Chapter 6

Collection System

Chapter 6 – **Collection System**

6.1 Introduction

The following chapter details the evaluation process and findings for the City's existing sewer collection system. Generally the existing hydraulic model was updated to reflect the existing collection system and calibrated using flow monitoring data recorded by City-owned equipment deployed for this sewer plan update. This update occurred in two steps: Step One – South Richland Sewer Review (discussed further in **Section 6.2.2.1**); Step Two – Remaining Richland Sewer Review. The hydraulic model was then used to evaluate the City's sewer collection system under the following three scenarios, each of which is discussed in this chapter:

- Existing Model and Analysis (**Sections 6.2 & 6.3**): Represents the current collection system within the City limits. Capacity issues identified at this stage indicate problems under today's conditions.
- Committed Model (**Section 6.5**): Represents the development within the entire UGA boundary; essentially everything the City has committed to serve and includes all parcels. Capacity issues identified at this stage indicate there may be problems upon build-out to the City's UGA boundary.
- Master Plan Model (**Section 6.6**): Represents the ultimate build-out of the future wastewater service area by including additional study areas identified by the City.

The above three scenarios also identify the modeling sequence that was followed for evaluating the City's sewer collection system: Existing System followed by the Committed Model and then the Master Plan Model. With the completion of the first two scenarios, an issues list was prepared to highlight the findings and is included in each discussion section.

The City's sewer collection system consists of manholes, gravity pipes, clean-outs, lift stations and force mains. The gravity collection pipes range in diameter from 8-inch to 54-inch and comprise a network of nearly 263 miles. The City's 14 lift stations act as local collection points for sewer flows within an area that cannot be served by extension of the existing gravity collection system. Lift station pumps convey wastewater through force main pipes that have a combined total length of greater than 5.5 miles. The lift stations are discussed in more depth in **Section 6.9**.

6.2 Existing System Model

6.2.1 General

The existing hydraulic model in this study was built using the City's GIS data and survey records and then analyzed using InfoSWMM modeling software. The City's previous comprehensive plan update (2004) utilized Hydra modeling software. For this update, InfoSWMM was chosen to provide a GIS-based modeling platform well suited to integrate the existing City data and the hydraulic model and to also provide a more sophisticated hydraulic modeling engine. The existing hydraulic model's primary purposes are to:

- Provide a snapshot of the current collection system flows
- Identify potential existing capacity issues
- Provide a platform for use in the Committed and Master Plan Models

The existing hydraulic model consists of two layers – 1) the System Layer and 2) the Flow Generation Layer. Each layer includes multiple parameters and corresponding assumptions that characterize the area and system being modeled. The assumptions are coupled with surveyed pipe inverts, record drawing data, flow monitoring data, characteristics learned from the physical system, similar studies done in the region, and general and historical knowledge gained through previous work for the City. Key assumptions used in the existing model are documented in **Appendix C**.

6.2.2 System Layer

The existing hydraulic model System Layer consists of the manholes, gravity sewer pipes, force mains, and lift stations in the collection system. A map of the Existing System is found on **Figure A1** and a map of the Sewer Collection System Basins (as described in **Section 1.4**) is found on **Figure 1-1**.

6.2.2.1 Existing Collection System Layer

The existing collection system layer was first updated from the previous hydraulic model (2004 update) in April of 2014 for the South Richland Sewer Review (SRSR) project. The objective of the SRSR project was to conduct hydraulic modeling for the gravity sewer system downstream of the UGA annexation area and the Badger South planned development in South Richland and to identify master plan improvements and alternatives. (A technical memorandum discussing the SRSR project results is included in **Appendix B**.) The previous hydraulic model was used as the main source of information for rim elevations, invert elevations, pipe sizes, and pipe lengths. The City's record drawings from improvement projects after 2004 were used to update the hydraulic model to current conditions. As depicted on **Figure A2**, there were five main areas of improvement projects that were added to the model: 1) Physical Sciences Facility at PNNL, 2) Logston Sewer Interceptor, 3) RY Basin Improvements (new lift station and abandon two existing lift stations), 4) Leslie Sewer Trunk, and 5) Badger Mountain South Development. Any missing or questionable data was reviewed with the City and then supplemented with record drawings, field checks, or survey if necessary. Missing or questionable data for trunk pipes was resolved by using data from the previous model or by straight-grading individual sections of pipe (i.e., interpolating an invert based on upstream and downstream inverts). These manholes and pipes are tagged in the model accordingly.

A portion of the collector pipes (8-inch and less) were added into the existing hydraulic model to facilitate flow routing; however, invert data was not verified or resolved because none of the 8-inch pipes were analyzed for capacity. The 8-inch pipes in the model perform the sole function of routing flows into the model and no physical data or capacity data on the 8-inch pipes should be utilized for decision making purposes. Only pipes 10 inches in diameter and larger were analyzed for capacity in the hydraulic model.

6.2.2.2 Lift Stations

Lift station and force main data were added to the existing hydraulic model based on the previous model and by record drawings and discussions with City staff. **Table 6-1** lists the current 14 lift stations that are operating in the City and that are represented in the model. **Figure A1** depicts the locations of these lift stations. Also included in **Table 6-1** are the design operating point's which were obtained from the City and from the previous General Sewer Plan. The lift stations are discussed in more depth in **Section 6.9**.

Table 6-1 – Existing Lift Stations

Lift Station	Year Constructed/ Last Major Rehabilitation	Design Operating Point (one pump)	Pump Description	Wet Well Dimensions
Battelle	2013	400 gpm at 20-ft	Flygt, 5 hp, NP-3102 MT 3~ 465	12-ft Diam.
Waterfront	1977	600 gpm at 43-ft	Fairbanks Morse, 15 hp	6-ft Diam.
Terminal Dr	1981	150 gpm at 24-ft	Fairbanks Morse, 3 hp	8-ft Diam.
Mental Health	2009	260 gpm at 21-ft	Vaughan, 5 hp, SP4C, 1170 rpm, 8.9in Imp	6-ft Diam.
Bradley	1999	180 gpm at 59-ft	Flygt, 10 hp, NP-3127	10-ft Diam.
Columbia Pt	2010	270 gpm at 52-ft	Flygt, 6.5 hp, NP-3102 SH 3~ 256	10-ft Diam.
Wellhouse Loop	1978	100 gpm at 12-ft	Hydromatic, 1.5 hp	6-ft Diam.
Duportail	1995	200 gpm at 45-ft	Flygt, 7.5 hp, NP-3127 HT 3~ 489	6-ft Diam.
Montana St	2015	970 gpm at 105-ft	Smith & Loveless, 30 hp, 4B3 1760 rpm	8-ft Diam.
Columbia Park Trail	2012	400 gpm at 50-ft	Flygt, 10 hp, NP-3127 MT 3~ 438	8-ft Diam.
Meadows South	1970's	100 gpm at 20-ft	Hydromatic, 3 hp	7-ft Diam.
Bellerive	2005	260 gpm at 80-ft	Flygt, 15 hp, NP 3153.180 HT, 229mm Imp	6-ft Diam.
Meadow Ridge	2007	245 gpm at 55-ft	Flygt, 10 hp, CP 3127.090 HT, 217mm Imp	6-ft Diam.
Dallas Rd	2012	260 gpm at 172-ft	Flygt, 35 hp, NP 3171 SH 3~ 277	8-ft Diam.

6.2.3 Flow Generation Layer

6.2.3.1 Water Meter Usage Data

Previous sewer modeling efforts assumed typical sanitary sewer unit flows based upon land use designation; however, a more precise method was utilized for the update of the existing hydraulic model. Sanitary unit flows for the existing model were based on recorded City water meter data from the period between December 2012 and February 2013. During these winter months, the vast majority of metered water used by customers is for potable use only (i.e. no irrigation) and discharged to the collection system. Therefore, use of meter data is a good indicator of base sanitary flow contribution. Since potable water service meter data was used, it provided actual usage data to generate sewer flows in the model rather than relying on typical unit flow data. This method yields a more precise representation of the existing flows in the system.

The average daily flow for each water meter was calculated from the average winter monthly volume recorded by each meter, yielding an average water use of 160 gpd per residential dwelling unit. Therefore an ERU was defined as 160 gpd. Based on 2.42 persons per household (reference **Section 2.11**), this yields a per capita flow of 66.1

gpd. Average daily flows were then adjusted by assumed peaking factors to reflect weekend and weekday diurnal curves. These diurnal curves were specific to each land use type and were adjusted during calibration of the model (See **Section 6.2.4**).

A majority of the City is characterized by residential flows. Since the highest average and peak residential flows usually occur on weekends, the majority of the trunk pipes will experience peak flows on the weekend; however, smaller basins with a high percentage of non-residential flows may experience peak flows during the weekdays. For example, a school generates the majority of its wastewater during the week, so the daily average was adjusted so that the majority of the flow is distributed throughout the week, and very little flow is distributed over the weekend. Therefore, the existing hydraulic model was built using factors to adjust the average daily flows from the water meters to average weekday and weekend flows to capture both maximum peak possibilities.

Currently, the City collects water meter data on a monthly basis, and the data is reported as a volume in hundreds of cubic feet. In the case of not being able to access a water meter for any reason (blocked by car, covered in snow, other) the City estimates the monthly value (on the lower end). The next month's meter reading might require a correction, however typically not a negative value.

6.2.3.2 Land Use

The land use types used in the existing hydraulic model are listed in **Table 6-2** and were generated from the land use codes provided with City water meter data.

Table 6-2 – Existing Model Land Use Types

➢ Assisted Living	➢ Open Space
➢ Church	➢ Public
➢ Commercial	➢ Residential - High Density
➢ Hospital	➢ Residential - Medium Density
➢ Hotel	➢ Residential - Low Density
➢ Industrial	➢ Restaurant
➢ Office	➢ School

The Residential Low Density land use type consists of all single-family dwelling units. Residential Medium Density consists of multi-family dwelling units with between two and four dwelling units, as well as condominiums, townhomes, mobile homes and RV parks. Residential High Density includes all apartments and multi-family dwelling units with greater than four dwelling units. Residential and non-residential unit flows are shown in **Table 6-3**.

Table 6-3 – Unit Flows

Parameter	Value
Residential Unit Flows [(a)]	
Low Density (Single family homes)	160 GPDU
Medium Density (Multi-family – 2 to 4-plex and condo/patio homes)	147 GPDU
High Density (Multi-family – >4-plex and apartments)	147 GPDU
Non-Residential Unit Flows [(b)]	
Assisted Living	3,300 GPAD
Church	150 GPAD
Commercial [(c)]	350 GPAD
Composite Commercial [(d)]	625 GPAD
Hospital	5,500 GPAD
Hotel	3,000 GPAD
Industrial [(e)]	60 GPAD
Industrial-Heavy [(f)]	3,000 GPAD
Office	350 GPAD
Public	540 GPAD
Restaurant	2,500 GPAD
School	170 GPAD

(a) *Based on 2.42 people per dwelling*
(b) *Based on winter water meter records divided by the net parcel area*
(c) *Includes range of commercial businesses, from convenience stores to big box stores.*
(d) *Combines commercial, hotel, restaurant, hospital, office, and public land-uses.*
(e) *Based on dry, non-significant, industrial flows only.*
(f) *Based on the 11 permitted City significant industrial users.*

Water meter data was coupled with land use to complete the flow generation layer. Although, the City's water meter data was supplied in spreadsheet format, the representing parcel number was also included and used to link the water meter to the parcel it served. Diurnal curves (the typical 24-hour shape of the flow) were then developed for each land use type. The initial diurnal curve patterns for each land use type were based on historical modeling experience and flow monitoring results. Diurnal curves for each land use type were then adjusted during calibration efforts to match the flow monitoring results. The diurnal curves used in the model can be found in **Appendix D**.

6.2.3.3 Infiltration and Inflow

Infiltration is groundwater entering the sewer through cracked pipes, faulty service connections or other deficiencies in the collection system. This can be groundwater from a high water table or rainfall induced groundwater. As mentioned in **Section 3.5**, the City has incorporated several inspection and rehabilitation measures to minimize infiltration. During calibration for the Existing Model, it was determined that there were specific areas within the City that accounted for more infiltration than others (as shown in **Figure 3-9**). This observation was confirmed by City staff.

Inflow is the flow of water directly into the sewer during and after a rainfall event due to direct connection to the sewer from storm drains, roof drains, parking lots, manholes, etc. The City has a low quantity of cross connections where storm drains or roof drains are connected to the sewer system. Therefore, inflow was assumed as 50 gpad.

6.2.4 Calibration

Calibration is the process of modifying various parameters and their assumed values in order to match flow monitoring data collected from multiple locations. Sewer flows were monitored at a total of eight locations in the collection system as shown on **Figure A3**. For the SRSR project (as described in **Section 6.2.2.1**) sewer flows were monitored at two locations, south of the Yakima River, between July 20th and August 8th, 2013. Six additional locations were selected and sewer flows were monitored between March 26th and May 12th, 2014. At each location, the City installed their flow monitoring equipment in the collection system manholes. During both flow monitoring periods there were no wet weather events with any measureable amounts of rainfall and therefore only dry weather flows were captured.

The flow monitoring data set, itself, has limitations that prevent a 'perfect' calibration between model output and real flows. Some of the factors affecting calibration include the level of uncertainty of the flow monitoring data and the low resolution of the water meter usage data. Additional limitations, not related to the flow monitoring data, include the diurnal curve patterns used, routing assumptions, normal fluctuations in wastewater flows from day-to-day, and the overall quantity of wastewater production. Considering these limitations, the model calibrated well, without significant changes to base assumptions or parameters, providing a high level of confidence in the existing model results and in the subsequent development of the master plan models. A complete listing of model assumptions and parameters are included in **Appendix C**.

6.2.4.1 Dry Weather Calibration

As discussed in **Section 6.2.3.1**, the model was calibrated to both weekend and weekday recorded flows. Recorded flows for individual days were plotted on a XY graph as the measured flow (in mgd) versus time (48 hour period of weekend and weekday) to show the uncertainty and variability of flow at any given point in the system. The flow monitoring data for the larger service areas showed less variability in flow values than the smaller service areas, which is a result of the number of customers upstream. An average weekend diurnal curve and average weekday diurnal curve were produced for each monitoring site based on the 30 minute average flow from the flow monitoring data. Final calibration graphs for dry weather flows are included in **Appendix D**.

6.2.4.2 Wet Weather Calibration

There were no wet weather events with any measureable amounts of rainfall during the flow monitoring period. Capturing a rainfall event during flow monitoring provides information regarding the impact the event has on the collection system at each flow monitoring site. Without a measureable event, a 3-hour short duration thunderstorm with a 2-year return period and a total rainfall amount of 0.424 inches (as per the DOE Stormwater Management Manual for Eastern Washington (SMMEW)) was used as the design storm. The short duration thunderstorm has a higher peak flow as compared to the 24-hour design storm. To simulate a worst-case condition in the model, the peak inflow from the storm event was aligned with the peak in the sanitary flow on the weekend. This results in a larger net return period for the storm event. The existing model includes a simulated rainfall event from the design storm. Final graphs for an assumed wet weather event are included in **Appendix D**.

6.3 Existing Model Analysis

6.3.1 Existing Model Analysis

The design storm discussed in **Section 6.2.4.2,** was incorporated into the calibrated model for analysis of the existing system capacity. Two measures of flow conditions in the collection system were used for evaluation of the existing model: flow depth over pipe diameter (d/D) and reserve capacity of the pipe. Based on these measures, there was only one minor instance of pipe surcharging and therefore, the existing collection system appears to have adequate capacity at today's flows.

Figures A4 and **A5** in **Appendix A** show Depth over Diameter (d/D) and Reserve Capacity for the Existing Model, respectively. Depth over diameter can be used to identify the extents of surcharging, and includes backwater effects from downstream pipe segments. Reserve capacity can be used to identify individual pipes that could be the root cause of the surcharging or limited capacity, but does not include the backwater effects from downstream pipe segments. This is evident when looking at both figures side by side. **Figure A5** shows several small instances of pipes being over capacity; however these issue pipes are not seen on **Figure A4** – these overcapacity pipes are pipes that were constructed nearly flat or with a reverse grade.

Table 6-4 contains a list of the areas where issues were identified in the system for the Existing Model. **Appendix E** has additional information and hydraulic grade line plots for each issue. **Appendix F** contains results from the Existing Model Analysis. Note that all existing model results and figures include the design storm event.

Table 6-4 – Existing Model Issues

Location	Issue	Reference	Recommended Action
Country Ridge Collector at Queensgate Drive	Surcharging ~ 0.10'	Appendix E, Section 2.1	Do Nothing / Monitor Pipe Flow Depth and Conditions

Table 6-5 contains a summary of each lift station and its remaining capacity given existing conditions. The lift stations have sufficient existing capacity.

Table 6-5 – Existing Model Lift Station Summary

Lift Station	Design Capacity (GPM)	Existing Peak Flow (GPM)[(a)]	Existing Peak Flow (% of Capacity)	Remaining Capacity (GPM)
Battelle	400	349[(b)]	87%	51
Waterfront	600	237	40%	363
Terminal Drive	150	19	13%	131
Mental Health	260	7	3%	253
Bradley	180	25	14%	155
Columbia Pt	270	58	21%	212
Wellhouse Loop	100	7	7%	93
Duportail	200	78	39%	122
Montana St	970	290	30%	680
Columbia Park Trail	400	41	10%	359
Meadows South	100	18	18%	82
Bellerive	260	82	32%	178
Meadow Ridge	245	7	3%	238
Dallas Rd	260	20	8%	240

(a) *Peak Flow values include a 10% factor of safety to reduce the potential for overloading the station. See Appendix C for further detail.*

(b) *Assumes Areva (SIU CR-IU008) is discharging at its MPL of 0.40 mgd.*

6.4 Committed Model

6.4.1 Introduction

The analysis of the Existing Model shows that the existing collection system is capable of handling existing design flows. The next step is to identify how the system will perform with future flows from areas to which the City has committed to provide service. The Committed Model represents the development within the entire UGA boundary; essentially everything the City has committed to serve and generally includes all parcels, developed or not. **Figure A6** identifies all the areas within the UGA that are currently undeveloped that were added to the Committed Model and developed based on their current land use. Additionally, the occupancy rates for multi-family housing are set equal to 100 percent to maximize sewer flows. The Committed Model is a tool to guide growth and expansion of the collection system and to also identify potential future deficiencies in the current collection system. The Committed Model's primary purposes are, therefore, as follows:

- Show the "uncommitted" capacity remaining in the collection system.
- Provide the size, approximate location, and depth for future sewer pipes 10 inches and greater in size to serve the UGA boundary.
- Identify potential capacity issues that may arise in the existing collection system as the City develops new areas beyond the City limits but within the UGA.
- Develop a base model to use in evaluating future wastewater service scenarios.

6.4.2 Committed Model System Layer

6.4.2.1 Trunk Pipes

The Committed Model system layer was developed to take advantage of existing and future public right-of-way and the low-lying areas along natural drainages. During the development of the system, the following information was taken into consideration:

- Selected master plan improvements identified in the SRSR project were included: the Bellerive Lift Station pump upgrade (600 gpm pump replacement) and East Badger South Lift Station.
- Development of the Badger South planned development was based on specific planning values provided by the developer's consulting engineer.
- The area west of the By-Pass Highway (SR 240) and southwest of the Richland Airport, currently zoned Agriculture, was not developed.
- Stevens Drive divides the upper North Richland area into two separate drainage basins with different land uses: the west area includes the Horn Rapids industrial park while the east area includes facilities related to the Pacific Northwest National Laboratory (PNNL).
- Undeveloped industrial area in and around the Horn Rapids Industrial Park (HRIP) was developed using a value of 1,250 gpad (as selected by the City) – this includes the Bechtel laydown yards generally south of Battelle Boulevard and west of Stevens Drive.
- Previous General Sewer Plan routing.

To reduce capital construction costs and operation and maintenance costs, the depth of future trunk pipes (10 inches and larger) was held to a minimum cover of 10-feet where possible while still providing service and minimizing the number of lift stations. It should be noted that these are planning level depths based upon the City's GIS contour layer data of 2-foot intervals. Detailed topographical survey will be needed on a project by project basis in order to refine pipe depths.

Sizing of future pipes was accomplished using the design parameters listed in **Table 6-6**. A portion of undeveloped land within the UGA has sufficient slope to allow trunk pipes to be constructed at steeper than minimum grade, thereby allowing for a reduction in trunk pipe sizes. In order to ensure that pipe downsizing, due to steeper than minimum slope, does not result in physical obstruction bottlenecks, care must be taken that trunks are designed and installed at the same or steeper slope than those listed in the results of the Committed Model. **Appendix G** lists the proposed sizes, inverts, and slopes of the future trunk pipes, and denotes any trunk pipes that require steeper than minimum slopes.

Table 6-6 – Future Pipe Design Parameters

Pipe Diameter (in)	Maximum Allowed Depth/Diameter	Minimum Slope
8	0.50	0.40%
10	0.55	0.28%
12	0.60	0.22%
15	0.65	0.15%
18	0.75	0.12%
21 to 30	0.75	0.10%
≥36	0.85	0.10%

The following is a list of assumptions used in the Committed Model:

- Each dwelling unit houses an average of 2.42 people based on the 2010 US Census data.
- The average sanitary flow per dwelling unit was 160 gpd, the residential unit flow determined during the calibration of existing model.
- Infiltration and inflow for future trunk pipes will be zero.
- Significant Industrial User's (SIU's), permitted by the City, are discharging at the Maximum Permitted Limit (MPL)
- Undeveloped residential areas were addressed by land use type and the following criteria (as described in **Appendix C**):
    - Low Density Residential – level of development based on parcel size
        - ≤ 1 acre = leave parcel flow as-is / no further development for Committed Model
        - > 1 acre = first reduce (by 23%) parcel size for non-buildable area, then subdivide parcel into 3.5 du/ac and multiply by the Low Density Unit Flow (160 gpdu) to calculate the Committed Model flows
    - Medium Density Residential – evaluate the density of each parcel based on the value of the following ratio: (Water Meter flow) / (number of du) / (Low Density Unit Flow)
        - ≥ 0.75 = leave parcel flow as-is for Committed Model
        - < 0.75 = update the parcel's Committed Model flow by the product (number of du)*(Low Density Unit Flow)
    - High Density Residential – same process as for Medium Density Residential

6.4.3 Committed Model Flow Generation Layer

6.4.3.1 Land Use and Unit Flows

Future flows were developed for the Committed Model which assume 100% development within the UGA boundary. **Figure A6** depicts the locations of all the assumed infill development.

Land use designations for the Committed Model were determined from existing land use and zoning designations as well as discussions with the City. The unit flows identified in **Table 6-3** were used to generate the Committed Model flows.

6.4.3.2 Flow Allocation

Similar to the Existing System Model, each parcel in the Committed Model service area was modeled by injecting flow into the nearest upstream manhole in the system layer. Some large master plan parcels were divided according to the proposed land use configurations and flows then injected into multiple locations based on topography and trunk pipe serviceability.

It is important to note that the service area boundaries for each trunk pipe within the Committed Model are based on aerial mapping and City/County contours, and therefore are approximate. Individual service area boundaries may change slightly as field survey is performed and development occurs. While safety factors built into the model allow for these minor changes, significant proposed changes or the cumulative effect of minor changes should be analyzed to prevent over-allocation of trunk capacity in the future.

6.5 Committed Model Analysis

6.5.1 Committed Model Analysis

The Committed Model analysis provides results assuming development within the entire UGA boundary, without the addition of any relief pipes or the correction of existing system deficiencies. This helps identify the priorities for Capital Improvement Projects. It should be noted that the Committed Model predicts that the total average daily flow at the WWTP will be 12.64 mgd, while in **Table 3-10** the average predicted flow to the WWTP in 20-years will be 8.24 mgd. As noted in the table, the 20-year flow is based on specific growth rate values for residential, commercial and industrial land uses; therefore, the Committed Model represents growth beyond the 20-year planning projections.

Figures A7 and **A8** show Committed Model results for Depth over Diameter and Reserve Capacity, respectively. As previously noted, depth over diameter can be used to identify the extents of surcharging, and includes backwater effects from downstream pipe segments; while reserve capacity can be used to identify individual pipes that could be the root cause of the surcharging or limited capacity, but does not include the backwater effects from downstream pipe segments.

Appendix G contains a tabular layout, by pipe model ID, of the results from the Committed Model analysis. The results list the upstream and downstream manhole information including rim and invert elevation and the data source, the pipe length, diameter, and slope, and Committed Model results including flow, velocity, d/D and reserve capacity. All Committed Model results and figures include the design storm event.

Table 6-7 contains a list of the areas where issues were identified in the system for the Committed Model. Each problem reach is identified by the general location and is discussed in detail in **Appendix E**. The issues are also grouped into the applicable CIP project number used in **Table 7-1** of **Chapter 7**.

Table 6-7 – Committed Model Pipe Capacity Issues

Location	Issue	Identified Under Existing Model Analysis	Reference	Recommended Action
Country Ridge Collector to Yakima River	Overflow		Appendix E, Section 3.1	Replace Pipe with Larger Diameter – Developer Driven Improvement
Leslie Rd Trunk Near Col. Park Trail	Pipe Nearing Capacity d/D = 0.95		Appendix E, Section 3.2	Replace Pipe Section – See *CIP **CP.1** for details*
Keene Rd Collector At Keene/Gage Int.	Surcharge ~ 0.10-ft		Appendix E, Section 3.3	Replace Pipe Section – See *CIP **CP.2** for details*
Upper North Interceptor	Surcharging of Local Collectors and Residential Services		Appendix E, Section 3.4	Replace Pipe Section – See *CIP **CP.3** for details*
Bellerive LS Downstream Piping	Surcharge ~ 3.0-ft		Appendix E, Section 3.5	Replace Pipe Section – See *CIP **CP.4** for details*
Logston Interceptor Logston Blvd	Pipe Nearing Capacity d/D = 0.86		Appendix E, Section 3.6	Do Nothing / Monitor Pipe Flow Depth and Conditions
Airport Collector On Hagen Rd	Pipe Nearing Capacity d/D = 0.82		Appendix E, Section 3.7	Do Nothing / Monitor Pipe Flow Depth and Conditions
Hwy 240 Interceptor Highway Crossing	Flat & Reverse Grade, Pipe Nearing Capacity d/D = 0.80		Appendix E, Section 3.8	Do Nothing / Monitor Pipe Flow Depth and Conditions

The City's lift stations, with the Committed Model scenario peak flow and remaining capacity are listed in **Table 6-8.**

Table 6-8 – Committed Model Lift Station Summary

Lift Station Name	Design Capacity (GPM)	Committed Model Peak Flow (GPM)[a]	Committed Model Peak Flow (% of Capacity)	Remaining Capacity (GPM)
Battelle[b]	400	349[b]	87%	51
Waterfront	600	260	43%	340
Terminal Drive	150	22	15%	128
Mental Health	260	7	3%	253
Bradley	180	84	47%	96
Columbia Pt	270	84	31%	186
Wellhouse Loop	100	10	10%	90
Duportail[c]	200	222[c]	Exceeds Capacity	-22
Montana St	970	298	31%	672
Columbia Park Trail	400	52	13%	348
Meadows South	100	20	20%	80
Bellerive[d]	260	540[d]	Exceeds Capacity	-280[d]
Meadow Ridge	245	11	4%	234
Dallas Rd[e]	260	2,450[e]	Exceeds Capacity	-2190[e]

(a) *Peak Flow values include a 10% factor of safety to reduce the potential for overloading the station. See Appendix C for further detail.*
(b) *See Section 6.6.2.1 for a discussion of build-out of the Battelle Lift Station drainage basin and peak influent flow conditions.*
(c) *See Section 6.6.2.2 for a discussion of build-out of Duportail Lift Station drainage basin and peak influent conditions.*
(d) *See Section 6.2.2.3 for a discussion of build-out of the Bellerive LS drainage basin and peak influent flow conditions.*
(e) *See Section 6.2.2.4 for a discussion of build-out of the Badger Mountain South development, the Dallas Road LS drainage basin and peak influent flow conditions.*

6.5.2 Committed Model Lift Station Analysis

Table 6-8 provides the results of the Committed Model scenario peak flows into the City's 14 existing lift stations. The difference between the existing lift station capacity and the peak flows is listed. Note that the peak flow values as predicted by the model include a 10% factor of safety (See **Appendix C** for further detail). As noted in the table, four lift stations have negative values which indicate additional capacity will be required to meet the Committed Model scenario peak flows. Each of these four lift stations are discussed in further detail in the following sections.

6.5.2.1 Battelle Lift Station

During the Committed Model scenario, it was assumed that Areva (SIU CR-IU008) would be discharging at its maximum permitted limit (MPL) of 0.40 mgd and a new gravity collection pipe would be constructed along Battelle Blvd, east of the lift station, to serve current and undeveloped areas. It was assumed the undeveloped areas along Battelle Boulevard would be developed at the industrial unit flow of 1,250 gpad. This new gravity pipe would provide a bypass around the existing lift station and reduce influent flows to only those from Areva. The model identified that peak flows into the Battelle Lift Station would be roughly the same as during the Existing Model scenario, at 350 gpm

(0.50 mgd), since other flows are relatively minor. Note that in order to connect to existing sewer services and maintain a minimum of 8-ft of cover, the pipe should be 12-in in diameter and constructed at a slope of 0.10%.

In January 2016, Areva has agreed to a modified permit limit of 0.258 mgd; lower than the current MPL.

6.5.2.2 Duportail Lift Station

During the Committed Model scenario, it was assumed that the apartments within the drainage basin are all at full (100%) occupancy and that a currently undeveloped area (44 acres) north of the lift station would be developed at a density of 5 units/acre using a unit flow of 160 gpdu (Note that this also excludes 23% of the area for roads and landscaping). Based on these assumptions, the model identified that peak flows into the Duportail Lift Station would be roughly 200 gpm (0.29 mgd), which matches the capacity of one of the existing pumps. It should be noted that the existing lift station is within the proposed alignment of the Duportail Bridge and Roadway extension project and is planned for relocation. (See **Figure 6-3** for additional detail) At this time it is not known when this improvement will occur, although it is predicted to take place within the next 5 to 10 years. At that time, larger pumps with additional capacity should be selected for the new lift station.

As noted in **Table 6-8**, the factored peak flows into the lift station are approximately 222 gpm and the flows are then pumped through an existing 6-inch force main. At this peak flow the flow velocity through the force main will be approximately 2.5 fps, therefore greater than the minimum velocity required for self-cleaning (2 fps) but less than the maximum recommended velocity (8 fps) as listed in Ecology's Orange Book reference. Downstream of the forcemain discharge manhole is the 12-inch Bypass Highway Interceptor and the Committed Model scenario identifies this pipe has 0.60 mgd of reserve capacity for this additional flow.

6.5.2.3 Bellerive Lift Station

During the Committed Model scenario, all undeveloped area within a lift station drainage basin was developed at its current zoning and routed to the lift station to account for future development – this also includes converting homes currently on septic systems. Within the Bellerive Lift Station drainage basin is the existing Rancho Reata neighborhood; therefore to identify ultimate build-out conditions for the lift station two scenarios were considered: Rancho Reata *not-included*, and Rancho Reata *included*. Given these two scenarios, the model identified that peak flows into the lift station were roughly 425 gpm (0.61 mgd) and 490 gpm (0.71 mgd), respectively. Each of the existing lift station pumps only has a capacity of 260 gpm (0.37 mgd) and therefore additional capacity will be required with build-out of this drainage basin area. (See **Figure 6-4** for additional detail)

As noted in **Table 6-8**, the factored peak flows into the lift station are approximately 550 gpm (assuming Rancho Reata *included*) and the flows are then pumped through an existing 6-inch force main. At this peak flow the flow velocity through the force main will be approximately 5.6 fps, therefore greater than the minimum velocity required for self-cleaning (2 fps) but less than the maximum recommended velocity (8 fps) as listed in Ecology's Orange Book reference. The forcemain currently discharges into 8-inch gravity sewer collection piping along Bellerive Drive, north of Gage Boulevard. At this location the gravity sewer pipe has a pipe slope of approximately 1.5% with a reserve capacity of approximately 0.40 mgd, however further downstream, the pipe slope flattens to approximately 0.50% and the reserve capacity decreases to 0.30 mgd. Surcharging occurs here and continues upstream for roughly 3,500-feet. Replacing the 8-inch piping with 12-inch piping (and matching the existing slope) will resolve the surcharging of the downstream collection system.

6.5.2.4 Dallas Road Lift Station

During the Committed Model scenario, all of the Badger South development was considered developed per its master plan. To handle the master plan flows, the developers planned to route sewer flows, generated on the east half of the development, to an east Badger South lift station that would be positioned near the northeast corner of the Badger South development. The East Badger South Lift Station is shown as a master planned lift station on **Figure 6-5**. The wastewater would then be pumped to the west and to the West Badger South Lift Station, a master plan lift station, adjacent to the existing Dallas Road lift station. The developer intends for the West Badger station to be a central lift station to service the entire Badger South development. Wastewater flows would then be pumped along the same alignment the Dallas Road forcemain follows – eventually discharging into the gravity collection system in Country Ridge. (See **Figure 6-5** for additional detail)

As noted in **Table 6-8**, the factored peak flows for build-out of the development at the west, central lift station are approximately 2,450 gpm (3.5 mgd). Influent flows at the lift station are from the West Badger South area and from the East Badger South lift station. Peak flows for West Badger South area are approximately 1,000 gpm (1.44 mgd) and the capacity of the East Badger South lift station pumps are approximately 1,300 gpm (1.87 mgd). At the West Badger South lift station flows are pumped through an existing 12-inch force main. At the factored peak flow the flow velocity through the force main will be approximately 7.0 fps, therefore greater than the minimum velocity required for self-cleaning (2 fps) but less than the maximum recommended velocity (8 fps) as listed in Ecology's Orange Book reference. As previously noted, the forcemain discharges into 8-inch gravity sewer collection piping through Country Ridge. At this location the gravity sewer pipe is roughly constructed at minimum slope and has less than 0.75 mgd of reserve capacity. Routing the Badger South flows through this existing collection system causes surcharging which then leads to localized flooding at several manhole locations. The surcharging continues as the collector crosses Keene Road and Queensgate Drive for a total of roughly 12,000-lineal feet of 8-inch piping that will need replacement.

Figure 6-1 – Intentionally Left Blank

City of Richland
2015 General Sewer Plan

Figure 6-2

Battelle Lift Station

Legend

- City Limits
- Interstate/Highway
- Major Streets
- City Lift Stations
- Manhole

Pipe Size (in)

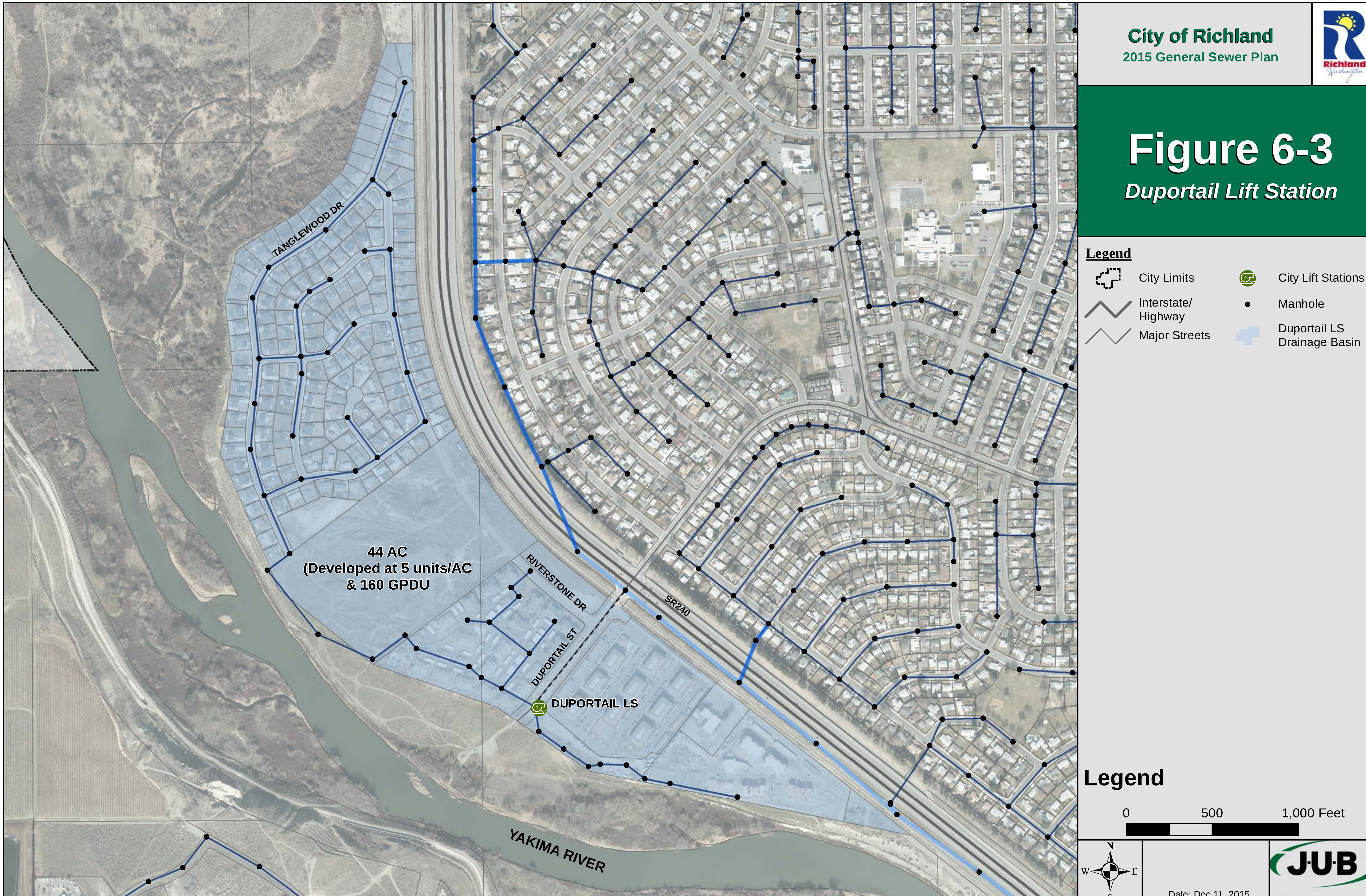
- Collector
- 10
- 12
- 15
- 18
- 21
- 24
- 27
- 30
- 36
- 42
- 54
- Forcemain

0 500 1,000 Feet

N W E S

Date: Dec 11, 2015

J-U-B ENGINEERS, INC.


City of Richland
2015 General Sewer Plan
Richland Washington
Figure 6-3
Duportail Lift Station
Legend
City Limits
Interstate/ Highway
Major Streets
City Lift Stations
Manhole
Duportail LS Drainage Basin
TANGLEWOOD DR
44 AC
(Developed at 5 units/AC
& 160 GPDU
RIVERSTONE DR
SR240
DUPORTAIL ST
DUPORTAIL LS
YAKIMA RIVER
Legend
0
500
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Date: Dec 11, 2015
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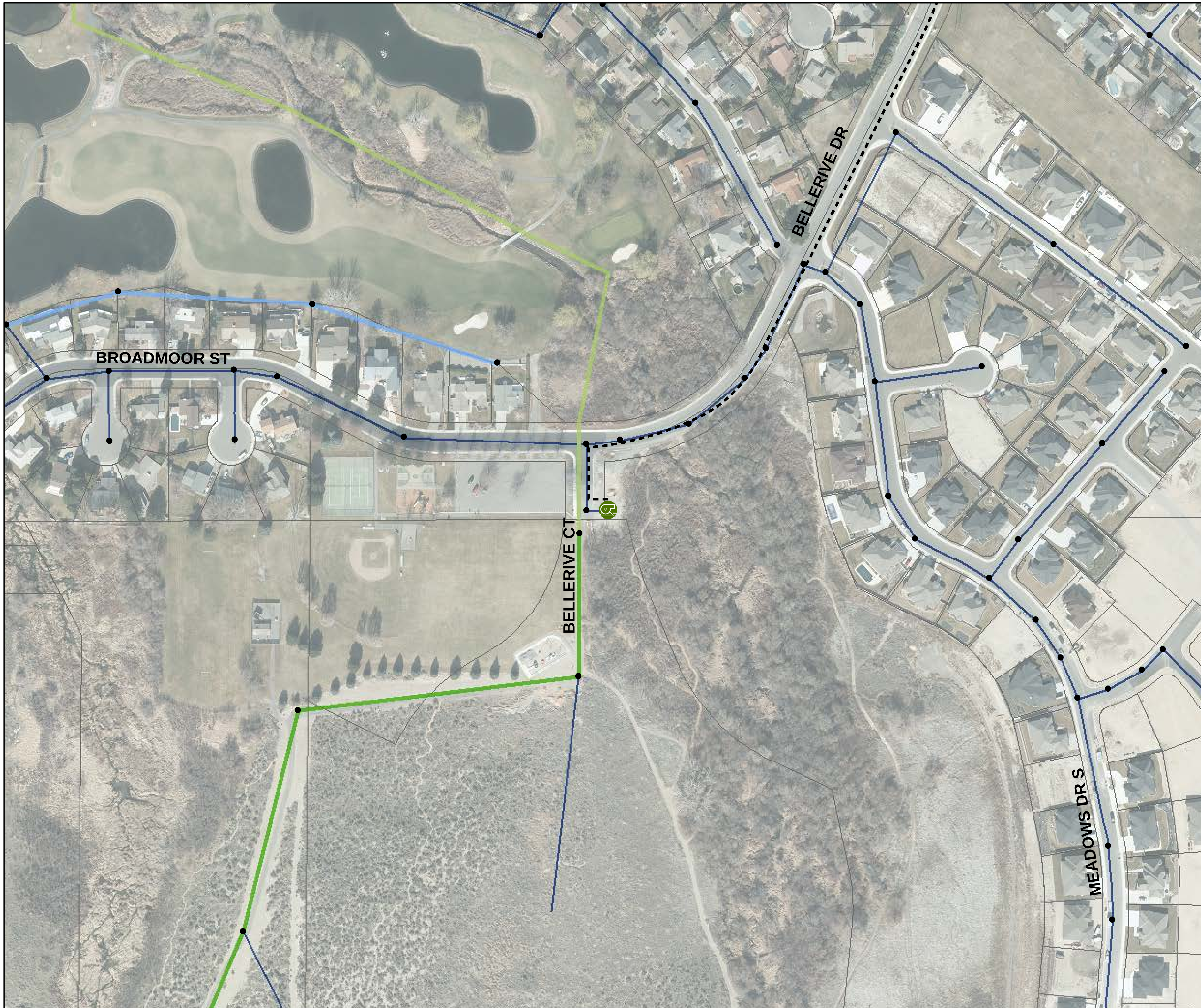
J·U·B ENGINEERS, INC.



City of Richland
2015 General Sewer Plan

Figure 6-4
Bellerive Lift Station

Legend

- City Limits
- Interstate/Highway
- Major Streets
- City Lift Stations
- Manhole

Pipe Size (in)

- Collector
- 10
- 12
- 15
- 18
- 21
- 24
- 27
- 30
- 36
- 42
- 54
- Forcemain

0 200 400 Feet

N W E S

Date: Dec 2, 2015

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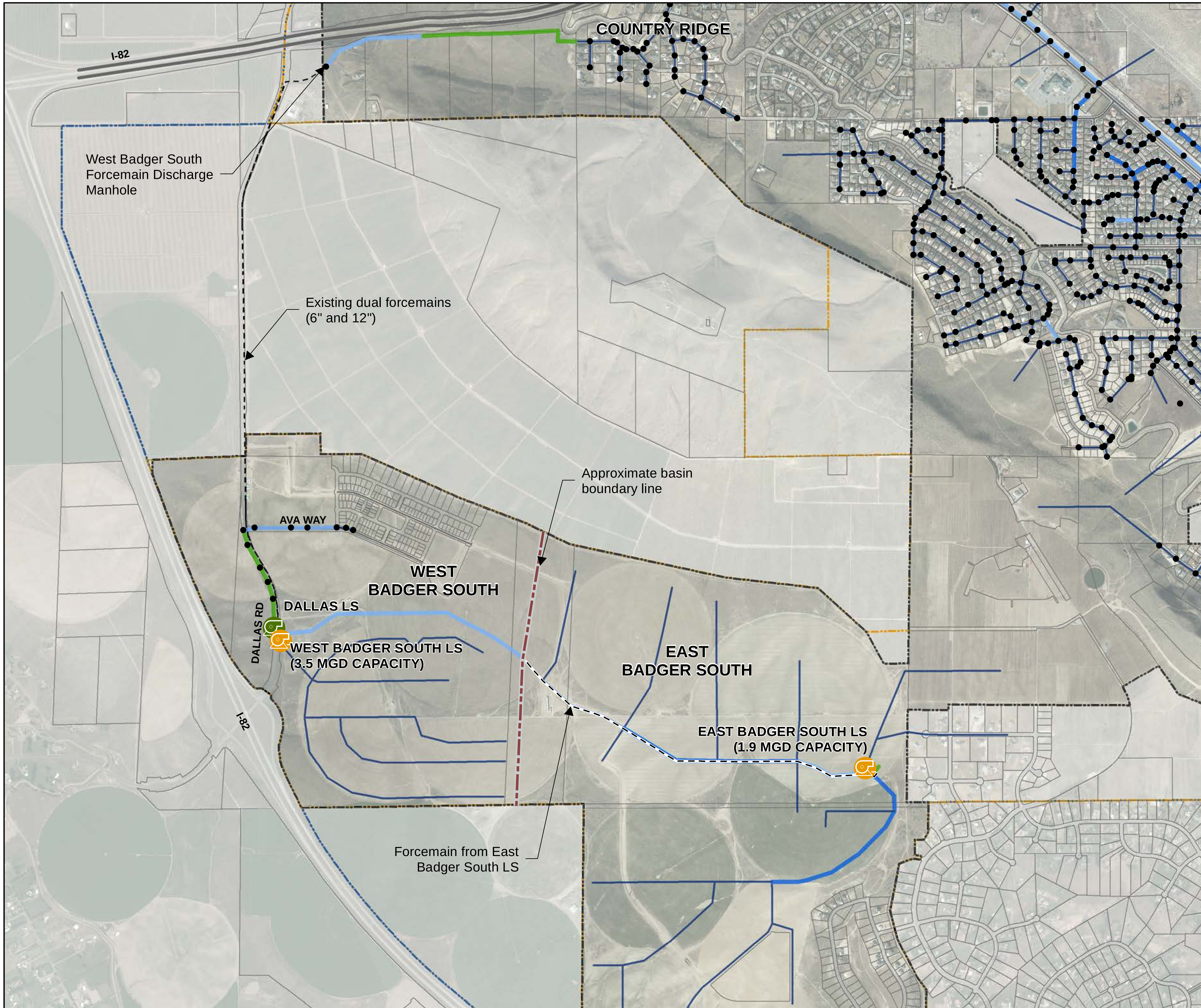
City of Richland
2015 General Sewer Plan

Figure 6-5
Dallas Lift Station



Legend

- City Limits
- UGA
- 50-yr Planning
- Interstate/Hwy
- Major Streets
- City Lift Station
- MP Lift Station
- Manhole

Pipe Size (in)

- Collector
- 10
- 12
- 15
- 18
- 21
- 24
- 27
- 30
- 36
- 42
- 54
- Forcemain
- MP Forcemain

0 1,500 3,000 Feet

N E S W

Date: Dec 11, 2015

J·U·B ENGINEERS, INC.

6.6 Master Plan Model

6.6.1 Master Plan Model Analysis

With modern materials and construction methods, it is expected that sewer pipes will exceed fifty years of service before rehabilitation or replacement is necessary. Therefore, a Master Plan Analysis is prepared as a model scenario by planning to a 50-year boundary. This ensures that all projects identified for the CIP (at the Committed Model stage) are further upsized to handle planned build-out flows – thus ensuring the pipes will provide reserve capacity for their design life. The 50-year boundary was identified by the City as the UGA boundary along the south part of Richland and the UGA boundary along the north part of Richland including a 1,350 acre portion of a land transfer from the Department of Energy's 300 Area at the request of TRIDEC.

Table 6-9 contains a list of the issues identified in the Master Plan Model. Similar to the existing and committed model results, each issue is identified by the interceptor name or general location and is discussed in detail in **Section 6.7.2** and included in **Appendix E**. The issues also reference the applicable CIP project number used in **Table 7-1** of **Chapter 7**. **Figure A10** identifies the Master Plan Model pipe sizes.

Between the initial conceptual layout and the final model results, several alignment changes were made to provide service to the study area extents, minimize the sewer depths, and eliminate the need for lift stations. The Master Plan scenario was developed so that the majority of the future trunk pipes are at planned depths of less than 20 feet below the existing ground surface, as shown in **Figure A11**, with the exception of portions of the collection system extension along Horn Rapids Road and adjacent to SR 240. In these basins, the ground topography varies widely with existing drainage depressions; however, as planned development extends to these areas, the pipe depths are expected to decrease.

Figures A12 and **A13** show the Depth over Diameter and the Reserve Capacity, respectively, for the Master Plan Model based on the existing pipe sizes. As previously noted, depth over diameter can be used to identify the extents of surcharging, and includes backwater effects from downstream pipe segments; while reserve capacity can be used to identify individual pipes that could be the root cause of the surcharging or limited capacity, but does not include the backwater effects from downstream pipe segments.

Appendix H contains results from the Master Plan Model Analysis. All Master Plan Model results and figures include the design storm event.

Table 6-9 – Master Plan Model Issues

Location	Issue	Identified Under Committed Model Analysis	Reference	Recommended Action
Country Ridge Collector to Yakima River	Overflow	X	Appendix E, Section 3.1	Replace Pipe with Larger Diameter – Developer Driven Improvement
Leslie Rd Trunk Near Col. Park Trail	Surcharge ~ 2.0-ft	X	Appendix E, Section 3.2	Replace Pipe Section – See *CIP* ***CP.1*** *for details*
Keene Rd Collector At Keene/Gage Int.	Surcharge ~ 0.10-ft	X	Appendix E, Section 3.3	Replace Pipe Section – See *CIP* ***CP.2*** *for details*
Upper North Interceptor	Surcharging of Local Collectors and Residential Services	X	Appendix E, Section 3.4	Reconfigure Interceptor and New Lift Station – See *CIP* ***CP.3*** *for details*
Bellerive LS Downstream Piping	Surcharge ~ 3.0-ft	X	Appendix E, Section 3.5	Replace Pipe Section – See *CIP* ***CP.4*** *for details*
Logston Interceptor Logston Blvd	Several Segments at Full Flow d/D ~ 1.00	X	Appendix E, Section 4.1	Do Nothing / Monitor Pipe Flow Depth and Conditions
Hwy 240 Interceptor Highway Crossing	Flat & Reverse Grade, Pipe Nearing Capacity d/D = 0.85	X	Appendix E, Section 3.7	Do Nothing / Monitor Pipe Flow Depth and Conditions
Airport Collector On Hagen Rd	Pipe Nearing Capacity d/D = 0.82	X	Appendix E, Section 3.8	Do Nothing / Monitor Pipe Flow Depth and Conditions

6.6.2 Assessment of Master Plan Model Results

The following sections discuss the issues identified in the Master Plan Model and provide further detail. Several of the issues are grouped together and discussed as a whole based on their location in the collection system. It should be noted that each of these system issues were discussed in the "Committed Model" discussion and are recommended to be further upsized in order to provide capacity for the Master Plan flows.

6.6.2.1 Country Ridge Collector and Badger South Development

As development continues within the Badger South area, more flow will be routed through the Country Ridge Collector. The existing collector pipe is 8-inch diameter both through the Country Ridge development and downstream to the intersection of Queensgate Dr and Jericho Rd (approximately 7,600 LF). It then increases to a mix of 12-inch and 15-inch diameter pipe and connects to the Yakima River inverted siphon crossing (approximately

4,500 LF). The bottleneck in this collection system is through the 8-inch piping, which was mainly constructed at minimum slope (0.40%) and only has a reserve capacity of 0.40 mgd at existing flows. Note that the capacity of the Dallas Road lift station is 260 gpm (0.37 mgd); therefore improvements to the Country Ridge Collector should precede a pump upgrade to the lift station.

The Master Plan Model identifies at build-out that the peak flow from the Badger South development will be approximately 2,300 gpm (3.3 mgd). This flow is out of the large planned lift station to be constructed adjacent to the existing Dallas Road lift station. Downstream pipe improvements through Country Ridge and to the Yakima River (approximately 12,000 LF) should be sized to accommodate this future peak flow. Using the Master Plan Model, it was determined that 18-inch gravity collection pipe (matching the existing pipe grades) would convey the future lift station discharge.

6.6.2.2 Leslie Road Trunk

In South Richland, drainage basins M, N, O, and the majority of P (see **Figure 1-1** for basin reference) drain half the City through a 21-inch trunk sewer pipe on Leslie Road – making this a critical part of the collection system. The trunk pipe follows Leslie Road northward and downhill toward Columbia Park Trail at a 6% grade before flattening to a 0.50% slope at the bottom of the hill. At this grade change the trunk pipe also decreases in diameter from 21-inch to 18-inch for a length of 120-feet before increasing in size to 30-inch. Both the Committed and Master Plan Model identify that surcharging will occur during these scenarios. Peak flows through the trunk pipe are approximately 4.7 mgd and surcharging of up to 2-feet occurs in the 18-inch section of piping.

6.6.2.3 Keene Road Collector

The Keene Road Collector is a 12-inch pipe that drains residential developments, on either side of Keene Road, and routes flows to the east and eventually to the Leslie Road trunk. From record drawings and previous sewer model data it was determined that the collector decreases in diameter to 10-inch pipe for a short 900-LF section before increasing to 12-inch. This bottleneck is located on Keene Road, just north of the intersection of Keene Road and Gage Boulevard. Both the Committed and Master Plan Model identify that the amount of surcharging that occurs here is minor (0.10-ft); however a decrease in diameter can prevent larger objects in the collection system from passing through.

6.6.2.4 Upper North Interceptor and Service Backups

The Upper North Interceptor (UNI) is located in a residential area of North Richland, generally north of McMurray Street and east of George Washington Way (G-Way). It is approximately 15,000 of 18-inch and 24-inch concrete interceptor pipe that drains from the diversion structure near the intersection of G-Way and University Drive down to a connection with the Lower North Interceptor (54-inch pipe) at the intersection of McMurray Street and G-Way. (See **Figure A1** for reference) The lower half (18-inch and 24-inch) of the UNI was constructed in the 1970's while the upper half (24-inch) was constructed in 1997. City sewer crews' note that there are several dropped pipe joints along the UNI that have been observed during routine CCTV inspection. Crews also note that homeowners, in specific areas along the UNI, have complained about backups or overflows into their basements when the UNI flows at or greater than half full. For that reason the diversion structure has been adjusted to keep flows below half pipe flow.

As part of this General Sewer Plan two alternatives were considered to resolve the service backups caused by the UNI. However, the Master Plan Model was first used to confirm that all master plan flows can be diverted away from

the UNI through control of the diversion structure. This was confirmed using the Master Plan Model and no surcharging was observed.
Working with City staff, the affected neighborhoods within this area of North Richland were identified and two improvement alternatives were considered:

A. Construct New Connection Piping on Davison Avenue for UNI & Construct New Lift Station at the Water Treatment Plant (WTP) (See **Figure 6-5**)

   With this alternative approximately 2,400-LF of new 18-inch piping would be constructed along Davison and connect two existing UNI manholes. The existing UNI piping at each manhole would be plugged to route flows down the new piping – bypassing a neighborhood area that has had service backups. This "disconnected" area would instead drain to the new lift station located in an open space adjacent to the WTP. The new lift station would serve several small cul-de-sac's near the intersection of Saint St and Davison Avenue.

B. Construct a Compact Lift Station near McArthur & Alexander & Construct New Lift Station at the Water Treatment Plant (WTP) (See **Figure 6-5**)

   With this alternative two lift stations would be constructed to alleviate service backups. A compact lift station with a small footprint would be located along Alexander Avenue, between MacArthur and Spengler Streets. This lift station would serve approximately 15 homes that are at a greater likelihood of a service backup. It would discharge back into the UNI. The new lift station at the WTP is also a part of this alternative, however fewer homes would be connected and therefore a smaller pump size may be considered.

The City did not choose an alternative at this time, but will revisit both alternatives and better identify the affected homes at the time this project begins design.

6.6.2.5 Bellerive Lift Station Drainage Basin

As previously noted in **Section 6.6.2.3,** within the Bellerive Lift Station drainage basin is the existing Rancho Reata neighborhood which is not currently in the City limits but is within City's UGA and the overall area of where this lift station can serve; therefore to identify ultimate build-out conditions for the lift station two scenarios were considered: Rancho Reata *not-included*, and Rancho Reata *included*. Given these two scenarios, the model identified that peak flows into the lift station were roughly 425 gpm (0.61 mgd) and 490 gpm (0.71 mgd), respectively. Each of the existing lift station pumps only has a capacity of 260 gpm (0.37 mgd) and therefore additional capacity will be required with build-out of this drainage basin area. The forcemain currently discharges into 8-inch gravity sewer collection piping along Bellerive Drive, north of Gage Boulevard. At this location the gravity sewer pipe has a pipe slope of approximately 1.5% with a reserve capacity of approximately 0.40 mgd, however further downstream, the pipe slope flattens to approximately 0.50% and the reserve capacity decreases to 0.30 mgd. Surcharging occurs here and continues upstream for roughly 3,500-feet. Replacing the 8-inch piping with 12-inch piping (and matching the existing slope) will resolve the surcharging of the downstream collection system.

It is important to note that previous planning for the Badger South development identified approximately half the developed flow to be routed east and to the Bellerive Lift Station. This planning discussion is included in the SRSR memo included in **Appendix B**. The concept included the design of the East Badger South Lift Station with a discharge forcemain routed along I-82 and down to the current endpoint of the Leslie Interceptor, near the intersection of Leslie Road and E Reata Road. This increase of sewer flows to the existing lift station would surpass a minor pump upgrade and instead require the construction of a larger lift station facility with a trench style design. An alternative to a new lift station was considered mainly in an effort to minimize long term O&M. The alternative project was the Meadow Springs Interceptor, which was an extension of the Leslie Interceptor. The alignment generally followed northward and across the Meadow Springs golf course, under Gage Boulevard and through the Canyon Terrace neighborhood to connect to existing 21-inch sewer north of the intersection of Leslie Road and Canyon Street. The Meadow Springs Interceptor was planned as 21-inch pipe through the golf course, then increasing in pipe size, north of Gage Boulevard, to 24-inch.

6.6.2.6 Logston Interceptor

The Logston Interceptor currently extends from Logston Boulevard north to Battelle Boulevard and serves the Horn Rapids Industrial Park area. The 9,000-LF interceptor pipe is 24-inch diameter and was constructed at 0.08% slope with several sections constructed at slopes (0.06%) flatter than the design. The capacity of a 24-inch pipe flowing full and at 0.08% slope is approximately 4.48 mgd. The Committed Model scenario identified that with the build-out of the Industrial Park and areas within the City's UGA (at the unit flow of 1,250 gpad), the interceptor would have an average depth over diameter (d/D) value of 0.72 and a d/D value of 0.86 at any bottlenecks (where 0.06% slope). The average reserve capacity during this scenario is 1.05 mgd with a value of 0.15 mgd at its bottleneck.

Under the Master Plan scenario, an additional 1,350 acres of area north of Horn Rapids Road and the UGA is expected to be added as part of a TRIDEC request area from the Department of Energy. The City plans for this area to be zoned industrial with the same unit flow (1,250 gpad). To best serve this additional area, the Logston Interceptor was extended at minimum slope (0.10%) northward over existing ground. The proposed area is generally flat with the existing ground sloping approximately 0.50% to the south. The proposed alignment for the Logston extension generally follows the existing ground low points and would first constructed to the west along Battelle Boulevard, approximately 1,000-feet, to the western properly line of the Areva facility. Construction would then turn northward, crossing Horn Rapids Road, and following the existing low areas of ground. The Master Plan Model scenario identified that with the additional TRIDEC area the Logston Interceptor would be at full flow capacity where constructed at 0.08% with areas of minor surcharging. It is important to note that the trunk pipe in this area has an average bury depth of 8-feet and there are no sewer services directly connected which would allow for a minimum amount of surcharging during only peak flow conditions.

6.6.2.7 SR 240 Crossing

Both the Committed and Master Plan Model identified the existing 18-inch pipe crossing the SR 240 highway was constructed at nearly a flat grade and that a connecting pipe was constructed with a reverse grade; causing wastewater to collect and puddle at low points along the piping. The affected piping includes a 200-LF bore crossing under the highway and a 220-LF section of reverse grade piping. It appears that additional 18-inch piping may be required to "catch" the existing piping invert elevation using the minimum pipe slope for 18-inch (0.12%).

6.6.2.8 Airport Collector

Both the Committed and Master Plan Model identified a 400-LF section of existing 12-inch pipe, just north of the Richland Airport, was nearing capacity (d/D = 0.82). In this area the pipe slope transitions from steeper than minimum slope to minimum slope. A hydraulic jump occurs at the manhole with the transition of pipe slope; however given the location of where this issue occurs (industrial area and near an airport) the City may choose to simply monitor future conditions for any issues.

6.7 Renewals and Replacements

As documented in **Section 6.3**, the hydraulic analysis of the sewer collection system trunks (10-inch and larger) indicated very few hydraulic bottlenecks in the existing collection system. However, the City has an aged collection system with known condition issues. Therefore, the City's Capital Improvement Plan (CIP) for the next ten years will included a renewed focus on a repair/replacement program for older portions of the collection system based upon condition and risk of failure factors. The replacement program prioritizes sewer pipes with the greatest need for replacement each budget year. The City's goals for the replacement plan included the following:

- Utilize and leverage existing GIS data, existing pipe condition assessment data and hydraulic model results.
- Develop a prioritizing ranking of all pipes in the collection system.
- Develop comprehensive prioritization tool to help City staff make informed engineering decisions about which sewer pipes to target with roadway and water system repair/replacement planning efforts.
- Provide ability for City staff to update the prioritization 'in-house'.
- Summarize priority pipes into projects that include budget level costs and preliminary construction method recommendations.

Through discussions and meetings with the City a prioritization method combining pipe condition, risk and hydraulic capacity was developed. This method is included in **Appendix Q**. Manhole condition was not included in the prioritization at this time, but could be added in the future as standardized manhole condition rating methods are adopted and applied (e.g. NASSCo-MACP).

6.7.1 Prioritization Criteria and Weighting

The prioritization method is composed of three main categories including likelihood of failure (pipe condition), consequence of failure (risk), and hydraulic capacity. The majority of the criteria in each category involved processes and/or data that are already collected by the City. Through workshops with City staff, each category and criteria was assigned a certain weighting value to reflect relative importance. The scoring results using this weighting helped to provide the City with an initial evaluation of their collection system and because these weights can be easily modified, they will likely be adjusted and fine-tuned over time as the City implements the replacement and rehabilitation program.

The Likelihood of Failure (LoF) category generally represents the condition of the pipe. It includes the following criteria:

- PACP Rating – standardized condition rating score from City's Granite XP scoring software.
- Pipe Material – clay, concrete, and steel pipes are given a higher weight, while PVC is given a lower weight.
- Pipe Age – pipe age is based on GIS data or estimated, with older pipes given a higher weight
- Time Since Last CCTV Inspection – pipes with a greater number of years since the last CCTV inspection are given a higher score.

- Cleaning Frequency – based on the City's Problem & Maintenance (PM) list where pipes with a higher frequency of cleaning are given a higher score.

The Consequence of Failure (CoF) category represents the risk and impact of a pipe failure and includes the following criteria:

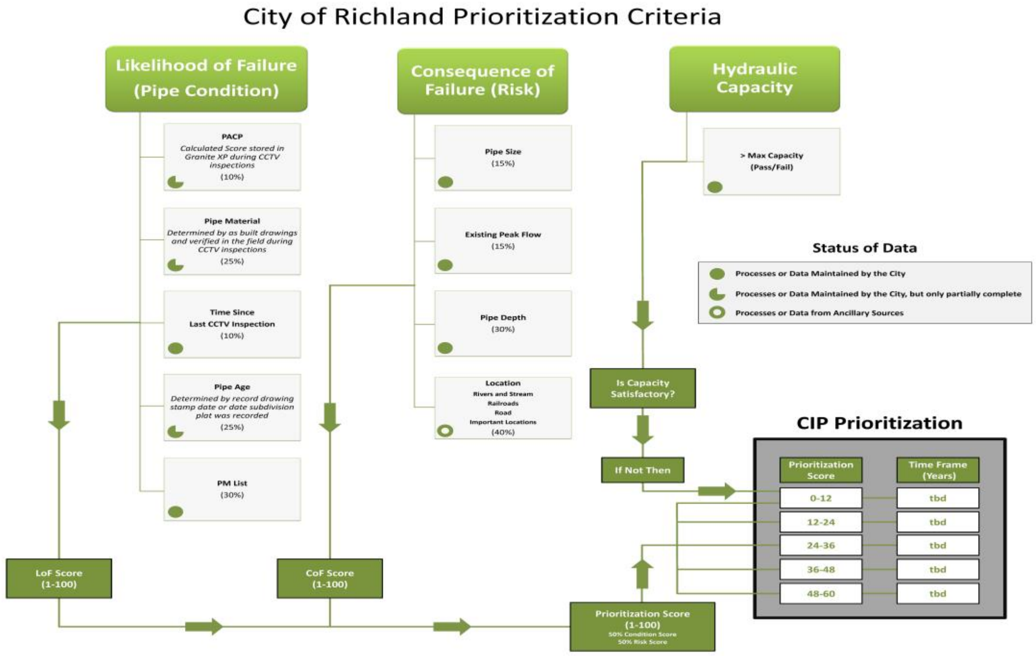
- Pipe Size – larger pipes generally serve a greater number of customers and have greater impact on disruption of service.
- Peak Flow – higher flows are given a higher weight because of the potential for overflows and backups to larger portions of town.
- Location – pipes that cross or run nearby existing water sources (well fields) and waterways are given a higher weight due to the potential environmental risks. Pipes that cross or are located in main arterial roads or cross railroads are given a higher weight due to the increased nuisance to citizens and disruption of service. Pipes that provide service to important locations such as hospitals, schools and major business centers that are sensitive to disruptions of service are given higher weights as well.
- Pipe Depth – deeper sewers are given higher weights due to the additional time required for repairs and increased disruption of service.

The Hydraulic Capacity category utilizes the current sewer model results to assign the highest priority to sewer lines that are over capacity – as these are automatically targeted for replacement independent of condition rating. As noted in **Section 6.3**, the City does not have any existing pipes that are over capacity and therefore this category was not used.

A score (0-100) was then assigned to each pipe for each scoring criteria. **Appendix Q** includes a detailed explanation of each criterion and how scores are calculated. Data field calculations are performed using functionality built into ESRI ArcMap. Each criterion score is then multiplied by the weighting factor and the two categories are summed individually. This results in a CoF score (0-100) and a LoF score (0-100). These two categories are then combined using the category weighting factors to establish a priority score for each pipe (0-100). An explanation of how the weighting factors are determined for both the overall categories (CoF and LoF) and each category criterion is included in the following paragraph. The higher the overall priority score the greater priority should be given for pipe rehabilitation or replacement. Several iterations were conducted by varying the weighting for various criteria. **Figure 6-6** outlines the final weighting criteria and prioritization process that was used.

In regards to the weighting factors used for both the LoF and CoF categories and the criterion within each category, generally little weight was assigned to pipe condition data criterion of the LoF category because the City does not have a significant amount of reliable pipe condition data. As noted in **Section 6.8**, developing an inventory of existing pipe condition is a goal for the City's replacement program. A significant amount of weight was associated to the pipe materials and age criterion of the LoF category because there is a reliable source of this information in the City's GIS system. A significant amount of weight was also associated to areas where the operations staff visits regularly for routine maintenance (i.e. Cleaning Frequency in the LoF category). In the CoF category, more weight was given to the location/proximity and pipe depth criterion and less to the flow and size.

Figure 6-6 – Prioritization Criteria



Figures summarizing the prioritization scoring are included in **Appendix Q**. The City can easily modify the individual criteria weighting or LoF/CoF weighting to further refine and customize the prioritization over time. This can be done using simple ArcMap field calculations without expending significant time.

6.7.2 Developing Pipe Condition Ratings

The City has not been actively performing CCTV assessment of existing pipes and does not have reliable information in regards to current condition. This results in a significant gap in data as the condition rating should be the backbone of a renewal/replacement program. Therefore, the City is budgeting for a large-scale effort to acquire this vital condition information over a three year period. The current pipe scoring and prioritization list that resulted from the effort in this plan provides a general guideline and target to identify those portions of the system that the City should immediately focus on gathering condition data for. Results are shown on **Figure Q2.**

Because most PVC pipe has been recently constructed, the City should focus on the assessment of all non-PVC pipe in the collection system. The City GIS data indicates that there is approximately 725,000 LF of non-PVC pipe in the system. A planning level cost for CCTV and pipe condition rating is approximately $2/LF. Therefore, this effort is expected to cost approximately $1.5 million. A typical rate of assessment is 2,000 LF per day. The City is planning on implementing this project over three years; therefore, approximately $500,000 of the renewals/replacement budget will be dedicated to CCTV and pipe condition rating. The existing prioritization scoring should be utilized to identify which areas of the City should immediately be targeted when developing the contracts for the CCTV and pipe condition rating effort.

Another benefit of the pipe condition scoring effort will be a database listing the types of defects that are found in the various pipes of the collection system. This data will then be valuable in determining the likely construction methods of rehabilitation. A preliminary construction method can be determined by evaluating the type of defects associated with each pipe segment. The City utilizes CUES Granite XP protocols during the CCTV process to code pipe defects. By joining the City's current CUES data table (CCTV defects table) and the City's current sewer pipe dataset in GIS and using tools within ArcMap, a preliminary construction method can be determined for each pipe segment. The City should ensure that this pipe defect data is included in the deliverable for the CCTV and pipe condition rating effort.

6.7.3 Updating the Prioritization

Once the pipe condition rating data has been gathered by the City, it can easily be incorporate into the overall pipe scoring system. Once completed, the backbone of the sewer rehabilitation and replacement program will be the City's existing sewer GIS data and CCTV results (CUES condition rating). This prioritization tool utilizes existing City datasets and the power of GIS to visualize LoF (condition rating) and CoF (risk rating) over the entire City. This tool will allow the City to make informed engineering decisions about each replacement or rehabilitation project.

Each pipe segment can be given an updated LoF score (condition) by joining the City's current CUES data table (CCTV results) to the City's current sewer pipe dataset in GIS and running a simple field calculation. This can be done as often as the City desires, whether it is yearly, quarterly, or more frequently. Additional LoF (condition) criterion that the City may wish to update regularly could include Time Since Last CCTV Inspection and O & M Frequency.

The CoF scores (risk) requires less frequent updates, because the criteria used in this category are less dynamic than the LoF (condition) criteria. Most of the data is derived from the existing sewer model, which is only updated and recalibrated approximately every five years. The 'Location' criteria are also nearly static with updates every five years being adequate.

Appendix Q details procedures to update all of the LoF and CoF criteria. We recommend that the LoF scores (condition) and overall prioritization scores be updated on a yearly basis, prior to establishing the CIP for the coming year. We recommend the CoF (risk) be updated when the existing sewer model is recalibrated, approximately every five years. As the sewer rehabilitation and replacement program is refined in coming years, the City can easily update the LoF, CoF, and overall prioritization scores more or less frequently as required.

6.7.4 Collection System Replacement Analysis

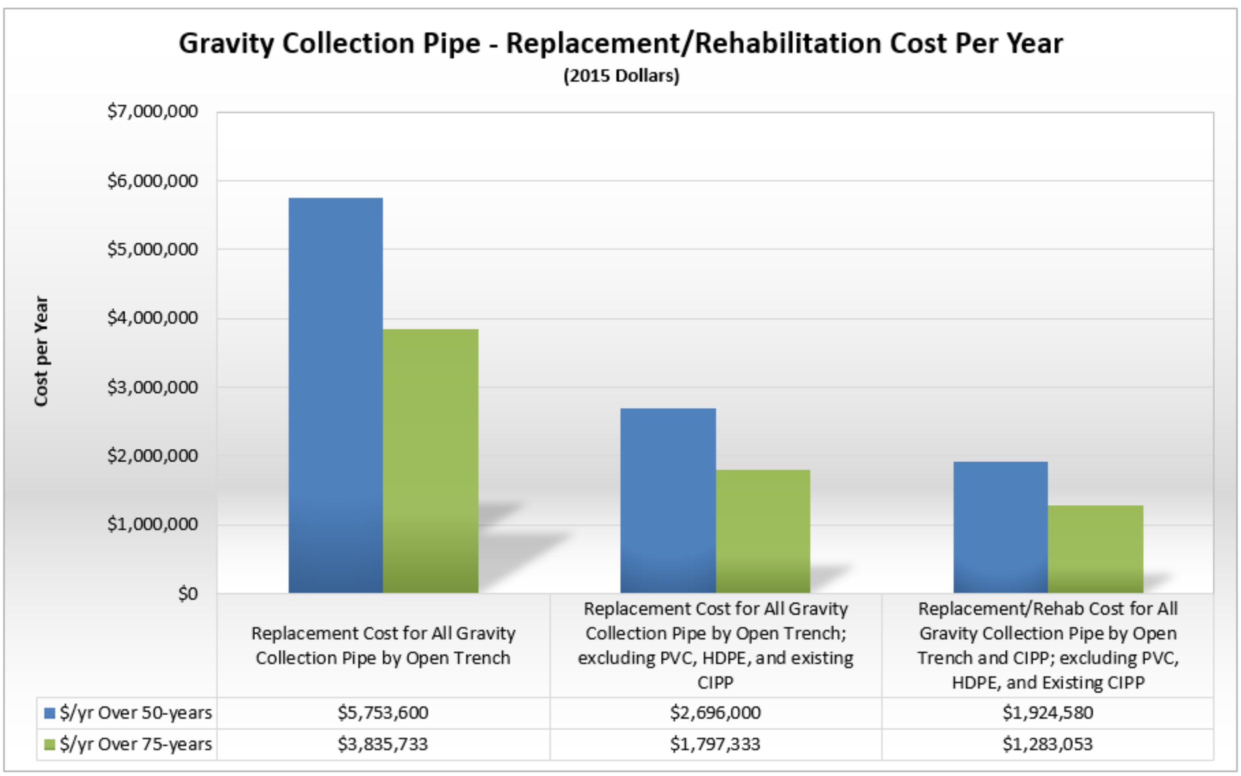
To determine a range that should be targeted for annual budgeting for collection system renewal/replacement, a total system replacement cost was calculated. The cost of total system replacement for the entire existing gravity sewer collection system via trench and replacement construction methods in the City is estimated to be approximately $288 million. This number assumes sewer collection mains only and not the service connections to each parcel.

Sewer collection system pipes have an expected lifetime that ranges from 50 to 75 years. Therefore, assuming that the collection system would need to be replaced every 50 years, beginning today the annual cost for system replacement in 2015 dollars is approximately $5.8 million per year. Assuming replacement every 75 years, the cost is approximately $3.8 million per year.

PVC pipe is expected to have a lifetime of approximately 100 years – and the oldest PVC pipe in the ground today is approximately 25 years old. Therefore, it is reasonable to assume that none of the existing PVC pipe will need to be replaced in the next 75 years. Similarly, it is reasonable to assume that all pipe that has been rehabilitated with CIPP methods will not require additional rehabilitation in the next 75 years – the City has been conducting CIPP rehabilitation on approximately 130,000 LF of pipe since1997. If we exclude all PVC pipe and CIPP pipe from the replacement cost analysis, the total collection system replacement cost decreases to approximately $135 million – assuming trench and replacement construction methods. Assuming that this non-PVC and non-CIPP portion collection system would need to be replaced every 50 years, the annual cost for system replacement in 2015 dollars is approximately $2.7 million per year. Assuming replacement every 75 years, the cost is approximately $1.8 million per year.

To take the analysis one step further, it is reasonable to assume that of the non-PVC pipe and non-CIPP pipe to be replaced, rehabilitation costs will decrease if the pipe can be rehabilitated with CIPP lining versus traditional trench and replace methods. Assuming that approximately half of this non-PVC pipe can be rehabilitated with CIPP while the other half is replaced via trenching, the total system replacement cost in 2015 dollars is approximately $96 million. Implementing the rehabilitation/replacement plan over 50 years is a cost of approximately $1.9 million per year. Assuming rehabilitation/replacement every 75 years, the cost is approximately $1.3 million per year.

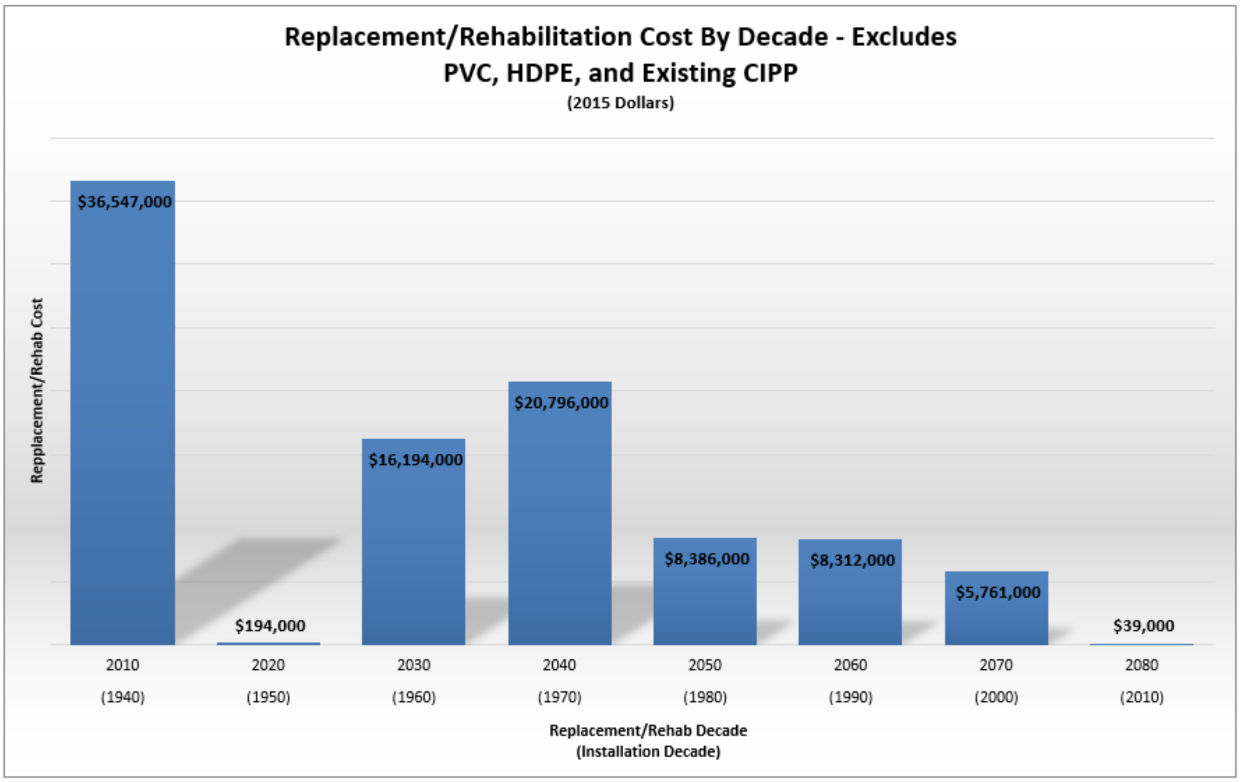
Figure 6-7 – System Replacement Costs



Based upon this analysis, the City should be budgeting somewhere between $1.3 million and $1.9 million (2015 dollars) annually for renewal and replacement of the sewer collection system. In the CIP presented in **Chapter 7**, $1.5 million dollars is budgeted annually beginning in 2017. **Chapter 8** addresses the financial impacts of the renewal/replacement program

It is worth noting that the above analysis does not take into account the age of the existing pipes. The City has limited data on pipe age; however, an estimate of pipe installation by the decade was developed in order to identify the potential timing of replacement. Figure 6-8 depicts potential cost of replacement per decade for the next several decades. This assumes a 75-year lifespan for the non-PVC pipe that has not yet been rehabilitated. Because a significant portion of the City was constructed in the 1940s, replacement of a large portion of the City is likely required soon. This emphasizes the need for CCTV inspection and condition rating of the system in order to verify if the pipes are near the end of their service life. As the City updates the GIS records on pipe installation years, this analysis can be further refined.

Figure 6-8 – Potential Timing of System Replacement Costs



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One other noteworthy item is in regards to service connections. The numbers above are based upon rehabilitation of sewer main pipes only and does not account for rehabilitation of the sewer service connections. As noted above, assuming non-PVC and non-CIPP pipes are replaced with a combination of open-trench and CIPP, this is approximately $96 million in 2015 dollars. If the City decided to also rehabilitate the approximately 9,000 sewer service laterals associated with these projects, it is estimated to add another $23 million to the project cost – or an increase of about 25%.

6.8 Lift Stations

This section discusses the existing condition of the 14 lift stations that the City maintains and includes the existing pump capacities. In addition, this section identifies any needs and upgrades for each lift station.

6.8.1 Description of Existing Facilities

The City currently maintains 14 lift stations. The existing lift stations are generally classified as belonging to one of two different categories, local service or interceptor service. In general, the interceptor lift stations receive flows from large service areas and their operation is important to the overall performance of the collection system. If an interceptor service lift station were to fail, it could have significant impacts as measured by the area affected by flooding. The Montana St lift station serves all of the RY sewer basin and is necessary to convey flows west of SR 240. All other City lift stations serve smaller drainage basins and are therefore classified as local service. For this reason, special attention and prioritization is given to the Montana St station. The City's 14 lift stations are shown in **Figure A1** and are listed in **Table 6-10**.

Table 6-10 – Existing Lift Stations

Lift Station Name
Battelle
Waterfront
Terminal Drive
Mental Health
Bradley
Columbia Pt
Wellhouse Loop
Duportail
Montana St
Columbia Park Trail
Meadows South
Bellerive
Meadow Ridge
Dallas Rd

The following is a general description of each of the lift station facilities.

Battelle Lift Station

The Battelle Lift Station is located in and serves an industrial area in the upper North area of Richland off Battelle Blvd. The station was originally constructed in 1995 as a duplex submersible pump style lift station with 25 hp pumps to serve a drainage basin of approximately 3,400 AC. In 2013, as a part of the Logston Sewer Interceptor project, the drainage basin was significantly decreased in size and routed to the new interceptor. The Battelle Lift Station drainage basin is now approximately 330 AC in size, serving mainly the area east of the lift station and fronting Battelle Blvd. In 2013 the original submersible pumps were replaced with smaller 5 hp Flygt pumps to convey the lower flows through a new, and much shorter, forcemain to the nearby interceptor pipe. The wet well is a 12-ft diameter precast manhole with a depth of that is approximately 18 feet. A 6-inch forcemain (130 LF) is used to transport the pumped sewage to the nearby gravity system discharge manhole located near the intersection of Battelle Blvd and Logston Blvd. The station is reported to have a capacity of 400 gpm (0.58 mgd) with one pump in operation.

Fencing is provided around the station and an intrusion alarm is also provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system.

Waterfront Lift Station

The Waterfront Lift Station is located north of the Penford processing facility in upper North Richland, adjacent to the Columbia River. It serves the processing facility and a small area of residential development, north of the lift station and east of Richardson Rd. The station was constructed in 1977 and has a typical wet-pit/dry-pit configuration. The dry well has a 3.5-ft diameter entrance tube and a 7-ft diameter factory built station that rests on a common foundation slab with the wet well. The wet well is a 6-ft diameter, 19 foot deep structure. Two 15 hp vertical non-clog Fairbanks Morse sewage type pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (260 LF) is used to transport the pumped sewage to the gravity system discharge manhole located near the intersection of Richardson Rd and Lindberg St. The station is reported to have a capacity of 600 gpm (0.86 mgd) with one pump in operation.

Fencing is provided around the station and an intrusion alarm is also provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system. The station is also equipped with a small ventilation fan, heater, and dehumidifier in the dry well. A sump pump is provided and the station does have an alarm in case a leak occurs inside the pump chamber.

No station improvements since 1977, although it is scheduled for replacement by a City standard, submersible style lift station in 2017.

Terminal Drive Lift Station

The Terminal Drive Lift Station is located near the south end of the Richland Airport. It serves the Columbia Basin Racquet Club facility and a small area of commercial parcels along Terminal Dr. The station was constructed in 1981 and has a typical wet-pit/dry-pit configuration. The dry well has a 3.5-ft diameter entrance tube and a 7-ft diameter factory built station that rests on a common foundation slab with the wet well. The wet well is an 8-ft diameter, 15 foot deep structure. Two 3 hp vertical non-clog Fairbanks Morse sewage type pumps are used in a duplex configuration to remove the sewage from the wet well. A 4-inch forcemain (390 LF) is used to transport the pumped sewage to the gravity system discharge manhole located Northeast of the lift station. The station is reported to have a capacity of 150 gpm (0.22 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system. The station is also equipped with a small ventilation fan, heater, and dehumidifier in the dry well. A sump pump is provided and the station does have an alarm in case a leak occurs inside the pump chamber.

Mental Health Lift Station

The Mental Health Lift Station is located on Stevens Drive, just north of the Kadlec Hospital campus. The lift station was originally constructed in 1973 as a submersible pump style lift station. In 2009 the lift station pumps were replaced with two 5 hp Vaughan chopper style pumps to address the constant ragging issues the City was experiencing in the flows to the lift station. The wet well is a 6-ft diameter precast manhole that has a depth of approximately 15 feet. A 4-inch forcemain (195 LF) is used to transport the pumped sewage to the gravity system discharge manhole located in Stevens Drive. The station is reported to have a capacity of 260 pm (0.37 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is provided. The alarms are transmitted to the wastewater treatment plant by the telemetry system.

Bradley Lift Station

The Bradley Lift Station is located east of George Washington Way, at the intersection of Comstock St and Bradley Blvd. It serves the commercial area along Bradley Blvd, mainly north of the Columbia Point Golf Course. The lift station was constructed in 1999 as a submersible pump style lift station. The wet well is a 10-ft diameter precast manhole that is approximately 24 feet deep. Two 10 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (1,774 LF) is used to transport the pumped sewage to the intersection of George Washington Way and Columbia Point Drive where it combines with the 6-inch forcemain from the Columbia Point Lift Station. The combined forcemain is 8-inch in size and discharges into the gravity system discharge manhole located near the intersection of Aaron Drive and Abbot Street. The station is reported to have a capacity of 180 gpm (0.26 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is present. The alarms are transmitted to the wastewater treatment plant by the telemetry system.

Columbia Point Lift Station

The Columbia Point Lift Station is located east of George Washington Way on Columbia Point Drive. It serves the commercial and multi-family area along Columbia Point Drive mainly north and east of the Columbia Point Golf Course. The lift station was constructed in 1999 as a submersible pump style lift station. The wet well is a 10-ft diameter precast manhole that is approximately 20 feet deep. Two 6.5 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (1,800 LF) is used to transport the pumped sewage to the intersection of George Washington Way and Columbia Point Drive where it combines with the 6-inch forcemain from the Bradley Lift Station. The combined forcemain is 8-inch in size and discharges into the gravity system discharge manhole located near the intersection of Aaron Drive and Abbot Street. The station is reported to have a capacity of 270 gpm (0.39 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is present. The alarms are transmitted to the wastewater treatment plant by the telemetry system.

Wellhouse Loop Lift Station

The Wellhouse Loop Lift Station No. is located at the intersection of Wellhouse Loop and Wyman St. It serves a mainly commercial area fronting Wellhouse Loop. The station was constructed in 1978 and has a typical wet-pit/dry-pit configuration. The dry well has a 3.5-ft diameter entrance tube and a 7-ft diameter factory built station that rests on a common foundation slab with the wet well. The wet well is a 6-ft diameter, 17 foot deep structure. Two 1.5 hp vertical non-clog Hydronix sewage type pumps are used in a duplex configuration to remove the sewage from the wet well. A 4-inch forcemain (50 LF) is used to transport the pumped sewage to the gravity system discharge manhole located at the same intersection. The station is reported to have a capacity of 100 gpm (0.14 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system. The station is also equipped with a small ventilation fan, heater, and dehumidifier in the dry well. A sump pump is provided and the station does have an alarm in case a leak occurs inside the pump chamber.

Duportail Lift Station

The Duportail Lift Station is located at the end of the cul-de-sac on Duportail Street, north of the Yakima River. It serves both the single family and the multi-family developments west of the By-Pass Highway (SR 240) and north of the River. The lift station was constructed in 1995 as a submersible pump style lift station. The wet well is a 6-ft diameter precast manhole that is approximately 23 feet deep. Two 7.5 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (860 LF) is used to transport the pumped sewage to the gravity system discharge manhole located near the intersection of Duportail Street and the By-Pass Highway. The station is reported to have a capacity of 200 gpm (0.29 mgd) with one pump in operation.

Fencing is provided around the station and an intrusion alarm is also provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system.

Montana St Lift Station

The Montana Lift Station is located at the end of Montana Street, north of Columbia Park Trail in the Richland Wye area. It serves all of the RY drainage basin which consists of all the area north and east of SR 240 and south of the Yakima River. The station was constructed in 1968 and has a typical wet-pit/dry-pit configuration. The dry well has a 3-ft diameter entrance tube and an 8-ft diameter factory built station that rests on a common foundation slab with the wet well. The wet well is a 6-ft diameter, 23 foot deep structure. Two 30 hp vertical non-clog Smith & Loveless sewage type pumps are used in a duplex configuration to remove the sewage from the wet well. The forcemain consists of both 8-inch AC (1,850 LF) and 10-inch PVC (866 LF) pipe and conveys sewage flows to the gravity system discharge manhole located west of the roundabout at Columbia Park Trail and N Steptoe Street. The station is reported to have a capacity of 970 gpm (1.40 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system. The station is also equipped with a small ventilation fan, heater, and dehumidifier in the dry well. A sump pump is provided and the station does have an alarm in case a leak occurs inside the pump chamber.

Columbia Park Trail Lift Station

The Columbia Park Trail Lift Station is located on Columbia Park Trail, adjacent to the Columbia River. It serves the Hanford Reach Interpretive Center and the Fowler and Tapteal Lift Station drainage basins (both these older style lift stations were abandoned). The lift station was constructed in 2012 as a submersible pump style lift station. The wet well is an 8-ft diameter precast manhole that is approximately 13 feet deep. Two 10 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (1,320 LF) is used to transport the pumped sewage to the gravity system discharge manhole located near the intersection of Columbia Center Blvd and Columbia Park Trail. The station is reported to have a capacity of 400 gpm (0.58 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system.

Meadows South Lift Station

The Meadows South Lift Station is located at the end of Meadows Drive South and adjacent to the Meadow Springs Golf Course. It serves an area of single family and multi-family development off Meadows Drive, generally west of Bellerive Drive. The station was constructed in the 1970's and has a typical wet-pit/dry-pit configuration. No record drawings were available, although the City clarified that the wet well is a 7-ft diameter, 21 foot deep structure. Two 3 hp vertical non-clog Hydromatic sewage type pumps are used in a duplex configuration to remove the sewage from the wet well. A 4-inch forcemain (90 LF) is used to transport the pumped sewage to the gravity system discharge manhole located in Blalock Court, north of the lift station. The station is reported to have a capacity of 100 gpm (0.14 mgd) with one pump in operation.

Fencing is provided around the station and an intrusion alarm is also provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system. The station is also equipped with a small ventilation fan, heater, and dehumidifier in the dry well. A sump pump is provided and the station does have an alarm in case a leak occurs inside the pump chamber.

Bellerive Lift Station

The Bellerive Lift Station is located on Bellerive Ct, east of Claybell Park in South Richland. It serves single family residential development along Broadmoor St (to the west) and the Heights at Meadow Sprints development (to the east). The lift station was constructed in 2005 as a submersible pump style lift station. The wet well is a 6-ft diameter precast manhole that is approximately 28 feet deep. Two 15 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (1,980 LF) is used to transport the pumped sewage to the gravity system discharge manhole located north of Gage Blvd, on Bellerive Dr. The station is reported to have a capacity of 260 gpm (0.37 mgd) with one pump in operation.

In 2011 the Leslie Sewer Trunk was constructed to the south and connected to the lift station. The 18-inch trunk pipe was designed to collect all flows south of the lift station, including the east half of the Badger Mountain South planned development. As discussed in the SRSR (see Appendix B) the increased sewer flows to the lift station will exceed the current capacity. The recommended improvements included extending a 21-inch pipe to the north, across the Meadow Springs Golf Course, and connecting to an existing 21-inch trunk pipe on Leslie Road, near the north edge of the Canyon Terrance subdivision. At the lift station there is a 13-ft vertical difference between the Leslie Sewer Trunk influent pipe invert and the local residential collector influent pipe invert and therefore, as part of the improvements, the City may choose to leave the lift station in place, to serve the local collector, in lieu of deepening the trunk pipe. Shallow groundwater was encountered during the lift station construction.

No fencing is provided around the station, but an intrusion alarm is present. The alarms are transmitted to the wastewater treatment plant by the telemetry system.

Meadow Ridge Lift Station

The Meadow Ridge Lift Station is located on Steptoe St, just south of the intersection with Gage Blvd. It serves commercial development south of Gage, between Bellerive Dr and Steptoe St. The lift station was constructed in 2007 as a submersible pump style lift station. The wet well is a 6-ft diameter precast manhole that is approximately 13 feet deep. Two 10 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (1,080 LF) is used to transport the pumped sewage to the gravity system discharge manhole located west of the lift station. The station is reported to have a capacity of 245 gpm (0.35 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is present. The alarms are transmitted to the wastewater treatment plant by the telemetry system.

Dallas Rd Lift Station

The Dallas Road Lift Station is located on Dallas Road, north of the I-82, Dallas Rd exit. It serves the west half of the mixed use development of Badger Mountain South. The lift station was constructed in 2012 as a submersible pump style lift station. The wet well is an 8-ft diameter precast manhole that is approximately 25 feet deep. Two 35 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. The lift station has dual forcemain pipes (9,510 LF of 6-inch and 12-inch) to accommodate increased sewer flows to the station. Currently only the 6-inch forcemain pipe is in use and transports the pumped sewage to the gravity system discharge manhole located generally southeast of the I-182/Dallas Rd undercrossing. The station is reported to have a capacity of 260 gpm (0.37 mgd) with one pump in operation.

Record drawings identify a future bypass of this lift station, to a second lift station that is not currently constructed. No phasing information was available at this time.

Fencing is provided around the station and an intrusion alarm is also provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system.

6.8.2 Proposed Improvements and Future Replacement Schedule

The City has been proactively upgrading the lift stations by replacing pumps and piping and electrical control systems. The following is a list that identifies the scheduled improvements for the lift station upgrades:

2014 – Montana Lift Station Standby Generator:
Installation of an on-site generator to operate the lift station during power outages.

2015 – Columbia Park Trail Lift Station Standby Generator:
Installation of an on-site generator to operate the lift station during power outages.

2017 – Waterfront Lift Station Replacement:
Replace this deficient wet-pit/dry-pit station with a submersible pump style lift station.

Lift stations must also be rehabilitated and replaced as necessary. Mechanical rehabilitation is often required every 15 to 30 years, while electrical upgrades are often required every 15 to 20 years. A major rehabilitation or replacement should be expected every 50 years. As shown in **Table 6-11,** several lift stations will need mechanical and/or electrical upgrades within the next 10 years. None of the lift stations are expected to undergo a major rehabilitation or replacement within the next 10 years, other than those identified in the previous section. For budgetary purposes, the following costs are assumed: a mechanical upgrade is $30,000 to $80,000 and an electrical upgrade is $25,000 to $55,000, depending on the lift station size (2015 dollars).

Table 6-11 – Lift Station Rehabilitation/Replacement Expectations

Lift Station Name	Year Constructed/ Last Major Rehabilitation	Comments	Rehabilitation Expected In…		
			Mechanical (15 to 30 years)	Electrical (15 to 20 years)	Major Rehabilitation/ Replacement (50 years ±)
Battelle	2013		20 - 30 years	20 - 30 years	40 - 50 years
Waterfront[1]	1977		0 - 5 years	0 - 5 years	5 - 15 years
Terminal Drive	1981		0 - 5 years	0 - 5 years	15 - 20 years
Mental Health	2009		10 - 15 years	10 - 15 years	40 - 50 years
Bradley	1999		5 - 10 years	5 - 10 years	30 - 40 years
Columbia Pt	1999		5 - 10 years	5 - 10 years	30 - 40 years
Wellhouse Loop	1978		0 - 5 years	0 - 5 years	5 - 15 years
Duportail[2]	1995		0 - 5 years	0 - 5 years	25 - 35 years
Montana St	1968		0 - 5 years	0 - 5 years	5 - 10 years
Columbia Park Trail	2012		20 - 30 years	20 - 30 years	40 - 50 years
Meadows South	1970's		0 - 5 years	0 - 5 years	5 - 10 years
Bellerive	2005		10 - 15 years	10 - 15 years	40 - 50 years
Meadow Ridge	2007		10 - 15 years	10 - 15 years	40 - 50 years
Dallas Rd	2012		20 - 30 years	20 - 30 years	40 - 50 years

(1) *Waterfront Lift Station is scheduled for a complete station replacement in 2017.*
(2) *Duportail Lift Station is scheduled for relocation and replacement as a part of the Duportail Bridge project. No schedule at this time.*