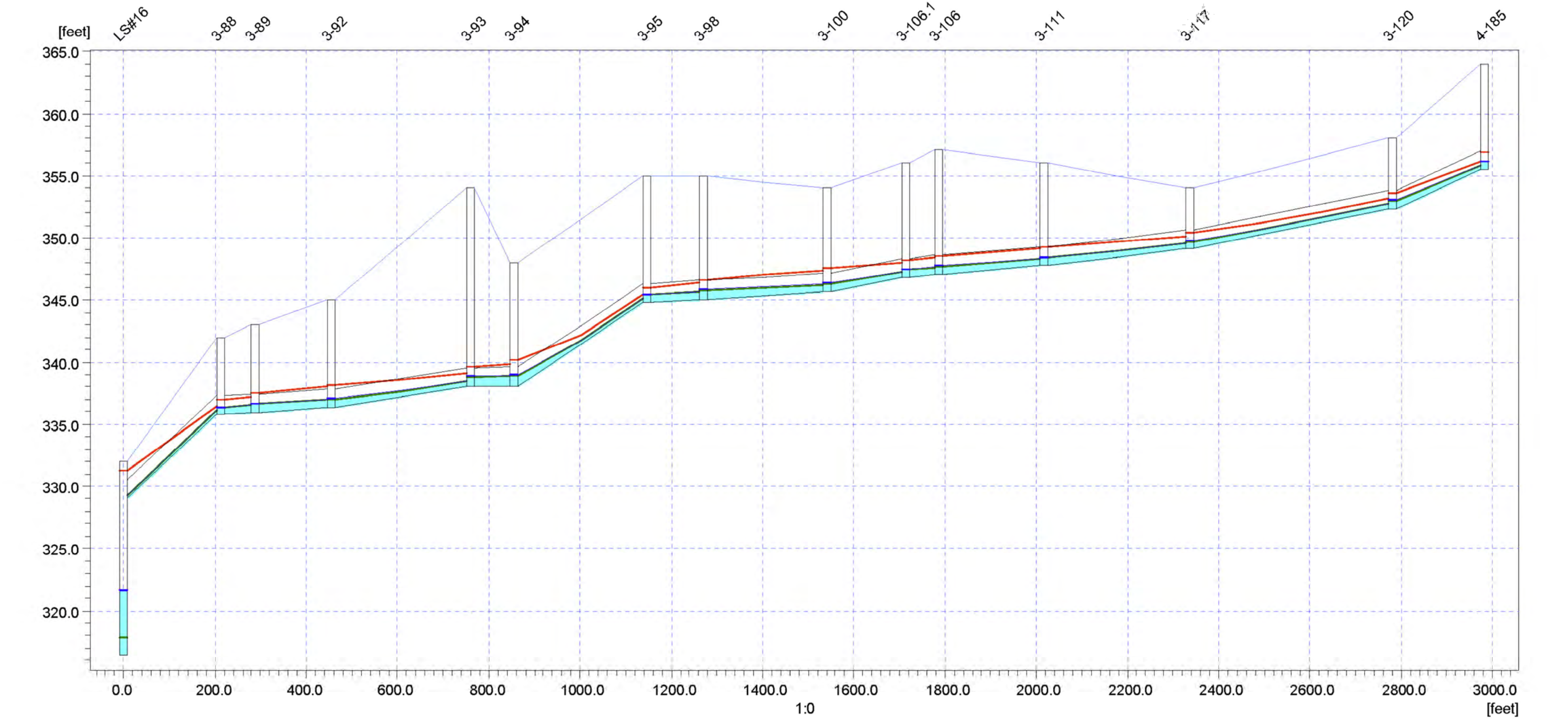
Sewer System: City of Lynnwood March 2011
GIS Base Data: Snohomish County
This map is a geographic representation based on information available. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.

LYNNWOOD WASHINGTON

EXISTING TRUNK G-1A - 2032 PEAK HOUR PROFILE LS 16 TO 4-185

City of Lynnwood
2012 Wastewater Comprehensive Plan Update
August 2012

Figure 8-4



Node	Ground Lev.	Invert lev.
LS#16	332.00	316.43
3-88	342.00	335.85
3-89	343.00	335.95
3-92	345.00	336.34
3-93	354.00	338.01
3-94	348.00	338.09
3-95	355.00	344.82
3-98	355.00	345.07
3-100	354.00	345.67
3-106.1	356.00	346.81
3-106	357.09	347.09
3-111	356.00	347.75
3-117	354.00	349.14
3-120	358.00	352.34

Length	Diameter	Slope o/oo
212.45	1.50	32.24
	1.50	1.36
169.03	1.50	2.31
305.91	1.50	5.46
95.41	1.50	0.84
289.64	1.50	23.24
122.97	1.50	2.03
272.22	1.50	2.20
174.05	1.50	6.54
	1.50	3.87
228.00	1.50	2.87
321.89	1.50	4.33
442.97	1.50	7.22
201.16	1.50	15.56

P:\Mapping\Maps_Generated\Lynnwood\11-10238.01\011\maps\Fig 8-5 Trunk G-1A 2032 Improv Profile 11x17.mxd 8/23/2012 ctolentino

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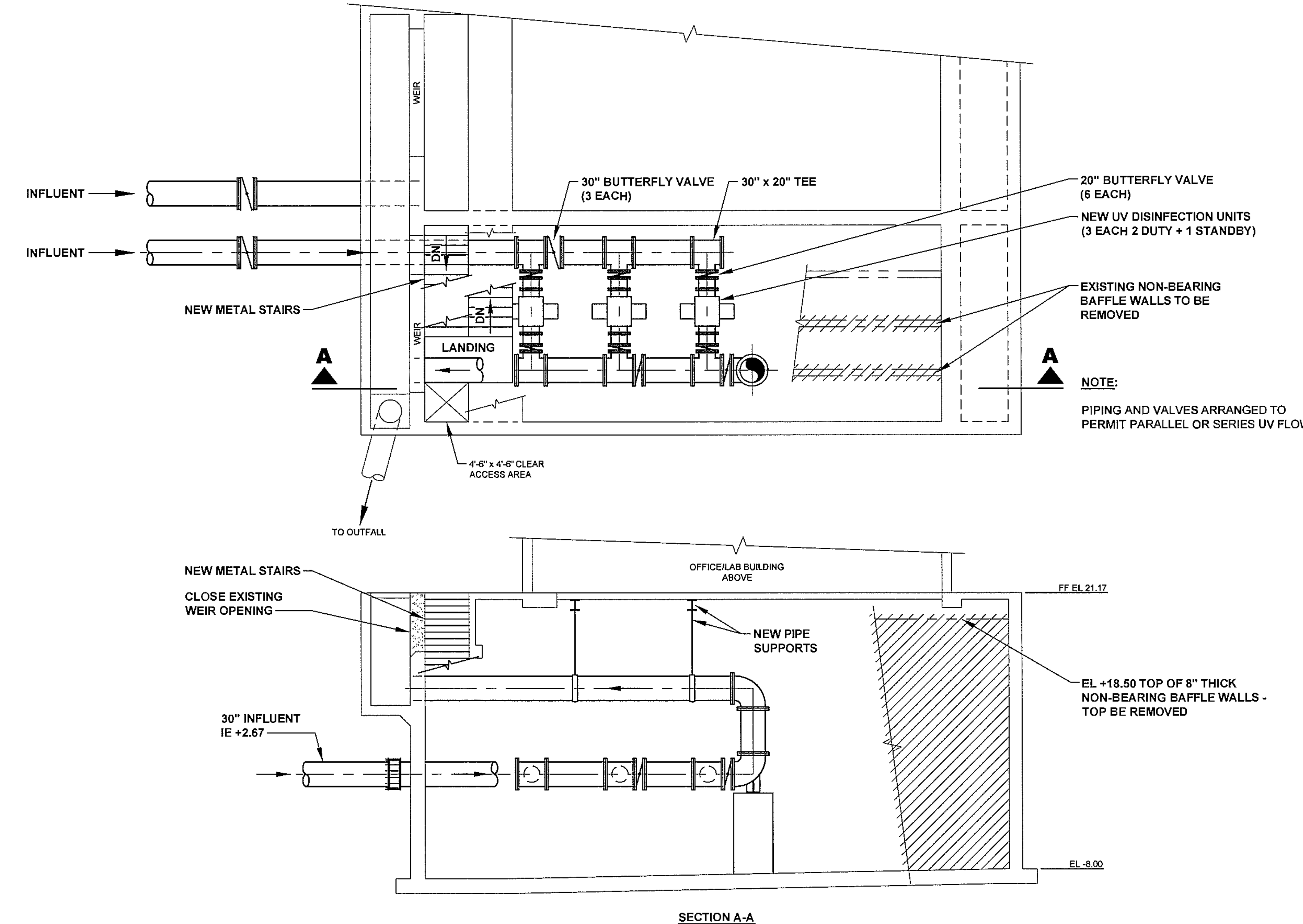
Sewer System: City of Lynnwood March 2011
GIS Base Data: Snohomish County
This map is a geographic representation based on information available. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.

LYNNWOOD WASHINGTON

TRUNK G-1A WITH 2032 IMPROVEMENTS PROFILE LS16 to 4-185

City of Lynnwood
2012 Wastewater Comprehensive Plan Update
August 2012

Figure 8-5



PLOT DATE & TIME AUG 23 2012 10:31:49
FILE NAME (UPDATED BY) S:\CAD\LYNNWOOD\11-10238 SEWER COMP PLAN\DWGS\F11-10238 FIG 8-7.DWG (GOC)
XREFS: NONE

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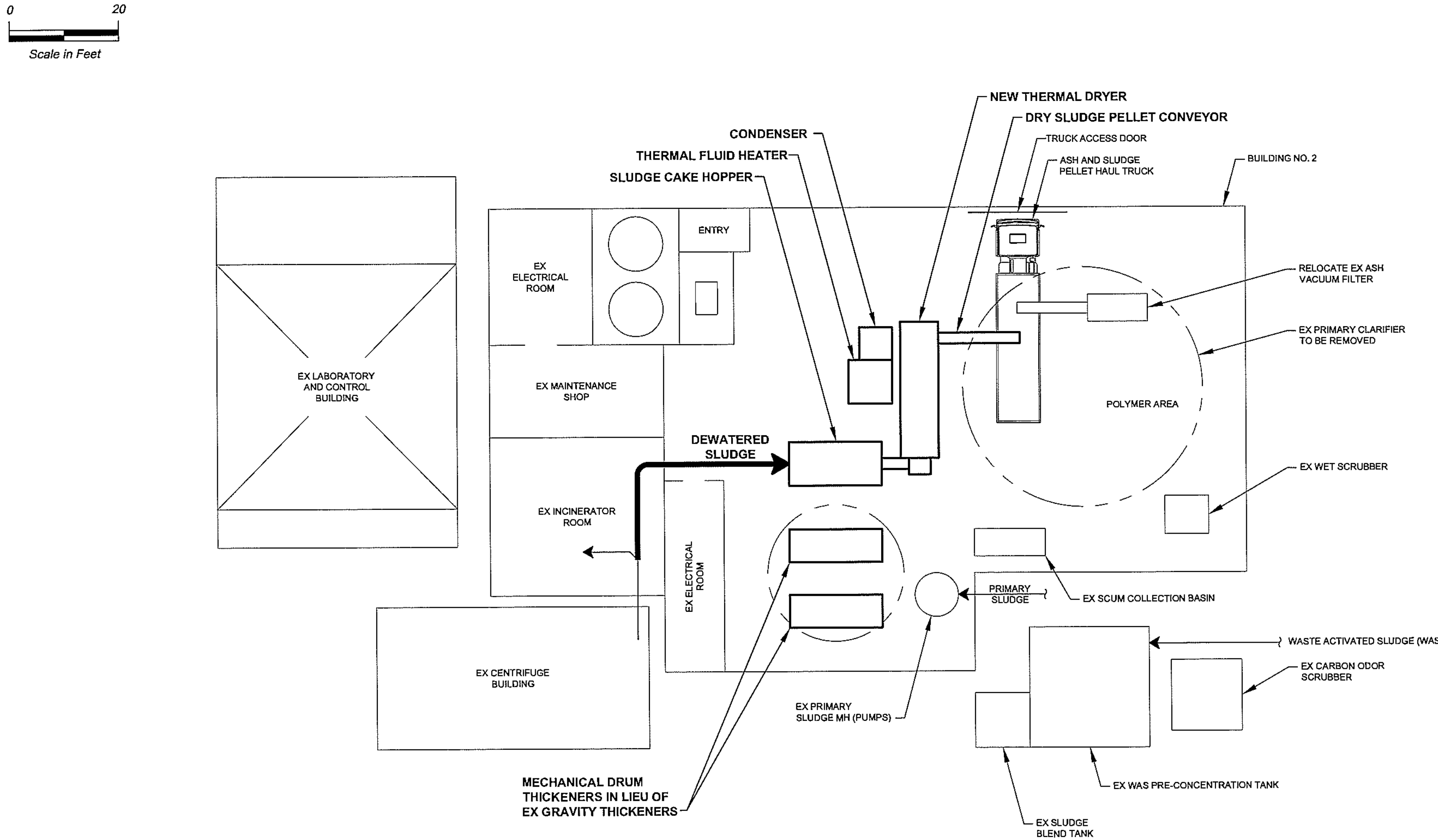
LYNNWOOD WASHINGTON

In-line UV Installation Concept

Lynnwood Wastewater Treatment Plant
August 2012

Figure **8-7**



PLOT DATE & TIME
JAN 26 2012 12:26:00
FILE NAME (UPDATED BY)
S:\CAD\LYNNWOOD\WWTP SLUDGE DRYER ADD.DWG (GOC)
XREFS: NONE

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Site Plan for Sludge Thickener and Dryer in Building No. 2

Lynnwood Wastewater Treatment Plant

August 2012

Figure

8-8

Chapter 9 Recommended Improvements

9.1 Conveyance Recommendations

Ongoing lift station improvements that are expected to be completed before 2018 include the following:

Lift Station No. 4	to be funded by developers
Lift Station No. 8	$5,800,000 in 2010 dollars
Lift Station No.16	$4,830,000 in 2010 dollars
City Total	$10,630,000 in 2010 dollars

Sewer piping improvements needed within the next six years as identified through the hydraulic model are summarized in Table 9-1.

Table 9-1 Modeled Sewer Piping Improvements Needed by 2018

Trunk	Description	Ex Dia (in)	Length (ft)	2032 Dia (in)	Est Const Cost	Est Project Cost
B-1	LS 8 Node 81 to 16-111	12	622	15	$ 239,000	$ 311,000
C-2C	LS 10 16-37 to 6-78	8 to 12	1,357	15 to 18	$ 578,000	$ 751,000
G-1A	LS 16 to 4-85	10 to 12	2,982	18	$ 156,000	$ 1,502,000
G-2A	4-185 to 17-134	8 to 10	5,439	10 to 12	$ 1,221,000	$ 1,587,000
I	LS 12 11-8 to 1091	8	5,762	10 to 12	$ 1,009,000	$ 1,311,000
J	WWTP 7-90 to 7-117	15	2,588	18	$ 1,003,000	$ 1,304,000
K-1	6-1 to 6-17	24	1,074	27	$ 582,000	$ 758,000
K-2	8-6 to 7-1	24	3,419	30 to 33	$ 2,138,000	$ 2,779,000
L	3-75 to 4-172	8	1,885	10	$ 88,000	$ 115,000
	Totals		**25,128**		**$ 8,014,000**	**$ 10,418,000**

The pipe improvements shown in Table 9-1 reflect the model results of pipe reaches that currently exceed the surcharge criteria, and show the diameter needed to accommodate peak hour flow projected for the year 2032. Additional pipe improvement will be needed by 2031 as summarized in Table 9-2.

Table 9-2 Additional Sewer Piping Improvements Needed by 2032

Trunk	Description	Ex Dia (in)	Length (ft)	2032 Dia (in)	Est Const Cost	Est Project Cost
B-1	81 to 16-111	8	1,016	10	$ 165,000	$ 214,000
C-1	LS 10 to 16-38	12	3,459	15 to 18	$ 2,123,000	$ 2,760,000
C-2B	16-37 to15-65	8	1,605	10	$ 260,000	$ 338,000
C-2C	16-37 to 16-78	8	777	12	$ 176,000	$ 229,000
D-4	5-9 to 5-60	8	1,420	10	$ 230,000	$ 299,000
E-2	5-7 to 16-28	8	2,307	10	$ 228,000	$ 296,000
E-3	LS 10 to 5-10	10 & 21	3,520	12 & 27	$ 1,237,000	$ 1,608,000
F-1	LS 10 to 3-6	18	267	21	$ 107,000	$ 139,000
I	1-23 to 11-8	8	322	12	$ 73,000	$ 95,000
K-1	6-1 to 6-17	24	1,370	27	$ 150,000	$ 195,000
K-2	7-2 to 6-1	18	261	21	$ 79,000	$ 103,000
K-3	6-16 to 6-161	36	2,676	39	$ 1,831,000	$ 2,380,000
	Totals		**19,000**		**$ 6,659,000**	**$ 8,656,000**

9.2 Liquid Stream Recommendations

Lynnwood should investigate installing an energy dissipating vortex in the existing drop structure or in Manhole 2A, just upstream of the existing influent Parshall flume. The energy dissipating vortex would slow the approach velocity to the flume and allow for more accurate influent flow measurement.

Lynnwood should implement ultraviolet disinfection or on-site hypochlorite generation in lieu of the existing chlorine gas system. The close proximity of residential homes to the plant and the need for emergency response planning (including the possible need for evacuation drills) could make the continued use of chlorine gas a public relations problem, even if it is not an actual emergency. Onsite generation is cheaper at $800,000 than UV at $1,700,000. However, UV poses safer and poses less risk to the surrounding community.

Lynnwood should plan for the eventual implementation of chemically enhanced primary treatment (CEPT), which will likely be necessary by about year 2030. CEPT may become necessary if the existing circular primary clarifies is removed to provide space for a sludge dryer. Planning should include reserving room in existing re-constructed Building No. 2 for storing and feeding coagulants to the existing rectangular primary clarifiers.

9.3 Solids Management Recommendations

A wet electrostatic precipitator will needed to be added to the existing incinerator emission controls for added particulate removal to meet regulatory requirement at an estimated cost of $1,000,000. It is likely that mercury can be source controlled through implementing and enforcing a dental amalgam pretreatment program.

Planning for installation of a sludge dryer is recommended with the cost estimated $3,000,000 including materials handling and ancillary equipment. Space could be provided in Building No. 2 by removing the existing circular clarifier, which may necessitate implementing CEPT. Sludge dried 90 percent into pellets would reduce the need for supplemental diesel fuel, provide and alternative disposal option and produce a soil amendment product meeting Class A standards that is more environmentally sustainable. However there is no immediate requirement to implement such an improvement.

9.4 Structural Recommendations

Both Building No. 1 and No. 2 have significant deterioration and have been recommended for rehabilitation for nearly a decade. Upgrade of No. 2 is the higher priority at an estimated cost of $500,000. However, the deterioration does not affect the treatment process effectiveness so it does not represent an immediately required improvement.

9.5 Six-Year Capital Improvement Program

Improvements recommended to be implemented during the next six years are organized in Table 9-3 to form the Capital Improvement Program.

Table 9-3 Six-Year Capital Improvement Program
Costs Shown in Millions

Improvement	2013	2014	2015	2016	2017	2018	Total Cost
Lift Stations							
LS 4	Developer Funded						
LS 8	$400,000	$5,400,000					$ 5,800,000
LS 16		$ 530,000	$4,300,000				$ 4,830,000
Subtotal	$400,000	$5,930,000	$4,300,000	$ -	$ -	$ -	$10,630,000
Conveyance							
Updated Flow Monitoring	$150,000						$ 150,000
B-1		$ 311,000					$ 311,000
C-2C		$ 751,000					$ 751,000
G-1A			$ 200,000	$1,302,000			$ 1,502,000
G-2A			$ 200,000	$1,387,000			$ 1,587,000
I				$ 100,000	$1,211,000		$ 1,311,000
J				$ 100,000	$1,204,000		$ 1,304,000
K-1					$ 80,000	$678,000	$ 758,000
K-2					$ 300,000	$2,479,000	$ 2,779,000
L					$ 15,000	$ 100,000	$ 115,000
Subtotal	$150,000	$1,062,000	$ 400,000	$2,889,000	$2,810,000	$3,257,000	$10,568,000
Treatment							
Disinfection Engineer Report	$ 30,000						$ 30,000
Disinfection Design				$ 240,000			$ 240,000
Disinfection Construction					$1,700,000		$ 1,700,000
Structural Assess Bldg 1 & 2	$ 80,000						$ 80,000
Solids Engineer Report	$ 50,000						$ 50,000
Screw Press Design		$ 50,000					$ 50,000
Screw Press Construction			$ 280,000				$ 280,000
Incinerator Engineer Report	$250,000						$ 250,000

Table 9-3 Six-Year Capital Improvement Program
Costs Shown in Millions

Improvement	2013	2014	2015	2016	2017	2018	Total Cost
Incinerator Upgrade Design		$ 50,000					$ 50,000
Incinerator Control Upgrade			$1,500,000				$ 1,500,000
Heat Exchanger Replacement				$ 500,000			$ 500,000
Electrostatic Precipitator		$ 1,000,000					$ 1,000,000
Subtotal	$410,000	$ 1,100,000	$1,780,000	$ 740,000	$ 1,700,000	$ -	$ 5,730,000
GRAND TOTAL	**$960,000**	**$ 8,092,000**	**$6,480,000**	**$ 3,629,000**	**$ 4,510,000**	**$ 3,257,000**	**$26,928,000**

9.6 Sewer Extensions into Undeveloped Areas

As noted on Figure 3-8, the existing sewer pipe system does not serve all developed parcels within the City sewer service area. Some vacant parcels that may be suitable for development are also not served. The City does not have a program for extending sewers to serve these parcels.

If and when these property owners desire sewer serve, pipes can be extended by Developer Extension Agreements or by formation of Local Improvement Districts (LID).

Chapter 10 Financial Program

10.1 Existing Sewer Rates

The City adopted a six-year sewer rate structure that became effective in January 2011 under Chapter 14.40 of the Lynnwood Municipal Code. The rate structure is organized into residential and commercial/industrial categories with a number of classifications within each category. Table 10-1 displays residential rates as an example.

Table 10-1 Monthly Residential Sewer Rates

Classification	2011	2012	2013	2014	2015	2016
Residential Single-Unit						
Mo. Base Rate (incl. 10 CCF)	$33.89	$35.38	$37.33	$38.73	$40.05	$41.38
Mo. Volume Charge > 10 CCF	$2.77**	$2.89**	$3.05**	$3.16**	$3.27**	$3.38**
Special Sewer Rate	See Section 14.40.040 Special Sewer Rates					
Residential Multiple/Mobile-Unit Combined						
Mo. Base Rate (incl. 10 CCF/Unit)	$29.06	$29.49	$29.49	$29.49	$29.49	$29.49
Mo. Volume Charge > 10 CCF/Unit	$2.72*	$2.76*	$2.76*	$2.76*	$2.76*	$2.76*
Special Sewer Rate	See Section 14.40.040 Special Sewer Rates					
Sewer Only Accounts						
Residential Single-Unit	$35.35	$36.91	$38.94	$40.41	$41.79	$43.18
Residential-Multiple-Unit per Unit	$30.53	$30.98	$30.98	$30.98	$30.98	$30.98

*Base rate plus volume over 10 CCF per unit
**For next additional 17.5 CCF (No charges are incurred for volumes over the additional 17.5 CCF per month)

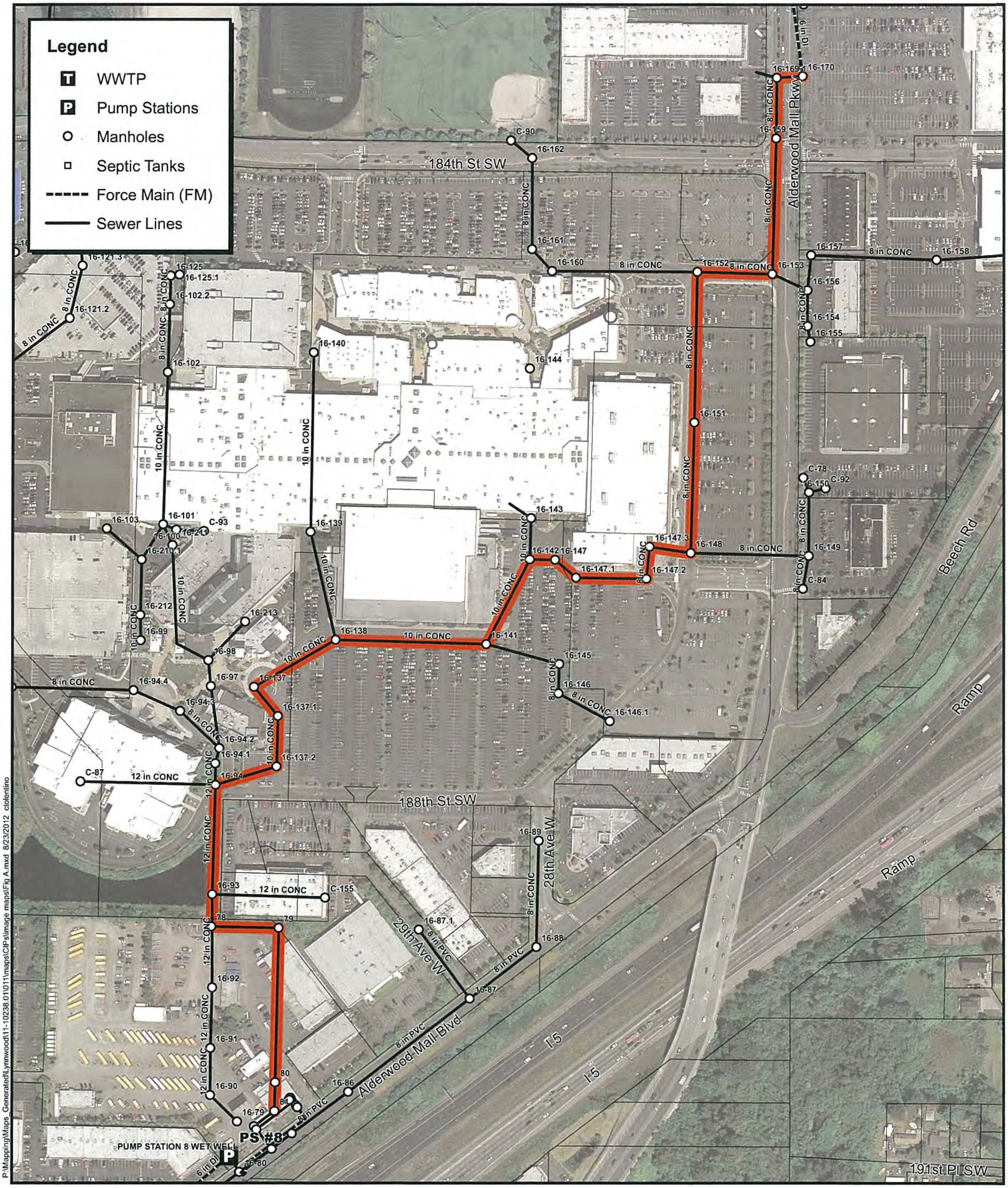
Commercial/industrial customers are billed based on water meter sizes 5/8 through 6-inches.

10.2 Financial Program

Financing of the Capital Improvement Program will be developed subsequently through a rate analysis during early 2013.

Appendix A

Modeled Trunks Plans and Profiles



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NORTH

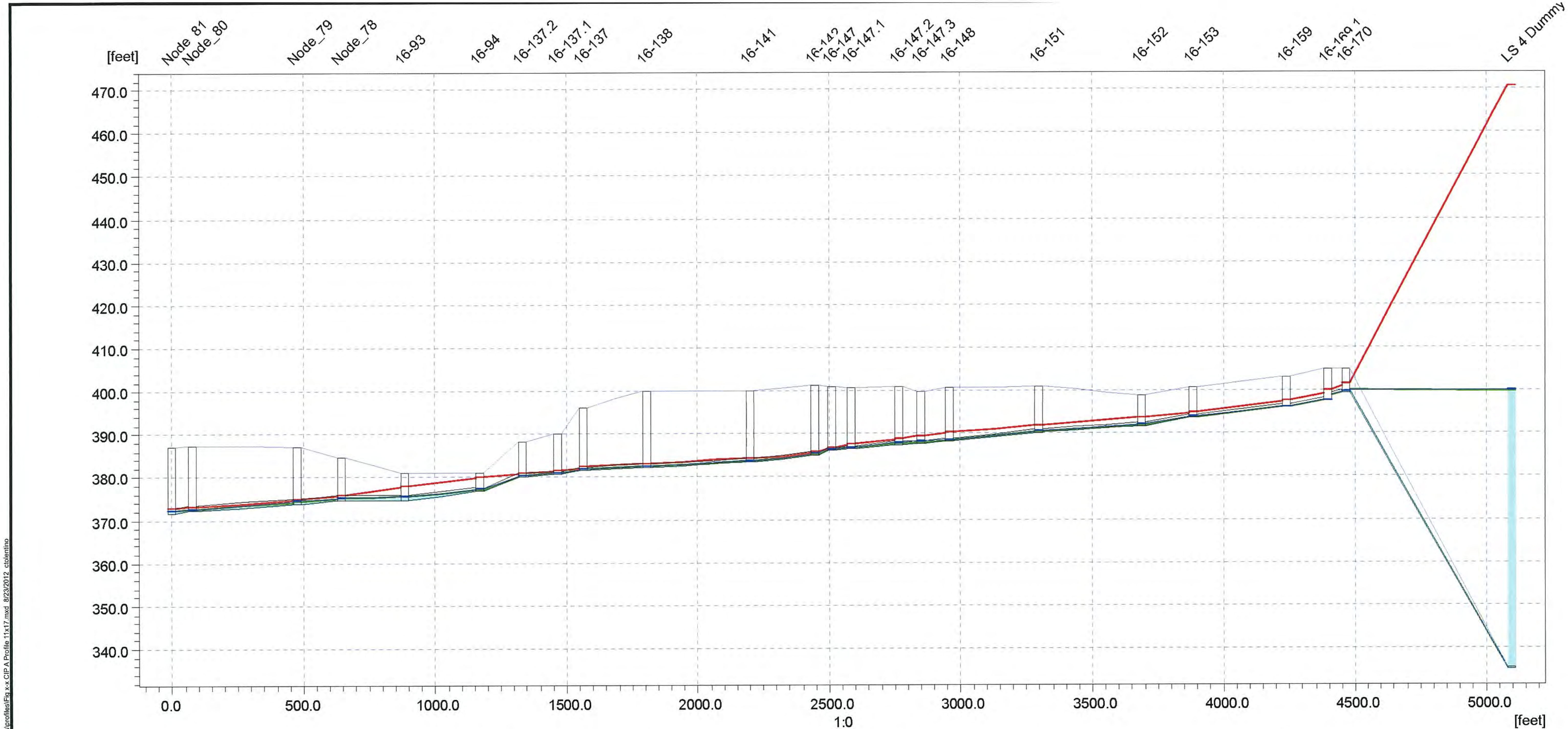
0 250 500 Feet

LYNNWOOD WASHINGTON

CIP A
PROFILE NODE 81 TO 16-170

City of Lynnwood
2012 Wastewater
Comprehensive Plan Update
August 2012

Figure
A



Ground Lev.	Invert lev.
387.00	371.77
387.09	373.87
384.69	374.67
381.00	374.80
381.00	376.68
388.19	380.09
390.10	380.66
396.25	381.52
400.03	382.33
399.92	383.42
401.25	385.00
400.76	386.46
400.83	387.53
399.72	387.57
400.59	388.17
400.88	390.22
398.91	391.71
400.57	393.57
403.07	396.07
405.00	397.53

Length	Diameter	Slope o/oo
399.04	1.25	4.26
	1.25	4.70
241.70	1.00	0.12
283.80	1.00	6.62
	0.83	20.84
	0.83	4.34
244.51	0.83	3.31
391.55	0.83	2.78
247.75	0.83	6.38
	0.67	5.82
339.52	0.67	5.57
390.63	0.67	3.40
196.84	0.67	9.45
351.96	0.67	7.10
	0.67	9.28
630.60	0.50	102.11

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Sewer System: City of Lynnwood March 2011
GIS Base Data: Snohomish County
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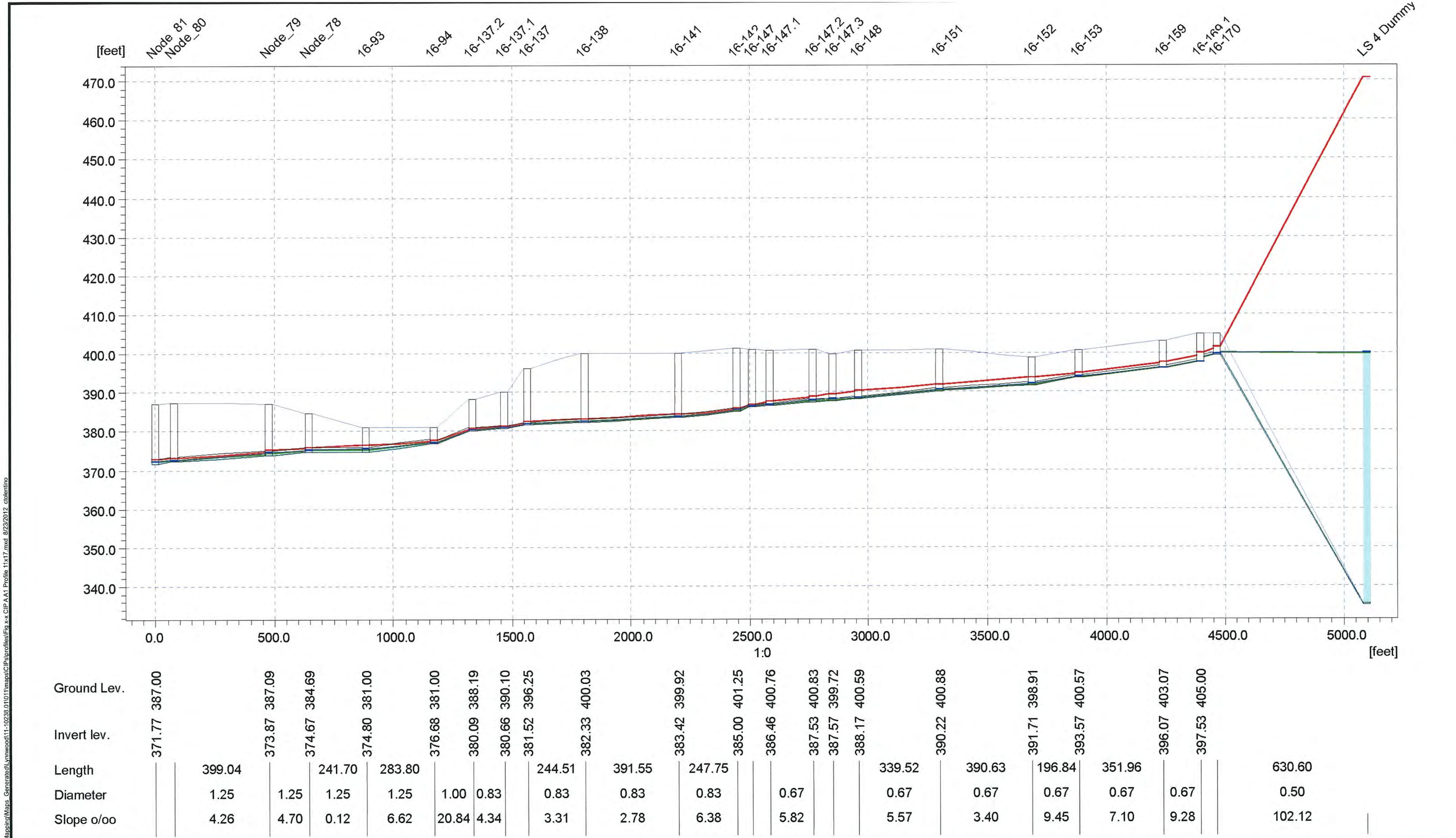
LYNNWOOD WASHINGTON

CIP A
PROFILE NODE 81 TO 16-170

City of Lynnwood
2012 Wastewater Comprehensive Plan Update
August 2012

Figure
x-x



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CIP A IMPROVEMENTS PROFILE NODE 81 TO 16-170

City of Lynnwood
2012 Wastewater Comprehensive Plan Update
August 2012

Figure
x-x