CITY OF DAYTON
Wastewater System Facilities Plan
Dayton, Oregon

CHAPTER 4

EXISTING WASTEWATER FACILITIES

Chapter Outline

CHAPTER 4 EXISTING WASTEWATER FACILITIES

4.1. INTRODUCTION

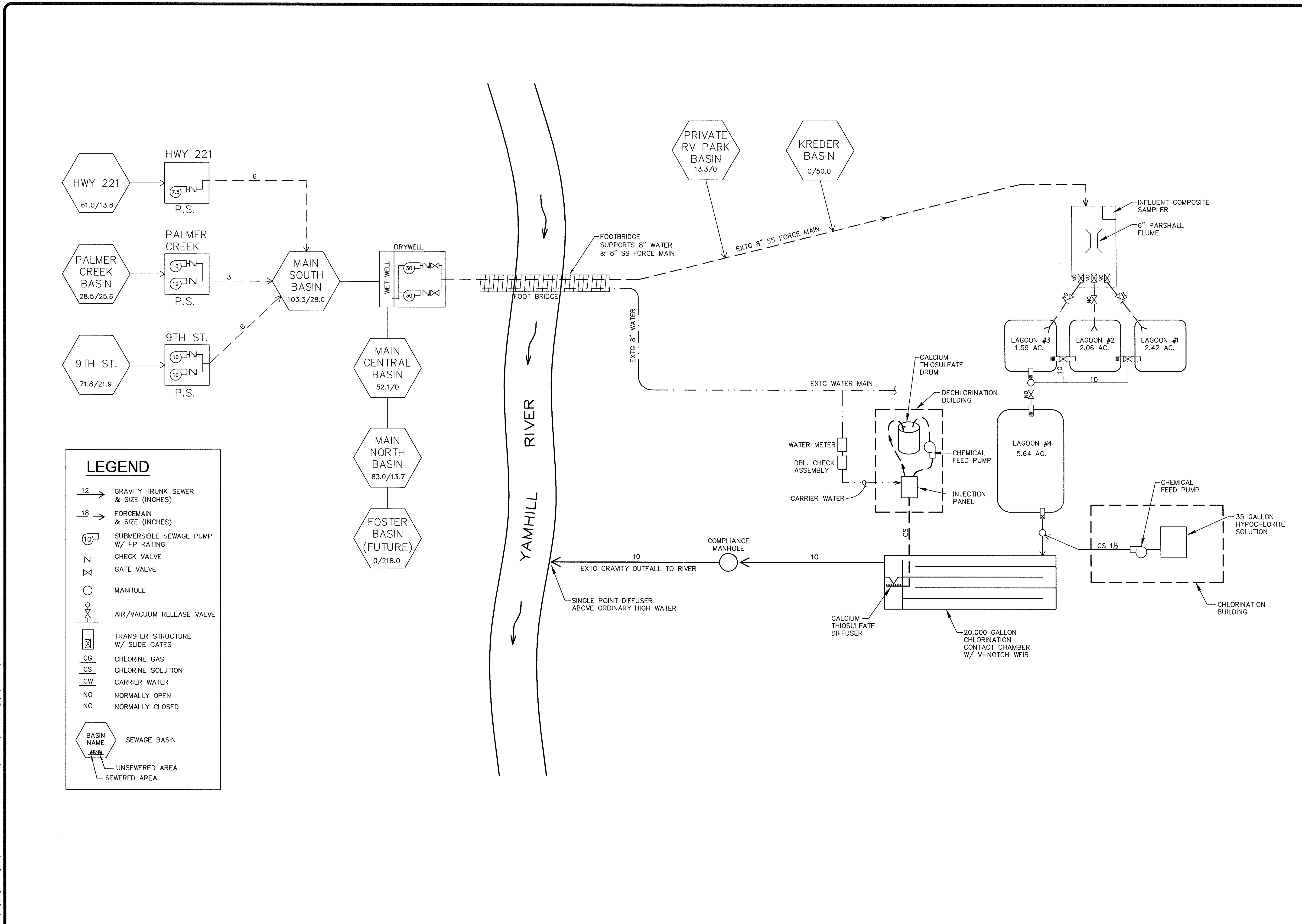
This section provides an overview of the existing wastewater facilities including the existing wastewater collection system, pump stations, and the wastewater treatment plant. It also summarizes known or reported problems related to each of these components.

4.2. GENERAL OVERVIEW OF EXISTING WASTEWATER FACILITIES

Dayton's wastewater facilities consist of a conventional gravity collection system that conveys wastewater from the users to one of four pump stations. Three of the four pump stations lift wastewater to the main collection system. The main collection system drains to the main pump station. The main pump station conveys wastewater over the Yamhill River (via a pipeline attached to a suspension bridge) to the treatment facility.

The treatment facility straddles Kreder Road southeast of the RV Park located along the Dayton Bypass (OSH 18). The treatment plant consists of a headworks, four facultative lagoons, a chlorination building with chlorine disinfection equipment, a chlorine contact chamber, dechlorination equipment, and an outfall to the Yamhill River. An overall schematic representation of the existing wastewater system is presented in Figure 4-1.

Figure 4-2 shows the existing wastewater collection facilities. The reader is encouraged to refer to these figures throughout the following discussion.



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R:\Dwg\Dayton City of\Wastewater Facilities Plan 2009\8-12-11\FIG 4-1.dwg (Layout1 tab)

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		REVISIONS	

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DSN. JW
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CKD. JW
DATE: JUN 2011

WESTECH ENGINEERING, INC.
CONSULTING ENGINEERS AND PLANNERS
3841 Fairview Industrial Dr. S.E., Suite 100, Salem, OR 97302
Phone: (503) 585-2474 Fax: (503) 585-3986
E-mail: westech@westech-eng.com

CITY OF DAYTON
WASTEWATER FACILITIES PLAN

EXISTING WASTEWATER FACILITIES PLAN
FIGURE 4-1

FIGURE
4–1

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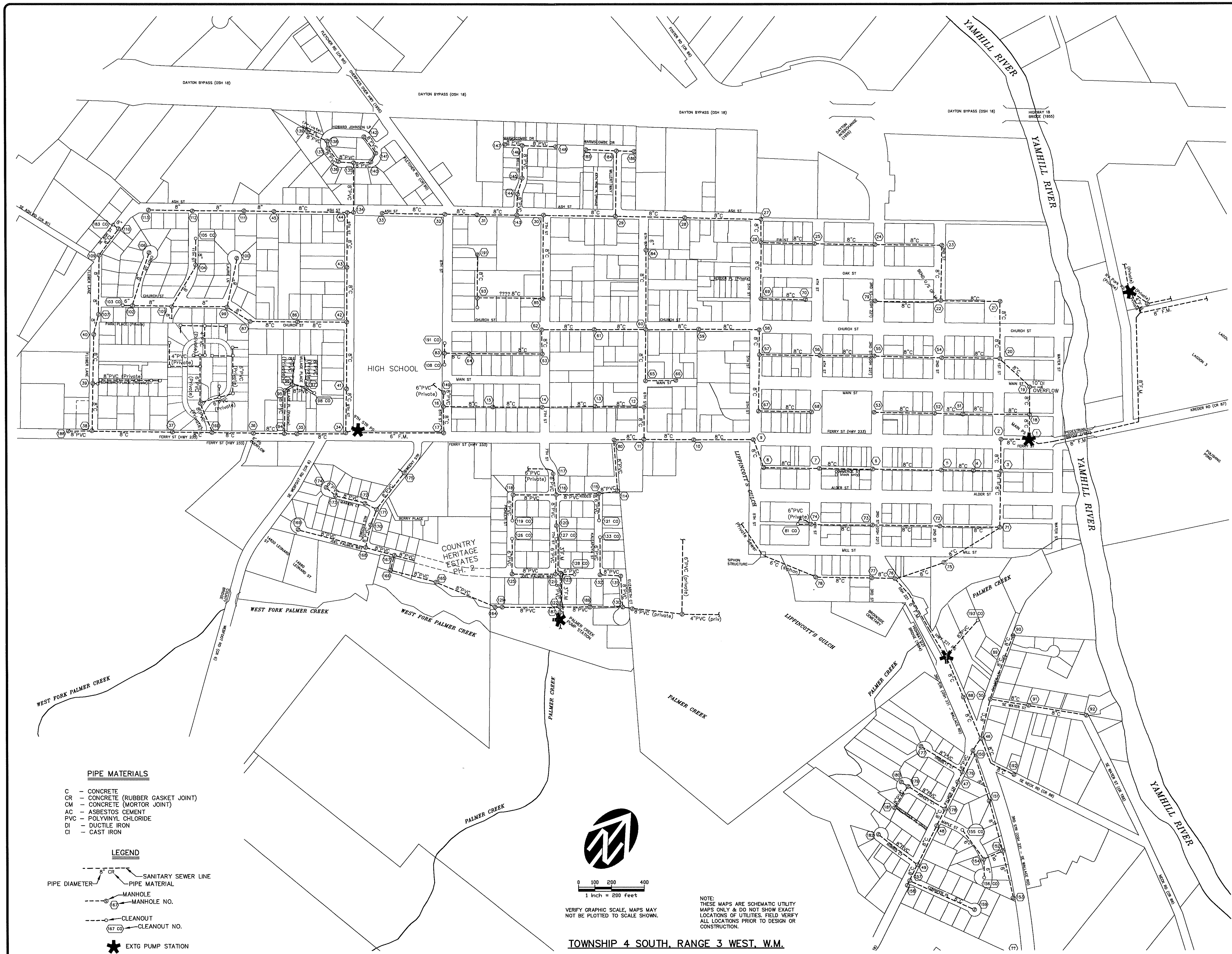

CITY OF DAYTON, OREGON

EXISTING MAP
COLLECTION SYSTEM
FIGURE 4-2

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DATE: JUN 2011

FIGURE
4-2

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4.3. HISTORY AND DEVELOPMENT OF SEWERAGE FACILITIES

Dayton's original sewer system was built in 1965 and 1966 and replaced all the individual septic tank systems . It served most of the area within the present City limits from 10th Street east to Water Street and from Mill Street north to Ash Street. This area also included that portion of town southeast of the HWY 221 bridge from Conifer Place north to Jarika Place.

Concrete pipe was used for the construction of the original gravity collection piping. Three sewage lift stations were constructed as part of the original system in 1966. These included the Main Pump Station, the 9th Street Pump Station, and the HWY 221 Pump Station. Though modified over the years, these pump stations are still in operation today. As originally constructed, the 9th Street and HWY 221 stations discharged into the upper ends of the gravity collection system that drained to the Main Pump Station. The Main Pump Station conveys all wastewater collected in the City to the treatment facility. The treatment facility was constructed in 1966 and consisted of two stabilization lagoons operated in parallel located at the current site north east of the Yamhill River. The Main Pump Station received raw wastewater from the entire collection system and pumped the sewage under the Yamhill River via a 6-inch forcemain into the treatment facility. Plant effluent from the 1966 treatment facility was then chlorinated and discharged to the Yamhill River through an 8-inch outfall pipeline.

In 1982, a new facultative lagoon treatment facility was constructed north of the existing wastewater lagoons on the east side of the Yamhill River. The new facility created 3 new treatment lagoons and utilized the two existing 1966 lagoons as polishing cells. The dike between the two existing lagoons was removed to form the single polishing lagoon (lagoon #4) that is currently in service. To convey wastewater to the upstream end of the new lagoons, the 6-inch diameter forcemain from the Main Pump Station was replaced with a new 8-inch forcemain. Also at that time, the Main Pump Station pumps and motors were upsized to convey the increased flow, while the wetwell and drywell remained unchanged. The 9th Street and HWY 221 Pump Stations, forcemains, and discharge locations remain unchanged since 1966. The only exception being the HWY 221 forcemain that was replaced when the new bridge was constructed in 1982. The 9th Street and HWY 221 Pump Stations operate by lifting sewage into the gravity collection system that drains to the Main Pump Station.

In the time since the lagoons were constructed, additional development in town resulted in the construction of new gravity mains and one additional pump station. The new Palmer Creek Pump Station was originally constructed in 2007 and is located south of Joe Palmer Way and east of the Elizabeth Court cul-de-sac. This pump station serves the Palmer Creek Basin composed of primarily single family residential services. The pump station discharges into the MH # 120, which drains into the main collection system as shown in Figure 4-2.

4.4. WASTEWATER COLLECTION SYSTEM

The City's existing sanitary sewage collection system collects wastewater from residences, businesses, industries, and public facilities and conveys the wastewater to the City's wastewater treatment plant via a combination of gravity collection and forcemains from one of four City owned pump stations. This section provides an overview of the existing wastewater collection system within the study area with an emphasis on flow routing and known and reported problems.

Although all public sewers within the study area are owned by the City, two other entities have jurisdiction over the right-of-ways within which the sewer mainlines are located. In addition to the City, the Oregon Department of Transportation (ODOT) has jurisdictional oversight for facilities constructed within the 3rd Street (HWY 221) and Ferry Street (HWY 233) right-of-ways. Yamhill

County has jurisdiction over Kreder Road (CR 87), Webfoot Road and the public roads located north of HWY 18.

4.4.1 User Connections

The City's system currently serves 799 user connections. The user connections are classified as shown in Table 4-1.

Table 4-1 | Sewer Connection Summary

(As of November 2009)

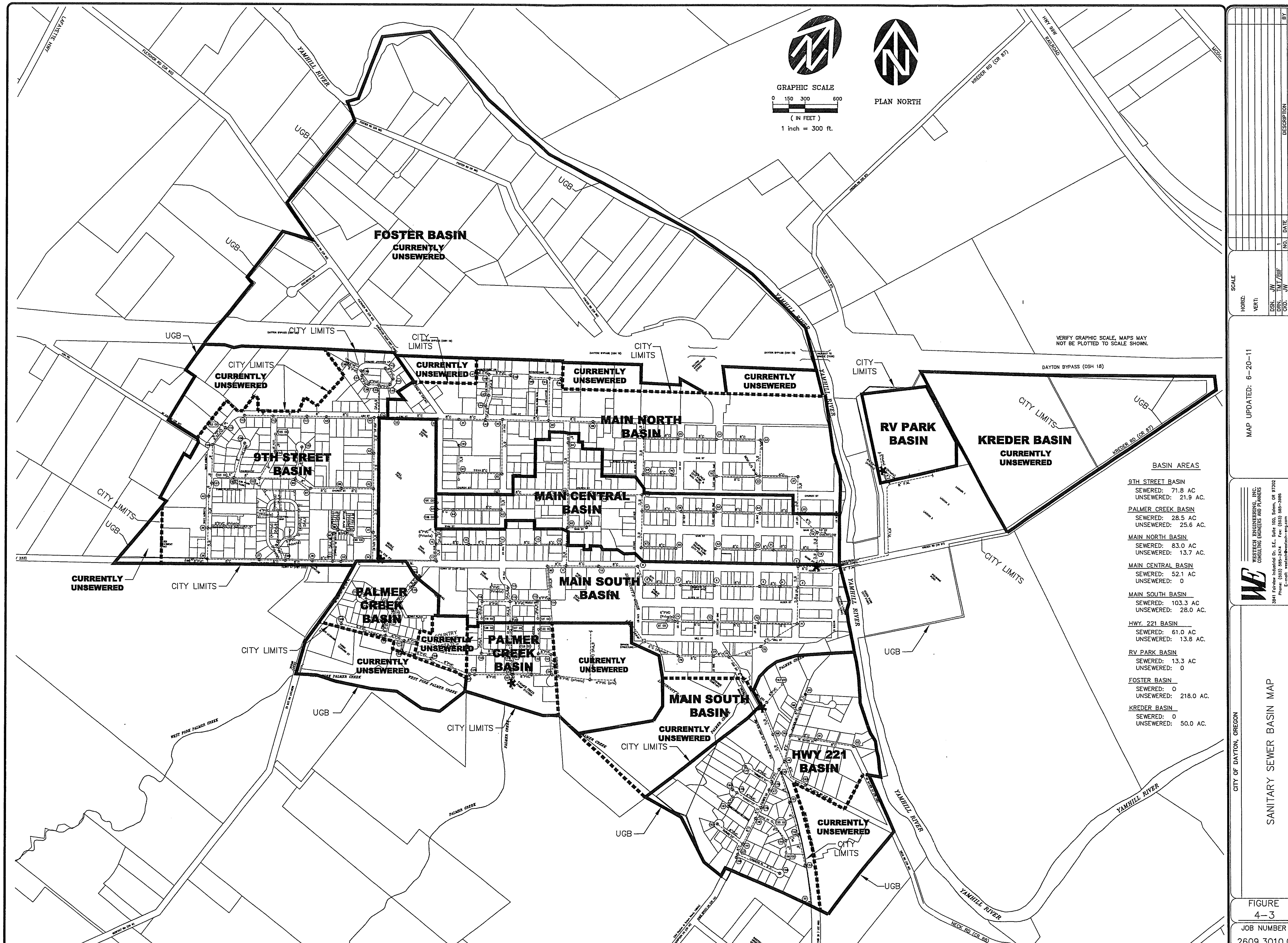
User Classification	Number of Services
Single Family Residential	745
Duplex	6
Triplex	3
Apartments	1
Housing Authority	2
Commercial	23
Public	5
Schools	6
Churches	7
RV Park	1
Total	799

4.4.2 Sewer Drainage Basins

To aid in the analysis of the collection system, it is convenient to divide the collection system into separate drainage basins. The basin boundaries are based on a combination of factors including topography, urban growth boundaries, as well as the existing drainage patterns and pump station locations. The collection system is divided into 9 distinct basins as shown in Figure 4-3. The major basins are generally named after the pump station that serves the basin. Table 4-2 lists the basin name and approximate area within each of the basins.

Table 4-2 | Collection System – Drainage Basin Areas

Sewer Basin Designation	Total Area (Acres)	Sewered Area (Acres)	Nonsewered Area (Acres)
9th Street	93.7	71.8	21.9
Palmer Creek Basin	54.1	28.5	25.6
Main North Basin	96.7	83	13.7
Main Central Basin	52.1	52.1	0
Main South Basin	131.3	103.3	28
HWY 221 Basin	74.8	61	13.8
RV Park Basin	13.3	13.3	0
Foster Basin	218	0	218
Kreder Basin	50	0	50
Totals	784	413	371



GRAPHIC SCALE
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(IN FEET)
1 inch = 300 ft.
PLAN NORTH
FOSTER BASIN
CURRENTLY UNSEWERED
9TH STREET BASIN
MAIN NORTH BASIN
MAIN CENTRAL BASIN
MAIN SOUTH BASIN
PALMER CREEK BASIN
RV PARK BASIN
KREDER BASIN
HWY 221 BASIN
CITY LIMITS
UGB
YAMHILL RIVER
DAYTON BYPASS (OSH 18)
KREDER RD (CR 87)
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BASIN AREAS
9TH STREET BASIN
SEWERED: 71.8 AC
UNSEWERED: 21.9 AC.
PALMER CREEK BASIN
SEWERED: 28.5 AC
UNSEWERED: 25.6 AC.
MAIN NORTH BASIN
SEWERED: 83.0 AC
UNSEWERED: 13.7 AC.
MAIN CENTRAL BASIN
SEWERED: 52.1 AC
UNSEWERED: 0
MAIN SOUTH BASIN
SEWERED: 103.3 AC
UNSEWERED: 28.0 AC.
HWY. 221 BASIN
SEWERED: 61.0 AC
UNSEWERED: 13.8 AC.
RV PARK BASIN
SEWERED: 13.3 AC
UNSEWERED: 0
FOSTER BASIN
SEWERED: 0
UNSEWERED: 218.0 AC.
KREDER BASIN
SEWERED: 0
UNSEWERED: 50.0 AC.
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DRN. TMT/BJF
CKD. JW
DATE JUN 2011
WESTECH ENGINEERING, INC.
CONSULTING ENGINEERS AND PLANNERS
CITY OF DAYTON, OREGON
SANITARY SEWER BASIN MAP
FIGURE
4-3
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4.4.3 Gravity Collection System

The City is served by a conventional gravity collection system that conveys wastewater to one of four pump stations. The original collection system was constructed in 1966 as part of the original sewer system. The original construction utilized primarily concrete pipe. The concrete joint type is unknown. Additions to the original system have utilized a variety of pipe materials including cast iron, and most recently PVC. In the late 1970's, the City embarked on infiltration reduction campaign that consisted of grouting manhole cracks, leaky main line concrete pipe joints as well as replacing many private sewer laterals. The City currently has approximately 42,700 feet of gravity sewer lines. As illustrated in Table 4-3 approximately 70% or 30,000 feet of the existing gravity sewer is concrete pipe, 28 % is composed of PVC, 1% cast iron, and 1% asbestos cement. The only asbestos cement pipe is located at the inverted siphon at Lippincott's Gulch. The current public works design standards allow only rubber gasketed PVC and ductile iron pipe for the construction of gravity sewers. These pipe materials have a long life and exclude most I/I from entering the collection system.

Table 4-3 | Gravity Sewer Pipe Inventory

			Pipe Size	
Pipe Material	**Pipe Length (ft)**	**% of Total**	**8-inch**	**6-Inch**
Concrete	29946	70%	29396	550
PVC	11883	28%	11083	800
Cast Iron	521	1%	521	0
Asbestos Cement	350	1%	350	0
		% of Total	**97%**	**3%**

As previously stated, the original collection system was primarily constructed with concrete pipe. Since installation, it has collected large quantities of groundwater and stormwater during wet periods of the year. The soils beneath Dayton consist generally of sands, silt and clay material. These soils are permeable and the groundwater levels vary from about 35 feet below ground surface (BGS) in the summer to about 7-8 feet BGS during the winter months. In the winter, when groundwater levels are high, many of the sanitary sewers are beneath the seasonal groundwater and collect large amounts of infiltration. Infiltration and inflow (I/I) is disruptive because it can overload the sewage lift stations, forcemains, and the stabilization ponds. Because the topography of Dayton is relatively flat and there are few basements, surcharged sanitary sewers have not traditionally been a major problem. However, this last spring there was a back up into the high school, due to high I/I flows that overloaded the gravity collection system below the 9^{th} St. Pump Station. As discussed later in this section, groundwater levels have a tremendous influence on the I/I flows within the sewer system. Major portions of the sewer system lie beneath the static groundwater level during most of the winter months, and many of these sewers collect large amounts of infiltration.

4.4.4 Existing Pump Stations

Wastewater is conveyed by the gravity collection system to one of four pump stations as discussed above. Table 4-4 contains a summary of some of the important characteristics of each of the pump stations. A more detailed description of each of the stations is presented in the following sections.

Table 4-4 | Summary of Existing Pump Stations

Category	9th Street Pump Station	Palmer Creek Pump Station	Highway 221 Pump Station	Main Pump Station
General				
• Basin served	9th Street	Palmer Creek	Palmer Lane	All Basins
• Construction date(s)	1965-66	1965-66	1965-66	1966 Modified 1982
• Type	Wetwell	Wetwell	Wetwell	Wetwell/Drypit
• Rated Capacity	± 266 gpm @ 60 ft TDH (±532 gpm both on)	± 111 gpm @ 90 ft TDH (±222 gpm both on)	± 266 gpm @ 60 ft TDH	±900 gpm @ 73 ft TDH (±1,800 gpm both on)
Wetwell				
• Type	Concrete bottom	Concrete w/hopper bottom	Concrete	Concrete
• Size	7 ft diameter	5 ft diameter	4 ft diameter	4 & 6 ft diameter
• Rim Elevation	±158 ft.	104 ft.	±120.92 ft.	109.5 ft.
• Influent Invert Elev.	±144 ft.	92.88 ft.	±111.67 ft.	8" (N) = ± 83.80 ft. 8" (E) = ± 95.02 ft
• Effluent Invert Elev.	±154 ft.			± 78.80 ft.
• Bottom Elev.	±137 ft.	89.33 ft.	±106.92 ft.	± 78.80 ft
• Depth (Rim to Bottom)	±21 ft	14.67 ft.	±14 ft.	±30 ft.
Pumps				
• Type	Submersible	Submersible	Submersible	Dry Pit Vertical Mount
• Number	2	2	1	2
• Manufacturer & Model	Reliance Model P18G2703E FLYGT	FLYGT 3129.09X2	FLYGT	Cornell Model # 4X4X14TRHV630 & Paco Model # 4X4X14TLHVC30
• Motor Size & Speed	7.5 HP & 10 HP	10 HP 1735 RPM	10 HP	30 HP 1165 RPM
• Power Supply	240-Volt 3-Phase	240-Volt 3-Phase	240-Volt 3-Phase	240-Volt 3-Phase
Force Main				
• Size & Type	6"	3" PVC	6" PVC	8" C900
• Length	± 520 ft.	± 512 ft.	± 540 ft.	± 2,120 ft
• FM Discharge	MH # 17	MH # 120	MH # 76	Headworks
• FM Discharge Location	Ferry & 8th St.	7th St. south of Rodeo	South of 3rd & Mill St.	WWTP Lagoons
• FM Discharge Elev.	±158.66	±155.8	± 131.18	± 119
Hydrogen Sulfide Control	none	none	none	none
Auxiliary Power				
• Type & Location	None	Cummings, fixed onsite	None	None
• Fuel Supply	N/A	Diesel (120 gallon)	N/A	N/A
• Transfer Switch	Pigtail for mobile generator hookup	Automatic		Pigtail for mobile generator hookup
• Trans. Switch Rating	unknown	400 AMP		unknown
Telemetry	Not fully installed	Not fully installed	Not fully installed	Not fully installed
Overflow				
• Location	MH # 34 Rim 9th & Ferry Streets	In Wetwell	In Wetwell	MH # 19 (Main St btw Water St. & 1st St.)
• Elevation	Rim El. ±156.6 ft.	8 inch IE = 97	8" IE El. ± 113.84 ft.	12"I.E. 90.09 ft.
• Discharge point	Storm drainage system	Palmer Creek	Palmer Creek	Yamhill River

4.4.4.1 9th Street Pump Station

The 9th Street Pump Station is located at the intersection of 9th Street and Ferry Street. This station is one of the original pump stations installed in 1966 and currently serves the 9th Street Basin. The station serves as a lift station to lift wastewater into the main gravity collection system where water flows by gravity to the Main Pump Station. The station is a wetwell type pump station with submersible, solids handling pumps mounted in the wetwell. This pump station mechanical system has not been significantly modified since originally constructed 1966, although the controls have been upgraded. One of the pumps was replaced in 2007. Currently, the City is working to add telemetry to this pump station. This station does not have temporary or a permanent auxiliary power generator. However, it does have pigtails and a manual transfer switch to connect a portable generator.

The pump station is equipped with two 7.5-horsepower, submersible, constant speed pumps. The pumps are Reliance Model P18G2703E. The pump motors are powered by 240-volt three phase power.

Pump operation is controlled by a float switch in the wetwell that controls two pumps (pump 1 and pump 2). From the bottom of the wetwell up, the float switches turn the pumps on and off at the following elevations. Pump 1 and 2 off at elevation = ±137.75 feet, start pump 1 and stop pump 2 at elevation = ±141.25 feet, and start pump 2 at elevation = ± 143.25 feet. The control system does not currently have an overflow alarm.

In the event of a prolonged pump station failure or long periods of heavy rainfall, sewage overflows can occur. Sewerage overflows occur out of the upstream manhole #34 lid. Sewage will flow from the manhole lid into the storm drainage system in Ferry Street that discharges upstream of Palmer Creek or into a private vacant lot. Currently, the City is in the construction stage of installing a new overflow that will be directly connected to the storm drain system, minimizing raw sewage spills onto the street and private property.

Westech and City personnel inspected the pump station in the spring of 2009. The equipment is in fairly poor condition. The pump station structure is over 40 years old. The pumping equipment and discharge piping are well over 40 years old and are showing signs of aging. The pumps and equipment are antiquated and replacement parts are becoming increasingly difficult to find. Discussions with Public Works revealed that both pumps will turn on and run constantly for several days during high flow events. This indicates that the station is undersized.

During most years, winter rains cause the groundwater levels to rise above the elevation of most of the collection system piping draining to the 9th Street Pump Station. Most of this gravity collection piping is old and accumulates large amounts of groundwater infiltration. During prolonged wet periods, the 9th Street collection system and pump station operate in a surcharged condition. According to Public Works, both pumps at the 9th Street Pump Station turn on and run continuously for several days in a row if not longer during high flow events. Typically, pump stations are designed to convey the peak flow to the station with the largest single pump out of service. The 9th Street Pump Station cannot accomplish this as demonstrated when one of the pumps went out in 2007 causing raw sewage to back up and flow down Ferry Street. During high-flow periods, the station cannot convey the peak flows with a single pump. Both pumps are required to convey peak flows. If one of the pumps fails again during these periods, sewage overflows could happen again. This station currently lacks adequate capacity and redundancy. Therefore, upgrades are necessary early in the planning period. These upgrades will be discussed in greater detail in **Section 6**.

4.4.4.2 Palmer Creek Pump Station

The Palmer Creek Pump Station was constructed in 2007 and is the newest pump station in the City. This pump station is located south of Joe Palmer Way and east of the Elizabeth Court cul-de-sac. The station is a duplex, submersible pump station. The pumps, inlet and discharge piping are housed in the wetwell. The pumps discharge into the existing 3-inch diameter, PVC forcemain that discharges into manhole #120. From this manhole, flow is routed by gravity through the Main South basin to the Main Pump Station. The forcemain is constructed at a continuously ascending grade from the pump station to the discharge manhole. There are no provisions to control the generation of hydrogen sulfide.

The pump station is equipped with two 10-horsepower, submersible, constant speed pumps. The pumps are ITT Flygt, model 3127.170 powered by 240-volt three phase power. The motors are directly coupled to the pump in a vertical configuration. The station is outfitted with an automatic transfer switch and a permanent, onsite, diesel generator to power the station in the event of a line power failure.

Pump operation is controlled by a Flygt control probes and FMC controller. The probe includes 10 sensors spaced at even intervals to sense the water level in the wetwell. Individual sensors are wired to provide pump on/off control and high level alarms. From the lowest to the highest the level probe provides pump off (89.83 feet), lead pump on (92.33 feet), lag pump on (92.83 feet), and high water alarm (93.33 feet).

In the near future, the City plans to install an autodialer and telemetry system. Conditions that will be monitored by the autodialer alarm system include the following.

- Overflow/Redundant Emergency High Water
- High Water Alarm
- Power Transfer (ATS switch to generator)
- Line Power Fail
- Power Fail (ATS)
- High Water Level
- Generator Fail

In the event of a prolonged pump station failure, sewage will flow out of the 8-inch diameter overflow (invert elevation =97 feet) pipe and flow southeast to discharge on the bank of Palmer Creek. Sewage will flow from the bank of Palmer Creek and eventually make it into Palmer Creek proper.

Westech and City personnel inspected the pump station in the spring of 2009. Since the pump station is very new there are no deficiencies at this time. The only item this pump station needs is to have the telemetry system completed. Therefore, upgrades should not be necessary through the planning period. The completion of the system wide telemetry system will be discussed in greater detail in **Section 6**.

4.4.4.3 Highway 221 Pump Station

The Highway 221 Pump Station is located on the east side of the Hwy 221 bridge on the southeast end of the City. The Hwy 221 Station was originally constructed in 1966 as part of the original collection system. The station is a wetwell type pump station with a single submersible, solids handling pump mounted in the wetwell. The City is currently working to finish installing the telemetry system. The pump station is situated just off the right side of the north bound lane, just before crossing the Hwy 221 Bridge. Vehicular access is poor, as Public Works staff must close the north bound lane of Hwy 221 to do any maintenance to the pump station.

The Hwy 221 Pump Station has only a single pump. Therefore, the station has no redundancy. The pump and discharge piping are housed in the wet well. The pump discharges into a 6-inch diameter, PVC, forcemain that is mounted beneath the Hwy 221 Bridge, spanning Palmer Creek. The forcemain discharges into manhole 76. From this manhole, flow is routed by gravity to the Main Pump Station. There are no provisions to control the generation of hydrogen sulfide.

The pump station is equipped with one 10-horsepower, submersible, constant speed, pump. The pump is manufactured by Flygt. The pump motor is powered by 240-volt three phase power. The pump station was originally constructed without provisions for auxiliary power.

Pump operation is controlled by a float switch in the wetwell. From the lowest to the highest, the float switch provides pumps off (Elev. = ±108.5 feet) and start pump (Elev. = ±111.75 feet). The control system does not currently have a high level or overflow alarm. The pump station does not currently have an active telemetry system.

In the event of a prolonged pump station failure, sewage will flow out of the 8-inch overflow pipe located in the wetwell. The 8-inch overflow pipe is routed so the overflow will flow by gravity and discharge on the bank of Palmer Creek.

Westech and City personnel inspected the pump station in the spring of 2009. The structural and mechanical equipment is in fairly poor condition. The pump station structure is over 45 years old. The pumping equipment and discharge piping are well over 45 years old and are showing signs of aging. The pump and equipment are antiquated and replacement parts are becoming increasingly difficult to find. Discussions with Public Works revealed that this pump runs constantly during high flow conditions, likely due to the fact that there is only one pump.

The Hwy 221 Pump Station does not have the required pump redundancy, lacks auxiliary power, and most of the pump equipment is antiquated, therefore upgrades are necessary early in the planning period. These upgrades will be discussed in greater detail in **Section 6**.

4.4.4.4 Main Pump Station

The Main Pump Station is located just northeast of the intersection of 1st and Ferry Streets in the northeast corner of the City adjacent the Yamhill River. The station was originally constructed in 1965 as a packaged wetwell/drypit Cornell pump station. In 1980, the pumps were upsized from 7.5 horsepower to 30 horsepower and the forcemain was replaced with a larger pipe. The pump station is situated in Ferry Street and has good vehicular access.

The Main Pump Station is a duplex, packaged station manufactured by Cornell. The wetwell has two 8-inch concrete inlets. The pumps and discharge piping are housed in a below grade 8-ft diameter steel chamber with a 3-ft diameter access tube and that is mounted on a concrete slab adjacent the concrete wetwell. The concrete wet well is approximately 30 feet deep and has two different diameters. The wet well has 48-inch diameter sections that sit on top of a larger 72-inch diameter bottom chamber that houses the two suction pipes for the pumps. The pumps discharge into an 8-inch diameter, C900 PVC, forcemain that discharges into the WWTP headworks. There are no provisions to control the generation of hydrogen sulfide.

The pump station is equipped with two 30 horsepower, self-priming, constant speed pumps. One pump is manufactured by Cornell and the other is manufactured by Paco. Both pump motors are 30hp and are powered by 240-volt three phase power. The motors are directly coupled to the pump. The pump station was originally constructed without provisions for auxiliary power. However, now the pump station has a pigtail and manual transfer switch to accommodate a mobile generator connection.

Pump operation is controlled by a Bubbletrol Control System. It is unknown at this time what level controls are set at in the wet well.

The pump station does not currently have an active telemetry or an autodialer alarm system.

In the event of a prolonged pump station failure, sewage will flow out the 12" overflow pipe in MH # 19 located just a half a block northeast of the 1st Street and Main Street intersection. Sewage will flow through the 12" overflow pipe a short distance and into the Yamhill River. At this time there is no alarm system to notify city personnel when an overflow occurs. The only way to know if an overflow occurs is to physically inspect MH# 19 during an overflow event.

Westech and City personnel inspected the pump station in the spring of 2009. The equipment is in reasonable condition considering that it is almost 30 years old. The Paco pump and motor had been rebuilt in the last few months. As such, the new pump and motor were in good condition. The older Cornell pump and motor are nearing the end of their life span. Both pumps are fairly antiquated, and replacement parts are expected to become more difficult to find.

An examination of the pump run times shows that at least one of the two pumps has been running constantly since the pump run-time meters were installed approximately one and a half years ago. The Main Pump Station operates in a surcharged condition during nearly all wet periods. This is caused by large amounts of groundwater infiltration into the gravity collection piping draining to the station. During high-flow periods, the station cannot convey the peak flows with a single pump. Both pumps are required. If one of the pumps were to fail during these periods, sewage overflows might occur. Therefore, this station does not have the required pumping capacity and redundancy, and upgrades are necessary early in the planning period. These upgrades will be discussed in greater detail in **Section 6**.

4.4.5 Raw Sewage Overflows

The City has reported a number raw sewage overflows in recent years. Including the 2005 calendar year, the City has reported 14 raw sewage overflow events. A detailed listing of these events is included in the NPDES permit evaluation report included in **Appendix B**. Of these 14 events, seven of the events occurred during dry weather conditions. This indicates that the cause of the overflows are often related to mechanical problems rather that high flows due to large amounts of infiltration and inflow. As noted above, there are significant shortcomings associated with most of the City's pumping stations. Some of the stations lack capacity to convey existing peak flows. In addition, some of the stations lack proper mechanical and electrical redundancy needed to ensure that overflows do not occur as a result of mechanical problems or power outages. Finally, some of the stations have not historically been equipped with telemetry equipment to enable operators to monitor high wetwell level conditions. Together, these shortcomings have led to numerous overflow events. In 2011, the City began the process of tying the telemetry systems at the pump stations to a central monitoring location. At the present time, all of the pump stations have been equipped with high level and overflow alarms with telemetry. This should help to reduce the occurrence of raw sewage overflows. In order to correct capacity problems and improve redundancy and reliability, the improvements described in Section 6 of this document will ultimately need to be implemented. Once implemented, the recommended improvements should eliminate the occurrence of most of the raw sewage overflows.

4.4.6 Sanitary Sewer Bridge, River or Stream Crossings

There are three locations in Dayton where sewer forcemains cross rivers, streams or bridges. These include the Yamhill River crossing (from the Main Pump Station to the WWTP), the Hwy 221/3rd Street bridge crossing over Palmer Creek (from HWY 221 Pump Station to gravity system), and the inverted siphon at Lippencott's Gulch. These crossings are key points in the collection system that are worthy of further discussion.

4.4.6.1 Existing Yamhill River Crossing

As shown in Figure 4-1, the 8-inch forcemain from the Main Pump Station to the existing headworks crosses the Yamhill River suspended under a pedestrian footbridge located at the east end of Ferry Street. The pedestrian footbridge is a 540-foot long wooden suspension bridge originally constructed in 1980. The pedestrian footbridge also supports the water main that conveys the domestic water from the watershed to the water distribution system.

In 2007, the City hired OBEC Consulting Engineers to inspect and evaluate the bridge structure and design some necessary structural repairs, which were completed in 2008. Also in 2008, OBEC completed a technical memo that includes life expectancy and cost analysis for the Ferry Street Bridge (see **Appendix E**). Per the OBEC memo dated 7/11/08, the pedestrian footbridge contains many timbers that are near the end of their useful life. Maintenance costs will continually increase as the bridge ages and may ultimately need to be replaced. It is anticipated that the timber glulam decking will need to be replaced sometime between 2011 and 2013, which is anticipated to cost the City approximately $200,000. Also, the timber bridge rail does not meet current safety design standards, and it is anticipated to cost up to $300,000 to upgrade the rail to current standards. Further, it is anticipated that some of the timber glulam columns, caps and girders will need to be replaced between 2018 and 2020, with estimated costs exceeding $200,000.

OBEC estimated the replacement cost of the Ferry Street Bridge would range from approximately $3.8 to $6.9 million, depending on the replacement bridge type. OBEC also mentioned that the design life of a bridge of this type (located in the Pacific Northwest) is 35 years. The bridge is now 30 years old.

The Newberg-Dundee Bypass Tier 2 Draft Environmental Impact Statement (released in June 2010) includes a plan to replace the existing pedestrian bridge with a new vehicular bridge at the same location (i.e., to connect Kreder Road and Ferry Street across the Yamhill River). See Chapter 6 for more discussions.

Since the forcemain from town to the treatment plant is connected to the footbridge, any improvements to the bridge must take into consideration the forcemain piping.

4.4.6.2 Palmer Creek Bridge Crossing (Hwy 221 Bridge)

As shown in Figure 4-2, a 6-inch sanitary sewer force main crosses Palmer Creek at the Hwy 221 Bridge. The forcemain is suspended under the bridge. The sanitary sewer force main conveys sewerage from the Hwy 221 Pump Station to the gravity sanitary sewer distribution system west of the bridge. The Hwy 221 Bridge is a concrete vehicular bridge constructed by ODOT in 1984. In addition to the sanitary sewer force main, the ODOT bridge also supports a water distribution line that conveys water from the distribution system to the portion of Dayton south of Palmer Creek.

The existing 6-inch sanitary sewer force main crossing of the ODOT bridge is in good condition, and is adequately sized to convey the projected sewage flows required for the south portion of Dayton.

4.4.6.3 Lippencott's Gulch Crossing

As illustrated in Figure 4-2, a 6-inch asbestos cement inverted siphon connects the school property on the west of Lippencott's Gulch to MH 78, east of the Gulch. This inverted siphon was originally constructed in 1965 and served the school property. The siphon is at capacity, but the condition or its exact location is unknown. Due to these issues, in 2009 the school district, as part of the expansion project, connected the new building and concession stands to MH #130 in Elizabeth Court located in the Palmer Creek Basin. If the school district expands in the future or transfers the existing sanitary flows from the siphon to the new connection at MH # 130, the district may be required to upsize the Palmer Creek Pump Station. If the flows are eliminated from the siphon, the City would abandon it due to the high replacement costs associated with lack of maintenance access

4.4.7 Infiltration and Inflow

The collection system is typical of many western Oregon sewer systems in that it experiences higher flows during the winter months because of infiltration and inflow (I/I). Base sewage flow estimated during the months of August and September is approximately 0.245 MGD. The I/I measured during the field investigation (discussed in the section below) was approximately 0.888 MGD. This increase in flow is strongly related to precipitation. The wastewater flows respond quickly and tend to track precipitation over time. There is typically a direct correlation between rainfall and wastewater flow.

The I/I problem is significant and is a major concern to the City. High I/I flows are problematic for a number of reasons. I/I utilizes reserve capacity of the gravity collection system that could be used to convey municipal sewage. Also, since all of the wastewater in Dayton is pumped to the treatment plant, I/I increases pump run times and power costs. Finally, I/I is a burden to the treatment facilities since it must be treated and discharged as though it was wastewater.

4.4.7.1 Field Investigation

Westech personnel measured I/I in several locations throughout the collection system during the late evening and early morning hours of March 31, 2010. The field work was completed immediately following a major winter storm. Field investigations included manhole inspections, and spot checking instantaneous flows in the sewers.

Flow mapping is a method used to help identify those portions of the collection system which contribute the most I/I and to quantify the amount of I/I originating within a section of the collection system. It consists of measuring the instantaneous flow in the sewer at strategic manholes in the collection system. The work is typically performed between midnight and 6 A.M. when sanitary flows are lowest. This allows the assumption that all flow in the collection system is from I/I. Typically, flow mapping is completed either during, or immediately following, a major winter storm so the I/I contributions are highest. The results of the flow mapping are shown on Figure 4-5.

Based upon the field investigations and discussions with public works the following statements can be made regarding I/I and the City's efforts to reduce I/I. The reader is encouraged to refer to Figure 4-5 during the following discussion.

- While the City has some base I/I throughout the winter months, the vast majority of the I/I is either direct inflow or rainfall induced infiltration. Once the soil is wet during the winter and early spring, a major rainstorm will result in increased flows observed at the WWTP within a few hours after the precipitation starts.

- Though I/I flows decrease after a major storm, prolonged dry periods (i.e., a few weeks) are required before flows to the WWTP return to near summer levels. This suggests that a significant volume of groundwater must be drained before I/I levels return to low flow values. Conceptually, one can envision that the I/I flows slowly drain a relatively large storage reservoir.

- The newer portions of the collection system that are constructed of PVC pipe materials contribute very little I/I.

- Inadequate pump station and trunk sewer capacity results in surcharging in the lower portions of the Main North, Central and South collection basins (areas close to the Main Pump Station).

- Surcharging of the lower portions of the Main South Basin downstream of the 9th Street Pump Station to manhole 9 occurs during very large storms as a result of inadequate capacity in the 8-inch trunk sewer. This also causes sewage to backup into the school.

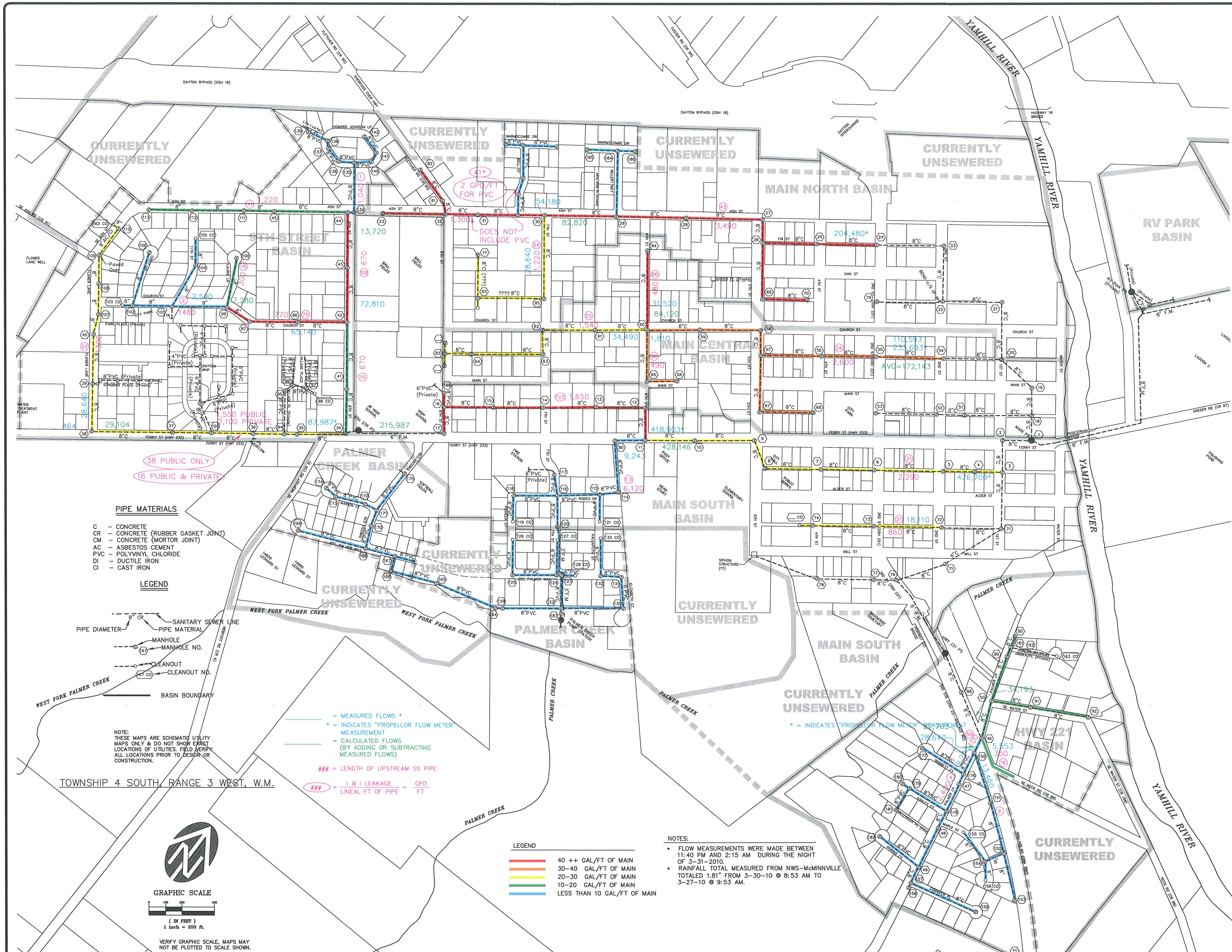
- I/I originates from all major components within the collection system – manholes, service laterals and sewer mains.

- Of 14 manholes that were inspected and were not surcharged, 8 manholes (approximately 60%) were observed to leak at various locations.

- Within Main North Basin collection system, approximately 98% of the I/I originates within the existing concrete pipe segment from manhole 33 to manhole 24. This particular pipe segment represents nearly all of the overall length of sewer main in this basin. Therefore, initial I/I reduction efforts might best be focused on this pipe segment.

- Within the Main South Basin collection system, the most severe I/I problems were observed between manholes 17 and 11. Therefore, future I/I reduction efforts might best be focused in this area.

- Total I/I measured during the field investigation was 0.888 MGD.



MAP UPDATED: 7-16-07

WESTECH ENGINEERING, INC.
CONSULTING ENGINEERS AND PLANNERS
3841 Fairview Industrial Dr. S.E., Suite 100, Salem, OR 97302
Phone: (503) 585-2474 Fax: (503) 585-3986

CITY OF DAYTON, OREGON

I / I MEASUREMENT
MARCH 31, 2010

FIGURE 4-4

JOB NUMBER 2609.3010.0

NO.	DATE	DESCRIPTION REVISIONS	BY
1			

SCALE HORIZ: VERT:
DSN. DM
DRN. DM/TJM
CKD. DM
DATE: JUN 2011

4.4.8 Description of Known Existing Collection System Deficiencies

Problems with the Collection System were identified from meetings and discussions with City staff and from field investigations. During major winter storms, the lower portions of the collection system surcharge due to a combination of inadequate pump station capacity, inadequate trunk sewer capacity, and large amounts of infiltration and inflow. The shortcomings in the existing system can generally be divided into the following categories; lack of capacity, end of useful life, and high infiltration and inflow. A short discussion of each of these categories follows. The deficiencies listed in this section are largely based on field observations and operational problems. Since components of the collection system (i.e., gravity collection piping) are not monitored on a full-time basis, this list of deficiencies should not be considered all-inclusive. As described in **Section 6**, several additional collection system deficiencies exist that are revealed through quantitative analysis.

- Lack of Capacity. This type of problem results from pipes that are too small to handle the peak sewage flows. This problem is a result of peak sewage flows increasing either due to development upstream or deterioration of the upstream system (i.e., increased I/I). Portions of the lower gravity collection piping appear to lack the capacity to convey peak flows.

- End of Useful Life. This type of problem is the result of old, damaged, or worn out facilities that no longer function as designed. The most common example of this type of problem includes broken or collapsed pipes. The correction of these types of problems requires replacement or reconstruction of the existing system.

- High Infiltration/Inflow. I/I flows in the collection system utilize capacity in the sewer mains which was intended for sanitary sewage. Unusually high amounts of I/I result in surcharged sewers, abnormally high pump run times and bypasses. As stated in prior sections, the lower end of the collection system surcharges. The systems operate in a surcharged condition for weeks at a time during all but the driest years.

  High amounts of infiltration and inflow is far and away the most significant problem in the City's collection system. It is the underlying cause of the capacity problems in the trunk sewers and pump stations. The recommended I/I correction measures are presented in **Section 6**. Table 4-5 illustrates the major known problem areas, as well as the category that the problem falls under.

Table 4-5 | Known Collection System Problem Areas

Location	Problem Category
1966 Concrete Collection Piping	End of Useful Life / High I/I / Lack of Capacity
9th Street Basin	High I/I
9th Street Trunk Sewer	High I/I / Lack of Capacity
9th Street Pump Station	Lack of Capacity / End of Useful Life
Hwy 221 Basin	High I/I
Hwy 221 Trunk Sewer	High I/I / Lack of Capacity
Hwy 221 Pump Station	Lack of Capacity / End of Useful Life
Main North Basin	High I/I
Main North Trunk Sewer	High I/I / Lack of Capacity
Main Central Basin	High I/I
Main Central Trunk Sewer	High I/I / Lack of Capacity
Main South Basin	High I/I
Main South Trunk Sewer	High I/I / Lack of Capacity
Main Pump Station	Lack of Capacity / End of Useful Life
Palmer Creek Pump Station	Lack of Capacity
Inverted Siphon from Elementary School	Lack of Capacity/End of Useful Life/Lack of Access

4.4.9 Collection System Non-Compliance Issues

The City has not received any Warning Letters from DEQ in recent years regarding problems in the collection system. However, the City has received Warning Letters for the 9th Street Pump Station & HWY 221 Pump Station overflows.

There are a number of areas in the collection system that will likely experience compliance problems unless significant upgrades are completed within the planning period. These include the replacement or reconstruction of over-capacity and faulty sewers that contribute large amounts of I/I. Continued I/I control efforts are needed in the collection system regardless if growth within the collection system occurs. The specific projects are discussed in more detail in **Section 6.**

4.5. WASTEWATER TREATMENT AND DISPOSAL SYSTEM

The City of Dayton owns, operates and maintains the wastewater treatment plant (WWTP) serving the City. The WWTP is located northeast of the City on the east side of the Yamhill River. The WWTP has four stabilization lagoons. The three primary lagoons normally operate in parallel, with the fourth lagoon serving as a polishing cell. The lagoons operate on a summer hold winter-discharge operational scheme. Treated wastewater is discharged through an outfall pipeline during the winter discharge season (November 1-April 30) to the Yamhill River. The plant was originally constructed in 1968, and has undergone one significant modification in 1980. In 1980 a new headworks structure and the three primary lagoons were constructed. The headworks includes a 6-inch Parshall flume for flow measurement and a composite sampler. In addition to the lagoons and headworks, the treatment facilities include two chemical buildings one with chlorine disinfection equipment and the other with dechlorination equipment. The plant also includes a chlorine contact chamber and effluent flow meter. A detailed listing of the design criteria for the treatment facilities is included in Table 4-6. The existing treatment plant plan is presented in Figure 4-5. The following sections provide brief descriptions of each to the individual unit processes that comprise the treatment facility.

Table 4-6 | Summary of Existing Treatment Facilities

Design Year	2000			
Design Population	2,295			
Design Flows				
• AAF	• 0.206 MGD			
• PAF	• 0.906 MGD			
Design Loads				
• BOD	• 390 PPD, 0.17ppcd			
• TSS	• 459 PPD, 0.20ppcd			
Lagoon Features (non aerated)	Lagoon # 1	Lagoon # 2	Lagoon # 3	Lagoon # 4
• Type	• Facultative	• Facultative	• Facultative	• Facultative
• Primary or Polishing	• Primary	• Primary	• Primary	• Polishing
• Normal Area	• 2.42 Ac	• 2.06 Ac	• 1.59 Ac	• 5.64 Ac
• High water Area	• 3.05 Ac	• 2.92 Ac	• 2.14 Ac	• 6.53 Ac
• Minimum Depth	• 3 Feet	• 3 Feet	• 3 Feet	• 3 Feet
• Maximum Depth	• 8 Feet	• 8 Feet	• 8 Feet	• 9.5 Feet
• Storage	• 4.46 MG	• 4.06 MG	• 3.04 MG	• 12.89 MG
• Minimum Freeboard	• 2 Feet	• 2 Feet	• 2 Feet	• 2 Feet
• Width at Dike Top	• 10 Feet	• 10 Feet	• 10 Feet	• 10 Feet
• Dike Slope	• 3:1 Feet	• 3:1 Feet	• 3:1 Feet	• 3:1 Feet
Influent Flow Measurement				
• Primary Device	• 6" Parshall Flume			
• Location	• Headworks			
• Measurement Range	• 0.0350 – 2.53 MGD			
• Meter Type	• Stevens Model AxSys CCR Flow Input Flowmeter System			
• Meter Range	• 0 – 2.53 MGD			
Influent Sampling				
• Location	• Headworks			
• Method	• Grab & Composite			
• Automatic Sampler	• SIGMA Portable Sampler			
Disinfection Facilities				
• Type	• Liquid Sodium Hypochlorite (5-gallon containers)			
• Location	• Disinfection Building			
• Chemical Feed Pump	• Pulsatron MP Series			
• Maximum Feed Rate	• 0.66 gph			
• Control System	• Manual			
• Injection Point	• Chlorine Contact Chamber Inlet			
• Contact Chamber	• Cast-in-Place Concrete Tanks with Baffles			
• Contact Volume	• 20,000 gallons			
• Minimum Contact Time	• 38 minutes at 0.906 MGD			
Dechlorination Facilities				
• Type	• Calcium thiosulfate Solution (55-Gallon Drums)			
• Location	• Chemical Building			
• Chemical Feed Pump	• Grundfos Series DME 2-16 (Rating = 0.66 gph)			
• Maximum Feed Rate	• 0.19 gph			
• Control System	• Manual			
• Injection Point	• Effluent Measurement Weir			
Effluent Flow Measurement				
• Primary Device	• 60° V-notch Weir			
• Location	• Contact Chamber Outlet			
• Measurement Range	• 0.0167 – 0.933 MGD			
• Meter Type	• Stevens Model AxSys CCR Flow Input Flowmeter System			
• Meter Range	• 0 – 1.0 MGD			

Table 4-6 | Summary of Existing Treatment Facilities

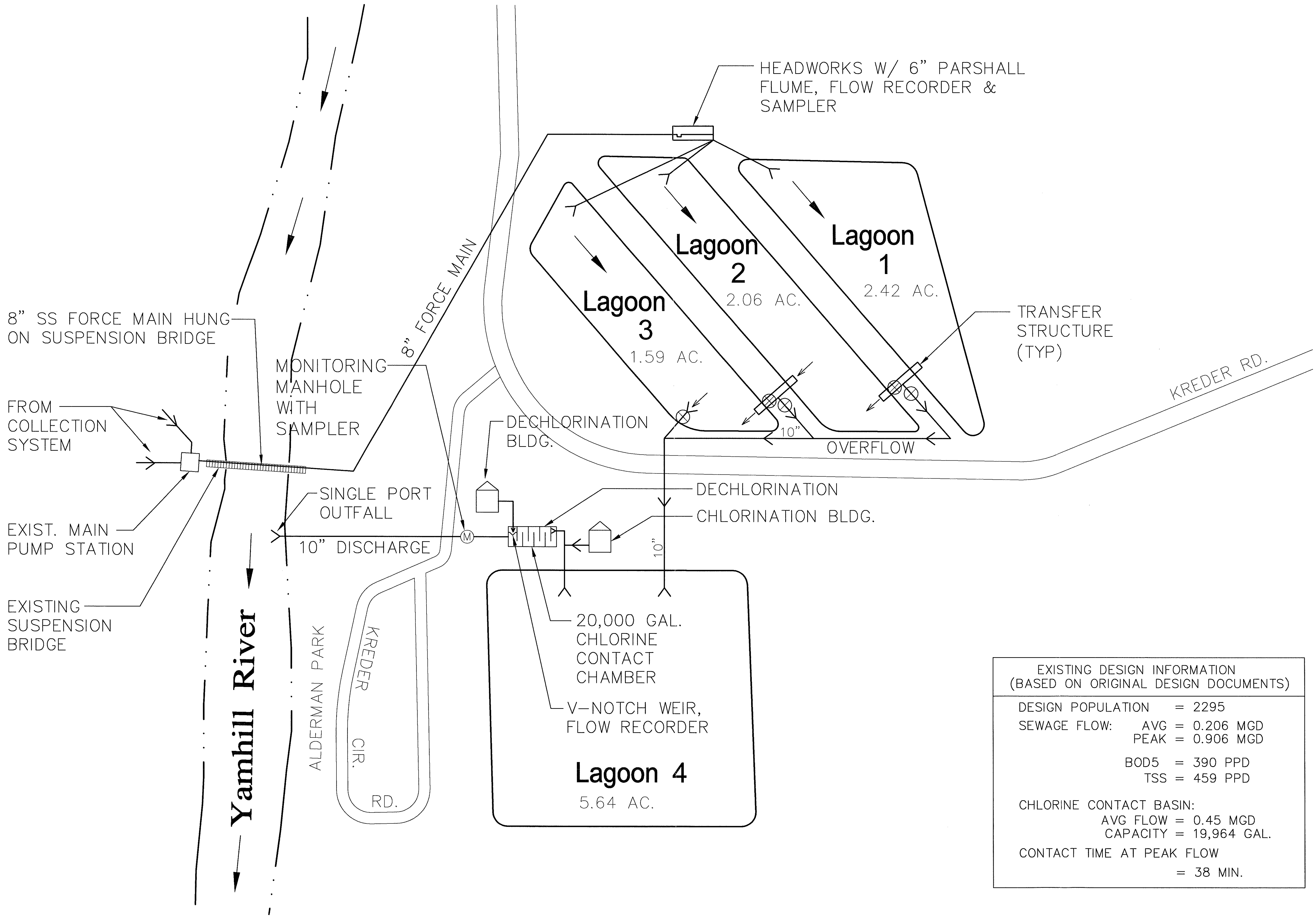
Effluent Sampling	
• Location	• Compliance Manhole
• Method	• Grab
• Automatic Sampler	• N/A
Telemetry System	None
Auxiliary Power	None
Outfall	
• Location	• Yamhill River
• Size & Type	• 10-inch Asbestos Concrete Single Port

4.5.1 Headworks

Discharge from the pump stations is delivered to the treatment facilities through an 8-inch diameter force main that discharges into the headworks structure at the WWTP. The headworks structure provides flow measurement, sampling and distributes raw sewage from the forcemain into three treatment lagoon cells. The headworks is not equipped with an energy dissipater at the forcemain discharge point or any screening, nor grit removal facilities.

Influent flow is measured at the 6-inch Parshall flume. A 12-inch stilling well is connected to the flume which houses the Stevens AxSys CCR Flow Input Flowmeter System. The Stevens Flowmeter System measures the depth of water in the 12" stilling well which corresponds to a flow value in the Parshall flume. The influent flow is measured by the float operated flow meter and recorded instantaneously via the integral 6" circular chart recorder. Total flows are recorded on the totalizer. There is no telemetry system for the remote monitoring of the flow signal. Over the past several years the influent flow metering system has experienced chronic inaccuracy leading to unrealistic flow measurements. Therefore, a new flow measurement system is recommended.

The headworks is equipped with three outlet pipes, with each pipe directed to a separate existing primary facultative lagoon. Flow from the headworks can be directed into either two or three of the existing primary lagoons by closing valves on the outlet piping discharge lines. Under normal operating conditions, flow is equally split to all three lagoons. Flow from the headworks is conveyed through three 6-inch discharge pipes to the three lagoons. Each of the discharge pipes has a single discharge port per Figure 4-5. The headworks is in good condition but is undersized to handle existing flows. This problem will get exacerbated due to the increased flow expected during the planning period. Please refer to **Section 7** for the recommend improvements.



EXISTING DESIGN INFORMATION
(BASED ON ORIGINAL DESIGN DOCUMENTS)

DESIGN POPULATION = 2295

SEWAGE FLOW: AVG = 0.206 MGD
PEAK = 0.906 MGD

BOD5 = 390 PPD
TSS = 459 PPD

CHLORINE CONTACT BASIN:
AVG FLOW = 0.45 MGD
CAPACITY = 19,964 GAL.

CONTACT TIME AT PEAK FLOW
= 38 MIN.

CITY OF DAYTON

EXISTING WASTEWATER TREATMENT FACILITY SITE PLAN

FIGURE 4-5

JOB NUMBER 2609.3010.0

WESTECH ENGINEERING, INC.
CONSULTING ENGINEERS AND PLANNERS
3841 Fairview Industrial Dr. S.E., Suite 100, Salem, OR 97302
Phone: (503) 585-2474 Fax: (503) 585-3986
E-mail: westech@westech-eng.com

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

DSN. DM
DRN. BF
CKD. DM
DATE: MAR 09

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REVISIONS

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R:\Dwg\Dayton City of\Wastewater Facilities Plan 2009\8-12-11\FIG 4-6.dwg (FIG 4-5 tab)

4.5.2 Facultative Lagoons

The four facultative lagoons provide biological treatment and biosolids digestion for the wastewater. The ponds were constructed using the native onsite clay soils. The lagoons have no synthetic liners. The first three lagoons were designed to operate in parallel, while the forth lagoon serves as a polishing cell. However, depending on the water elevations in lagoons 1, 2 and 3, two of the three primary lagoons (1 & 2, 1& 3, or 2 & 3) may be operated in parallel, while lagoon 4 serves as a polishing cells. The lagoons are intended to provide both storage of wastewater during the non-discharge periods and treatment to secondary standards. Lagoons 1 to 3 are designed to operate between a minimum depth of 3-feet and a maximum depth of 8-feet (storage depth of 5 feet). The fourth cell is designed to operate between a minimum depth of 3-ft and a maximum depth of 9.5 feet (storage depth of 6.5 feet). These design parameters provide for two feet of freeboard from the top of the dikes. Please refer to Table 4-6 for additional information.

Lagoon 1, 2, and 3 are 2.42 acres, 2.06 acres, 1.59 acres, respectively, while cell 4 is 5.64 acres. The total water surface area for all four cells is 11.71 acres. Under normal operation, flow is equally split between the first three cells at the headworks structure. The lagoon inlet piping was reconstructed in 1980 and includes single ports to distribute wastewater to each lagoon. The maximum water level in each lagoon is controlled by a control structure that allows flow from the lagoons to operate in parallel or series. The overflow piping is an unvalved 10-inch diameter pipe connected to the control structures. Therefore, if the water level in one lagoon rises above the inlet elevation of the overflow pipe, water will flow down gradient from Lagoons 1 through 4 sequentially. Overflow of this nature will only occur when the water in one lagoon is above the overflow elevation. Under this scenario, the water level will cascade down to lagoon 4. Lagoon 4 does not have overflow piping, so an overflow would overtop the dike and eventually flow into the Yamhill River.

Treated effluent is withdrawn from lagoons (1-3) at a point near the south end of each lagoon. Sluice gates are used to control the flow of effluent from each lagoon. Effluent from the Lagoon 4 outlet structure flows through a 10-inch diameter pipeline to the chlorine contact chamber.

The three primary lagoons have been in service since 1980, while lagoon four has been in service since 1968. In 1980 the dikes of lagoon four were raised 4.5 feet. In 2008, DEQ and City noticed a wet spot along the outer east bank of lagoon four. The City hired Foundation Engineering to investigate the seepage. The Preliminary Geotechnical Investigation dated October 7, 2008 revealed that water was traveling from the lagoon along the construction joint between the top of the 1968 dike and the bottom of the 1980 dike addition, to the surface of the dike exterior (refer to Appendix H). The investigation recommended monitoring the leakage and if it worsened, then measures would be required to fix the leak. Since then the leakage has not increased. However, if the City uses this lagoon in the future improvements, this leak should be fixed.

Access during winter months can be challenging as the lagoons existing gravel roadway surface has deteriorated. If the existing lagoons are used, the existing access roads along the lagoons will need a new all weather surface.

During the summer of 2010 the City conducted a lagoon leakage test. The leakage measured was under the maximum allowable 1/4-inch per day, the DEQ threshold for existing lagoons. The details of the leakage test are described in **Section 4.5.2.1**.

Biosolids tend to accumulate in the lagoons over time. Generally, the majority of the biosolids in lagoon systems collect in the primary cells. Therefore, we would assume that the first three lagoon cells would have the majority of the biosolids. Based on inspection, the facultative lagoons have accumulated a biosolids over the years that will need to be removed early in the planning period. We have estimated a budget amount of $400,000 to remove the existing biosolids. However, the City should conduct a biosolids survey early in the planning period to determine the quantity of biosolids in the lagoons. Once the quantity of biosolids are determined the biosolids removal budget can be further refined.

Since 2007, the City has had to either discharge before the start of the discharge season (November 1st) in order to avoid overtopping the lagoons, or extend the discharge season past May 1st to draw the lagoons down to low operating levels in order to maximize storage for the summer hold period. These discharges out of the allowed discharge season demonstrate the lack of hydraulic treatment capacity of the existing lagoons. Also, based on the existing BOD loading rates that enter the lagoons, the loading rates exceed the maximum design threshold of 50 pounds BOD per acre per day for primary cells and 35 pounds BOD per acre per day for all the lagoons. Therefore, it appears that the lagoons are over loaded with BOD.

As discussed in greater detail in **Section 7**, the existing lagoons lack hydraulic storage and organic treatment capacity. This should be expected since the existing flows, loads and population exceed the values used for the design of the facility. The City should expect that the WWTP will continue to have permit compliance issues in the future. Please refer to **Section 7** for the recommended improvements.

4.5.2.1 Lagoon Leakage Testing Results

From August 17 through August 26, 2010, Westech Engineering staff completed field measurements to determine the leakage rate of the existing lagoons. The field measurements included lagoon evaporation, rainfall, influent flows, and lagoon water levels. The seepage rate from the lagoons was determined by completing a water balance. Leakage testing was restricted to August because rainfall could be eliminated from the water balance, minimizing the introduction of error.

Evaporation was measured using a standard US Weather Bureau steel pan 47.5-inches in diameter by 10-inches deep and a hook gauge with graduations in one-thousandths of an inch. Evaporation from an evaporation pan occurs at a faster rate than that of a lagoon, therefore a pan coefficient of 0.70 was used to correlate the measured pan evaporation rate to the lagoon evaporation rate. The lagoon water levels were monitored in Lagoon 3 and 4 by the separate stilling wells. It was assumed that the leakage of Lagoons 1 though 3 would be similar due to the close proximity and similar construction. No rainfall was measured during this time period.

The water balance was completed and the seepage rate for cells 1 though 3 and cell 4 was 0.23-inches per day and 0.16 inches per day, respectively. Current DEQ standards require that the seepage rate from existing lagoons not exceed ¼ inches per day. For new lagoons, DEQ requires a seepage rate less than 1/8 inches per day. Based on the leakage testing results, all four lagoons leak more than 1/8-inch per day but less than the maximum allowable ¼-inch per day. If these lagoons are to be incorporated in future plant expansions, the City may want to consider lining the lagoons to avoid any potential leakage problems in excess of ¼-inch per day. As noted above, the City should plan to remove the biosolids in the lagoons during the next planning period. It is likely that the accumulated biosolids are helping to seal the bottom of the lagoon. Therefore, the seepage rates from the lagoons may increase when the biosolids are removed.

4.5.3 Chlorination Building and Chlorine Disinfection Equipment

The lagoon effluent was disinfected by a chlorine gas disinfection system when the treatment plant was expanded in 1980. In 2002 the City replaced the gas chlorination system with liquid sodium hypochlorite system. The disinfection equipment is housed in a chemical feed building. The chemical building is a single room wood frame building with inside dimensions of approximately 6-ft by 6-ft. The building is located adjacent the chlorine contact chamber. The building houses the bulk sodium hypochlorite solution in 5-gallon containers, a 40 gallon temporary hypochlorite tank and a manual chemical feed pump. The dosage and feed rate are set manually at the chemical feed pump which is adjusted based on the flow through the chlorine contact chamber. Chlorine contact time is provided in the contact chamber. The contact chamber is a baffled concrete structure that provides approximately 20,000 gallons of contact volume. The chemical feed building, chlorination equipment, effluent flow meter, and chlorine contact chamber were all constructed as part of the 1980 WWTP expansion. These facilities have not been significantly modified over the years and largely operate as originally designed except for the dechlorination system.

4.5.4 Chlorine Contact Chamber & Effluent Flow Measurement

The chlorine contact chamber and effluent flow measurement system was originally installed in 1980. The concrete chlorine contact chamber is open to the atmosphere and has a total volume of 20,000 gallons. The peak flow capacity of the chlorine contact chamber, while still providing the 30-minute minimum contact time, is 0.760 MGD.

Effluent flow from the chlorine contact chamber exits through a 60° V-notch weir. The head over the weir is measured in a 12-inch stilling well that houses a level transducer that provides the input to the Stevens AxSys CCR Flowmeter System. The Stevens Flowmeter System measures the depth of water in the 12" stilling well which corresponds to a flow value in the V-notch weir. The maximum flow capacity of the 60° V-notch weir is 0.932 MGD.

4.5.5 Calcium Thiosulfate Dechlorination System

The dechlorination system is similar in many respects to the chlorination system. The dechlorination system includes carrier water, chemical feed pump, 55 gallon calcium thiosulfate drum, and an injection panel housed in the dechlorination building. The chemical feed pump is a Grundfos Series DME 2-18 that has a maximum feed rate of 0.66 gph. The design calcium thiosulfate feed rate is 0.19 gph, much less than the chemical feed pump capacity.

A 55-gallon drum of calcium thiosulfate, located in the dechlorination building, provides the source chemical. Calcium thiosulfate solution is withdrawn from the drum via a chemical feed pump. The calcium thiosulfate solution is mixed with the carrier water in the injector panel and pumped to the V-notch weir for dechlorination. Measurements to confirm dechlorination are taken at the compliance manhole located downstream of the V-notch weir.

4.5.6 Yamhill River Outfall

A 10-inch diameter pipe conveys effluent from the chlorine contact chamber to the Yamhill River as shown in Figure 4-5. The total length of drainage piping between the contact chamber and Yamhill River is approximately 310 feet. Treated effluent flows by gravity from the chlorine contact chamber to the outfall point in the Yamhill River. The existing outfall discharge is located above the ordinary high water level. Therefore, the outfall is exposed during most of the year. This is a shortcoming that does not conform to modern wastewater disposal practices. Typical design practice is to discharge treated effluent below the water level. This should be addressed during the planning period

4.5.7 Description of Existing Treatment System Deficiencies

The existing treatment facilities have functioned relatively well since their construction in 1980 with some minor maintenance and system upgrades. However, due to the fact that the existing wastewater flows and loads exceed the design wastewater flows and loads of the existing WWT,P the City will likely begin to experience permit compliance problems early in the planning period. Table 4-7 summarizes the known deficiencies within the treatment system. Refer to **Section 7** for the recommended improvements.

Table 4-7 | Known Treatment System Deficiencies

Location	Deficiency
Headworks	- Unreliable flow measurement equipment - Parshall Flume is under capacity
Facultative Lagoons	- Lack of hydraulic storage capacity - Lack of organic treatment capacity - Need to perform biosolids survey
Yamhill River Outfall	- Outfall is above ordinary high water level.

4.6. EXISTING SANITARY SEWER FUNDING MECHANISMS

Funding for the City's existing wastewater system comes from two major sources, user fees and system development charges (SDC). Since SDCs cannot be used to finance operation, maintenance (O&M), and replacement costs of a sewer system, the O&M and repair costs must be financed from the user fees.

4.6.1 Sewer User Fee

The City's sewer use regulations (**Appendix C**) provide the method for assessing sewer user fees. The sewer users are billed on a monthly basis for sanitary sewer service. Sewer fees are fixed and tied to the type of land use per Table 4-8.

Table 4-8 | Existing Base Sewer Rates

Description	Comments	2008/09 Rate
Single Family Residence	--	$25.00
Multi-Family & Mobil Home Parks	Per Unit	$25.00
Commercial		$22.00
Taverns/Restaurants		$31.00
Churches, Lodges & Clubs		$25.00
Hotels & Motels	Per room/unit	$25.00
Offices		$25.00
Laundromats	First washer	$19.00
	Each additional washer	$17.00
Schools:		
August 26-June 25		
• Elementary	Per 18 students (ADA[1])	$25.00
• Junior High	Per 16 Students (ADA)	$25.00
• High School	Per 11 students (ADA)	$25.00
June 26-August 25 w/out summer school		
• Elementary	X 1 EDU[2]	$25.00
• Junior High & High School	X 4 EDU	$25.00
June 26-Aug. 25 with summer school		
• Elementary	Per 18 students (ADA)	$25.00
• Junior High	Per 16 Students (ADA)	$25.00
• High School	Per 11 students (ADA)	$25.00
RV Park w/out wet facilities	First 2 spaces	$19.00
	Each additional space	$9.00
RV Park with wet facilities	Per space per month	$9.00
• Care taker office	Per unit	$19.00
• Park Facility	Kitchen, bathroom, up to 4 showers	$19.00
	Each additional shower	$9.00
• Laundry	First washer	$19.00
	Each additional washer	$17.00

(1) ADA = Average Daily Attendance as of January 1 of the current calendar year.
(2) EDU = Equivalent Dwelling Unit

The fixed fee structure shown in Table 4-8 allows unlimited usage per billing cycle for all users.

4.6.2 Sewer SDC

The City's sewer use regulations (**Appendix C**) provide the method for assessing sewer SDC's. The actual SDC fees are set by resolution of the Council. In July of 1999, the City updated the sanitary sewer SDC fees per Table 4-9. The 1999 table of all the utility SDC's are included in **Appendix C**. SDC fees are currently based on the water meter size. There are two parts to the SDC fee, the reimbursement fee portion and the improvement fee portion. The reimbursement part of the SDC goes into an account which can be used to reimburse developers for public sanitary sewer upgrades associated with private developments. The improvement portion of the SDC goes to the City and is used for future sanitary sewer improvements or upgrades.

Table 4-9 | Existing Sewer SDC Schedule

Meter Size	Reimbursable Fee	Improvement Fee	Total Sewer SDC
5/8 and 3/4 Inch	$483	782	$1,265
1 Inch	$643	$1,039	$1,682
1 ½ Inch	$966	$1,564	$2,530
2 Inch	$1,288	$2,085	$3,373
3 Inch	$1,933	$3,127	$5,060
4 Inch	$2,577	$4,170	$6,747
6 Inch	$3,866	$6,254	$10,120
8 Inch	$5,154	$8,339	$13,493