FOUNTAIN SANITATION DISTRICT
COLLECTION SYSTEM MASTER PLAN UPDATE

FINAL

MARCH 21, 2017

Prepared for:

Fountain Sanitation District
901 S. Santa Fe Avenue
Fountain, CO 80817

Prepared by:

Bohannan Huston

Engineering
Spatial Data
Advanced Technologies

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Prepared by:

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Jerry Edwards, PE Date	Johanna Phillips, EI Date

Bohannan Huston

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ABBREVIATIONS AND ACRONYMS

AAD	Average Annual Day (flow)
BHI	Bohannan Huston, Inc.
BOD5	5-day Total Biochemical Oxygen Demand
CCMD	Colorado Centre Metropolitan District
CCTV	Closed Circuit Television Video
CDPHE	Colorado Department of Public Health and Environment
CSU	Colorado Springs Utilities
d/D	Ratio of flow depth (d) to pipe diameter (D)
EPA	Environmental Protection Agency
FSD	Fountain Sanitation District
gal	Gallons
GOPP	Grease/Oil Program Permit
GPCD	Gallons Per Capita Per Day
GPD	Gallons Per Day
GPM	Gallons Per Minute
HDTRWRF	Harold D. Thompson Regional Water Reclamation Facility
I-25	Interstate 25
IGA	Intergovernmental Agreement
I/I	Infiltration and Inflow
LFMSDD	Lower Fountain Metropolitan Sewage Disposal District
MGD	Million Gallons per Day
NOAA	National Oceanic Atmospheric Administration
NPV	Net Present Value
O&M	Operations and Maintenance
PD	Peak Day
PVC	Polyvinyl Chloride
RJCII WWTF	Richard J. Christian II Wastewater Treatment Facility
USGS	U.S. Geological Survey
VC	Vitrified Clay
WWTF	Wastewater Treatment Facility

1 INTRODUCTION

1.1 PURPOSE AND SCOPE

This document provides an update to the *Sanitary Sewer Collection System Master Plan for Fountain Sanitation District* (FSD or District) that was completed in 2004. The major efforts associated with this update include the following:

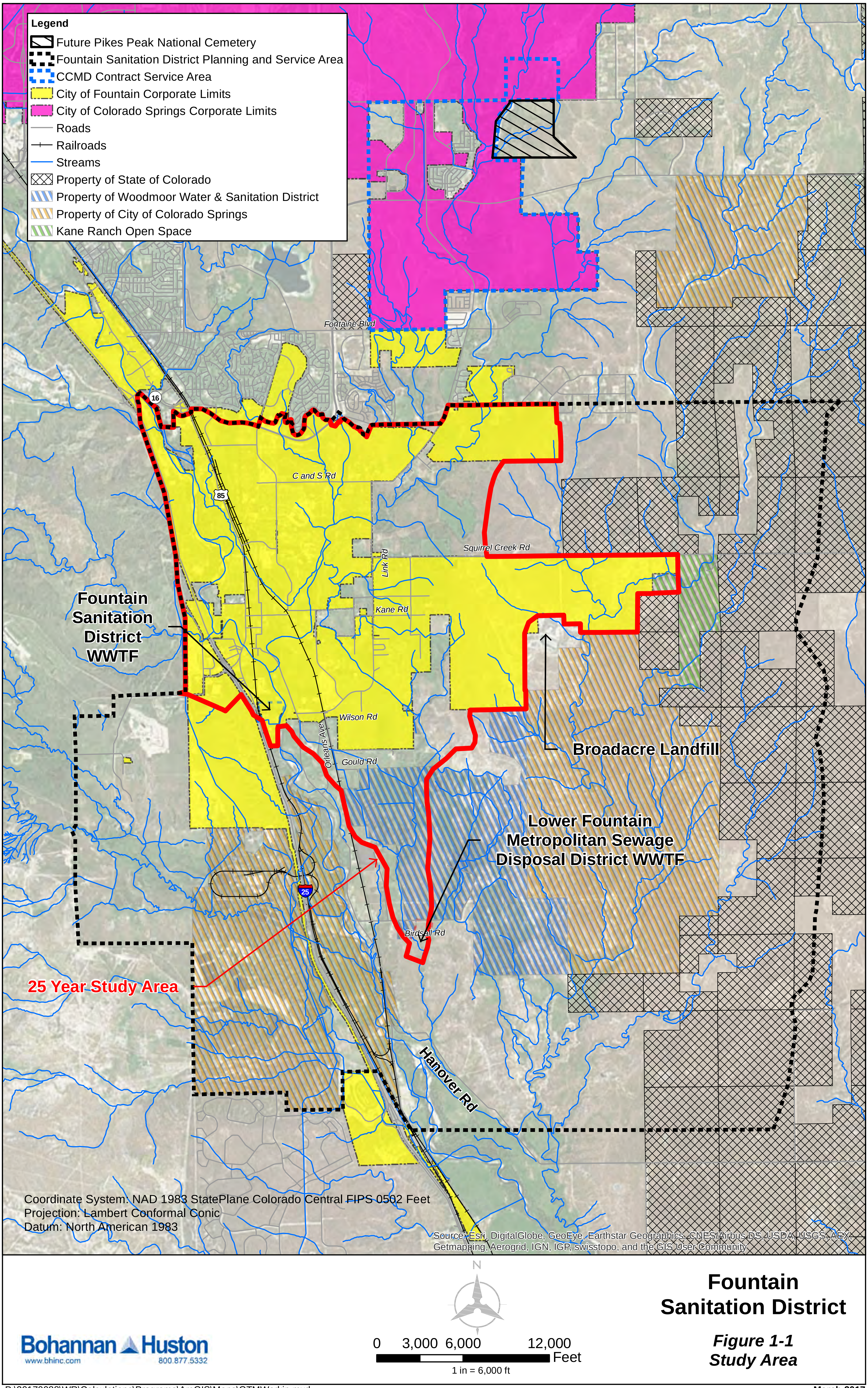
1. Evaluate the general condition of the collection system facilities and summarize historical replacement and rehabilitation activities.
2. Evaluate the capacity of the collection system and determine collection system improvements required to convey current and projected wastewater flows.
3. Prepare a phased plan of improvements with budget level project costs.
4. Deliver a hydraulic model of the collection system upon completion of the report.

The intent of this report is to document the evaluations and recommended improvements, through the planning period, for the FSD collection system. The District shares capacity with the Colorado Centre Metropolitan District (CCMD) in the LFMSDD treatment and conveyance facilities. The shared capacities are established by the Sewage Treatment and Disposal Agreement as amended and approved on January 15, 2009 (2009 Service Agreement). Evaluation of the CCMD service area is not a part of this project. It is assumed for all evaluations that the treatment and conveyance capacity available to FSD is the actual capacities less the amount owned or shared by CCMD.

A 25-year planning period is established for evaluation of recommended improvements. Evaluations within this time period are highly susceptible to changes in the economic drivers that create the need for collection facilities especially toward the latter part of the time frame. Most of the FSD Contract Service Area is large, undeveloped parcels with much of it being outside the drainage basins of the existing wastewater treatment facilities (WWTF). Generally, it will take many decades for areas outside these drainage basins to develop.

1.2 STUDY AREA

The study area for this report is shown of Figure 1.1 on the following page. The study area is the limit for which flow projections and extensions of service are evaluated. A description of the areas shown on Figure 1.1 and their importance to development of the study area are summarized following the exhibit.



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Fountain Sanitation District

Figure 1-1 Study Area

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Author: jphillips

March 2017

25-year Study Area: The 25-year study area (study area) for FSD is defined to be the area within the contract service area and within the existing WWTFs drainage basins. Except for currently known planned unit developments and planned land use, it excludes new areas where pumping would be required to deliver the flows to the existing treatment facilities. Wastewater flow projections and recommended improvements are determined for the 25-year planning period.

FSD Planning and Service Area: This area was established as the "Contract Service Area" by the District in its planning commencing with the area wide Facility 201 plan in 1976. No competing system may provide service within the Contract Service Area without the approval of the District. At present, the District delivers over half of its wastewater flows to the LFMSDD's Harold D. Thompson Regional Water Reclamation Facility (HDTRWRF or HDT Facility). The remainder of the flows are delivered to the District's Richard J. Christian II Wastewater Treatment Facility (RJCIIWWTF or RJC Facility).

CCMD Contract Service Area: This area was established by formation of the CCMD. The CCMD is obligated to provide wastewater disposal and treatment only for those areas not within the City of Colorado Springs corporate limits. All flows from the unincorporated portions of the CCMD service area are currently delivered to the LMFSDD's HDT Facility.

City of Fountain Corporate Limits: The FSD Contract Service Area incorporates most of the City of Fountain Corporate Limits. Areas on the north side of the city, outside of the FSD Contract Service Area, are provided wastewater disposal and treatment by the Widefield Water and Sanitation District and Security Sanitation District.

City of Colorado Springs Corporate Limits: As described above, the CCMD is not obligated to provide wastewater disposal and treatment for the incorporated areas within its contract service area.

Lower Fountain Metropolitan Sewage Disposal District RWRF: The Lower Fountain Metropolitan Sewage Disposal District (LFMSDD) was created in 1985. By definition, it was formed to provide regional wastewater management. The LFMSDD owns the HDT Facility located on this site. The construction of the HDT Facility was completed in 2013 to provide wastewater treatment for FSD and CCMD.

Fountain Sanitation District WWTF: The Richard J. Christian II Wastewater Treatment Facility was constructed in 1998 and is solely owned by the District.

Property of State of Colorado: The State of Colorado owns large tracts of property on the east side of the FSD Contract Service Area. These tracts are set aside for wildlife management or similar activities and will not be developed.

Property of City of Colorado Springs: The City of Colorado Springs has purchased large tracts of property to secure the water rights for use by the City of Colorado Springs. It is expected that these tracts will remain undeveloped within the 25-year planning period of this report.

Property of Woodmoor Water and Sanitation District: The Woodmoor Water and Sanitation District has purchased large tracts of property to secure the water rights for use by the Woodmoor WSD. It is expected that these tracts will remain undeveloped within the first 10 years of the planning period.

1.3 PREVIOUS REPORTS

Fountain Sanitation District Sanitary Sewer Collection System Master Plan, Black & Veatch, June 2003 – This study evaluated the existing wastewater collection system for Fountain Sanitation District to provide long range planning for management of the anticipated buildout of the service area. A calibrated model of the wastewater collection section was used to perform the planning year evaluations. Possible relief and expansion sewers were recommended based on projected 2010 flows and requirements for buildout of the service area. It was recommended that the Little Ranches pumping station and force main be upgraded, as well as construction of a new pumping station and force main to serve the proposed Valley Ranch Subdivision. Options for addressing the wastewater treatment capacity through build-out of the study area were also provided.

Fountain Sanitation District Sanitary Sewer Collection System Master Plan 2004 Enhancements, Black & Veatch, November 2004 – Additional details were requested by Fountain Sanitation District following the issued Sanitary Sewer Collection System Master Plan. The enhancements were documented by a series of technical memoranda. The tasks included development of a long-term flow and rainfall monitoring program (not implemented), a review of concerns with the Colorado Centre force main, an analysis of the Race Street lift station for possible abandonment, a review of wastewater treatment plant process (RJC Facility) and a summary evaluation of treatment plant expansion options. Ultimately, FSD entered into a Service Contract with the LFMSDD and others, and the HDT Facility along with the LFMSDD interceptor sewer system was constructed.

1.4 ACKNOWLEDGEMENTS

Bohannan Huston, Inc. gratefully acknowledges the cooperation and assistance of many people in the preparation of this report. The names of the people who participated in this study and who were instrumental in executing the project and in preparing the report are presented below.

Fountain Sanitation District

James Heckman .... District Manager

Becky Cox .... GIS Analyst

Tim Long .... Operations Superintendent

GMS, Inc.

Roger Sams .... Consulting Engineer to the District

Bohannan Huston, Inc.

Jerry Edwards .... Project Manager

Rob Richardson .... QA/QC Manager

Johanna Phillips .... Project Engineer

Greg Maynard .... GIS Specialist

2 EXISTING FACILITIES

2.1 GENERAL

The collection system comprises two major drainage basins which convey flows to two separate treatment facilities. Figure 2.1 on the following page shows the existing District facilities.

2.2 LFMSDD SEWAGE TREATMENT AND DISPOSAL AGREEMENT

The Lower Fountain Metropolitan Sewage Disposal District (LFMSDD) was created in 1985. By definition, it was formed to provide regional wastewater service. The LFMSDD owns the HDT Facility. The construction of the HDT Facility was completed in 2013 to provide wastewater service for FSD and CCMD. The FSD and the CCMD share ownership and capacity in the HDT Facility and in the new interceptor sewers that were constructed as part of the agreement as summarized below:

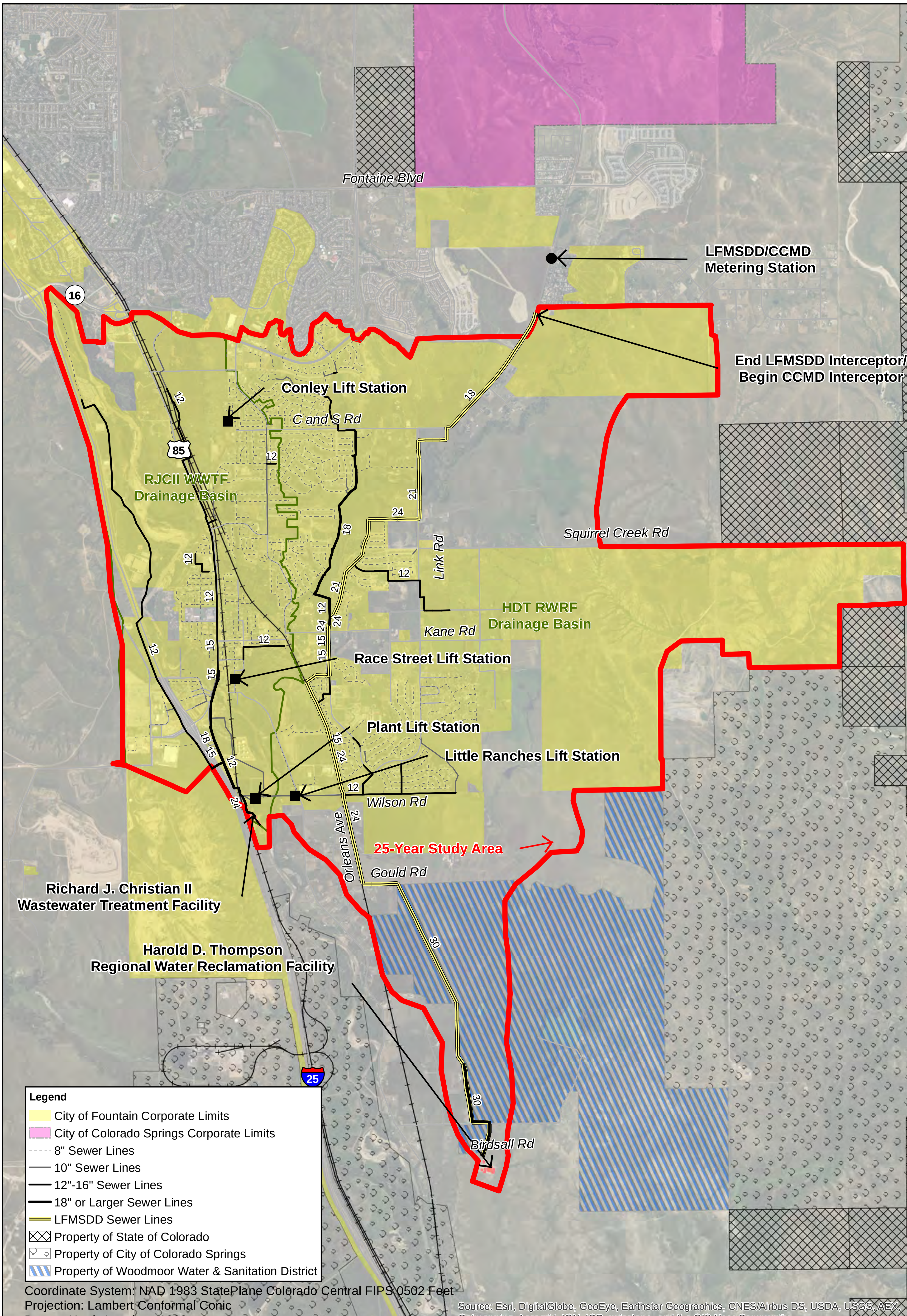
- Treatment
    - CCMD owns 25 percent of the total 2.5 MGD capacity of HDT Facility (equivalent of 0.625 MGD).
    - FSD owns 75 percent of the total 2.5 MGD capacity of the HDT Facility (equivalent of 1.875 MGD).
- Collection
    - CCMD owns 1 MGD maximum daily flow in the interceptor which delivers flows to HDT Facility.
    - FSD owns the remaining capacity.

In the lower reaches near the HDT Facility, a secondary 30-inch diameter sewer interceptor was installed for a portion of the alignment at the District's sole expense. The parallel sewer was constructed because of deep cuts required for construction and the desire to avoid reopening those in the future when additional conveyance capacity is required by FSD.

2.3 EXISTING FACILITIES DESCRIPTION AND INVENTORY

2.3.1 TREATMENT FACILITIES

As noted earlier, the collection system comprises two major drainage areas which convey flows to two separate treatment facilities. Flows within the Fountain Creek drainage basin are conveyed to the RJC Facility.



N

0 2,000 4,000 8,000 Feet

1 in = 4,000 ft

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Figure 2-1
Existing Facilities

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Author: jphillips

March 2017

The RJC Facility was built in July 1998. This treatment facility utilizes an extended aeration activated sludge process. The treatment process facilities were designed for a maximum month average daily flow of 1.56 MGD and an organic loading capacity of 2,298 pounds of 5-day total biochemical oxygen demand (BOD5). Treatment facility improvements were completed in 2006 increasing the maximum month average daily flow to 1.908 MGD and an organic loading capacity to 5,808 pounds of 5-day total biochemical oxygen demand (BOD5). The design peak flow is 2.73 MGD. The influent sewers, channels, and process piping were designed with a hydraulic capacity of 5.0 MGD to facilitate future expansion.

Flows within the Jimmy Camp Creek drainage basin are conveyed to the Harold D. Thompson Regional Water Reclamation Facility. The HDT Facility was completed in 2013 and has a hydraulic design capacity of 2.5 MGD (30-day average). The facilities were designed for an organic loading capacity of 9,744 pounds per day of BOD5. This facility utilizes an extended aeration activated sludge process with nutrient control and the option to convert to a 4-stage Bardenpho process. Table 2.1 provides a summary of the two existing treatment plants.

Table 2.1 – Capacity of Existing Treatment Facilities

Name	Year Placed into Service	MM Design Capacity[1] (MGD)	FSD Owned Portion (MGD)	CCMD Owned Portion (MGD)
Richard J. Christian II Wastewater Treatment Facility	1998	1.906	1.906	0
Harold D. Thompson Regional Water Reclamation Facility	2013	2.50	1.875	0.625

1 Maximum Month (MM) design capacity

The total treatment capacity (30-day average) of the FSD system, excluding the capacity in the HDT Facility which is owned by CCMD, is 3.781 MGD.

2.3.2 LIFT STATIONS

There are four lift stations in the collection system.

- The Little Ranches lift station was originally constructed to pump flows collected in the Jimmy Camp Creek drainage basin to the RJC Facility. Upon completion of the LFMSDD treatment plant and interceptor sewers, it was taken out of service because it was no longer required. It is currently back in service to serve a small development located just north of the lift station. This area has an average daily flow rate that is much less than the capacity of the station. The pumps in the station have not been replaced since it was repurposed to serve only the new development so the station sees limited use during the day. A 10-inch force main extends over 2,600 feet west to connect to the influent sewer for RJC facility. The Little Ranches pump station includes three 750-GPM pumping units, for a total rated capacity of 2,250 GPM (3.2 MGD). The firm rated capacity with one pump out of service is 1,500 GPM (2.1 MGD).

- The Conley lift station currently serves approximately 60 residential service connections in the northwest portion of the service area. This small station collects flow from cul-de-sacs accessed off Conley Boulevard, which include nine manholes. It is located within Conley Subdivision west of Conley Boulevard and south of Maram Way. The District keeps a spare pump in inventory at all times.
- The Race Street lift station is located on Race Street one block south of Indiana Avenue. This station currently serves approximately 25 residential service connections and is located in the southwestern portion of the service area. It collects flow from only four manholes and approximately 925 feet of gravity sewers. According to a 2004 memorandum, the pumps in the pump station are standard pumps that are sold by a local retailer and are easy to acquire. Further, it was reported that the pumps in the pump station have to be replaced about once every three years. The District keeps a spare pump in inventory at all times.
- The Plant pump station, located near RJC Facility, serves less than 50 residential service connections.

2.3.3 PIPELINES

The total length of sewers within the collection system is approximately 500,000 linear feet (95 miles) including 46,380 linear feet (8.5 miles) of shared capacity lines from the development of LFMSDD. The length of existing lines within the system summarized by diameter, and including the LFMSDD lines, is shown in Table 2.2. Table 2.3 summarizes the year that existing lines were installed by length and diameter.

Table 2.2 – Length of Existing Pipelines by Material[1]

Diameter	Plastic[2]	Vitrified Clay Pipe (VCP)	Ductile Iron Pipe (DIP)	Concrete Truss Pipe (CTP)	Unknown Material	Lined – Unknown Material
4[3] & 6	2,310	630	0	0	0	0
8	277,630	56,710	5,360	11,570	3,630	0
10	17,710	7,000	80	1,100	2,660	0
12	45,910	1,120	1,900	500	1,660	0
15 & 16[4]	15,240	0	60	0	0	250
18	10,750	4,610	600	0	0	0
21	11,450	0	0	0	0	0
24	3,990	0	1,540	0	0	0
30	13,310	0	0	0	0	0
Total	**398,300**	**70,100**	**9,540**	**13,170**	**7,960**	**250**

[1] Totals in this table include LFMSDD pipelines
[2] Plastic pipe includes PVC, ABS, and HDPE materials
[3] Less than 400 feet of 4-inch VCP pipeline in inventory
[4] Less than 400 feet of 16-inch DIP and HDPE pipeline in inventory

Table 2.3 – Length of Existing Pipelines by Year Installed[1]

Diameter	Before 1970	1970-1979	1980-1989	1990-1999	2000-2009	2010-Present
4 and 6	630	0	360	1,950	0	0
8	52,350	26,770	56,240	34,040	163,030	38,490
10	4,300	9,770	3,310	780	8,310	2,060
12	1,710	9,830	14,620	8,500	12,120	4,300
15 and 16	350	4,080	8,070	0	0	3,440
18	5,290	990	0	6,310	3,090	10,350
21	0	0	0	0	4,140	1,710
24	1550	0	0	340	5,440	9,070
30	0	0	0	0	9,730	10,780
Total	**66,170**	**51,440**	**82,600**	**51,930**	**205,860**	**80,220**

[1] Totals in this table include LFMSDD pipelines

The tables above include lines that have shared capacity with LFMSDD. Table 2.4 identifies the length and diameter of these lines within LFMSDD. There are three aerial sewer lines within the system which cross over Fountain Creek or a tributary to Fountain Creek and two lines crossing under Jimmy Camp Creek.

Table 2.4 – LFMSDD Pipelines

Diameter (inches)	Length (feet)
18	10,350
21	5,860
24	14,510
30	15,660
Total	**46,380**

[1] All pipelines are constructed of plastic materials including PVC and HDPE
[2] All pipelines were constructed between 2009 and 2013

2.3.4 MANHOLES

There are 1,814 sanitary sewer manholes in the FSD Service Area, as included in the GIS, and 139 sanitary sewer manholes that serve the shared capacity lines for LFMSDD. Table 2.5 and Table 2.6 list the construction material and diameter of existing manholes, respectively.

Table 2.5 – Existing Manholes: Materials of Construction[1]

Material	Number of FSD Manholes	Number of LFMSDD Manholes
Concrete	1,641	139
Brick	32	0
Concrete and Brick	29	0
Unknown	112	0
Total	**1,814**	**139**

[1] Pump stations are not included

Table 2.6 – Existing Manholes: Diameter[1]

Diameter (feet)	Number of FSD Manholes	Number of LFMSDD Manholes
4	1,049	0
5	84	59
6	3	80
Unknown	678	0
Total	**1,814**	**139**

[1] Pump stations are not included

2.4 CONDITION OF EXISTING FACILITIES

2.4.1 GENERAL

FSD reports a low rate of sewer blockages and overflows in the collection system. According to the District Manager, the last reportable “spill” incident took place in 1999. The Colorado Department of Public Health and the Environment (CDPHE) considers a “spill” reportable if it reaches state waters. No residential backup has occurred within the system since 2013.

A review of historical wastewater flows show that I/I is not a significant problem in the FSD collection system. Almost 60 percent of the collection system has been constructed since year 2000. This newer system results in a low amount of infiltration and inflow (I/I) in the system. In addition, the District has an ongoing pipeline and manhole rehabilitation program for their older facilities.

The District has documented issues of flat lines and grease in lines on Race Street, Wilson Road, Illinois Avenue, Cherry Circle, and Monterey Way. They routinely clean and maintain these pipelines. There is a capital improvement plan for replacement of a line from Monterey Way to Comanche Court due to slope issues of the line.

2.4.2 Replacement and Rehabilitation Activities

The District has a proactive maintenance, replacement, and rehabilitation program in effect. FSD acquired a CCTV truck in 2005 to perform their own inspections and assist with identifying pipelines to be relined.

Ongoing rehabilitation to sewer lines occurs annually. Existing VCP pipes are lined with a thermosetting plastic resin. On average, approximately 2000 feet of pipe are relined each year. The budget for this effort is established at $100,000 per year. A total of approximately 8,500 feet of pipe have been relined. The District reports that in recent years an average of 10 manholes are rehabilitated with a cementitious or epoxy coating. Point repairs up to 4 feet in length are made by District staff with cured-in-place repair system.

The District has installed rain caps on sewer manholes that are prone to flooding. GIS records identify at total of 102 rain caps on FSD manholes and an additional 139 on LFMSDD manholes, comprising over 12 percent of all manholes. They continue to assess vulnerable manhole locations and install additional rain caps as necessary.

2.4.3 Grease and Oil Program

Non-residential properties which prepare and sell food or where vehicle parking and automotive services occur, must obtain a Grease/Oil Program Permit (GOPP) from FSD. FSD performs an inspection of the property in order to specify within the permit if a grease trap, grease interceptor, or sand/oil interceptor is required. The District’s continued enforcement of the GOPP minimizes problems in the collection system and is considered effective.

3 WASTEWATER FLOWS

3.1 HISTORICAL POPULATION

Historical service connections were provided by the District. FSD currently provides service to over 8,100 system connections (residential, commercial, and industrial) as summarized in Table 3.1.

Table 3.1 – Historical Service Connections

Year	Residential Connections	Commercial and Industrial Connections	Total Connections
2002	4587	145	4732
2003	4925	148	5073
2004	5371	152	5523
2005	6042	154	6196
2006	6544	156	6700
2007	6881	160	7041
2008	7000	163	7163
2009	7114	164	7278
2010	7178	170	7348
2011	7251	172	7423
2012	7510	179	7689
2013	7680	190	7870
2014	7803	195	7998
2015	7920	201	8121
2016	8004	216	8220

The 2014 population for the City of Fountain was reported by the U.S. Census Bureau to be 27,781 with an average household density of 2.95 people per household. Using the Fountain average household density and applying to the number of residential connections, the estimated year 2015 service population for the District is approximately 23,400 persons (7,920 x 2.95 = 23,364).

3.2 HISTORICAL WASTEWATER FLOWS

3.2.1 AVERAGE ANNUAL DAILY FLOWS

Historical average annual daily (AAD) flows were provided by the District for each of the treatment facilities and are shown in Table 3.2.

Table 3.2 – Historical Average Annual Day(AAD) Flows by Treatment Facility

Year	Average Annual Day Flow (MGD)		
	RJC Facility	HDT Facility[1]	Total
2000	1.04	-	1.04
2001	1.10	-	1.10
2002	1.12	-	1.12
2003	1.09	-	1.09
2004	1.12	-	1.12
2005	1.15	-	1.15
2006	1.18	-	1.18
2007	1.26	-	1.26
2008	1.28	-	1.28
2009	1.32	-	1.32
2010	1.33	-	1.33
2011	1.31	-	1.31
2012	1.20	-	1.20
2013	1.15[1]	0.12[2]	1.28
2014	0.55	0.83	1.38
2015	0.57	0.88	1.45
2016	0.58[3]	0.92[3]	1.50

[1] The RJC facility was treating an average of 1.25 MGD between January and September 2013. After the HDT facility came online in October, influent flows dropped to an average of 0.82 mgd.

[2] The HDT facility was brought on line in October 2013 and treated an average daily flow of 0.46 MGD for the last three months of the year.

[3] Year 2016 flows based on flow data through November 2016 for RJC Facility and through mid-December 2016 for HDT Facility.

To evaluate daily flow patterns and historical wet weather flows, the District provided daily flows for both treatment facilities for year 2015. Only partial year 2016 flow data was available at the time the detailed analyses were conducted. Figure 3.1 on the following page shows the daily flows for each of the two treatment plants for 2015.

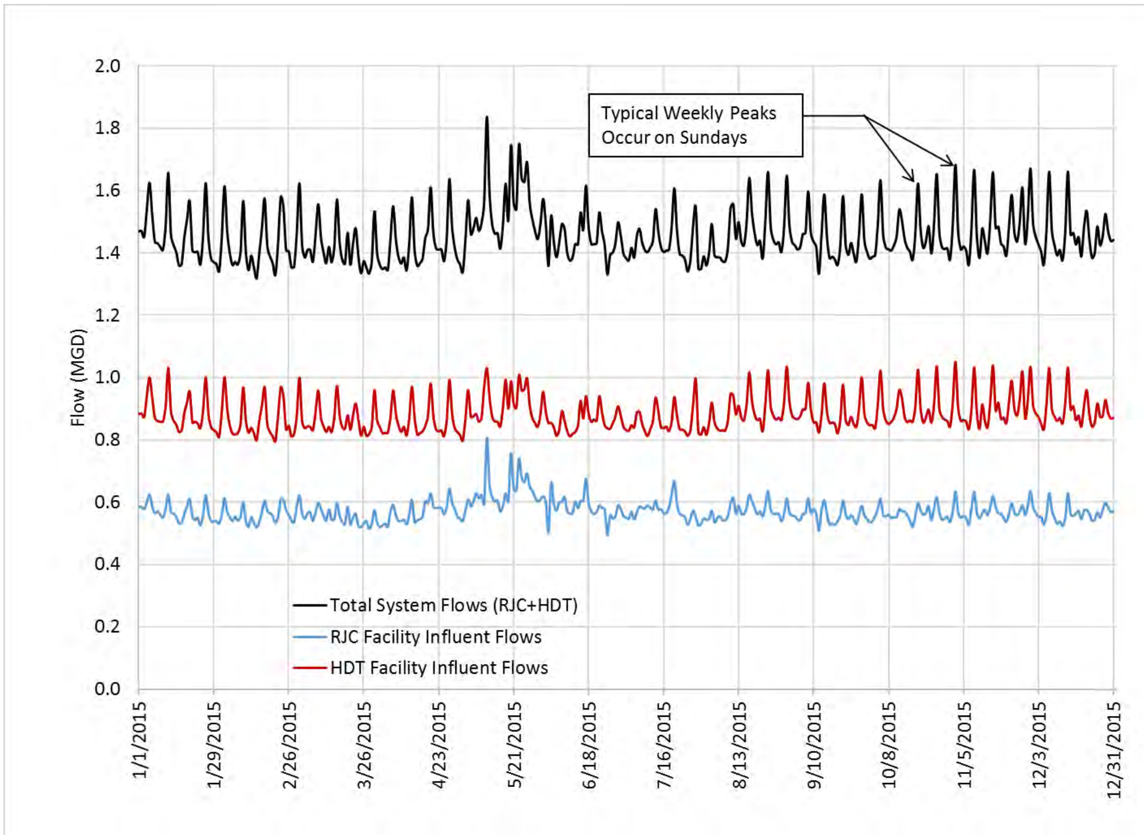

Figure 3.1 – Year 2015 Daily Flows by Treatment Facility

A review of the raw data indicated that a weekly 24-hour peak flow was recorded most every Monday morning, representing the typical Sunday usage. These recorded weekly peak flows can be seen in Figure 3.1 as recurring peaks.

In addition to the daily flows for each treatment plant, daily flows for 2015 were provided for the CCMD metering station. Figure 3.2 on the following page shows the daily flows for the CCMD metering station together with the daily flows for the HDT Facility for year 2015.

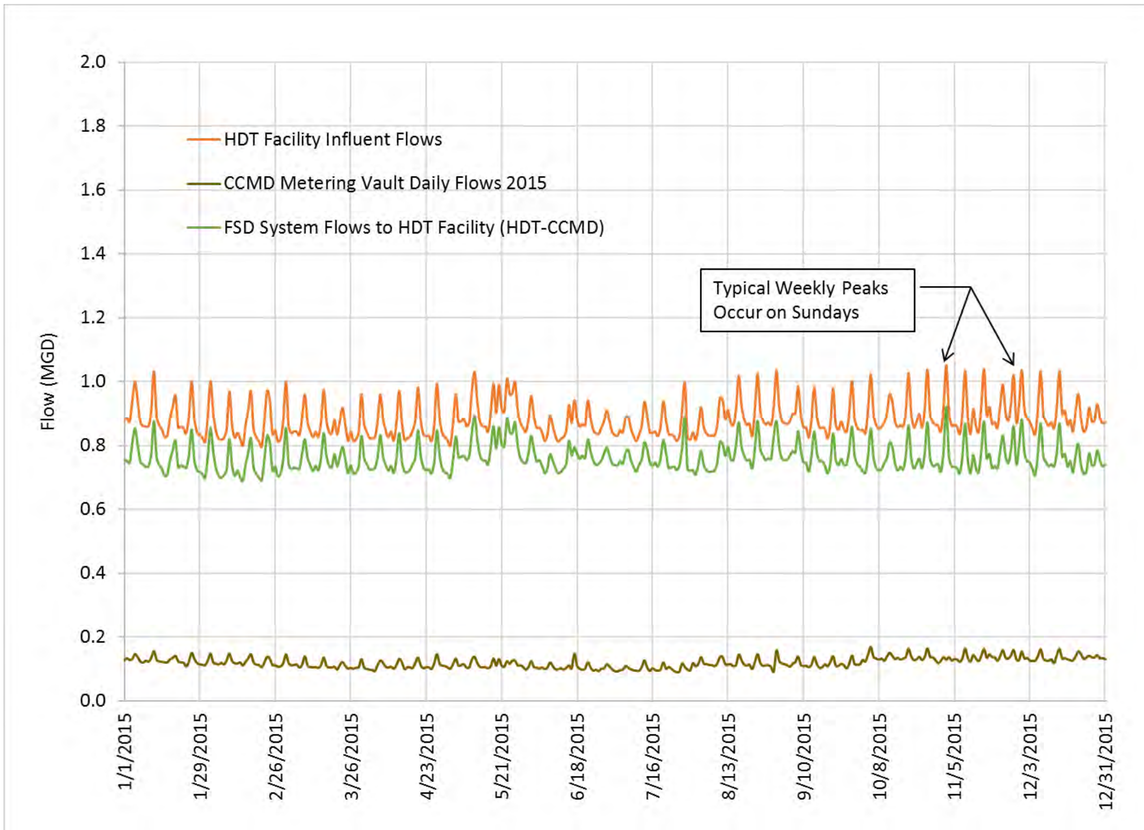

Figure 3.2 – Year 2015 Daily Flows for Thompson Plant and CCMD Meter Vault

A review of Figure 3.2 shows that during the wet weather period in 2015, where flows noticeably increase at the HDT Facility, there are no noticeable increase in flows at the CCMD metering station.

3.2.2 Average Dry Weather Flows

Following a review of the daily recorded flows, additional data was requested and received from the District for the dry-weather period covering September 2 through September 25, 2015 (24 days). This additional data consisted of instantaneous flows recorded every 15 minutes. Instantaneous 15-minute flows for the week of Sunday, September 13 through Saturday, September 19, 2015, were used as the basis for the average dry weather flows for each plant. Figure 3.3 displays the influent flows for each plant during this dry weather week.

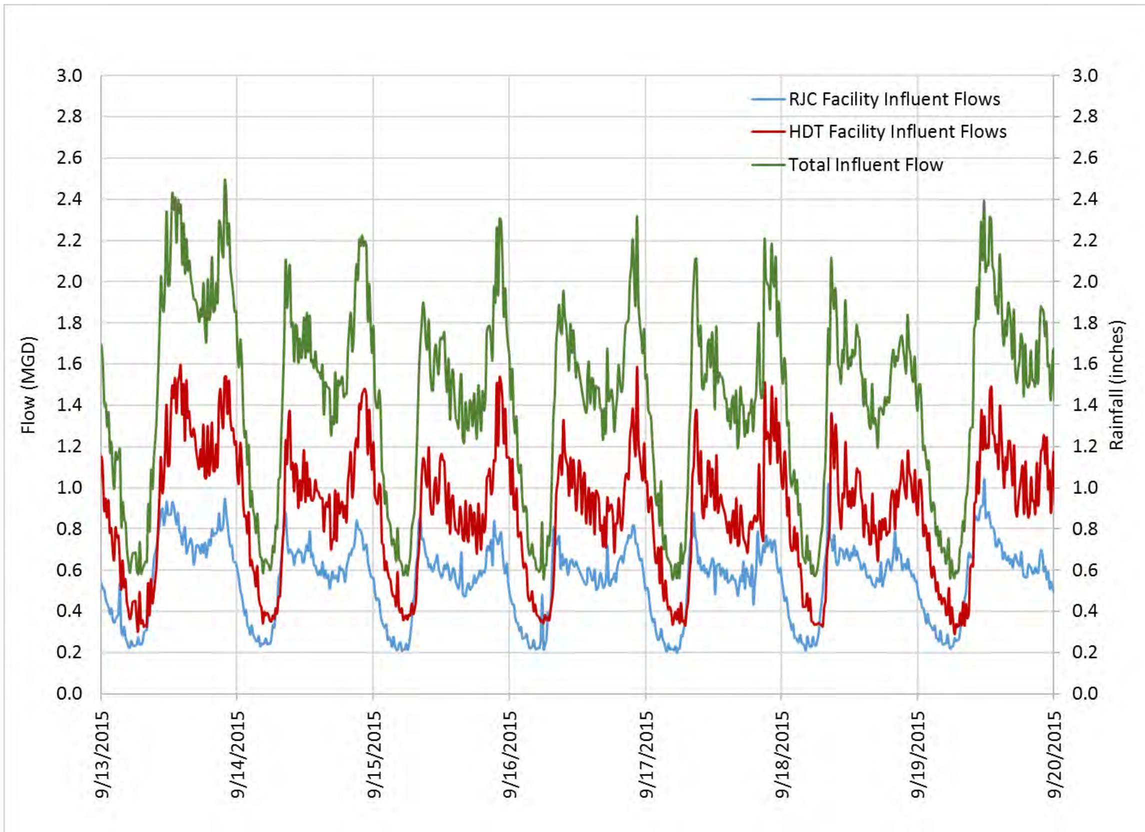

Figure 3.3 – Dry Weather Flows by Plant: Year 2015 Dry Weather Week

The average dry weather flow and the instantaneous peak flow are summarized for each plant for the dry weather week. Table 3.3 lists the ADWF and the instantaneous Peak Dry Weather Flow (PDWF), as well as the Average Annual Daily Flow (AADF) for each plant.

Table 3.3 – Year 2015 Dry Weather Flow by Treatment Facility

Plant	Year 2015 Dry Weather Flow[1]				
	Dry Weather Flow (MGD)	Annual Average Day (AAD) Flow (MGD)	24-hour Dry Weather Flow: AAD Flow (Ratio)	1-hour Maximum Dry Weather Flow (MGD)	1-hour Maximum Dry Weather Flow: AAD Flow (Ratio)
RJC Facility	0.551	0.572	0.97	1.04	1.89
HDT Facility	0.877	0.883	0.99	1.59	1.82

[1]Flows recorded for the dry-weather week of September t 13 through September 19, 2015

The average flow during dry periods is only slightly less that the average annual daily flow. For projection of future demands, the initial demand calculated based on land use will represent the dry weather flow (DWF) and will be multiplied by 1.05 to represent a conservative value for the average annual daily flow (AAD).

Using the estimated service population of 23,400 individuals in 2015 and the AAD flow for both plants, the average wastewater generated per capita was approximately 61 gallons per capita per day (gpcd).

3.2.3 WET WEATHER FLOWS

Historical rainfall records were reviewed and used to identify periods of high rainfall for year 2015 and 2016. An extended period of wet weather was identified in May 2015 and another in April 2016. Additional flow data was requested and received from the District for the wet-weather period of April 30 to May 24, 2015 (25 days), and for the wet-weather period from April 9 through May 2, 2016 (25 days). The additional data consisted of instantaneous flows recorded every 15 minutes.

Recorded 15-minute instantaneous flows for one week in each of the two wet weather periods were evaluated in detail. Recorded flows and rainfall for the selected wet weather week in 2015 are shown on Figure 3.4. Recorded flows and rainfall for the selected wet weather week in 2016 are shown on Figure 3.5.

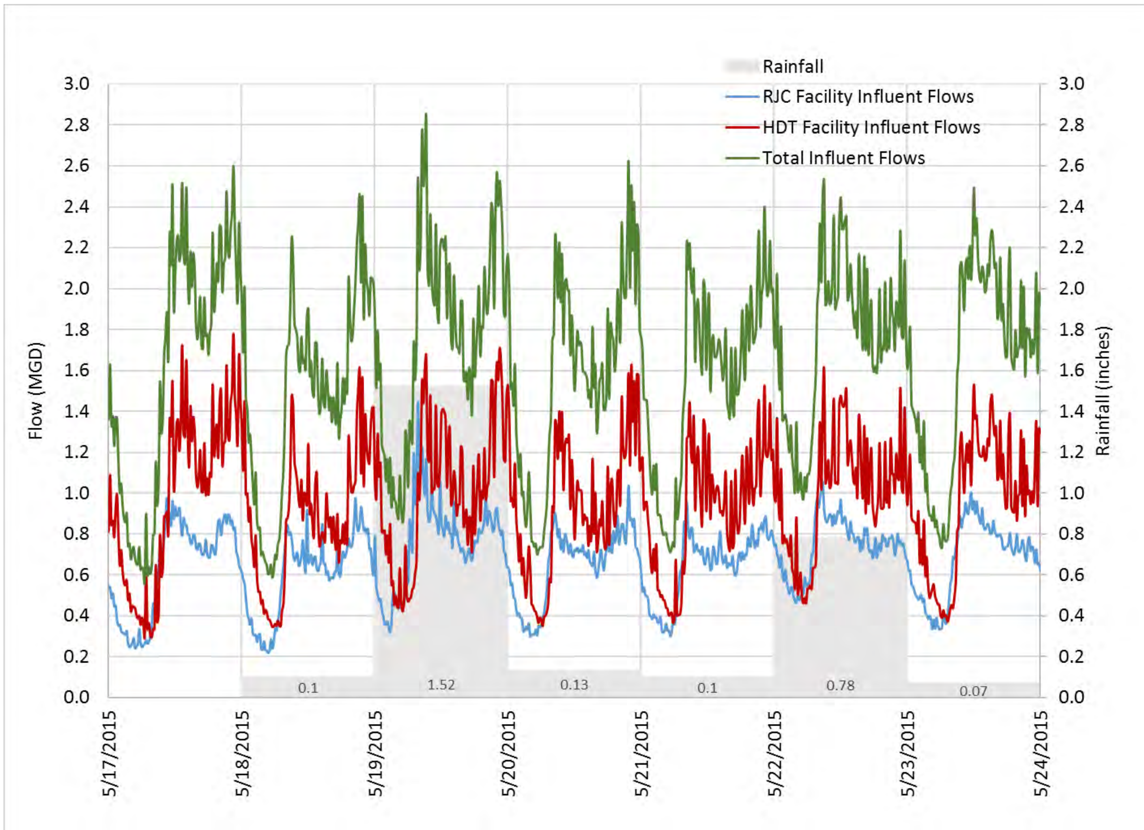

Figure 3.4 – Wet Weather Flows by Plant: Year 2015 Wet Weather Week

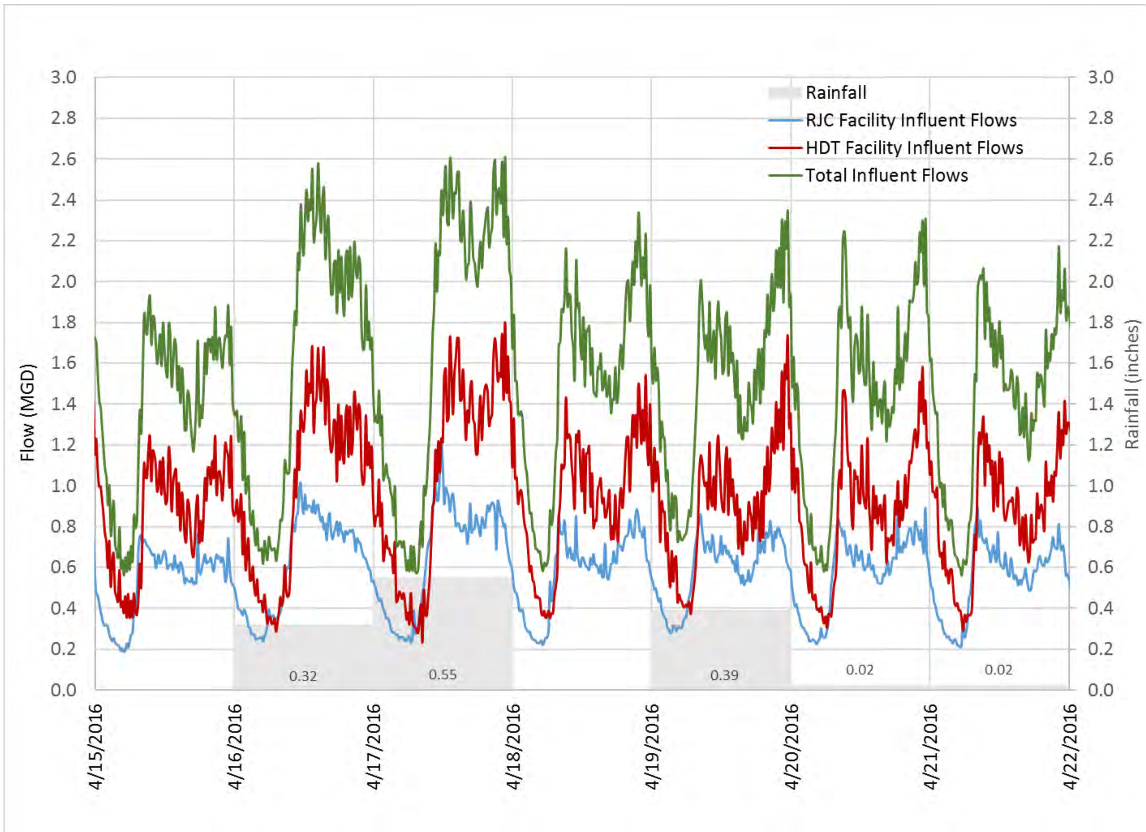

Figure 3.5 – Wet Weather Flows by Plant: Year 2016 Wet Weather Week

During the 2015 wet weather week, the RJC Facility saw a greater percentage increase in flows than the HDT Facility. The ratio of 24-hour flows to ADWF and the ratio instantaneous peak flows to ADWF were both higher for the RJC Facility as shown in Table 3.4.

Table 3.4 – May 2015 Wet Weather Flows by Treatment Facility

Plant	ADWF (MGD)	May 2015 Wet Weather Flows			
		24-Hour Wet Weather Flow (MGD)	24-Hour Wet Weather Flow: ADWF (Ratio)	1-hour Peak Flow (MGD)	1-hour Peak Flow: ADWF (Ratio)
RJC Facility	0.55	0.76	1.38	1.45	2.63
HDT Facility	0.88	1.02	1.16	1.78	2.03

During the 2016 wet weather week, the percentage increase in 24-hour flows for both plants was minimal. However, similar to the 2015 wet weather week, the ratio of 24-hour flows to ADWF and the ratio instantaneous peak flows to ADWF were both higher for the RJC Facility as shown in Table 3.5.

Table 3.5 – April 2016 Wet Weather Flows by Treatment Facility

Plant	ADWF (MGD)	May 2015 Wet Weather Flows			
		24-Hour Wet Weather Flow (MGD)	24-Hour Wet Weather Flow: ADWF (Ratio)	1-hour Peak Flow (MGD)	1-hour Peak Flow: ADWF (Ratio)
RJC Facility	0.55	0.58	1.06	1.21	2.19
HDT Facility	0.88	0.92	1.04	1.80	2.05

3.2.4 DETERMINATION OF PEAKING FACTORS

The rainfall intensity-duration relationships for the Fountain area are available in the Technical Paper 40, "Rainfall Frequency Atlas of the United States," published by the former U.S. Weather Bureau. The 24-hour rainfall for different return periods for the Fountain area are shown in Table 3.6.

Table 3.6 – Fountain Area Rainfall and Return Periods

Return Period (years)	Total 24-hour Rainfall (inches)[1]
1	1.4
2	1.7
5	2.4
10	2.9
25	3.3
50	3.8
100	4.2

[1]From Technical Paper 40, "Rainfall Frequency Atlas of the United States."

Historical rainfalls for five wet weather days were analyzed for this report. The five days are those within with the highest recorded rainfall in 2015 and 2016. The rainfall of 1.52 inches that occurred on May 19, 2015, approximates a 1.5-year storm event. The remainder of the rainfall events were less than the design 1-year, 24-hour rainfall. For each of the five wet weather days, the 24-hour flow and the 1-hour peaking factors were determined as shown on Table 3.7 and Table 3.8 for each plant.

Table 3.7 – RJC Facility Historical Wet Weather Peaking Factors

Date	24-hour Rainfall (inches)	Average Day (MGD)	24-hour Wet Weather Flow		1-Hour Wet Weather Flow	
			MGD	Ratio to AAD	MGD	Ratio to AAD
5/19/2015	1.52	0.57[1]	0.760	1.334	1.447	2.539
5/22/2015	0.78	0.57[1]	0.743	1.303	1.053	1.848
4/16/2016	0.32	0.56[2]	0.619	1.106	1.016	1.815
4/17/2016	0.55	0.56[2]	0.656	1.171	1.211	2.162
4/19/2016	0.39	0.56[2]	0.589	1.053	0.861	1.538

[1]Year 2015 AAD

[2]Only partial flow data was available for year 2016 with a noticeable increase in flows throughout the year. The average day shown here is for April 2016.

Table 3.8 – HDT Facility Historical Wet Weather Peaking Factors

Date	24-hour Rainfall (inches)	Average Day (MGD)	24-hour Wet Weather Flow		1-Hour Wet Weather Flow	
			MGD	Ratio to AAD	MGD	Ratio to AAD
5/19/2015	1.52	0.84[1]	1.013	1.151	1.709	1.308
5/22/2015	0.78	0.84[1]	1.004	1.140	1.605	1.296
4/16/2016	0.32	0.90[2]	0.975	1.084	1.680	1.204
4/17/2016	0.55	0.90[2]	1.041	1.157	1.798	1.286
4/19/2016	0.39	0.90[2]	0.906	1.006	1.731	1.118

[1]Year 2015 AAD

[2]Only partial flow data was available for year 2016 with a noticeable increase in flows throughout the year. The average day shown here is for April 2016.

Using the historical information, equations were developed that relate the peaking factor to the rainfall. Separate graphs were developed for each plant's collection system and are provided in Appendix A. Using these graphs, wet weather peaking factors are developed for each return period, for each plant, as shown in Table 3.9.

Table 3.9 – Design Wet Weather Peaking Factors by Return Period

Return Period (years)	Design 24-hr Rainfall (inches)	24-hour Wet Weather Flow Peaking Factor		1-hour Wet Weather Flow Peaking Factor	
		RJC Facility	HDT Facility	RJC Facility	HDT Facility
1	1.4	1.36	1.19	2.40	1.34
2	1.7	1.44	1.22	2.59	1.37
5	2.4	1.62	1.29	3.04	1.44
10	2.9	1.75	1.34	3.36	1.49
25	3.3	1.86	1.38	3.61	1.53
50	3.8	1.99	1.43	3.93	1.58
100	4.2	2.09	1.47	4.19	1.62

3.3 PROJECTED WASTEWATER FLOWS

3.3.1 METHODOLOGY

Projected wastewater flows were developed for average annual daily (AAD) and for peak wet weather conditions. These projected flows are used to develop scenarios for evaluating the future capacity of the system. The flows were developed based upon buildout within the 25-year study area. Existing and planned land use data was provided by FSD. The methodology for developing the projected wastewater flows consisted of the following:

- AAD flows within the study area were developed based on land use categories and the per acre unit flow rates.
- AAD flows for CCMD were added to the study area flows.
- 24-hour wet weather flows were developed for the selected return period by applying the peaking factors for that return period to the projected AAD flow.
- 1-hour wet weather flows were developed for the selected return period by applying the peaking factors for that return period to the projected AAD flow.

3.3.2 EVALUATION OF UNIT FLOW RATES

Unit flow rates (gallons per day per acre) for each land use category were presented in the 2004 Master Plan Report prepared by others. Initial flow projections using the previous unit flow rates generated projected flows much higher than considered reasonable.

BHI “calibrated” the previous unit flow rates by comparing flow projections using unit rates to historical flows. A buffer of 200 feet was applied to the existing sewer lines, and then the unit flow rates were applied to the existing land use within the buffer. Applying the original unit rates to the buffered area generated an AAD flow of 2.84 MGD, not including any flow contribution from CCMD. However, the actual AAD measured and provided by the District for 2015 was only 1.45 MGD; therefore, BHI reduced the unit flow rates by approximately 50 percent. The revised unit flow rates were then applied to the existing land use, and this generated an AAD flow (without CCMD) of about 1.5 MGD. Through this application, the revised unit rates are considered calibrated. Table 3.10 shows the revised unit rates for each land use category.

Table 3.10 – Land Use Based Unit Flow Rates

Land Use Category	Basis for Build-out Units per Acre	Selected Design Build-Out gpd per acre	Selected BO PE per acre
Agriculture	Unsewered	0	0.0
Business Park	15 gpcd, 10 empl/unit, 5 units/ac	390	11.0
Community Commercial	15 gpcd, 15 empl/unit, 3 units/ac	390	11.0
Downtown Mixed Use	25 gpcd, 15 empl/unit, 3 units/ac	620	17.6
Large Lot Residential	1 to 3	260	7.3
Mineral Extraction	Unsewered	0	0.0
Mixed Residential	7-20 du/ac, 2.5 people/du	1180	33.7
Mobile Homes	10-12 du/ac, 2.5 people/du	1525	43.6
Multi-family Dwellings	10-12 du/ac, 2.5 people/du	1525	43.6
Neighborhood Commercial	15 gpcd, 15 empl/unit, 3 units/ac	390	11.0
Open Space	Unsewered	0	0.0
Park	Unsewered	0	0.0
Planned Industrial	1000-1500 gpd/ac	650	18.3
Public	15 gpcd, 5 empl/unit, 3 units/ac	110	3.0
Regional Commercial	15 gpcd, 15 empl/unit, 3 units/ac	390	11.0
Single Family Dwellings	2-7 du/ac, 3.5 people/du	540	15.4
Small Office/Warehouse	15 gpcd, 15 empl/unit, 3 units/ac	130	3.7
Village Center	15 gpcd, 15 empl/unit, 3 units/ac	390	11.0
Village Commercial	15 gpcd, 15 empl/unit, 3 units/ac	390	11.0
School	25 gpcd, 200-800 students/school, 5 acres	1290	36.6

The future land use for the study area was provided to the District by the City of Fountain. The information was prepared by the City of Fountain within the City of Fountain Comprehensive Plan Update dated April 2013. Future land use is shown on Figure 3.6 on the following page.

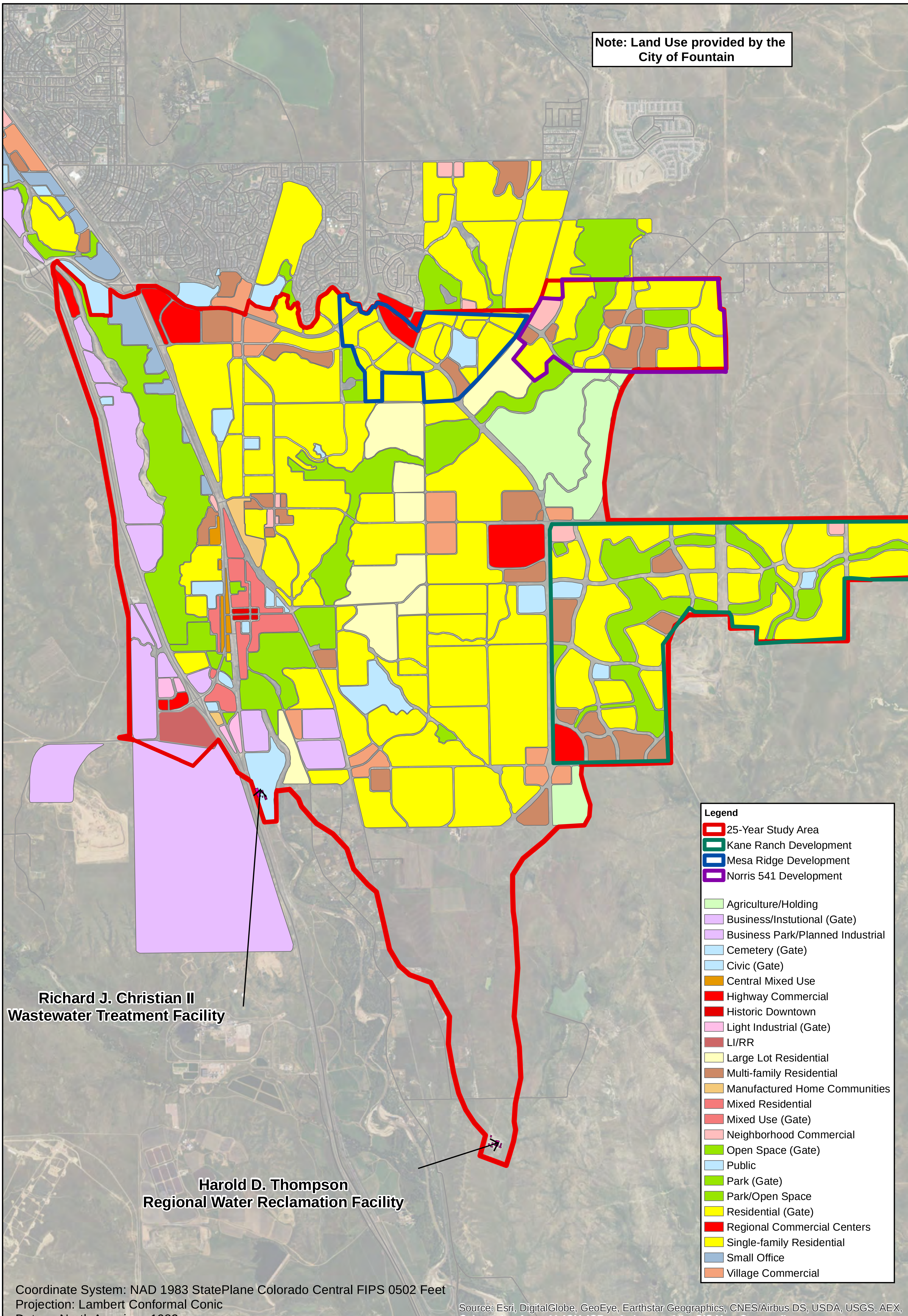

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Figure 3-6
Planned Land Use

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Author: jphillips

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3.3.3 EXISTING CONDITION DESIGN FLOWS

Using the "calibrated" land use based unit rates, AAD flows were calculated for the entire system and for each of the treatment facilities. Wet weather flows were then estimated for a 10-year rainfall event using the design peaking factors reported earlier. Table 3.11 shows that the design AAD flow of 1.6 MGD is slightly greater than current demands (2015) and represents a conservative flow.

Table 3.11 – Existing System Design Flows for 10-year Storm Event[1]

	AAD Flow (MGD)	24-hour Wet Weather Flow (MGD)	1-hour Wet Weather Flow (MGD)
RJC Facility	0.62	1.09	2.08
HDT Facility	1.00	1.35	1.49
Total	1.62	2.44	3.57

[1] Based on Land Use, Modified Per Acre Rates, CCMD 24-hour wet weather flow of 0.15 MGD and 10-year Storm Event.

3.3.4 DESIGN FLOWS

The calibrated unit flow rates were then used to estimate the projected wastewater flows for the 25-year study area at build out and for a short-term flow projection. The short-term projection assumes development of the Mesa Ridge and Norris 541 developments in addition to the existing uses. Based on conversations with the District, these two planned unit developments are anticipated to see significant development within the next 5 to 10 years.

Portions of the study area did not have planned future land use. BHI reviewed all three known PUDs and determined the blended flow generation per acre for each as shown in Table 3.12. For areas without planned future land use, it was assumed that densities would develop similarly to the Kane Ranch planned unit development (PUD) as shown on Figure 3.6. The other two PUDs contained much less open space; therefore, they had a higher unit flow rate.

Table 3.12 – Planned Unit Developments

	Flow Generation (GPD/acre)
PUD 1-Kane Ranch	438
PUD 2-Mesa Ridge[1]	523
PUD 3-Norris 541[1]	507
Design Per Acre Flow Rate[2]	450

[1] Larger flow generation per acre is due to less open space.
[2] Design unit flow rate is for areas without future land use designation.

As described earlier in this report, CCMD owns 1.0 MGD maximum day capacity in the LFMSDD interceptor sewer. Both the short-term and the long-term projections include 0.75 AAD flow contribution from CCMD. While it is unlikely that this high a flow will be contributed by CCMD in the short term, it provides a conservative basis to evaluate the collection system. Table 3.13 summarizes the projected AAD flows for CCMD.

Table 3.13 – CCMD Design Flows

	AAD Flow (MGD)	24-hour Wet Weather Flow (MGD)	1-hour Wet Weather Flow (MGD)
CCMD Flows	0.75	1.00	1.11

Table 3.14 summarizes the projected AAD flows for the study area at build-out.

Table 3.14 – Projected AAD Flows for Study Area at Build-Out

Description	Projected System AAD (MGD)
Areas within 25-Year Study Area with Land Use[1]	4.57
Areas within 25-Year Study Area without Land Use[2]	0.68
CCMD Flow Contribution	0.75
Total Area within 25-Year Study Area	6.00

[1] Areas with future land use as available from and provided by the City of Fountain.
[2] Areas with no specific future land use designation.

Table 3.15 provides the short term AAD flows and wet weather flows for each facility.

Table 3.15 – Short Term Design Flows

	AAD Flow (MGD)	24-hour Wet Weather Flow (MGD)	1-hour Wet Weather Flow (MGD)
RJC Facility	0.62	1.09	2.08
HDT Facility	2.19	2.74	3.93
Total	2.81	3.83	5.01

[1] Based on Land Use, Modified Per Acre Rates, CCMD 24-hour wet weather flow of 1.0 MGD and 10-year Storm Event.

Table 3.16 provides the study area AAD flows and wet weather flows for each facility.

Table 3.16 – Build-Out Design Flows

	AAD Flow (MGD)	24-hour Wet Weather Flow (MGD)	1-hour Wet Weather Flow (MGD)
RJC Facility	1.15	2.01	3.86
HDT Facility	4.85	6.49	7.22
Total	6.00	8.50	11.1

[1] Based on Land Use, Modified Per Acre Rates, CCMD 24-hour wet weather flow of 1.0 MGD and 10- year Storm Event.

Table 3.17 summarizes the projected flows for the entire system for each design condition.

Table 3.17 – Summary Design Flows

	AAD Flow (MGD)	24-hour Wet Weather Flow (MGD)	1-hour Wet Weather Flow (MGD)
Existing	1.50	2.22	3.40
Short Term	2.81	3.83	5.01
Build-Out	6.00	8.50	11.08

[1] Based on Land Use, Modified Per Acre Rates, CCMD 24-hour wet weather flow of 1.0 MGD and 10- year Storm Event.

4 EVALUATION OF SYSTEM

4.1 COLLECTION SYSTEM MODEL

A hydraulic model was created to evaluate the capacity of the existing collection system. The model was created in H_2OMAP Sewer by Innovyze. H_2OMAP Sewer is a GIS-based modeling software for use in planning, design and evaluations of sanitary, storm, and combined sewer collection systems.

4.1.1 FACILITIES CONSTRUCTION

A hydraulic model was developed over 10 years ago for the previous master plan. It had not been used recently, and many discrepancies were noted between it and the existing GIS files of the collection system. The new model of the collection system was created based on the most current GIS database provided by the District. All sewer lines 10 inches in diameter and greater, as well as some 8 inches in diameter, were imported as links into the model. Manholes and lift stations located along these lines were also imported into the model. The model pipes and manholes maintain the same IDs as GIS.

For manholes that did not contain rim elevations in the GIS database, a rim elevation was assumed from contour data from the USGS 10-meter Digital Elevation Model. For sewer lines missing invert elevations and a slope in the database, assumptions were made using known slopes of nearby upstream and downstream pipes. For problematic areas, information was field-surveyed, and available record drawings were examined by FSD to provide more accurate information. Sewer lines that required assumptions of invert elevations or slope are identified within the description field of the link in the model. It is recommended that missing information be acquired to more accurately simulate the demands that future developments would put on the system.

4.1.2 WASTEWATER LOAD ALLOCATION

Within the H_2OMAP software, a load allocator is used to assign wastewater flows to nodes or manholes within the model. To use the load allocator, subbasins were created in GIS using contour data from the USGS 10-meter Digital Elevation Model and land use data. The subbasins and land use data clipped to those subbasins was imported into the model from GIS. The land use based factors used to estimate wastewater generation rates can be seen in Table 3.10. These factors were input into the load allocator to correspond with the land use categories. Loads were then allocated to the appropriate manholes identified for each of the subbasins.

Subbasins were created to clip to existing land use (a buffer of 200 feet from the existing sewer lines) to model a scenario of the existing system. Other subbasins were created to clip to future land use to model scenarios for future build-out.

4.1.3 MODEL CALIBRATION

To calibrate the model and ensure that the load allocation was performed correctly, a scenario of the existing system for 2015 was developed to compare to the 2015 AAD flows for each plant. As provided previously in Table 3.11, the AAD flow for the RJC facility was 0.62 MGD and for the HDT facility was 0.88 MGD. The values at each facility in the model after running the existing system simulation were 0.65 for the RJC facility and 1.0 for the HDT facility. Therefore, based on the 2015 flow data, the model was considered calibrated.

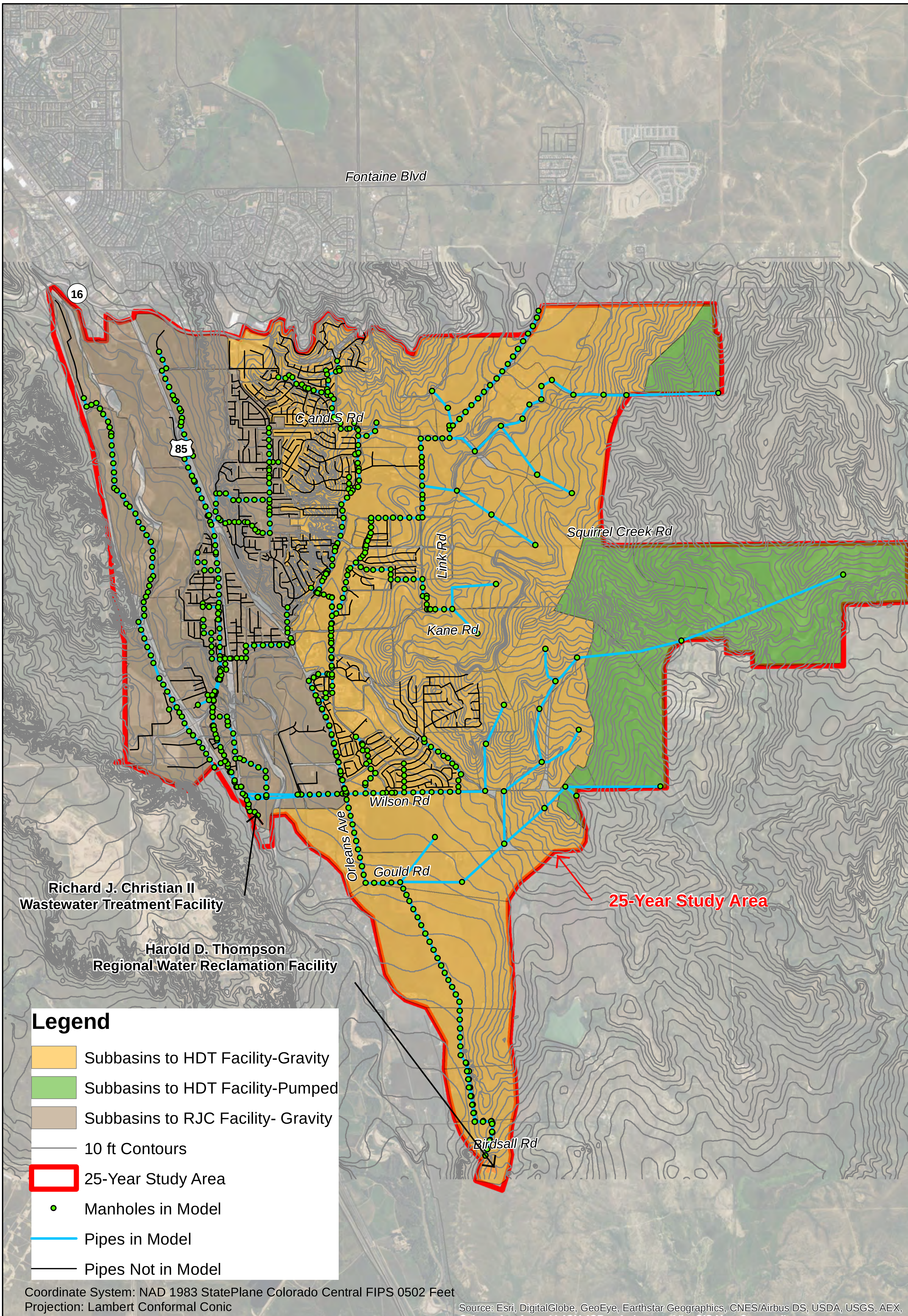
4.1.4 DEVELOPMENT OF BUILD-OUT MODEL

The collection system model was expanded with the addition of future facilities as required to serve build-out development of the 25-year study area. Subbasins were developed using 10-foot contour information, and the location and capacity of existing facilities were considered. After the conceptual layout was developed, build-out demands were allocated based on land use as described previously in this report. The collection system model was then used to size the future facilities and to evaluate other issues associated with development and increased flows. The discussion of system evaluations is presented later in this chapter. The build-out system model and the subbasin boundaries are shown on Figure 4.1 on the following page.

4.2 EVALUATION CRITERIA

Hydraulic analyses were conducted under 24-hour wet-weather flow conditions for existing, short term, and build-out conditions. Additional simulations were conducted with varying increased peaking factors to determine the sensitivity of the system to more extreme peaking conditions. The modeling results were used to evaluate the capacity of collection system components.

Gravity pipeline capacities were evaluated based on the ratio of the depth of flow to pipe diameter (d/D). Pipeline capacities were determined to be at capacity at a modeled depth of flow of 80 percent of the internal pipe diameter (d/D = 0.8). Sewer lines that had a d/D of greater than 0.8 were identified and improvements were developed.



N

0 2,000 4,000 8,000 Feet

1 in = 4,000 ft

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Figure 4-1
Build-Out Study Area Subbasins

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Author: jphillips

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Lift station capacities were evaluated, and future stations were sized based on having a minimum firm capacity (with the largest pump out of service) equal to the peak 1-hour flow into the station. Existing lift stations and force mains were reviewed in brief. The existing Conley, Race Street, and Plant lift stations will experience little or no growth and are adequately sized with no reported capacity concerns. The Little Ranches lift station has significantly greater capacity than the projected flows under current operation of the system using two treatment facilities. The capacity of the 10-inch force main for the lift station to the RJC Facility was reviewed relative to conveying flows from the RJC Facility to the LFMSDD interceptor sewer should the RJC Facility be considered for retirement in the future.

In addition to the above, the following design criteria established by CDPHE Water Quality Control Division were used as guidelines for sizing of new sewers in undeveloped areas:

- Sewer mains shall be a minimum of 8 inches in diameter except for small diameter sewer system components other than service lines.
- The design annual average day flow velocity in gravity flow pipelines shall be no less than 2 feet per second.
- Minimum manhole internal diameter shall be 4 feet.
- Minimum velocity in force main piping shall be no less than 2 feet per second.
- Maximum velocity in force main piping shall be no greater than 7 feet per second.
- Pumping capacity shall be sufficient to maintain the wet well water surface level below design maximum high water level at peak 1-hour flows. Pumping equipment must accommodate the design flow velocities in the force main.

4.3 EXISTING SYSTEM

4.3.1 EXISTING SYSTEM DEFICIENCIES

The existing collection system model revealed only one problem area under a 10-year wet weather event simulation. The problem area involves 8-inch lines along Illinois and Indiana Avenue. Following the reporting of the modeling results, District staff conducted field surveys of the pipelines and the model inverts for this area. The model inverts and slopes were adjusted accordingly, and the problem area remained with only slightly varying results. The problem area and the d/D ratios are shown in Figure 4.2. District staff noted this area is a known area of concern, and an improvement has previously been engineered for replacement.

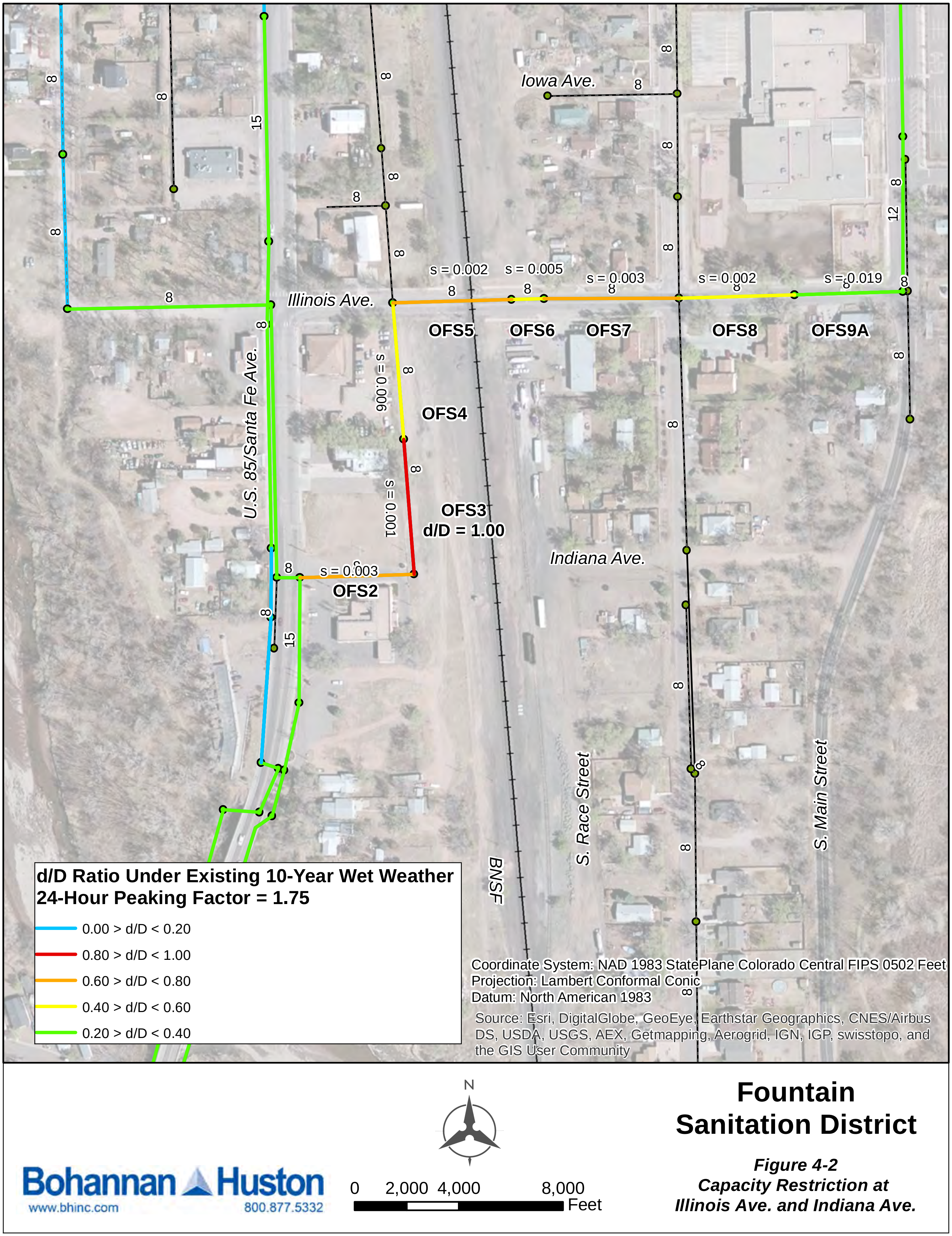

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Figure 4-2
Capacity Restriction at Illinois Ave. and Indiana Ave.

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4.3.2 WATCH LIST PIPES

A 1-hour peak flow for the 10-year design event was simulated for build-out conditions. Only two existing pipes were identified with d/D ratios greater than 0.8. In both cases, the d/D was less than 1.0 and the pipes were placed on the watch list (Table 4.1). Conditions in these pipelines should continue to be monitored, and the pipelines should be considered for replacement if conditions deteriorate or flows increase greater than expected in the future.

Table 4.1 – Watch List Pipes

ID	Description	Existing Pipe Information		
		Length (ft)	Diameter (in)	Average Depth (ft)
W1	North of Comanche Village Dr., FMS1 and FMS 2	404	8	13
W2	Santa Fe Ave. (US Highway 85), SFS41	156	8	9

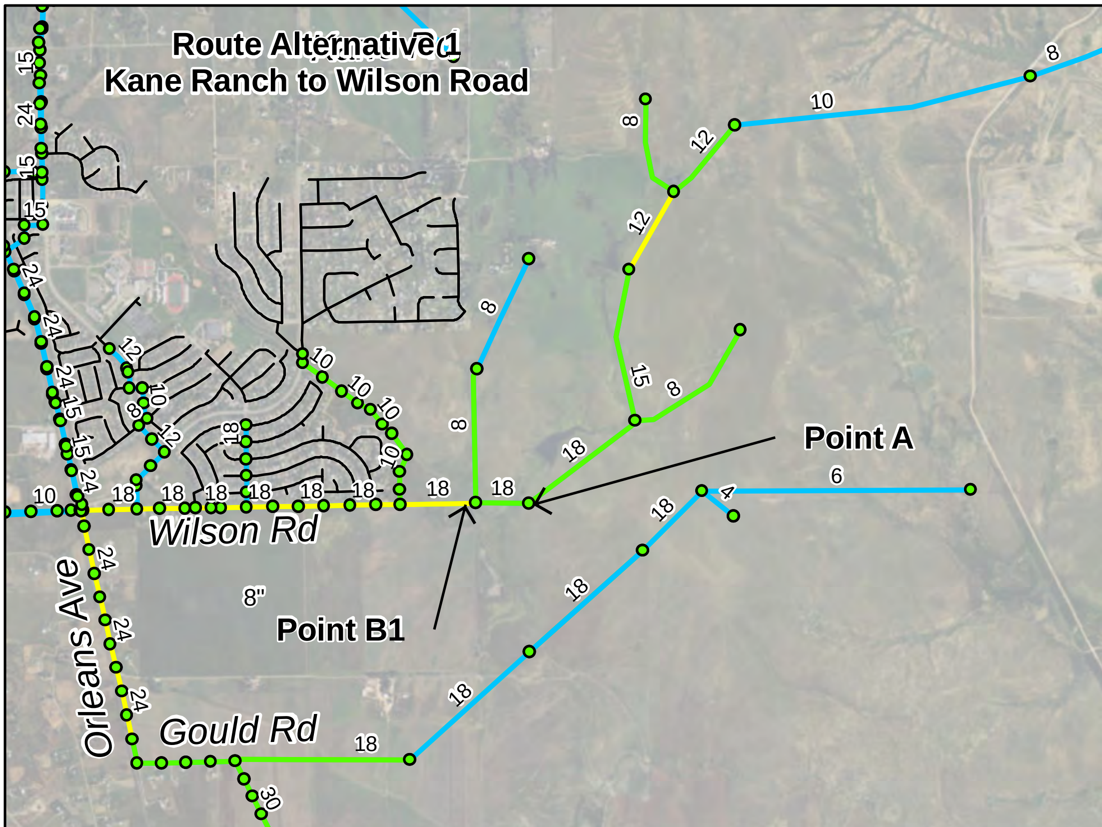
4.4 GROWTH RELATED EVALUATIONS

The collection system model was used to evaluate issues associated with development and subsequent increased flows. Full build-out of the study area and the associated required facilities were evaluated first. Short term evaluations were then conducted to verify which facilities would be required to serve short-term expected development. The evaluations of issues associated with increased flows are summarized in the following sections.

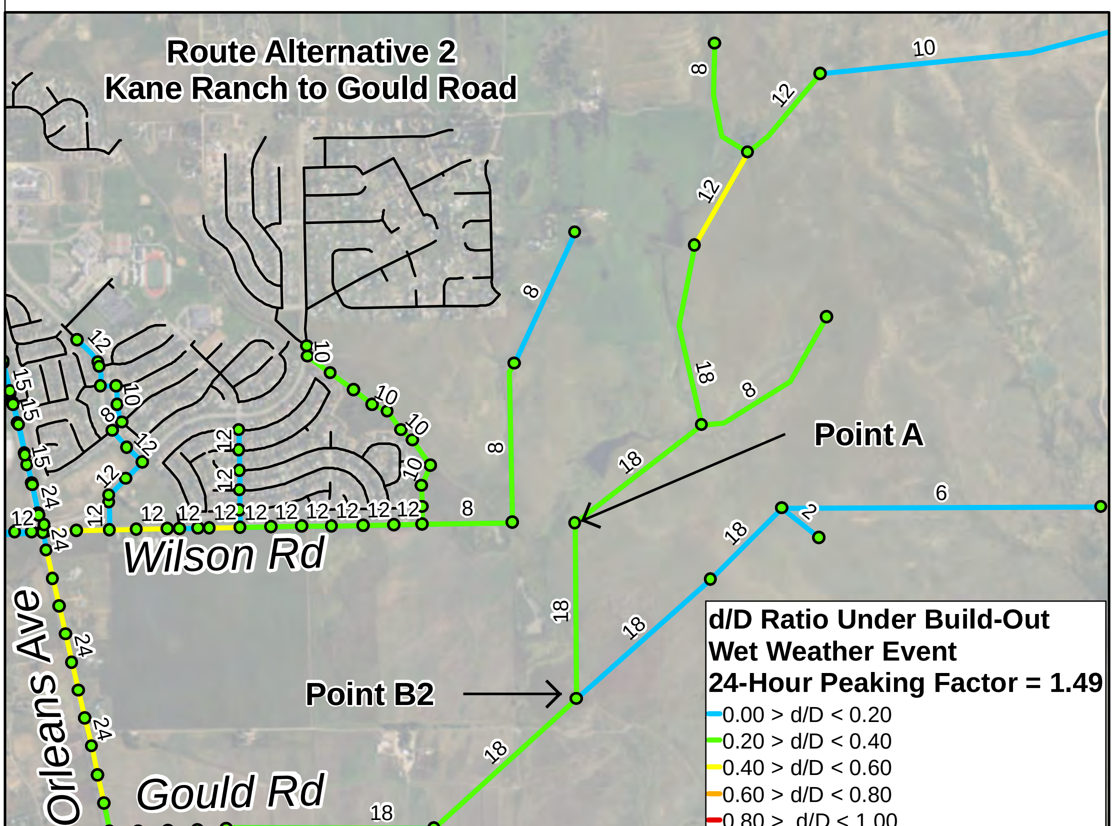
4.4.1 KANE RANCH FLOWS

Much of the Kane Ranch development will need to be pumped for service. Long-term hydraulic analyses were conducted to evaluate the best delivery point for those pumped flows. Two routing alternatives were evaluated with the modeling results illustrated on Figure 4.3. The key points of the evaluation are summarized below:

- If the pumped flows from the Kane Ranch development are delivered to the existing 12-inch pipeline along Wilson Road, it overloads the existing sewers along Wilson Road.
- If flows are routed south to the existing LFMSDD interceptor sewer on Gould Road, no overloading occurs in the existing collection system.



Note: Majority of flows to Point A are generated from the Kane Ranch Development



d/D Ratio Under Build-Out Wet Weather Event
24-Hour Peaking Factor = 1.49

- 0.00 > d/D < 0.20
- 0.20 > d/D < 0.40
- 0.40 > d/D < 0.60
- 0.60 > d/D < 0.80
- 0.80 > d/D < 1.00

N

0 2,000 4,000 8,000 Feet

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Figure 4-3
Long Term Routing of Kane Ranch Flows

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4.4.2 LFMSDD INTERCEPTOR SEWER AND RJC FACILITY

The capacity of the LFMSDD interceptor sewer south of Wilson Road was evaluated to assess its potential to convey flows pumped to it from the RJC Facility should the RJC Facility be retired in the future. A model scenario was developed to simulate the retirement of the RJC Facility and the subsequent delivery of flows to the HDT Facility. All flows would be pumped from the RJC Facility to the LFMSDD interceptor sewer at Wilson Road and Orleans Avenue.

Detailed review of the LFMSDD interceptor south of Wilson Road reveals two areas that are flatter than the others as summarized below:

- Just south of Wilson Road there are eight reaches of 24-inch pipe with a slope of about 0.3 percent, a full pipe capacity of 8.0 MGD (d/D = 1.0), and a design capacity of 7.8 MGD (d/D of 0.8).
- Further south, 22 reaches of 30-inch diameter pipe, directly north of the HDT Facility, have a slope of about 0.14 percent, a full pipe capacity of about 9.9 MGD (d/D = 1.0), and a design capacity of 9.7 MGD (d/D = 0.8). During initial construction of the LFMSDD pipeline, the District also had constructed a parallel pipeline that parallels the lowest 15 reaches of the 30-inch pipe. This parallel pipeline is currently not in service but can be connected and utilized when pipeline flows exceed 9.7 MGD.
- All other reaches of the pipeline have a capacity of 13 MGD or greater at design depth.

The projected flows in the lower reaches of the LFMSDD interceptor sewer are shown in Table 4.2.

Table 4.2 – Build-Out Peak Flows to Lower LFMSDD Interceptor

Plant	24-Hour (Peak Day)		1-hour (Peak Hour)	
	24-Hour Wet Weather Flow (MGD)	24-Hour Wet Weather Flow: ADWF (Ratio)	1-hour Peak Flow (MGD)	1-hour Peak Flow: ADWF (Ratio)
HDT Facility Only[1]	6.5	1.34	7.2	1.49
HDT Facility + RJC Facility[2]	8.5	1.42	11.1	1.85

[1] Build-out flows generated within the HDT Facility drainage basis, plus peak flows from CCMD.
[2] Build-out flows generated within the HDT Facility drainage basis, plus peak flows from CCMD, plus peak flows from the RJC Facility should it be retired.

Comparisons of pipeline capacity to potential flows are summarized below:

- All reaches of the LFMSDD interceptor, south of Wilson Road, can adequately convey the projected long-term 1-hour wet weather flows for the HDT basin of 7.2 MGD.
- If in the future, flows from the RJC Facility are conveyed to the LFMSDD interceptor for treatment at the HDT Facility, the capacities of the two sections of flatter slope (south of Wilson Road) would be exceeded under long-term wet weather flows of 11.1 MGD.
  - For the eight reaches, directly south of Wilson Road, the design capacity (d/D = 0.8) of 7.8 would be exceeded, and these reaches would likely need to be paralleled.
  - For the lowest 22 reaches, the design capacity (d/D = 0.8) of 9.7 MGD would also be exceeded. However, if sufficient equalization storage is provided, peak wet-weather flows generated in the RJC Basin could be attenuated before being delivered to the LFMSDD interceptor, possibly eliminating the need for paralleling this lowest section of reaches. These lower reaches should be evaluated further if conveyance of flows generated in the RJC Basin to the HDT Facility is considered. The evaluation should review the size and cost effectiveness of providing equalization storage versus new pipelines. It is important to note that 15 of these lower 22 reaches are already paralleled with an unused pipeline and would not require new construction.

Figure 4.4 shows the locations of the critical pipe reaches in the lower reaches of the LFMSDD interceptor sewer south of Wilson Rd.

4.4.3 SHORT-TERM IMPROVEMENTS

A short-term improvement scenario was developed to evaluate the addition of the Norris and Mesa Ridge developments. No problems were expected because there were no problems under the build-out condition. Improvements needed to serve Norris and Mesa Ridge developments are identified as short-term improvements later in this report. This model scenario was included in the final model delivered to the District.

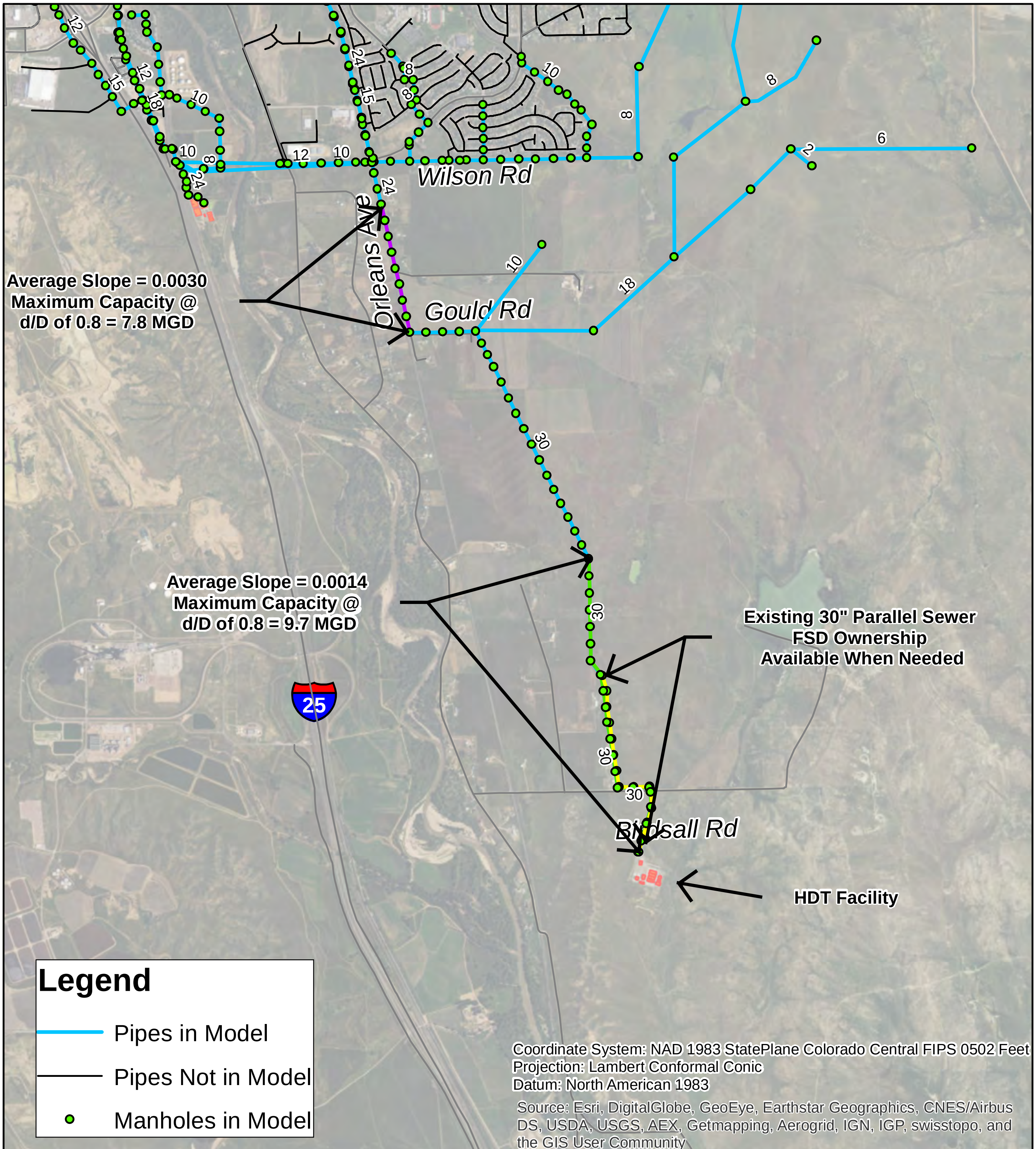

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Figure 4-4
Capacity Considerations of LFMSDD Interceptor

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5 FINDINGS AND RECOMMENDATIONS

5.1 FINDINGS

The primary findings from this master plan study are summarized below:

- Historical flows show relatively low peaking factors at both treatment facilities under wet weather conditions. This finding is supported by the relatively young age of the collection system with over 80 percent of the system less than 50 years old and almost 60 percent less than 20 years old. In addition, the District has a very proactive maintenance program. They conduct their own CCTV inspections and have an on-going annual program to rehabilitate existing pipelines determined to be in poor condition. In summary, the proactive measures practiced by the District are effective.
- Hydraulic modeling shows that, except for one location, the existing trunk system has sufficient capacity to convey 24-hour design flows for the 10-year storm event. In addition to this location, two reaches of sewer were overloaded under the peak 1-hour design flows for the 10-year storm event. Those two reaches have been placed on a watch list for continued monitoring and potential replacement.
- The lower 22 reaches of the LFMSDD interceptor sewer is flatter than upstream of that location. A pipeline was constructed parallel to the lowest 15 of these reaches at the sole expense of the District. The parallel pipeline is currently not in service but can be connected and utilized when pipeline flows exceed 9.7 MGD in the existing line.
- The 10-inch force main between the Little Ranches lift station and the RJC Facility has a capacity of about 2.5 MGD at a maximum velocity of 7 feet per second. The peak 1-hour flow to the RJC Facility at build-out of the study area is projected to be 3.9 MGD and would result in a velocity of about 11 fps. This force main would need replacement should the RJC Facility be retired in the future.

5.2 BASIS OF COSTS FOR RECOMMENDED IMPROVEMENTS

The overall project costs have been conservatively estimated for this budget level of detail. As such, actual construction cost may be lower than those identified. Any potential increases in the costs due to inflation are not reflected in the prices.

Construction costs cover the material, equipment, labor and services necessary to build the proposed project. Prices used in this study were obtained from a review of previous reports and pertinent sources of construction cost information. Construction costs are not intended to represent the lowest prices which may be achieved but rather are intended to represent a median of competitive prices submitted by responsible bidders.

The total project cost necessary to complete a project consists of expenditures for construction costs; contingencies; all necessary engineering services; such overhead items as legal, administrative, and financing services; and land acquisition.

Factors such as unexpected construction conditions, the need for unforeseen mechanical and electrical equipment, and variations in final quantities are a few examples of items that can add to planning level estimates of project cost. To cover such contingencies, an allowance of 20 percent of the construction cost has been included.

Engineering services may include preliminary investigations and reports, site and route surveys, foundation explorations, preparation of design drawings and specifications, regulatory agency approvals, engineering services during construction, construction observation, construction surveying, sampling and testing, start-up services, and preparation of operation and maintenance manuals. Overhead charges cover such items as legal fees, financing fees, and administrative costs. The costs presented in this report include a 20 percent allowance for engineering services, legal, and administrative costs.

The cost of land acquisition is not included in the project costs presented in this report. In most cases, no property acquisition is required for any of the potential improvements. Any new facilities would be constructed in property deeded to the District during development. The construction of pipelines will generally not require purchase of private property or acquisition of easements. Pipeline routes, insofar as possible, follow public streets and roads.

It is assumed that the costs for constructing new facilities into currently undeveloped areas will be borne by the developer, and the facilities will then be deeded to the District. This includes all recommended capital improvements identified in this report as “Development Driven.”

In considering the estimates presented in this report, it is important to realize they are reported in year 2017 dollars, and future changes in the cost of materials, equipment, and labor will cause comparable changes in project costs.

5.2.1 PIPELINES

Probable construction costs for sanitary sewer line replacement are based upon unit costs of $10.00 per diameter-inch per linear foot of PVC pipe.

The construction cost for new sanitary sewer pipes in areas to be developed is based on unit costs of $8.00 per diameter-inch per linear foot of PVC pipe. The unit cost includes manholes assumed to be installed every 400 feet.

The probable construction cost for new force mains in areas to be developed is based on unit costs of $8.00 per diameter-inch per linear foot of PVC pipe.

5.2.2 LIFT STATIONS

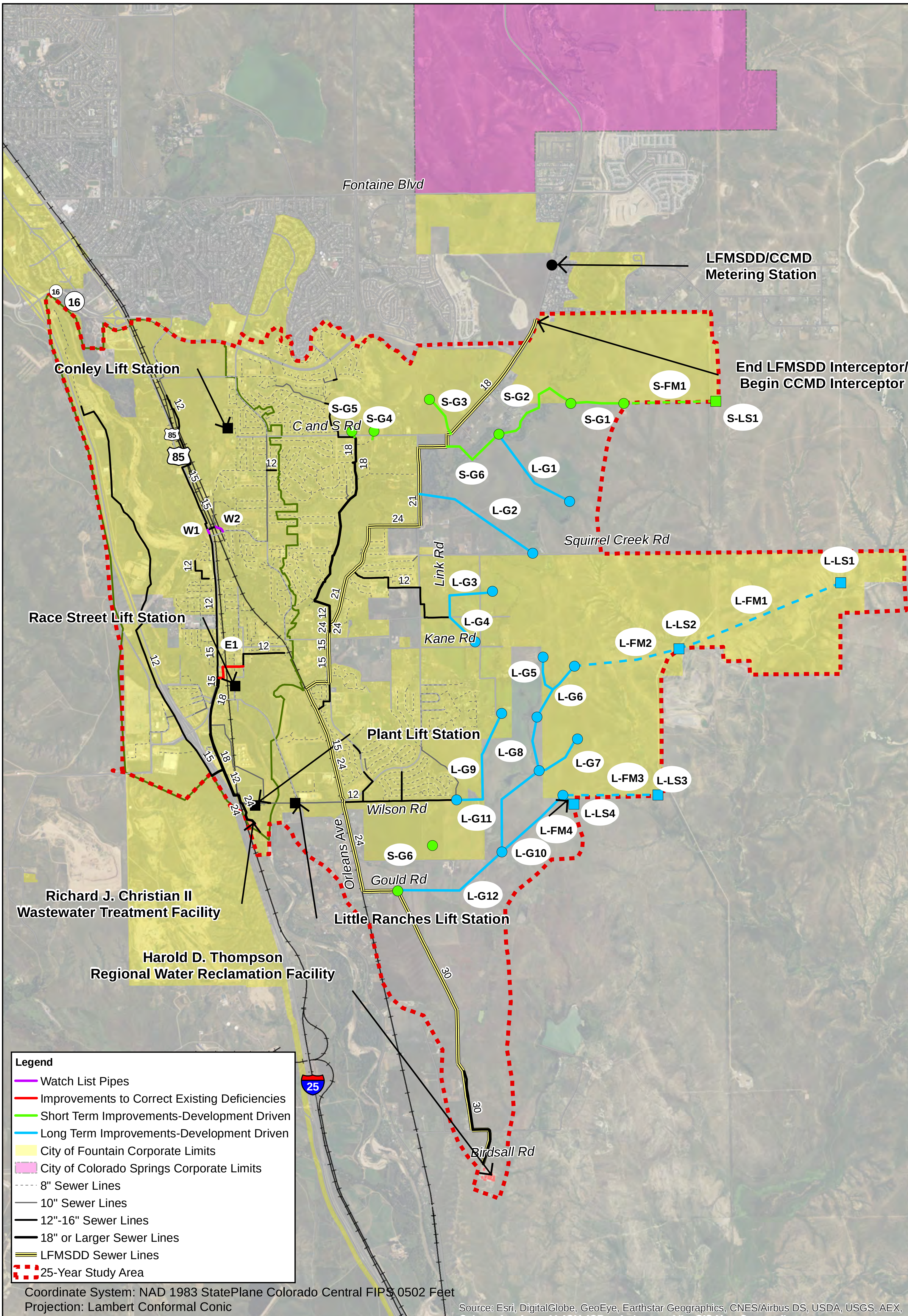
The project cost for lift stations can vary considerably. Costs are used herein that would be representative of a duplex pump station with a standby generator. The estimated total unit project costs include site work, lift station construction, site piping, controls, and miscellaneous appurtenances. The construction cost of various sized lift stations is provided in Table 5.1.

Table 5.1 – Construction Cost for Lift Stations

Size	Construction Cost	Unit Cost
1.0 MGD / 1,440 GPM	$800,000	560 $/GPM
0.75 MGD / 1,080 GPM	$700,000	650 $/GPM
0.5 MGD / 720 GPM	$580,000	800 $/GPM
0.25 MGD / 360 GPM	$430,000	1,200 $/GPM
Less Than 0.25 MGD		Varies

5.3 RECOMMENDED IMPROVEMENTS

Recommended improvements are shown on Figure 5.1 and summarized and tabulated in the following sections.



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N

0 2,000 4,000 8,000 Feet

1 in = 4,000 ft

Fountain Sanitation District

Figure 5-1
Recommended Improvements

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Author: jphillips

March 2017

5.3.1 EXISTING SYSTEM DEFICIENCIES

Recommended replacement pipelines to resolve existing system deficiencies are shown in Table 5.2. The only identified deficiency was discussed in detail earlier in this report. We have included costs for replacement of eight sections of pipe with relatively flat slopes. The exact slope on these sections of pipes should be determined by more precise survey, and the recommended improvements should be modified accordingly.

Table 5.2 – Improvements for Existing Deficiencies

ID	Description	Recommended Improvement			Probable Project Cost[1]
		Length (ft)	Diameter (in)	Average Depth (ft)	
E1	Replace eight reaches of pipe near BNSF Railroad and Illinois Avenue	1,600	12	12	**$ 722,000.00**

[1] Probable project costs include a 20% contingency and a 20% allowance for engineering, legal, and administrative costs.

5.3.2 SHORT-TERM DEVELOPMENT DRIVEN IMPROVEMENTS

Recommended new trunk pipelines, force mains, and lift stations are required to serve projected short-term development of the Mesa Ridge and Norris 541 developments (as shown previously on Figure 3.6) and are summarized in Tables 5.3 and 5.4.

Table 5.3 – Short Term Development Driven Improvements

ID	Description	Improvement Pipe Needed			Probable Project Cost[1]
		Length (ft)	Diameter (in)	Average Depth (ft)	
S-G1	Gravity Sewer	2,340	8	10	**$ 148,980.00**
S-G2	Gravity Sewer	4,590	12	12	**$ 441,030.00**
S-G3	Gravity Sewer	1,830	8	10	**$ 116,780.00**
S-G4	Gravity Sewer	380	8	8	**$ 24,200.00**
S-G5	Gravity Sewer	290	8	13	**$ 18,290.00**
S-G6	Gravity Sewer	2,940	18	9	**$ 422,640.00**
S-G7	Gravity Sewer	2,500	10	15	**$ 200,000.00**
S-FM1	Force Main	1,980	4	10	**$ 63,410.00**

[1]Probable project costs includes a 20% contingency and a 20% allowance for engineering, legal, and administrative costs.

Table 5.4 – Short Term Development Driven Lift Stations

ID	Description	Lift Station Needs			Probable Project Cost[1]
		Firm Capacity (gpm)	TDH (ft)	Horsepower	
S-LS1	Lift Station	111	29.5	1.23	**$ 430,000.00**

[1]Probable project costs includes a 20% contingency and a 20% allowance for engineering, legal, and administrative costs.

5.3.3 Long-Term Development Driven Improvements

New trunk pipelines, force mains, and lift stations required to serve projected long-term development through build-out of the 25-year Study Area are summarized in Tables 5.5 and 5.6.

Table 5.5 – Long Term Development Driven Pipelines

ID	Description	Improvement Pipe Needed			Probable Project Cost[1]
		Length (ft)	Diameter (in)	Average Depth (ft)	
L-G1	Gravity Sewer	4,390	8	9	**$ 281,070.00**
L-G2	Gravity Sewer	5,700	8	10	**$ 364,450.00**
L-G3	Gravity Sewer	2,890	8	10.	**$ 185,010.00**
L-G4	Gravity Sewer	1,600	8	10	**$ 102,100.00**
L-G5	Gravity Sewer	1,620	8	9	**$ 103,610.00**
L-G6	Gravity Sewer	2,790	12	9	**$ 267,610.00**
L-G7	Gravity Sewer	2,280	8	8	**$ 145,590.00**
L-G8	Gravity Sewer	2,400	18	10	**$ 344,930.00**
L-G9	Gravity Sewer	5,140	8	11	**$ 329,070.00**
L-G10	Gravity Sewer	3,650	18	11	**$ 525,880.00**
L-G11	Gravity Sewer	4,380	18	10	**$ 630,410.00**
L-G12	Gravity Sewer	5,210	18	10	**$ 750,250.00**
L-FM1	Force Main	7,650	8	10	**$ 489,710.00**
L-FM2	Force Main	4,660	10	10	**$ 373,140.00**
L-FM3	Force Main	4,160	6	10	**$ 199,700.00**
L-FM4	Force Main	620	2	10	**$ 9,900.00**

[1]Probable project costs includes a 20% contingency and a 20% allowance for engineering, legal, and administrative costs.

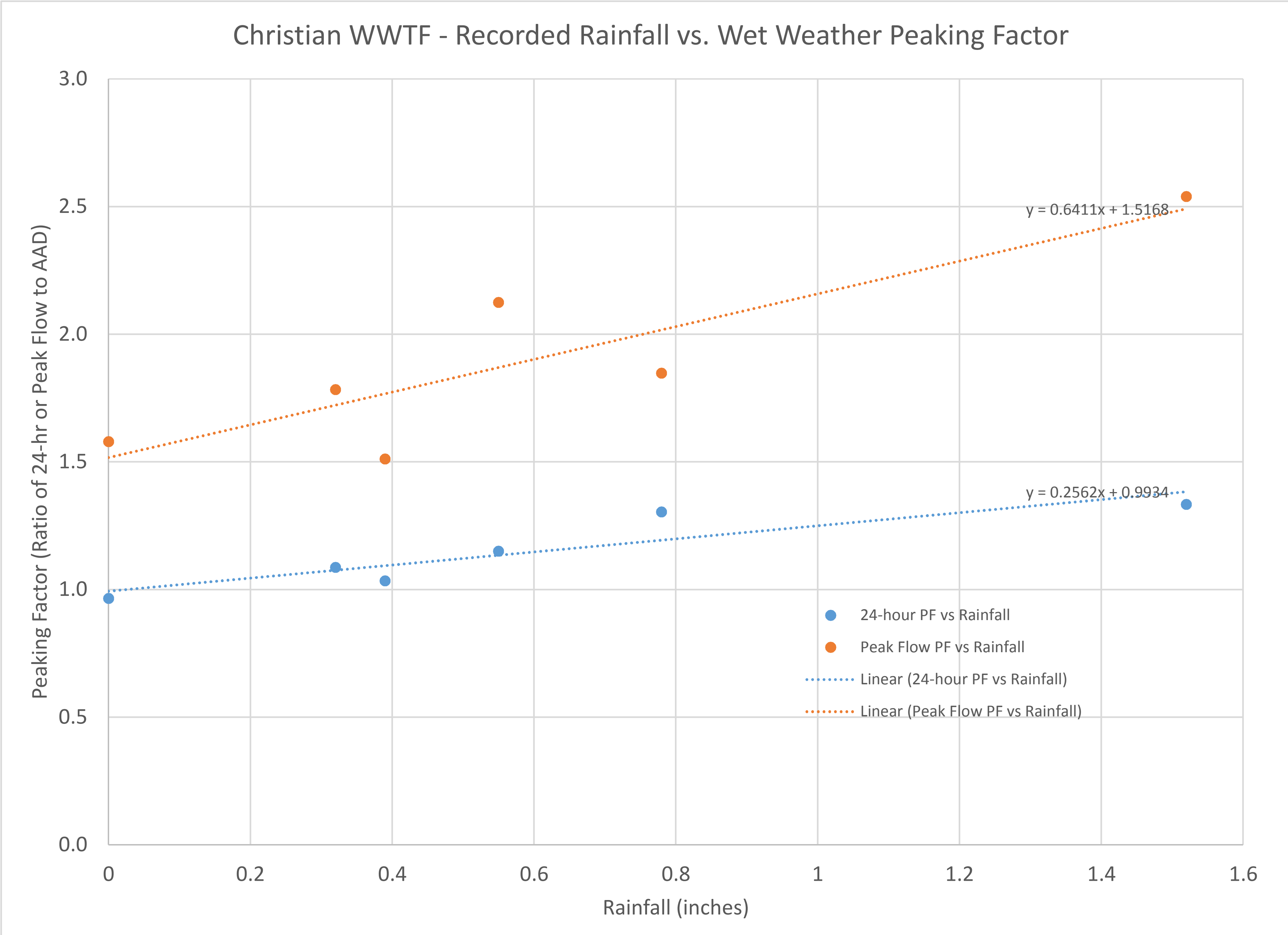
Table 5.6 – Long Term Development Driven Lift Stations

ID	Description	Lift Station Needs			Probable Project Cost[1]
		Firm Capacity (gpm)	TDH (ft)	Horsepower	
L-LS1	Lift Station	980	160	57	**$ 700,000.00**
L-LS2	Lift Station	310	45	5	**$ 430,000.00**
L-LS3	Lift Station	340	70	9	**$ 430,000.00**
L-LS4	Lift Station	30	25	0.25	**$ 60,000.00**

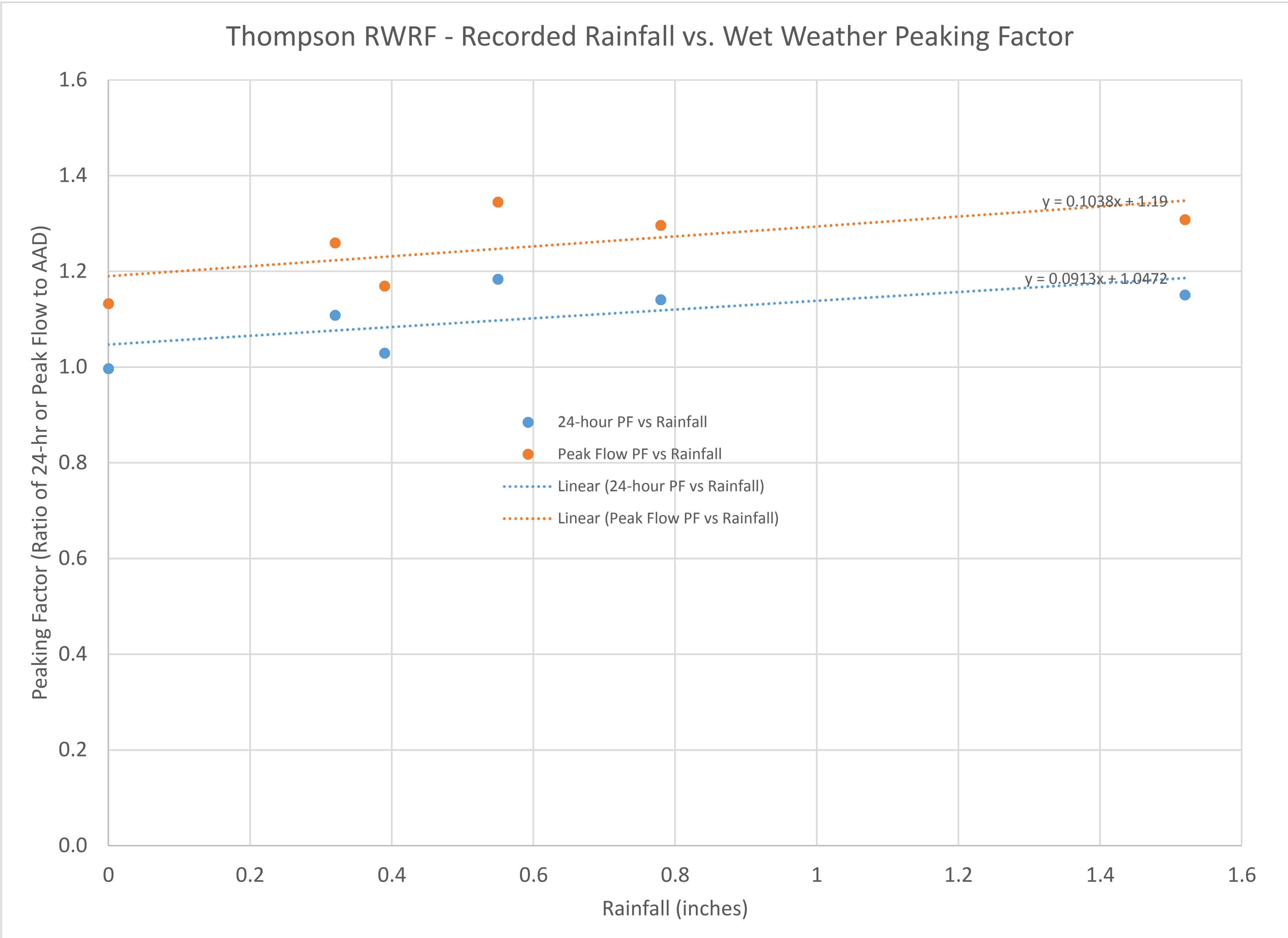
[1]Probable project costs includes a 20% contingency and a 20% allowance for engineering, legal, and administrative costs.

APPENDIX A –

PEAKING FACTOR GRAPHS



Christian WWTF - Recorded Rainfall vs. Wet Weather Peaking Factor
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Peaking Factor (Ratio of 24-hr or Peak Flow to AAD)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
Rainfall (inches)
y = 0.6411x + 1.5168
y = 0.2562x + 0.9934
24-hour PF vs Rainfall
Peak Flow PF vs Rainfall
Linear (24-hour PF vs Rainfall)
Linear (Peak Flow PF vs Rainfall)



Thompson RWRF - Recorded Rainfall vs. Wet Weather Peaking Factor
Peaking Factor (Ratio of 24-hr or Peak Flow to AAD)
Rainfall (inches)
y = 0.1038x + 1.19
y = 0.0913x + 1.0472
24-hour PF vs Rainfall
Peak Flow PF vs Rainfall
Linear (24-hour PF vs Rainfall)
Linear (Peak Flow PF vs Rainfall)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6

APPENDIX B –

SOUTHWEST AREA INDUSTRIAL DEVELOPMENT

REVIEW

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MEMORANDUM

DATE: February 26, 2016

TO: Fountain Sanitation District

FROM: Jerry Edwards, PE; Johanna Phillips, EI

SUBJECT: **Southwest Industrial Development Review**
Appendix B to 2016 Collection System Master Plan

Fountain Sanitation District requested that BHI perform a review of the facilities required to serve a potential industrial development located west of I-25 and within the FSD Planning and Service Area, as shown on Figure B-1 at the end of this memorandum. This area was not included in the Study Area for the 2016 Collection System Master Plan Update (2016 MP Update).

The potential industrial site would be located on properties currently owned by Edward C. Levy Company dba Schmidt Construction, Inc. This area was referred to in the 2004 Master Plan as the Christian Ranch Development. Adjacent land previously owned by the Martin Marietta Corporation has been acquired by the City of Fountain for purposes of water resource management. That area, located west of Interstate Highway 25, south of Charter Oak Ranch Road, north of City of Colorado Springs property and east of the west lines of Sections 7 and 18, will not be a source of wastewater generation. There has been no determination made that the City of Fountain will produce water treatment residuals that could be discharged to the FSD wastewater management system.

The collection system previously developed to serve the Christian Ranch Development is also shown on Figure B-1. As shown on Figure B-1, wastewater flows generated from a development in this general location would naturally flow to the southeast and would require a lift station to pump wastewater flows to a treatment facility. The flows could be delivered to either the Richard J Christian II Wastewater Treatment Facility (RJC Facility) or the Harold D. Thompson Regional Water Reclamation Facility (HDT Facility).

BHI identified the potential facilities required to convey up to 350,000 gpd of projected short-term wastewater to a treatment facility. Two alternatives were developed as shown in Figure B-1, one to convey flows to the RJC Facility, and a second to convey flows to the HDT Facility. At buildout of this part of the FSD Planning and Service area, approximately 3,100 acres (Schmidt Construction, Inc. property) may require wastewater management services.

BHI developed budget level costs for the design and construction of the new lift station, force main, and gravity sewer for each alternative. Alternative B, delivery of flows to the HDT Facility, would require a crossing of I-25, three railroads and Fountain Creek. However, Alternative A would connect to an existing 15-inch gravity sewer on the west side of I-25 and would not require highway, railroad or creek crossings. The budget level costs are provided within Table B.1 and Table B.2 below.

Table B.1 – Alternative A: To RJC Facility

Description	Firm Capacity Or Length (gpm)	Unit	Unit Cost	Probable Cost[1]
Lift Station	250[2]	gpm	$1,500 / gpm	$500,000
4" Force Main	10,300	feet	$32 / foot	$330,000
8" Gravity Sewer	5,400	feet	$64 / foot	$350,000
Subtotal – Probable Construction Cost				**$1,180,000**
Contingency and ELA Allowance				**$470,000**
Total Probable Project Cost				**$1,650,000**

[1] Based on unit costs presented in Table 5.1 of 2016 Collection System Master Plan Update. Probable project costs include a 20% contingency and a 20% allowance for engineering, legal, and administrative costs.
[2] Pumping rate with full flow equalization; 500 gpm required for planning without full flow equalization.

Table B.2 – Alternative B: To HDT Facility

Description	Firm Capacity Or Length (gpm)	Unit	Unit Cost	Probable Cost[1]
Lift Station	250[2]	gpm	$1,500 / gpm	$500,000
8" Gravity Sewer	9,900	feet	$64 / foot	$630,000
4" Force Main	9,000	feet	$32 / foot	$290,000
Crossing of I-25	200	feet	$600 / foot	$120,000
BNSF & CSU Railroad Crossings	200	feet	$600 / foot	$120,000
Fountain Creek Crossing; Trenchless Install	1,450	feet	$600 / foot	$870,000
Union Pacific Railroad Crossing	100	feet	$600 / foot	$60,000
Subtotal – Probable Construction Cost				**$2,590,000**
Contingency and ELA Allowance				**$1,036,000**
Total Probable Project Cost				**$3,626,000**

[1] Based on unit costs presented in Table 5.1 of 2016 Collection System Master Plan Update. Probable project costs include a 20% contingency and a 20% allowance for engineering, legal, and administrative costs.
[2] Pumping rate with full flow equalization; 500 gpm required for planning without full flow equalization.

Based on this review, it is significantly less costly to deliver the flows to the RJC Facility. This matter must be considered in planning for the future of the RJC Facility.

From the 2016 Master Plan Update, the long-term development driven wastewater flows for the RJC Facility, including the additional wastewater flows generated from this potential development, are shown in Table B-3.

Table B.3 – RJC Facility Projected Flows with SW Industrial Development

	Short-Term AAD Flow[1] (MGD)	Long-Term AAD Flow[1] (MGD)
RJC Facility	0.62	1.15
SW Potential Development	0.35	2.02[2]
Total	0.97	3.17

[1] Flows from Table 3-16 of 2016 Collection System Master Plan Update.
[2] Table 3-10 Planned Industrial unit flow of 650 gpd/acre from 3,100 acres.

The design and permitted maximum monthly flow of the RJC facility is 1.906 MGD. Based on an assumed maximum month to average annual day ratio of 1.3, the short term maximum month loading projected in Table B-3 above would be 1.26 MGD (0.97 MGD x 1.3 = 1.26 MGD). However, at buildout of the FSD Planning and Service area, the maximum month loading projected in Table B-3 would be 4.12 MGD (3.17 MGD x 1.3 = 4.12 MGD). Additional treatment capacity would be required to accept this buildout load. The influent sewers, channels, and process piping at the RJC facility were designed with a hydraulic capacity of 5.0 MGD and would also require upgrading to accept instantaneous peak flows of about 6 to 7 MGD under this loading scenario.

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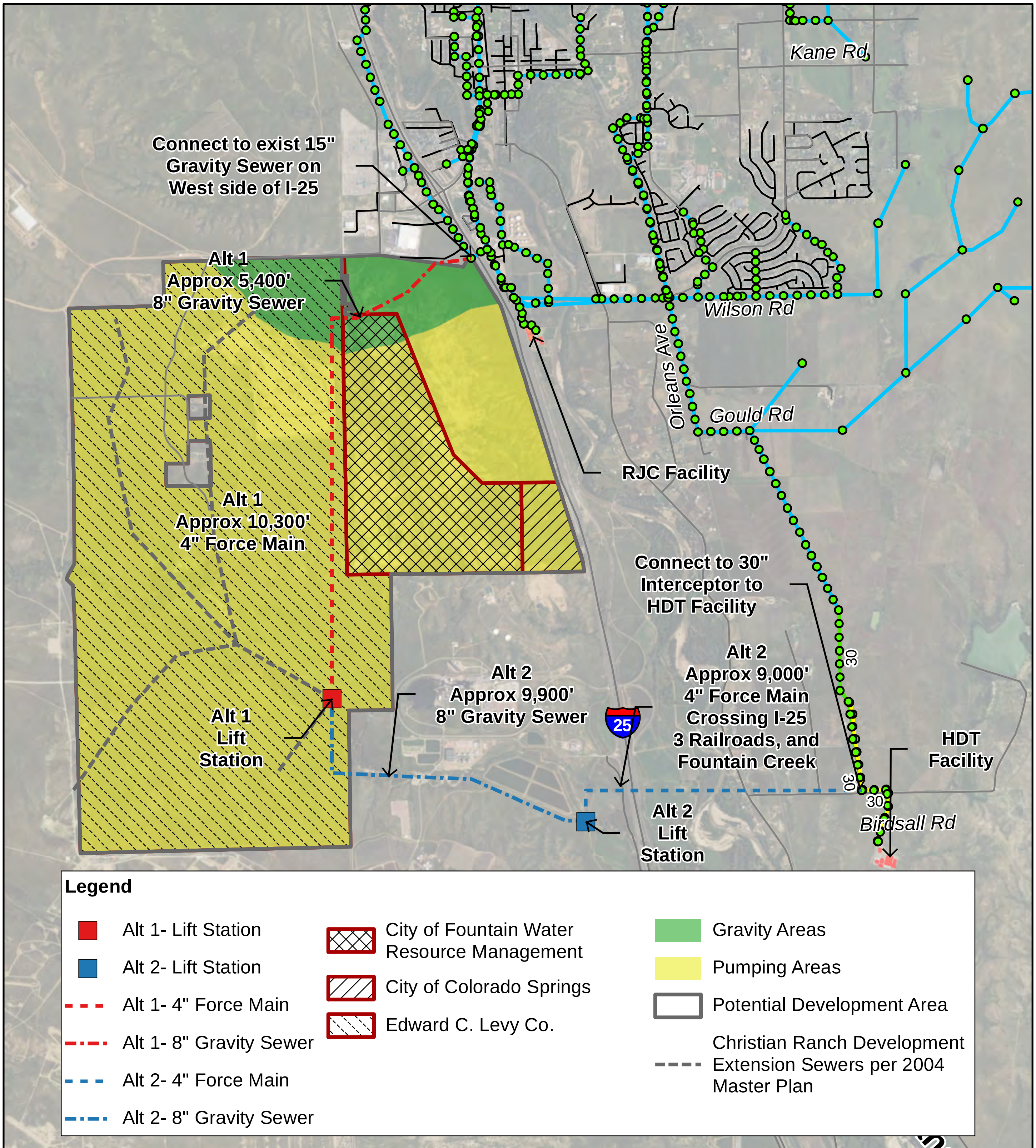

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0 2,000 4,000 8,000 Feet

Fountain Sanitation District

Figure B-1
Southwest Area Industrial Development

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Author: gmaynard

March 2017