Water and Wastewater Master Plan

City of Colleyville

Prepared by:

3010 Gaylord Parkway, Suite 190
Frisco, TX 75034

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Garver Project No.: 13088150

Engineer's Certification:
I hereby certify that this Water and Wastewater Master Plan was prepared by Garver under my direct supervision for the City of Colleyville.

STATE OF TEXAS
JEFFREY LYNN SOBER
103772
LICENSED
PROFESSIONAL ENGINEER

12/29/14

Jeffrey L. Sober, PE
State of Texas PE License 103772

STATE OF TEXAS
LANCE P. KLEMENT
113630
LICENSED
PROFESSIONAL ENGINEER

12/29/14

Lance P. Klement, PE
State of Texas PE License 113630

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Glossary

Terms and Abbreviations used in this report are as follows:

AAD	Annual average day
AWWA	American Water Works Association
CCTV	Closed Circuit Television Video
CIP	Capital Improvements Plan
EST	Elevated Storage Tank
FCV	Flow Control Valve
FM	Flow Monitoring
ft	Feet
GIS	Graphical Information System
gpcd	Gallons per capita per day
gpd	Gallons per day
gpm	Gallons per minute
GST	Ground Storage Tank
HGL	Hydraulic Grade Line
hp	Horsepower
I/I	Infiltration/Inflow
IFC	International Fire Code
LF	Linear Feet
MDD	Maximum day demand
MG	Million gallons
MGD	Million gallons per day
mg/l	Milligrams per liter
OPCC	Opinion of Probable Construction Cost
PH	Peak hour
PRV	Pressure Reducing Valves
MG	Million gallons
N.A.	Not Available
No.	Number
PS	Pump Station
psi	Pounds per square inch
RDII	Rainfall-derived infiltration and inflow
rpm	Revolutions per minute
SCADA	Supervisory Control and Data Acquisition
SS	Sanitary Sewer
TWDB	Texas Water Development Board
USGS	United States Geological Survey
TRA	Trinity River Authority of Texas

VFD	Variable Frequency Drive
Sewer Basin	Sanitary sewer drainage basin

1.0 Executive Summary

This Master Plan accomplishes the following objectives:

- Projected water usage and sewer loading rates based on existing conditions
- Updated design criteria for future planning and project development
- Condition assessments of all existing facilities, with recommendations for near and long term improvements
- Condition assessments of a large portion of system manholes, with recommendations for rehabilitation improvements
- Flow monitoring and a ranking of sewer basins based on RDII
- Updated City water and wastewater models
- Updated City water and wastewater GIS
- A list of water projects to reach identified 20-year growth plans and maintain existing infrastructure
- A list of wastewater projects to reach identified 20-year growth plans and maintain existing infrastructure

1.1 Population and Flow Projections

A combination of TWDB water master planning, regional city planning, and Colleyville's own projections were utilized to develop recommended population projections. These projections are shown in Table ES-1.

Table ES- 1: Population Projections 2020 - 2070

Year	Population
2010 US Census	22,807
2014	23,600
2020	25,800
2030	27,000
2034	27,420
2040	27,700
2050	28,000
2060	28,000
2070	28,000

These projections were incorporated in the projected water usage rates displayed in Table ES- 2, based on a combination of billed City usages and TWDB long-term flow projections. It is recommended to utilize TWDB projected estimates for infrastructure improvements planning.

Table ES- 2: Summary of Current and Projected Water Usages

Daily Demand	2014 Usage		2034 Usage	
	gpcd	MGD	gpcd	MGD
Base	122	2.879	122	3.345
Average Day (For Rate Study)	303	7.031	303	8.308
Average Day (For Improvements Planning)	303	7.031	342	9.378
Maximum Day	823	19.423	823	22.567

Sewer usage projections were developed by a combination of previous billed usages and flow rates. The current and projected average usage numbers are displayed in Table ES-3, with further detail on peak flow rate determination in Section 11.0 of this Master Plan.

Table ES- 3: Summary of Current and Projected Wastewater Average Daily Billed Flows

Year	Population	Average Flow (gpcd)	Total Average (MGD)
2012	23,005	97.1	2.253
2014	23,600	97.1	2.292
2034	27,420	97.1*	2.662

** Assumes with conservation measures, there will be no increase in gpcd*

1.2 Design Criteria

Design criteria were evaluated and updated for both water and wastewater systems, to determine the need for near and long term improvement projects. Summaries of those criteria are identified in Sections 5 and 10.

1.3 Existing Facility Assessments

Facility assessments were conducted on water and sewer facilities to evaluate the current condition of those facilities, and to recommend improvements, if warranted.

1.3.1 Water Facility Evaluation

The water facility evaluation is presented as Appendix A of this Master Plan. The water facilities were generally found to be in good condition, with the exception of the Overland Trail Ground Storage Tank (GST). Two Capital Improvements Projects involving minor site work at most locations and the demolition of the Overland Trail GST are included as Groups U and V. Water improvements projects were grouped in alphabetical order and assigned numerical priority as presented in Appendix D Water Capital Improvement Plan.

1.3.2 Wastewater Facility Evaluation

1.3.2.1 Lift Station Evaluation

A lift station evaluation was conducted at two of the City's active lift stations. The results of the assessment are documented in Appendix C, and show that minor improvements are warranted to maintain TCEQ compliance. However, these minor improvements for lift stations were not significant to merit separate project designation.

1.3.2.2 Manhole Condition Assessment

A City-wide manhole condition assessment and manhole survey was conducted on approximately one third of the City's wastewater system manholes and documented in a supplemental report to this Master Plan. The assessed manholes were then recommended for one of two priority levels of improvements, if warranted. These numbers were then projected to the entire City to account for manholes which were not surveyed due to line size or access. A summary of those findings is displayed in Figure ES -1.

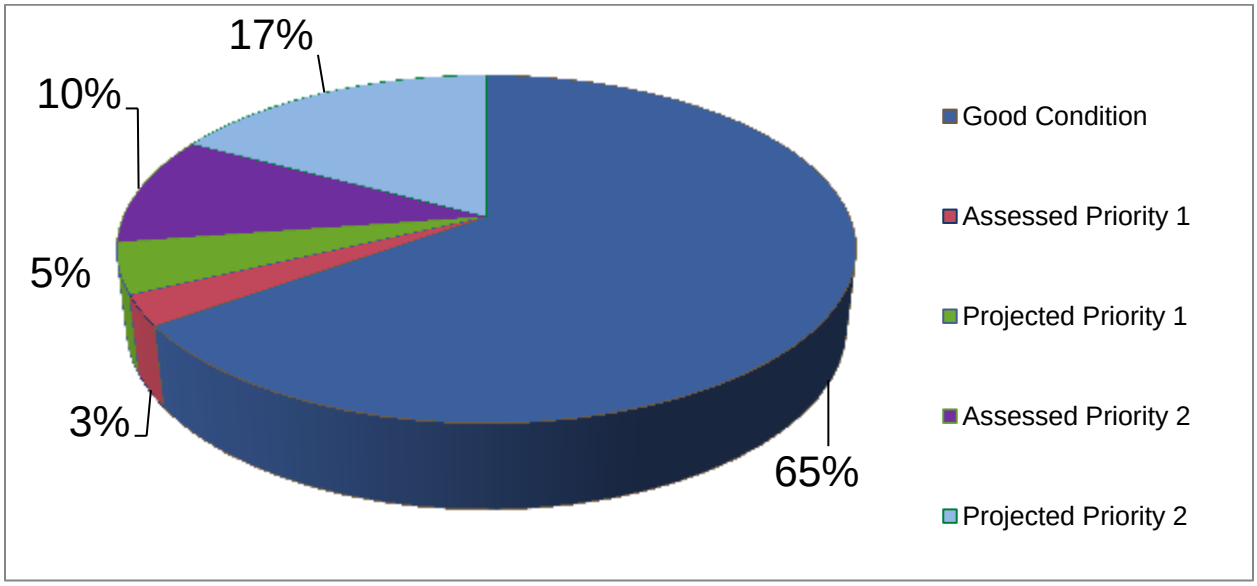


Figure ES- 1 - Manhole Assessment Results and Recommendation

1.4 Wastewater Flow Monitoring

A wastewater flow monitoring program was conducted for two months in 2014 to evaluate existing flows from the wastewater system. The flow monitoring program and results are included as a supplemental report to this Master Plan. The monitoring information was utilized to calculate the sewer basin ranking displayed in the report and summarized in Section 9, which also provides an estimated RDII per inch of precipitation. The RDII level of 1.2% of the total flow is relatively small (compared to aged systems which may experience RDII above 10%), and shows that the City has a sewer collection system which is fairly resilient to RDII.

The following conclusions were presented in the flow monitoring program:

- Dry weather flows are adequately transported without surcharge. Despite relatively low velocities, only one site exhibited debris deposition.
- The estimated RDII volume of 7.75 MG per year is relatively minor compared to the projected yearly flow of 624.2 MG (1.2%). However, repair and rehabilitation of the sanitary sewer system will assist in reducing the RDII volume, and non-flow I/I such as that attributed to dry-weather groundwater.
- Hydraulic data from the metering sites and rainfall gauges was available and utilized for hydraulic model calibration.
- Priority ranking of the sewer basins provides guidance for analysis of future I/I identification studies, such as smoke testing or CCTV.

1.5 Models

Full models of existing and future water and wastewater demands were developed and implemented into the City's current water models. These updates are included as digital files. The models implemented usage and future build out to identify improvement regarding flow rates, pressures, fire flows, and water age for the water system, and capacity and future growth needs for the wastewater system.

1.6 Water System Improvements

A list of recommended water improvement projects by priority is identified in Appendix D Water Capital Improvements Plan and summarized in the following Table ES-4. The projects are primarily driven by fire flow, capacity, and condition. The total project costs are shown in 2015 dollars, and have also been escalated in Appendix D to a projected cost, based on time of implementation and 3% escalation.

Table ES- 4: Summary of Proposed Water Capital Improvements

Project Identification				Schedule			2015 Cost ($1000)
Project	**Description**	**Location**	**Flexibility**	**Primary Trigger**	**Secondary Trigger**	**Trigger Date**	**OPCC**
1	Group Y	Area 1	Low	City-Identified	None	Oct-15	$1,363
2	Group Z	Area 2	Low	City-Identified	None	Oct-15	$627
3	Group A	Low Plane	Low	Regulatory	Capacity	Oct-15	$3,844
4	Group AB	Area 4	Low	City-Identified	None	Oct-16	$372
5	Group AC	Area 5	Low	City-Identified	None	Oct-16	$1,427
6	Group AA	Area 3	Low	City-Identified	None	Oct-17	$619
7	Group U	High Plane	High	Condition	Operational	Oct-17	$104
8	Group B	High/Low	Low	Capacity	Operational	Oct-18	$312
9	Group V	High/Low	High	Condition	Operational	Oct-18	$217
10	Group E	Low Plane	Medium	Fire Flow	City-Identified	Oct-18	$487
11	Group D	Low Plane	Medium	Condition	Fire Flow	Oct-19	$2,599
12	Group F	Low Plane	Medium	Condition	Operational	Oct-20	$2,328
13	Group L	Low Plane	Medium	Fire Flow	City-Identified	Oct-21	$1,333
14	Group I	High Plane	Medium	Fire Flow	Operational	Oct-21	$1,374
15	Group G	Low Plane	Medium	Fire Flow	Condition	Oct-23	$4,025
16	Group H	Low Plane	Medium	Condition	Operational	Oct-25	$1,636
17	Group J	Low Plane	Medium	Fire Flow	None	Oct-26	$985
18	Group K	Low Plane	Medium	Fire Flow	None	Oct-27	$5,414
19	Group M	Low Plane	Medium	Fire Flow	None	Oct-28	$2,638
20	Group W	Low Plane	Medium	Condition	City-Identified	Oct-29	$948
21	Group X	Low Plane	High	Capacity	City-Identified	Oct-30	$403
22	Group C	Low Plane	Medium	Capacity	Operational	Oct-30	$763
23	Group Q	Low Plane	Medium	City-Identified	Operational	Oct-30	$1,138
24	Group N	Low Plane	Medium	Fire Flow	Condition	Oct-31	$1,553
25	Group R	Low Plane	High	City-Identified	Condition	Oct-32	$711
26	Group S	High Plane	High	City-Identified	Operational	Oct-32	$104
27	Group T	Low Plane	High	City-Identified	None	Oct-33	$2,540
				Total 2015 OPCC:			**$39,863**

In addition to the recommended water capital improvements, operational recommendations such as storage levels in the tanks and associated water ages, and restructuring of the City's zoned water restrictions were recommended.

1.7 Wastewater System Improvements

A list of recommended wastewater projects by priority is identified in Appendix E, and summarized in the following Table ES-5. The projects are primarily driven by condition and capacity. The project costs are displayed in 2015 dollars, with a three percent estimated escalation per year projected to the proposed

implementation year in the Appendix. A cost comparison of taking the lift stations out of service was not recommended under current conditions.

Table ES- 5: Summary of Proposed Wastewater Capital Improvements

Project Identification				Schedule			2015 Cost ($1000)
Project	Description	Location	Flexibility	Primary Trigger	Secondary Trigger	Trigger Date	OPCC
1	Group X	Area 3	Low	City-Identified	None	Oct-17	$1,556
2	Group O	City-Wide	Low	Condition	None	Oct-18	$185
3	Group H	City-Wide	Low	City-Identified	Condition	Oct-18	$497
4	Group C	North	Low	Capacity	None	Oct-18	$193
5	Group F	North	Low	Capacity	Condition	Oct-22	$616
6	Group A	West	Low	Capacity	None	Oct-22	$632
7	Group T	South	Low	City-Identified	Condition	Oct-22	$624
8	Group G	East	Low	Capacity	None	Oct-22	$325
9	Group U	City-Wide	High	Condition	None	Oct-22	$2,259
10	Group P	City-Wide	Medium	Condition	None	Oct-22	$560
11	Group Q	City-Wide	Medium	Condition	None	Oct-22	$218
12	Group R	City-Wide	Medium	Condition	None	Oct-23	$318
13	Group E	East	Medium	Capacity	None	Oct-24	$1,020
14	Group V	City-Wide	High	Condition	None	Oct-26	$2,259
15	Group D	South	Medium	Capacity	None	Oct-26	$531
16	Group S	City-Wide	High	Capacity	None	Oct-26	$993
17	Group W	City-Wide	High	Condition	None	Oct-32	$2,259
				Total 2015 OPCC			**$15,044**

2.0 Introduction

2.1 City Summary

The City of Colleyville is a community located in northeast Tarrant County, Texas. The City is situated north and central to the Dallas/Fort Worth Metroplex, and comprises 13 square miles of mostly (~85%) developed area. The City is primarily residential, with a light commercial, institutional, and retail areas concentrated through the City's core along Colleyville Boulevard, and on the eastern and western fringes of the City limits.

The City is a wholesale customer of the Trinity River Authority (TRA) for treated water and wastewater treatment. The City does not own or operate any water or wastewater treatment facilities. Therefore, Colleyville's conveyance and storage infrastructure is critical to providing adequate water and sewer services to its citizens.

2.2 Objectives

The City has commissioned this Water and Wastewater Master Plan in order to evaluate the current condition of the existing infrastructure, and to adequately prepare for future growth and facility maintenance through a 20 year planning period. The Master Plan accomplishes the following:

- Documents field evaluations of existing water and sewer pumping stations
- Documents field evaluations of water storage facilities
- Documents field surveys of identified, accessible sanitary sewer manholes, as well as field assessments and measurements for each manhole
- Documents flow monitoring on identified sanitary sewer drainage basins, and evaluate wet weather response
- Identifies design criteria and constraints, including historical and projected population and usage
- Updates City GIS with new water and wastewater base maps
- Develops a water distribution system model for 6-inch and larger pipelines for current and 2034 demands
- Develops a wastewater collection system model for 8-inch and larger pipelines and implements current and 2034 demands
- Evaluates operational parameters for improvements to system operation and energy efficiency
- Develops a Capital Improvements Plan (CIP) to maintain the existing system and provide for future growth

2.3 Acknowledgements

Staff members throughout the City, including the Public Works and Community Development departments, were integral to the development of this Master Plan. Garver and our consultant team is sincerely grateful for their dedication to this effort.

3.0 Population and Flow Projections

3.1 Population Trends

Historical and projected population trends shown in Figure 1 are based on an analysis conducted of Texas Water Development Board (TWDB) 2011 and 2013 data, historical data, and population projections by the City based on existing and future land development projections. These projections are averaged into the "Combined Projection" trend line to display a conservative population growth expectation that accommodates all projections, but stays fairly close to the City's own holding-capacity method projected growth estimates.

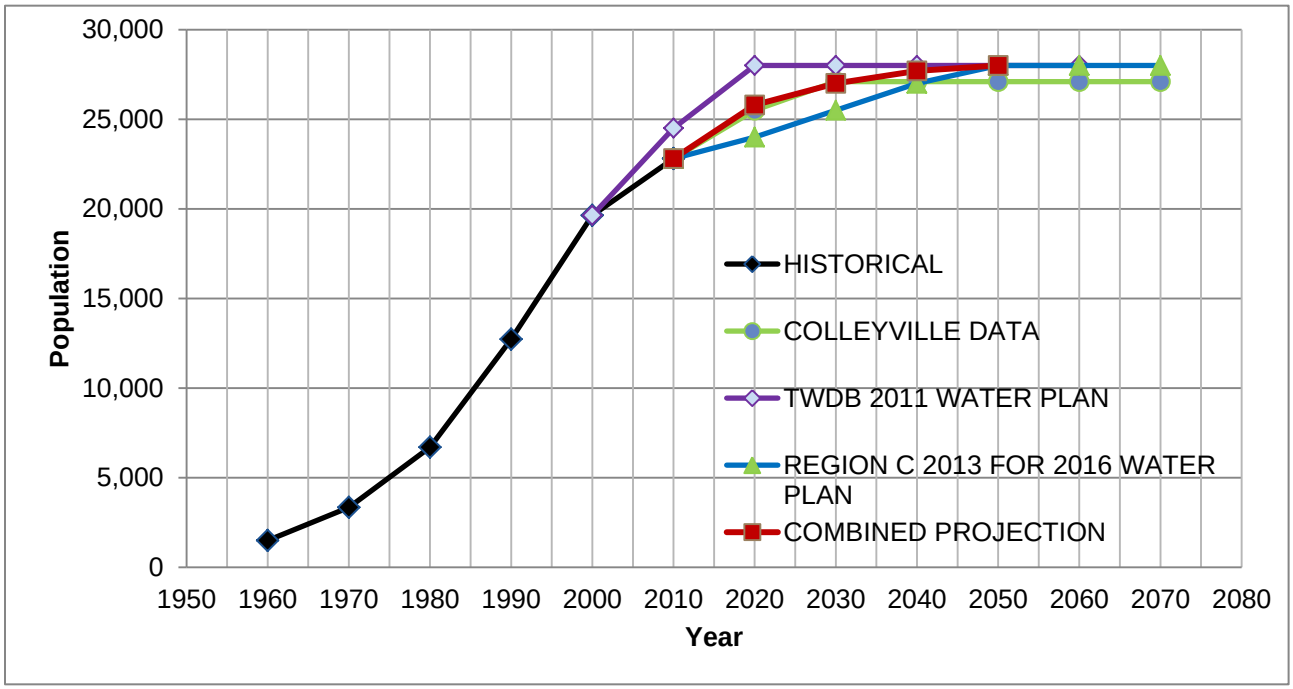


Figure 1: Comparison of Population Projections

The anticipated populations for 2020-2070 are provided in Table 1. The current population of 23,600 residents and a projected 2034 population of 27,420 will be used for this report's 20 year planning period.

Table 1: Population Projections 2020 - 2070

Year	Population
2010 US Census	22,807
2014	23,600
2020	25,800
2030	27,000
2034	27,420
2040	27,700
2050	28,000
2060	28,000
2070	28,000

3.2 Water Historical Use Data

3.2.1.1 Base Demand

Through interviews with City staff and observed diurnal demand curves, it is estimated that a majority of maximum day demand is attributed to lawn and landscape irrigation. This correlates to a sharp increase in monthly usage during warm weather months. Therefore, the previous five years of billed meter usage for January and February, when most irrigation systems would not be active, was used to calculate the base flow demand. This demand is useful for comparing peak flow events and correlating them to time of year.

It is assumed that the January billing cycle is 31 days, while the February billing cycle is 28 days. The average demand per day was calculated to be 2,837,850 gallons. Using a 5 year population average of 23,203, this equates to a base gallons per capita consumption of 122 gpcd. Utilizing the current population and this calculated per capita demand, the total current base demand is estimated at 2.879 MGD. This information is summarized in Table 2.

Table 2: Current Base Demand Summary

Average 5 year Demand For Jan/Feb (MGD)	2.838
5 Year Population Average	23,203
Per Capita Base Consumption (gpcd)	122
2014 Population	23,600
Total 2014 City-Wide Base Demand (MGD)	2.879

3.2.1.2 Average Demand

The average demand is calculated from the average usage throughout the entire year. This demand is useful for estimating yearly usage and identifying usage patterns. The previous four years of City billed metered usage are shown in the following Figure 2 versus the annualized lake evaporation recorded during that time. The lake evaporation rate utilizes a combination of wind, temperature, humidity, rainfall, and other factors to quantify a water balance. A higher lake evaporation rate indicates that the environment is drying out more quickly, and increases during time of drought. As a result, it is anticipated that the City's water usage would increase as users increase lawn and landscape irrigation.

The yearly total water usage closely follows the annual average evaporation rates. However, 2012 exhibits increased usage relative to the evaporation rate. This is likely caused by high system demands to compensate for an extremely dry year in 2011. Therefore, lake evaporation will be considered as an indicator of water usage for the City for the maximum day evaluation. This allows for more accurate understanding of historical maximum flow rates, and increases confidence in future projections based on those historical maximum usage rates.

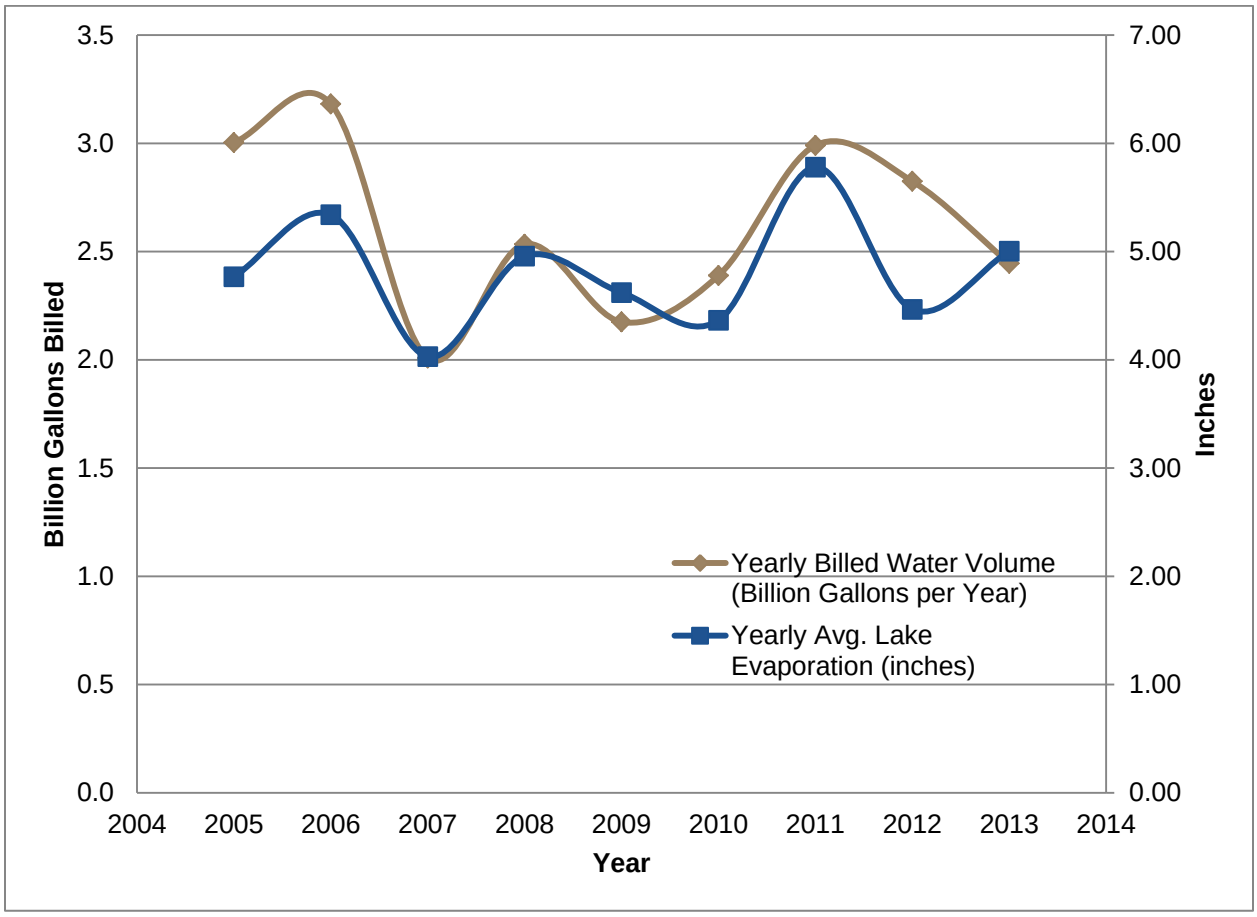


Figure 2: Colleyville Metered Usage per Year (Billions of Gallons) and Lake Evaporation (Inches)

Total usage appears to have decreased significantly in 2006 with the implementation of a time of day irrigation restriction, and stayed below that high demand year even in historical drought conditions experienced in 2011. Zone watering descriptions implemented in 2013 will be discussed further in this Master Plan.

The previous billings provide an average yearly metered usage of 2,565 MG. Using the five year average population over this time, the daily average usage equates to 7.031 MG, or 303 gpcd. Total current average day usage at the current population results in an average daily water usage rate of 7.151 MG. This information is summarized in Table 3.

Table 3: Current ADD Summary

Average 5 year Demand Per Day (MG)	7.031
5 Year Population Average	23,203
Per Capita Average Consumption (gpcd)	303
2014 Population	23,600
Total City-Wide 2014 Average Demand (MG)	7.151

3.2.1.3 Maximum Day

Figure 3 displays the maximum day of historical usage in the previous four years of billed meter usage and the recorded lake level evaporation during that time period. As detailed in the previous section, the recorded lake level evaporation allows a better understanding of causes for increased usage rates.

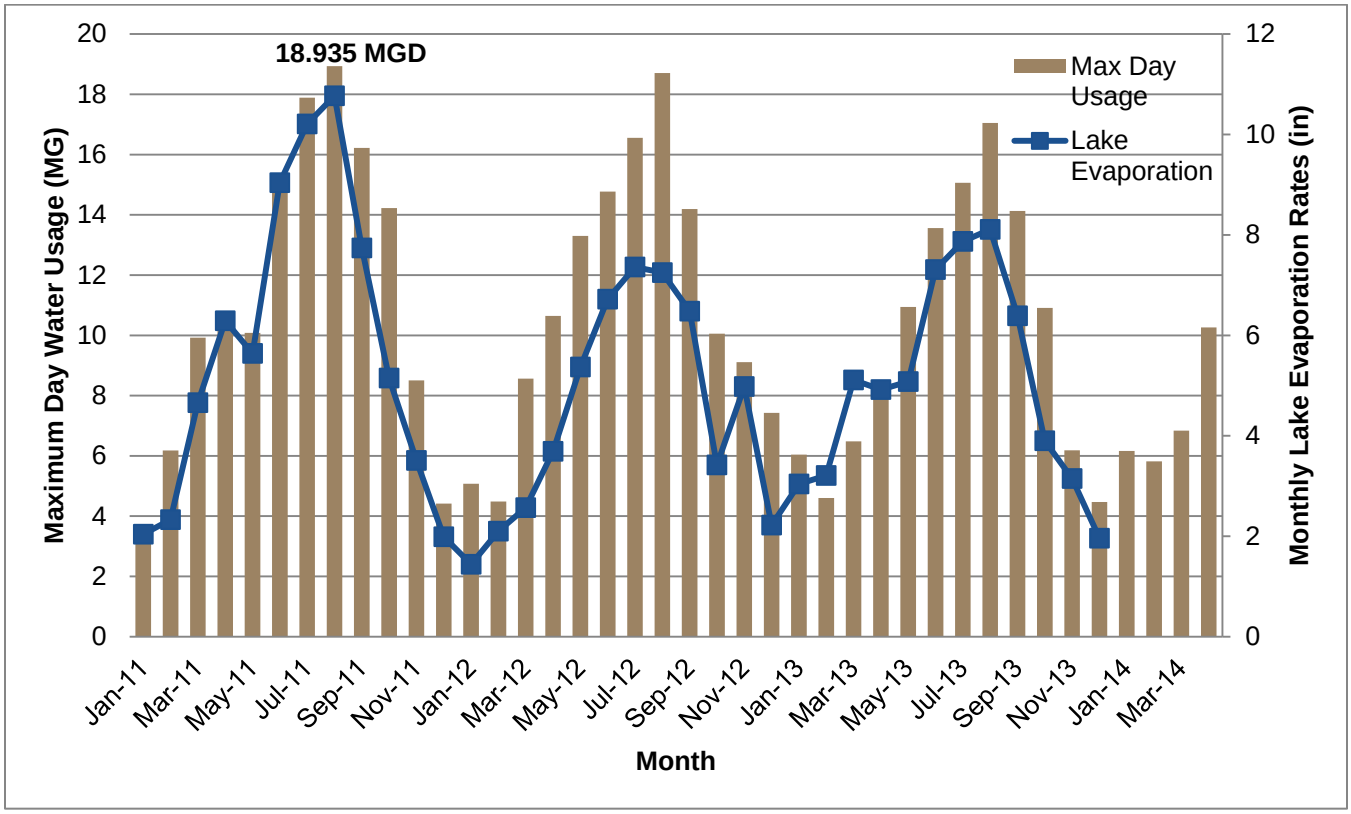


Figure 3: Peak Day Historical Usage Compared to Lake Evaporation

Figure 3 reveals a correlation to a historical MDD in August 2011 of 18.935 MG and evaporation rates 45% greater than 2012 and 2013. Zoned irrigation schedules implemented in 2013 appear to have reduced maximum usage by approximately 10% during similar weather conditions in 2012 and 2013.

The maximum usage in 2011 occurred during historical drought conditions which have not been experienced following 2011. However, in 2013, the MDD still approximated 90% of 2011 levels when population growth is accounted. Drought conditions similar to 2011 will occur in the future, and 2011 usage rates may be experienced again, regardless of current watering restrictions. Therefore, this evaluation will assume 2011 usage rates for the 20 year planning period. Historical records indicate a 2011 population of 23,005 citizens, which equates to a per capita maximum day demand of 823 gpcd. A current MDD utilizing 823 gpcd peak and current population estimates results in a 2014 peak flow rate of 19.423 MG. These calculations are summarized in Table 4.

Table 4: Current Maximum Day Demand Summary

Year	Population	MDD (gpcd)	MDD (MG)
2011	23,005	823	18.935
2014 (projected)	23,600	823	19.423

3.3 Water Projected Use Data

3.3.1 Base Demand

Future growth of the City is anticipated to mirror the current land use zones (residential, light commercial, etc.). TWDB projections, detailed in the following section, show a usage reduction of 1% over the planning period due to anticipated water conservation measures and the implementation of more efficient appliances. This reduction is relatively insignificant when compared to the current base demand of 122 gpcd. Therefore, a future rate of 122 gpcd will be maintained for planning purposes. When applied to the year 2034 population, the future base demand average day equals 3.345 MGD.

3.3.2 Average Day

The TWDB conducts regular State Water Plans to enable communities and entities with the necessary information to plan infrastructure for future water needs. This data is also useful in interpreting future usage and providing the tools necessary for long term rate study usage.

Utilizing historical usage and long term forecasting of usage and conservation trends, TWDB has published Board-approved future water usage rates for Colleyville as shown in Table 5.

Table 5: TWDB Usage Projections

Year	Total Usage (MG)	Gallons per capita/day
2020	3,037	346
2030	3,196	343
2040	3,361	341
2050	3,473	340
2060	3,470	339
2070	3,470	339

The immediate increase of per capita water consumption is projected from the continuous rise in City usage as full build-out continues, and future decrease is attributed to the implementation of water saving measures. As calculated from these projections, the 2034 average day usage is 342 gpcd, or 9.378 MGD. While this projection is 13% higher than the current usage rate of 303 gpcd, the TWDB estimates were based on the maximum year of record 2011 usage numbers. This reflects a conservative approach for planning purposes. Utilizing this approach will better prepare the City if those demands are realized, but may artificially inflate future billing estimates if current conservation trends continue.

It is recommended to utilize TWDB projected estimates for infrastructure improvements planning. However, for future rate and usage analysis, it is recommended to utilize the historical per capita usage average of 303 gpcd. At a 2034 population, that would equate to 8,308,260 gallons. These recommended usage rates are displayed in Table 6.

Table 6: Current and Projected Average Day Demands

Year	Population	Average Usage (gpcd)	Average Usage (MGD)
2011	23,005	342	7.868
2014	23,600	303	7.150
2034 (For Usage Planning)	27,420	303	8.308
2034 (For Improvements Planning)	27,420	342	9.378

3.3.3 Maximum Day

The historical peak day usage of 823 gpcd was applied to the 2034 population of 27,420, for a total future MDD of 22.567 MG. The 2011, 2014, and 2034 MDD total numbers are displayed in Table 7. Further discussion regarding maximum day design criteria is located in Section 5.

Table 7: Current and Project Maximum Day Demands

Year	Population	Maximum Day (gpcd)	Maximum Day (MGD)
2011	23,005	823	18.935
2014	23,600	823	19.423
2034	27,420	823	22.567

3.4 Water Usage Summary

Table 8 provides a summary of the previous historical and future usage demands.

Table 8: Summary of Current and Projected Water Usages

Daily Demand	2014 Usage		2034 Usage	
	gpcd	MGD	gpcd	MGD
Base	122	2.879	122	3.345
Average Day (For Rate Study)	303	7.031	303	8.308
Average Day (For Improvements Planning)	303	7.031	342	9.378
Maximum Day	823	19.423	823	22.567

3.5 Wastewater Historical Use Data

The previous three years of TRA-metered monthly wastewater flows were provided by the City. An average daily flow rate of 2.253 MGD was calculated from that data. This is summarized in Figure 4.

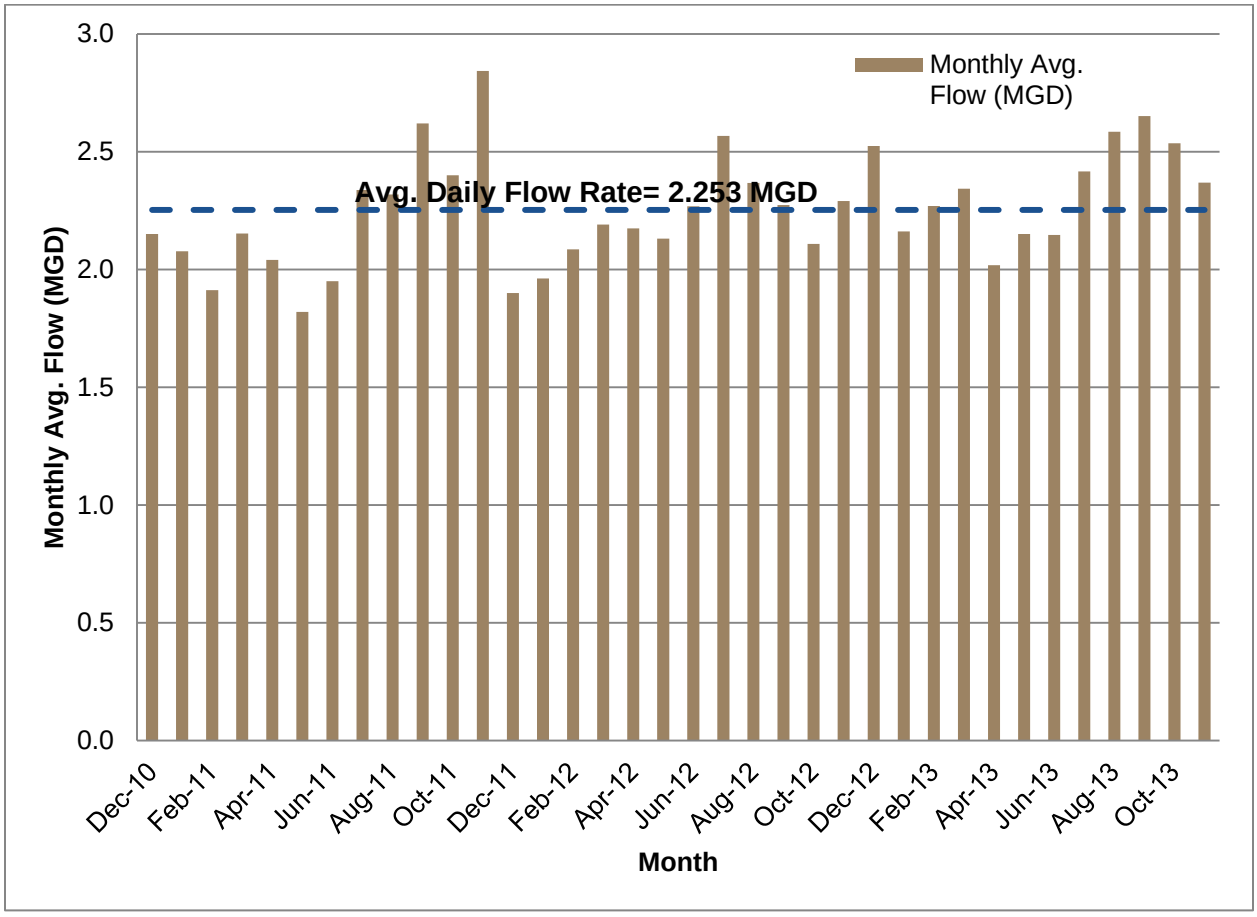


Figure 4: Average Wastewater Flow Rate Based on TRA Billings

A 2012 population of 23,205 was calculated from available population data. This was applied to the calculated average daily flow rate for an average flow of 97.1 gpcd. This number is in accordance with industry standards to assume per capita wastewater flow rates of 100 gpcd. Therefore, the calculated number will be utilized for current and projected sanitary sewer average day loadings. Current and future average flows are displayed in Table 9.

Table 9: Summary of Current and Projected Wastewater Average Daily Billed Flows

Year	Population	Average Flow (gpcd)	Total Average (MGD)
2012	23,005	97.1	2.253
2014	23,600	97.1	2.292
2034	27,420	97.1	2.662

Historical peak day flow rates are unknown. Therefore, the flow monitoring results discussed later in this Plan will be utilized to document current and projected peak flow rates.

4.0 Existing Water System

4.1 Overview

The City's water system consists of the items summarized below and in Table 10:

- 195 miles of pipelines;
- two pressure planes, high and low;
- two pump stations: Overland Trail Pump Station and L. D. Lockett Pump Station;
- two ground storage tanks, one at each pump station; and
- three elevated storage tanks: Bransford Elevated Tank, Hall Johnson Elevated Tank and McPherson Elevated Tank.

An overall map of the existing water system is included as Exhibit 1.

The City of Colleyville purchases wholesale treated water from TRA, and has the ability to receive this water from four metered take points. The southern meter is located at Jackson and Cheek Sparger, the western meter is located at 4892 Bransford Road, the eastern meter is located at 5485 Pool Road and the northwest meter, which supplies the LD Lockett Pump Station, is on Glade Road at Little Bear Creek in Hurst.

Table 10: Water System Facilities

Facility	Description and Size	Type	Pressure Plane
Pump Stations			
Overland Trail Pump Station	3.5 MGD (four pumps) • Two 500 gpm • Two 750 gpm	Horizontal split case	High
LD Lockett Pump Station	11.5 MGD (eight pumps) Low plane: • Two 550 gpm • Two 1,200 gpm High plane: • Two 750 gpm • Two 1,500 gpm	Horizontal split case	High and Low
Ground Storage Tanks			
Overland Trail GST (*Currently out of service)*	1.2 MG	Riveted steel	High
LD Lockett GST	5.0 MG	Concrete	High and Low
Elevated Storage Tanks			
Bransford EST	1.0 MG	Welded steel	Low
Hall Johnson EST	1.5 MG	Composite	Low
McPherson EST	1.0 MG	Composite	High

A facility summary and condition assessment was conducted by Garver in a separate Technical Memorandum, and included in this Master Plan as Appendix A. The Memorandum identifies improvements that are recommended further in this Master Plan's Capital Improvements Plan.

4.2 Distribution System

The water distribution system consists of 1,027,850 total feet of pipeline, as shown in Table 11.

Table 11: Distribution System Pipeline Inventory

Size (in.)	Total Length (ft)	Portion of System (%)
4	4,177	0.4
6	373,276	36.3
8	414,106	40.3
10	123,182	12.0
12	64,500	6.3
16	34,382	3.3
20	13,305	1.3
24	922	0.1
Total	**1,027,850**	**100%**

The pipelines range in age, material, and condition. A formal condition assessment of the pipelines was not conducted as a part of this Master Plan. However, known condition concerns were identified through input by City staff, and incorporated into model recommendations. Pipes identified by City staff as condition concerns and in need of replacement are as follows:

- Glue-joint 6-inch pipe north of McPherson EST along Overland Trail Drive
- Asbestos cement 6-inch pipe directly south of McPherson EST and along Bandit Trail
- Asbestos cement 6-inch pipe along the western end of John McCain Road
- Asbestos cement 6-inch pipe along Pleasant Run Road from Tinker Road to John McCain Road
- 8-inch pipe along the eastern portion of John McCain Road, which is causing a known bottleneck
- 6-inch glue-joint pipe along Tinker Road to the intersection of Colleyville Boulevard
- A combination of glue-joint and asbestos cement 6-inch pipe in the neighborhoods west of Bransford Road
- Asbestos cement 6-inch pipe in the area surrounding LD Lockett Park
- 2-inch bottleneck at Black Drive
- Glue-joint 6-inch pipe along Cheshire Drive
- Asbestos cement 8-inch pipe along Spring Hollow Road
- Combinations of 6-inch and 10-inch pipes along Pool Road
- An 8-inch bottleneck along Glade Road and Springs Court to Hollowbrook Court
- Asbestos cement 12-inch along Cheek-Sparger near Springhollow to Martin

These pipes have been identified and summarized within Capital Improvements Plan projects proposed in Appendix D of this Master Plan.

The TRA supply pipelines, (27-, 33-, and 36-in diameter) also traverse the City, but are not included in this evaluation. The City does not own or operate these TRA supply pipelines.

4.3 Pump Stations

The City of Colleyville owns and operates two water pump stations: Overland Trail Pump Station and LD Lockett Pump Station. A field investigation of each pump station and recommended improvements is located in Appendix A, and summarized in Section 4.5.

4.3.1 Overland Trail Pump Station

Overland Trail Pump Station is a 3.5 MGD facility with four horizontal split case pumps; two pumps have a capacity of 500 gpm and two pumps have a capacity of 750 gpm. The pump station site contains three buildings: Pump Building, Chlorine Building and Ammonia Building. The pump station is currently offline, due to concerns with the condition of the existing ground storage tank.

4.3.2 LD Lockett Pump Station

LD Lockett Pump Station is an 11.5 MGD facility with eight horizontal split case pumps:

- two 550 gpm pumps for low plane average flow
- two 1,200 gpm pumps for low plane peak flow
- two 750 gpm pumps for high plane average flow
- two 1,500 gpm pumps for high plane peak flow

The pump station site contains three buildings: Pump Building, Rechlorination Building and Dechlorination Building. The pump station contains several vaults for low and high plane surge valves, tank fill and drain control, rechlorination and dechlorination.

4.4 Storage Tanks

This City of Colleyville maintains two ground storage tanks and three elevated storage tanks:

- Overland Trail Ground Storage Tank
- LD Lockett Ground Storage Tank
- Bransford Elevated Tank
- Hall Johnson Elevated Tank
- McPherson Elevated Tank

4.4.1 Overland Trail Ground Storage Tank

The 1.2 million gallon Overland Trail Ground Storage Tank is a riveted steel tank approximately 140 feet in diameter with a height of approximately 25 feet. The tank is reported to have been relocated from an air base near San Angelo many years ago by the Leonard Brothers Water Supply Corporation. When the City of Colleyville acquired the portion of the Leonard Brothers water system inside the city limits, the tank was included with the purchase. The tank is showing signs of distress and is heavily corroded and leaking. The upper portion of the tank appears to be buckling as evidenced by the ladder bowing out from being originally straight. Due to its poor condition, the tank is currently out of service.

4.4.2 LD Lockett Ground Storage Tank

The 5 million gallon LD Lockett Ground Storage Tank is a concrete tank approximately 150 feet in diameter with a height of approximately 50 feet. The tank is fed from the TRA northwest connection, and provides a reservoir of water for the pump station.

4.4.3 Bransford Elevated Storage Tank

The Bransford Elevated Storage Tank is a 1.0 million gallon welded steel tank approximately 75 feet in diameter with a height of approximately 150 feet. The tank was originally built in 1982.

4.4.4 Hall Johnson Elevated Storage Tank

The Hall Johnson Elevated Storage Tank is a 1.5 million gallon composite tank approximately 60 feet in diameter with a height of approximately 140 feet. The tank was originally built in 1988. It is co-located with the City of Colleyville Public Works Service Center.

4.4.5 McPherson Elevated Storage Tank

The 1.0 million gallon McPherson Elevated Tank is a composite tank approximately 70 feet in diameter and approximately 140 feet in height. The tank was originally built in 1999.

4.5 Facility Assessment

A facility assessment was conducted of all storage and pumping facilities described in Sections 4.3 and 4.4. The purpose of the field evaluation study is to:

- Compare TCEQ requirements to the existing installations
- Identify any regulatory upgrades needed
- Evaluate the overall condition of those facilities
- Identify any conditional improvements needed
- Include any major capital expenditures within the CIP

A technical memorandum presenting the findings of those field evaluations is included in this Master Plan as Appendix A, and summarized in Table 12. These costs have been identified in CIP projects, located in Appendix D of this report.

Table 12: Water Facilities Field Evaluation Results

Facility		Description	Improvements Needed	Forecasted Cost
Overland Trail	PS	Well maintained, good condition	Decommissioning	$75,000
	GST	Poor condition	Minor site improvements, tank demolition	
LD Lockett	PS	Well maintained, good condition	None	$10,000
	GST	Minor site erosion, overgrown vegetation, poor site security	Site grading, erosion and vegetation control, adjustment of flap valve, and fencing	
Bransford EST		Minor site erosion, overgrown vegetation, minor site security issues, standpipe corrosion, interior walkway and metal plate erosion	Replace interior tank components, repaint interior and exterior components, remove site vegetation, remount fencing	$135,000
Hall Johnson EST		Well maintained site, good facility condition	None	None
McPherson EST		Erosion issues near overflow, paint is peeling on piping	Drainage channel, repainting exterior piping	$12,500

4.6 Watering Zones

4.6.1 Overview

Texas water systems that rely on surface water have become increasingly concerned with dwindling supplies of available fresh water. As populations increase and more land is developed, total and per capita usage also increases. The Texas Water Development Board and Texas Legislature has recognized the value that conservation contributes towards protecting these water supplies, and has mandated Water Conservation Plans which identify water conservation measures for public utilities or wholesale providers serving 3,300 or more connections.

The City's surface water is supplied by the Tarrant Regional Water District (TRWD) to the TRA for treatment and conveyance to member cities. Therefore, the City's Conservation Plan is also reliant upon the TRWD's Water Conservation Plan, intended water usage conservation, and triggers for water usage reduction measures.

As identified by the TRWD's 2013 Plan, time of day irrigation requirements were implemented in 2006 by the City as a proactive response to conservation efforts. The 2013 Plan identified incentives for

conservation, including water use reduction due to increased price and irrigation limits of 2 times per week for residential customers. The City implemented these recommendations as part of the City's conservation Plan, and applied a tiered rate structure to promote reduced usage. Additionally, zoned watering restrictions were implemented in 2013 with the onset of Stage 1 of the TRWD drought contingency plan. The zones were implemented in an effort to restrict water usage and clearly define borders that citizens could clearly identify. The zone boundaries are shown in the following Figure 5.

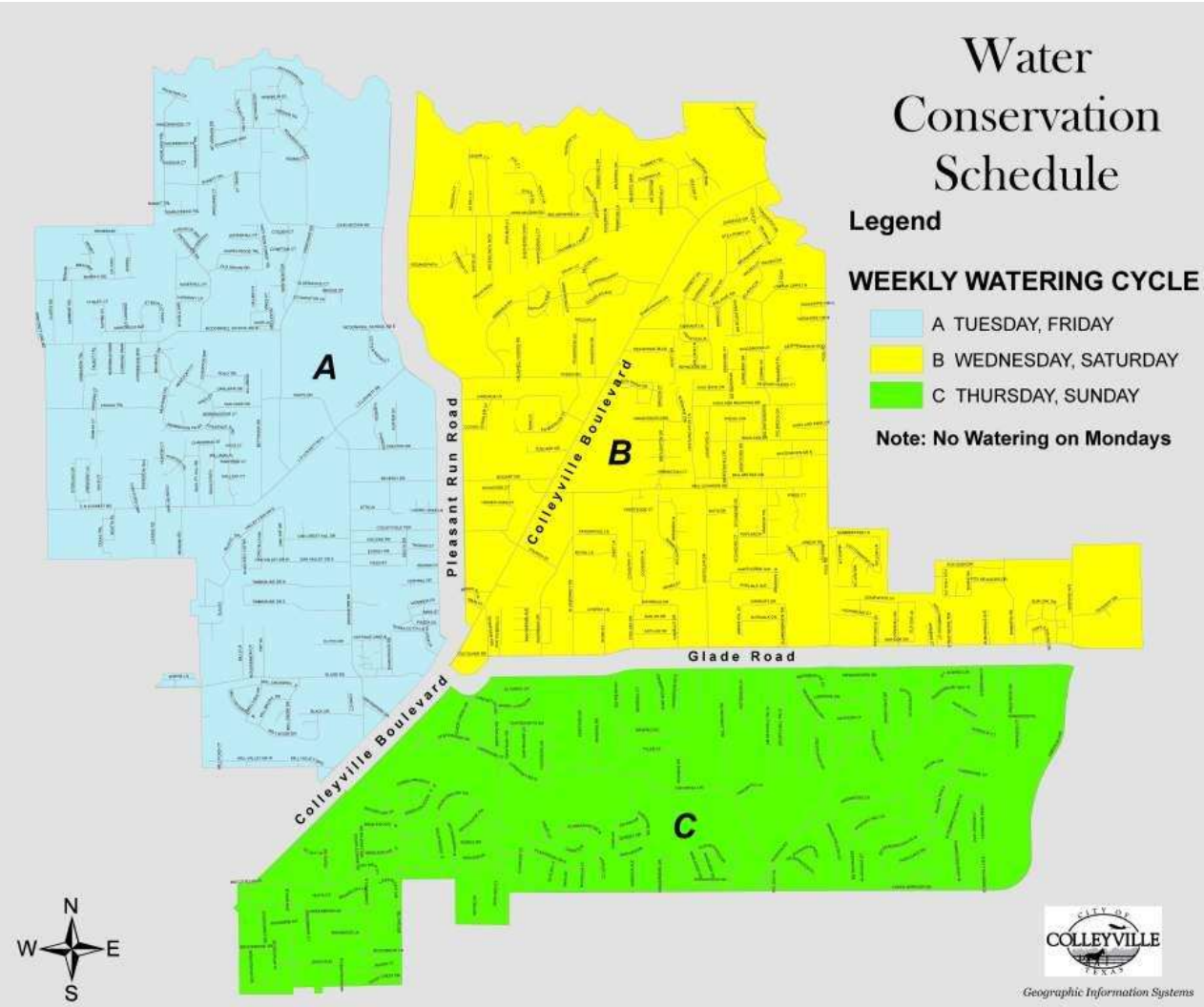


Figure 5: Colleyville Zone Restrictions

4.6.2 Impact on Total Demand

The previous Figure 3 of this Master Plan identifies a 10% decrease in maximum day water usage from 2012 to 2013, compared to a 30% increase in lake evaporation during that same time period. This indicates that the implementation of a watering restriction schedule does have a positive impact on reducing overall City consumption. However, as previously detailed in Section 2, the historic maximum

day demand may be experienced again, if weather conditions similar to 2011 occur. While the zone watering schedules may limit average daily usage, it is unknown and unlikely that they will have an impact on reducing the maximum day demand.

4.6.3 Impact on Zone Usage

Appendix B contains a technical memorandum that evaluates the need for Overland Trail in the current or future water system. The study found that the zone watering restrictions play a key factor in the maximum demands the system will experience, depending on which zone is allowed to water the day that high demand is experienced. The difference between zones is due to the variation in number of meter connections per zone. Zone B is the largest of the zones, therefore it is anticipated that the total maximum demand of 19.423 MG would occur on that watering day. Anticipated maximum day demands relative to the watering day is summarized in Table 13.

Table 13: Maximum Day Demands Relative to the Zone Watering Day

Watering Zone Day	Base Usage City-Wide (MG)	Zone-Specific Maximum Day Usage (MG)	Total Maximum Day Usage (MG)
A	7.091	8.903	15.993
B	7.091	12.332	19.423
C	7.091	10.646	17.736

The study determined that because maximum demand could hit on any of these watering days, zone water restrictions would have an unintended consequence of localizing maximum demands within each zone. This resulted in larger infrastructure requirements in each of those zones to prepare for maximum day, rather than if the demand were distributed evenly across the City. Though the study focused primarily on the high plane and Overland Trail, similar infrastructure sizing increases would be necessary in tanks and pipelines throughout the distribution network to accommodate the zone-localized demands.

The estimated difference in the high pressure plane between the current (with zone restrictions) anticipated demand, existing high plane system storage capacity without Overland Trail, and anticipated demand without zone restrictions is shown in Table 14.

Table 14: Comparison of Storage Requirements

Maximum Day Zone A High Pressure Plan Demand (MG)	2014	2034
Current Zone Restrictions	5.38	6.23
Existing High Plane Storage Capacity	5.32	5.32
Difference*	**-0.06**	**-0.91**
Proposed Zone-less Restrictions	2.76	3.20
Existing High Plane Storage Capacity	5.32	5.32
Difference*	**2.56**	**2.12**

**Positive = available storage, negative = storage shortage*

4.6.4 Conclusion

It is recommended that the zoned watering restrictions would be replaced with a City-wide watering restriction schedule. This will save the City capital expenditure, reduce localized demands, and maintain the infrastructure necessary to meet the proposed maximum day demand. A proposed watering schedule is presented in Section 8.

5.0 Water Design Criteria

5.1 Site Security, Maintenance and Housekeeping

The Texas Commission on Environmental Quality (TCEQ) provides requirements to ensure that public water systems supply safe drinking water to their customers. The recommendations identified within this report and in Appendix A for facility improvements were founded on the requirements set forth by TCEQ:

§290.43. Water Storage
(e) Facility security. All potable water storage tanks and pressure maintenance facilities must be installed in a lockable building that is designed to prevent intruder access or enclosed by an intruder-resistant fence with lockable gates. Pedestal-type elevated storage tanks with lockable doors and without external ladders are exempt from this requirement. The gates and doors must be kept locked whenever the facility is unattended.

§290.46. Minimum Acceptable Operating Practices for Public Drinking Water Systems.
(m) Maintenance and housekeeping. The maintenance and housekeeping practices used by a public water system shall ensure the good working condition and general appearance of the system's facilities and equipment. The grounds and facilities shall be maintained in a manner so as to minimize the possibility of the harboring of rodents, insects, and other disease vectors, and in such a way as to prevent other conditions that might cause the contamination of the water.

5.2 Flow, Pressure and Storage

In addition to site improvements, TCEQ also regulates water system design requirements. The following Table 15 compares TCEQ requirements to those adopted by previous Colleyville Capital Improvements Plans. Design criteria were selected for this Master Plan based on a combination of industry standards, International Fire Code recommendations, City preference, and TCEQ requirements.

In regards to high service pumping, TCEQ requirements were selected as the primary criteria, because the City is not a typical isolated system and available pressure and flow from the TRA supply lines will reduce required high service pump sizing. The fire flow and maximum day analyses will ensure adequate high pressure pumping is in place during these scenarios.

Table 15: Water System Design Criteria Summary

System Element	TCEQ	Industry / Colleyville Standard	2014 Master Plan
Water Supply	0.6 gpm/connection min.	1.62 gpm/connection (Max Day)	1.62 gpm/connection (Max Day)
Min. Normal Operating Pressure	35 psi min.	35 psi @ Max Day demand	35 psi @ Max Day demand
Min. Residual Pressure Under Combined Fire and Drinking Water Demands	20 psi	20 psi	20 psi
Min. Static Pressure		40-60 psi	40 psi
Elevated Storage	100 gal/connection 200 gal/connection if no ground storage Must be at least 80 feet above highest service connection in the pressure plane served.	15% of Maximum day Demand	15% of Maximum day Demand
Ground Storage		130 gal/capita	130 gal/capita
Total Storage	200 gal/connection	Sum of elevated and ground storage requirements.	Sum of elevated and ground storage requirements.
Fire Flows	None	Residential = 1,000 gpm Lt Mercantile = 1,500 gpm Industrial and Principal Mercantile = 3,000 gpm	Residential = 1,500 gpm Lt Mercantile = 1,500 gpm (Based on City preference and IFC guidelines) Industrial and Principal Mercantile = 3,000 gpm Must be maintained for 3 hours
High Service Pumping	If elevated storage is at least 200 gal/connection, each pressure plane or pump station must provide 0.6 gpm/connection All pump stations must have two or more pumps per pressure plane.	Max Day demand plus fire flows	If elevated storage is at least 200 gal/connection, each pressure plane or pump station must provide 0.6 gpm/connection All pump stations must have two or more pumps per pressure plane.

(Continued on following page)

Table 15: Water System Design Criteria Summary (Continued)

System Element	TCEQ	Industry / Colleyville Standard	2014 Master Plan
Water Mains	Number of Conn. / Min Line Size (in.) 10 / 2 25 / 2.5 50 / 3 100 / 4 150 / 5 250 / 6 >250 / 8 No new lines under 2" in diameter are allowed.	Minimum main providing fire protection: 8" Minimum line size serving Single Family Residential: 6" Minimum line size serving other than Single Family Residential: 8" Maximum looped line length: 3,000 feet Maximum dead end length: 1,200 feet Along SH-26, minimum 10" line on both sides hwy.	Minimum main providing fire protection: 8" Minimum line size serving Single Family Residential: 6" Minimum line size serving other than Single Family Residential: 8" Maximum looped line length: 3,000 feet Maximum dead end length: 1,200 feet Along SH-26, minimum 10" line on both sides
Maximum Pipe Velocities	Sewer: 10 ft/s Water: No direction	Sewer: 10 ft/s Water: <10 ft/s	Sewer: 2-10 ft/s Water: <10 ft/s

Section 3 of this Plan detailed the current and future flow rates that will be applied to the model. The water system will be modeled utilizing evenly distributed odd/even restrictions across the City. It will also be assumed that a minimum of twice weekly watering restrictions will remain in place for the planning period.

6.0 Water Model Development

The modeled distribution system for the City of Colleyville consists of the pipe network, supply connections, elevated and ground storage, and pumping infrastructure that allow the City to supply potable water to its customers. A hydraulic model representation of the physical system has been created using Bentley WaterGEMS V8i SELECTseries 4 (WaterGEMS). The following sections detail components of the hydraulic model development process.

6.1 Pipe Network

The pipe network was developed based on the City's GIS database. The existing GIS pipe database was imported into WaterGEMS using the ModelBuilder tool, which allowed the pipe locations, lengths, and sizes listed in the GIS database to be directly converted to pipe attributes within the hydraulic model. The connectivity of the pipe network was refined through discussions with City personnel, cross-referencing with the previous hydraulic model, and engineering judgment to determine the most reasonable assumptions where little data existed.

6.2 Elevated Storage Tanks

The base, minimum, and maximum (overflow) levels for each elevated tank were specified in the model based on engineering drawings for each of the tanks. These values are listed in Table 16. Storage versus level relationships were set in the model based on volume charts from the tank manufacturer or design drawings.

Table 16: Elevated Tank Elevations

Facility	Base Elevation (ft)	Minimum Elevation (ft)	Maximum Elevation (ft)
High Plane			
McPherson EST	691.00	800.00	840.00
Low Plane			
Bransford EST	661.50	755.00	790.00
Hall Johnson EST	657.00	750.00	790.00

6.3 TRA Meter Station Connections

The City's distribution system is supplied with water from TRA through four (4) supply connections. The Northwest connection supplies water to the LD Lockett GST. The East, South, and West connections supply water directly into the system's low plane. The TRA connections serve as boundary conditions in the hydraulic model, and accurate representation of these elements is critical in developing an accurate model of system hydraulics.

Data at each meter station was provided for the first day of each month for the period of May 2013 through May 2014. The types of data provided at each station are summarized in the following Table 17.

Table 17: Data Provided at Meter Stations

Facility	Flow Rate	Pressure
Northwest Meter Station	Yes	Yes
South Meter Station	Yes	No
West Meter Station	Yes	Yes
East Meter Station	Yes	Yes

The flow and pressure data at the Northwest, West, and East Meter Stations were analyzed. The flow and pressure pairs corresponding to non-zero flow rates were used to develop the graph shown in Figure 6. The results show the average pressure at each of the three stations is approximately 90 psi and that the pressure values are generally within the range of 80 to 100 psi. However, there is not a discernable trend relating pressure values to flow rates at any of the meter stations.

The East, South, and West Meter Stations were included in the City's previous water distribution system model. The elevations for these three meter stations were taken from that model:

- East Meter Station: 569.60 feet
- South Meter Station: 583.10 feet
- West Meter Station: 592.50 feet

The elevation of the Northwest Meter Station was set as 602 feet based on elevation contour maps. The elevations of the meter stations are used, in conjunction with the pressure values, to set hydraulic grade line boundary conditions for the distribution system model.

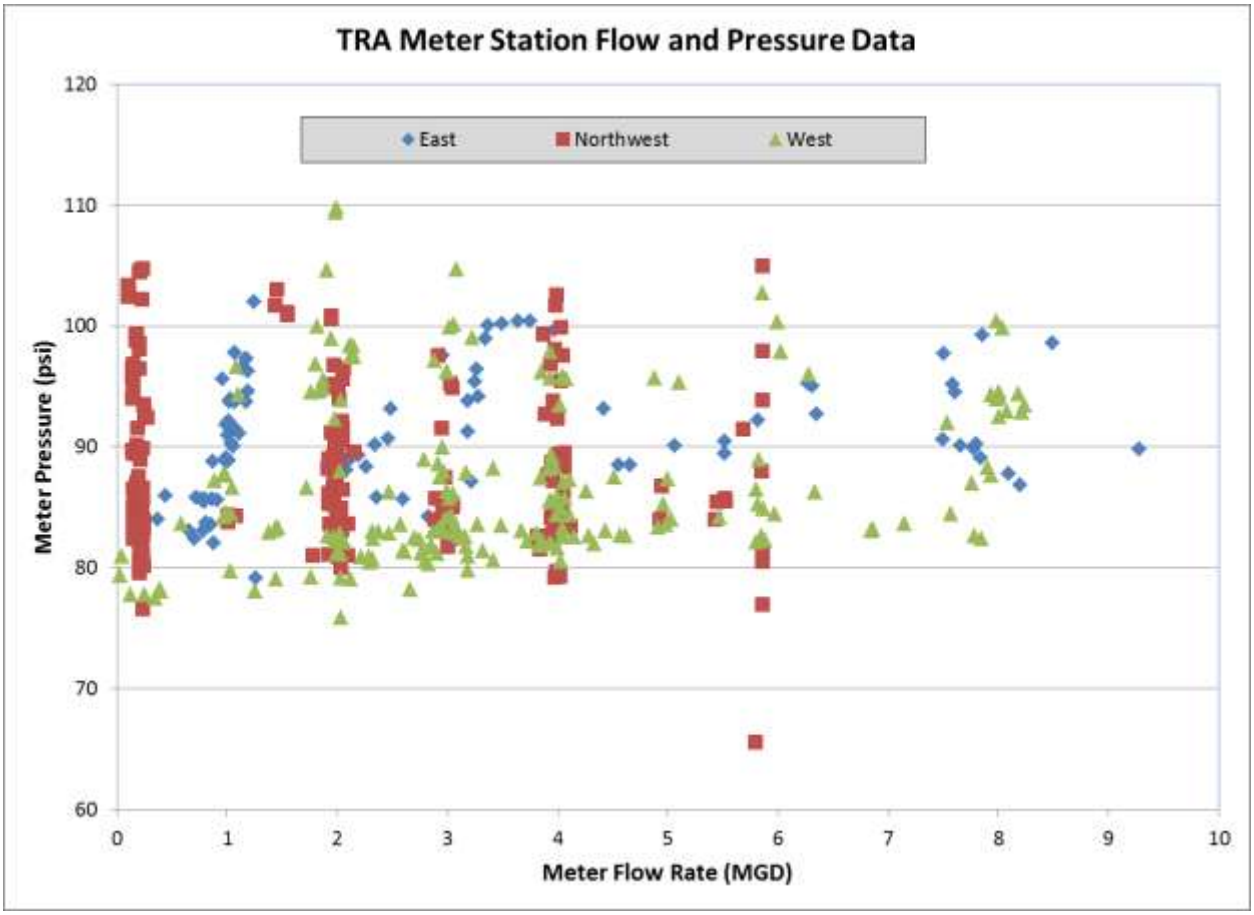


Figure 6: TRA Meter Station Flow and Pressure Data

6.4 Junction Elevations

The elevations of the junctions in the model were specified using the TRex (Terrain Extractor) tool in WaterGEMS. An elevation contour shapefile created using USGS 5-foot contours was imported via the TRex tool, and the tool automatically assigns elevations throughout the model. The model elevations were spot checked by comparing to the junction elevation in the previous model and surveyed elevations from the manhole survey for the sanitary sewer model to verify the elevation import process. For the elevation data set, the elevation accuracy is generally in the range of ± 5.0 feet (2.2 psi). This level of accuracy is generally acceptable for distribution system modeling related to system master planning.

6.5 LD Lockett PS

The ground storage tank (GST) and pumps at the LD Lockett PS were incorporated into the model using data provided by the City. The following parameters were used to describe the LD Lockett GST:

- Elevation (Minimum): 658.00 ft
- Elevation (Maximum): 699.00 ft
- Diameter: 150.00 ft

There are eight pumps at the LD Lockett PS. Two pumps were included in the model with each of the following four designations:

- High Plane Average Flow
- High Plane Peak Flow
- Low Plane Average Flow
- Low Plane Peak Flow

The pump curves for each set of pumps were defined in the model using multiple point curves.

6.6 Overland Trail PS

The ground storage tank and pumps at the Overland Trail PS were included in the base model set-up. However, these elements were made inactive, due to the current state of the pump station and GST. Additionally, it is recommended that this facility be decommissioned. Refer to Appendix B for additional information regarding the recommendation to decommission the Overland Trail PS.

6.7 Pressure Zone Isolation Valves

Isolation valves were included in the distribution system model in order to allow for separation of the high and low planes within the system. The locations of the isolation valves were developed from maps provided by the City and through discussions with City personnel. Each isolation valve was assigned to a pipe, and the status of the isolation valves was set to “Closed” in order to prevent flow between the zones.

6.8 Demands

The overall model demands (or projected water usages) outlined in Section 3 were used to create model demand alternatives in the hydraulic model. A total of four demand alternatives were created:

- 2014 Average Day Demand
- 2014 Maximum Day Demand
- 2034 Average Day Demand
- 2034 Maximum Day Demand

The procedure for spatial distribution of the demand across the City’s network is described in Section 6.8.1, and the diurnal curves utilized to develop the temporal variation in demands throughout the day are described in Section 6.8.2.

6.8.1 Spatial Variability of Demand

The billing record data from 2013 was processed to create the following data sets for each meter:

- Minimum Day Period: average of January, February, March, and December usage
- Average Period: average of May and June usage
- Maximum Day Period: average of July, August, and September usage

The meter locations were geocoded based on parcel and road centerline shapefiles provided by the City. This allowed meter data to be assigned to the correct spatial location within the City.

The Minimum Day Period usage was used as the base demand within the model. This approach was based on the assumption that water usage for lawn and landscape irrigation is limited during this period, so the demands associated with the Minimum Day should be representative of the base demands throughout the year. Separate base demand shapefiles were created for the high and low planes.

The difference between the Average Day and Minimum Day usage was used to develop the spatially variable demands associated with water consumption in excess of the base demand during average demand periods. Separate demand shapefiles were created for the high and low planes.

The difference between the Maximum Day and Minimum Day usage was used to develop the spatially variable demands associated with water consumption in excess of the base demand during peak demand periods. In order to account for variations in demands due to the three-zone water restrictions, shapefiles were created for each of the following groups:

- High plane meters
- Low plane meters in Zone A
- Low plane meters in Zone B
- Low plane meters in Zone C

Following the decision to decommission the Overland Trail GST and PS facility (detailed in Appendix B), these shapefiles were combined into only high plane meters and low plane meters.

6.8.2 Diurnal Demand Curves

Diurnal curves are used to capture the temporal variation in demands throughout a daily period. Tank levels (GST and EST) and TRA meter flow rates were provided for the first day of each month for the period of May 2013 through May 2014, and this data was analyzed to estimate the system-wide usage on an hourly basis. Specifically, a volumetric balance was applied to the in-system storage and incoming flows through the TRA metering stations to calculate change in storage and usage each hour. Multiple days were used to create representative low, average, and high demand curves:

- Low (base) demand: December 1, January 1, and February 1
- Average demand: May 1, June 1, October 1, and November 1
- High demand: August 1, September 1

The system wide diurnal curves showing hourly usage in MGD versus hours of the day are shown in Figure 7.

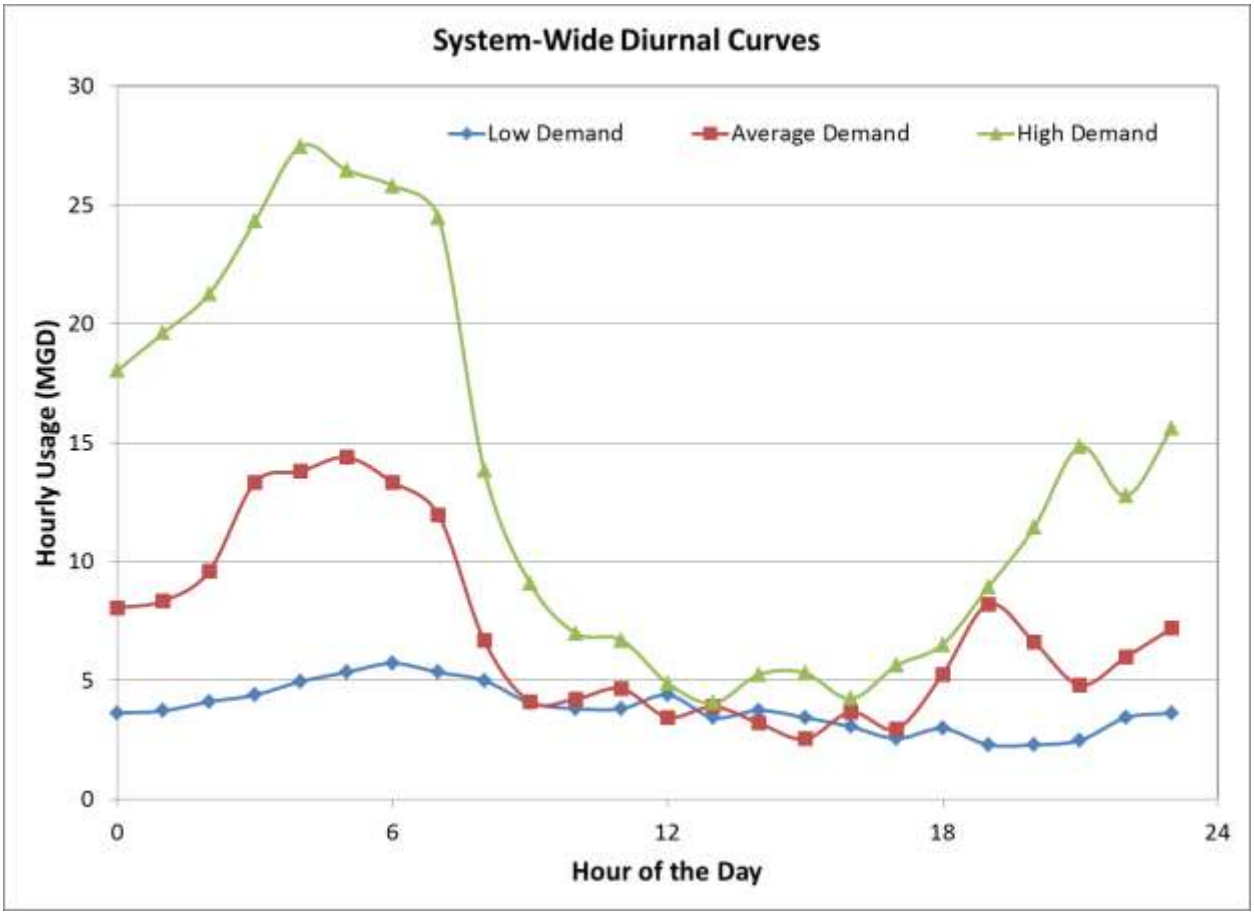


Figure 7: System-Wide Diurnal Curves

The curves clearly show early morning and late night usage signatures which are likely a result of irrigation demands within the system. For the high demand days (August 1 and September 1), the average usage was 13.48 MGD and the peak hour usage was 27.47 MGD. The ratio of the peak to the average for this curve is approximately 2.04, meaning the peak hour usage for this curves is 2.04 times the average usage.

The data from the three curves above was combined to create two diurnal curves for application to the model:

- Base diurnal curve
- Excess diurnal curve

Base Diurnal Curve. The low demand curve was normalized by dividing by the average usage during the day to yield the base diurnal curve. This diurnal curve is applied to the base model demands.

Excess Diurnal Curve. An excess diurnal curve was developed by normalizing the values of the average and high demand curves above the low demand curve. The excess diurnal curve is applied to excess model demands, where the excess demands are the difference between the total and base demands for the alternative.

The two diurnal curves are shown in the following Figure 8.

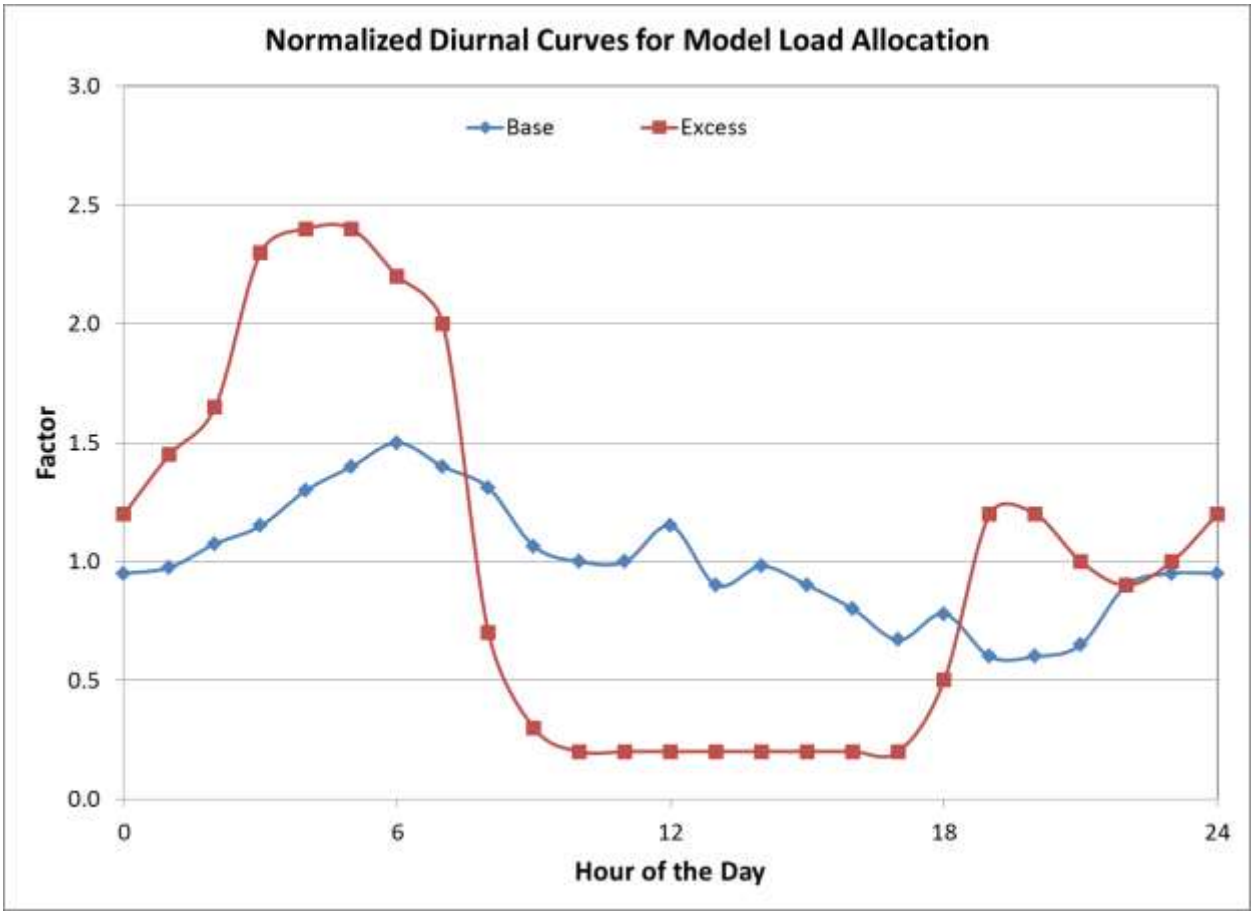


Figure 8: Normalized Diurnal Curves for Model Load Allocation

6.9 Model Calibration

6.9.1 Pressure Readings

City personnel collected field data for model calibration on June 30, 2014. The data collection consisted of static pressures at hydrants as well as flow and pressure information for flow tests. Additionally, LD Lockett pump operations, elevated storage tank levels, and pressures at TRA meter stations were provided for the times of the field tests to allow accurate boundary conditions to be specified in the hydraulic model. Pressure values collected in the field were converted to hydraulic grade lines by combining the pressure data with elevation data for the hydrant locations.

Standards have not been set in the United State regarding calibration of hydraulic models. However, general guidelines are presented in different texts, including Chapter 7 of “Advanced Water Distribution Modeling and Management” by Walski et al. The guidelines for master planning for systems consisting of 24-inch pipes and smaller are as follows:

- During fire flow tests, the model should predict the HGL within 5-10 feet at calibration data points.

Additionally, it is a generally accepted practice that the degree of accuracy of the calibration is directly related to the precision of the information used in the calculations. Pressure readings are assumed to be known to within ± 1 psi (2.3 feet), and the topographic data set used to set elevations in the model is assumed to have an error of ± 5.0 feet. Therefore, the accuracy of the elevation and pressure data, which sets the lower limit of the practical accuracy of the model, is a total of 7.3 feet.

The static results are presented graphically in Figure 9. Each marker on the graph represents a calibration point pair, which consists of a HGL from the field measurements and a HGL from the model at the same spatial location. There are three dashed lines. The dark dashed line in the middle represents the 1:1 line. Markers falling on the 1:1 line have the same HGL value for the field measurement and model results. Markers above and to the left of the 1:1 line have higher model HGL values than field measured HGL values, while markers below and to the right of the 1:1 line have lower model HGL values than field measured HGL values. The two lighter dashed lines represent ± 7.3 feet, which is the lower limit of accuracy, as discussed in the paragraph above.

The majority of the results within the low plane are within ±7.3 feet. There is no systematic error within the low plane, and it is likely that the majority of the error is due to errors in model elevations and local differences in pipe characteristics and demands between the model and real physical system. However, despite the differences between field data and model results, the hydraulic model provides accurate overall trends in hydraulic grades throughout the low plane system and the model is calibrated appropriately for master planning purposes.

In general, the hydraulic grade line values calculated based on the pressure measurements in the high plane were higher than the values that were predicted by the hydraulic model. Specifically, based on the SCADA data provided for the field data collection period, the McPherson EST set the hydraulic grade line for the high plane at values around 820 feet. The pumps at LD Lockett did not run during the data collection period, so the hydraulic grade line at the McPherson EST should have been the maximum hydraulic grade in the high plane. Therefore, the hydraulic grade line of the McPherson EST was calibrated by 8 feet in order to obtain the calibration shown in Figure 9. It is recommended that the level transducer in the McPherson EST and the LD Lockett pump station be checked for calibration to ensure accurate water level and pressure measurements are obtained.

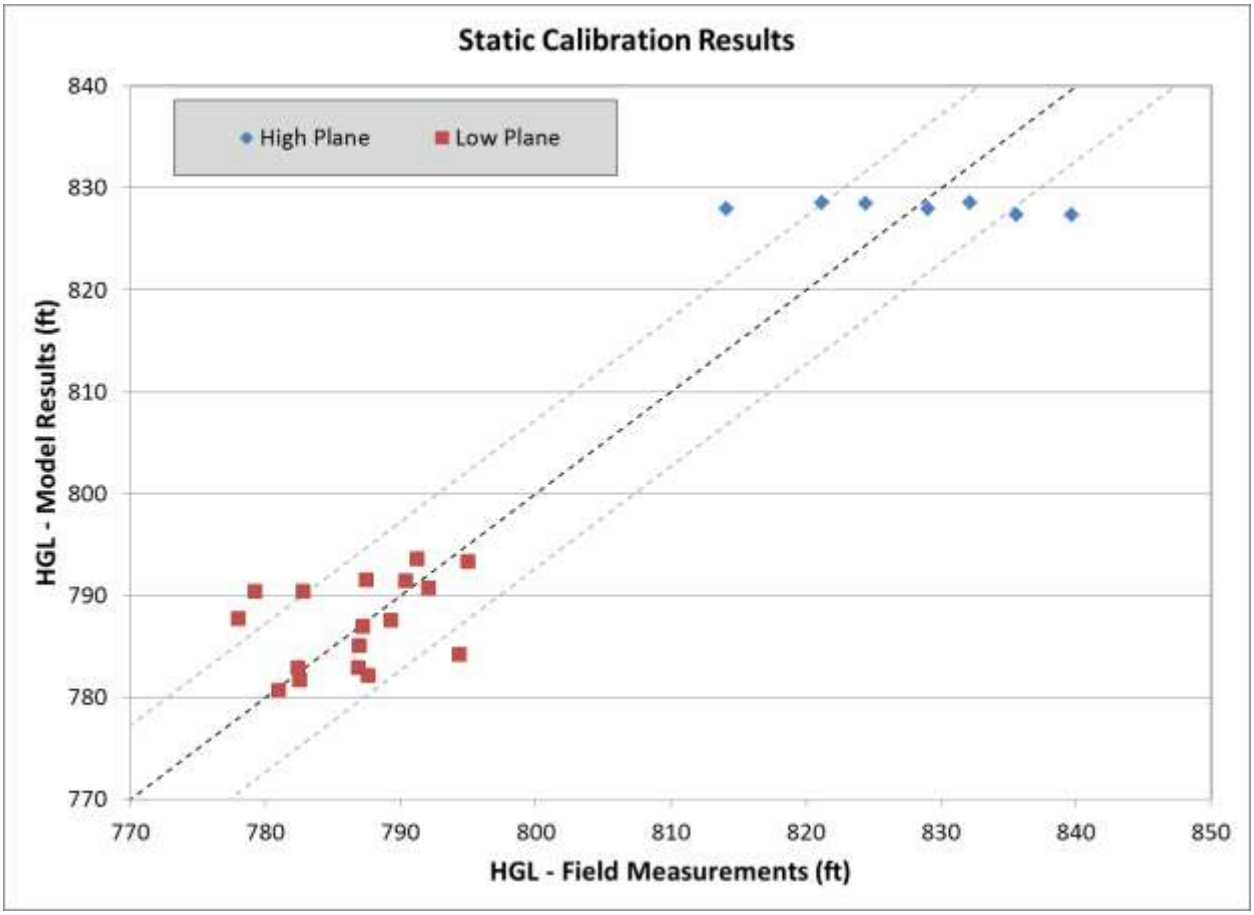


Figure 9: Static Calibration Results

6.9.2 Flow Tests

City personnel also conducted six (6) flow tests. The flow and pressure data collected allows the change in pressure experienced in the field to be compared to the change in pressure calculated within the hydraulic model. The flow test calibration results are presented in Figure 10. The six markers on Figure 10 correspond to each of the six flow tests. Each marker represents a calibration point pair, which consists of a pressure drop from the field measurements and a pressure drop from the model at the same spatial location. The dashed line represents the 1:1 line, and markers falling on the 1:1 line have the same pressure drop for the field measurements and model results. Markers above and to the left of the 1:1 line have higher pressure drops in the model than for field measurements, while markers below and to the right of the 1:1 line have lower pressure drops in the model than for field measurements.

The absolute value of the error for the pressure drop during the hydrant flow tests ranged from 0.5 to 4.3 psi, which corresponds to a range of 1 to 10 feet. In general, the pressure drops are slightly higher in the model than for the field data. There was a single test in the low plane which showed less pressure drop in the model than in the field. The hydrants used for this flow test were in the southwest portion of the system near the intersection of Colleyville Blvd and Greenbriar Lane. Pipe roughness values for pipes crossing Little Bear Creek were increased (i.e., the Hazen-Williams C values were decreased) in order to

increase frictional losses to this location. It is possible that there is a closed valve within the system or significant siltation of pipes crossing Little Bear Creek that restrict flow beyond the level predicted by the model. The difference in change in pressure between the model and field is approximately 2 psi, which is a reasonable level of error for a full system hydraulic model.

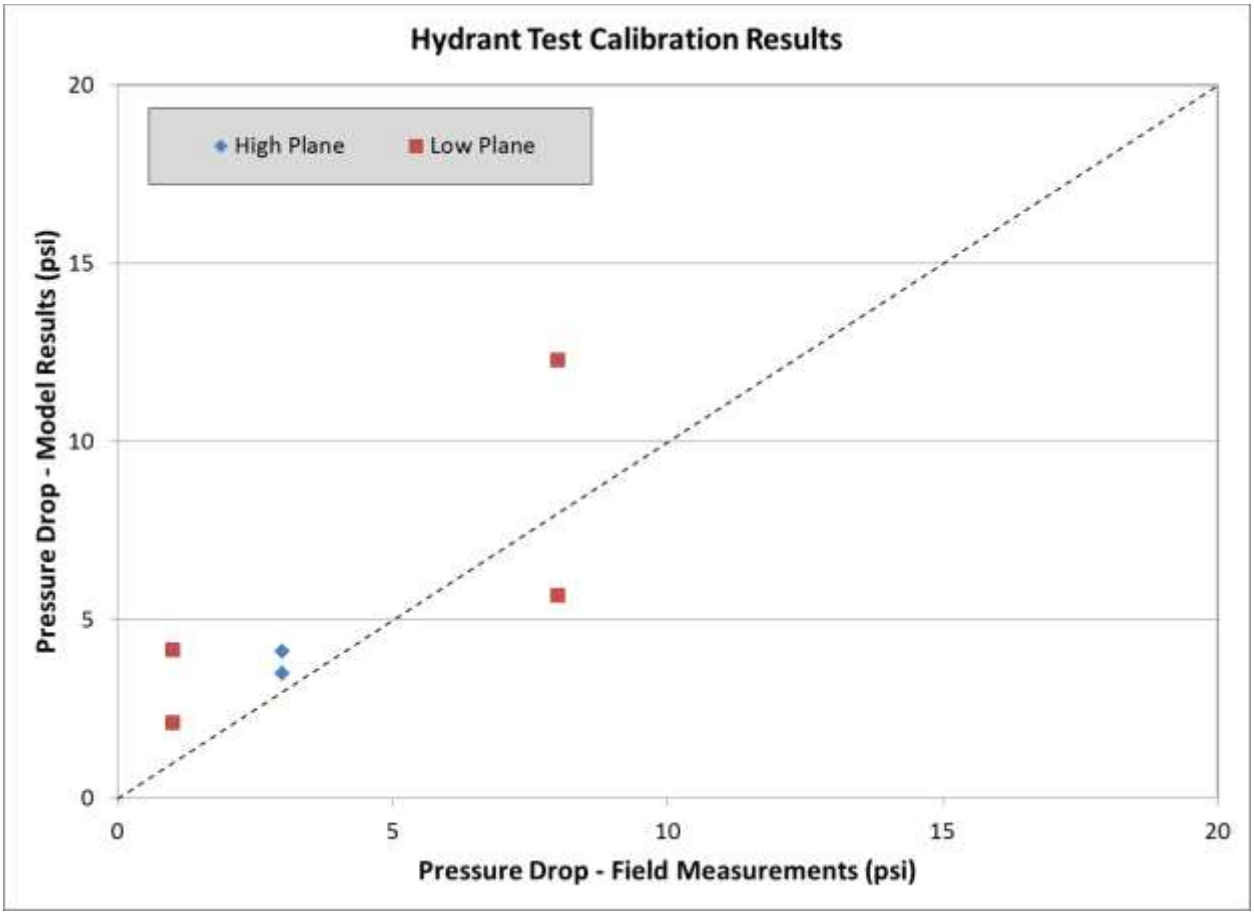


Figure 10: Hydrant Test Calibration Results

7.0 Water Model Results

Model scenarios were created for current and future system conditions. The Current model included existing system elements, as well as improvements which are already included in the City's existing capital plans. Current model scenarios do not include the Overland Trail GST and PS because it is recommended that facility be decommissioned. Current model simulations are performed under both average and maximum day demand scenarios using 2014 demand values.

Infrastructure improvements were identified through discussions with City staff, evaluation of facilities, and analysis of model results. Improvements to address issues for the Current model scenarios were incorporated into the hydraulic model to assess the utility of the proposed projects.

The Future model is based on anticipated demand conditions for the 20-year horizon (2034). The Future model includes the improvements incorporated into the Current model to address system deficiencies. Additional improvements to address issues for the Future model were incorporated into the hydraulic model to assess the utility of the proposed projects.

7.1 Current Water Model Results

7.1.1 Average Day Demands

The current system was analyzed under average day demands. The total average day demand was divided into base and excess components for application of diurnal curves, and applied using the previously detailed spatial variability. These amounts are shown in the following Table 18.

Table 18: Current Water Model Average Day Demands

Demand Condition	Base Demand (MGD)	Excess Demand (MGD)
Average Day	2.879	4.152

7.1.1.1 HGL and Pressure (Average Day)

The HGL in the system is strongly influenced by the HGL at boundary points (e.g., water levels in ESTs, HGLs at TRA connection points). Additionally, the amount of flow within the system can have a major impact on HGL level. In general, under average demand conditions, the HGL does not vary as significantly across the system because head loss in water mains is relatively minor, as compared to maximum day or fire flow conditions. The HGL varies throughout the day as boundary conditions change (e.g., tank levels rise or fall, pump status changes, TRA connection values vary). Minimum and maximum HGL values within each of the pressure planes are shown in Table 19.

Table 19: Current Water Model Average Day HGL Results

Parameter	Statistic	High Plane	Low Plane
Minimum HGL (ft)	Minimum	796.3	751.8
	Mean	817.5	761.1
	Maximum	820.0	770.0
Maximum HGL (ft)	Minimum	830.0	784.5
	Mean	837.6	789.6
	Maximum	857.7	804.1

As the results in Table 19 show, there is a significant HGL difference between the high and low planes. The HGL is generally about 50 feet higher in the high plane than the low plane, which is expected given the operating ranges of the ESTs in each zone. The HWL for the McPherson EST is 840.0 feet, which is 50 feet higher than the HWL for the Bransford and Hall-Johnson EST in the low plane.

Pressure at junctions in the model is calculated using the HGL and elevation. Statistics for the minimum and maximum pressure results within each of the pressure planes are shown in Table 20.

Table 20: Current Water Model Average Day Pressure Statistic Results

Parameter	Statistic	High Plane	Low Plane
Minimum Pressure (psi)	Minimum	42	34
	Mean	66	63
	Maximum	87	90
Maximum Pressure (psi)	Minimum	48	42
	Mean	75	75
	Maximum	99	104

See Exhibit 2 for modeled pressures throughout the system. The distribution of pressures throughout each pressure plane is similar. The mean minimum pressure is around 65 psi in each plane, and the mean maximum pressure is approximately 75 psi in each plane. The noteworthy difference between the results for the two pressure planes is that the minimum pressure in the low plane drops below our design criteria minimum of 35 psi during the simulation.

The minimum pressure (34 psi) occurs in the high-elevation area along Woodcrest Court south of Brookridge Drive. The model results are sensitive to the settings used to define the operational parameters for the system. Specifically, the pressures of less than 35 psi in the average day simulation could readily be alleviated through an adjustment to the tank level controls for the system. Refer to Section 8 for additional information on system operations.

7.1.1.2 Velocity (Average Day)

The velocities in the model range from 0.0 to 11.8 fps for the average day scenario. The maximum velocity in each pipe during the model is shown in Exhibit 3. The majority of the high velocities (greater than 5 fps) are associated with the main supply lines and LD Lockett PS suction and discharge lines for the pumps. The pipes with velocities greater than 10 feet per second, which is identified as the maximum velocity criteria, are located at the high plane discharge header coupling for the LD Lockett PS and the 6-inch line along Summertree Lane between Pool Road and Maple Lane. The piping associated with the LD Lockett PS is assumed to be properly designed for existing pump station infrastructure, so improvements are not recommended to address high velocities in these lines. Replacement of the 6-inch line along Summertree Lane with a 10-inch line is recommended in the capital plan to address concerns with excessive velocities scouring the line.

7.1.1.3 Tower Levels (Average Day)

Variations in tank levels in the McPherson, Bransford, and Hall-Johnson EST over the three-day simulation are shown in Figures 11 and 12.

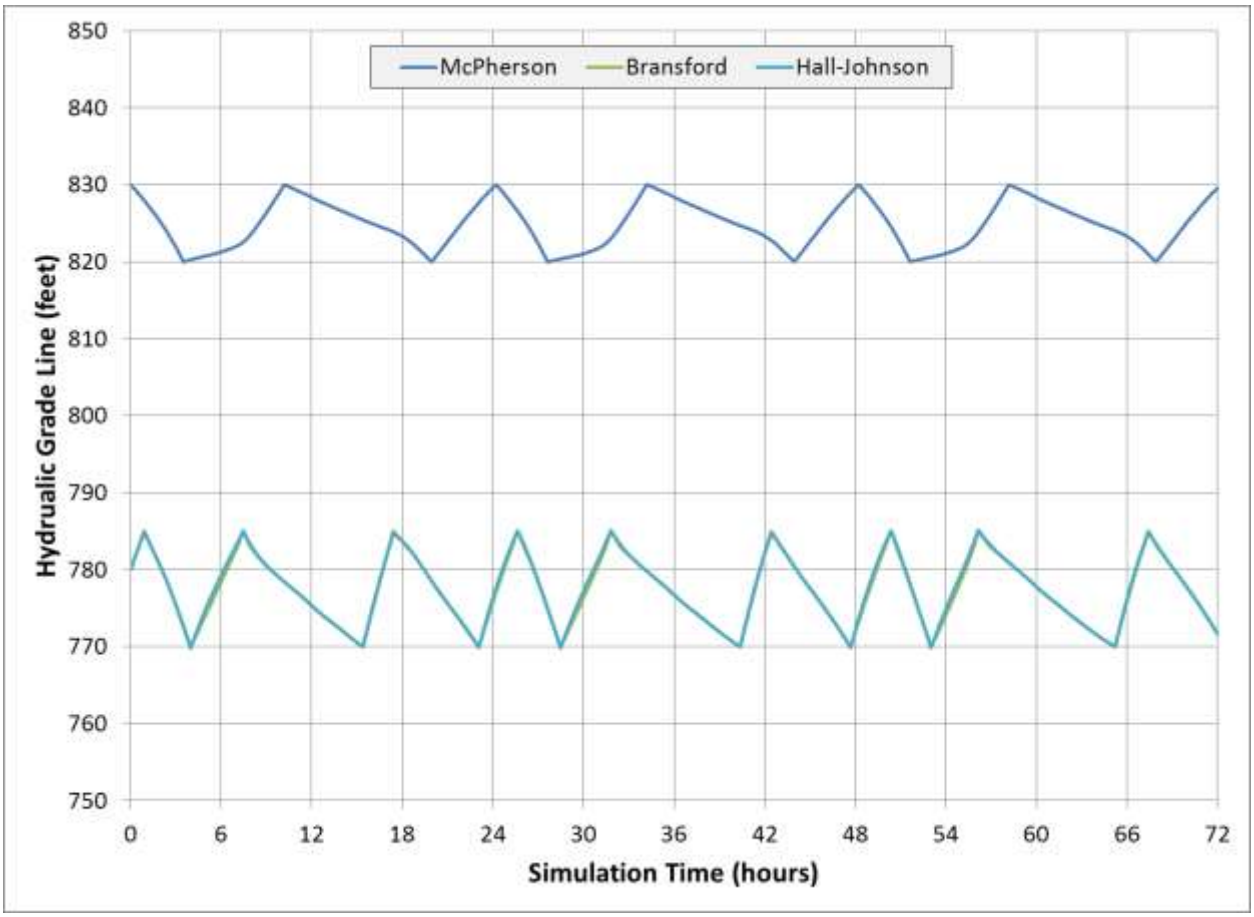


Figure 11: HGL versus Time for Tank Levels on Average Day

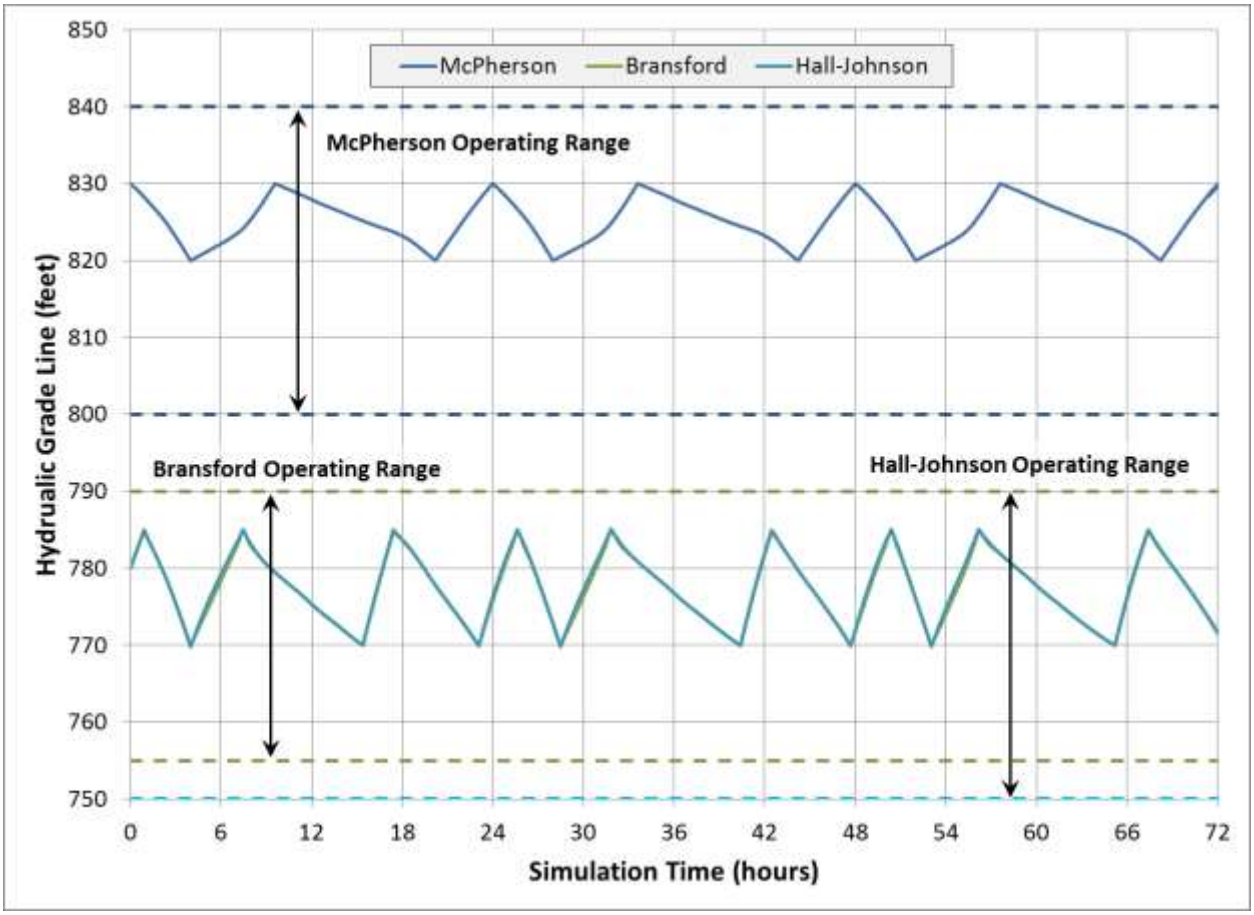


Figure 12: Current Water Model Average Day HGL for Existing ESTs

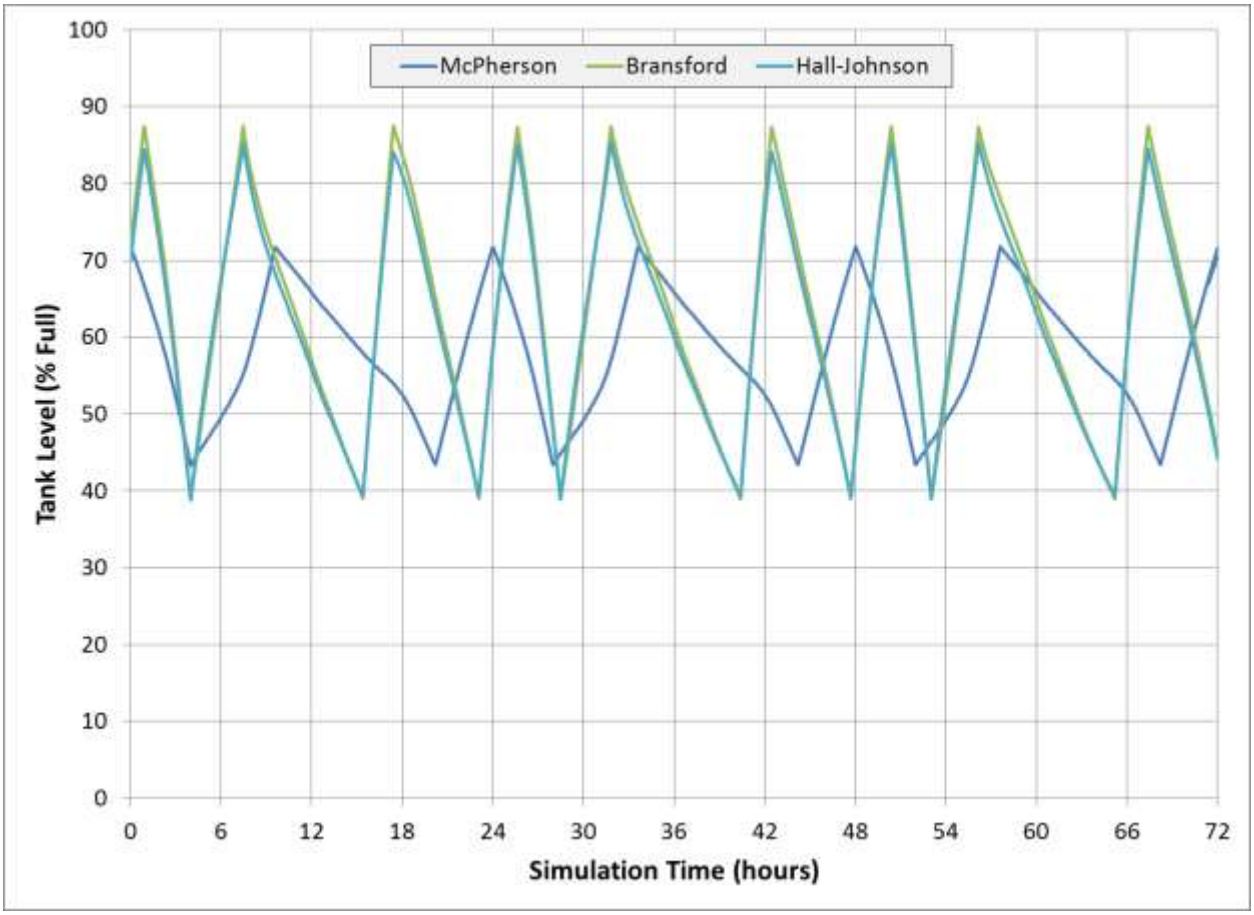


Figure 13: Current Water Model Average Day Tank Level Variations for Existing ESTs

The HGL for the McPherson EST varies between 820 and 830 feet, which corresponds to a range of approximately 43 to 72 percent full.

The HGLs for the Bransford and Hall-Johnson EST vary between 770 and 785 feet, which correspond to a range of approximately 39 to 86 percent full. During the average day simulation, the Bransford and Hall-Johnson EST levels remain balanced throughout the simulation. One contributing factor to the balanced tank levels in the Bransford and Hall-Johnson EST is that the low plane pumps at LD Lockett are not utilized during the average day simulation. The low-plane discharge line from LD Lockett enters the distribution system at the Bransford EST. Therefore, LD Lockett preferentially fills the Bransford EST. Adequate capacity of the lines between the Bransford and Hall-Johnson EST is required to convey water from the Bransford EST east to the Hall-Johnson EST to maintain similar water levels when the low-plane pumps at LD Lockett are operating.

7.1.1.4 Water Age

Water age modeling was conducted to assess the water quality in the distribution system. Water quality analysis is concerned with levels of residual disinfection throughout the distribution system, as well as formation of disinfection by-products (DBPs). TRA utilized chloramines for residual disinfection, and the

City continues the use of chloramines in their system. The City uses equipment at the LD Lockett GST and PS to maintain adequate chloramine levels entering the distribution system through that facility. Residual disinfectant and DBP levels are not specifically assessed in this study. Rather, water age is used as a surrogate for water quality within the system. In general, higher water age is associated with lower residual disinfectant levels and higher DBP levels.

For this evaluation, the following classifications were used to assess water age in the system.

Table 21: Water Age Classifications

Water Age (days)	Water Quality Level
0 – 2	Good
2 – 3	Fair
3 – 5	Concerning
5 +	Extremely Concerning

The water quality classifications listed in Table 21 are general guidelines used in this study to provide a means of identifying areas of concern. System water quality is related to additional factors, including temperature, pH, combined chlorine residual, and free ammonia.

The water age calculations provide the water age after water enters the Colleyville system through any of the four TRA meter stations. The water age is set to 0.0 days for water at each of the meter stations. As mentioned above, the City utilizes equipment at LD Lockett to maintain chloramine levels. Therefore, it is assumed for the purposes of this study that the water age will be reset at LD Lockett and the water age will be 0.0 days for water leaving the GST. For all water age modeling, all tanks in the system were simulated using a completely mixed model. The maximum water age for each junction within the system is shown in Exhibit 4.

The water age in the low plane is generally good. The majority of the main lines within the low plane have maximum water age values between 0 and 2 days. The low water age can be attributed to good turnover in the tanks within the pressure plane. Maximizing tank turnover minimizes hydraulic detention time in the EST. This, in turn, limits the maximum water age in portions of the system that receive water out of the EST.

The water age in the high plane is slightly higher than the water age in the low plane. The lowest maximum water age values in the high plane are in the southern areas of the pressure plane, closest to the LD Lockett PS. The areas around the McPherson EST have maximum age values in the 2-3 day range, while some areas in the northern section of the high plane have water age in the 3-5 day range. The high plane has a larger ratio of elevated storage volume to system demands. While approximately 15% of the demand is in the high plane, the high plane contains approximately 29% of the elevated storage for the system. While the elevated storage volume in the high plane is detrimental to water quality to a degree, the elevated storage is essential for providing adequate service during emergency conditions such as fires or disruptions to operations at LD Lockett GST and PS.

The main contributing factor to the higher water age in the high plane (as compared to the low plane) is the high plane is fed through LD Lockett, whereas the low plane can get water directly through the TRA meter stations. The water levels and water age in the LD Lockett GST are shown in Figure 14.

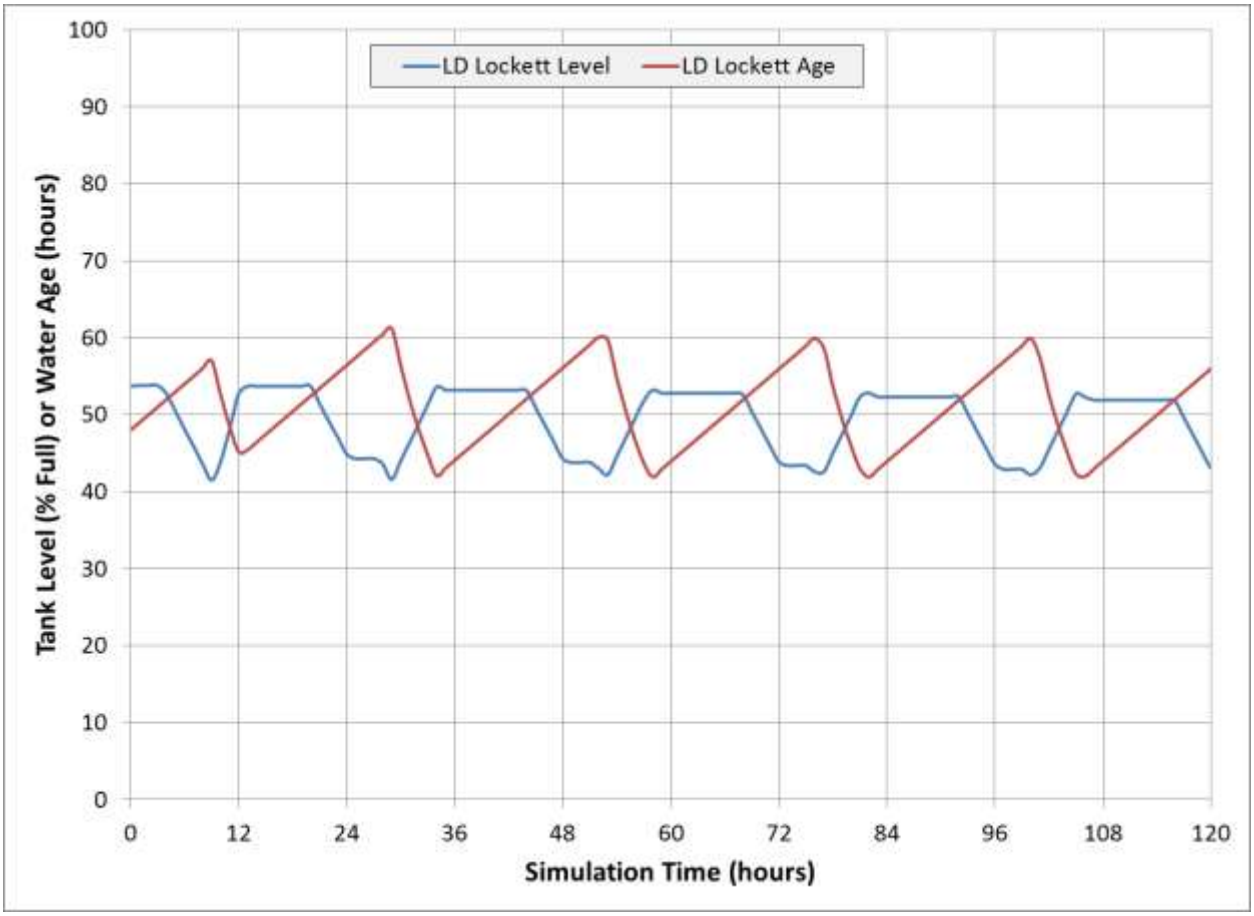


Figure 14: Current Water Model Average Day LD Lockett GST Water Levels & Water Age

The LD Lockett GST is fed by TRA through the Northwest TRA Meter Station. Water leaves the LD Lockett GST and is conveyed by pumps to either the high or low planes within the Colleyville system. There are four general situations that can occur during normal operations:

1. Inflow from TRA; pumps off
    - This situation only occurs once during the simulation, 11 to 12 hours into the simulation. Under this situation, the tank is filling, as shown in Figure 14. The water in the tank continues to age, but the inflow of fresh water is a large enough percentage of the total volume to reduce the water age.
2. Inflow from TRA; pumps on
    - This situation occurs repeatedly during the simulation, including at times 29 to 33 hours, 53 to 57 hours, and 77 to 81 hours. Under this situation, the tank is filling because the flow rate through the TRA Meter Station is higher than the pumped flow rate. Figure 14

shows the tank level increases during these periods. The water age decreases during these periods because there is a significant inflow of fresh water.

3. No flow from TRA; pumps off
    - This situation occurs repeatedly during the simulation, including at times 13 to 19 hours, 35 to 43 hours, and 59 to 67 hours. Under this situation, the tank level is constant because there are no inflows or outflows. The water age increases linearly during these periods because the water is detained in the GST. For example, at time 13 hours, the average water age is 45.4 hours. At time 19 hours, the average water age is 51.4 hours.
4. No flow from TRA; pumps on
    - This situation occurs repeatedly during the simulation, including at times 20 to 24 hours, 44 to 48 hours, and 68 to 72 hours. Under this situation, the tank is draining because no water is entering the GST through the meter station from TRA. Figure 14 shows the tank level decreases during these periods. The water age increases linearly during these periods because there is no inflow of fresh water. For example, at time 20 hours, the average water age is 52.4 hours. At time 24 hours, the average water age is 56.4 hours.

In summary, the water age in LD Lockett decreases when new water from the Northwest TRA Meter Station is entering the GST. The change in water age during these times is a function of the flow rates into and out of the GST, as well as the age and volume within the GST. During periods when there is no inflow to the GST from TRA, the average water age increases linearly with time.

The water age in LD Lockett varies between 41.5 and 61.2 hours (1.73 and 2.55 days). Therefore, the maximum water age is 1.73 days without accounting for any aging in the TRA supply system or within the actual high plane system. Additionally, water on the northern side of the high plane is also subjected to additional hydraulic detention time in the McPherson EST (similar to the detention of low plane water within the Bransford and Hall-Johnson EST).

Water age in the high plane can be minimized by maintaining lower levels and higher turnover rates for the LD Lockett GST. However, changes to operations to address water age have potential implications on resiliency for emergency conditions. Specifically, maintaining relatively low levels in the LD Lockett GST and the ESTs in the system to minimize system water age could leave Colleyville without adequate emergency storage in the event of a shutdown of supply service from TRA.

As shown in Exhibit 4, there are locations throughout both pressure planes with high water age. The majority of these locations are junctions in the model with little to no demand. The automated spatial demand allocation procedure did not guarantee demands at each junction. Junctions with small demand that are located at the end of dead-end pipes will have high water age values. As an example of potential issues with water quality on dead end lines with low demands, consider a 6-inch diameter dead end line with a length of 400 feet. The volume of the line is approximately 587 gallons. If a single service is located at the end of this 400 LF main and the average usage for the customer is 100 gpd, the average hydraulic detention time will be 5.87 days in that portion of line. Water with an age of 2 days at the start of the dead end line will age from good to fair quality to extremely concerning quality because the demand on the line is insufficient. Similar conditions can also occur on looped lines if there is not a significant hydraulic gradient around the loop that forces water to circulate and the demands on the looped lines are low. Isolated model junctions with high water age may be indicative of locations with water age issues, or

demands may have been placed on nearby junctions during the demand allocation process. However, these locations warrant additional attention to ensure water quality issues are addressed.

7.1.1.5 Available Fire Flow

The available fire flow was calculated for each junction in the model based on a steady-state scenario with the pumps at LD Lockett off and the water levels in the ESTs in the system at half full. This operational set-up for running the fire flow simulations is based on guidance in the Third Edition of the AWWA M32 Manual titled Computer Modeling of Water Distribution Systems. Flow was allowed to enter the system through the TRA meter stations during the simulation. The available fire flow at each junction in the model (supply lines excluded) is shown in Exhibit 5.

There are potential issues with meeting the desired 1,500-gpm available fire flow at select points throughout the system. Isolated junctions that do not meet the criterion are frequently at the end of 6-inch, dead-end lines. There are three areas where the 1,500-gpm fire flow criterion is not met on a large scale, and all three of these areas occur in the low plane. These areas include:

- The high-elevation area on the southwest edge of the system is an area with widespread issues meeting available fire flow targets, both for residential areas (1,500 gpm) and commercial areas (3,000 gpm).
- The area south of Cotton Belt Trail and west of Bransford Road has significant issues meeting the 1,500-gpm residential fire flow criterion, despite being in close proximity to the Bransford EST and the 20-inch water line along Bransford Road.
- The north and northwest areas of the low plane (generally north of John McCain Road and west of Trianon Court) do not meet the 1,500-gpm residential fire flow criterion. These areas are located a significant distance from the elevated storage within the low plane, and there is not a well-connected skeleton of large diameter lines to convey water from the TRA meter stations and ESTs across the low plane.

System improvements have been recommended to address issues with available fire flow.

7.1.2 Maximum Day Demands

The current system was then analyzed under maximum day demands. The total maximum day demand was divided into base and excess components for application of diurnal curves. The breakdown of these demands is shown on the following Table 22.

Table 22: Current Water Model Maximum Day Demands

Demand Condition	Base Demand (MGD)	Excess Demand (MGD)
Maximum Day	7.000	12.423

7.1.2.1 HGL and Pressure (Maximum Day)

The HGL (and pressure) within the system will usually vary more significantly on maximum day than during average day conditions, due to increased flow and head loss in water mains throughout the

system. Additionally, storage tank levels may be depleted more significantly during maximum day conditions than during average day conditions when supply infrastructure is able to better keep pace with demands. Minimum and maximum HGL values within each of the pressure planes are shown in Table 23.

Table 23: Current Water Model Maximum Day HGL Results

Parameter	Statistic	High Plane	Low Plane
Minimum HGL (ft)	Minimum	824.3	687.8
	Mean	825.4	733.2
	Maximum	827.1	774.8
Maximum HGL (ft)	Minimum	839.6	785.4
	Mean	853.2	790.9
	Maximum	895.2	819.1

As the results in Table 23 show, there is a significant difference between the high and low pressure planes during maximum day conditions. In the high plane, the difference between the mean values for the minimum and maximum HGLs is about 28 feet (12 psi), whereas the variation in mean values is about 58 feet (25 psi) in the low plane. The difference between the lowest values for the minimum and maximum HGL within each pressure plane is even larger, with a difference of only 15.3 feet in the high plane (839.6 vs. 824.3) and a difference of 97.6 feet in the low plane (785.4 vs 687.8). The low plane covers more area than the high plane, and the overall conveyance capacity of the pipe network is not as robust, which causes the large variation in HGL during peak demand conditions.

Pressure at junctions in the model is calculated using the HGL and elevation. Statistics for the minimum and maximum pressures within each of the pressure planes are shown in Table 24.

Table 24: Current Water Model Maximum Day Pressure Statistic Results

Parameter	Statistic	High Plane	Low Plane
Minimum Pressure (psi)	Minimum	46	15
	Mean	69	51
	Maximum	93	78
Maximum Pressure (psi)	Minimum	52	45
	Mean	81	76
	Maximum	114	104

The high plane performs very well during maximum day conditions, and the minimum pressure of 46 psi is well above the TCEQ minimum pressure of 35 psi for normal operating conditions. The high plane is relatively small and has a solid network of large diameter lines. Additionally, the McPherson EST is centrally located, and it provides about 28.6% of the total elevated storage for the Colleyville system, while the demands in the high plane only constitute about 15% of the total demands.

The low plane experiences widespread issues with maintaining adequate pressure during maximum day demand conditions, as shown in Exhibit 6. The most notable areas are the high-elevation area along

Woodcrest Court south of Brookridge Drive. The model predicts minimum pressures of less than 25 psi in this area. Additionally, minimum pressures less than 35 psi are widespread in the Monticello Parkway area north of John McCain Road and in the area around Peter's Path and David Lane.

7.1.2.2 Velocity (Maximum Day)

The velocities in the model range from 0.0 to 18.7 fps for the average day scenario. The maximum velocity in each pipe during the model is shown in Exhibit 7. The majority of the high velocities (greater than 5 fps) are associated with the main supply lines and pump stations. Specifically, the supply line for LD Lockett and the supply line from the East TRA Meter Station towards the Hall-Johnson EST show relatively high velocities. An example of a distribution line with high velocities is the 12-inch line along Hall-Johnson east of the Hall-Johnson EST. The model predicts velocities in excess of 8 fps for portions of this pipe.

7.1.2.3 Tower Levels (Maximum Day)

Variations in tank levels in the McPherson, Bransford, and Hall-Johnson EST over the three-day simulation are shown in Figure 15.

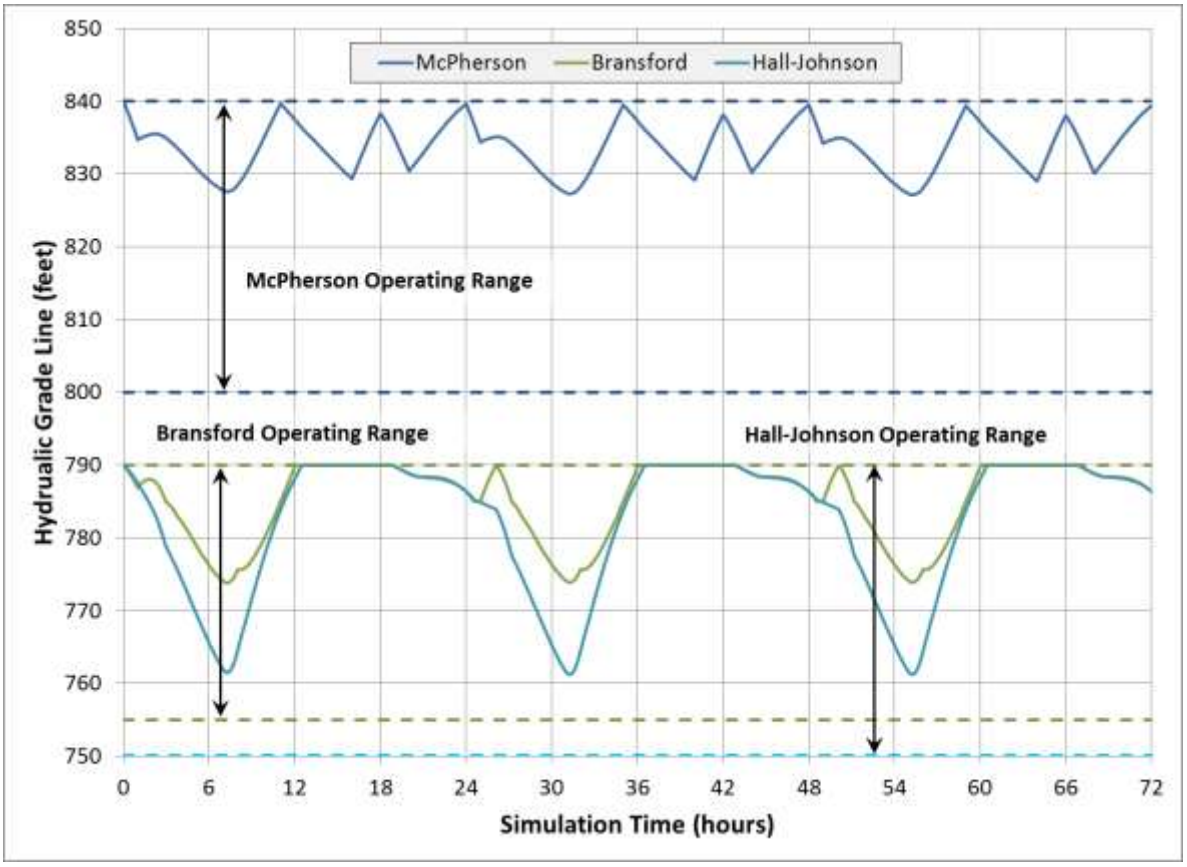


Figure 15: Current Water Model Maximum Day HGL for Existing ESTs

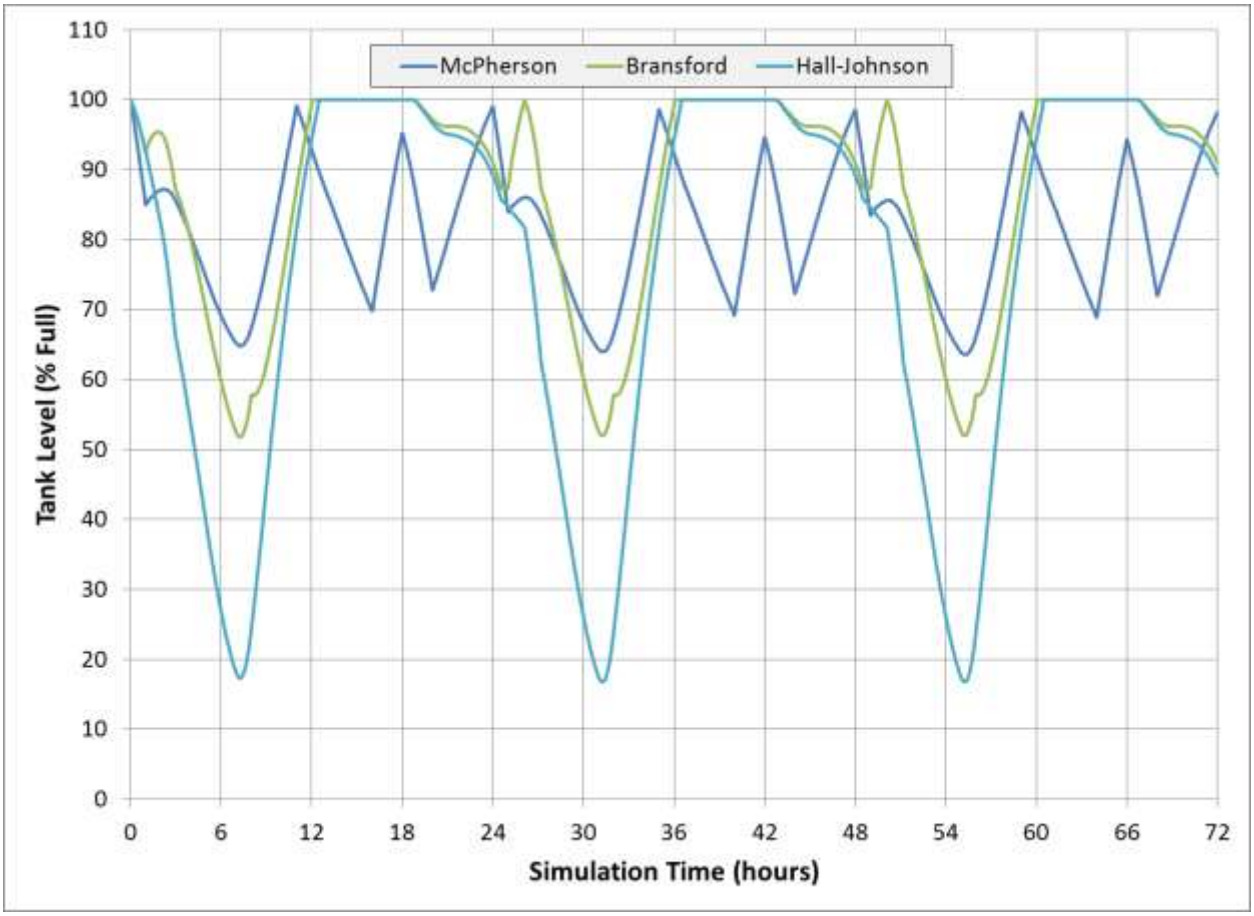


Figure 16: Current Water Model Maximum Day Tank Level Variations for Existing ESTs

The HGL for the McPherson EST varies between 826.9 and 840.0 feet, which corresponds to a range of approximately 63 to 100 percent full.

The HGL for the Bransford EST varies between 773.1 and 790.0 feet, which corresponds to a range of approximately 49 to 100 percent full. The HGL for the Hall-Johnson EST varies between 760.3 and 790.0 feet, which correspond to a range of approximately 15 to 100 percent full. During the average day simulation, the Bransford and Hall-Johnson EST levels remained balanced throughout the simulation, as discussed in the previous section. During the maximum day simulation, the Hall-Johnson EST levels dropped significantly more than the levels in the Bransford EST, with a minimum level in the Hall-Johnson EST of almost 13 feet less than the minimum level in the Bransford EST.

The peak flow low plane pumps at LD Lockett are set to run from 1 AM to 8 AM during the maximum day simulation. The pumps were set to run at this time to offset peak demands and maintain overall storage levels within the low plane. The HGL and tank level graphs for the maximum day simulation show that the Bransford EST has a more pronounced response to the operations at LD Lockett than the Hall-Johnson EST does. Specifically, the Bransford EST fills during the early hours of the first day of the simulation after

the pumps at LD Lockett turn on. In contrast, levels in the Hall-Johnson EST continue to fall, even after the pumps at LD Lockett turn on. The Hall-Johnson EST does receive some benefit from the LD Lockett pump operations, which is seen through the lessened rate of decrease in level once the LD Lockett pumps turn on. However, during period when LD Lockett is operating, the two ESTs in the low plane are not balanced. In contrast, during the afternoon (when the LD Lockett pumps remain off) the Bransford and Hall-Johnson ESTs remain in better balance.

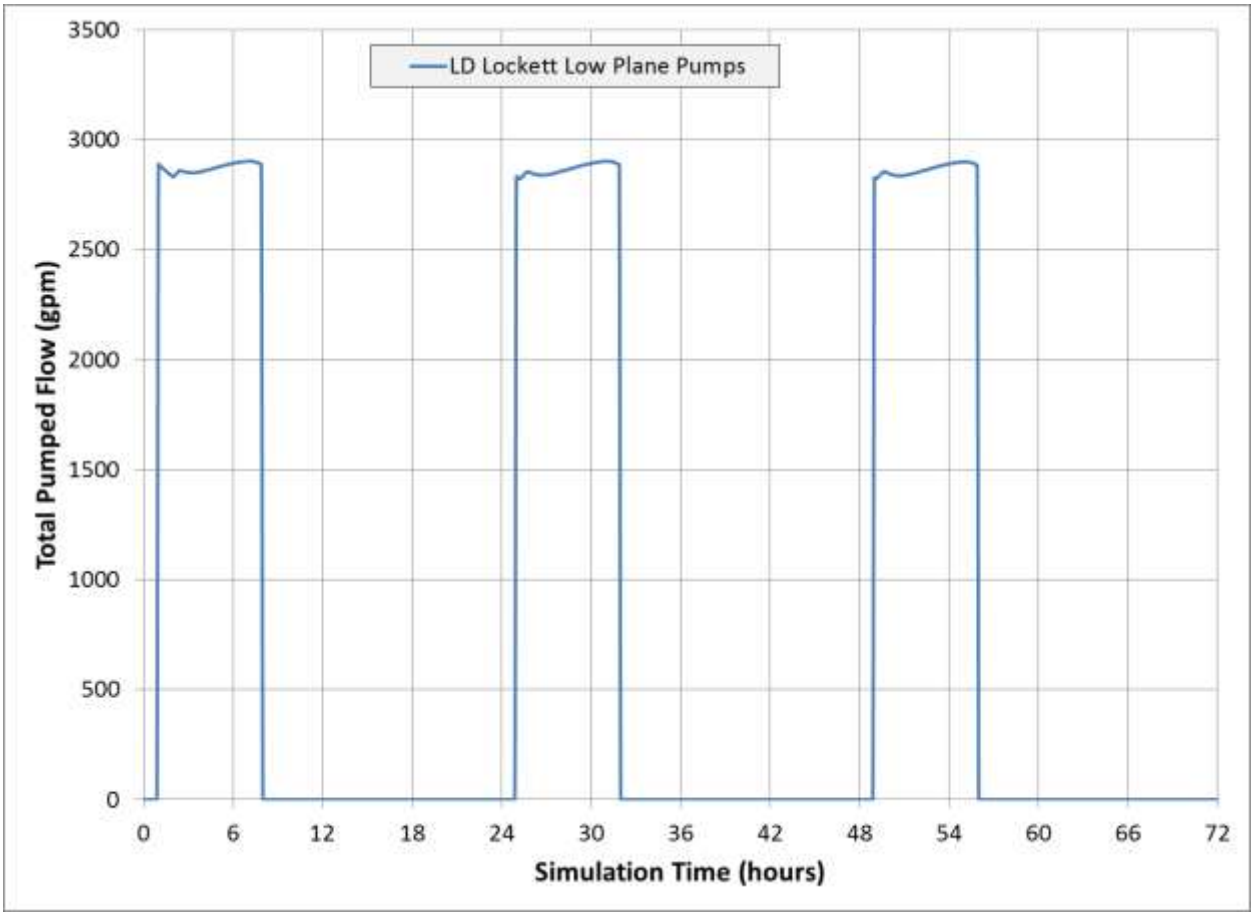


Figure 17: Current Water Model Maximum Day LD Lockett Total Pumped Flow

7.1.2.4 Water Age (Maximum Day)

The maximum water age for each junction within the system is shown in Exhibit 8. The water age in the low plane is generally good. The majority of the main lines within the low plane have maximum water age values between 0 and 2 days. The low water age can be attributed to good turnover in the tanks within the pressure plane. The area on the northwest side of the low plane has the highest water age. This is logical given that area is the most hydraulically distant point from the water sources for the low plane.

The water age in the high plane is higher than the water age in the low plane. As was the case for the average day demand simulation results, the maximum water age for the high plane increases northward through the zone. This is anticipated because water enters the south side of the pressure plane from LD Lockett. The northwest side of the high plane has maximum water age values in the 2-3 day range, so the water quality for this area is anticipated to be fair.

As shown in Exhibit 8, there are locations throughout both pressure planes with high water age. The majority of these locations are junctions in the model with little to no demand. These locations are potential points of water quality concern in the system, but additional investigation is warranted to confirm poor water quality. Overall, there are not any large-scale areas of concern in regard to water age during maximum day demand conditions.

7.1.2.5 Available Fire Flow (Maximum Day)

The available fire flow was calculated for each junction in the model based on a steady-state scenario with the pumps at LD Lockett off and the water levels in the ESTs in the system at half full. Flow was allowed to enter the system through the TRA meter stations during the simulation. The systems demands were set as the average maximum day demands, rather than peak hour demands. Either maximum day or peak hour demands can be used for assessing available fire flows, and use of peak hour demands will result in a conservative analysis. However, given the magnitude of peak hour demands for the Colleyville system, maximum day demand conditions were selected for fire flow analysis to avoid potentially costly overdesign of the distribution network to address extreme emergency conditions with a very low probability of occurrence.

The available fire flow at each junction in the model (supply lines excluded) is shown in Exhibit 9. The issues associated with the available fire flow during maximum day demand conditions are similar to those shown for average day conditions. There are potential issues with meeting the desired 1,500-gpm available fire flow at select points throughout the system. Refer to Section 7.1.1.5 for additional description of these areas.

Additionally, the commercial and institutional area on the east side of town adjacent to SH 121 does not have sufficient fire flow during maximum day demand conditions. The available fire flow in this area is generally between 2,000 and 3,000 gpm, which is below the needed fire flow of 3,000 gpm.

7.2 Near-Term Water System Improvements

7.2.1 Description

The distribution system analysis identified issues with system capacity, namely low pressures and inadequate fire flows. Additionally, pipes were identified that experienced excessive velocities. Conceptual improvements were designed to address the hydraulic issues, as well as to improve the condition and operations of the system. Triggers, explained in detail in Section 8.1, were developed in order to identify and rank proposed capital projects. The projects have been divided into groups, and they are explained in significant detail in Section 8.1 and Appendix D. The near-term improvements, shown in Exhibit 10, were incorporated into the current model to verify hydraulic issues were addressed.

Each of the improvements is described in detail in the Water System Capital Improvements Plan, so each of the individual project groups will not be covered here. However, the two main projects to improve system hydraulics are discussed below. Additionally, select model results following incorporation of the proposed changes are also presented within this section.

7.2.2 Additional Storage Evaluation

A criterion was identified in the design criteria for elevated storage volume. The elevated storage criterion was set at 15% of maximum day demand in each pressure plane. The existing and target storage (over the 20-year planning horizon) are shown for each pressure plane in Figure 18. For this evaluation, it has been assumed that 15% of the system demands are in the high plane and 85% of the system demands are in the low plane.

The McPherson EST provides adequate storage for the high plane. This is supported through the hydraulic modeling, which shows the levels in the McPherson EST are maintained above 60% full during the maximum day demand simulation.

However, the target storage for the low plane surpasses the existing storage in the Bransford and Hall-Johnson EST during 2015-2016. Therefore, addition of new storage within the low plane is recommended, as a deficiency of approximately 0.4 MG will be present by 2034. This recommendation is supported by the hydraulic modeling, which showed the Hall-Johnson EST dropping to less than 20% full during the maximum day demand scenario and demand issues beginning to appear in the southwest portion of the system.

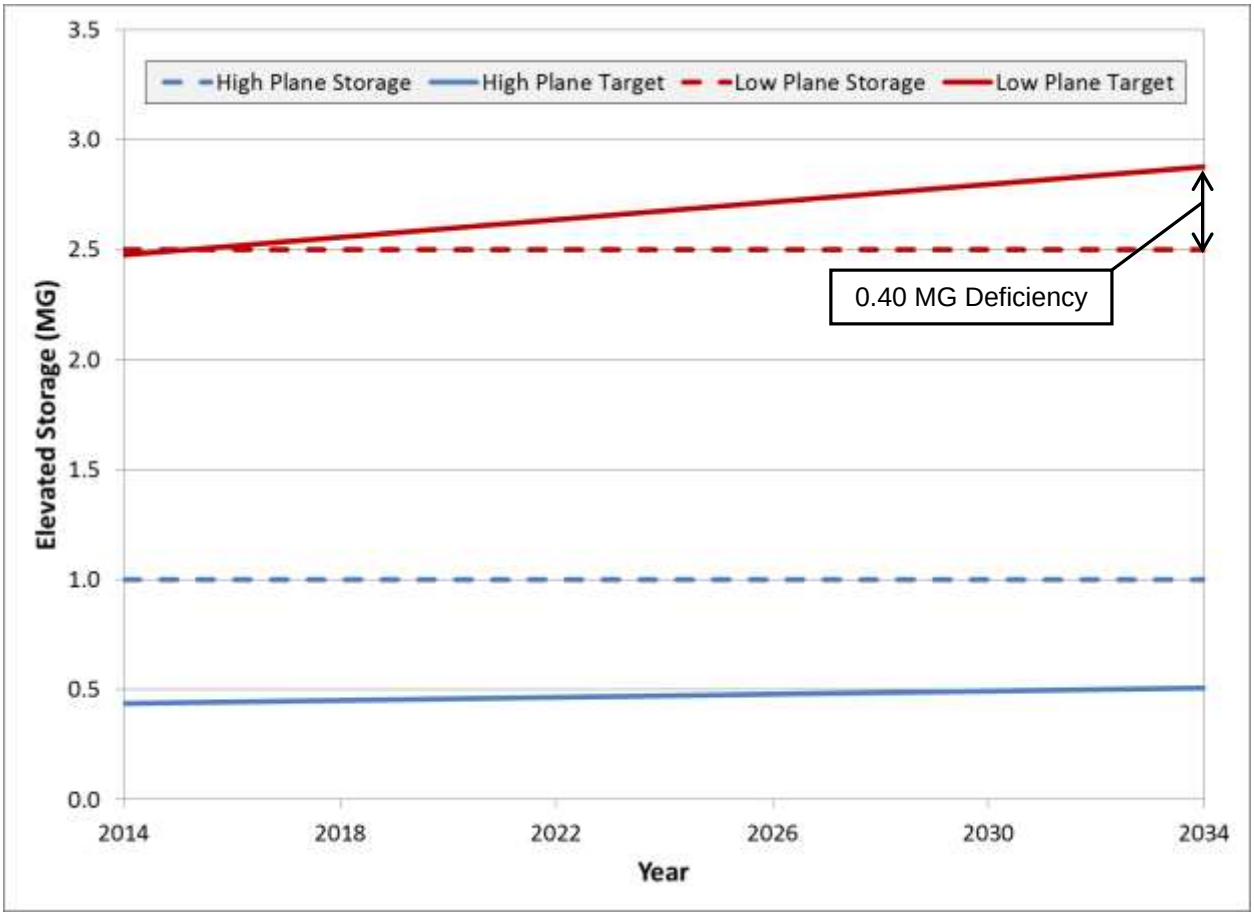


Figure 18: High & Low Pressure Plane Existing & Target Storages

The proposed area for new storage is the area of Greenbriar Lane and Colleyville Boulevard. Refer to Group A in Exhibit 10 for the proposed location of the new elevated storage tank. This siting area has an obvious impact on the low pressures and inadequate fire flows in the adjacent area of the low plane, and provides an area of high elevation to serve the entire system. Further evaluation of tank siting should be conducted during detailed design.

The benefits of the new storage tank will be experienced throughout the low plane because the increased storage tank volume will help stabilize the HGL across the pressure plane. However, complimentary improvements are also needed in this area, including improvements to address low pressure and inadequate fire flow in the Monticello Parkway area.

While the target elevated storage difference is 0.4 MG, it is recommended to design a 1.0 MG storage tank. This recommendation is based on modeled system demands that show a benefit to the tower levels throughout the low plane system as storage size is increased. Additionally, fire flow demands of 3,000 gpm sustained for 3 hours, per IFC guidelines, equates to 0.5 MGD; therefore, it is recommended that this amount plus usage demand be accounted for in sizing this EST. Lastly, cost differences between a 0.5 MG and a 1.0 MG elevated or storage tank are anticipated to be approximately 20%, which is relatively minimal considering the previously listed factors.

7.2.2.1 Storage Improvements Comparison

To determine if the target design criteria requiring the increased elevated storage was justified, an alternatives evaluation between an EST and a GST with a BPS was conducted to address the system capacity and fire flow deficiencies in that area. Both alternatives utilize 1.0 MG storage tank.

A ground storage tank offers ease of access and maintenance, but requires additional pumping, infrastructure, and operations and maintenance costs. An elevated storage tank provides a low maintenance alternative that can withstand power outages and provides more resilient storage, but does require more costly maintenance when required due to access considerations.

The proposed elevated storage tank would be backfed through the distribution system. A diameter of 75 feet has been selected, with a low water level of 760, and a high water level of 790. Inflow/outflow will be accomplished by means of an altitude valve. No additional booster pumping to fill the tower is anticipated.

The GST and BPS option will behave similarly to the EST option. When the HGL in the area around the BPS drops to 770 ft, the BPS will turn on and provide water to the system through redundant 250 gpm pumps rated at 117 feet of head. When peak demand occurs (and to accomplish fire flow needs), additional redundant 3,000 GPM pumps at 117 feet of head are required.

The VFD-controlled pumps would maintain a set HGL of 775, and would operate as long as they are needed to maintain that level. As the demand reduces, the flow through the pumps reduces to a shut-off point. During this time, the GST will refill by means of a pressure-sustaining valve on the tank fill line. This valve slowly fills the tank, in order to prevent sudden demand on the system that may cause the pumps to re-engage. No backup power was provided at either location for the alternatives comparison.

The following Tables 25 and 26 show the estimated life-cycle costs for a 20 year planning horizon for each alternative.

Table 25: Elevated Storage Tank Cost Analysis

Elevated Storage Tank Option					
Label	**Diameter (ft.)**	**Base (ft.)**	**LWL (ft.)**	**HWL (ft.)**	Estimated Costs
1.0 MG EST	75	648	760	790	$ 2,200,000
				Subtotal	$ 2,200,000
				Contingency (40%)	$ 880,000
				Appurtenances (electrical, SCADA, etc.)	$ 440,000
				Engineering	$ 528,000
				OPCC	$ 4,048,000
				20-Year Life Cycle Energy Costs	Negligible
				20-Year O&M Costs	$ 400,000
				Total Estimated Life-Cycle Costs	**$ 4,348,000**

Table 26: Ground Storage Tank and Booster Pump Station Cost Analysis

Ground Storage Tank Option					
Label	**Diameter (ft.)**	**Base (ft.)**	**LWL (ft.)**	**HWL (ft.)**	Estimated Costs
1.0 MG GST	75	648	648	678	$ 1,200,000
Label	**Quantity**	**Cost (EA)**			
High Demand Pumps	2	$110,000			$ 220,000
Low Demand Pumps	2	$35,000			$ 70,000
Valving and Connecting Piping	1 (LS)	$120,000			$ 120,000
Pump Station Building	1 (LS)	$400,000			$ 400,000
				Subtotal	$ 2,010,000
				Contingency (40%)	$ 804,000
				Appurtenances (electrical, SCADA, etc.)	$ 402,000
				Engineering	$ 482,000
				OPCC	$ 3,698,000
				20-Year Life Cycle Energy Costs	$ 100,000
				20-Year O&M Costs	$ 357,000
				Total Estimated Costs	**$ 4,155,000**

Although the ground storage tank is less expensive in overall construction costs, the 20-year life cycle costs for pumping electricity and O&M exceed the 20-year life cycle O&M costs for the elevated storage tank. Elevated storage O&M costs include two condition rehabilitation/repainting projects, while O&M

costs for the ground storage tank include two repainting projects along with service and repair for the pumps.

Comparisons on similar situations typically find in favor of ground storage tanks, as high service pumping is necessary to provide pressure to fill the elevated storage tanks, thereby increasing the elevated life cycle costs. However, the City is able to utilize the pressure provided by TRA's supply line, and avoid significant O&M costs of operating and maintaining high service pumps.

Based on the analysis presented, an 1.0 MG elevated storage is recommended as the proposed improvement project. The EST will provide greater operational flexibility and system resiliency, such as in the case of a system-wide loss of power. Additionally, City staff will not be required to operate and maintain an additional pump station. This improvements project has been identified as Water Group A. If the GST and BPS are preferred, further evaluation of pump and storage sizing should be completed during detailed design to ensure the most efficient pumps are selected.

7.2.3 System Balancing Improvements

Additions of two pressure reducing valves are recommended to allow flow from the high plane to the northwest area of the low plane. During normal demand conditions, no flow will pass through the valves. However, during high-demand or emergency conditions, flow will be allowed through the valves to maintain acceptable pressures. If the proposed pressure reducing valves are incorporated, it will improve the responsiveness of the system to high-stress conditions, and reduce operational needs, at minimal capital costs. Minimal additional piping is associated with these improvements, and the total project is identified as Water Improvement Group B.

7.2.4 Pressures

The recommended improvements address all the issues with inadequate pressures. As shown in Exhibit 11, the minimum pressure is at least 35 psi at each junction within the system under maximum day demand conditions. The single red circle north of the McPherson EST is immediately downstream of one of the pressure reducing valves, and the minimum pressure at this location is equal to 35 psi.

The red circles near the LD Lockett GST and PS are on the supply line from the TRA metering station to the LD Lockett GST. The model assumes the TRA meter station is closed when there is not flow to the LD Lockett GST, so the supply line for the LD Lockett GST has the same HGL as the GST when the tank is not filling. A pressure of 35 psi is equivalent to approximately 81 feet. The GST is less than 81 feet tall, so the pressures adjacent to the GST are less than 35 psi.

7.2.5 Velocities

The recommended improvements address the issues with excessive velocities. Exhibit 12 shows the maximum velocities in the system during maximum day demand conditions. The velocities remain high (5 to 10 fps) in some lines, but these are generally main supply lines. The majority of the lines in the system have velocities less than 3 fps, which will prevent excessive head loss across the system.

7.2.6 Tower Levels

The addition of the third EST within the low plane helps maintain tank levels within the low plane as shown in Figure 19. The Hall-Johnson EST remains close to 30% full during maximum day demand conditions, rather than dropping to approximately 15% full.

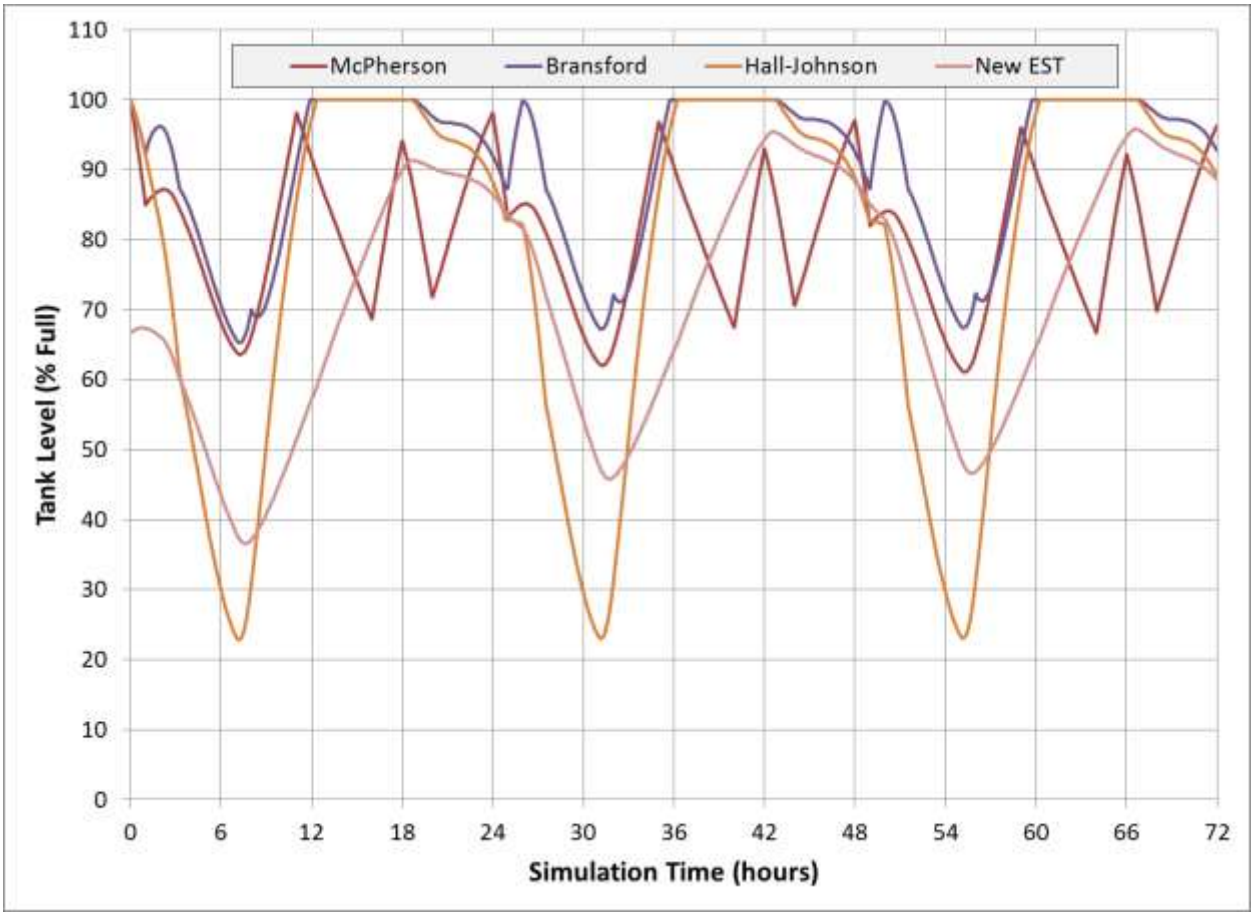


Figure 19: Low Plane Tank Levels with New EST Addition

7.2.7 Water Age

Addition of storage within a distribution system can have a negative impact to water quality if water age is increased significantly. The 10 States Standards (Great Lakes – Upper Mississippi River Board) specifically addresses this issue in discussion of sizing for finished water storage (Part 7 of their Recommended Standard for Water Works 2012 Edition), which states that excessive storage capacity should be avoided to prevent potential water quality deterioration problems.

With the addition of a 1-MG EST in the low plane, the total elevated storage in that pressure plane is 3.5 MG and the total elevated storage in the system is 4.5 MG. Therefore, the high plane still has a higher ratio of elevated storage to demand than the low plane.

The maximum water age following implementation of the near-term improvements is shown in Exhibit 13. The maximum water age for the area adjacent to the new EST is 2-3 days. This water age is similar to the northwest side of the high plane. The maximum water age in the majority of the system is 0-2 days. There is a potential impact to water quality in the southwest area of the low plane where the elevated storage is added. However, the potential negative issues are small in comparison to the positive impacts to system pressures, available fire flows, and resilience to emergency conditions.

7.2.8 Available Fire Flows

The recommended improvements address the majority of the issues with fire flows. As shown in Exhibit 13, the available fire flow is greater than 1,500 gpm for almost all the junctions in the system, and the available fire flow is at least 3,000 gpm along the main lines in the principle mercantile areas. The areas where fire flows set in the design criteria were not met are areas of concern discussed below.

7.2.8.1 Additional Areas of Concern

Three areas of concern were identified related to fire flow deficiencies following the improvements, as described below. These areas of concern were not identified as individual projects, as it is believed they can be addressed through individual improvements along each line section, if warranted.

The existing 6-inch pipeline that parallels the 8-inch pipeline along the boundary between the high and low pressure planes along Westcoat in Figure 20 does not have adequate fire flow. Hydrants along this area should be served by the 8-inch pipeline that is part of the high pressure plane system.

Figure 20: Parallel Lines Along Westcoat Drive

There is a long 6-inch service line to a house north of John McCain Road between Holly Lane and Trianon Court. The model shows 1,500 gpm is not available for fire flow at the end of the service, but adequate fire flows are available on John McCain Road and Trianon Court, shown on Figure 21.

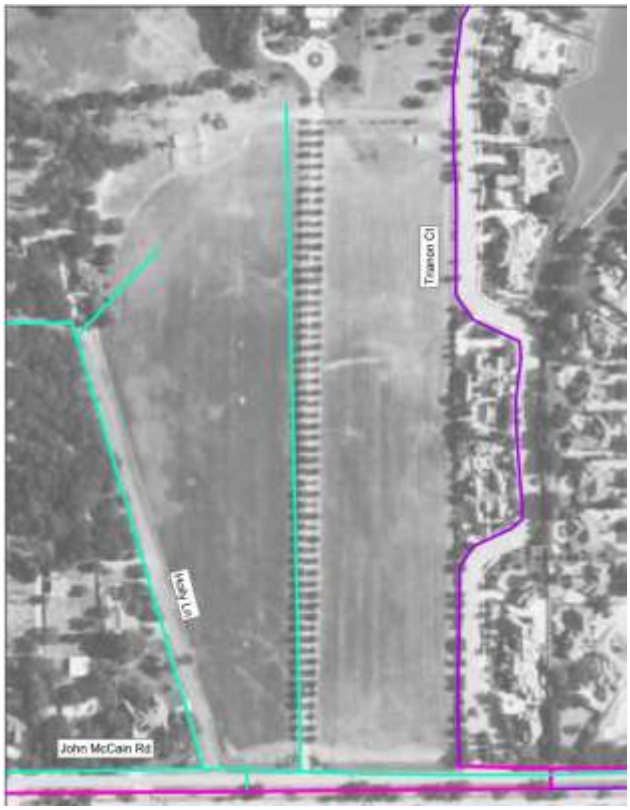


Figure 21: 6" Service Line Along John McCain

The proposed improvements result in 3,000 gpm available fire flow along the main lines for the retail area adjacent to SH 121 on the east side of the system. However, fire flows of 3,000 gpm are unavailable off many of the 8-inch and smaller lines. These areas are shown in light blue in Figure 22.

Figure 22: Commercial Fire Flows Along SH-121 and Glade Road

Further field evaluation and as-built verification into each of these areas of concern is recommended to determine where local fire flows would be supplied, and to ensure adequate coverage.

7.3 Future Water Model Results

The future model was utilized to assess system behavior over the 20-year planning horizon. The infrastructure for the future model was the existing system plus the near-term improvements shown in Exhibit 10 and documented in the CIP. The demands in the model were also updated to represent anticipated conditions at the end of the 20-year planning horizon.

In order to apply future demands to the system, undeveloped parcels were identified by cross-referencing the City's GIS parcel map with the existing water meters. For the purposes of this study, parcels were assumed to be undeveloped if they did not have a current water meter. The City's 2025 Future Land Use plan was utilized to classify the undeveloped parcels by land use type. Specifically, the undeveloped parcels were divided into three categories:

1. Single Family Residential

2. Commercial, Industrial, or Institutional
3. Park or Open Space

The undeveloped parcels identified are presented by category in Exhibit 27. The undeveloped parcels were assigned demands in order to achieve the desired total system demands. The existing average day demands were escalated based on the increase in per capita demands presented in previous sections. Then, the additional demand necessary to reach the 2034 average day demand was calculated. The additional demand to be allocated to the undeveloped parcels for maximum day demand conditions was calculated as the difference between the maximum day demands for 2034 and 2014.

The single family residential areas were divided into lots based on average parcel size in adjacent areas. The total number of potential lots identified was 1,356. The commercial-industrial-institutional and park-open space areas were evaluated on a per acre basis. The total areas for those categories were 252.2 and 391.8 acres, respectively. Each classification was assigned a per unit load for average and maximum day, as shown in Table 27. These unit loads were estimated based on average values for Colleyville and industry standards, and they were adjusted in order to achieve the appropriate levels of new system demands for both average and maximum day demand scenarios.

Table 27: Demands for Undeveloped Parcels

Classification	Unit	Number of Units	Average Day Demand (per unit)	Maximum Day Demand (per unit)
Single Family Residential	Lot	1,356	413	994
Commercial-Industrial-Institutional	Acre	252.2	1,250	3,008
Park-Open Space	Acre	391.8	1,100	2,647

7.3.1 Average Day Demands

The system was analyzed under future average day demands. The total average day demand was divided into base and excess components for application of diurnal curves, as shown on Table 28.

Table 28: Future Water Model Average Day Demands

Demand Condition	Base Demand (MGD)	Excess Demand (MGD)
Average Day	3.345	6.033

7.3.1.1 HGL and Pressure (Average Day)

The improvements identified based on the current system model provide adequate capacity for future average day demand conditions. The HGL and pressure results for the future average day system are presented in the following tables. While the experienced conditions will be influenced by actual system

operations, the simulation shows that minimum pressures are sustained during the simulation while still allowing adequate tank cycling (all tanks cycle at least 40% during the course of the day). Adequate tank cycling indicates that water quality would be maintained with these improvements. See Tables 29 and 30 for a tabulation of average day HGL and pressure results.

Table 29: Future Water Model Average Day HGL Results

Parameter	Statistic	High Plane	Low Plane
Minimum HGL (ft)	Minimum	822.0	760.5
	Mean	822.7	771.3
	Maximum	823.8	790.7
Maximum HGL (ft)	Minimum	833.7	782.7
	Mean	838.8	790.9
	Maximum	856.7	804.2

Table 30: Future Water Model Average Day Pressure Results

Parameter	Statistic	High Plane	Low Plane
Minimum Pressure (psi)	Minimum	45	38
	Mean	68	67
	Maximum	92	100
Maximum Pressure (psi)	Minimum	49	44
	Mean	75	76
	Maximum	99	104

The minimum pressures are greater than 35 psi for all junctions within the distribution system. Refer to Exhibit 15 for a visual representation of the minimum system pressures. No additional improvements are necessary to improve pressures during average day conditions.

7.3.1.2 Velocity (Average Day)

The maximum velocity in each pipe during the model simulation is shown in Exhibit 16. As was the case for simulations with the existing system, the majority of the high velocities (greater than 5 fps) are associated with the pump stations and main supply lines. No additional improvements are necessary to address velocities that occur during average demand conditions.

7.3.1.3 Tower Levels (Average Day)

Variations in tank levels in the McPherson, Bransford, Hall-Johnson, and the New EST over the three-day simulation are shown in Figure 23.

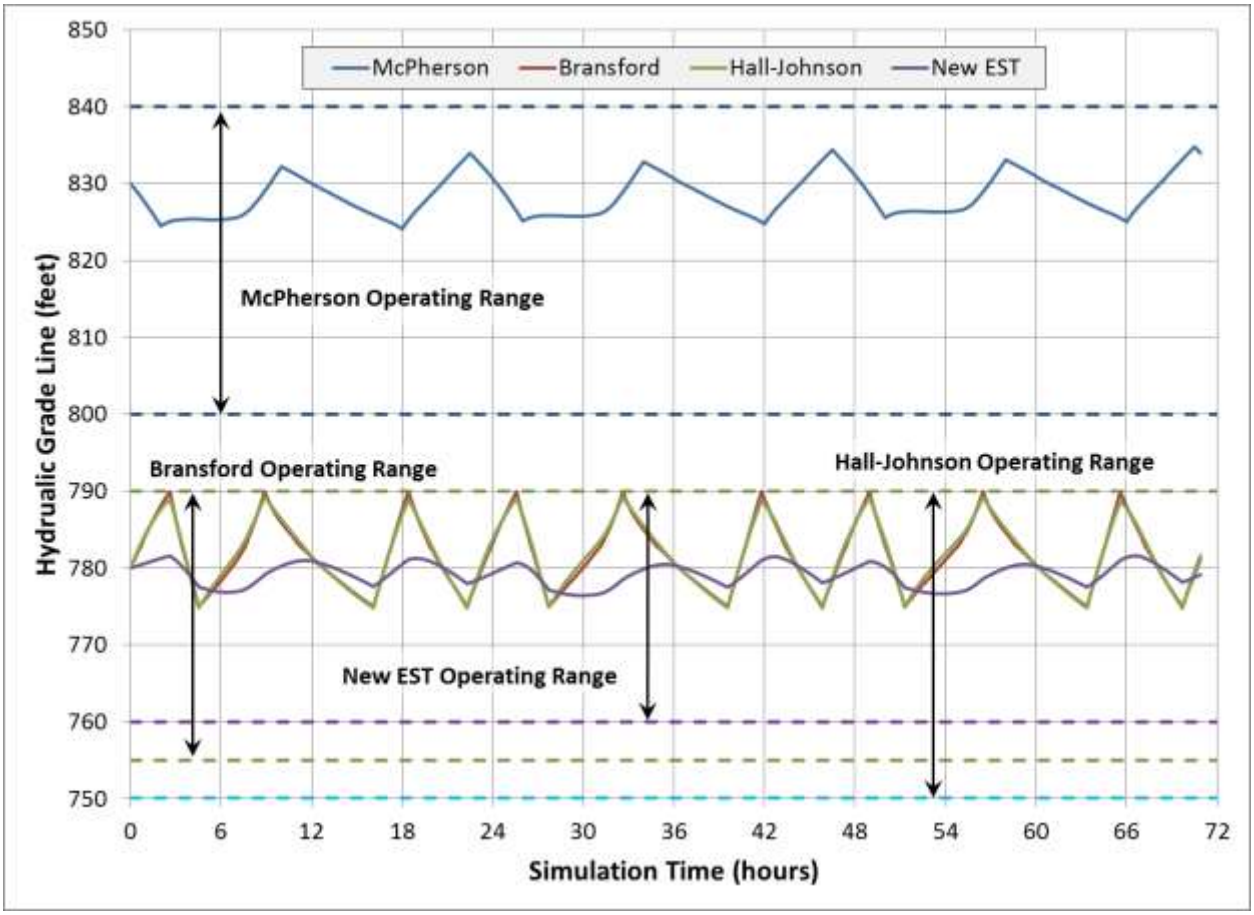


Figure 23: Future Water Model Average Day Tank Level Variations for Existing and New ESTs

The HGL for the McPherson EST varies between 824.5 and 832.2 feet, which corresponds to a range of approximately 56 to 78 percent full.

The HGLs for the low plane tanks vary, with the Bransford and Hall-Johnson ESTs maintaining similar levels and the New EST having less variation.

7.3.1.4 Water Age (Average Day)

The maximum water age for each junction within the system is shown in Exhibit 17. The water age in the low plane is generally good. The majority of the main lines within the low plane have maximum water age values between 0 and 2 days, with the area adjacent to the New EST in the southwest area having slightly higher water age. The higher maximum water age in this area is due to the relatively low turnover in the New EST, relative to the Bransford and Hall-Johnson ESTs. Operational adjustments within the low plane could be used to generate additional turnover in the New EST if decreasing water quality becomes an issue in this area.

The water age in the high plane is higher than the water age in the low plane. As with previous scenarios presented, the southern portion of the high plane has low water age (less than 1 day) while the maximum water age in the northwest portion of the high plane is in the 3-5 day range, which indicates potential for water quality issues. Flushing of lines on the northwest side of the high plane may be necessary to maintain system water quality.

As shown in Exhibit 17, there are locations throughout both pressure planes with high water age. The majority of these locations are junctions in the model with little to no demand. The automated spatial demand allocation procedure did not guarantee demands at each junction. Junctions with small demand that are located at the end of dead-end pipes will have high water age values, as discussed previously.

7.3.1.5 Available Fire Flow (Average Day)

The available fire flow was calculated for each junction in the model based on a steady-state scenario with the pumps at LD Lockett off and the water levels in the ESTs in the system at half full. Flow was allowed to enter the system through the TRA meter stations during the simulation. The available fire flow at each junction in the model (supply lines excluded) is shown in Exhibit 18. The only deficiencies are the areas of concern identified for the current maximum day model (see Section 7.2.6). Therefore, no additional improvements are necessary to address available fire flow under future average day conditions.

7.3.2 Maximum Day Demands

The future system was analyzed under maximum day demands. The total maximum day demand was divided into base and excess components for application of diurnal curves. These demands are shown in Table 31.

Table 31: Future Water Model Maximum Day Demands

Demand Condition	Base Demand (MGD)	Excess Demand (MGD)
Maximum Day	8.133	14.434

7.3.2.1 HGL and Pressure (Maximum Day)

The HGL (and pressure) within the system will usually vary more significantly on maximum day than during average day conditions, due to increased flow and head loss in water mains throughout the system. Additionally, storage tank levels may be depleted more significantly during maximum day conditions than during average day conditions when supply infrastructure is able to better keep pace with demands. Minimum and maximum HGL values within each of the pressure planes are shown in Table 32, while the pressure statistics are shown in Table 33.

Table 32: Future Water Model Maximum Day HGL Results

Parameter	Statistic	High Plane	Low Plane
Minimum HGL (ft)	Minimum	805.7	702.4
	Mean	812.0	733.5
	Maximum	823.6	779.9
Maximum HGL (ft)	Minimum	838.5	784.2
	Mean	850.5	796.7
	Maximum	891.0	826.9

Table 33: Future Water Model Maximum Day Pressure Statistic Results

Parameter	Statistic	High Plane	Low Plane
Minimum Pressure (psi)	Minimum	38	28
	Mean	64	51
	Maximum	86	75
Maximum Pressure (psi)	Minimum	51	45
	Mean	80	78
	Maximum	112	106

The minimum pressures are shown in Exhibit 19. The high pressure plane performs relatively well under maximum day demand conditions. The pressures are maintained above the 35 psi minimum at all locations.

There are pressure issues in the low plane that are most prominent in the areas north of the Bransford and Hall-Johnson ESTs. There are two major issues identified by this simulation, and these two issues are related. The first issue is the inability to maintain water levels in the Hall-Johnson EST. The level in the Hall-Johnson EST drops, which causes low pressures in the area adjacent to the tower. The second issue is the inability for water to be conveyed from the Bransford EST to the northeast to serve the north-central portion of the low plane. Addition of a new line along LD Lockett Road up to Pleasant Run Road increases the ability of water to flow from the Bransford EST to this area, which reduces demand on the Hall-Johnson EST and helps maintain balanced levels in those two ESTs. This improvement is presented in Appendix D as Water Group C.

7.3.2.2 Velocity (Maximum Day)

The maximum velocity in each pipe during the model simulation is shown in Exhibit 20. A number of the pipes with high velocities are similar to those identified in previous simulations. However, there are additional pipes with high velocities under this demand scenario. Specifically, both pipes that connect the Bransford EST to the area along Tinker Road and Pleasant Run Road (northeast of the Bransford EST) have velocities in the 5 to 10 fps range. The high velocities, and associated high head loss values, contribute to the issues with tank level balancing and sub-standard pressures described in Section 7.3.2.1.

7.3.2.3 Tower Levels

Future variations in tank levels in the McPherson, Bransford, Hall-Johnson, and the new EST over the three-day simulation are shown in Figure 24.

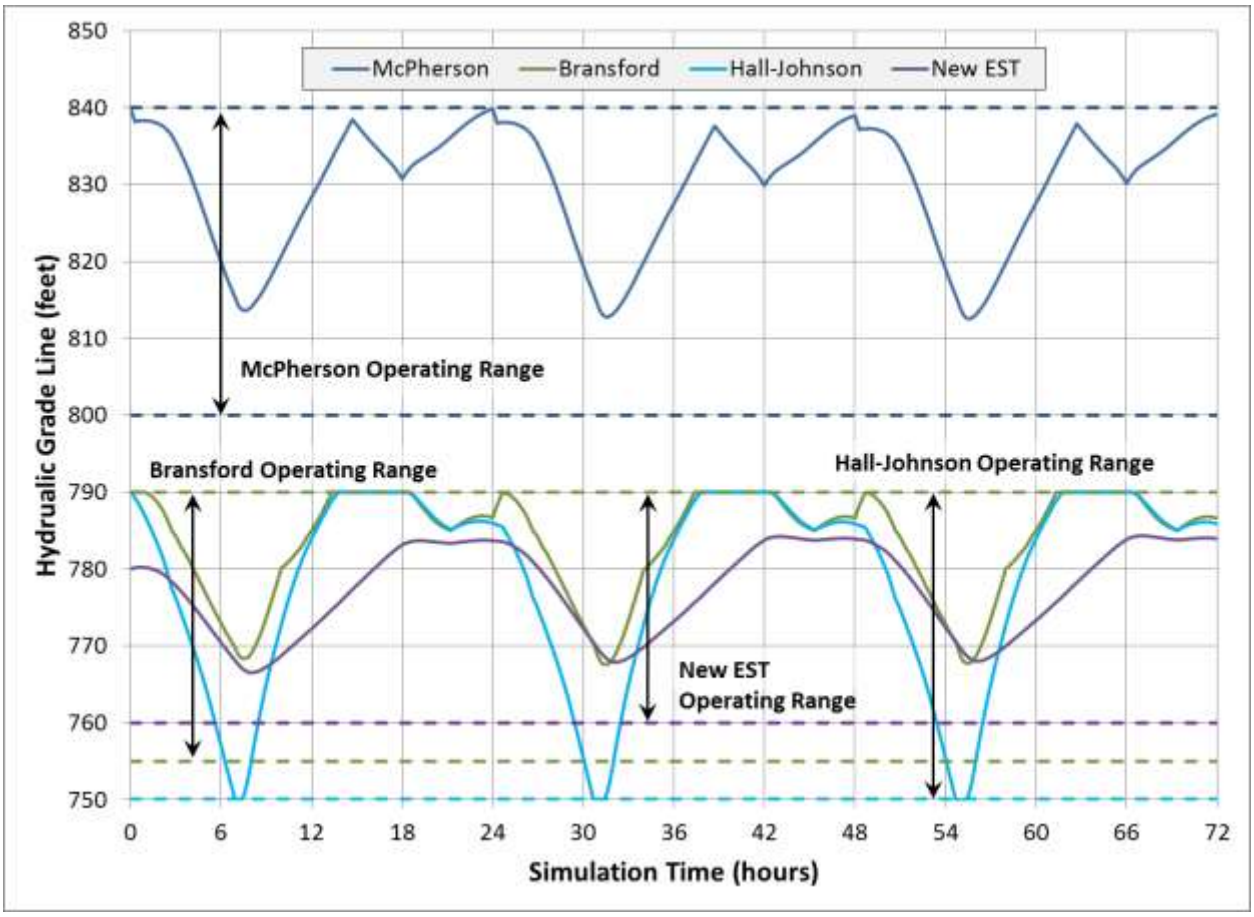


Figure 24: Future Water Model Maximum Day HGL for Existing and New ESTs

The McPherson EST volume is significantly depleted during the peak morning demand. However, the high plane pumps at LD Lockett are able to refill McPherson during off-peak times.

In the low plane, the levels in the Hall-Johnson EST are completely depleted by the peak morning demand, while the other two tanks maintain significant volume. This suggests improvements are warranted to help balance tank levels, as described above. The peak flow low plane pumps at LD Lockett are set to run from 0 to 10 AM during the maximum day simulation. The pumps were set to run at this time to offset peak demands and maintain overall storage levels within the low plane. The HGL and tank level graphs for the maximum day simulation show that the Bransford EST has a more pronounced response to the operations at LD Lockett than the Hall-Johnson EST does, and when LD Lockett is operating, the Bransford and Hall-Johnson ESTs are not balanced. In contrast, during periods when the LD Lockett PS is not pumping into the low plane, the Bransford and Hall-Johnson ESTs remain in better

balance. This suggests the low plane pumping capacity does not significantly contribute to levels in the Hall-Johnson EST.

7.3.2.4 Water Age (Maximum Day)

The water age under maximum day demand conditions is shown in Exhibit 21. As Figure 24 shows, there is significant tank turnover under maximum day demand conditions, which results in low water age in the system. The maximum water age in each pressure plane is 0 to 2 days, except for the low demand junctions in each zone that show high water age for all simulations.

7.3.2.5 Available Fire Flow

The available fire flow was calculated for each junction in the model based on a steady-state scenario with the pumps at LD Lockett off and the water levels in the ESTs in the system at half full. Flow was allowed to enter the system through the TRA meter stations during the simulation. The available fire flow at each junction in the model (supply lines excluded) is shown in Exhibit 22. The only deficiency, other than the areas of concern identified previously, is on junction northeast of the Bransford EST at the end of a long dead-end line. Extension of this dead-end line to connect to the water line along Emerald Drive is recommended as an improvement as part of a group identified to improve looping within the distribution system.

7.4 Future Water Improvements

7.4.1 Description

The distribution system analysis identified issues with system capacity, namely low pressures and inadequate fire flows. Conceptual improvements were designed to address the hydraulic issues, as well as to improve the condition and operations of the system. Triggers, explained in detail in Section 8.1, were developed in order to identify and rank proposed capital projects. The projects have been divided into groups, and they are explained in significant detail in Appendix D. The long-term improvements, shown in Exhibit 23, were incorporated into the current model to verify hydraulic issues were addressed.

Each of the improvements is described in detail in Appendices D and G.

7.4.2 Pressures

The recommended improvements address all the issues with inadequate pressures. As shown in Exhibit 24, the minimum pressure is greater than 35 psi at each junction within the system under maximum day demand conditions.

7.4.3 Tower Levels

Prior to the improvements, the Hall-Johnson EST went empty due to the peak demands. The improvements, specifically the new 16-inch line along LD Lockett Road, help balance tank levels, as shown in Figure 25.

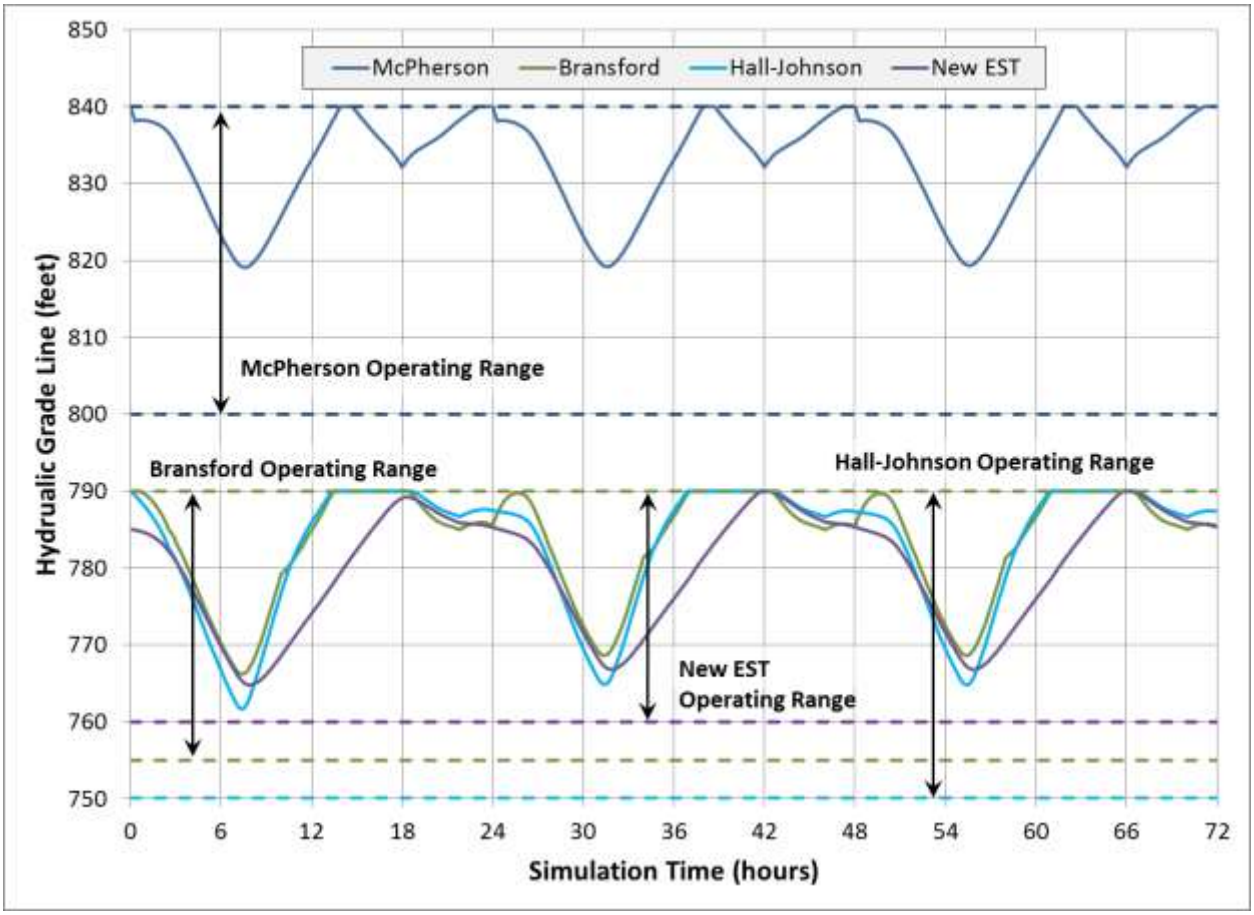


Figure 25: HGL for Existing ESTs and New EST with Future Water Improvements

The McPherson EST levels are maintained at higher levels because less water is needed to backfeed the low plane from the high plane following implementation of the identified improvements. This helps maintain volumes in case of emergency situations, such as disruption of flows from LD Lockett PS or a fire within the high plane or northwest portion of the low plane.

The Hall-Johnson and new ESTs have minimum levels of approximately 20%, while the Bransford EST reaches a minimum level of approximately 30%. Compared to previous simulations, the ESTs in the low plane are markedly more balanced.

7.4.4 Velocity

The maximum velocities are shown in Exhibit 25. The maximum velocities on the lines to the east and northeast from the Bransford EST are decreased to the 3 to 5 fps range as a result of the improvements to balance tower levels in the low plane.

7.4.5 Available Fire Flows

The additional looping in the system alleviates the issues with inadequate fire flows presented previously. The fire flows available following implementation of the proposed long-term improvements are shown in Exhibit 26.

8.0 Recommended Water Improvements

8.1 Project Identification and Ranking

In order to accurately define the need for an improvement to the system, several key factors were identified as a trigger for a project. These triggers were developed in conjunction with the previously established design criteria. One identified area of need may have several triggers which necessitate a need for one set of improvements to address all issues (i.e., one area of low pressure may not meet TCEQ requirements or recommended fire flows).

The identified triggers are as follows (in order of priority, from greatest to least):

1) Regulatory
   The regulatory requirements which would qualify in this category include TCEQ regulations identified in the design criteria section. For example, failing to meet a minimum residual pressure of 20 psi in the model under combined drinking water and fire flow demands would activate this trigger, or failure to meet minimum TCEQ storage capacity. Additionally, TCEQ design and facility requirements such as site security would activate this trigger.

2) Capacity
   This trigger is activated if a section or area of the system is not able to provide the modeled flow during peak demand or elevated storage to all meters within that portion of the system.

3) Fire flow
   This trigger is activated during the fire flow modeling scenario, if that portion of the system is not able to meet the minimum required flow rates.

4) Condition
   Condition triggers are activated based upon deteriorating conditions of existing infrastructure. This trigger is activated if the field investigations of the water storage and pumping facilities determine rehabilitation is needed, and/or the asset is reaching the end of its useful life. City staff input was utilized to identify known pipe condition issues in the distribution system.

5) City-Identified
   City-identified triggers include areas of pipeline City staff have identified that is anticipated to be replaced (such as replacing 4-inch pipe with a larger size transmission). Policies that impact an improvement, such as requiring new 10-inch pipe for any improvements along Colleyville Boulevard or the defined looping and dead end criteria, can also activate this trigger.

6) Operational
   Operational triggers are activated when an improvement will provide increased operational benefit, such as decommissioning aged infrastructure. Looping and dead end requirements would also be captured in this trigger (as those types of improvements would improve water quality and minimize flushing requirements).

Once these triggers were identified, any area within the existing system that activated one of these criteria was identified to be addressed through improvements. A modeling analysis was conducted to determine the most cost-effective improvements to implement, while providing the maximum impact with minimal infrastructure for current and future system growth.

Improvements that shared the same triggers where then categorized into alphabetical groupings based on proximity and need being addressed. For example, a low fire flow in one portion of town may be addressed by multiple new pipelines. These pipelines would share the same trigger, and the same geographic area. Therefore, they would be grouped into one alphabetized category (i.e., Group C).

The improvements were then divided into two categories:

- Near-term, based on immediate need and current (2014) model
- Long-term, based on future (2034) model with near-term improvements implemented

The improvements were then ranked numerically, with near-term ranked first. The most critical, immediate needs were given highest priority, and consideration was also given to the greatest service area impacts for each improvement. The resultant project identification and rankings list, located in Appendix D, provides the City with a directory of the most critical needs addressed in near-term and long-term projects.

8.2 Renewal and Replacement Projects

Approximately 120,000 LF of existing water line has been identified for replacement with the proposed Capital Improvements Plan. A Congressional Budget Office study was completed in 2002 to evaluate the investment levels required for water and wastewater infrastructure. While the study did not conclude a target wastewater line replacement program, it did recommend a water replacement program from 0.6% to 1.0% of existing water lines per year. The identified projects equal an average replacement rate of 0.6% over the course of the 20 year CIP. Therefore, additional renewal and replacement projects have not been identified, as it is anticipated that recommended projects will accommodate those needs.

8.3 Water Zone Restrictions

It is recommended to convert the zone restrictions to City-wide restrictions based on odd/even address watering schedules, or similar. TRWD has required a watering schedule for their major customers as follows:

- Monday- No watering allowed
- Tuesdays and Fridays- Non-residential sites
- Wednesdays and Saturdays- Residential addresses ending in even numbers
- Thursdays and Sundays- Residential addresses ending in odd numbers

This schedule format is recommended for the City. However, it is recommended that this schedule be closely coordinated with the TRA and the TRWD to ensure that it meets the best interest of their regional usage patterns.

These measures should be implemented as soon as feasible in a non-peak usage time (from October through February), to allow customers adequate time to reprogram irrigation timers and to raise public

awareness of the new restrictions. Implementing new restrictions during a non-peak time will also reduce the possibility of higher demand caused by unequal customer implementation of the new restrictions.

A City-wide educational effort is recommended to gain public participation in implementing the new watering restrictions. Active community participation, such as water sprinkler programming seminars, circulars attached to water bills, and local media advertisements, will raise awareness of the new restrictions and encourage public acceptance and involvement.

8.4 Operations and Energy Efficiency Recommendations

There are a significant number of operational considerations related to water distribution systems, as many of the factors associated with the system are interconnected. Some of the major operational issues are described below.

8.4.1 Tank Levels

As previously identified, the target storage in the system is 15% of the maximum day demand. For operational purposes, the 2012 Edition of the 10 States Standards recommends that the minimum storage capacity for systems not providing fire protection should be equal to average daily consumption. However, fire flow storage requirements may be reduced when the source facilities have sufficient capacity with standby power to supplement peak demands of the system (such as the TRA supply system to Colleyville). Additionally, the 10 States Standards also recommends that excessive storage capacity be avoided to prevent potential water quality deterioration problems.

For operational purposes, the current average day demand is compared to the existing system storage volume in Table 34. The current average day demand is also compared to the storage volume that would be available following construction of a new 1-MG tank. Additionally, the projected future average day demand (based on the TWDB per capita demand projection) is compared to the future storage volume.

Table 34: Current and Future ADD Versus Available Storage at Peak Tank Levels

Condition	Demand (MGD)	Storage (MG)	Ratio of Demand to Storage (%)
Current ADD with existing storage	7.03	8.5	83
Current ADD with additional 1-MG storage tank	7.03	9.5	74
Future ADD	9.38	9.5	99

At maximum tower levels, all scenarios maintain ADD storage. During current ADD or less, tower levels may be reduced to achieve greater tank turnover. However, as the demand exceeds the ADD in the current scenario, tank levels should be raised as well.

Under the future ADD scenario, the tower levels should be maintained at maximum level to provide the recommended storage. Additionally, LD Lockett PS and TRA meter station parameters should be specified to achieve full storage levels prior to peak demand periods in order to improve resiliency to emergency events.

8.4.2 Tank Mixing and Cycling

Modeling for the water age study assumed tanks in the system were completely mixed. The degree of mixing within tanks depends on a number of factors, including the size, geometry, fill/draw rates, inlet/outlet piping, and tank cycle range. Inadequate mixing can result in tank stratification, with the result being that water in the top level remaining in the tank for extended periods that lead to water quality issues.

The tanks should be evaluated to identify potential issues related to inadequate mixing. The following evaluation methods are available:

1. Desktop study. This evaluation would consist of review of plans and/or as-built records for the tanks, as well as fill and draw data, to determine if further study is warranted.
2. Field investigation. Water quality sampling would be performed on each tank in question to determine the degree of stratification and water quality degradation in the tank.
3. CFD evaluation. Computation fluid dynamics modeling of the tank could be performed to assess potential changes to tank mixing based on operational changes or installation of tank mixing devices.

Tank mixing is important for water quality. However, it is only one component. Adequate turnover of the tank, which limits hydraulic detention time in the tank, is also necessary. For example, in a completely mixed tank, if 25% of the water is replaced each day, the average hydraulic detention time for the water in the tank will be 4 days. However, if 50% of the water is replaced each day, the average hydraulic detention time will be only 2 days.

8.4.3 Water Age Versus Residual

As mentioned in Section 7.1.1, water age is typically an indicator of disinfection by-products (DBPs) for systems that utilize chlorine disinfection as their primary disinfectant. As the water ages, more DBPs are formed. These levels can approach or exceed EPA limits for those constituents, depending on total age, organic carbons in the water, and water temperature. Water ages exceeding 3 days are generally considered an area of concern.

However, the City currently uses chloramines for all disinfection. Chloramines produce far less regulated DBPs than typical free chlorine. The negative aspect to chloramine use is that the residuals disappear fairly quickly, and nitrification becomes a concern when low chloramine levels are present. Therefore, water ages over 3 days are a concern for chloraminated systems for reasons of low residual levels and the possibility for nitrification.

TCEQ recommends the following steps to limiting nitrification and improving water quality within chloraminated systems:

- Optimize the chloramination process.
  This would require bench top studies to evaluate the current water supply by TRA, which may change seasonally, and evaluating what levels of chlorine and ammonia must be fed to maintain the proper residual.

- Reduce water age.
  This has been accomplished to the extent possible through the existing infrastructure by the proposed projects. Additional steps the City could take would include implementation of a flushing program targeting areas of concern, implementation of tank mixing apparatus (which come in solar-powered models to maintain energy efficiency) and seasonally adjusting the tower levels, depending on demand. However, care must be taken to maintain the adequate storage requirements within each tank to cover average day demand.
- Replace aging infrastructure.
  This is accomplished through the implementation of the proposed Capital Improvements Plan.

Executing these recommended steps will assist in reducing overall water age, while improving chloramine residual throughout the system.

8.4.4 Pressure-Reducing Valves

The PRVs recommended between the high and low pressure planes in Water Group B are intended to allow flow from the high plane to the northwest side of the low plane to maintain pressures in that area during high demand periods or emergency events.

The settings on the PRVs could impact the system in a number of ways:

- Increase downstream (low plane) pressure setting
  - Higher flows from high zone to low zone
  - More turnover on high plane system, which reduces water age in the high plane
  - Additional flow needed through the LD Lockett PS, which increases operational costs
  - Potential impacts to emergency storage in high plane
- Decrease downstream (low plane) pressure setting
  - Lower flows from high zone to low zone
  - Less turnover on high plane system
  - Lower energy costs for high plane pumps at LD Lockett

Pressure settings should be evaluated regularly and may need to be adjusted seasonally based on general patterns in system demands and performance of the distribution system. An initial set point of 35 psi for the northern PRV (PRV-2), and 50 psi for the southern valve (PRV-1) is recommended, and was utilized in all scenarios with PRVs implemented.

8.5 Water Infrastructure Improvements

See Appendix D for detailed infrastructure improvements, cost estimates descriptions, and recommendations. A contingency factor of 20% has been applied to accommodate unforeseen design considerations, and changes in market pricing,

A summary of proposed projects, time-frame for implementation, and forecasted costs is displayed in Table 35.

Table 35: Water Capital Improvements Summary

Project Identification				Schedule			2015 Cost ($1000)[1]
Project	**Description**	**Location**	**Flexibility**	**Primary Trigger**	**Secondary Trigger**	**Trigger Date**	**OPCC**
1	Group Y	Area 1	Low	City-Identified	None	Oct-15	$1,363
2	Group Z	Area 2	Low	City-Identified	None	Oct-15	$627
3	Group A	Low Plane	Low	Regulatory	Capacity	Oct-15	$3,844
4	Group AB	Area 4	Low	City-Identified	None	Oct-16	$372
5	Group AC	Area 5	Low	City-Identified	None	Oct-16	$1,427
6	Group AA	Area 3	Low	City-Identified	None	Oct-17	$619
7	Group U	High Plane	High	Condition	Operational	Oct-17	$104
8	Group B	High/Low	Low	Capacity	Operational	Oct-18	$312
9	Group V	High/Low	High	Condition	Operational	Oct-18	$217
10	Group E	Low Plane	Medium	Fire Flow	City-Identified	Oct-18	$487
11	Group D	Low Plane	Medium	Condition	Fire Flow	Oct-19	$2,599
12	Group F	Low Plane	Medium	Condition	Operational	Oct-20	$2,328
13	Group L	Low Plane	Medium	Fire Flow	City-Identified	Oct-21	$1,333
14	Group I	High Plane	Medium	Fire Flow	Operational	Oct-21	$1,374
15	Group G	Low Plane	Medium	Fire Flow	Condition	Oct-23	$4,025
16	Group H	Low Plane	Medium	Condition	Operational	Oct-25	$1,636
17	Group J	Low Plane	Medium	Fire Flow	None	Oct-26	$985
18	Group K	Low Plane	Medium	Fire Flow	None	Oct-27	$5,414
19	Group M	Low Plane	Medium	Fire Flow	None	Oct-28	$2,638
20	Group W	Low Plane	Medium	Condition	City-Identified	Oct-29	$948
21	Group X	Low Plane	High	Capacity	City-Identified	Oct-30	$403
22	Group C	Low Plane	Medium	Capacity	Operational	Oct-30	$763
23	Group Q	Low Plane	Medium	City-Identified	Operational	Oct-30	$1,138
24	Group N	Low Plane	Medium	Fire Flow	Condition	Oct-31	$1,553
25	Group R	Low Plane	High	City-Identified	Condition	Oct-32	$711
26	Group S	High Plane	High	City-Identified	Operational	Oct-32	$104
27	Group T	Low Plane	High	City-Identified	None	Oct-33	$2,540
				Total 2015 OPCC:			**$39,863**

[1] Project Costs are the Opinion of Probable Construction Costs (OPCC) based in year 2015, and include Engineering and Contingency. A 3 percent escalation for inflation to the Trigger Date month and year has been added for a Forecasted Cost in Appendix D.

Figure 26 illustrates the breakout of projects by primary trigger, based on percentage of forecasted total project costs.

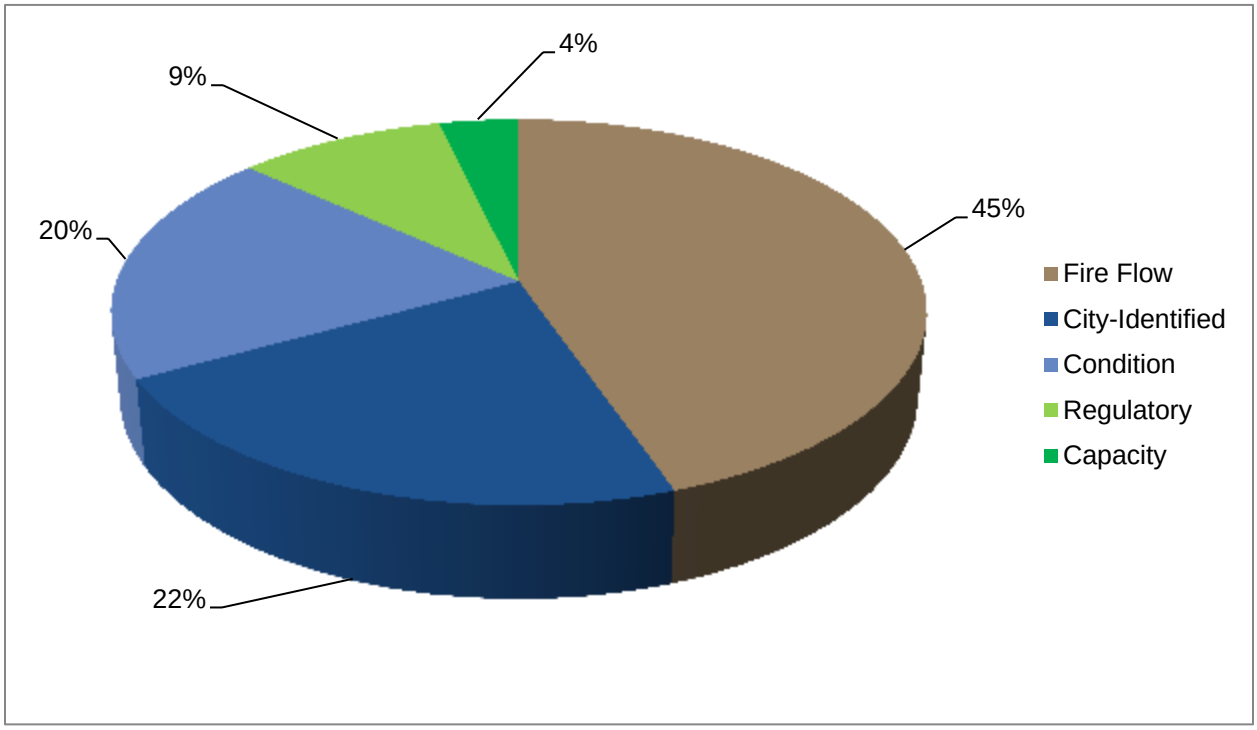


Figure 26: Distribution of Water Project 2015 OPCC by Primary Trigger

9.0 Existing Wastewater System

9.1 Overview

The City's water system consists of the items summarized below:

- 838,997 linear feet of sanitary sewers;
- 3,029 manholes; and
- Three lift stations: Overland Trail, Reserve, and a third (Lift Station 839) that is anticipated to be transferred to private ownership. For the purposes of this report, Lift Station 839 was modeled but not evaluated during the lift station facility investigation conducted by Garver.

An overall map of the existing wastewater system is included as Exhibit 28.

The City of Colleyville has an agreement in place to discharge wastewater to the TRA. Twelve major sewer drainage basins exist within the town, and send flow to two TRA interceptors. A northern interceptor follows Big Bear Creek, ranges in size from 12-inch to 15-inch, and accepts four of the twelve major basins. A southern interceptor runs along Little Bear Creek, ranges in size from 36-inch to 42-inch, and accepts the remaining eight drainage basins. These major drainage basins are displayed in Exhibit 30. Several smaller drainage basins (accounting for approximately 30.6% of the total flow) also exist, and enter the two interceptors at various connections along each pipeline.

Several studies were conducted as a part of this Master Plan to evaluate the condition of the existing infrastructure and evaluate current flow rates. Those items include:

- Pipeline and facility condition assessment workshop
- Manhole survey and condition assessment
- Lift station evaluation and condition assessment
- Sewer basin temporary flow monitoring

Descriptions of those efforts and the resultant findings are summarized in the following subsections.

9.2 Sanitary Sewer

The existing sanitary sewer infrastructure varies greatly in age, material utilized, and condition. The City's core contains the oldest infrastructure and associated piping, while the remaining City area has been developed fairly recently and contains a large percentage of PVC pipelines. An inventory of existing pipelines by diameter is displayed in Table 36.

Table 36: Wastewater System Sewer Inventory

Size (in.)	Total Length (ft)	Portion of System (%)
4	5,232	0.6
6	548,874	65.4
8	220,081	26.2
10	38,026	4.5
12	17,248	2.1
15	5,758	0.7
18	2,730	0.3
21	1,048	0.1
Total	**838,997**	**100**

9.2.1 Condition Interviews

The City currently implements an aggressive sewer pipeline cleaning, assessment, and replacement program, which conducts regular cleaning and condition assessments of existing pipelines and rehabilitates or replaces those pipelines as needed. In July 2014, a workshop was conducted to identify known areas of concern relating to pipeline condition. City staff indicated the following known problem areas:

- The Quail's Path line to Quality Hill is in need of replacement
- There is a spot near the soccer fields where a 4-inch pipeline increases to 6-inch and has a tendency to go septic.
- Across Colleyville Blvd at Tinker, a 6-inch sewer pipeline across the highway needs to be upsized to an 8-inch.
- On Manning between Manholes 1930 and 1931 the pipeline is in need of replacement.
- Rustic Oaks has areas near a beaver dam that are causing flooding into the manholes.

The first four bullet points are included in the current CIP as Sewer Improvements Group H. Further information is needed to identify which manholes are affected by the beaver dam flooding, and to determine a proper course of action.

9.3 Manholes

The City's collection system contains 3,029 manholes. In order to accurately assess the relative condition of this infrastructure, a manhole system inventory and assessment was conducted. The inventory and assessment evaluated manholes on sewer lines 8-inches and greater. Manholes on secured, private property were not included due to restricted access.

The survey and assessment was comprised of a manhole lid survey to obtain horizontal and vertical survey values, and a manhole condition assessment to obtain invert elevations, evaluate the current condition of each manhole, and recommend and prioritize improvements as needed.

This information has the following beneficial effects:

- Saves operation and maintenance costs by clearly identifying problematic areas
- Reduces the chance for manhole failure
- Identifies infiltration/inflow sources
- Improves the accuracy of the City's model
- Improves planning and design of future improvements
- Extends the useful life of the manhole, upon repair

A description of these evaluations follows.

9.3.1 Manhole Lid Surveys

A City-wide survey was conducted to take vertical and horizontal survey coordinates of manhole lids with a precision of +/- 0.01 feet. The survey intended to utilize the City's geodetic control system to establish horizontal and vertical benchmarks. However, during the course of the survey it was determined the control system contained an amount of error. Therefore, NAD 83 was utilized as the standard datum for

survey values. This data was then exported into the City's existing wastewater model, and the modeled manhole locations were adjusted to match the actual surveyed data.

9.3.2 Manhole Condition Assessment

9.3.2.1 Overview

The manhole condition assessment was conducted in July 2014 by Pipeline Analysis, and is furnished as a supplemental report to this Master Plan. Each identified manhole was inspected for defects, photographed on interior and exterior, and logged by handheld GPS coordinates and address. The purpose of performing the manhole condition assessment is to allow the City to be proactive in its approach to operating and maintaining the system, reduce infiltration and inflow (I/I), and reduce the likelihood of component failure. Rehabilitation methods can be used to correct noted defects and extend the life of the manhole while improving the overall performance of the collection system.

The defect inspection included:

- Inspection of the casting/cone and manhole cover condition
- Identification of the lid type
- Inflow potential based on surrounding conditions
- Material and diameter of the manhole
- Manhole walls inspected for integrity and signs of any defects such as missing mortar or root intrusion
- Bench was checked for type and depth of debris, any signs of settlement, and any silt deposits noted with any depth of silt recorded
- Influent and effluent lines were inspected and compared to existing maps and any corrections noted

Invert measurements and rim survey information was also taken at each manhole, to enable correct modeling of the infrastructure, including pipe slopes and diameters. This data was compiled in conjunction with the City's existing wastewater system model.

9.3.2.2 Defects

Upon inspection, any visible defects were noted and photographed. One manhole may have several defects, as shown in Figures 27-30. These figures display photographed defects for Manhole 1640.

Figure 27: Manhole 1640 Light Defect- Broken Frame

Figure 28: Manhole 1640 Moderate Defect – Bad Pipe Seal

Figure 29: Manhole 1640 Defect - Evidence of Groundwater Intrusion

Figure 30: Manhole 1640 Light Defect – Defective Joint

9.3.2.3 Rehabilitation Recommendation

One of eight groups of rehabilitation methods were then recommended to address each defect. Those rehabilitation method groups are identified in the following Table 37. Individual rehabilitation methods are identified in the report.

Table 37: Manhole Rehabilitation Methods

Group	Description
1	Replace manhole ring and cover with water-tight ring and cover, or install T-cone stopper/cap in cleanout
2	Realign and seal manhole ring and cover
3	Raise manhole or mainline cleanout to grade
4	Repair chimney/cone and coat
5	Clean manhole, remove roots, repair as needed, and coat with rigid polyurethane or equal
6	Repair/construct manhole bench and invert
7	Install stainless steel inflow protector insert
8	Stop I/I, clean, repair pipe seal and/or seam and seal coat area

Costs were then associated with each rehabilitation group, depending on the severity and condition of the identified defect. A priority level of either 1 (repair needing attention as soon as practical) or 2 (repair needing attention as funding is available) was also generated, based on severity of the defects, potential for I/I, and root presence. As an example, the defects noted in Figures 26-29 resulted in a recommendation of Priority 2 Group 5 improvements totaling $4,219. A sample Priority 1 defect is displayed in Figure 31.

Figure 31: Manhole 2158 Severe Defect - Priority 1 Debris and Bad Pipe Seal

9.3.2.4 Summary

Of the 1,134 manholes inspected, 393 were noted as requiring rehabilitation. 85 were ranked as Priority 1 improvements needed, while 308 ranked as Priority 2. The distribution of the manholes that were inspected was scattered across the majority of the city. These results are likely a representative sample for the collection system, and it is expected that similar results would be found if the remainder of the manholes were inspected. Based on that conclusion, approximately 663 manholes would have defects requiring repairs of some type in the currently uninspected portion of the collection system. A summary of the current and projected manhole rehabilitation estimates are in the following Table 38 and Figure 32.

Table 38: Manhole Condition Assessment Summary

		Requiring Rehabilitation		Replacement by Priority	
	Manholes Assessed	Manholes	% of Total	Priority 1	Priority 2
Surveyed	1,134	393	35	85	308
Remaining (projected)	1,895	663	35	142	521
Total (projected)	**3,029**	**1,055**	**35**	**227**	**829**

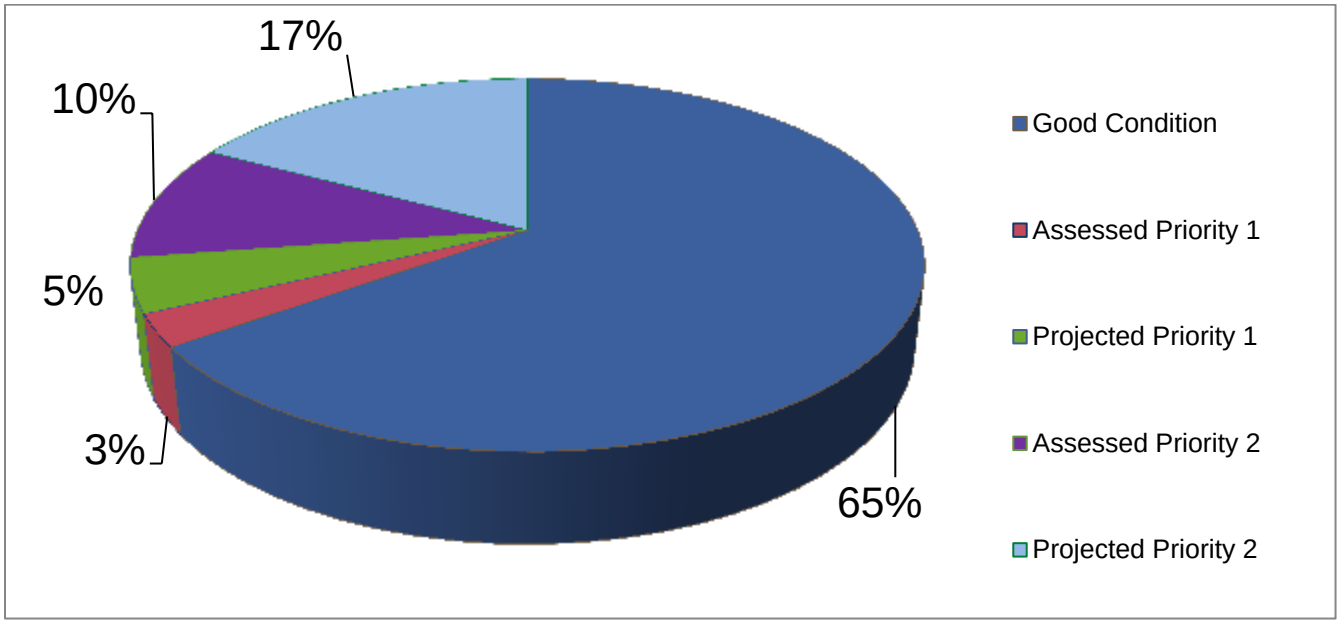


Figure 32: Assessed and Projected Manhole Conditions

An assessment of the remaining manholes is recommended to continue this effort and continue to update the City's GIS layers with more accurate data. Projects have been identified within the Capital Improvements Plan to address identified Priority 1 and 2 manholes, conduct an assessment of the remaining manholes, and address those anticipated Priority 1 and 2 manholes.

9.4 Lift Stations

The City currently owns and operates three lift stations, with plans to transfer one of the lift stations to a private owner. The following lift stations were also previously identified but have been removed from the collection system:

- Lift Station 988 on Colleyville Blvd at Tennison has been removed.
- Lift Stations 1091 on Emerald Drive near the railroad has been abandoned
- Lift Station 699 on Briarcliff Ct has been abandoned.

As such, these lift stations were not included in the model. Therefore, this Plan considers only the Overland Trail and Reserve lift stations. Table 39 presents the two lift stations' description, size, and overall condition.

Table 39: Wastewater System Lift Stations

Facility	Description and Size	Overall Condition
Overland Trail Lift Station	70 GPM and 5 hp each, two total	Good
Reserve Lift Station	98 GPM and 5 hp each, two total	Good
General Physical Condition Rating Guidelines: *Good: no immediate attention required.* *Fair: requires some initial repair to remain in adequate working condition.* *Poor: requires replacement or reconstruction in the immediate future.*		

This Master Plan conducted site assessments for both lift stations. Those assessments are summarized in the following section, and included as Appendix C.

9.4.1 Lift Station Site Evaluations

9.4.1.1 Overview

Garver conducted a site assessment of both lift stations in March, 2014. The assessment included evaluation of service areas, record documents, O&M maintenance records, equipment cut sheets, surveys, and associated code requirements. The findings from that assessment were then compared to TCEQ regulations, and a list of proposed recommendations was developed. The detailed evaluations is located in Appendix C of this Master Plan.

<u>Overland Trail Lift Station</u>
This lift station was constructed in 1990 and renovated in 2010. It services a small area in the surrounding neighborhood, comprising of approximately 30 houses. The flow is conveyed to the lift station by gravity, where it is then pumped to the Big Bear Creek TRA interceptor.

The lift station is controlled by a series of float switches, with remote SCADA notification of alarms. VFDs with soft starters were recently installed in 2010 to convert single phase to three phase power at the location. The pumps are powered by 5 hp motors. Site security is maintained by chain link fence.

<u>The Reserve Lift Station</u>
This lift station was constructed in 2007 as part of The Reserve subdivision. The lift station services the surrounding neighborhood, which includes approximately 40 houses. At build-out, the station is anticipated to serve 77 houses. The flow is conveyed to the lift station by gravity, and then pumped to the 36-inch Little Bear Creek TRA interceptor.

The lift station is also controlled by a series of float switches, with remote SCADA notification of alarms. The pumps are on/off, with VFDs utilized to convert the incoming 3-phase power. Site security is maintained by a brick fence.

9.4.1.2 Findings

The assessment revealed the following findings for each site:

Overland Trail

- The station has been well maintained and is in very good condition following the renovations completed in 2010.
- The pumps have been sized to handle the demands and flows they will experience at peak flows and will be able to keep up with the sewer being delivered by gravity to the station during higher flow periods.
- The electrical and controls are in good operating condition and have not posed any problems for the City staff.
- The site security procedures for locking all the equipment hatches and panels are serving their purpose.
- There is no barbed wire atop the fence.
- The grading of the site allows for proper drainage of the site without ponding or flooding.
- The vent for the lift station is smaller than the recommended size per Chapter 217.
- The impellers on the pumps are routinely becoming clogged from material in the wet well.

The Reserve

- The station has been well maintained over the past seven years and is in very good condition.
- The pumps have been sized to handle the demands and flows they will experience at peak flows and will be able to keep up with the sewer being delivered by gravity to the station during higher flows periods.
- The electrical and controls are in good operating condition and have not posed any problems for the City staff.
- The site security procedures for locking all the equipment hatches and panels are serving their purpose.
- There is no barbed wire atop the fence.
- The grading of the site allows for proper drainage of the site without ponding or flooding.
- There is some grease build up in the wet well.
- The top of the pump rails are exhibiting rust which could be a result of corrosion.

9.4.1.3 Recommendations

It is recommended to increase the vent size to 4” and install barbed wire at Overland Trail to comply with TCEQ regulations. It is also recommended to coordinate with the pump VFD manufacturer to increase ramp time, and allow a faster start to the pumps. This will likely reduce instances of clogging.

It is also recommended to implement barbed wire at The Reserve lift station, per TCEQ regulations. The grease accumulation should be monitored, and a liquid degreaser added as needed. Several products (such as Dawn Chemical or Continental Research Corporation Floating Lift Station Degreaser) are

available which are not harmful to treatment systems, biodegradable, and effective at reducing grease build-up. Additionally, the pump rails should be monitored for corrosion, and replaced with higher grade stainless steel if condition becomes a concern.

The total cost to implement the fencing and vent improvements is anticipated to be below $10,000. Therefore, this has not been identified within the Master Plan as a CIP project.

9.5 Flow Monitoring

The purpose of the flow monitoring study is to establish current dry and wet weather flows at key locations and major drainage basins within the City. This data is then used to calibrate the hydraulic model, and identify areas with the greatest I/I for future investigation.

Pipeline Analysis conducted the sanitary sewer flow monitoring from April 13th, 2014 to June 14th, 2014. Data and key findings were presented in a report, dated June 2014, that accompanies this Master Plan. The report presents wastewater flow data and rainfall totals, along with RDII and drainage basin rankings. The modeling section of this Master Plan further refines the data by applying diurnal demand patterns and incorporation of additional base flow considerations for revised RDII estimates.

Wastewater flow data was obtained every 15 minutes from twelve temporary flow meters installed inside manholes, on twelve individual basins as displayed in Exhibit 30. Manholes were selected based on the amount of coverage area for each individual basin. Detailed area piping maps are within the report, and a summary of each location is displayed in Table 40.

Table 40: Temporary Flow Meter Location Summary

Flow Meter	Manhole ID	Pipe Size
Colleyville 001	2832	10
Colleyville 002	1723	8
Colleyville 003	1239	10
Colleyville 004	32	10
Colleyville 005	2379	15
Colleyville 006	2213	10
Colleyville 007	673	10
Colleyville 008	1086	15
Colleyville 009	186	10
Colleyville 010	1909	12
Colleyville 011	101	8
Colleyville 012	325	18

By utilizing the available water meter connection data, it was determined that 69.4% (7,252) of the current water connections were within the extents of the flow monitoring basins. It is assumed that each water connection corresponds to a sewer connection. Therefore, this monitoring study captures 69.4% of the total sanitary sewer flow from the City.

Rainfall totals were also gathered from eight temporary rainfall gauges installed throughout the City during the duration of the monitoring period. The gauge data was implemented into a spatial analyst software (ESRI) to distribute the point preciptiation totals across the entire area and determine averaged rainfalls in each basin. Rainfall gauge location and the measured distribution of an experienced rainfall event are shown in the following Figure 33.

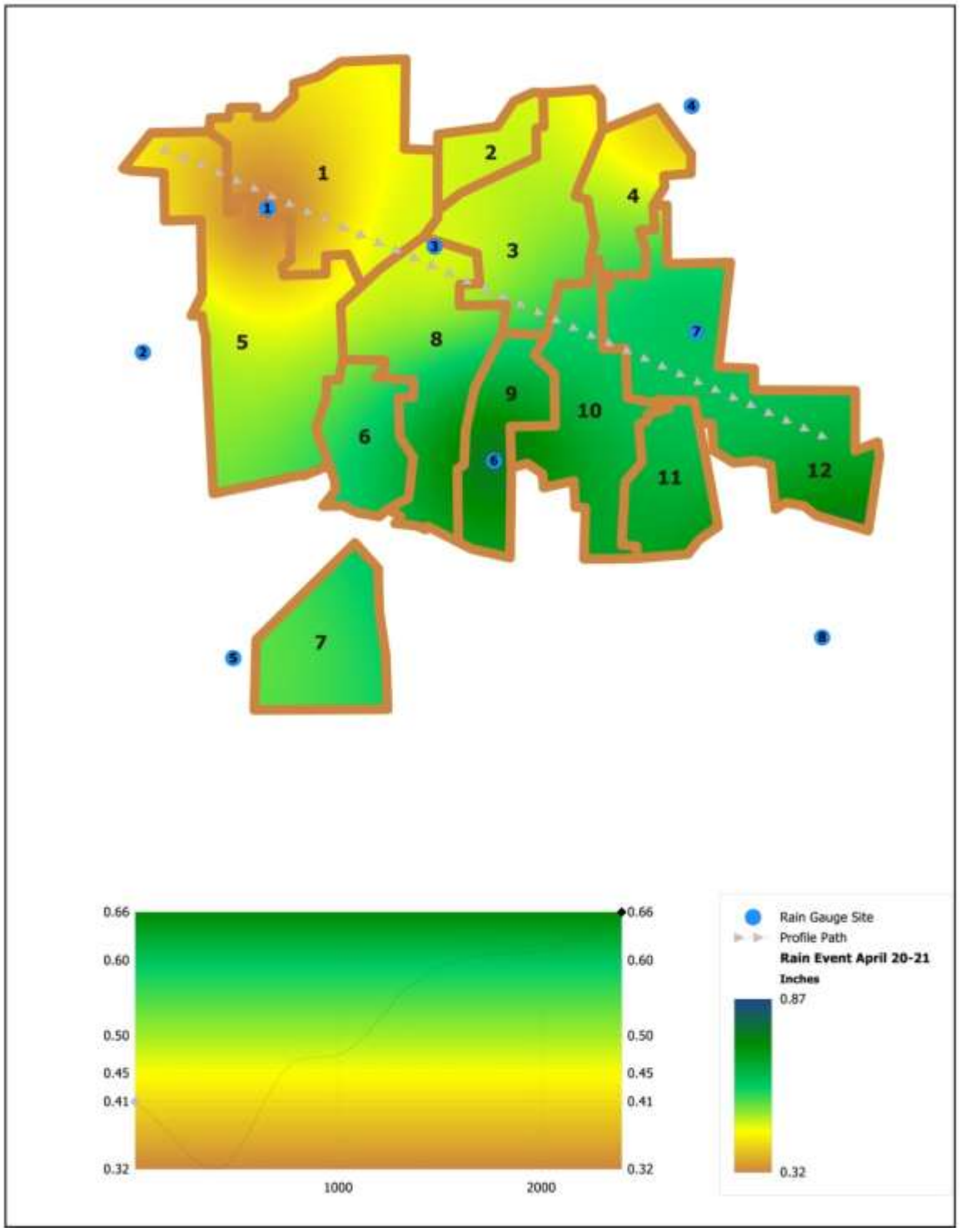


Figure 33: Temporary Rain Gauge Location and Precipitation Distribution for Sample Rain Event

9.5.1 Dry Weather Events

Total daily wastewater volume during dry weather was determined from analysis of a dry period, which occurred from April 24th-30th. The dry period is also expected to result in lower groundwater potential. Table 2 within the report displays a summary of daily average flows, peaking ratios, and velocities experienced during this time. Dry weather surcharge was not observed during the monitoring period. Relatively little silt deposition was visible, despite low scouring velocities (<2.0 fps) experienced during this timeframe. A summary of the dry weather flows per basin is displayed in Table 41.

Table 41: Average Dry Weather Flow Per Basin (MGD)

Basin	Average Flow (MGD)
001	0.08
002	0.02
003	0.03
004	0.04
005	0.12
006	0.10
007	0.08
008	0.07
009	0.07
010	0.06
011	0.08
012	0.35
Total Monitored Flow (69.4%)	1.10
Unmonitored Flow (30.6%)	0.49
Total Dry Weather Flow	**1.59**

9.5.2 Wet Weather Events

Rain events were experienced during four significant storms. Wet weather surcharging was not observed at any of the metering sites during these events. The events are summarized in the following Table 42.

Table 42: Major Rainfall Events During Flow Monitoring

Date(s)	Average Total Rain (in.)
April 20-21	0.56
May 8	0.73
May 12-14	0.87
June 8-9	1.06

A smaller event occurred on May 27th, but did not register an appreciable average across all 8 monitoring sites. The Wastewater Model Development section of this Master Plan details how peak flow rates were developed from this data and incorporated into the wastewater model.

A wet weather analysis conducted in the Pipeline Analysis report summarizes the system responses observed during the wet weather periods. The analysis computes the volume of RDII which entered the collection system during the storm. From this data, the discrete RDII was calculated for the storm event. The total discrete RDII for each meter basin was normalized using the average rainfall which established the rainfall dependent infiltration/inflow volume per inch of rainfall.

The following Figure 34 presents the results of this analysis in the ranking of the metered basins based on the observed wet weather flows, and the total RDII for the experienced storm events. It is important to note that this priority ranking does not take into account basin footage.

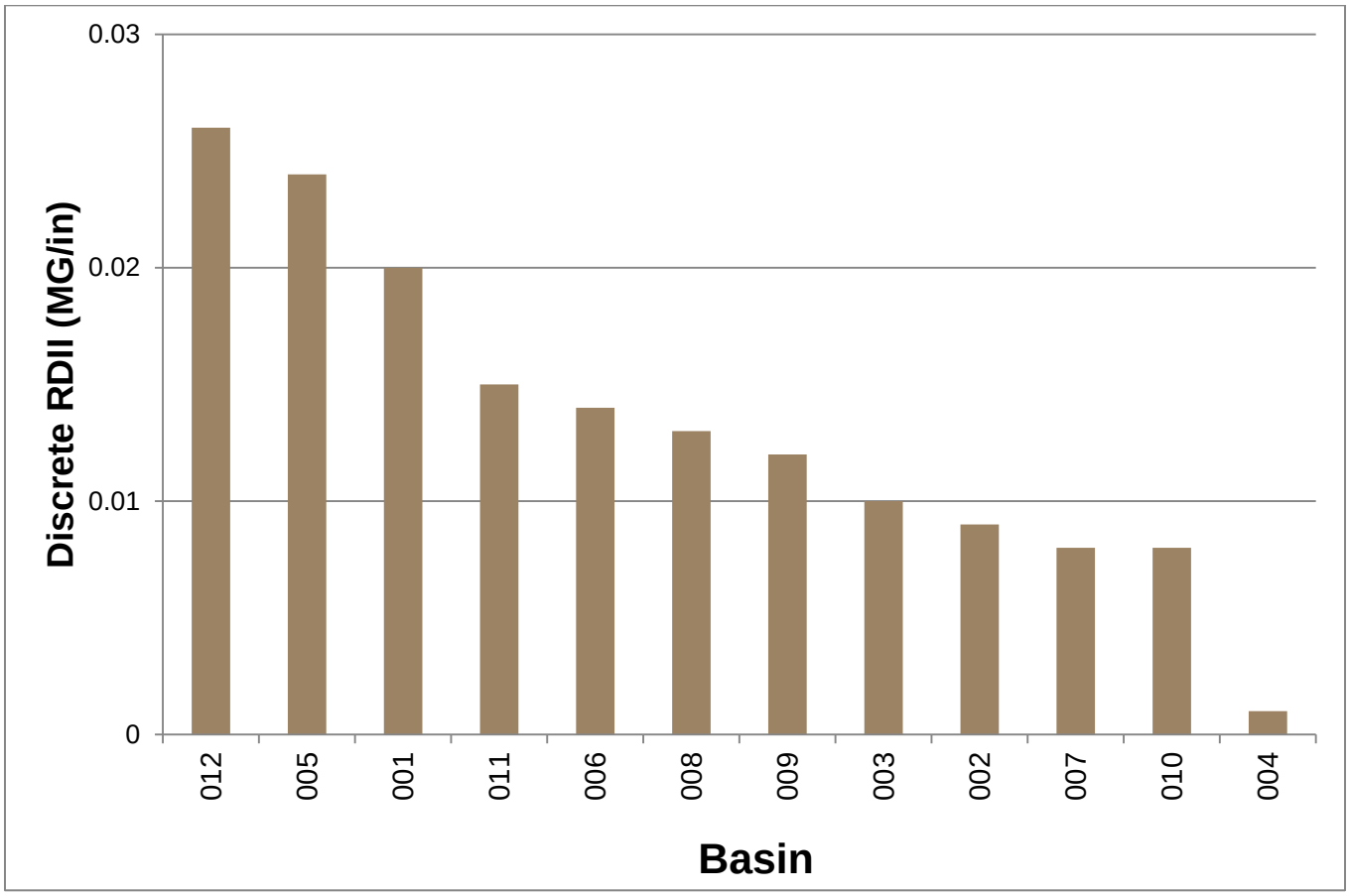


Figure 34: Ranking of Basins Based on Total RDII Volume

In order to normalize the basin rankings, the RDII has also been applied to the total linear feet of pipe within each basin. The results are displayed in the following Figure 35. These basin rankings represent the volume of flow in MG for every inch of precipitation, per 1,000 linear foot of pipe. The basins range from 500 gallons per inch of precipitation per 1,000 feet in Basin 006, to 30 gallons in Basin 004.

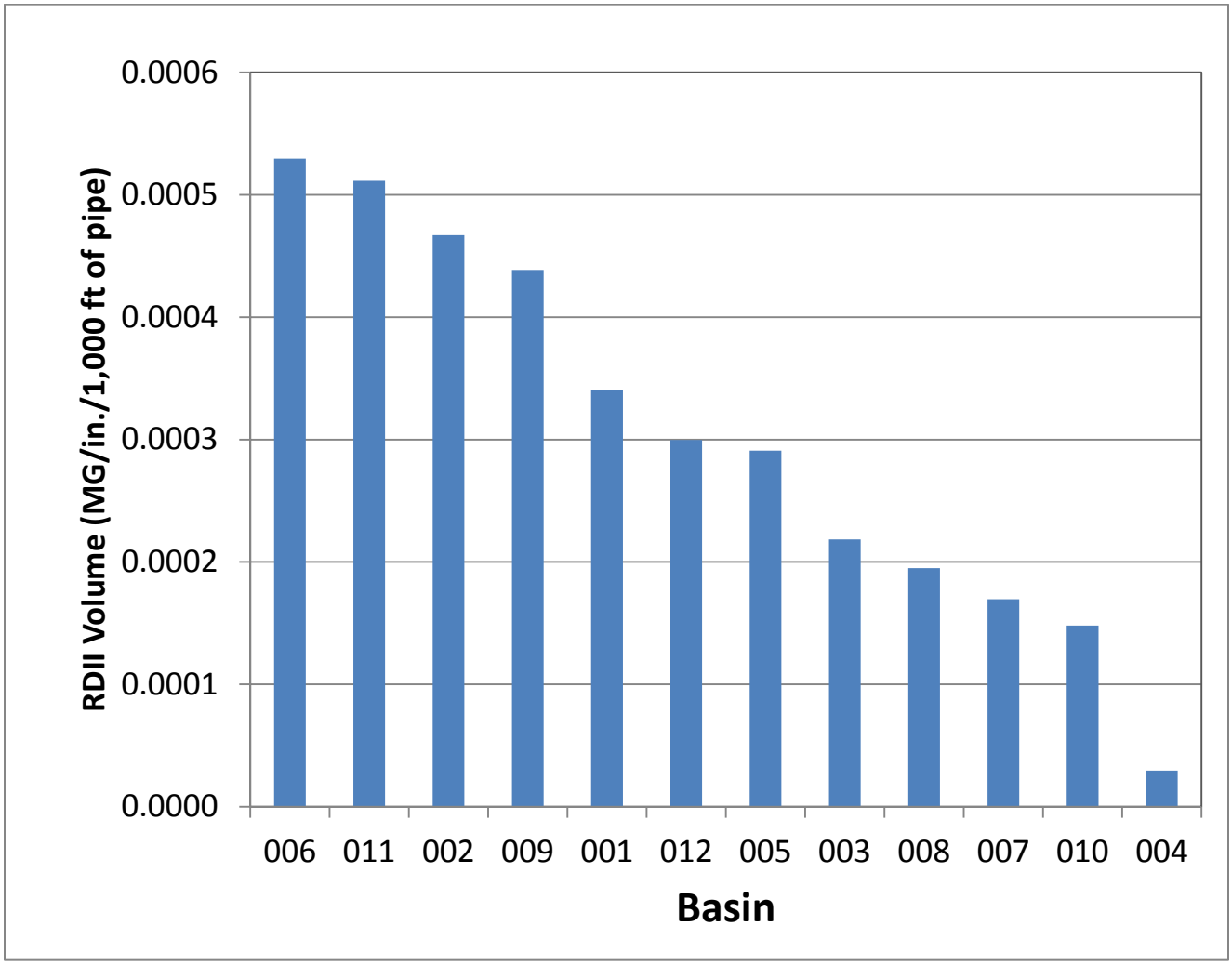


Figure 35 - Ranking of Basins By RDII Normalized to LF of Pipe in Basin

The total calculated rate under this method of RDII for the metered system is 0.16 million gallons per inch of rainfall. As previously noted, this RDII rate accounts for 69.4% of the existing sanitary sewer system. Therefore, the total RDII by this method is estimated to equal 0.23 million gallons for every inch of rainfall. Using the normal annual precipitation for Colleyville of 33.7 inches, the total annual RDII volume is projected to be 7.75 million gallons.

Further refinement of these RDII numbers is located in Section 11, with an alternative method utilize to calculate total RDII volumes. This alternative method was necessary in order to extrapolate peak flows to model within the system.

9.5.3 Average Flow Rate Evaluation

Table 43 displays the average flow rate in each basin for the duration of the flow monitoring study. A total average flow of 1.71 MGD is estimated for the entire City.

Table 43: Average Flow Per Basin Throughout Monitoring Period

Basin	Average Flow (MGD)
001	0.083
002	0.019
003	0.037
004	0.030
005	0.129
006	0.087
007	0.084
008	0.075
009	0.069
010	0.074
011	0.088
012	0.415
Total Monitored Flow (69.4%)	1.19
Unmonitored Flow (30.6%)	0.52
Total Daily Average Flow	**1.71**
Estimated Yearly Flow	**624.2**

The total average flow is 24% less than the City's average billed flow rate from TRA (2.253 MGD), and 10% less than the lowest billed month on record (1.82 MGD), as shown in Figure 36.

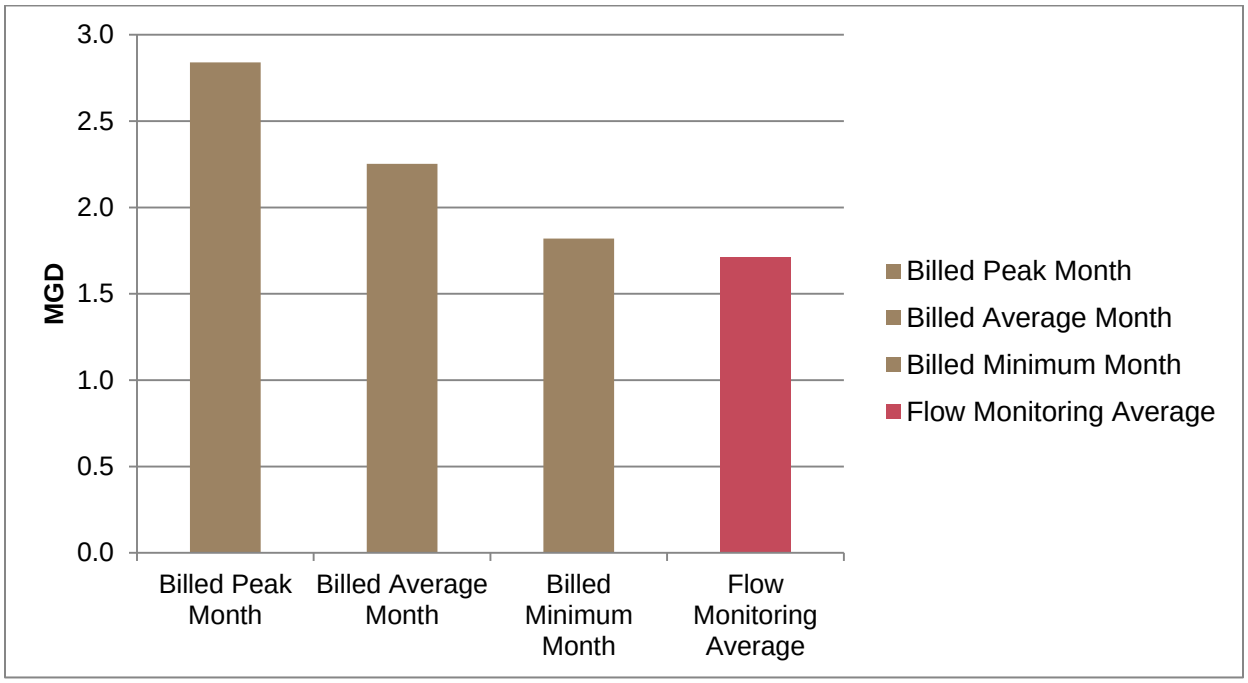


Figure 36: Comparison of Monitored to Billed Flow Rates

It is unknown what may be contributing to the difference between billed meter flow rates and field flow measurements. Possibilities include:

- Greater than 30.6% flow allocation to the unmetered portions of the City
- Portions of the City's billed usage may not be metered (i.e., flows are estimated)

Further investigation of the TRA metered flows is recommended, to ensure that all flow from the City is accurately being measured.

9.5.4 Conclusion

The following conclusions were presented in the flow monitoring program:

- Dry weather flows are adequately transported without surcharge. Despite relatively low velocities, only one site exhibited debris deposition.
- The estimated RDII volume of 7.75 MG per year is relatively minor compared to the projected yearly flow of 624.2 MG (1.2%). However, repair and rehabilitation of the sanitary sewer system will assist in reducing the RDII volume.
- Hydraulic data from the metering sites and rainfall gauges is now available and utilized for hydraulic model calibration.
- Priority ranking of the sewer basins provides guidance for analysis of future I/I identification studies, such as smoke testing or CCTV.

9.6 Infiltration

The flow monitoring study determined that there was not a significant inflow problem in the collections system pipelines. The manhole inspections did find that there is visible infiltration occurring throughout the collection system at the manholes. Though a portion of this flow likely increases during wet weather and contributes to RDII. It is also likely that a majority of this flow occurs independent of wet weather events and is therefore not captured in the RDII studies.

This groundwater that enters the system increases system flow rates and capacity requirements, and inflates the flow rate sent to TRA for treatment. The City is able to save costs on infrastructure and TRA billings by implementing manhole improvements which will limit this inflow.

In order to estimate the amount of infiltration entering the system, the average flow was compared to published flow rates of urban areas. The average flow (cumulative) for the twelve monitored basins was reported by Pipeline Analysis as 1.098 MGD. This cumulative flow rate is estimated to represent a base sewer flow of approximately 66.2 gallons per capita per day (gpcd). This estimate was developed using the following steps:

1. The sum of the average dry weather flows from each basin was calculated as 1.10 MGD.
2. The total number of matched (geocoded), non-sprinkler water meters was found to be 10,450.
3. Of the 10,450 matched, non-sprinkler water meters, it was determined that 69.4% (7,252) of them were within the extents of the flow monitoring basins.

4. The system-wide average dry weather flow was estimated to be 1.59 MGD by dividing the average flow in the flow monitoring basins by the percentage of matched, non-sprinkler meters in the area.
5. The average per capita dry weather flow was estimated by dividing the system-wide dry weather flow by the current population of 23,600. The result is approximately 67 gpcd.

Standard engineering practice is to assume indoor water usage is equivalent to base wastewater flow. Water uses for almost 1,200 households across fourteen cities in North America were analyzed for a survey of water use conducted for the American Water Works Association Research Foundation (AWWARF). The results are reported in “Residential End Use of Water” by Mayer et al. (1999). The mean daily per capita indoor usage ranged from 57.1 gpcd in Seattle to 83.5 gpcd in Eugene, and the overall mean daily per capita indoor usage was reported as 69.3 gpcd. The study also reports that the water leakage rates and implementation of water conserving fixtures impacts the average indoor usage rates.

The estimated base sanitary sewer loading for the City of Colleyville (67 gpcd) is consistent with the average indoor usage reported in the study for AWWARF (69.3 gpcd). The mean daily per capita indoor usage for the City of Colleyville is currently unknown, though the average total water usage rate City-wide is previously identified as 122 gpcd. It is reasonable to assume that indoor residential wastewater usage falls within the range between the mean value reported for the sites in Seattle (57.1 gpcd) and the base sanitary sewer loading rate estimated for the Colleyville system (67 gpcd).

Application of these numbers results in a range of estimates for base infiltration into the sewer system of 0.0 to 9.9 gpcd (0.0 to 14.2% of dry sewer flows from base infiltration), or 0 to 85.28 million gallons on a yearly basis.

10.0 Wastewater Design Criteria

Similar to the water distribution system, the wastewater system is regulated by criteria set by the TCEQ and general engineering practice. Those criteria are identified in the following Table 44, along with criteria used in designing master plan improvements.

Table 44: Wastewater Design Criteria

System Element	TCEQ			Industry/Colleyville Standards	2014 Master Plan
Flow Design	Dry weather flows + I/I			Dry weather flows plus I/I	Dry weather flows + I/I
Pipe Materials	Based on engineering judgment			PVC	PVC
Capacity	Accommodate current and future flows without surcharge			Accommodate current and future flows without surcharge	For all pipes, 75% full or less during peak flows
Wastewater Peaking Factor	Qpeak/Qavg = 4.0 for minor collectors. Main trunk, interceptor, and outfall sewers shall be designed to convey expected flow			Q	
Minimum Pipe Diameter	6-inch, excluding service laterals and force mains			Residential 6-inch, non-residential 8-inch	8-inch
Pipe Sizing and Minimum Slope	Size (in.)	Min Slope (%)	Max Slope (%)		
	6	0.50	12.35		Same
	8	0.33	8.40		Same
	10	0.25	6.23		Same
	12	0.20	4.88		Same
	15	0.15	3.62		Same
	18	0.11	2.83		Same
	21	0.09	2.30		Same
	24	0.08	1.93		Same
	27	0.06	1.65		Same

11.0 Wastewater Model Development

The wastewater collection system for the City of Colleyville consists of the service pipelines, collection pipelines, interceptors, lift stations, and connections to TRA interceptors on the north side along Big Bear Creek, and on the south side along Little Bear Creek. A hydraulic model representation of the physical system has been created using Bentley SewerGEMS V8i SELECTseries 3 (SewerGEMS). The following sections detail components of the hydraulic model development process.

11.1 Manholes

Manhole information was obtained through survey and manhole assessment. Specifically, geographical locations and rim elevations were surveyed for the manholes on sewer pipelines of size 8-inch and larger. The distances from the rim of the manhole to the invert of each incoming or outgoing pipe were collected for the manholes during the manhole assessment. These two data sets were combined to obtain invert elevations for pipelines associated with the surveyed manholes.

The majority of the manhole information was collected along the sewer lines of size 8-inch and larger. However, there were some data gaps along these lines, as well. These data gaps were due to manholes that could not be located or accessed, as well as manholes with lids that could not be removed during the field investigation process. Additionally, some 6-inch lines were included in the model for connectivity between 8-inch lines within the system. Engineering judgment was used in setting elevations where survey information was not available. Specifically, if data gaps were present between upstream and downstream manholes where data was known, the average slope was used between the known points to set manhole inverts for manholes. If the line was located at the boundary of the modeled basin (i.e., the downstream end of the line feeds the most upstream manhole in a reach or the upstream end of the line drains the most downstream manhole of a sewer basin), the minimum slope per TCEQ requirements was assumed for that particular sewer pipeline.

11.2 Sewer Pipelines

Sewer pipelines were imported into the model based on upstream and downstream manholes. The pipe inverts were defined based on the surveyed data whenever possible. When survey data was not available, either minimum slope or average slope were utilized based on data available, as described in the manholes section above. A Manning's roughness value of 0.013 was used for all pipes. All 8-inch and larger sewer lines were modeled, as well as 6-inch lines necessary to connect areas of 8-inch lines.

11.3 Lift Stations

The Overland Trail Lift Station and The Reserve Lift Station were incorporated into the model based on record drawings. Specifically, the influent gravity line, wet well, pumps, and force main were included for each of the facilities. Pump curves were included in the model using the multiple point curve option.

11.4 Connections to TRA Interceptors

An outfall was created in the SewerGEMS model at the downstream end of each of the modeled sewer basins. In general, this location corresponded to the location of a TRA interceptor. A free outfall condition was utilized for each outfall in the model, which assumes that the TRA line has adequate capacity. The

modeling implication is that the TRA interceptors did not cause any backwater effects in the modeled system. The invert elevations of the connections to the TRA system were not surveyed. Therefore, the lines connecting to the TRA system were assumed to have minimum slopes.

11.5 Spatial Delineation of Flows

Thiessen polygons were utilized in the spatial allocation of flows within the sewer basin. Thiessen polygons define areas around a set of points; for this application, the set of points was the shapefile containing all of the City's manholes. Thiessen polygons have the property that each point within the polygon is closer to that polygons sample point (manhole) than to any other sample point (manhole) in the set.

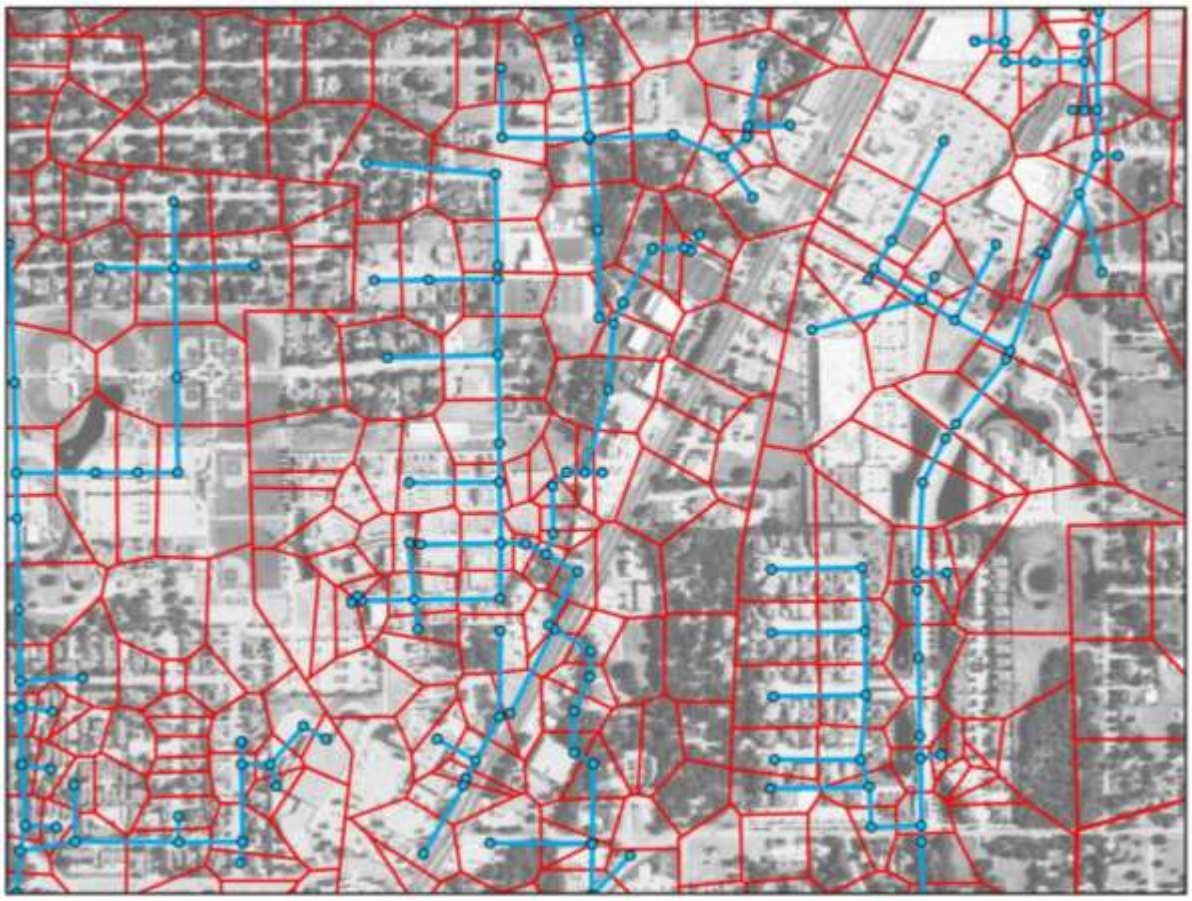


Figure 37: Thiessen Polygons Based on System Manholes

The Thiessen polygons for the area along Colleyville Boulevard, approximately between Glade Road (south) and Hall-Johnson Road (north), is shown in Figure 37. Each of the polygons outlined in red is a Thiessen polygon for a wastewater collection system manhole in the GIS database.

The wastewater system model did not contain all of the manholes and sewer lines in the system, so additional work was required to prepare the Thiessen polygons so they could be used to load the model.

Referring to Figure 37, the modeled system is shown in blue, with the circles representing model manhole locations and the lines corresponding to model pipes. There are areas of the collection system where there are manholes that are not modeled. This is apparent from Figure 37 because there are Thiessen polygons that do not contain a model manhole.

The manhole and sewer line data for the entire system was used to aggregate service basins to the first place their loading contributions would impact the model system. For example, an 8-inch sewer line might have multiple branches off of it with 6-inch lines. The Thiessen polygons associated with each of those 6-inch lines will be aggregated and applied to the model at the connection of the 6-inch line to the 8-inch line. Each Thiessen polygon was marked with the number of the manhole to which it contributes, and the Thiessen polygons associated with a given manhole were dissolved into a single polygon in the next step of the process. The dissolved Thiessen polygons were utilized for both the dry weather and wet weather load allocation. The dissolved Thiessen polygons are shown in Figure 38.

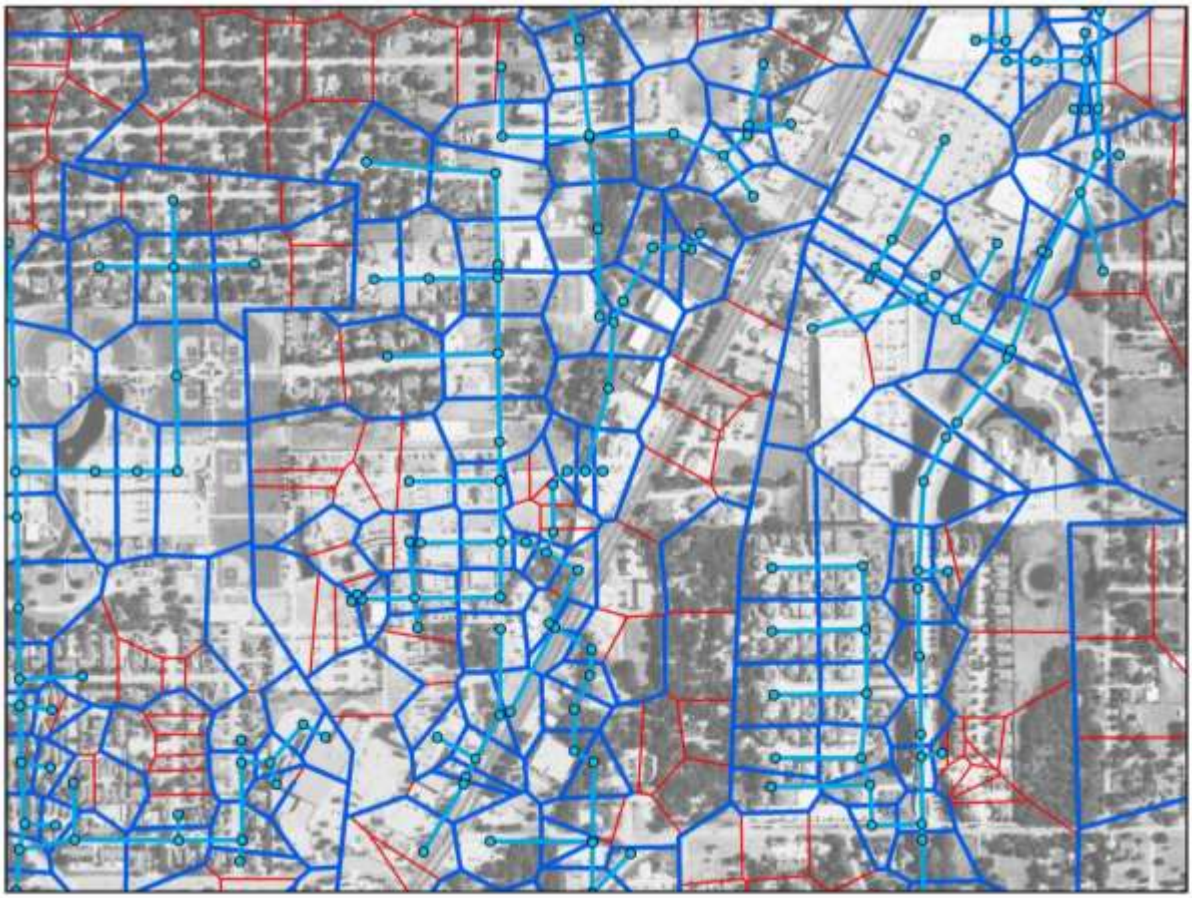


Figure 38: Dissolved Thiessen Polygons Based on Model Manholes and System Connectivity

Figure 38 shows the dissolved Thiessen polygons (blue outlines) overlain on top of the original Thiessen polygons (red outlines). The model network is shown in blue, as well. For a significant portion of the area shown in Figure 38, the original and dissolved Thiessen polygons are aligned, which results in the red outlines being completely covered. An example of this is for the manholes associated with the five parallel

sewer lines conveying flow to the N-S line along San Gabriel Avenue in the bottom right quadrant of Figure 38. In contrast, the area along Colleyville Boulevard from about Main Street to Church Street does not have any modeled manholes. This area drains to the south, and the model manhole associated with that Thiessen polygon is the one on the east side of the sewer pipe that crosses Colleyville Boulevard south of Main Street.

11.6 Dry Weather (Base) Flows

Dry weather base flows were predicated on flow monitoring data and water billing records during the low demand months of the year. The flow monitoring period of April 24-30 was used as the dry weather period. An example of the average diurnal curves during the dry weather period is shown in Figure 39.

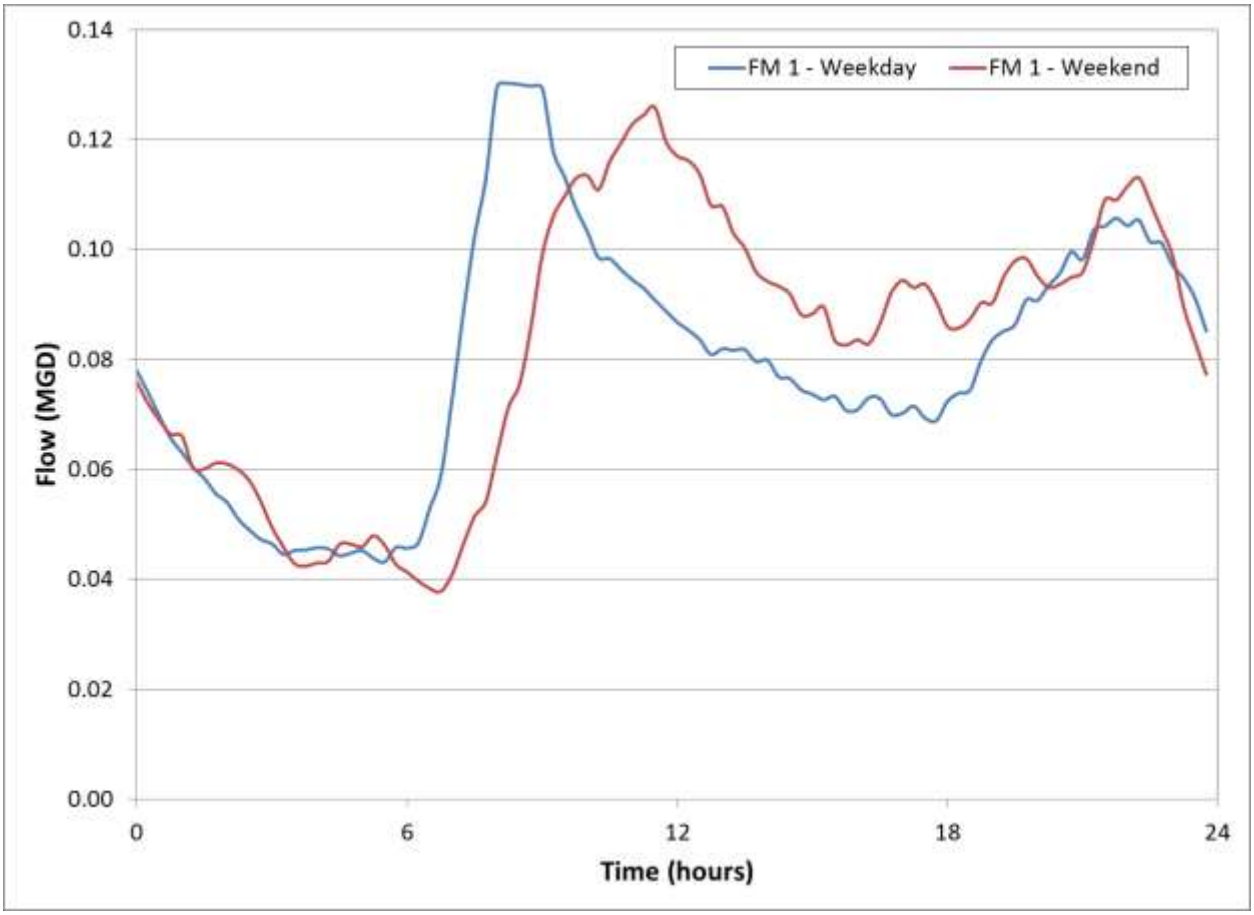


Figure 39: Average Hydrographs for FM 1 During the Dry Weather Period (April 24-30)

The following steps were used to develop and allocate the dry weather flows:

1. The average dry weather flow rate for each of the twelve flow monitoring locations was determined based on the dry weather period of April 24-30.
2. The average water usage in each flow meter basin was calculated during minimum demand months. Water meters dedicated to irrigation usage were not included in this demand aggregation because irrigation water is not returned directly to the wastewater collection system.

3. The return ratio was calculated for each basin. The return ratio is the ratio of the average dry weather (base) flow to the metered water consumption in each basin, and it represents the portion of the water used that is returned to the system.
4. The average dry weather loading for each manhole in the model was determined by finding the average demand during the minimum months within the polygon (from the dissolved Thiessen polygons) for the manhole and multiplying that value by the return ratio for the sewer basin.
5. Peaking factors based on the dry weather flow monitoring were used to determine the peak dry weather flow for each basin. For each of the twelve basins, the weekday peak flows were greater than the weekend peak flows. Therefore, the peaking factors are based on the weekday peak flow rates.
6. The peak dry weather loading for each manhole was calculated by applying the peaking factor for the basin to the average dry weather loading for the manholes within the basin.

The average and peak dry weather flow for each flow monitoring location are shown in Table 45.

Table 45: Flow Meter Dry Weather Flow Data

Flow Meter	Average Flow (MGD)	Peak Flow (MGD)	Peaking Factor
1	0.0805	0.1302	1.62
2	0.0199	0.0468	2.35
3	0.0346	0.0663	1.92
4	0.0367	0.0920	2.51
5	0.1181	0.1978	1.67
6	0.0973	0.1674	1.72
7	0.0814	0.1159	1.42
8	0.0651	0.0950	1.46
9	0.0739	0.1005	1.36
10	0.0587	0.1325	2.26
11	0.0819	0.1574	1.92
12	0.3842	0.6001	1.56

11.7 Wet Weather (RDII) Flows

11.7.1 RDII Estimates from Flow Monitoring

Flow monitoring measurements provided the total flow rate at twelve monitoring locations in the City wastewater collection system. The total wastewater flow rate is a combination of the base (dry weather) loading and the rainfall-derived infiltration and inflow (RDII). The base (dry weather) loading is comprised mainly of loading from service connections and base infiltration into the system. The RDII is the additional infiltration and inflows to the system resulting from precipitation events. Examples of RDII contributions are illegal connections to stormwater components (e.g., downspouts plumbed into the wastewater collection system), infiltration at manhole lids, and additional infiltration due to elevated groundwater levels during and following rain events.

To complete the assessment of the rainfall-derived infiltration and inflow (RDII) to the wastewater system, it is necessary to remove the base loading from the total flow signal. The total flow for FM 1 for Rainfall Even 5 during the period of June 8-9 is shown in red in Figure 40. The representative dry weather flow for FM 1 is shown in blue in the figure. Average base weekday flow rates were used for weekdays, while average base weekend flow rates were used for weekends.

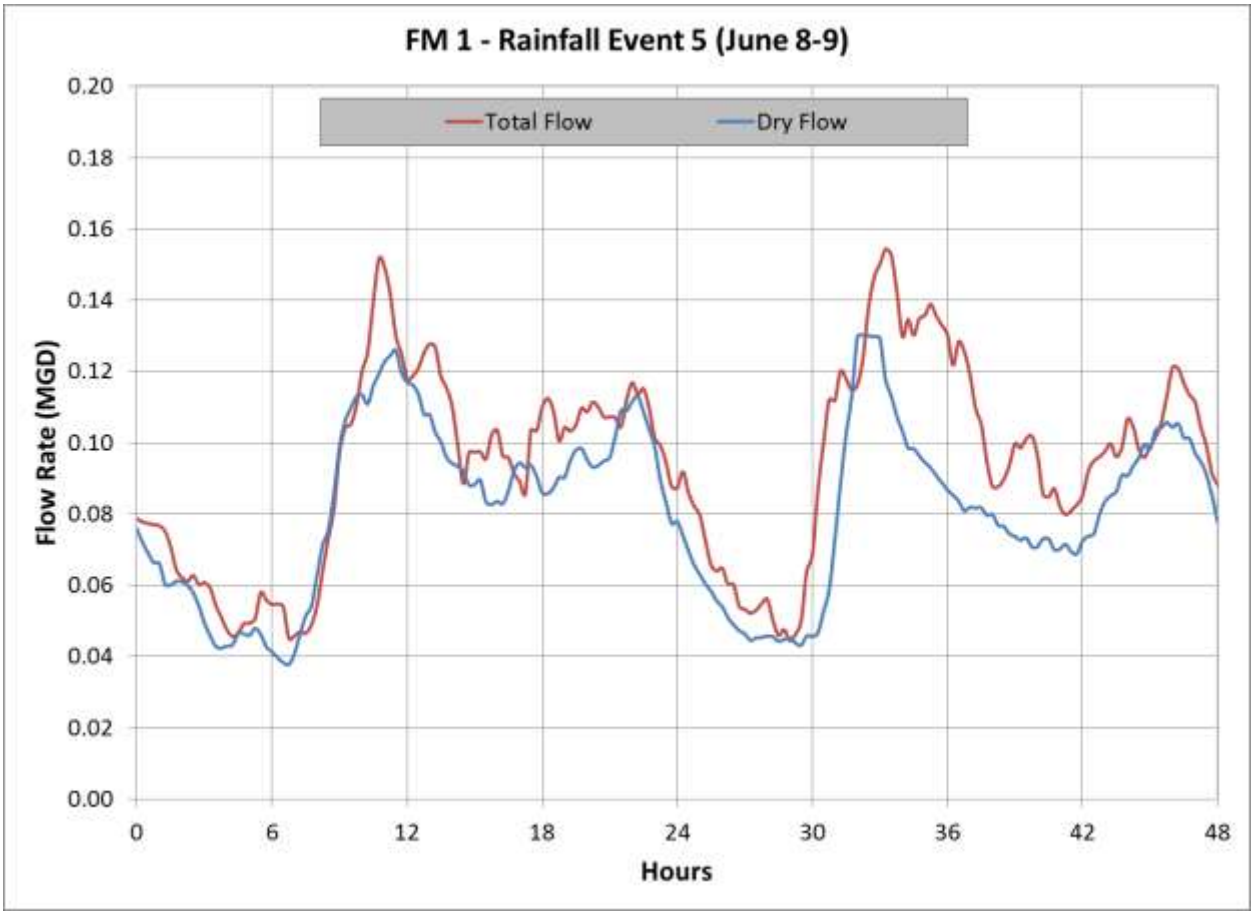


Figure 40 - Sample Rainfall Event Flow Monitoring

The average base flow rates were subtracted from the total flow rates measured during the periods associated with the precipitation events. The resulting hydrograph was the estimated RDII hydrograph for the given event. Figure 41 shows the total, representative, and estimated RDII hydrographs for June 8-9 for FM 1. The RDII hydrograph is shown in green. Additionally, the cumulative event rainfall for Rain Gauge 1 is shown in purple, and it is referenced to a secondary y-axis. Rain Gauge 1 is the rain gauge closest in proximity to FM Basin 1.

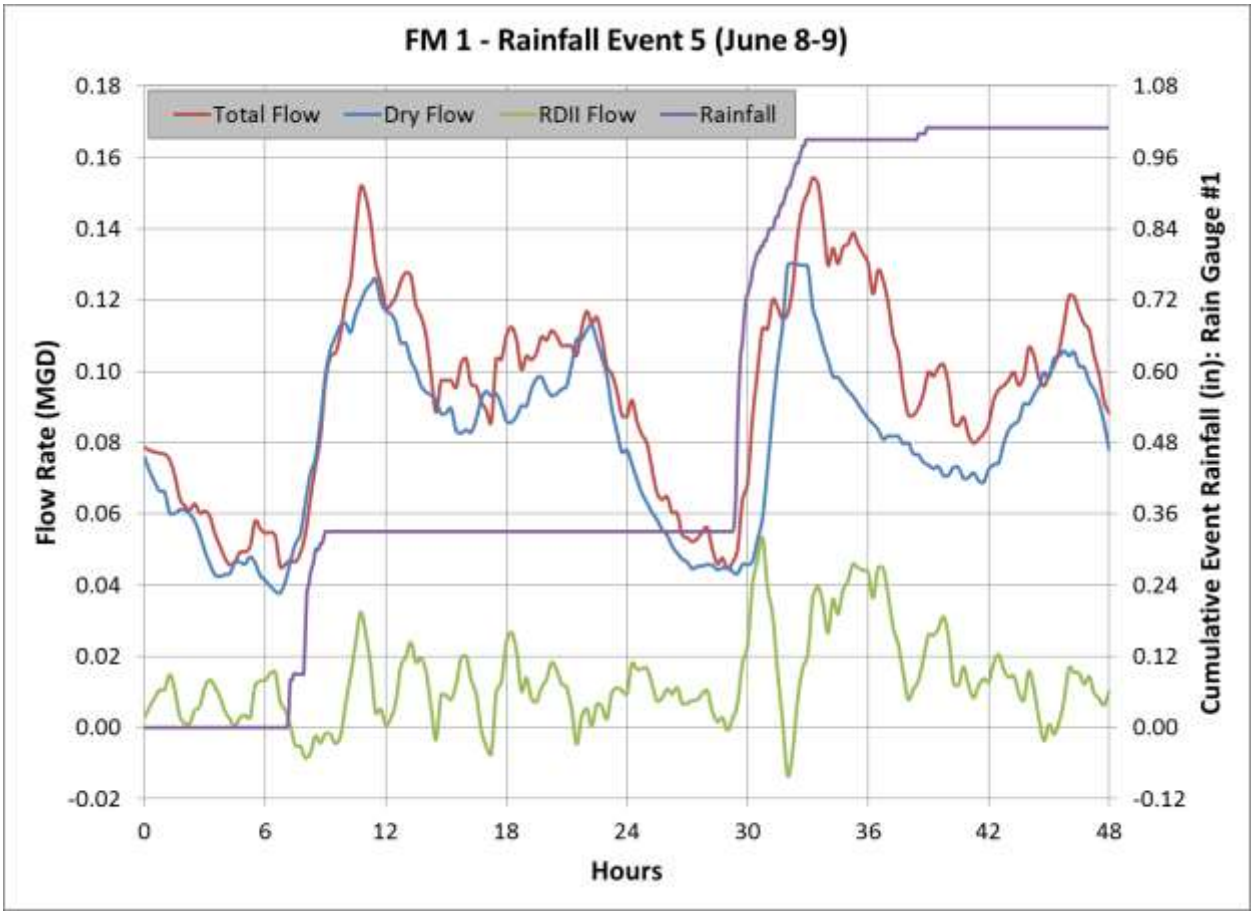


Figure 41: FM 1 Hydrographs for Rainfall Event 5

There are subtle differences in the dry weather flow diurnal curves for the two days. June 8 was a Sunday, so the dry weather flow was approximated using the weekend average, whereas June 9 was a Monday. The weekday average flow has a slightly higher morning peak, and the morning peak occurs earlier. Additionally, the mid-day base loadings are higher for the weekend average than for the weekdays.

Figure 41 shows that the RDII response for this event is fairly small relative to the base flow in the system. Additionally, there is a more significant response to the larger amount of precipitation that occurred on June 9 than for the smaller amount of rainfall on June 8. There are a number of factors that could be contributing to this. In general, larger rainfall amounts will result in more pronounced RDII values. Additionally, the rainfall on June 8 would likely increase the soil moisture content, which would make the antecedent conditions more favorable to produce RDII for the subsequent event.

The RDII hydrographs were used to estimate the RDII peak flow and total volume associated with each precipitation event during the flow monitoring period. The results for FM Basin 1 are shown in Table 46.

Table 46: FM Basin 1 Results

Rainfall Event	Dates	Rainfall (in)	RDII Peak Flow (MGD)	RDII Volume (MG)
1	April 20-21	0.41	0.0437	0.0200
2	May 8	0.71	0.0595	0.0070
3	May 12-14	0.89	0.0418	0.0150
4	May 27	0.34	0.0392	0.0030
5	June 8-9	1.00	0.0533	0.0140

11.7.2 RDII Peak Flow and Volume Coefficients

Numerous techniques exist for developing RDII flows for basins. Based on the data collected during the flow monitoring period, a method that correlates RDII peak flow and total flow to total event precipitation was utilized. Specifically, for each flow monitoring location, a linear trend line was fit to each set of data point (i.e., peak flow vs. rainfall and total RDII volume vs. rainfall). The data points and fit line are shown for FM 1 in Figure 42.

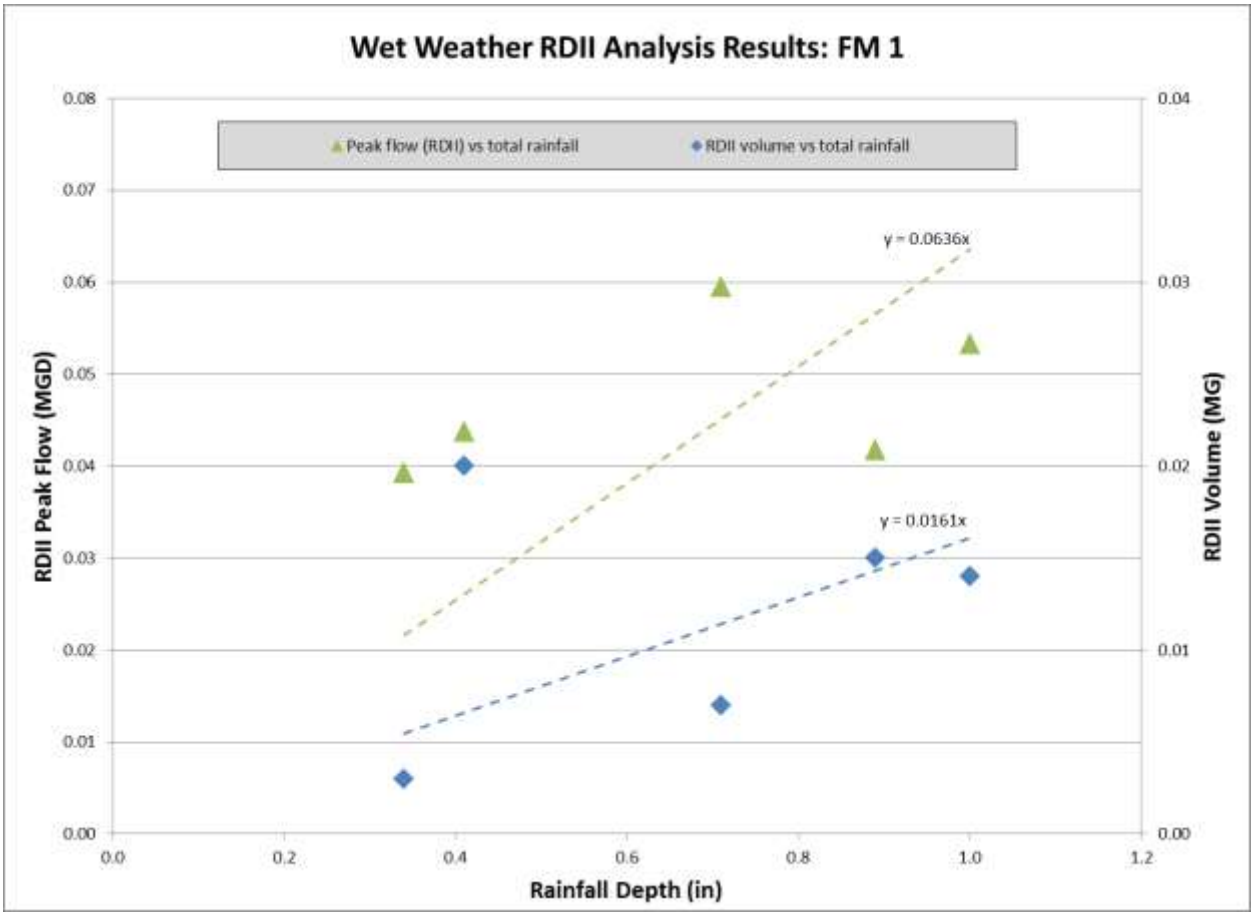


Figure 42: FM 1 Wet Weather RDII Analysis Results

The slopes of the best fit lines are used to project the RDII peak flow rate and volume for different event rainfall totals. The values for each basin are shown in Table 47. The units are MGD/inch (RDII peak flow) and MG/inch (RDII volume).

Table 47: RDII Peak Flow Slope and Volume Slope FM Basins Results

FM Basin	RDII Peak Flow Slope (MGD/inch)	RDII Volume Slope (MG/inch)
1	0.0636	0.01610
2	0.0179	0.00701
3	0.0361	0.00825
4	0.0140	0.00193
5	0.1415	0.01634
6	0.0645	0.01634
7	0.0317	0.00681
8	0.0916	0.01501
9	0.0537	0.00758
10	0.0653	0.00863
11	0.0956	0.02007
12	0.1463	0.02201

11.7.3 Design Storm

The design storm selected for the wastewater system analysis was the 5-year, 6-hour storm event. The total precipitation associated with the design event was calculated using the rainfall intensity-duration-frequency coefficients for the 5-year precipitation event for Tarrant County. The frequency coefficients for that storm are shown in Table 48.

Table 48: 5- Year Precipitation Event Intensity-Duration Frequency Coefficients

Coefficient	Value
e	0.778
b	66
d	8.5
Coefficients are for units of inches and minutes.	

The intensity for the 5-year, 6-hour storm event is calculated as 0.665 in/hr. This yields a design storm event rainfall of 3.99 inches.

11.7.4 Design Storm RDII Peak Flow

The peak flow for each basin was estimated based on the design storm event rainfall depth of 3.99 inches and the RDII peak flow coefficient. The RDII peak flow rates for each of the flow monitoring points are shown in Table 49.

Table 49: RDII Peak Flow Rates for FM Basins

FM Basin	Projected Design Storm RDII Peak Flow (MGD)
1	0.2538
2	0.0714
3	0.1440
4	0.0559
5	0.5646
6	0.2574
7	0.1265
8	0.3655
9	0.2143
10	0.2605
11	0.3814
12	0.5837

11.7.5 RDII Allocation

In order to model the impacts of RDII throughout the sewer basin, it was necessary to distribute the RDII peak flow loading to individual manholes, similar to the procedure for allocation of the base loading. For the RDII peak flow, area contributing flow to a given manhole was as the basis for spatial distribution. For example, if 25% of a given sewer basin drains to a single manhole, then the RDII peak flow assigned to that manhole was 25% of the basin total. The dissolved Thiessen polygons discussed previously were utilized for the RDII allocation, as described in the following steps:

1. The effective contributing area for each sewer basin was determined. The effective area was calculated by subtracting the area for undeveloped parcels from the total area.
2. The peak flow for each sewer basin was normalized by the effective contributing area for the sewer basin.
3. The effective contributing area for each manhole was used to determine the peak RDII loading at the manhole, using undeveloped parcel areas and the dissolved Thiessen polygons. The loading was calculated by multiplying the normalized peak flow (flow per area) by the area associated with the manhole.

11.8 Load Allocation in Non-Monitored Basins

Load allocation for non-monitored basins was based on the same principles as the loading for monitored basins. Specifically, a return ratio was used for estimating dry weather loading, and a per area value was used for RDII flow allocation. For each non-monitored sewer basin or sewer basin, the dry and wet weather factors were assigned based on values for adjacent basins using one of following two criteria:

- The larger value from two adjacent basins
- 120% of the value for an adjacent basin

As shown in Tables 47 and 49, there is variation in dry and wet weather load factor values between sewer basins in Colleyville. Therefore, the approach was to be conservative in assigning values to non-monitored basins without being over-conservative and creating capacity issues where they do not actually exist.

11.9 Future Load Allocation

The existing model loads were used as the basis for developing loading for future (2034) model scenarios. The dry and wet weather loads for the existing model were carried over to the future scenarios. The additional loads to account for future build-out were based on undeveloped parcels.

The undeveloped parcels were used to estimate the volume and spatial distribution of additional dry weather wastewater loads to account for future conditions. Undeveloped parcels were classified, based on the 2025 Land Use Plan for the City of Colleyville, using the following category groups:

- Single Family Residential
- Commercial-Industrial-Institutional
- Parks and Open Space

Dry weather load allocation was completed using an approximate number of lots for residential areas and land area for commercial-industrial-institutional parcels. The overall dry weather loading rates include a base value (150 gpd/lot for residential and 1,000 gpd/acre for commercial-industrial-institutional) and a peaking factor (2.5). Wet weather load allocation was completed based on land area. Refer to Table 50 for additional details.

Table 50: Dry and Wet Weather Load Allocation

	Dry Weather		Wet Weather	
Land Use Type	Unit	Value (gpd/unit)	Unit	Value (gpd/unit)
Residential	Lot	375	Acre	Existing per unit RDII value for basin
Commercial-Industrial-Institutional	Acre	2,500	Acre	
Parks and Open Space	Acre	0	Acre	

Undeveloped parcels were assigned to manholes based on topography and existing sewer lines. Specifically, the loads from each undeveloped parcel were assigned to the first modeled manhole to which they would contribute flows. This process is very similar to the Thiessen polygon assignment for the existing load allocation. Loads for any undeveloped parcels that drain to a basin that is not present in the model were not incorporated into the model.

12.0 Wastewater Model Results

12.1 Existing Pipe Slopes

The minimum pipe slopes for gravity sanitary sewer lines required by TCEQ are specified to achieve a velocity of 2 fps in pipes flowing at least 50% full. Sub-standard pipe slopes can lead to issues such as deposition of materials suspended in the flow and ponding of sanitary flows within the collection system.

There are 81 sewers within the modeled system that have slopes less than the TCEQ minimum required slopes. The locations of these pipes can be seen graphically in Exhibit 31.

12.2 Current Wastewater Model Results

12.2.1 Flow Capacity

Under existing dry weather flow conditions, the system has adequate capacity with the exception of four pipeline segments. The diameter, length, slope, and ratio of flow to design capacity for these four pipeline segments are shown in Table 51.

Table 51: Existing Sewer Exceptions to Meet Flow Capacity

Pipe Label	Diameter (in)	Length (ft)	Slope (ft/ft)	Flow / Design Capacity (%)
2477-2478	10	235.2	0.00010	121.5
1261-779	10	64.4	-0.00014	107.3
1480-1677	8	36.5	-0.00014	104.6
3019-3020	10	67.0	0.00100	52.7

All four of these sewer segments have sub-standard pipe slopes. Moreover, the surcharged pipes (i.e., pipes with flow to design capacity ratios greater than 100%) have slopes that are either adverse or very close to zero. Pipes with adequate slopes utilize the bottom slope of the pipes to facilitate gravity flow, pipes with flat or adverse slopes rely on higher upstream depths to provide the gradient on the water surface elevation needed to convey the wastewater flows downstream under gravity conditions. For this reason, long pipes with sub-standard slopes are particularly susceptible to surcharge conditions.

Under existing wet weather design flow conditions, the system has 56 sewer segments with inadequate capacity, as previously defined in the sewer design criteria. The majority of the lines with inadequate capacity have sub-standard slopes or are pipes that have been manually set to minimum slope within the model because either the upstream or downstream pipe invert was not obtained during the field data collection process. Pipes with inadequate capacity are shown graphically in Exhibits 32 and 33.

12.2.2 Velocity

Minimum slope guidance is provided in an effort to achieve cleansing velocities within gravity pipes in the wastewater system. In general, a target cleansing velocity of 2 fps is used. During dry weather periods, flow rates are relatively low, which results in lower flow depths and velocities. Deposition of sediments

can occur during these periods. Elevated velocities during high-flow events will carry some of the deposits downstream to the system outfall, provided adequate scour velocities are achieved.

Analysis of the current system under wet weather design peak flows reveals significant portions of the system do not see self-cleansing velocities during high-flow events. Specifically, 808 of the pipes within the system have velocities less than 2 fps during the wet weather peak flow analysis. Sub-standard velocities occur for the following reasons:

- Pipes have inadequate (sub-standard) slopes (Note: adequate self-cleansing velocities can occur in pipes with inadequate slopes provided sufficient head – due to significant surcharging of the upstream manhole – is provided upstream)
- Pipes have adequate (greater than minimum) slopes but insufficient flows, so flow depths and velocities are low

The velocities for the pipes during the current wet weather design peak flow event are shown in Exhibit 34

12.3 Future Wastewater Model Results

12.3.1 Flow Capacity

Under future dry weather flow conditions, the system has adequate capacity with the exception of three pipes. The pipes with the most significant capacity issues are the same pipes as for the current system analysis. The diameter, length, slope, and ratio of flow to design capacity for these four pipes are shown in Table 52.

Table 52: Existing Pipe Exceptions to Meet Future Flow Capacity

Pipe Label	Diameter (in)	Length (ft)	Slope (ft/ft)	Flow / Design Capacity (%)
1480-1677	8.0	36.5	-0.00014	209.6
2477-2478	10.0	235.2	0.00010	148.4
1261-779	10.0	64.4	-0.00014	137.2

Under future wet weather design flow conditions, the majority of lines with inadequate capacity have sub-standard slopes or are pipes that have been manually set to minimum slope within the model because either the upstream or downstream pipe invert was not obtained during the field data collection process. Pipes with inadequate capacity are shown graphically in orange and red in Exhibits 35 and 36.

12.3.2 Velocity

Analysis of the future system under wet weather design peak flows reveals significant portions of the system do not see self-cleansing velocities during high-flow events, despite additional sanitary loads due to growth within the City. Specifically, 764 of the pipes within the system have velocities less than 2 fps during the wet weather peak flow analysis. The velocities for the pipes during the future wet weather design peak flow event are shown in Exhibit 37.

12.4 Wastewater Improvements

12.4.1 Description

System improvements were identified to address capacity issues for the current and future scenarios. Pipes with flow to design capacity ratios exceeding the design criteria were identified, and improvements were designed to provide adequate capacity. The identified improvements were developed to minimize the extent of the impacts. The following situations were encountered during the improvement development process to achieve adequate pipe capacities:

1. Capacity was achieved by increasing pipe diameter and using the existing pipe slope. This type of change limits improvements to the deficient pipe(s).
2. Capacity was achieved by adjusting pipe slope between existing manholes. This type of change limits improvements to the deficient pipe(s). Pipe diameters may have been increased as well.
3. Capacity was achieved by adjusting pipe slope of the deficient pipe(s) and one or more upstream or downstream pipes. In general, it is advisable to utilize extra slope downstream of the deficiencies because that type of change results in the sewer becoming deeper overall. This limits potential issues with service connections and collector lines. Pipe diameters may have been increased as well.

System improvements were not specified in the Master Plan to address sub-standard slopes or inadequate self-cleansing velocities if major complaints were not present in an area, provided that the modeled sections provided adequate capacity. Section 14.0 gives further information regarding identification and implementation of recommended improvements. Exhibit 38 shows Near Term Improvements that need to address current capacity issues.

12.5 Lift Station Gravity Bypass

12.5.1 Introduction

The City expressed interest in identifying gravity bypass alternatives for two lift stations on the northwest edge of the system:

- Overland Trail Lift Station
- The Reserve Lift Station

12.5.2 Overland Trail Lift Station

Overland Trail Lift Station collects flows from a relatively small area: (1) along Overland Trail between Sagebrush Street and Bandit Trail (west of Overland Trail), (2) along Chisholm Court, and (3) along Bandit Trail between Overland Trail and Windswept Trail. The force main from the Overland Trail Lift Station goes south along Overland Trail to Bandit Trail, then east along Bandit Trail to tie into the existing gravity network.

There are approximately 30 houses that contribute flow to the Overland Trail Lift Station. Assuming an average base sanitary sewer loading rate of 150 gpd/lot, the total daily dry weather loading is 4,500 gpd (1.643 MG/year). The total land area contributing flow to the Overland Trail Lift Station is approximately 21 acres. The average wet weather RDII volume for FM basins 1 and 5, which are in the same vicinity as

the area for the Overland Trail Lift Station, is between 0.000020 and 0.000025 MG/in/ac. The average annual precipitation is approximately 37.9 inches (NWS location station in Grapevine). Therefore, the annual RDII volume for the 21 acres can be estimated as 0.016 to 0.020 MG/year. The total flow through the Overland Trail Lift Station on an annual basis is estimated at 1.82 MG/year.

12.5.2.1 Proposed Gravity Bypass Route

The preferred gravity bypass alternative for Overland Trail, assuming the gravity bypass will tie into the City's system and not be discharged to an adjacent system, follows the same alignment as the existing force main. Therefore, under a gravity bypass alternative, the flows for the Overland Trail Lift Station will still be directed to the same location in the system, and there will not be an adverse impact on the system because the peak flows for the gravity bypass should be less than or equal to the peak flows from the lift station. This alignment, and associated depths of bury, are shown in Exhibit 40. Directional drilling for depths of bury greater than 20 feet is considered.

12.5.2.2 Cost Comparison

The current flow rate of 1.82 MG/year equates to a single pump run time of 433 hours, or 18 days. Utilizing a 5 horsepower, assumed motor efficiency of 70%, and energy cost of $0.10/kwh, the total yearly anticipated electrical costs are $240/year, with estimated annual maintenance costs of $1,000/year, for a 20 year life cycle cost of $18,448.

The estimated cost to construct the Overland Trail bypass is shown in the following Table 53.

Table 53: Estimated Cost to Construct Overland Trail Gravity Bypass

Diameter (in.)	Length (ft.)	Avg. US & DS Depth (ft.)	Pipe Cost	Manholes Each	Manholes Cost	Total Estimated Cost
6	221.20	8.80	$ 30,822	1	$ 8,647	$ 39,469
6	497.70	13.80	$ 69,351	1	$ 8,647	$ 77,997
6	101.10	27.40	$ 24,978	1	$ 8,647	$ 33,625
6	398.60	36.00	$ 98,480	1	$ 9,892	$ 108,372
6	288.80	12.00	$ 40,242	1	$ 8,647	$ 48,889
6	392.60	6.00	$ 54,706	2	$ 17,294	$ 71,999
Lift Station Decommissioning (Lump Sum)						$ 75,000
					Subtotal	$ 455,352
					Contingency (20%)	$ 91,070
					Engineering	$ 81,963
					OPCC	**$ 628,385**

12.5.2.3 Conclusion

The cost to maintain the lift station is calculated to be much less than the capital cost to install a gravity bypass. The high cost for installation of the pipeline is due to the required pavement reconstruction throughout the residential neighborhood, deep bury depth throughout the middle section of the line alignment, and deep directional boring. In addition, the new section of gravity bypass will also need continuing operations and maintenance. Due to the large difference in cost to maintain and operate the lift station versus installation of a gravity bypass, it is recommended to maintain the existing lift station.

12.5.3 The Reserve Lift Station

The Reserve Lift Station collects flows from the new development in the area around Rembrandt, north of Murphy Road. The force main from The Reserve Lift Station goes east along Rembrandt, then south to tie into the gravity network for FM basin 5.

There are approximately 110 houses that will contribute flow to The Reserve Lift Station once the area is fully developed. This does not include additional flows from the area currently under development to the west, which encompasses approximately 17 acres. Assuming an average base sanitary sewer loading rate of 150 gpd/lot, the total daily dry weather loading is 16,500 gpd (6.023 MG/year). The total land area contributing flow to the Overland Trail Lift Station is approximately 65 acres. The average wet weather RDII volume for flow monitoring basins 1 and 5, which are in the same vicinity as the area for the Overland Trail Lift Station, is between 0.000020 and 0.000025 MG/in/ac. The average annual precipitation is approximately 37.9 inches (NWS location station in Grapevine). Therefore, the annual

RDII volume for the 65 acres can be estimated as 0.049 to 0.062 MG/year. The total flow through The Reserve Lift Station on an annual basis is estimated at 6.08 MG/year.

12.5.3.1 Proposed Gravity Bypass Route

The gravity bypass alternative for The Reserve Lift Station does not follow the same alignment as the existing force main. The preferred gravity bypass for The Reserve Lift Station is east along Rembrandt, then continuing east behind the houses on the north side of Eventide Way and connecting to the existing network in the area between Vanguard Court and Stillwater Circle (East Bypass Alternative shown in Exhibit 41).The gravity bypass would discharge to a manhole that is within FM basin 1. Therefore, this gravity bypass alternative would have the following impacts to the system:

- Increase flows along a main interceptor for FM basin 1
- Decrease flows along the main interceptor for FM basin 5

The increase in flows for FM basin 1 do not result in any additional improvements being required above those which are recommended to address existing capacity concerns. Therefore, it is not anticipated that the gravity bypass will result in any additional costs to improvements in the receiving sewer basin.

The decrease in flows for FM basin 5 reduces the severity of the issues for that sewer basin. Modifications are still recommended to address capacity concerns, although the urgency of the improvements would be reduced:

- Group B would become a lower priority because the flow to design capacity ratio would drop from 114.4 to 89.5% for the pipe with the most severe capacity issues.
- The downstream pair of pipes recommended for Group C would remain a high priority, although the severity of the issue decreases significantly. The upstream pair (2477-2478 and 2478-1277) would become a lower priority, and that improvement could be delayed.
- Group F would become a lower priority because the flow to design capacity ratio would drop from 112.3 to 80.0% for the pipe with the most severe capacity issues.
- Group I would remain a low priority project.

12.5.3.2 Cost Comparison

The current flow rate of 6.08 MG/year equates to a single pump run time of 1,034 hours, or 43 days. Utilizing a 5 horsepower, assumed motor efficiency of 70%, and energy cost of $0.10/kwh, the total yearly anticipated electrical costs are $551/year, with estimated annual maintenance costs of $1,000/year, for a 20 year life cycle cost of $23,000.

The estimated cost to construct The Reserve bypass is shown in the following Table 54.

Table 54: Estimated Cost to Construct the Reserve Bypass

Diameter (in.)	Length (ft.)	Avg. US & DS Depth (ft.)	Pipe Cost	Manholes Each	Manholes Cost	Total Estimated Cost
8	500.00	13.00	$108,097	1	$ 8,647	$ 116,744
8	475.00	36.00	$117,742	1	$ 9,892	$ 127,634
8	265.00	45.00	$ 65,688	1	$ 9,892	$ 75,580
8	232.00	43.00	$ 57,508	1	$ 9,892	$ 67,400
8	500.00	34.00	$123,939	1	$ 9,892	$ 133,831
8	500.00	35.00	$123,939	1	$ 9,892	$ 133,831
8	500.00	20.00	$123,939	1	$ 9,892	$ 133,831
8	65.00	10.00	$ 14,053	2	$ 17,294	$ 31,346
Lift Station Decommissioning (Lump Sum)						$ 75,000
					Subtotal	$ 895,198
					Contingency (20%)	$ 179,040
					Engineering	$ 161,135
					OPCC	**$ 1,235,373**

Though the flow redirection would result in the de-prioritizing of several projects, those projects would still require implementation in the 20 year planning period. Additionally, the difference between operation and maintenance life cycle cost of the existing lift station versus the 20-year amortized construction cost of a new very deep gravity bypass is very significant.

Therefore, it is recommended that The Reserve lift station be kept operational. However, additional decommissioning alternatives may be available if the neighboring connections outside Colleyville's service area to the northwest are available and willing to receive flow. It is anticipated that route will be shorter distance and shallower depth of bury. Further investigation and coordination with those utilities is needed if this alternative is determined to be feasible.

13.0 Recommended Wastewater Improvements

In evaluating recommended wastewater improvements, for line sections it is assumed the furthest downstream manholes will be replaced. Any manholes located along proposed improvements that are associated with the executed manhole evaluation have been removed from that group cost estimate and are included in the line group (as the manhole will likely need replacement, due to revised flow lines).

13.1 Project Identification and Ranking

Similar to the previously detailed water project identification and ranking, several key factors were identified as triggers for a project. These triggers were developed in conjunction with the previously established design criteria. The improvements recommended in following sections address all triggers for that identified need.

The identified triggers are as follows (in order of priority, from greatest to least):

1) Capacity
This trigger is activated if a section or segment of sewer is not able to provide the modeled flow during peak demand or elevated storage to all meters within that portion of the system. Trigger limits are identified in the design criteria and are as follows:
- For all pipes, any pipe greater than 75% full

2) Condition
This trigger is activated if a known condition requires repair or replacement of the infrastructure. Individual condition triggers have been identified in the manhole assessment report. For pipeline groupings, if greater than 50% of the manholes on each group's sewer segments have been identified as needing repair, the condition trigger is activated. City staff input was also solicited to identify known pipe condition issues in the collection system.

3) Operational
Operational triggers are activated when an improvement will provide increased operational benefit, such as decommissioning aged infrastructure. This includes any lift station decommissioning.

4) City-identified
City-identified triggers include pipelines that City staff have identified as anticipated to be replaced.

In addition to these triggers, areas of the collection system were identified as not meeting minimum slope requirements, but not triggered specifically for minimum slope requirements. The existing pipelines would only be replaced for minimum slope issues, if those issues resulted in a previously listed trigger.

Once these triggers were identified, a modeling analysis was conducted to determine the most cost-effective improvements to implement, while providing the maximum impact with minimal infrastructure for near and long term system growth. Alphabetical project groupings were developed in localized areas to address the identified improvements.

The improvements were then divided into three levels of flexibility, depending on the current condition of the need, and anticipated criticality of the improvements. All improvements were then ranked numerically. The most critical, lowest flexibility needs were given highest priority, and consideration was also given to the greatest service area impacts for each improvement. For capacity triggers, higher priority was given to those pipes which exhibited greater than 95% capacity during peak flow conditions.

The resultant project identification and rankings list, located in Appendix E, provides the City with a directory of the most critical needs addressed in near-term and long-term projects.

13.2 Renewal and Replacement Projects

The City has identified 23,190 LF of pipe to be replaced due to condition concerns. This represents 1% of the City's total pipeline inventory, which addresses needs that have developed in the previous 5 years. Therefore, this level of renewal and replacement is expected to occur on a 5-year basis. The first group of renewal and replacement is represented in the previously identified projects with condition triggers. Three additional phases (Project Groups U, V, and W) identify the same percentage of replacement, distributed by pipe size according to the City's current inventory.

13.3 Wastewater Infrastructure Improvements

See Appendix E for detailed infrastructure improvements, cost estimates descriptions, and recommendations. A contingency factor of 20% has been applied to accommodate unforeseen design considerations, and changes in market pricing,

A summary of proposed projects, time-frame for implementation, and current and forecasted costs is displayed in Table 55.

Table 55: Wastewater Capital Improvements Summary

Project Identification				Schedule			2015 Cost ($1000)[1]
Project	Description	Location	Flexibility	Primary Trigger	Secondary Trigger	Trigger Date	OPCC
1	Group X	Area 3	Low	City-Identified	None	Oct-17	$1,556
2	Group O	City-Wide	Low	Condition	None	Oct-18	$185
3	Group H	City-Wide	Low	City-Identified	Condition	Oct-18	$497
4	Group C	North	Low	Capacity	None	Oct-18	$193
5	Group F	North	Low	Capacity	Condition	Oct-22	$616
6	Group A	West	Low	Capacity	None	Oct-22	$632
7	Group T	South	Low	City-Identified	Condition	Oct-22	$624
8	Group G	East	Low	Capacity	None	Oct-22	$325
9	Group U	City-Wide	High	Condition	None	Oct-22	$2,259
10	Group P	City-Wide	Medium	Condition	None	Oct-22	$560
11	Group Q	City-Wide	Medium	Condition	None	Oct-22	$218
12	Group R	City-Wide	Medium	Condition	None	Oct-23	$318
13	Group E	East	Medium	Capacity	None	Oct-24	$1,020
14	Group V	City-Wide	High	Condition	None	Oct-26	$2,259
15	Group D	South	Medium	Capacity	None	Oct-26	$531
16	Group S	City-Wide	High	Capacity	None	Oct-26	$993
17	Group W	City-Wide	High	Condition	None	Oct-32	$2,259
				Total 2015 OPCC			**$15,044**

[1] Project Costs are the Opinion of Probable Construction Costs (OPCC) based in year 2015, and include Engineering and Contingency. A 3 percent escalation for inflation to the Trigger Date month and year has been added for a Forecasted Cost in Appendix E.

Figure 43 illustrates the breakout of projects by primary trigger, based on percentage of 2015 OPCC.

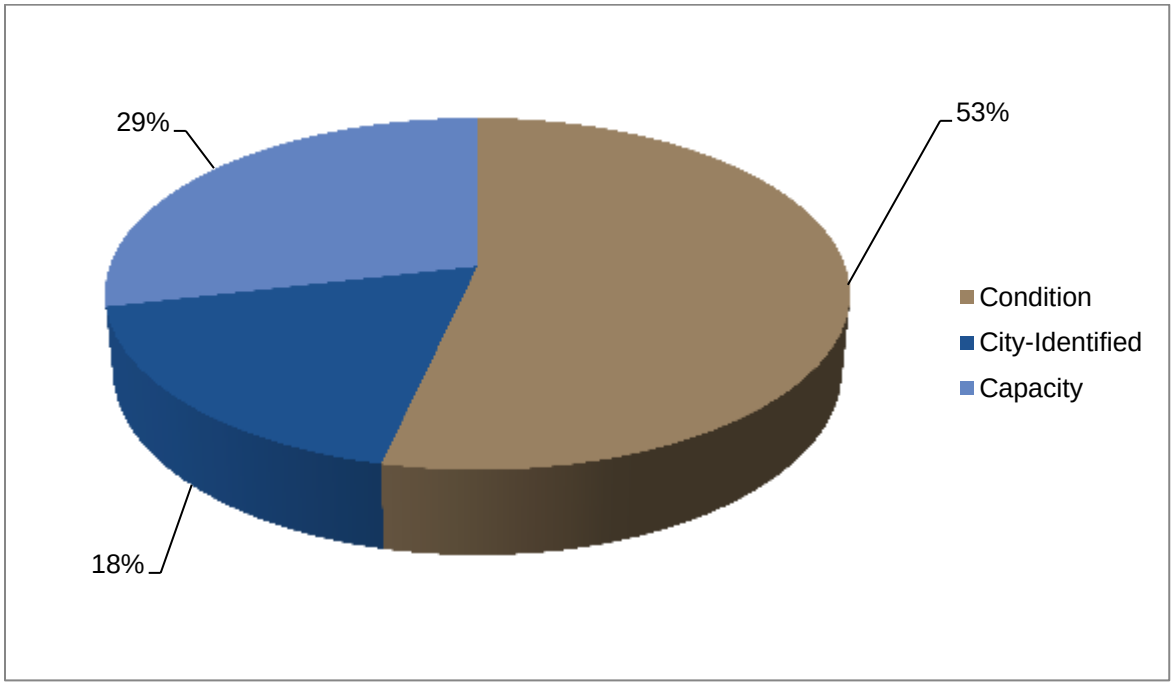


Figure 43: Distribution of Wastewater Total 2015 Project Costs by Primary Trigger

14.0 Conclusion

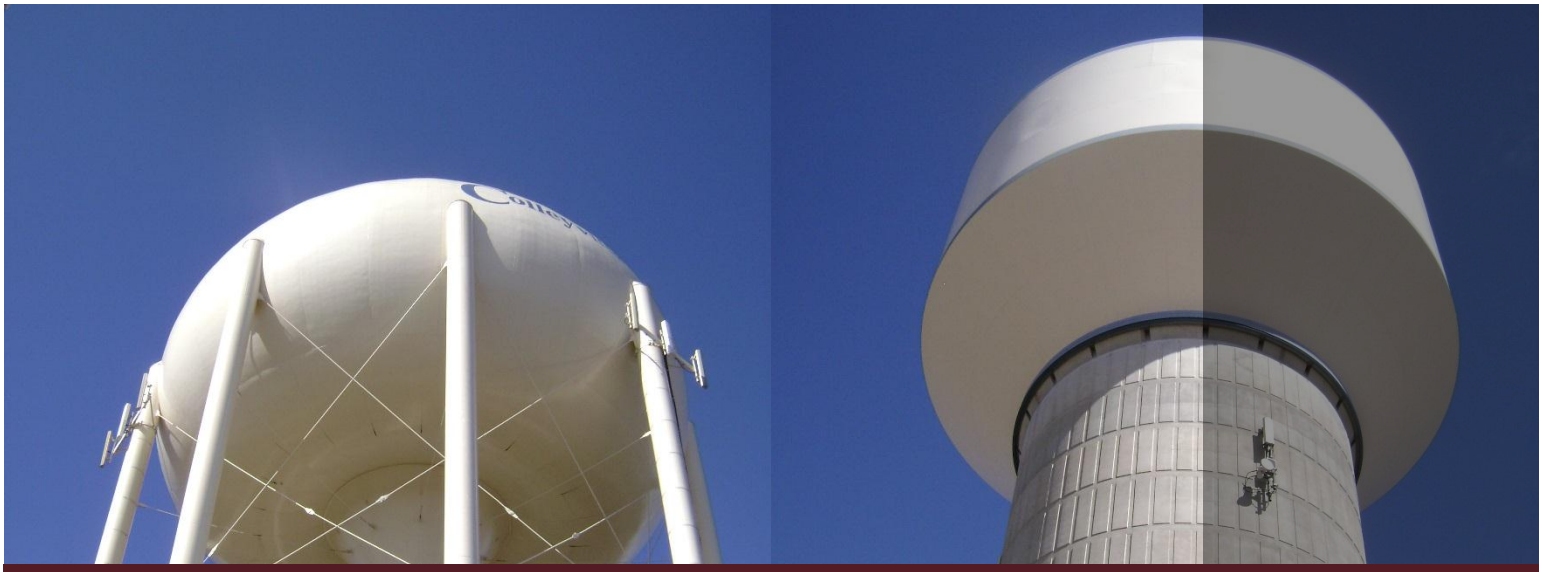
This Master Plan has identified key recommendations for water and sewer infrastructure and operations, based on multiple field investigations, condition assessments, and model simulations.

The Plan provides key information for future planning efforts as well, including:

- Projected water usage and sewer loading rates based on existing conditions
- Updated design criteria for future planning and project development
- Condition assessments of all existing facilities, with recommendations for near and long term improvements
- Condition assessments of a large portion of system manholes, with recommendations for near and long term rehabilitation projects
- Flow monitoring and a ranking of sub-basins based on RDII
- Updated City water and wastewater models
- Updated City water and wastewater GIS
- A list of water projects to reach identified 20-year growth plans and maintain existing infrastructure
- A list of wastewater projects to reach identified 20-year growth plans and maintain existing infrastructure

14.1 Recommendations

It is recommended to implement the Water and Wastewater CIPs concurrently, as funding allows. This will allow the City to be equipped for the 20-year build out that is anticipated. This Plan should be re-evaluated at a minimum of every 5 years, to ensure projected growth and usage rates have not significantly changed.



Technical Memorandum

Water/Wastewater Master Plan

Water Facilities Evaluation

Prepared For:

City of Colleyville

July 2014

GARVER

GarverUSA.com

Technical Memorandum

Water/Wastewater Master Plan
Water Facilities Evaluation

City of Colleyville, Texas

Engineer's Certification

I hereby certify that this Technical Memorandum for the Water/Wastewater Master Plan – Water Facilities Evaluation was prepared by Garver under my direct supervision for the City of Colleyville, Texas.

Lance P. Klement, PE
State of Texas PE License 113630

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1.0 Executive Summary

The City of Colleyville (City) has commissioned Garver to perform field investigations of its water facilities in order to develop an accurate understanding of the condition and expected service life of the existing facilities. The City's water system includes the following facilities:

- Two pump stations: Overland Trail Pump Station and L. D. Lockett Pump Station;
- Two ground storage tanks (GST), one at each pump station; and
- Three elevated storage tanks (EST): Bransford Elevated Tank, Hall Johnson Elevated Tank and McPherson Elevated Tank.

Minor site security and rehabilitation improvements are recommended at LD Lockett, Bransford EST, and McPherson EST. No improvements are recommended for Hall Johnson. It is recommended to decommission and demolish Overland Trail.

2.0 Introduction

2.1 Project Scope

A condition assessment of the water facilities was accomplished by means of a site visit where an engineering team evaluated site security, paving and grading, mechanical equipment (piping and pumps), and instrumentation and controls based on visual observations. In-depth structural and electrical condition assessments were not performed. The team also conducted interviews with operations and maintenance staff and reviewed all available data regarding the facilities such as plans, maintenance records and TCEQ inspection documentation. The findings are documented in this technical memorandum, along with background data, recommendations for replacement and repair (if needed) and cost estimates to undertake any recommendations.

2.2 Site Description

The facilities are located primarily in the northwestern portion of the city, with the exception of the Hall Johnson EST which is centrally located. Figure 1 identifies the location of each water facility.

Figure 1: Water Facilities Site Map

Overland Trail Pump Station and GST are located at 7508 Overland Trail, between Wagonwheel Court and Sagebrush Street. Figure 2 provides an overview of the site location. The pump station and GST are located in a residential neighborhood and the property is bordered by homes on the north, south and west sides.

Figure 2: Overland Trail Pump Station and GST Site Location

L.D. Lockett Pump Station and GST are located at 729 West L.D. Lockett Road, between Reatta Parkway and Chalford Common. Figure 3 provides an overview of the site location. The pump station and GST are bordered by homes on the north and west sides and baseball fields on the south side.

Figure 3: L.D. Lockett Pump Station and GST Site Location

Bransford EST is located at 6015 Bransford Road, across from Bransford Park and near the intersection of Bransford Road and L.D. Lockett Road. Figure 4 provides an overview of the site location. The tank site is adjacent to a rail line along northwest boundary and bordered by trees along the east.

Figure 4: Bransford EST Site Location

Hall Johnson EST is co-located with the City of Colleyville Public Works Service Center at 1601 Hall Johnson Road. Figure 5 provides an overview of the site location.

Figure 5: Hall Johnson EST Site Location

McPherson EST is located in McPherson Park, 240 West McDonwell School Road. Figure 6 provides an overview of the site location.

Figure 6: McPherson EST Site Location

3.0 Assessment Process

To have an effective assessment of a water facility, it is important to understand the outcomes desired before the assessment is completed. This serves as a quality control measure and insures the end product meets high standards. The approach used in the assessment process is summarized below.

3.1 Data Collection and Review

This task included assembling and reviewing all available and relevant documents, which include the following:

1. Record documents
2. Operation and maintenance records
3. Equipment cut sheets and catalogs
4. Pump performance curves
5. Existing site surveys
6. Code requirements for the site
    a. TCEQ, including *§290.43. Water Storage* and *§290.46. Minimum Acceptable Operating Practices for Public Drinking Water Systems*
    b. Army Corp of Engineers
    c. International Building Code
    d. City/County Ordinances and Code
    e. National Fire Protection Association Codes
    f. OSHA Compliance

3.2 Field Review and Condition Assessment

3.2.1 Field Review

Garver sent a team of engineers to the water facilities on March 13, 2014 to complete the field review and staff interviews. The investigations were based solely on visual observations related to general site security including access, gates, lighting; general site drainage; and general conditions of pipes, pumps and valves. No in-depth structural or electrical condition assessments were performed. Note that the field investigations of each facility do not constitute a full condition assessment. The observations were made based on the investigation team members' experience.

3.2.2 Condition Assessment

Prior to conducting the field assessment of the facilities, a matrix checklist was developed to be able to systematically evaluate the facility and equipment. The field review included photographing all equipment and appurtenances as well as visually observing fit and function.

3.3 Identification of Deficiencies

Once the documentation review and site assessments were completed, the deficiencies in the system could be identified and addressed. The following categories were addressed and compared against existing codes, Best Available Practices, or other criteria applicable to each category.

- Site Security
- Site Civil
- Access
- Pumping and Mechanical
- Instrumentation and Controls

4.0 Overland Trail Pump Station and Ground Storage Tank

4.1 Facility History and Overview

The site currently consists of a Pump Building, Chlorine Building, Ammonia Building, ground storage tank, perimeter fencing with vehicular gate, and associated grading and drainage provisions. Renovations to the facility in 1998 included replacement of the Pump Building and Chlorine Building, addition of the Ammonia Building, and paving improvements. The current pump station is a 3.5 MGD facility with four horizontal split case pumps; two pumps have a capacity of 500 gpm and two pumps have a capacity of 750 gpm.

The tank is reported to have been relocated from an air base near San Angelo many years ago by the Leonard Brothers Water Supply Corporation. When the City of Colleyville acquired the portion of the Leonard Brothers water system inside the city limits, the tank was included with the purchase. The tank is riveted steel construction. The finished floor is at an elevation of 717.4 and the tank's overflow is at an elevation of 740.0. The tank is approximately 140 feet in diameter and has a capacity of 1.2 million gallons. The inlet is 8-inch ductile iron and the outlet/pump suction is 24-inch ductile iron. The tank is also provided with a 10-inch overflow pipe and 12-inch flapper valve on the overflow. The tank has an 18-inch diameter roof access hatch and a 24-inch diameter access hatch on the west sidewall. The tank is also equipped with an external ladder, electronic and manual water level indicator, a 36-inch and a 12-inch mushroom vent. Safety features include a safety climb system. Table 1 summarizes the pump station and tank design data.

Table 1: Overland Trail Pump Station and GST Design Data

Parameter	Value
Location	7508 Overland Trail
Original Construction	Unknown
Pump Station	
Improvements	1998
Description of Improvements	Replaced Pump Building and Chlorine Building; Added Ammonia Building; Paving Improvements
Number of Pumps	4
Pump Capacity (gpm)	Pumps 1 and 2: 750 Pumps 3 and 4: 500
Rated Total Dynamic Head (TDH) (ft)	Pumps 1 and 2: 150 Pumps 3 and 4: 150
Type of Pump	Horizontal split case

Manufacturer	Fairbanks Morse
Model	2823A
Volts/Ph/Hz	460/3/60
Motor HP	Pumps 1 and 2: 40 Pumps 3 and 4: 30
RPM	1800
Tank	
Type of Tank	Ground Storage
Construction	Riveted Steel
Improvements	1998
Description of Improvements	Replaced tank controls and inlet/outlet piping
Capacity (MG)	1.2
Height to Overflow (ft)	22.6
Diameter (ft)	140
Inlet/Outlet Diameter (in)	8/24
Overflow (in)	10

4.2 Service Area

The pump station and ground storage tank service the High Plane pressure zone; however, they are presently offline due to the poor condition of the ground storage tank. The High Plane pressure zone is also serviced by L.D. Lockett Pump Station and Ground Storage Tank and McPherson Elevated Storage Tank. Figure 7 presents a pressure plane distribution map and identifies the facilities that serve each pressure plane and watering zone.

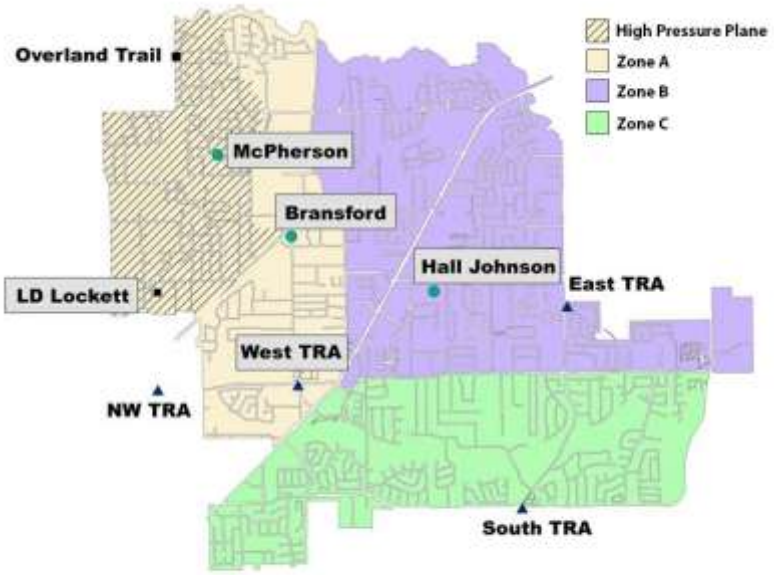

Figure 7: Pressure Plan Distribution

4.3 Plan Reference

The 1998 Record Drawings prepared by Knowlton-English-Flowers, Inc. for the "Overland Trail Pump Station Improvements" were referenced for the design of the pump station and tank site.

4.4 Control Description

Pumps 1 and 2 alternate operation, and Pumps 3 and 4 alternate operation. The pumps are run to maintain pressure in the distribution system, and they start and stop based primarily on level in McPherson Elevated Storage Tank. Additionally, the pumps shut down on a low level signal from the Overland Trail Ground Storage Tank. Facility alarms include GST high water level and pump failure.

Gaseous chlorine and anhydrous ammonia are fed by direct gas feeders. Each gas feeder has a manual control dial in addition to an integrated control panel for automatic adjustments. In automatic operation the feeders are flow paced based on flow meter signal feedback and input dose set-point. The chlorine dose is set based on the residual measured by the ammonia analyzer.

4.5 Condition Assessment

4.5.1 Site Security

The site is located in a residential neighborhood. Site access is restricted by a wrought iron fence with a locked gate across the main entrance to the east, an 8 foot tall wooden fence across the northern boundary, and an intruder-resistant chain link fence across the western and southern boundaries. The chain link portions of the perimeter fencing have barbed wire.
During field investigations the overall site appeared to be fairly well kept, with the exception of the perimeter vegetation. Consideration should be given to removing vegetation and overhung tree limbs along the fence to improve perimeter visibility and limit climbing access.

Figure 8: Overgrown Perimeter Vegetation at Overland Trail

4.5.2 Site Civil

While the site generally appears well drained, erosion has occurred under the northern fence and the landscaping in that area needs attention.

Figure 9: Erosion at Overland Trail Northern Fence

4.5.3 Access

The site has sufficient access for maintenance, as the access gate opens to Overland Trail and will easily accommodate a work truck. There is a permanent bridge crane in the Pump Building to facilitate pulling the pumps for service, if needed.

The access hatch on the top of the GST is only 18 inches in diameter. The small diameter and the presence of a crossbar running directly beneath the access hatch prevent entry for diver inspection of the tank.

4.5.4 Pumping and Mechanical

The interior of the Pump Building is in good condition. The pumps, valves and piping have a very good appearance with intact coatings and no signs of corrosion or leakage. City staff reports the equipment to be in excellent operational condition.

Figure 10: Interior Photographs of Overland Trail PS Pump Building

The interior of the Chlorine Building is in good condition. The piping, valves, chlorine feed and scrubber equipment appear to be in like-new condition, but were not in operation at the time of the field investigations

Figure 11: Interior Photographs of Overland Trail PS Chlorine Building

The interior of the Ammonia Building is in good condition. The piping, valves and ammonia feed equipment appear to be in like-new condition. This facility was not in operation at the time of the field investigations and was being used for storage.

Figure 12: Overland Trail PS Ammonia Building Used for Storage

4.5.5 Instrumentation and Controls

The instrumentation and controls for the pump station, tank and chemical systems consist of motor control centers, pressure indicators, level switches and an ammonia analyzer. The controls are all in good working condition and operate as designed and intended.

4.5.6 Structural

The overall condition of the exterior of the buildings is very good. The walls appear to be plumb with no evidence of settlement or undulations in the walls. Visual assessments determined that the masonry units and grout did not show signs of signficant weathering, erosion or cracking.

Figure 13: Exterior Photographs of Overland Trail PS Buildings

The ground storage tank is showing signs of distress and is heavily corroded and leaking. The upper portion of the tank appears to be buckling as evidenced by the ladder bowing out from being originally straight. Due to its poor condition, the tank is currently out of service.

Figure 14: Evidence of Overland Trail Ground Storage Tank Buckling

Figure 15: Corrosion on Overland Trail Ground Storage Tank

4.5.7 Summary

Table 2 summarizes the findings of the condition assessment based on ratings of good, fair and poor. A condition assessment rating of *good* indicates that no immediate attention is required. A rating of *fair* indicates that some initial repair or minor improvement is needed for the facility to remain in adequate working condition. A rating of *poor* indicates that improvements, replacement or reconstruction are needed in the immediate future.

Table 2: Overland Trail Pump Station and GST Condition Assessment Ratings

Aspect	Rating
Site Security	Fair
Site Civil	Fair
Pump Station Access	Good
Tank Access	Poor
Pumping and Mechanical	Good
Instrumentation and Controls	Good
Tank	Poor

4.6 Conclusions

The following conclusions were developed from the evaluation of the Overland Trail Pump Station and GST:

- Perimeter vegetation is overgrown and the landscaping requires attention.
- The grading of the site generally allows for proper drainage without ponding or flooding, with the exception of the northern boundary.
- Access to mechanical equipment is adequate, but the access hatch on the tank roof is smaller than the recommended size per TCEQ and a crossbar underneath the hatch prevents interior access by a diver.
- The pump station buildings and associated equipment have been well maintained and are in very good condition.
- The electrical and controls are in good operating condition and have not posed any problems for the City staff.
- The ground storage tank is in poor condition and inoperable at this time.

4.7 Recommendations

A separate Overland Trail Technical Memorandum provided by Garver evaluated the water distribution system's need for the facility. As detailed in the memo, the facility is no longer needed for the City's existing or future infrastructure.

The City has expressed a desire to sell the property. If that option is pursued, it is recommended to fully decommission and demolish the GST, and to decommission and salvage the existing pump station. The yard piping should be decommissioned outside of the existing property line, in order to prevent the future parcel owner from having direct access to the City's water supply.

4.8 Cost Implications

Complete demolition of the existing ground storage tank, along with reclamation of existing pumping and electrical in the pump station (while leaving the existing booster station building and site security intact), is expected to cost approximately $75,000.

5.0 L.D. Lockett Pump Station and Ground Storage Tank

5.1 Facility History and Overview

The site currently consists of a Pump Building, Rechlorination Building, Dechlorination Building, ground storage tank, perimeter fencing with vehicular gate, and associated grading and drainage provisions. The pump station and ground storage tank were constructed in 2008. The pump station is an 11.5 MGD facility with eight horizontal split case pumps:

- two 550 gpm pumps for low plane average flow;
- two 1200 gpm pumps for low plane peak flow;
- two 750 gpm pumps for high plane average flow; and
- two 1500 gpm pumps for high plane peak flow.

The tank is pre-stressed concrete construction. The finished floor is at an elevation of 637.0 and the tank's overflow is at an elevation of 675.0. The tank is approximately 150 feet in diameter and has a capacity of 5.0 million gallons. The inlet is 20-inch ductile iron with 12-inch Tideflex mixing outlets and the outlet/pump suction is 30-inch ductile iron. The tank is also provided with a 16-inch ductile iron recirculation line, 24-inch ductile iron drain line, 16-inch ductile iron overflow pipe, and 16-inch flapper valve on the overflow. The tank roof has a 20-inch diameter access/inspection hatch and a 36-inch by 36-inch manway with lockable hasp. A 36-inch diameter manway is located on the tank sidewall. The tank is also equipped with an external ladder, electronic and manual water level indicator, and a 30-inch mushroom vent. Safety features include a safety climb system. Table 3 summarizes the pump station and tank design data.

Table 3: L.D. Lockett Pump Station and GST Design Data

Parameter	Value
Location	729 West L.D. Lockett Road
Original Construction	2008
Pump Station	
Improvements	None
Description of Improvements	N/A
Number of Pumps	8
Pump Capacity (gpm)	Low Plane, Avg Flow: 550 Low Plane, Peak Flow: 1200 High Plane, Avg Flow: 750 High Plane, Peak Flow: 1500
Rated Total Dynamic Head (TDH) (ft)	Low Plane, Avg Flow: 154 Low Plane, Peak Flow: 184 High Plane, Avg Flow: 204 High Plane, Peak Flow: 227
Type of Pump	Horizontal split case
Manufacturer	Aurora
Model	Low Plane, Avg Flow: 421 Low Plane, Peak Flow: 421 High Plane, Avg Flow: 421 High Plane, Peak Flow: 410

Volts/Ph/Hz	460/3/60
Motor HP	Low Plane, Avg Flow: 40 Low Plane, Peak Flow: 100 High Plane, Avg Flow 6: 75 High Plane, Peak Flow: 125
RPM	Low Plane, Avg Flow: 1200 Low Plane, Peak Flow: 1200 High Plane, Avg Flow: 1200 High Plane, Peak Flow: 1800
Tank	
Type of Tank	Ground Storage
Construction	Pre-stressed Concrete
Improvements	None
Description of Improvements	N/A
Capacity (MG)	5.0
Height to Overflow (ft)	38
Diameter (ft)	150
Inlet/Outlet Diameter (in)	20/30
Overflow (in)	16

5.2 Service Area

The pump station and ground storage tank service the High Plane pressure zone. The High Plane pressure zone is also serviced by Overland Trail Pump Station and ground storage tank (which are presently offline due to the poor condition of the ground storage tank) and McPherson Elevated Storage Tank.

Figure 7, in Section 4 above, presents a pressure plane distribution map.

5.3 Plan Reference

The 2009 Record Drawings prepared by Dannenbaum for the “L.D. Lockett Road Ground Storage Tank and Pump Station” were referenced for the design of the pump station and tank site.

5.4 Control Description

The pumps are run to maintain pressure in the distribution system, and they start and stop based primarily on level in the elevated storage tanks. Additionally, the pumps shut down on a low level signal from the L.D. Lockett Ground Storage Tank. Facility alarms include GST high water level and pump failure. No information was provided for the control of the chlorination and dechlorination systems.

5.5 Condition Assessment

5.5.1 Site Security

Site access is restricted by an ornamental wrought iron fence with a locked gate across the main entrance and an 8 foot tall intruder-resistant chain link fence around the remainder of the site. The chain

link fabric along the southern portion of the fence needs to be straightened and remounted. It has been reported that the southern fence is being breached by citizens to retrieve balls from the adjacent ball fields. The City should consider posting signs along the fence informing the public that it is a violation of federal law to enter the site without permission.

Figure 16: L.D. Lockett PS Perimeter Fencing

5.5.2 Site Civil

During field investigations the overall site appeared to be fairly well kept and well drained. Minor erosion has occurred near the northern side of the ground storage tank and needs attention.

5.5.3 Access

The site has sufficient access for maintenance, as the access gate opens to Chalford Common and can accommodate a work truck. There is a truck ramp at the Pump Building and a permanent bridge crane within the building to facilitate pulling the pumps for service, if needed.

The 36-inch by 36-inch access hatch on the top of the GST and the 36-inch manway in the sidewall allow for adequate access to the interior of the tank.

5.5.4 Pumping and Mechanical

The interior of the Pump Building is in good condition. The pumps, valves and piping have a very good appearance with intact coatings and no signs of corrosion or leakage. City staff reports the equipment to be in excellent operational condition.

Figure 17: Interior of L.D. Lockett PS Pump Building

The interior of the Rechlorination and Dechlorination Buildings also appear to be in good condition. The piping, valves, chlorination feed and dechlorination equipment are visually in very good to like-new condition.

Figure 18: Interior of L.D. Lockett PS Rechlorination and Dechlorination Buildings

5.5.5 Instrumentation and Controls

The instrumentation and controls for the pump station, tank and chemical systems consist of motor control centers, pressure indicators, and level switches. The controls are all in good working condition and operate as designed and intended.

5.5.6 Structural

The overall condition of the exterior of the buildings is very good. The walls appear to be plumb with no evidence of settlement or undulations in the walls. Visual assessments determined that the masonry units and grout did not show signs of signficant weathering, erosion or cracking.

Figure 19: Exterior of L.D. Lockett PS Buildings

The pump station contains several vaults for low and high plane surge valves, tank fill and drain control, rechlorination and dechlorination. The vaults and their appurtenances all appear to be in like-new condition.

Figure 20: L.D. Lockett PS Vaults

The exterior of the tank appears to be in very good to like-new condition with no evidence of cracking or erosion. Interior inspection conducted by a City contractor in December 2013 indicated that the interior is in good condition and one duckbill inlet valve requires replacement. Additionally, silt was cleaned from the tank during the 2013 inspection.

Figure 21: Lockett Ground Storage Tank

5.5.7 Summary

Table 4 summarizes the findings of the condition assessment based on ratings of good, fair and poor. A condition assessment rating of *good* indicates that no immediate attention is required. A rating of *fair* indicates that some initial repair or minor improvement is needed for the facility to remain in adequate working condition. A rating of *poor* indicates that improvements, replacement or reconstruction are needed in the immediate future.

Table 4: L.D. Lockett Pump Station and GST Condition Assessment Ratings

Aspect	Rating
Site Security	Poor
Site Civil	Fair
Access	Good
Pumping and Mechanical	Good
Tank	Good
Instrumentation and Controls	Good

5.6 Conclusions

The following conclusions were developed from the evaluation of the L.D. Lockett Pump Station and GST:

- The pump station is less than a decade old and has been well maintained over its lifetime such that it is in very good condition.
- The property is being breached by citizens along the southern boundary, which has also caused the chain link fabric to become misshapen.
- The grading of the site allows for proper drainage without ponding or flooding, with the exception of the northern boundary.
- Access to mechanical equipment is adequate.

- The pumps are adequately sized to handle current demands and flows.
- The electrical and controls are in good operating condition and have not posed any problems for the City staff.

5.7 Recommendations

The following recommendations were developed based on the evaluation described herein:

- Post signs along the fence line informing the public that it is a violation of federal law to enter the site without permission; this will help mitigate continued entrance by non-authorized persons on the property. Additionally, straighten and remount the chain link fabric along the southern portion of the fence.
- Fill and re-grade the northern boundary of the site to address present erosion issues and prevent further erosion.

5.8 Cost Implication

Site improvements to the fencing and grading are expected to cost $10,000.

6.0 Bransford Elevated Storage Tank

6.1 Facility History and Overview

The Bransford Elevated Storage Tank was constructed in 1982 by Hydrostorage Inc. The construction of the tank site included the tank, perimeter fencing with vehicular gate, and associated grading and drainage. The tank is located across from Bransford Park near the intersection of Bransford Road and L.D. Lockett Road.

The tank is welded steel construction. The finished floor of the tank's base is at an elevation of 755.0 and the tank's overflow is at an elevation of 790.0. The bowl of the tank is approximately 75 feet in diameter and has a capacity of 1.0 million gallons. A single 24-inch pipe serves as both the inlet and outlet for the tank. The tank is also equipped with a 24-inch overflow pipe. The tank has a 36-inch access tube with 24-inch manway for bowl access, 24-inch combination access tube hatch and vent, 24-inch diameter roof hatch, and a 24-inch roof vent. Safety features include handrails, ladder platforms, safety climbs, and obstruction lights on top of the tank. Table 5 summarizes the tank design data.

Table 5: Bransford Elevated Storage Tank Design Data

Parameter	Value
Location	6015 Bransford Road
Type of Tank	Elevated Storage
Construction	Welded Steel
Year Constructed	1982
Improvements	1999
Capacity (MGD)	1.0
Height to Overflow (ft)	128.5
Max Head Range (ft)	35
Bowl Diameter (ft)	75
Inlet/Outlet Diameter (in)	24
Overflow (in)	24

6.2 Service Area

The tank is one of two elevated tanks that service the Low Plane pressure zone. Figure 7, in Section 4 above, presents a pressure plane distribution map.

6.3 Plan Reference

The 1999 Record Drawings prepared by Deltatek Engineering for the “Bransford Road Elevated Storage Tank Improvements” were referenced for the design of the tank.

6.4 Control Description

The level controls in the tank send signals to L.D. Lockett Pump Station. The pump station is equipped to meet average and peak demands with the pumps dedicated to the Low Pressure Plane.

6.5 Condition Assessment

6.5.1 Site Security

Site access is restricted by an intruder-resistant chain link fence with locked gate. The chain link fabric and the barbed wire atop the fence need to be remounted and/or tightened in several locations.

During field investigations it was also noted that the overall site appeared to be fairly well kept, with the exception of the perimeter vegetation. Consideration should be given to removing vegetation along the fence line to improve perimeter visibility and limit climbing access.

Figure 22: Chain Link Fence at Bransford Elevated Storage Tank

6.5.2 Site Civil

The tank site is generally well kept and well drained.

Figure 23: Vegetation at Bransford Elevated Storage Tank

6.5.3 Access

The site has sufficient access for maintenance, as the access gate opens to Bransford Road and can accommodate a work truck. A personnel door provides personnel access at the tank base. The tank is also equipped with a 24-inch access tube manway, a 24-inch diameter combination access tube hatch and vent, and a 24-inch roof hatch. These hatches provide access to the bowl for inspection and maintenance purposes.

6.5.4 Mechanical

Corrosion was not noted on any exterior piping, but the appearance of the piping attached to the overflow and drain vault could be improved by painting.

Figure 24: Exterior Piping at Bransford Elevated Storage Tank

The exterior of the tank appears to be in very good to like-new condition with no peeling, blistering or pitting of the tank coating. The tank itself has a locked door.

Figure 25: Exterior Coating of Bransford Elevated Storage Tank

Some of the interior piping and valves appear to be close to needing paint.

Figure 26: Interior Piping at Bransford Elevated Storage Tank

The tank is well maintained, but due to its age and some deterioration beginning to be noted on the interior, it will require rehabilitation in the near future. Interior inspection conducted by a City contractor in December 2013 indicated that the structure is in good condition with only light corrosion of submerged wall plates and appurtenances such as access ladder and overflow flume. The interior roof plates were designated as being in fair condition with moderate corrosion around the braces and supports. Floor grating on the internal catwalk is in poor condition and requires replacement. Submerged coatings generally did not show signs of peeling, blistering or pitting with the exception of internal floor plates in fair condition with some blistering and pitting. Sandy silt and metal debris from catwalk were removed during the 2013 inspection and additional cleaning would not be required.

6.5.5 Instrumentation and Controls

Tank level readings are relayed to the SCADA system through the use of the City's radios.

6.5.6 Summary

Table 6 summarizes the findings of the condition assessment based on ratings of good, fair and poor. A condition assessment rating of *good* indicates that no immediate attention is required. A rating of *fair* indicates that some initial repair or minor improvement is needed for the facility to remain in adequate working condition. A rating of *poor* indicates that improvements, replacement or reconstruction are needed in the immediate future.

Table 6: Bransford Elevated Storage Tank Condition Assessment Ratings

Aspect	Rating
Site Security	Fair
Site Civil	Good
Access	Good
Mechanical	Fair
Instrumentation and Controls	Good

6.6 Conclusions

The following conclusions were developed from the evaluation of the Bransford Elevated Storage Tank:

- The chain link fabric and the barbed wire atop the fence need to be remounted and/or tightened in several locations.
- Perimeter vegetation is overgrown and the landscaping requires attention.
- The grading of the site generally allows for proper drainage without ponding or flooding.
- Corrosion was not noted on piping and valves, but the appearance could be improved and additional protection provided by painting.
- The tank is well maintained, but due to its age and some deterioration beginning to be noted on the interior, it will require rehabilitation in the near future.

6.7 Recommendations

The following recommendations were developed based on the evaluation described herein:

- Remount chain link fabric and barbed wire along perimeter fence line.
- Remove and/or or trim perimeter vegetation to improve perimeter visibility and limit climbing access.
- Paint/repaint piping and valves to provide protection and increase the life of the piping and valves.
- Replace walkway and repaint roof and floor plates.

6.8 Cost Implication

Site improvements are expected to cost $10,000. Painting of valves, piping, and internal component replacement and painting is expected to cost $125,000, for a total facility cost of $135,000.

7.0 Hall Johnson Elevated Storage Tank

7.1 Facility History and Overview

The Hall Johnson Elevated Storage Tank was constructed in 1988 by Landmark. The construction of the tank site included the tank, perimeter fencing with vehicular gate, and associated grading and drainage. It is co-located with the City of Colleyville Public Works Service Center.

The tank is a composite construction with a concrete base and a welded steel bowl. The finished floor of the tank's base is at an elevation of 744.5 and the tank's overflow is at an elevation of 790.0. The bowl of the tank is approximately 88 feet in diameter and has a capacity of 1.5 million gallons. A single 24-inch

stainless steel pipe serves as both the inlet and outlet for the tank. The tank is also equipped with a 16-inch stainless steel overflow pipe with flapper valve. The tank has a 42-inch access tube with 24-inch manway for bowl access, 30-inch combination access tube hatch and vent, 30-inch diameter roof hatch, 24-inch roof vent and a separate a vacuum/pressure relief vent. Safety features include handrails, ladder platforms, safety climbs, and obstruction lights on top of the tank. The tank has also been provided with cathodic protection. Table 7 summarizes the tank design data.

Table 7: Hall Johnson Elevated Storage Tank Design Data

Parameter	Value
Location	1601 Hall Johnson Road
Type of Tank	Elevated Storage
Construction	Composite
Year Constructed	1988
Improvements	None
Description of Improvements	N/A
Capacity (MGD)	1.5
Height to Overflow (ft)	132.5
Max Head Range (ft)	40
Bowl Diameter (ft)	88
Inlet/Outlet Diameter (in)	24
Overflow (in)	16

7.2 Service Area

The tank is one of two elevated tanks that service the Low Plane pressure zone. Figure 7, in Section 4 above, presents a pressure plane distribution map.

7.3 Plan Reference

The 1988 Record Drawings prepared by Knowlton-English-Flowers, Inc. for the “Hall Road Elevated Tank” were referenced for the design of the tank site.

7.4 Control Description

The level controls in the tank send signals to L.D. Lockett Pump Station. The pump station is equipped to meet average and peak demands with the pumps dedicated to the Low Pressure Plane.

7.5 Condition Assessment

7.5.1 Site Security

The site is co-located with the City of Colleyville Public Works Service Center. There is an eight foot intruder-resistant chain link fence around the tank site. The fence has barbed wire. There is one vehicular access gate on the east side of the property. The vehicular gate is equipped with a padlock to restrict access and the personnel door on the tank itself is equipped with a keyed lock.

7.5.2 Site Civil

The overall site appears reasonably well kept and appears well drained.

Figure 27: Overall Site Photographs at Hall Johnson Elevated Storage Tank

7.5.3 Access

Access to the site is provided via a service road from Hall Johnson Road and past the City of Colleyville Public Works Service Center parking lot. The vehicular gate at the east side of the site allows vehicle access to the site and also provides an entrance to the 12 foot high by 12 foot wide overhead roll up door at the base of the tank. A 36-inch by 80-inch personnel door provides personnel access at the tank base. The tank is also equipped with a 24-inch diameter manway for bowl access, a 30-inch combination access tube hatch and vent, and a 30-inch diameter roof hatch. These hatches provide access to the bowl for inspection and maintenance purposes.

7.5.4 Mechanical

The exterior of the tank appears to be in very good to like-new condition. The tank itself has locked doors.

Figure 28: Exterior of Hall Johnson Elevated Storage Tank

The interior of the tank provides work and storage space. The area appears to be fairly well organized.

Figure 29: Storage Area of Hall Johnson Elevated Storage Tank

Interior inspection conducted by a City contractor in December 2013 indicated that the interior is in good condition, including the submerged coatings which did not show signs of peeling, blistering or pitting. Additionally, light sediment was cleaned from the tank during the 2013 inspection.

7.5.5 Instrumentation and Controls

Tank level readings are relayed to the SCADA system through the use of the City's radios.

7.5.6 Summary

Table 8 summarizes the findings of the condition assessment based on ratings of good, fair and poor. A condition assessment rating of *good* indicates that no immediate attention is required. A rating of *fair* indicates that some initial repair or minor improvement is needed for the facility to remain in adequate working condition. A rating of *poor* indicates that improvements, replacement or reconstruction are needed in the immediate future.

Table 8: Hall Johnson Elevated Storage Tank Condition Assessment Ratings

Aspect	Rating
Site Security	Good
Site Civil	Good
Access	Good
Mechanical	Good
Instrumentation and Controls	Good

7.6 Conclusions

The following conclusions were developed from the evaluation of the Hall Johnson Elevated Storage Tank:

- The location of the site behind the City of Colleyville Public Works Service Center, along with barbed wire fencing appears to be adequate in maintaining a secure facility.
- The grading of the site generally allows for proper drainage without ponding or flooding.
- The tank and piping are well maintained.

7.7 Recommendations

Based on the visual observations made and input from Staff, no improvements are needed at this time.

7.8 Cost Implications

No costs for improvements are anticipated.

8.0 McPherson Elevated Storage Tank

8.1 Facility History and Overview

The McPherson Elevated Storage Tank was constructed in 1998 by Landmark. The construction of the tank site included the tank, perimeter fencing with vehicular gate, and associated grading and drainage. The tank is located in a park area on McDonwell School Road, west of Westcoat Drive.

The tank is a composite construction with a concrete base and a welded steel bowl. The finished floor of the tank's base is at an elevation of 701.0 and the tank's overflow is at an elevation of 840.0. The bowl of the tank is approximately 69.5 feet in diameter and has a capacity of 1.0 million gallons. A single 16-inch stainless steel pipe serves as both the inlet and outlet for the tank. The tank is also equipped with an 8-inch stainless steel overflow pipe. The tank has a 30-inch diameter tank floor hatch, a 30-inch diameter

roof hatch, a vent, a 24-inch exhaust flange, a vacuum/pressure relief vent, and a 30-inch combination access tube hatch and vent. Safety features include handrails, ladder platforms, safety climbs, and obstruction lights on top of the tank. Table 9 summarizes the tank design data.

Table 9: McPherson Elevated Storage Tank Design Data

Parameter	Value
Location	240 McDonwell School Rd
Type of Tank	Elevated Storage
Construction	Composite
Year Constructed	1998
Improvements	None
Capacity (MGD)	1.0
Height to Overflow (ft)	139
Max Head Range (ft)	40
Bowl Diameter (ft)	69.5
Inlet/Outlet Diameter (in)	16
Overflow (in)	8

8.2 Service Area

The tank services the High Plane pressure zone and is the only elevated storage within this pressure zone. Figure 7, in Section 4 above, presents a pressure plane distribution map.

8.3 Plan Reference

The 1998 Record Drawings prepared by Kimley-Horn and Associates, Inc. for the “1 M.G. High Plane Elevated Storage Tank” were referenced for the design of the tank site.

8.4 Control Description

The level controls in the tank can send signals to both the Overland Trail Pump Station and the L.D. Lockett Pump Station. The pump stations are equipped to meet average and peak demands with the pumps dedicated to the High Pressure Plane. Currently, Overland Trail Pump Station is not in use, therefore the L.D. Lockett Pump Station serves and the sole source to fill the McPherson Elevated Tank.

8.5 Condition Assessment

8.5.1 Site Security

The site is located within a park that contains a splash park, basketball court and hiking trail. There is an eight foot chain link perimeter fence with privacy slats around the site. The fence does not have barbed wire. There is one vehicular access gate and one three foot wide personnel gate on the west side of the property. The vehicular gate is equipped with a padlock to restrict access and the personnel door is equipped with a lock. Additionally, the electric panel located near the gates is equipped with a padlock.

8.5.2 Site Civil

The tank site is relatively well graded for drainage during rain events. The majority of the site is grass-covered which reduces the amount of runoff. The tank's overflow pipe terminates on the northwest side of the tank above a concrete splash pad. Due to erosion from overflow water in the past, a concrete channel was extended to help alleviate erosion that was occurring. While this reduced the erosion, additional measures need to be taken to repair some of the washed out areas and prevent further soil loss. Erosion has occurred on the north side of the tank and requires attention.

8.5.3 Access

Access to the site is provided via a park road from McDonwell School Road to the parking lot on the west side of the site where a large vehicle gate and a three foot wide personnel gate are located. Site access is restricted by a chain link fence with locked gates. The vehicular gate allows vehicle access to the site and also provides an entrance to the 10 foot high by 12 foot wide overhead roll up door at the base of the tank. A 3 foot wide by 7 foot tall personnel door provides personnel access at the tank base. The tank is also equipped with a 30-inch diameter tank floor hatch, a 30-inch diameter roof hatch, and a 30-inch combination access tube hatch and vent. These hatches provide access to the bowl for inspection and maintenance purposes.

8.5.4 Mechanical

The exterior of the tank appears to be in very good to like-new condition.

Figure 30: McPherson Elevated Tank

The valves appear to be in good operating condition and no problems were noted. The flap valve on the overflow pipe appears operational and seats well.

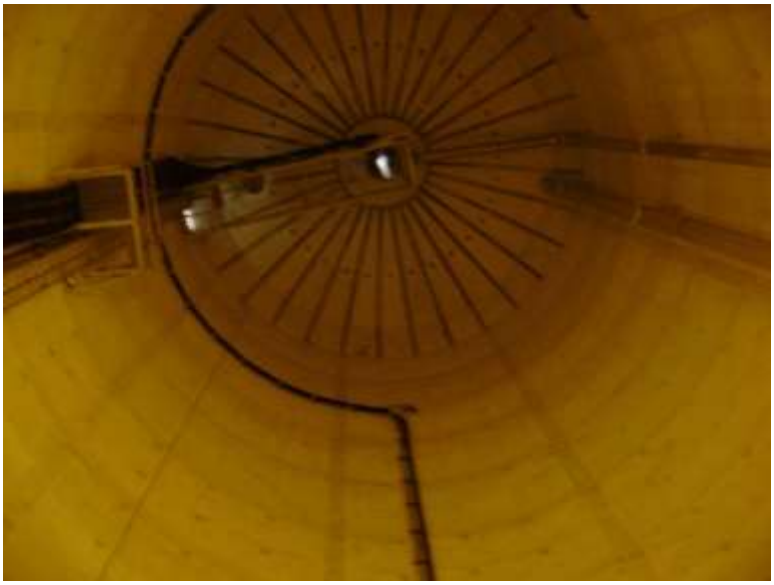

Figure 31: Inside of Base of McPherson Tank

Figure 32: Electric Control Panel and Peeling Paint at McPherson Tank

Interior inspection conducted by a City contractor in December 2013 indicated that the structure is in good condition. The side manway hatch was designated as being in fair condition. The screen had detached from the vent and required replacement. Submerged coatings generally did not show signs of peeling, blistering or pitting with the exception of internal floor plates with minor blistering. Sand and silt were removed during the 2013 inspection and additional cleaning would not be required.

8.5.5 Instrumentation and Controls

Tank level readings are relayed to the SCADA system through the use of the City's radios.

8.5.6 Summary

Table 10 summarizes the findings of the condition assessment based on ratings of good, fair and poor. A condition assessment rating of *good* indicates that no immediate attention is required. A rating of *fair* indicates that some initial repair or minor improvement is needed for the facility to remain in adequate working condition. A rating of *poor* indicates that improvements, replacement or reconstruction are needed in the immediate future.

Table 10: McPherson Elevated Storage Tank Condition Assessment Ratings

Aspect	Rating
Site Security	Good
Site Civil	Fair
Access	Good
Mechanical	Fair to Good
Instrumentation and Controls	Good

8.6 Conclusions

The following conclusions were developed from the evaluation of the McPherson Elevated Storage Tank:

- There is proper grading for the site, although there is erosion that has been caused by the tank overflow.
- The tank appears to be in very good to like new condition.
- The piping and controls appear to be in good condition; however, paint is peeling on some of the piping inside the base of the tank.

8.7 Recommendations

The following recommendations were developed based on the evaluation described herein:

- Repairs need to be made where the ground has been eroded due to the tank's overflow piping runoff and additional measures should be taken to prevent future soil loss and erosion. Extending the concrete drainage channel or placing riprap at the end of the existing channel would help remedy the erosion.
- Repaint the piping to provide protection and increase the life of the pipe.

8.8 Cost Implications

Site civil and painting improvements are expected to cost $12,500.

9.0 Summary

9.1 Recommended Improvements

Table 11 displays a summary of recommended improvement costs at each location. Engineering and professional services have not been included in this cost evaluation, as it is anticipated that these projects would be developed and executed by the City engineering staff.

Table 11: Summary of Recommended Improvements

Site	Improvements Description	Estimated Cost
Overland Trail	GST demolition, equipment salvage	$75,000
LD Lockett	Site security, site drainage	$10,000
Bransford	Site security, painting, and interior component replacement	$135,000
Hall Johnson	None	$0
McPherson	Site grading and painting	$12,500
	Total	**$232,500**

3010 Gaylord Parkway
Suite 190
Frisco, TX 75034

TEL 972.377.7480
FAX 972.377.8380

www.GarverUSA.com

TECHNICAL MEMORANDUM

To: City of Colleyville **Date:** September 5, 2014

Attn: Bob Lowry, P.E.

From: Jeff Sober, P.E.

RE: Overland Trail GST and PS

Introduction. The purpose of this technical memorandum (TM) is to determine the feasibility of permanently decommissioning versus rehabilitating the Overland Trail ground storage tank (GST) and pump station (PS). The combination of those facilities is referred to simply as Overland Trail.

Overland Trail is located within the high pressure plane of the City of Colleyville's distribution system, and is presently offline due to the poor condition of the existing GST. Currently, the LD Lockett GST and PS (LD Lockett) supplies all water that is pumped into the high plane. LD Lockett also contains a low plane PS which relies on the same GST as the high plane PS. However, this memorandum focuses only on the high plane system, with the understanding that improvements to the high plane will benefit the low plane (by increasing flow capacity for the LD Lockett low plane facility).

High Plane Overview
The high plane covers the northwest portion of the City's water distribution system (see Figure 1).

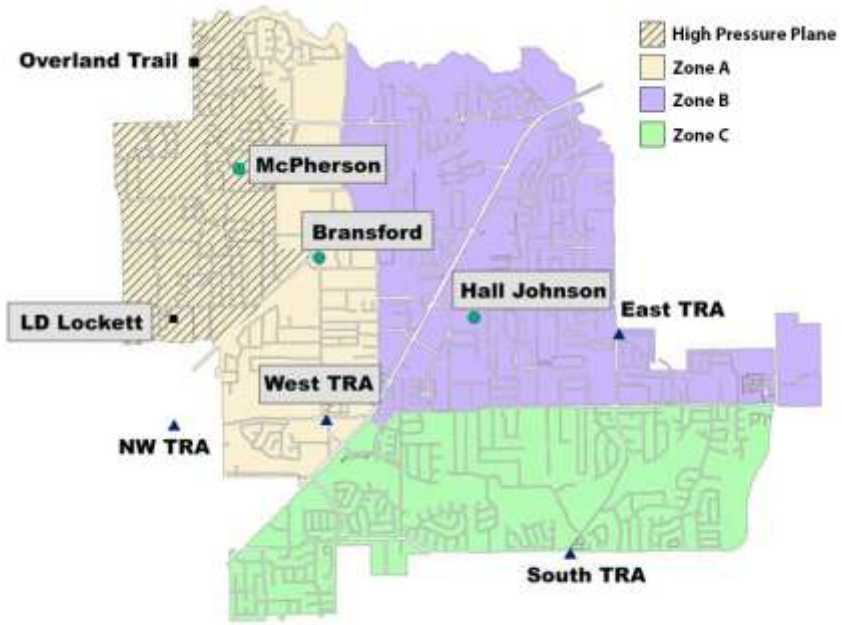

Figure 1 – Pressure Plane Distribution

Approximately 15% of the system resides within the high plane, with the majority of the system (approximately 85%) being covered by the low plane. The high plane pumps at LD Lockett are the only entry route for water into the high plane. Storage is provided within the high plane by the McPherson elevated storage tank (EST), which has a capacity of approximately 1 million gallons (MG). Overland Trail is also located within the high plane, and is supplied by the high pressure plane interconnecting piping.

High Plane Supply and Storage

Active

Water is currently conveyed to the high plane by two sets of pumps at LD Lockett, as displayed in following table.

LD Lockett High Plane Pump Station					
Type	# of Pumps	Shutoff Head (ft)	Design Flow (gpm)	Design Head (ft)	Pumping Capacity (MGD)
Average Flow	2	211	750	204	2.16
Peak Flow	2	264	1,500	227	4.32

The design head for the peak flow pumps is greater than the shutoff head for the average flow pumps. Therefore, assuming the peak flow pumps are operating near their design point, the average flow pumps will not be able to operate concurrently with the peak flow pumps. The maximum flow rate of water from LD Lockett into the high plane is estimated at 3,000 gpm (4.320 MGD). The McPherson EST provides storage within the high plane, with a maximum volume of 1.000 MG.

Inactive

Overland Trail consists of a 1.2 MG GST and four pumps, as shown in the following table.

Overland Trail Pump Station				
Type	# of Pumps	Design Flow (gpm)	Design Head (ft)	Pumping Capacity (MGD)
Average Flow	2	500	150	1.44
Peak Flow	2	750	150	2.16

However, the pumps at Overland Trail are limited by the size of the GST, and are capable of supplying the total GST amount of 1.2 MG.

Total

The current maximum total amount of water that can be utilized with the high plane is the sum of the volume of water that can be pumped into the zone through LD Lockett and the storage within the McPherson EST. Additional capacity is available if Overland Trail is brought back into service.

The maximum day supply for the existing infrastructure is shown in the following table.

Facility	Maximum Daily Supply Volume (MG)
LD Lockett	4.320
McPherson EST	1.000
Total (Active Facilities)	**5.320**
Overland Trail (Inactive)	1.200

Demand Analysis
The planning horizon for the current study is 20 years, so this analysis included evaluations with present (2014) and future (2034) demand levels.

Overall System Demands. The current and projected water demands are summarized in the following table.

Demand Condition	Current Usage		2034 Future Usage	
	(gpcd)	(MGD)	(gpcd)	(MGD)
Base	122	2.879	122	3.345
Average	303	7.031	342	9.378
Maximum	823	19.423	823	22.567

High Plane Demands. The City's system is currently divided into three watering zones, and watering is allowed on the following schedule:

- Zone A: Tuesday and Friday
- Zone B: Wednesday and Saturday
- Zone C: Thursday and Sunday
- No watering: Monday

Daily data provided by the City was analyzed to develop a relationship between high and low plane demands and the relationship of watering zones. The average total system usage on no watering days for the period of July 1 through August 14, 2014 was 7.091 MGD. This value was assumed to be the base summer usage.

The maximum day usage of 19.423 MGD is expected to occur on a Zone B watering day, as Zone B contains the largest number of meters and historically produces the maximum demand since watering zones were implemented. Assuming the base usage remains constant at 7.091 MGD, the excess usage on the maximum day would be 12.332 MGD. The maximum excess usage for Zone B (12.332 MGD) is 1.71 times the peak Zone B excess usage during the period of record. This ratio of 1.71 is used for all zones in developing the anticipated maximum day usage by watering zone, as shown in the following table.

Watering Zone	Base Usage (MGD)	Excess Usage for Period of Record (MGD)	Excess Usage Peak Ratio (MGD)	Maximum Day Excess Usage (MGD)	Total Maximum Day Usage (MGD)
A	7.091	5.196	1.713	8.903	15.993
B	7.091	7.197	1.713	12.332	19.423
C	7.091	6.213	1.713	10.646	17.736

The water meter billing records from 2013 were then analyzed and applied to each zone and pressure plane, as follows:

Watering Zone	Pressure Plane	Number of Meters	Percentage of Meters with Zone	Percentage of Meters within System
A	High	1,541	49.1%	14.2%
A	Low	1,595	50.9%	14.7%
B	Low	4,503	100.0%	41.6%
C	Low	3,195	100.0%	29.5%

Zone A Maximum Day Demands. The usage within each pressure plane is the combination of the base and excess demands for the plane. Based on the previous meter data, the total of Zone A Maximum 15.993 MGD is divided as follows:

- High Plane: 5.384 MGD
- Low Plane: 10.610 MGD

For future (2034) projected maximum day usage, it is assumed that the average per capita demand will remain the same and that the ratio of high plane to low plane usage will remain constant for Zone A watering days. This assumption was made following investigation of the number of undeveloped parcels within different planes and watering zones. The projected maximum day usage for Zone A watering days is presented in the following table.

Area	2014 Usage (MGD)	2034 Usage (MGD)
Total System	15.993	18.582
High Plane	**5.384**	**6.255**
Low Plane	10.610	12.327

Comparison of Usage and Supply on a Daily Basis
The existing infrastructure is insufficient to supply peak demands if the maximum usage exceeds the maximum supply. Comparison of the maximum day usage values (current and projected) with the existing maximum supply reveals a deficiency for both demand scenarios.

	2014	2034
Maximum Supply Capacity (MGD)	5.320	5.320
Maximum Usage (MGD)	5.384	6.255
Supply Surplus/ Deficit (MGD)	**-0.064**	**-0.935**

Comparison of Usage and Supply on an Hourly Basis

A scenario was developed with the hydraulic model to assess the high plane under existing maximum day demand conditions. Specifically, the scenario was set-up with demands representative of a maximum day event during a Zone A watering day. Diurnal curves were also generated and implemented based on historical usage rates.

The model was set-up with the following conditions, which attempt to maximize the ability of the infrastructure to provide adequate supply to the high plane:

- McPherson EST Initial Level: 840.00’ (at overflow level)
- LD Lockett PS High Plane Peak Flow Pumps: ON at McPherson EST level of 838.00’, OFF at McPherson EST level of 839.95’
- LD Lockett GST Initial Level: 693.00’
- NW TRA Meter Station Open/Close Status: OPEN at LD Lockett GST level of 688.00’, CLOSED at LD Lockett GST level of 693.00’
- NW TRA Meter Station Maximum Flow: 4,200 gpm

It is assumed that the high plane average flow pumps at LD Lockett cannot operate concurrent to the high plane peak flow pumps. Therefore, the high plane average flow pumps were not set to turn on during this model simulation.

Two scenarios were completed as follows:

- 2014 Max Day with no Overland Trail
- 2014 Max Day with Overland Trail

Figures 2 and 3 document the simulation results.

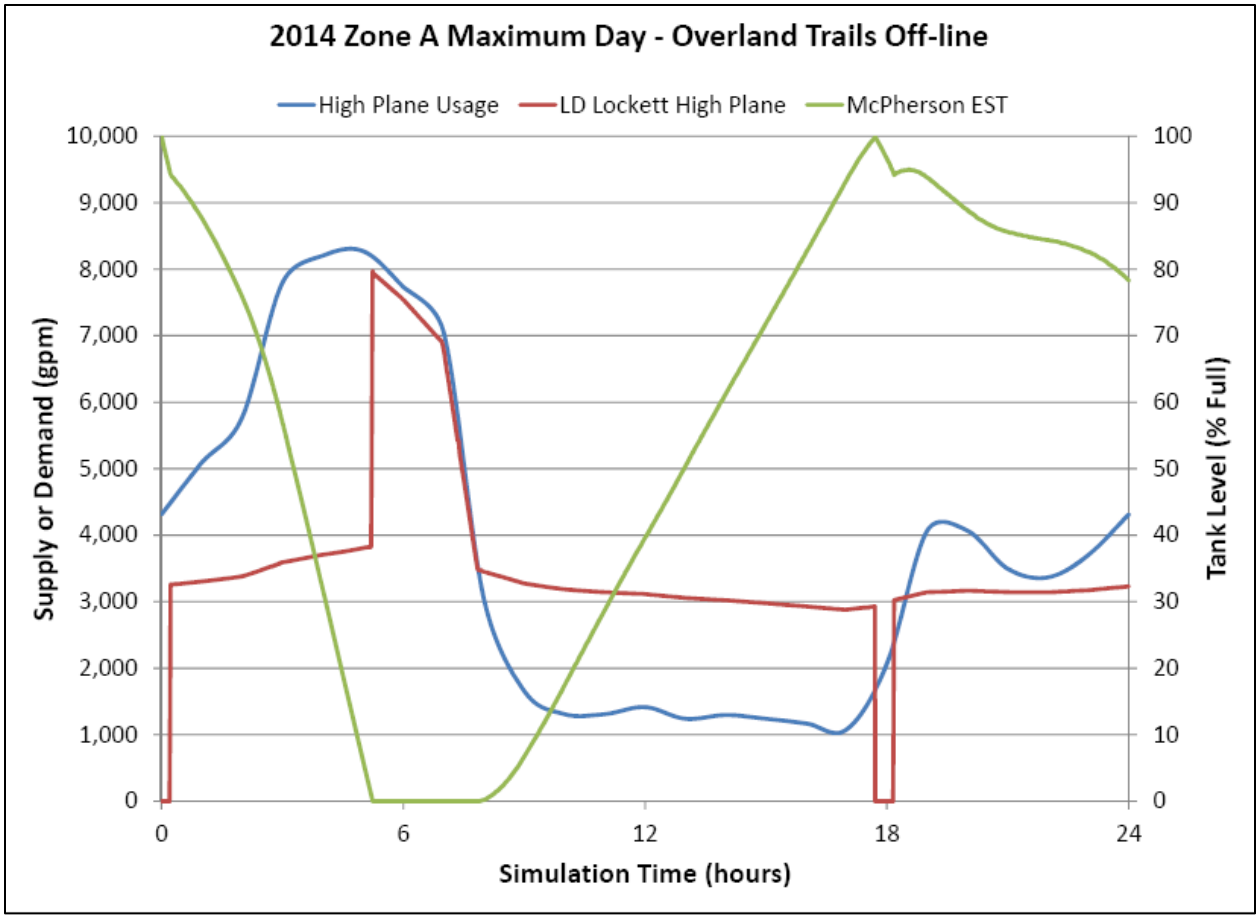

Figure 2 – Simulation with OT off-line

Consistent with the previous Daily Basis calculations, the system is unable to keep up with a peak day demand without Overland Trail online. McPherson EST is drained for a significant portion of time, and at approximately the 5 hour mark, the system hits 0 pressure, which unrealistically increases LD Lockett's pumping capacity. This is not a feasible scenario.

The following Figure 3, however, demonstrates that placing Overland Trail back online helps the system to recover from the hourly peak demand, without draining McPherson EST for an extended period of time.

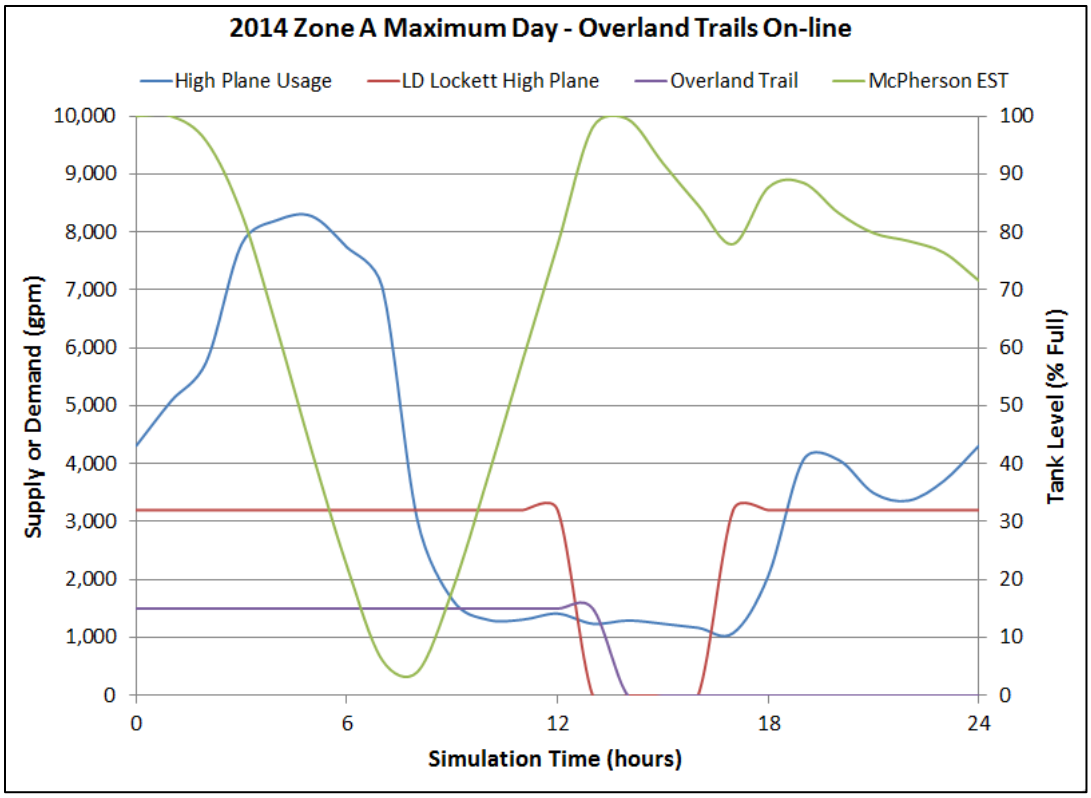

Figure 3 – Simulation with OT on-line

It was then assumed that a future scenario without Overland Trail would not be feasible, and modeling was conducted to determine the required sizing for the Overland Trail GST, assuming all Overland Trail pumps were utilized in a peak day scenario. As displayed in Figure 4, a GST volume of 2 MG provides adequate storage within the system to avoid extended draining of the McPherson EST. However, the tank is drained for a period of approximately one hour, and the system is completely dependent upon LD Lockett and Overland Trail for that time period, until the system demand decrease allows the tanks to catch back up. This scenario can be avoided by implementation of future pumping upgrades at either Overland Trail or LD Lockett.

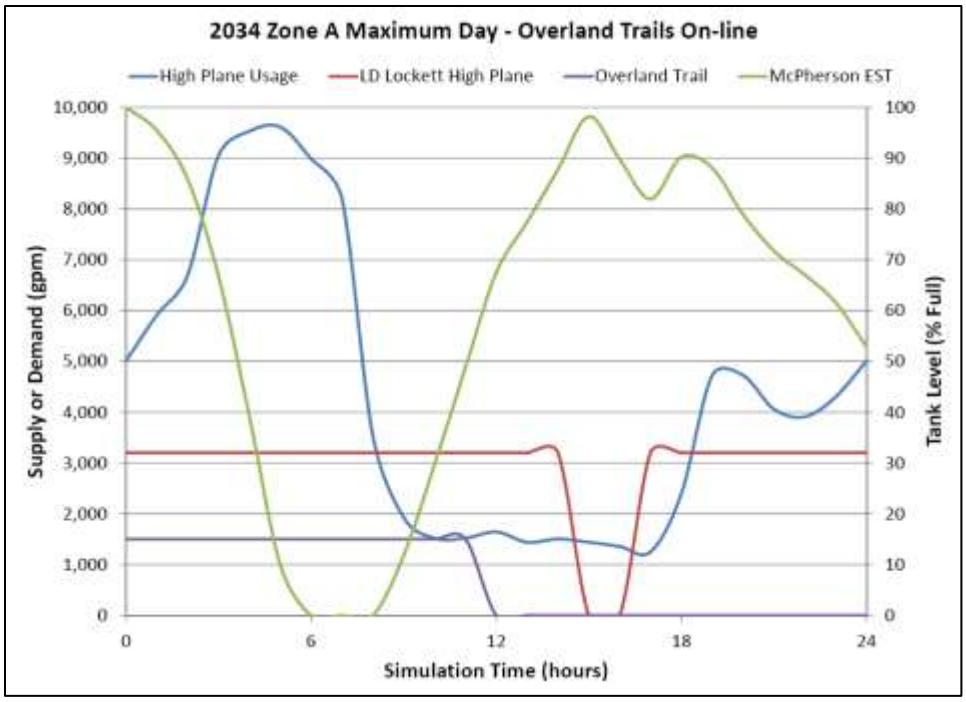

Figure 4 - Future Simulation with OT Online

Alternative to Improvements under Zone Watering Restrictions
The high pressure plane meters account for 14.2% of the total system demand. Assuming that the City elects to remove zone watering restrictions and water on an alternative City-wide usage schedule (such as odd/even address usage), the peak demand in each zone could decrease significantly. Potential high pressure plane usage numbers are shown in the following table

Max Day High Pressure Plane Usage (MG)		
	2014	2034
Current zone restrictions	5.384	6.255
Existing system capacity	5.32	5.32
Zoneless city-wide restrictions	2.76	3.20

Summary and Conclusions. The existing (2014) and projected (2034) maximum day usage in the high plane are estimated at 5.384 and 6.255 MG, respectively. Utilizing historical data, population and flow projections, and current zone watering restrictions, the existing infrastructure (i.e., LD Lockett and McPherson EST) is not capable of meeting this current and future demand.

Rehabilitation of Overland Trail with a new 2.0 MG GST and utilizing the existing pumping infrastructure would result in an addition of supply to the high plane adequate to cover present needs, and allow for the City to implement future pumping improvements to fully address the future peak day.

However, an alternative to these improvements would be the implementation of a new watering policy that provides equal restrictions across the City, and removes zones entirely. This scenario would not require additional maximum day storage and pumping capacity infrastructure within the high plane, and would avoid identified capital improvements for the City of Colleyville, while providing the same overall peak demand capacity. This is the preferred approach by the City, and will be incorporated in existing and future models.

Technical Memorandum

Water/Wastewater Master Plan

Lift Stations Evaluation

Prepared For:

City of Colleyville

July 2014

GARVER

GarverUSA.com

Technical Memorandum

Water/Wastewater Master Plan
Lift Stations Evaluation

City of Colleyville, Texas

Prepared by:

3010 Gaylord Parkway
Suite 190
Frisco, TX 75034

July 2014

Garver Project No.: 13088150

Engineer's Certification

I hereby certify that this Technical Memorandum for the Water/Wastewater Master Plan – Lift Stations Evaluation was prepared by Garver under my direct supervision for the City of Colleyville, Texas.

Lance P. Klement, PE
State of Texas PE License 113630

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Appendix A Wastewater Lift Station Evaluation Form

1.0 Executive Summary

The City of Colleyville (City) has retained Garver, LLC to perform an evaluation of its lift stations to determine compliance with the current Texas Commission on Environmental Quality's (TCEQ) 30 Texas Administrative Code (TAC) Chapter 217 regulations.

2.0 Introduction

2.1 Project Scope

The City of Colleyville's wastewater collection system is comprised of 896,000 LF of sewer lines, over 3,029 manholes, and three lift stations. One of the lift stations is a simplex grinder pump that serves an individual residence. As this station is maintained by the resident it was not included in the assessment. The two lift stations that were evaluated were the Reserve and Overland Trails. The scope of service was to perform a condition assessment of the lift stations, which was accomplished by means of a site visit where an engineering team evaluated the site, piping, pumps, instrumentation and controls. The findings are documented in this report, along with background data, field notes, recommendations for replacement and repair and associated cost estimates to complete any recommendations.

Structural and electrical condition assessments were not performed and the field investigations are based on visual observations and review of plans for the facilities in regards to general condition of the pumping facilities, security, drainage, and TCEQ compliance. Field investigation of the facilities is not a full condition assessment.

2.2 Site Description

Both lift stations are located on the northwest side of the city. See Figure 1 for the location map. The Overland Trails Lift Station is located at 7303 Overland Trail near the intersection of Overland Trail and Bandit Trail. Overland Trail is oriented north to south and the lift station is located approximately twelve feet from the road. See Figure 2 for the Site Location. The lift station is located adjacent to two houses and has a slatted chain link fence around the exterior with the required signage posted on the gate. The site consists of a wet well cover with a hatch, associated control panels and electrical equipment, and a water tap. Three sewer valve boxes are located adjacent to the wet well cover. The site is in good condition with no visible maintenance issues. The grade is uneven due to the aggregate used as ground cover, but the site does not appear to have drainage problems.

Figure 1: Lift Station Site Map

Figure 2: Overland Trail Site Location

The Reserve Lift Station is located at 900 Rembrandt near the intersection of Rembrandt and Monet. Rembrandt is oriented east to west and the lift station is located approximately ten feet from the curb. See Figure 3 for the Site Location. The lift station is bordered by a house on the west side and a green space with trees and a detention pond to the east. The site has a brick fence around the boundaries of the site with the required signage posted on the gate. The site consists of a wet well with a hatch, associated control panels and electrical equipment which are under a metal canopy, and a water tap. A valve vault is located adjacent to the wet well. The entire site is paved with concrete and drainage does not appear to be a problem. The site is in good condition with no visible maintenance issues.

Figure 3: Reserve Site Location

3.0 Assessment Process

To have an effective assessment of a lift station, it is important to understand the parameters, procedures, and outcomes desired before the assessment is completed. This serves as a quality control measure and insures the end product meets high standards. The approach used in the assessment process is summarized below.

3.1 Data Collection and Review

This task included assembling and reviewing all available and relevant documents provided the City, which include the following:

1. Record documents
2. Operation and maintenance records
3. Equipment cut sheets and catalogs
4. Pump performance curves
5. Existing site surveys
6. Code requirements for the site
    a. TCEQ
    b. Army Corp of Engineers
    c. International Building Code
    d. City/County Ordinances and Code
    e. National Fire Protection Association Codes
    f. OSHA Compliance

3.2 Field Review and Condition Assessment

3.2.1 Field Review

Garver sent a team of engineers to the lift station sites on March 13, 2014. On this day the field review and staff interviews were completed.

3.2.2 Condition Assessment

Prior to conducting the field assessment of the facilities, a matrix checklist was developed to be able to systematically evaluate the facility and equipment. This checklist compromises the bulk of the review. Results of the checklists are provided in Appendix A. The field review included photographing and dimensioning all equipment and appurtenances as well as visually observing fit and function.

3.3 Identification of Deficiencies

Once the documentation review and site assessments were completed, the deficiencies in the system could be identified and addressed. The following categories were addressed and compared against existing codes, Best Available Practices, or other criteria applicable to each category.

- Mechanical
- Controls
- Instrumentation
- Safety
- Environmental
- Performance

4.0 Overland Trails Lift Station

4.1 Lift Station History and Overview

The lift station was originally constructed in 1990 and was renovated in 2010. The renovation consisted of replacing the cover and hatch, discharge piping, floats, control wiring, and repainting the interior of the

wet well. The existing wooden fence was replaced with a chain link fence with privacy slats and a mow strip around the base of the fence. A slab was poured for the new electric and control panels.

4.2 Service Area

The lift station services a small area in the surrounding neighborhood, including approximately 30 houses. The service area is shown in Figure 4. The flow from the lift station is conveyed approximately 900 feet and is discharged into a manhole. It then gravity flows to the northeast, eventually entering the 12-inch TRA interceptor.

4.3 Plan Reference

The 2010 Overland Trail Lift Station renovation plans, designed by Deltatek Engineering, were referenced for the modifications performed at the facility site.

Figure 4: Overland Trails Service Area

4.4 Control Description

The lift station pumps are controlled by a series of mercury floats that pump the lift station wet well down according to set elevations. The floats terminate into the control panel where they activate a motor contactor to start and stop the pumps. The control box has three switches present, a master on/off, a pump # 1 hand/off/auto and a pump # 2 hand/off/auto. The control panel is also connected to SCADA so that staff can monitor alarms and run status reports. The lift station also has two 60 amp breakers in the control panel with the pump switches. An electrical disconnect is in a separate panel. The control schematic for the lift station is shown in Figure 5 below.

4.5 Condition Assessment

4.5.1 Site Security

The site is located in a residential development and has an eight foot, chain link perimeter fence with privacy slats. There is one access gate which can accommodate a vehicle. The gate is equipped with a padlock to restrict access. Additionally, there are padlocks on the access hatch of the wet well, the control panel, and the electrical disconnect panel. The fence does not have barb wire.

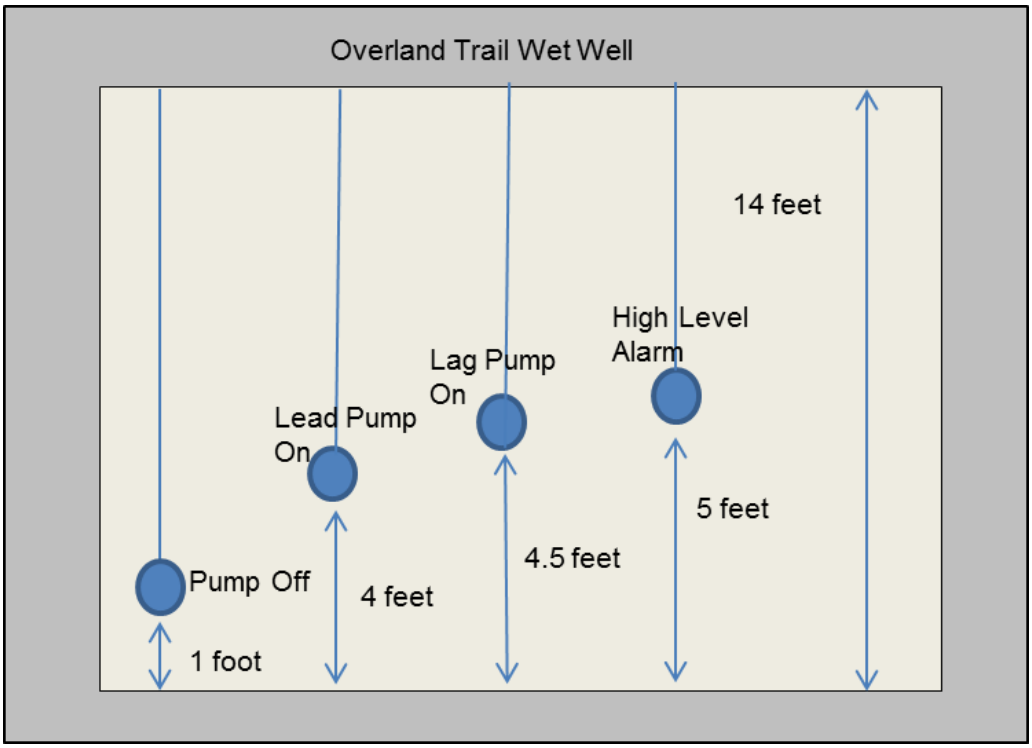


Figure 5: Overland Trail Control System Schematic

Figure 6: Fencing and Locks at Overland Trail Lift Station

4.5.2 Site Civil

The lift station site is well graded for drainage during rain events and the ground is covered with large aggregate rather than grass or concrete, which acts as a semi-permeable surface and reduces the amount of runoff from the site. While there is a slight lip between the aggregate and the mow strip around the fenced perimeter, there does not appear to be any drainage problems or areas that would pond water. The concrete pad for the control panel and the mow strip appear to be in good shape. There does not appear to be any deterioration or significant cracking. The piping and supports also appear to be in good condition. There are signs of minor rusting on the piping and supports in the wet well, but they appear to be surface rust and not extensive.

The gravity line discharging into the wet well appears to be in good condition with no signs of corrosion or improper sealing. There did not appear to be any leakage around the pipe penetration. The three valve boxes with the isolation valves are plastic and do not show any signs of wear or cracking and appear to be in good condition, as does the piping and valves in the boxes.

4.5.3 Access

The site has sufficient access for maintenance, as the access gate opens to Overland Trail and will easily accommodate a work truck and provide access to the wet well. There is also a hoist attached to the wet well head to aid the maintenance personnel in pulling the pumps out of the wet well for service.

The hatch access to the top of the wet well is a Halliday aluminum hatch. The hatch does not have any paint or coatings and is fastened to the top of the wet well with two hinges and is secured with a padlock. The hatch is a 3’-6” x 3’-6” hatch opening and provides adequate access to the pumps, controls, etc.

4.5.4 Pumping and Mechanical

The Overland Trails Lift Station was originally constructed in 1990 and was renovated in 2010 to replace the pumps with two Hydromatic model HPGH500M3-2 submersible pumps. The pumps have 5 HP motors with a design point of 70 gpm at 104 feet of TDH. The station has only single phase power; therefore a VFD was utilized to convert power from single phase to three phase. The lift station has a service area that includes 30 houses. Based on an average of 3 people per house and an assumed value of 80 gallons per day per capita of wastewater and a peaking factor of 4, the pumping capacity needed to provide service for these households is 25 gpm. The lift station has a firm capacity of 70 gpm, which is more than sufficient for the service area.

It was noted during the staff interviews that the pumps have been experiencing excessive instances of clogging. The clogging has increased since the soft starts were installed. The operators feel that the slower startup of the pump may allow debris to be pulled into the impeller and clog it before the impellers get up to full speed where they would normally cut up the material. The pump housings do not appear to have any major concerns and the City staff stated that the impellers are still sharp and in good operating condition.

4.5.5 Controls and Instrumentation

The controls for the lift station consist of the mercury floats, motor starters and control switches. The controls are all in good working condition and operate as designed and intended. Visual inspection of the floats indicates they float freely and operate the pumps at the design points.

4.5.6 Condition Assessment

Figure 7: Chain Link Fence at Overland Trail Lift Station

Figure 8: Control Panel at Overland Trail Lift Station

Figure 9: Valve Boxes at Overland Trail Lift Station

Figure 10: Wet Well Vent at Overland Trail Lift Station

Figure 11: Pump Controls at Overland Trails Lift Station

Table 1: Overland Trails Condition Assessment Ratings

Aspect	Rating
Site Security	Fair
Site Civil	Good
Access	Good
Pumping and Mechanical	Fair
Controls and Instrumentation	Good

4.5.7 Lift Station Design Data

Table 2: Overland Trails Lift Station Design Data

Lift Station	Overland Trail
Address	7303 Overland Trail
Year Constructed	January 1990
Recent Improvements	February 2010
	Renovated to replace pumps, fencing, and SCADA
Capacity one pump, gpm	70
Rated Total Dynamic Head (TDH), ft	104
Type of Pump	Submersible Grinder
Manufacturer	Hydromatic
Model	HPGH500M3-2
Volts/Ph/Hz	230/3/60 (VFD used to convert single phase to 3 phase power)
Motor HP	5
Impeller Dia.	6.25-inches
RPM	3450
Force Main Length, ft	900
Force Main Diameter	3-inch

4.5.8 30 TAC Chapter 217 Requirements

Table 3: 30 TAC Chapter 217 Requirements

Does Station have some available capacity	Yes
217.35 Backup Power Requirements	Since less than 100 gpm, station has connections for portable generator, SCADA provided,
217.61 Design Considerations	
217.61 a Level Controls provided,	Yes, floats
217.61 b Above 100 year flood pain	Yes, above 100 yr.
217.61 c Separate wet well and valve vaults	Yes
217.61 d Pump Cycle time greater than 6 minutes.	Yes
217.61 e Wet well Ventilation	No, passive 3-inch vent
217.64 Pump Type is acceptable	Submersible grinder, yes
217.63 Lift Station Piping (and Valves)	Yes
217.64 Emergency Provision for Lift Station	Yes, alarms and SCADA provided
217.69 Force Main Design	Velocities greater than 3 fps ok for grinder pumps
Velocity in Force Main (one pump)	3.2 fps
217.93 Security Fencing	No barb wire

4.6 Conclusions

The following conclusions were developed from the evaluation of the Overland Trail lift station:

- The station has been well maintained and is in very good condition following the renovations completed in 2010.
- The pumps have been sized to handle the demands and flows they will experience at peak flows and will be able to keep up with the sewer being delivered by gravity to the station during higher flow periods.
- The electrical and controls are in good operating condition and have not posed any problems for the City staff.
- The site security procedures for locking all the equipment hatches and panels are serving their purpose.
- There is no barb wire atop the fence.
- The grading of the site allows for proper drainage of the site without ponding or flooding.
- The vent for the lift station is smaller than the recommended size per Chapter 217.
- The impellers on the pumps are routinely becoming clogged from material in the wet well.

4.7 Recommendations

While the vent appears to be sized appropriately for the lift station, it is only 3 inches in diameter which is less than the minimum required by TCEQ. The vent pipe should be enlarged to 4 inches in diameter or a variance obtained from the TCEQ for the smaller vent.

The fence does not currently have the single strand of barb wire atop the fence as required per Chapter 217 regulations. Barb wire should be added to the fence to meet the regulations.

A further investigation should be performed to determine if the slow starts of the pumps are making them susceptible to clogging.

4.8 Cost Implications

Cost to be inserted here once consensus on recommendations is reached.

5.0 Reserve Lift Station

5.1 Lift Station History and Overview

The lift station was originally constructed in 2007 as part of the Reserve subdivision. The construction of the facility consisted of the perimeter fencing with vehicular gate, wet well, valve vault, electrical and control panels with canopy to protect staff from the weather, and concrete paving inside the facility.

5.2 Service Area

The lift station services the surrounding neighborhood of the Reserve development, which currently includes approximately 40 houses. At build out, the lift station will serve approximately 77 houses. The service area is shown in Figure 12. The flow from the lift station is conveyed approximately 1,400 feet and is discharged into a manhole. It then gravity flows to the southeast, eventually entering the 36-inch TRA interceptor on the south side of the city.

Figure 12: Overland Trails Service Area

5.3 Plan Reference

The plans for the Reserve Lift Station were developed in June 2007 by Carter and Burgess as part of the plan set for the Reserve subdivision development. These plans have been used to confirm dimensions, sizing of electric and control settings for this assessment.

5.4 Control Description

The lift station pumps are controlled by a series of mercury floats that pump the lift station wet well down according to set elevations. The floats terminate into the control panel where they activate a motor contactor to start and stop the pumps. The control box has three switches present, a master on/off, a pump # 1 hand/off/auto and a pump # 2 hand/off/auto. The lift station also has two 60 amp breakers in the control panel with the pump switches. An electrical disconnect is in a separate panel. The control schematic for the lift station is shown in Figure 13 below. The control panel is also connected to SCADA so that staff can monitor alarms and run status.

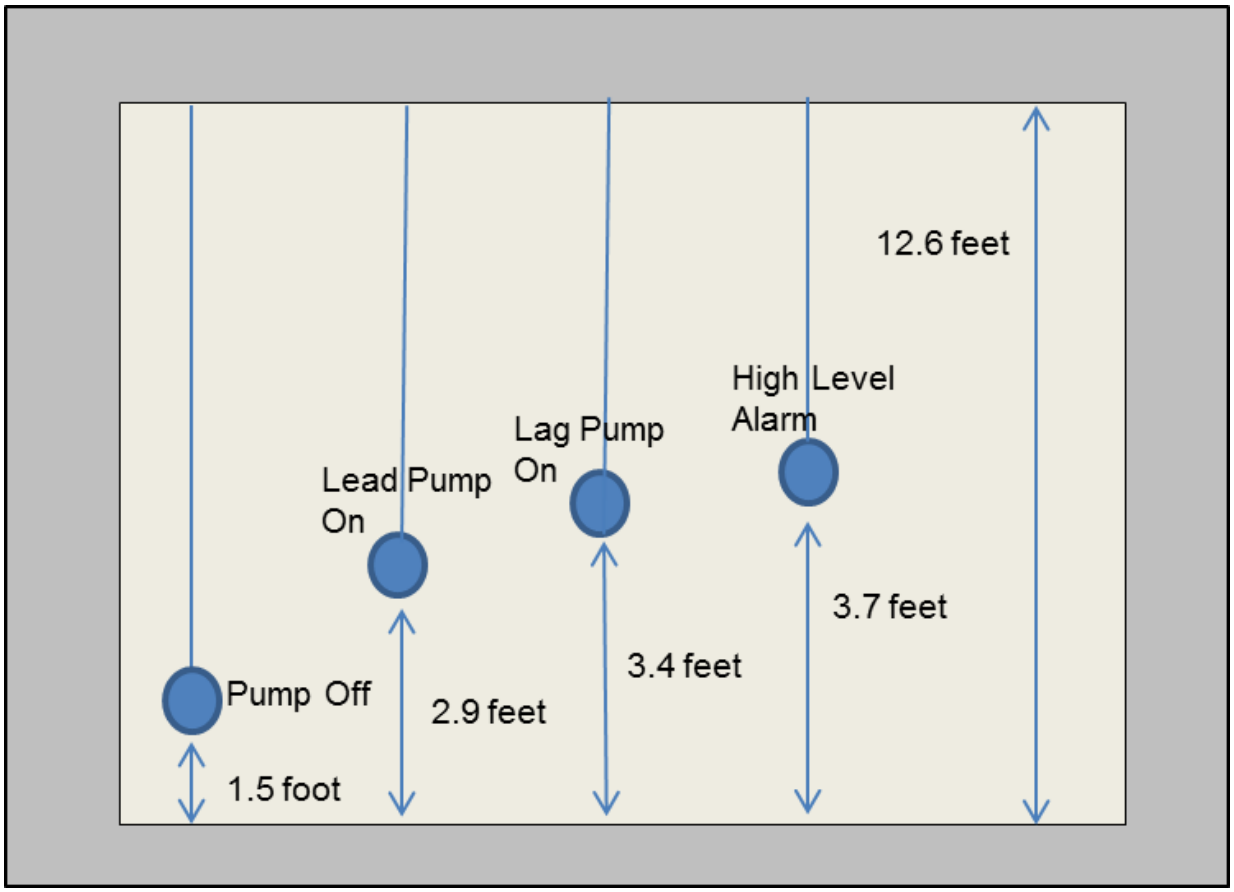


Figure 13: Reserve Control System Schematic

5.5 Condition Assessment

5.5.1 Site Security

The site is located in a residential development and has an eight foot, brick perimeter fence. There is one access gate which can accommodate a vehicle. The gate is equipped with a padlock to restrict access. Additionally, there are padlocks on the access hatch of the wet well, each of the control and electrical panels, and the electrical disconnect panel. The fence is not equipped with barb wire.

5.5.2 Site Civil

The lift station site is well graded for drainage during rain events. Inside the perimeter fence, the facility was constructed with a concrete slab. This allows for easier maintenance but increases the amount of runoff during rain events. The grade of the slab and the gaps in the base of the brick wall allow runoff to exit the site efficiently without ponding. The site is located adjacent to a detention pond which serves the neighborhood. The elevation difference between the lift station site and the detention pond is significant and the lift station site is located above the 100-year flood plain.

The concrete slab appears to be in good shape. There does not appear to be any deterioration or significant cracking. The piping and supports also appear to be in good condition, although there are signs of minor rusting on the piping and supports in the wet well. The supports at the top of the rails in the wet well are rusting as well. There is evidence of water in the valve vault which has led to rusting of the piping and valves in the vault.

The gravity line discharging into the wet well appears to be in good condition with no signs of corrosion or improper sealing. There did not appear to be any leakage around the pipe penetration.

5.5.3 Access

The site has sufficient access for maintenance, as the access gate opens to Rembrandt and will easily accommodate a work truck and provide access to the wet well. There is adequate room to back a hoist truck into the site to aid the maintenance personnel in pulling the pumps out of the wet well for service.

The hatch access to the top of the wet well is a Halliday aluminum hatch. The hatch does not have any paint or coatings and is fastened to the top of the wet well with two hinges and is secured with a padlock. The hatch is a 3’-0” x 4’-0” hatch opening and provides adequate access to the pumps, controls, etc.

5.5.4 Pumping and Mechanical

The Reserve Lift Station was constructed in 2007 and was renovated in 2010 to replace the pumps with two Hydromatic model HPGF500M2-4 submersible pumps. The pumps have 5 HP motors with a design point of 98 gpm at 48 feet of TDH. The station has only single phase power; therefore a VFD was utilized to convert power from single phase to three phase. The lift station has a service area that ultimately includes approximately 77 houses. Based on an average of 3 people per house and an assumed value of 80 gallons per day per capita of wastewater and a peaking factor of 4, the pumping capacity needed to provide service for these households is 51 gpm. The lift station has a firm capacity of 98 gpm, which is

more than sufficient for the service area, as it is currently planned. Some grease was observed in the wet well, but staff did not note any problems associated with the grease.

5.5.5 Controls and Instrumentation

The controls for the lift station consist of the mercury floats, motor starters and control switches. The controls are all in good working condition and operate as designed and intended. Visual inspection of the floats indicates they float freely and operate the pumps at the design points.

5.5.6 Condition Assessment

Figure 14: West Fence at Reserve Lift Station

Figure 15: East Fence at Reserve Lift Station

Figure 16: Entrance Gate to Reserve Lift Station

Figure 17: Mounted Control Panels at Reserve Lift Station

Figure 18: Interior of Control Panel at Reserve Lift Station

Figure 19: Float and Grease in Wet Well at Reserve Lift Station

Table 4: Reserve Lift Station Condition Assessment Ratings

Aspect	Rating
Site Security	Fair
Site Civil	Good
Access	Good
Pumping and Mechanical	Good
Controls and Instrumentation	Good

5.5.7 Lift Station Design Data

Table 5: Reserve Lift Station Design Data

Lift Station	The Reserve
Address	900 Rembrandt
Year Constructed	June-07
Recent Improvements	N/A
Capacity one pump, gpm	98
Rated Total Dynamic Head (TDH), ft	48
Type of Pump	Submersible Grinder
Manufacturer	Hydromatic
Model	HPGF500M2-4
Volts/Ph/Hz	230/3/60 (VFD used to convert single phase to 3 phase power)
Motor HP	5
Impeller Dia.	8.875 inches
RPM	1750
Force Main Length, ft	1,403
Force Main Diameter	4-inch

5.5.8 30 TAC Chapter 217 Requirements

Table 6: 30 TAC Chapter 217 Requirements

Does Station have some available capacity	Yes
217.35 Backup Power Requirements	Since less than 100 gpm, station has connections for portable generator, SCADA provided
217.61 Design considerations	
217.61 a Level controls provided,	Yes, Floats
217.61 b Above 100 year flood pain	Yes, Above 100 yr.
217.61 c Separate wet well and valve vaults	Yes
217.61 d Pump cycle time greater than 6 minutes.	Yes
217.61 e Wet well ventilation	Passive 6-inch Vent
217.63 Lift Station Piping (and Valves)	Yes
217.64 Pump Type is acceptable	Submersible Grinder
217.64 Emergency provision for lift station	Yes, Alarms and SCADA provided
217.69 Force Main Design	Velocities less than 3 fps ok for Grinder pumps.
Velocity in Force Main (one pump)	2.3
217.93 Security Fencing	No barb wire

5.6 Conclusions

The following conclusions were developed from the evaluation of the Reserve lift station:

- The station has been well maintained over the past seven years and is in very good condition.
- The pumps have been sized to handle the demands and flows they will experience at peak flows and will be able to keep up with the sewer being delivered by gravity to the station during higher flows periods.
- The electrical and controls are in good operating condition and have not posed any problems for the City staff.
- The site security procedures for locking all the equipment hatches and panels are serving their purpose.
- There is no barb wire atop the fence.
- The grading of the site allows for proper drainage of the site without ponding or flooding.
- There is some grease build up in the wet well.
- The top of the pump rails are exhibiting rust which could be a result of corrosion.

5.7 Recommendations

Based on the observed grease in the wet well, we would recommend the City's maintenance personnel inspect the lift station on regular intervals to observe the grease and determine if it is becoming a problem. If the grease continues to accumulate, a degreaser should be utilized to break up the grease and allow it to be flushed out of the wet well.

Staff should also monitor the top of the pump rails to prevent severe corrosion from occurring to the rails.

5.8 Cost Implication

Cost to be inserted here once consensus on recommendations is reached.

APPENDIX A

Wastewater Lift Station Evaluation Form

Appendix D

Water Capital Improvements Plan

1.0 Executive Summary

This Water Capital Improvements Plan utilizes the model criteria and simulations detailed in the Master Plan to develop proposed projects from the recommended improvements. These improvements are identified for a 20-year planning horizon.

Shorter CIP programs can be developed by limiting the timeframe for the proposed projects as desired. For instance, a current 5-year CIP can be developed by utilizing all projects from the 2015-2020 planning horizon. A summary of all proposed project costs and schedules is displayed on Page 4, while a proposed schedule is located on Page 5. Project descriptions are in order based on priority ranking following this summary.

1.1 Identification and Ranking

Each project is identified by up to two of the six known triggers detailed in the Master Plan (regulatory, capacity, fire flow, condition, City-identified, or operational). As these triggers activated a need, alphabetical project groupings were developed in localized areas to address that need. These were then ranked numerically based on the criticality of the project and service area impacted.

1.1.1 Timeline

This priority was then applied to an overall timeline, in order to meet a 20-year planning horizon. Each project has also been assigned a flexibility rating of low, medium, or high. Projects with higher flexibility can be extended later in the planning horizon, depending on the City's available funding or changing system conditions which may impact the need for the project (such as unexpected delayed City-wide growth delaying the need for capacity improvements).

The project priority dictates the trigger date. Projects with higher priority will trigger sooner than those with lower priority. The trigger date is then utilized to capture anticipated costs for the life of the project, by escalating the total estimated 2015 costs at a rate of 3% to the trigger date for the engineering and construction items.

1.2 Cost Development

Costs estimates were prepared for each individual project, based on industry standards and the 2015 bidding environment. These costs are report level estimates, and should be re-evaluated as each project nears the trigger date. Each project has the following costs associated with the total forecasted project costs:

- Opinion of Probable Construction Cost (OPCC)

  The OPCC is the report level estimate of Contractor's bid price once the project has been designed and is ready for the bid phase to begin. It represents a combination of the estimated total construction costs, engineering, and a 20% estimating contingency.

- Engineering

  Engineering includes estimates of professional services needed to bid each project, including survey, geotechnical, deed research (as needed), preliminary, and final design of all improvements. This cost represents 15% of the OPCC. Construction engineering is not included, since those services are assumed to be provided by City staff.

- Forecasted Project Costs

  Forecasted project costs are the Opinion of Construction Costs (OPCC) with a 3 percent escalation for inflation to the Trigger Date month and year.

Costs for tanks, specialty construction, and demolition were developed from manufacturer and contractor quotations. Water Main and sewer installation costs were developed from a combination of Dallas Water Utilities' Average Costs Manual, recent neighboring city water and sewer project bid tabulations, and 2014 water and sewer bid tabulations provided by the City of Colleyville. The developed cost estimates are presented in Table 1, and individual item descriptions follow.

Table 1 –Water Line Cost Estimates

PVC Water Line Cost Estimates ($ per LF)					
Size	General Improvements	Connections & Valves	Line BOTOC	Open Cut	Total Pipe Install
6	$44	$58	$150	$78	$187
8	$46	$62	$160	$84	$200
10	$48	$64	$180	$92	$213
12	$50	$66	$200	$98	$224
16	$52	$68	$240	$104	$238
20 (DIP)	$54	$70	$260	$120	$258

Assumes 10% BOTOC

Bid Item Descriptions:

General Improvements: All sitework, backfill, erosion control, rehabilitation of existing structures, testing, and contractor overhead costs.

Connections and Valves: Cost for all service connections and connections to existing pipes, including isolation valves and hydrants.

Line BOTOC: (By Other Than Open Cut): The cost for installation of C906 HDPE where boring is required, such as sensitive utility areas or across highways or railroads.

Open Cut: This line item contains the estimated cost of materials and labor to install C900 PVC in an open trench

Total Pipe Install: This is the summary of all previous line items, and utilized on a linear footage basis for the cost estimates, depending on line size. As noted in Table 1, it is assumed that 10% of project linear footage will require line install BOTOC.

1.2.1 Project Descriptions and Legend

Individual descriptions are included for each project, along with associated aerials. Figure 1 displays the legend for each of these aerials. For line projects, summary

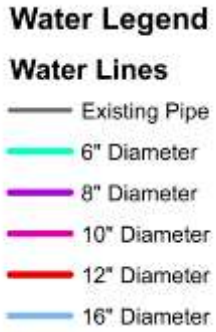


Figure 1 – Aerial Legend

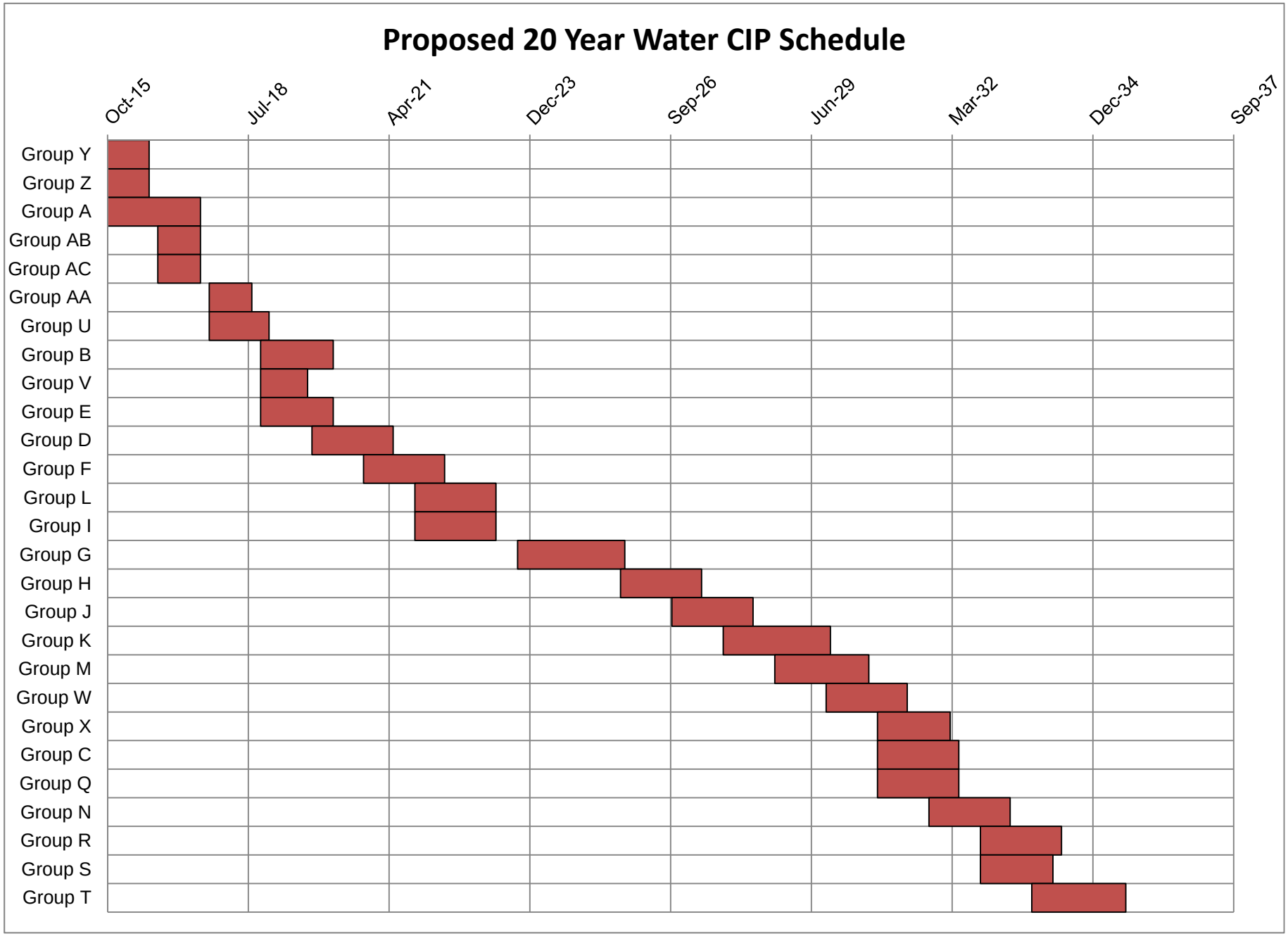
sheets displaying the total linear footage of pipe and the individual pipe segments identified for improvements are included. These line segments correspond to the line segment labels within the updated model provided with the Master Plan.

1.3 Cost and Schedule Summary

A summary of all proposed projects, costs, and schedules follows on the next page, with a proposed project schedule on the following page.

Water CIP Project Summary

Project Identification				Schedule							2015 Costs ($1,000)	Forecasted Cost ($1,000)		
Project	Description	Location	Flexibility	Primary Trigger	Secondary Trigger	Trigger Date	Project Complete	Engineering /Design	Bid/ Construction	Total Project Duration	OPCC	Construction	Engineering	OPCC
1	Group Y	Area 1	Low	City-directed	None	Oct-15	Jul-16		9	9	$1,363	$1,404		$1,404
2	Group Z	Area 2	Low	City-directed	None	Oct-15	Jul-16		9	9	$627	$645		$645
3	Group A	Low Plane	Low	Regulatory	Capacity	Oct-15	Jul-17	9	12	21	$3,844	$3,546	$516	$4,062
4	Group AB	Area 4	Low	City-directed	None	Oct-16	Jul-17		9	9	$372	$394		$394
5	Group AC	Area 5	Low	City-directed	None	Oct-16	Jul-17		9	9	$1,427	$1,514		$1,514
6	Group AA	Area 3	Low	City-directed	None	Oct-17	Jul-18		9	9	$619	$677		$677
7	Group U	High Plane	High	Condition	Operational	Oct-17	Nov-18	4	9	13	$104	$98	$14	$113
8	Group B	High/Low	Low	Capacity	Operational	Oct-18	Feb-20	7	9	16	$312	$305	$46	$351
9	Group V	High/Low	High	Condition	Operational	Oct-18	Aug-19	4	6	10	$217	$213	$31	$244
10	Group E	Low Plane	Medium	Fire Flow	City-directed	Oct-18	Feb-20	7	9	16	$487	$477	$72	$549
11	Group D	Low Plane	Medium	Condition	Fire Flow	Oct-19	Apr-21	9	9	18	$2,599	$2,620	$393	$3,013
12	Group F	Low Plane	Medium	Condition	Operational	Oct-20	Apr-22	9	9	18	$2,328	$2,417	$363	$2,780
13	Group L	Low Plane	Medium	Fire Flow	City-directed	Oct-21	Apr-23	9	9	18	$1,333	$1,426	$214	$1,639
14	Group I	High Plane	Medium	Fire Flow	Operational	Oct-21	Apr-23	9	9	18	$1,374	$1,470	$220	$1,690
15	Group G	Low Plane	Medium	Fire Flow	Condition	Oct-23	Oct-25	12	12	24	$4,025	$4,704	$685	$5,389
16	Group H	Low Plane	Medium	Condition	Operational	Oct-25	Apr-27	9	9	18	$1,636	$1,969	$295	$2,264
17	Group J	Low Plane	Medium	Fire Flow	None	Oct-26	Apr-28	9	9	18	$985	$1,221	$183	$1,405
18	Group K	Low Plane	Medium	Fire Flow	None	Oct-27	Oct-29	12	12	24	$5,414	$7,121	$1,037	$8,158
19	Group M	Low Plane	Medium	Fire Flow	None	Oct-28	Jul-30	9	12	21	$2,638	$3,574	$520	$4,095
20	Group W	Low Plane	Medium	Condition	City-directed	Oct-29	Apr-31	9	9	18	$948	$1,284	$193	$1,477
21	Group X	Low Plane	High	Capacity	City-directed	Oct-30	Feb-32	7	9	16	$403	$562	$84	$646
22	Group C	Low Plane	Medium	Capacity	Operational	Oct-30	Apr-32	9	9	18	$763	$1,065	$160	$1,224
23	Group Q	Low Plane	Medium	City-directed	Operational	Oct-30	Apr-32	9	9	18	$1,138	$1,589	$238	$1,827
24	Group N	Low Plane	Medium	Fire Flow	Condition	Oct-31	Apr-33	9	9	18	$1,553	$2,232	$335	$2,567
25	Group R	Low Plane	High	City-directed	Condition	Oct-32	Apr-34	9	9	18	$711	$1,053	$158	$1,211
26	Group S	High Plane	High	City-directed	Operational	Oct-32	Feb-34	7	9	16	$104	$153	$23	$176
27	Group T	Low Plane	High	City-directed	None	Oct-33	Jul-35	9	12	21	$2,540	$3,989	$581	$4,569
										Total 2015 OPCC:	**$39,862,934**	**Total Forecasted OPCC:**		**$54,081,992**


Proposed 20 Year Water CIP Schedule
Oct-15
Jul-18
Apr-21
Dec-23
Sep-26
Jun-29
Mar-32
Dec-34
Sep-37
Group Y
Group Z
Group A
Group AB
Group AC
Group AA
Group U
Group B
Group V
Group E
Group D
Group F
Group L
Group I
Group G
Group H
Group J
Group K
Group M
Group W
Group X
Group C
Group Q
Group N
Group R
Group S
Group T

A proposed spending schedule follows. This spending schedule and associated project trigger dates should be updated as City funding is further refined.

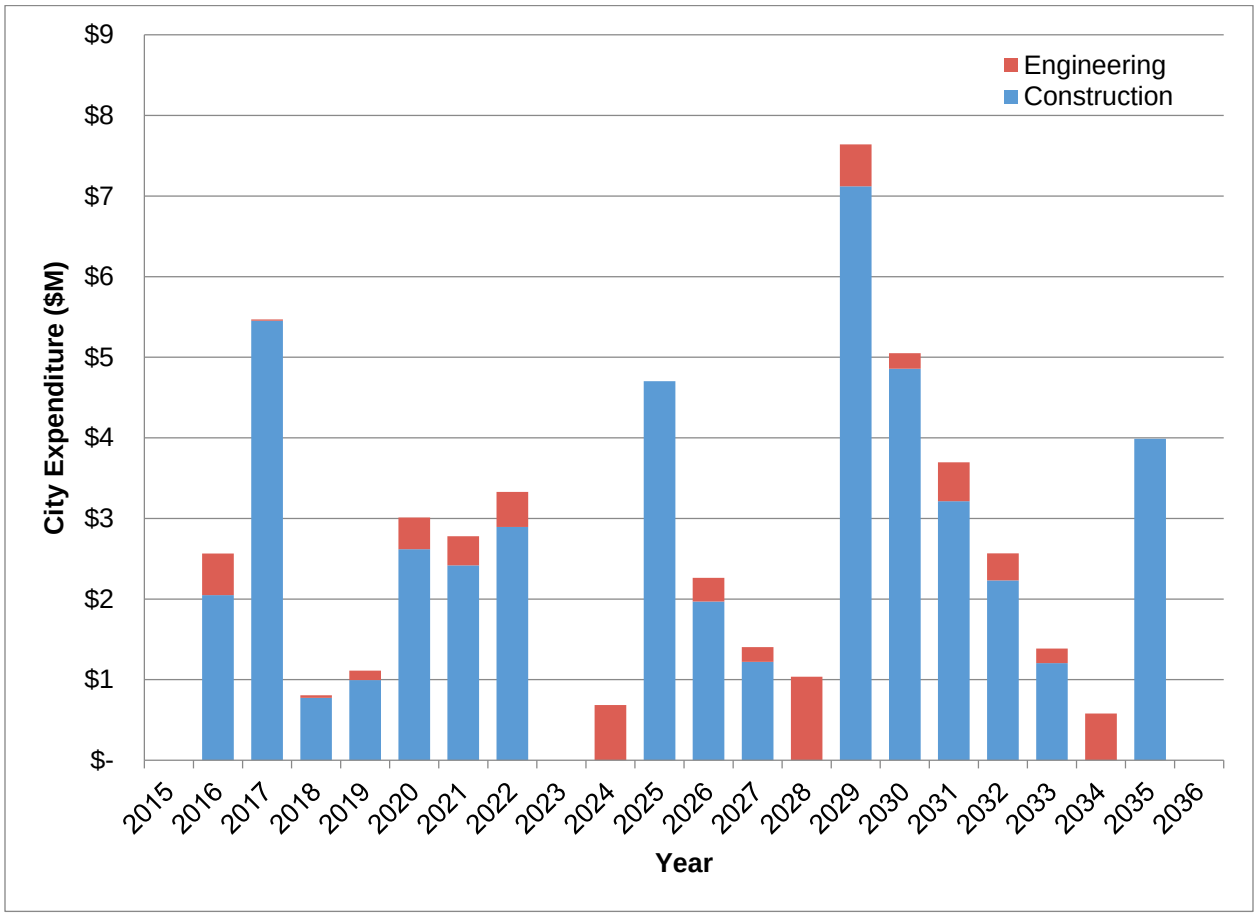


Figure 2 - Proposed Spending Schedule for Water CIP

Individual project descriptions are detailed in the following pages.

Project 1: Water Group Y Capital Improvements

The project includes installation of new 6", 8", and 12" PVC as well as rehabilitation of existing water infrastructure such as valves and gutter systems within Area 1. This project involves the installation of approximately 588 LF, 7,119 LF, and 40 LF or 6", 8" and 12" PVC pipeline respectively.

Justification

This project was identified as a need in the 2013 Water and Sewer Utility Improvements CIP.

Unintended Consequences

None identified.

Special Considerations

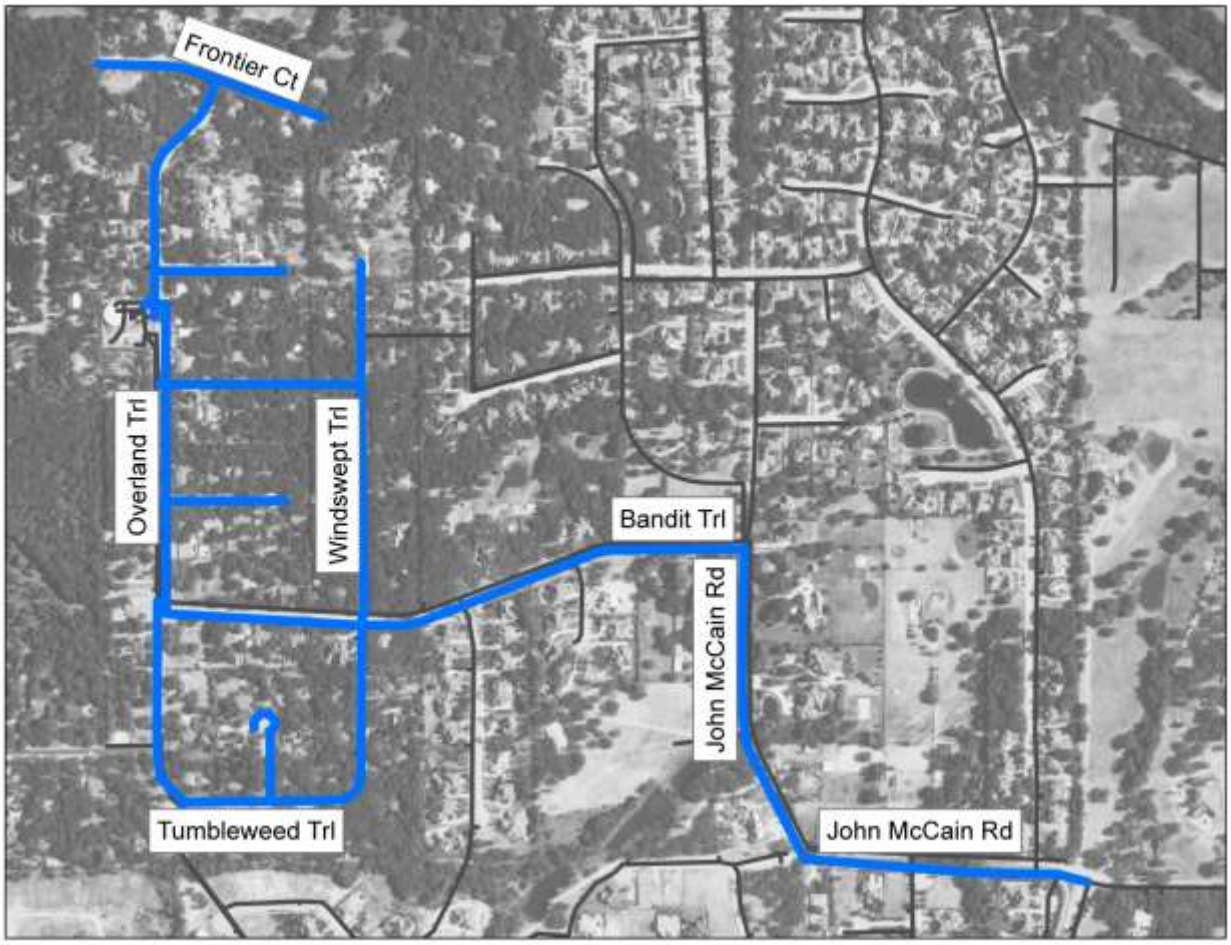
None identified.

Potential Alternatives

None identified.

Project Identification		
Number:	1	
Location:	Area 1	
Flexibility:	Low	
Schedule		
Primary Trigger:	City-Identified	
Secondary Trigger:	None	
Trigger Date:	Oct 2015	
Project Complete:	July 2016	
Project Implementation (Months)		
Engineering/Design:	-	
Bid/Construction:	9	
Total Project Duration:	**9**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$1.36	$1.40
Engineering	$ -	$ -
OPCC	**$1.36**	**$1.40**

Project 1 Aerials



Project 2: Water Group Z Capital Improvements

Project 2 specifies the installation of approximately 91 LF and 4,373 LF of 6" and 8" PVC respectively. In addition, the project involves the rehabilitation of valves and other water infrastructure along Area 2 as specified by the City.

Justification

This project was identified as a need in the 2013 Water and Sewer Utility Improvements.

Unintended Consequences

None identified.

Special Considerations

None identified.

Potential Alternatives

None identified.

Project Identification		
Number:	2	
Location:	Area 2	
Flexibility:	Low	
Schedule		
Primary Trigger:	City-Identified	
Secondary Trigger:	None	
Trigger Date:	Oct 2015	
Project Complete:	July 2016	
Project Implementation (Months)		
Engineering/Design:	-	
Bid/Construction:	9	
Total Project Duration:	**9**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.63	$0.65
Engineering	$ -	$ -
OPCC	**$0.63**	**$0.65**

Project 2 Aerial

Project 3: Water Group A Capital Improvements

The project includes installation of a new 1 MG elevated storage tank, 271 LF of new 16" PVC distribution pipe, and 518 LF of new 12" PVC.

Project Identification		
Number:	3	
Location:	Low Plane	
Flexibility:	Low	
Schedule		
Primary Trigger:	Regulatory	
Secondary Trigger:	Capacity	
Trigger Date:	Oct 2015	
Project Complete:	July 2017	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	12	
Total Project Duration:	**21**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$3.34	$3.55
Engineering	$0.50	$0.52
OPCC	**$3.84**	**$4.06**

Justification

A new elevated storage tank in this area addresses elevated storage concerns plane-wide across the low pressure plane. The tank also addresses local pressure and fire flow concerns in the southwest portion of the system. The 16" and 12" upgraded piping provides additional fire flow throughout this area. Furthermore, the additional elevated storage provides extra security to the system in the event of a power outage or supply issue. Contingency money may be utilized for property acquisition if the tank is not able to be located on existing City property.

Unintended Consequences

Water age throughout the system will increase as a result of the installation of this tank. The tank has become an unsafe structure for personnel performing operational and maintenance activities on and around the tank. The cost to maintain the tank will continue to escalate due to the age and condition of the structure. Though unlikely, an increase in pressure in that area may also cause old or weak pipes near that location to rupture.

Special Considerations

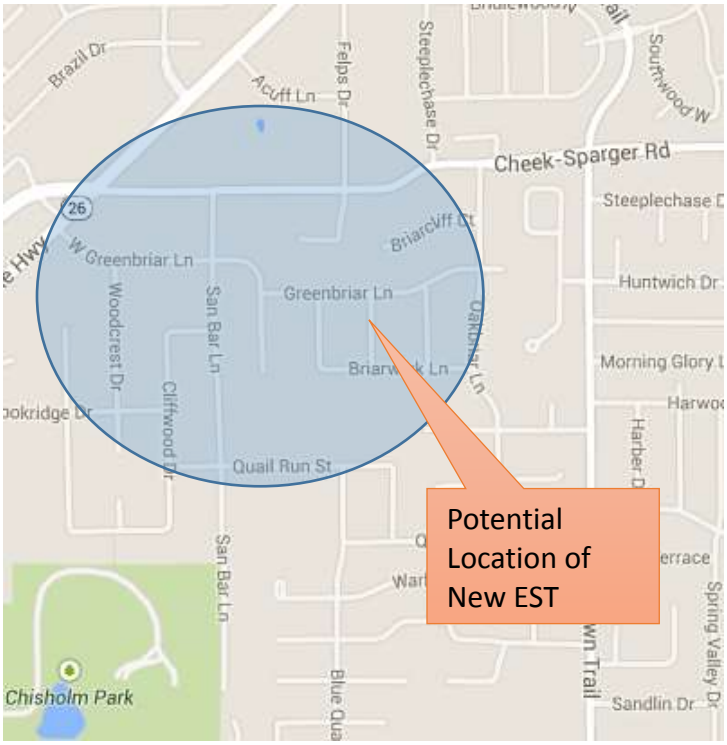
Consideration must be given to the final siting of the tank, as this will have secondary impacts throughout the model and a large impact on surrounding citizens. Public input is recommended to ensure that the aesthetics of the new tank meet the City's expectations.

Potential Alternatives

An alternatives evaluation between a ground storage tank with booster station or the proposed elevated storage tank was conducted in the Master Plan. While the EST remains the recommended improvement, the GST with booster station is a potential alternative. Additionally, relocating the proposed location is possible, but may require additional piping infrastructure to alleviate local pressure and flow concerns.

Group A					
Label	**Diameter (in.)**	**Length (ft.)**			**Total Cost**
P-2086	16	280			$ 65,000
P-1897	12	280			$ 66,000
P-1898	12	250			$ 58,000
Label	**Diameter (ft.)**	**Base (ft.)**	**LWL (ft.)**	**HWL (ft.)**	
1 MG EST	75	648	760	790	$ 2,200,000
				Subtotal	$ 2,388,000
				Contingency (20%)	$ 478,000
				Appurtenances (electrical, SCADA, etc.)	$ 478,000
				Engineering	$ 502,000
				OPCC	**$ 3,844,000**

Project 3 Aerial



Project 4: Water Group AB Capital Improvements

Project 4 encompasses installation of 48 LF and 2,679 LF of 6" and 8" PVC pipeline throughout Area 3 as well as various rehabilitation projects.

Justification

This project was identified as a need in the 2013 Water and Sewer Utility Improvements.

Unintended Consequences

None identified.

Special Considerations

None identified.

Potential Alternatives

None identified.

Project Identification		
Number:	4	
Location:	Area 4	
Flexibility:	Low	
Schedule		
Primary Trigger:	City-Identified	
Secondary Trigger:	None	
Trigger Date:	Oct 2016	
Project Complete:	July 2017	
Project Implementation (Months)		
Engineering/Design:	-	
Bid/Construction:	9	
Total Project Duration:	**9**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.37	$0.39
Engineering	$ -	$ -
OPCC	**$0.37**	**$0.39**

Project 4 Aerials

Project 5: Water Group AC Capital Improvements

Project 5 involves water line renewals of Area 5 including the installation of 179 LF and 10,027 LF of 6" and 8" PVC respectively.

Justification
This project was identified as a need in the 2013 Water and Sewer Utility Improvements.

Unintended Consequences
None identified.

Special Considerations
None identified.

Potential Alternatives
None identified.

Project Identification		
Number:	5	
Location:	Area 5	
Flexibility:	Low	
Schedule		
Primary Trigger:	City-Identified	
Secondary Trigger:	None	
Trigger Date:	Oct 2016	
Project Complete:	July 2017	
Project Implementation (Months)		
Engineering/Design:	-	
Bid/Construction:	9	
Total Project Duration:	**9**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$1.43	$1.51
Engineering	$ -	$ -
OPCC	**$1.43**	**$1.51**

Project 5 Aerials

Project 6: Water Group AA Capital Improvements

Project 3 encompasses installation of 171 LF and 4,030 LF of 6" and 8" PVC pipeline throughout Area 3.

Justification

This project was identified as a need in the 2013 Water and Sewer Utility Improvements.

Unintended Consequences

None identified.

Special Considerations

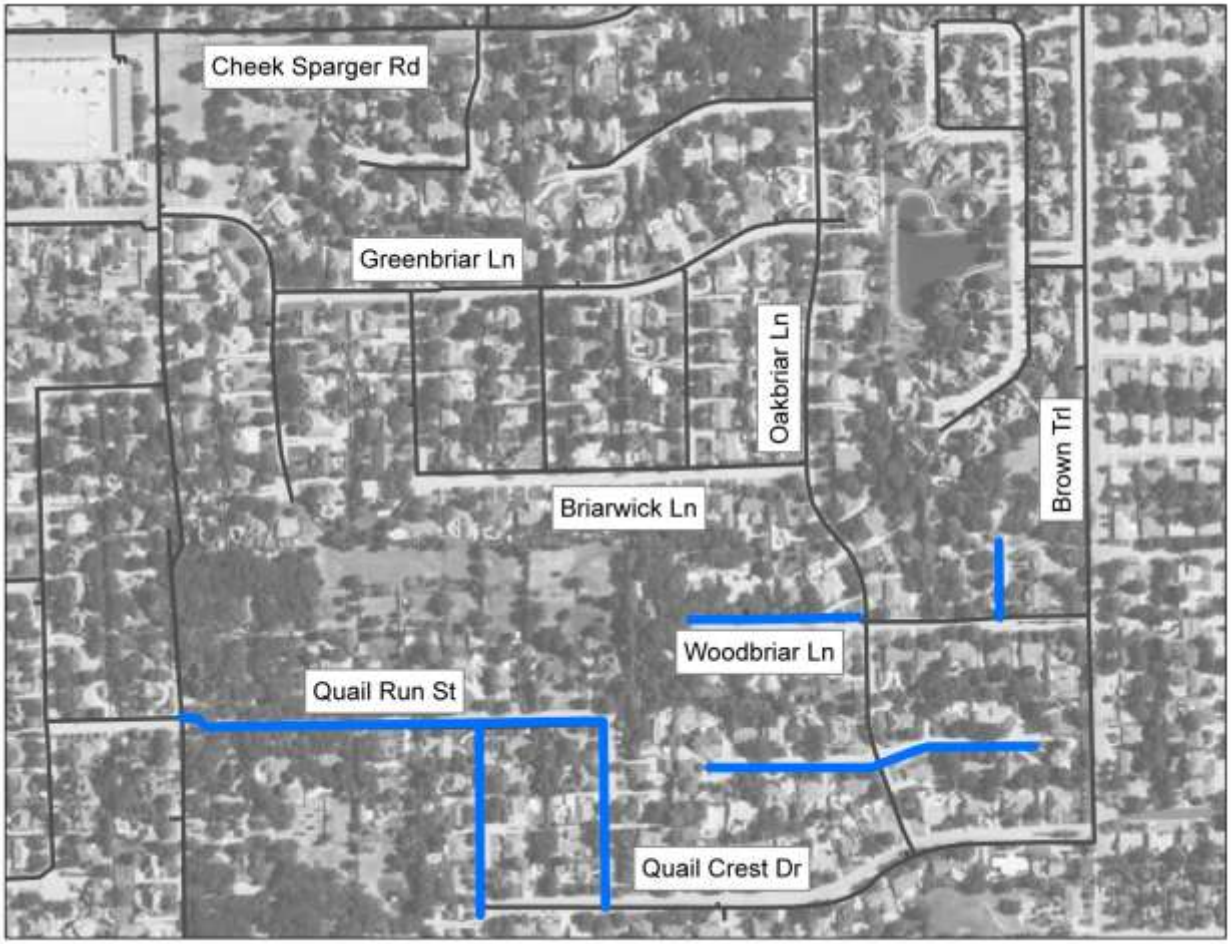
None identified.

Potential Alternatives

None identified.

Project Identification		
Number:	6	
Location:	Area 3	
Flexibility:	Low	
Schedule		
Primary Trigger:	City-Identified	
Secondary Trigger:	None	
Trigger Date:	Oct 2017	
Project Complete:	July 2018	
Project Implementation (Months)		
Engineering/Design:	-	
Bid/Construction:	9	
Total Project Duration:	**9**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.62	$0.68
Engineering	$ -	$ -
OPCC	**$0.62**	**$0.68**

Project 6 Aerials



Project 7: Group U Overland Trail Decommissioning and Demolition

Project Description

The project includes decommissioning of the existing Pump Building, Chlorine Building, Ammonia Building, and salvage of major mechanical equipment such as pumps and valves for further use by the City. The project also includes demolition of the ground storage tank and yard piping that connects directly to the City's distribution system.

Project Identification		
Number:	7	
Location:	High Plane	
Flexibility:	High	
Schedule		
Primary Trigger:	Condition	
Secondary Trigger:	Operational	
Trigger Date:	Oct 2017	
Project Complete:	Nov 2018	
Project Implementation (Months)		
Engineering/Design:	4	
Bid/Construction:	9	
Total Project Duration:	**13**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.09	$0.10
Engineering	$0.04	$0.01
OPCC	**$0.10**	**$0.11**

Justification

A hydraulic evaluation of the water distribution system determined that the facility is no longer needed for the City's existing or future infrastructure.
The existing 1.2 million gallon tank was originally installed in the City after being relocated from an air base near San Angelo and has become dilapidated over the years. Numerous leaks have become evident due to the corrosion on the interior of the tank. Lack of adequate coating system on the interior and exterior have allowed the tank to rust and corrode leading to water quality concerns in addition to the water loss from leaks that have developed.

The bolted steel tank is experiencing structural failure. The steel panels near the top of the tank have buckled and flex inward/outward as the water level in the tank rises and lowers, exerting or releasing pressure on the panels. The ladder on the southeast side of the tank has bowed outward from the tank creating a precarious and potentially dangerous situation for the operations staff.

Yard piping and valves on the tank have developed leaks that require repairs for the tank to operate efficiently.

Unintended Consequences

The tank has become an unsafe structure for personnel performing operational and maintenance activities on and around the tank. The cost to maintain the tank will continue to escalate due to the age and condition of the structure.

Special Considerations

None identified.

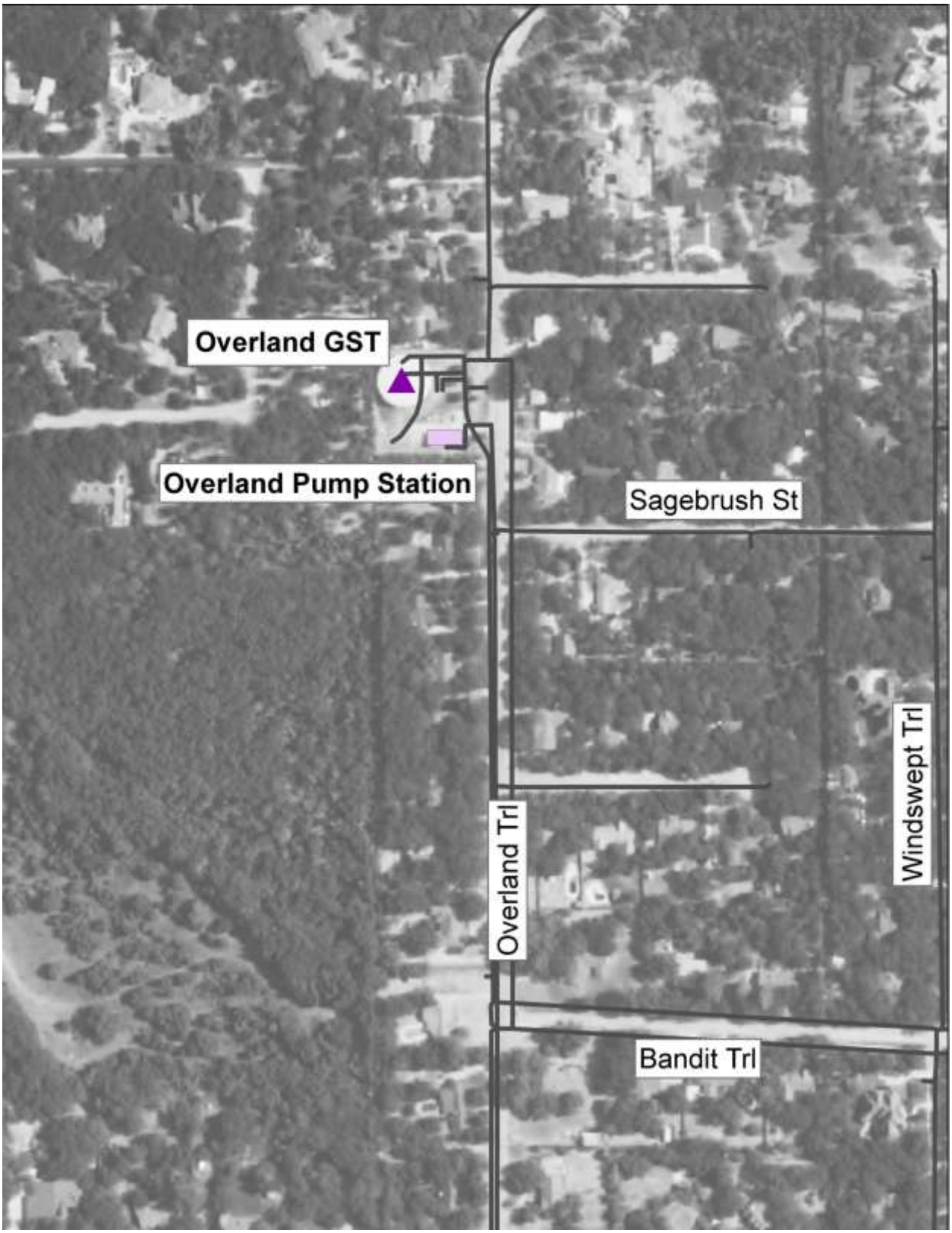
Potential Alternatives

The City may choose to sell the land the tank sits on with the tank still intact, or leave the tank as-is. If this occurs, it is recommended that all facilities be isolated from the water supply system to avoid any security concerns.

Group U	
Description	**Total Cost**
Overland Trail Decommission	$ 75,000
Subtotal	$ 75,000
Contingency (20%)	$ 15,000
Engineering	$ 14,000
2015 OPCC	**$ 104,000**

Project 7 Photographs

15
10


Overland GST
Overland Pump Station
Sagebrush St
Windswept Trl
Overland Trl
Bandit Trl

Project 8: Water Group B Capital Improvements

Project Identification	
Number:	8
Location:	High/Low
Flexibility:	Low
Schedule	
Primary Trigger:	Capacity
Secondary Trigger:	Operational
Trigger Date:	Oct 2018
Project Complete:	Feb 2020
Project Implementation (Months)	
Engineering/Design:	7
Bid/Construction:	9
Total Project Duration:	**16**

	2015 Costs ($ Millions)	Forecasted Costs ($ Millions)
Construction	$0.27	$0.31
Engineering	$0.04	$0.05
OPCC	**$0.31**	**$0.36**

Project Description

The project includes the addition of two pressure release and flow control valves to regulate flow from the northeast section of the high pressure plane into the northwest portion of the low pressure plane. The project also includes associated vaults and connecting 8" and 12" piping.

During normal and low flow demands, no flow is expected to pass through the valves. However, during high demand or emergency situations, flow will be allowed through the valves to maintain acceptable pressures.

Justification

Currently during periods of maximum demand, the City staff manually open valves to allow flow from the high plane to back feed the low plane. The addition of these valves allows the pressure and flow to regulate automatically, which provides pressure and fire flow support to this portion of the low plane.

Unintended Consequences

As detailed in the Master Plan, water age in the high pressure plane is approximately 2 days older than low pressure plane water. The water age in the low plane will be increased with blending from the high plane, but the high plane age would also be expected to decrease due to increased turnover. Additionally, it is only anticipated that these valves would operate in peak to near-peak conditions, when water age throughout the system is relatively low. Therefore, it is not believed that these connections will cause significant effects to water age or chlorine residual levels.

Special Considerations

Operational adjustments will need to be made once the valves are installed, to ensure the proper pressure settings are incorporated.

Potential Alternatives

Installation of a ground storage tank with booster station, elevated storage tank, or increase of transmission mains are all potential alternatives. However, these alternatives are all estimated to be greater cost and coordination effort.

Group B			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-1500	12	130	$ 28,000
P-1502	12	120	$ 27,000
P-1503	12	20	$ 5,000
P-1504	12	20	$ 4,000
P-1639	8	20	$ 3,000
P-1640	8	10	$ 2,000
P-1641	8	10	$ 1,000
PRV-2	8	0	$ 60,000
PRV-1	12	0	$ 100,000
		Subtotal	$ 227,000
		Contingency (20%)	$ 46,000
		Engineering	$ 41,000
		2015 OPCC	**$ 313,000**

Project 8 Aerials

Project 9: Group V Facilities Improvements

Project Identification		
Number:	9	
Location:	High/Low	
Flexibility:	High	
Schedule		
Primary Trigger:	Condition	
Secondary Trigger:	Operation	
Trigger Date:	Oct 2018	
Project Complete:	Aug 2019	
Project Implementation (Months)		
Engineering/Design:	4	
Bid/Construction:	6	
Total Project Duration:	**10**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.19	$0.21
Engineering	$0.03	$0.03
OPCC	**$0.22**	**$0.24**

Project Description

The project includes security, coating and civil/site improvements at multiple water facilities.

L.D. Lockett Pump Station:

- Signs should be posted along the southern fence line informing the public that it is a violation of federal law to enter the site without permission. Additionally, the chain link fabric along the southern portion of the fence should be straightened and remounted.
- The northern boundary of the site should be filled and re-graded to address erosion issues and prevent further erosion.

Bransford Elevated Tank:

- The chain link fabric and barbed wire along the perimeter fence line should be remounted and perimeter vegetation should be removed and/or trimmed.
- Piping and valves should be painted/repainted.
- The tank walkway should be replaced and roof and floor plates repainted.

McPherson Elevated Tank:

- Repairs need to be made where the ground has been eroded due to the tank's overflow piping runoff. The concrete drainage channel should be extended or riprap placed at the end of the existing channel to help remedy the erosion.
- Piping and interior components should be repainted.

Justification

The water facilities identified herein are key in providing required storage capacity and adequate pressure for the City's water distribution system. Overgrown landscaping and fencing in disrepair pose a security concern by limiting site visibility and providing climbing access where the perimeter of each site can be breached. At the L.D. Lockett site, non-authorized individuals are currently breaching the fence adjacent to the ball fields to retrieve balls. Additionally, non-coated metal surfaces, and surfaces where the coating is peeling, at these sites will begin to corrode and will experience an overall decrease in useful life.

Unintended Consequences

None identified.

Special Considerations

These improvements can be completed through the utilization of in-house personnel and are likely not required to be bid to a consultant or contractor, unless additional contract labor is needed for painting.

Potential Alternatives

None identified.

Group V	
Description	**Total Cost**
LD Lockett site improvements	$ 10,000
Bransford EST site and painting improvements	$ 135,000
McPherson EST site and painting improvements	$ 13,000
Subtotal	$ 158,000
Contingency (20%)	$ 32,000
Engineering	$ 29,000
2015 OPCC	**$ 218,000**

Project 9 Photographs

L.D. Lockett Pump Station Site

Bransford Elevated Tank Site

McPherson Elevated Tank Site

McPherson EST
W McDonwell School Rd
Bettinger Dr
Bransford EST
Hunter Trl
W LD Lockett Rd
Brandon Rd
Oak Crest Hills Dr
Oak Valley Dr

Project 10: Water Group E Capital Improvements

Project Description

The project includes the addition of an 8" PVC line to connect Apple Valley Drive to Bills Lane, and replacement of a 2-inch pipeline along Black Drive with an 8-inch pipe. This project addresses looping issues of the pipeline in that area and bottleneck concerns of the existing 2-inch pipe.

Justification

Individual dead-end pipelines exist on Apple Valley Drive and Bills Lane, and existing 2-inch bottleneck on Black Drive. These connector pipes will address those dead ends, provide increased fire flow to the residences in that area, and remove a known bottleneck.

Unintended Consequences

No known unintended consequences exist with the construction of this line.

Special Considerations

New right of way will be necessary through the Apple Valley Drive area, as well as reconstruction of private property. Therefore, the line alignment may need shifted, depending on the landowner cooperation.

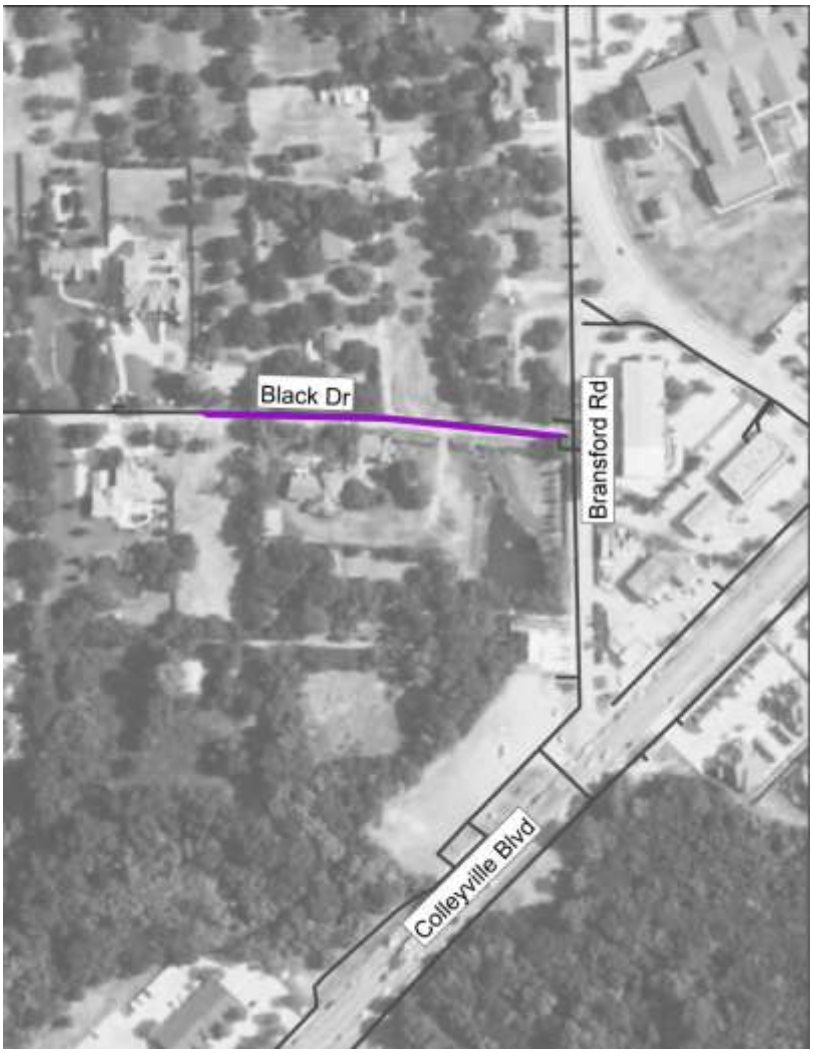
Potential Alternatives

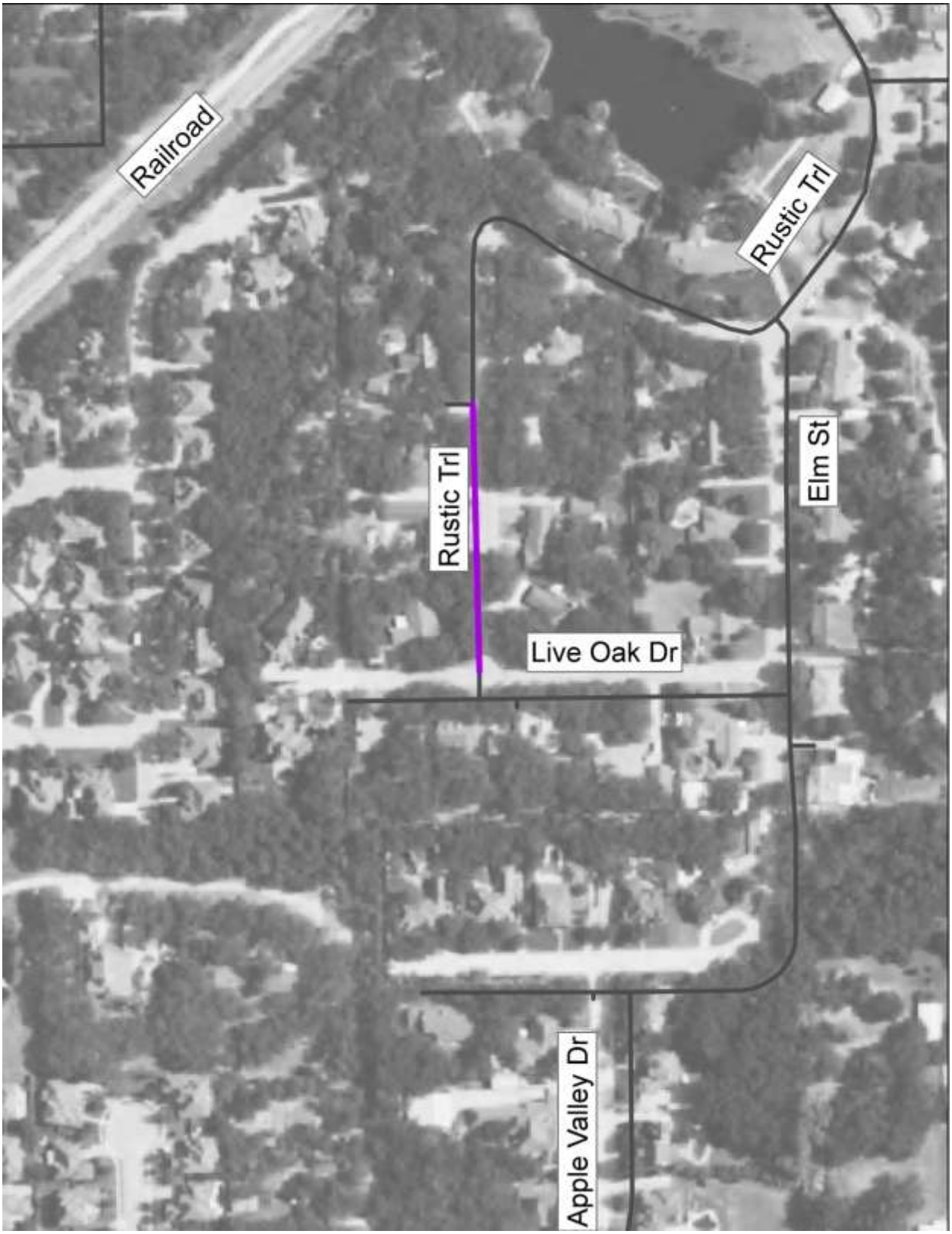
An alternative looping scenario would require installation of 8" line to the 6" line off Glade Road. However, that line would be insufficient to provide adequate fire flow to the area. Therefore, fire flow improvements (such as upsizing the 6" to the private residence) would be necessary.

Project Identification		
Number:	10	
Location:	Low Plane	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Fire Flow	
Secondary Trigger:	City-Identified	
Trigger Date:	Oct 2018	
Project Complete:	Feb 2020	
Project Implementation (Months)		
Engineering/Design:	7	
Bid/Construction:	9	
Total Project Duration:	**16**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.42	$0.48
Engineering	$0.05	$0.07
OPCC	**$0.49**	**$0.55**

Group E			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-1893	8	750	$ 148,000
P-2139	8	630	$ 126,000
P-2075	8	400	$ 80,000
		Subtotal	$ 354,000
		Contingency (20%)	$ 71,000
		Engineering	$ 64,000
		2015 OPCC	**$ 488,000**

Project 10 Aerials




Railroad
Rustic Trl
Rustic Trl
Live Oak Dr
Elm St
Apple Valley Dr

Project 11: Water Group D Capital Improvements

Project Description

The project includes installation of 8,535' of new 8, 10, and 12-inch PVC lines in the northern part of the low pressure plane, near Bransford Tank. This will replace existing glue joint and asbestos cement pipes to address condition concerns, and increase line sizing to provide increased fire flow to this area.

The pipeline segments are primarily small interconnecting piping. One neighborhood loop improvement is included along Polo Trail.

Project Identification		
Number:	11	
Location:	Low Plane	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Condition	
Secondary Trigger:	Fire Flow	
Trigger Date:	Oct 2019	
Project Complete:	April 2021	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	9	
Total Project Duration:	**18**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$2.26	$2.62
Engineering	$0.34	$0.39
OPCC	**$2.60**	**$3.01**

Justification

The existing 6" glue joint and asbestos cement pipe in the area has caused the City concern about the integrity of the pipe. In addition, interconnecting 12-inch and 8-inch piping causes the 6-inch piping in the area to bottleneck during high demand and fire flows.

Replacement of these lines will result in decreased maintenance, adequate fire flows, and increased pipeline capacity.

Unintended Consequences

There are no known unintended consequences caused by the replacement of these lines.

Special Considerations

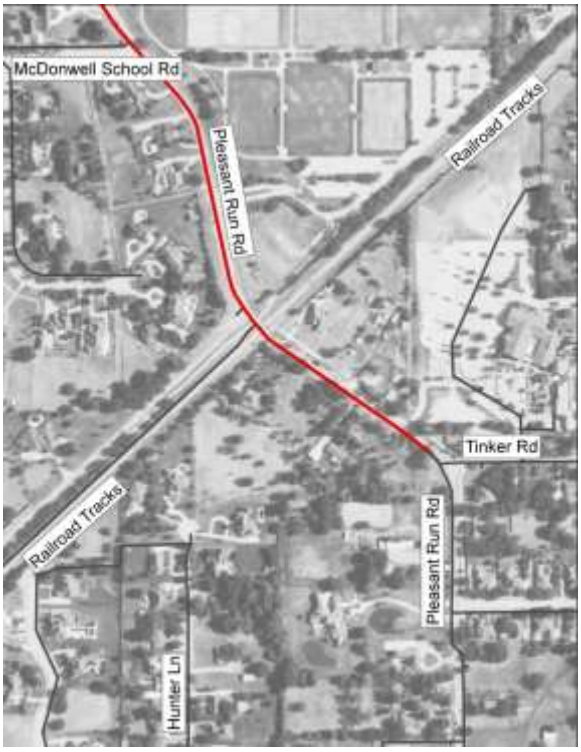
Special considerations for connections to existing pipelines and service connections must be given, as these pipeline segments are in dense residential areas. Any construction should be coordinated with future roadway projects.

Potential Alternatives

None identified.

Group D			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-1508	10	50	$ 10,000
P-1509	10	390	$ 82,000
P-1510	10	280	$ 59,000
P-1511	10	1090	$ 232,000
P-1512	10	40	$ 8,000
P-1513	10	450	$ 95,000
P-1514	10	270	$ 57,000
P-1519	12	800	$ 178,000
P-1520	12	90	$ 19,000
P-1522	12	910	$ 204,000
P-1523	12	220	$ 48,000
P-1524	12	170	$ 39,000
P-1525	12	1240	$ 277,000
P-1753	10	30	$ 5,000
P-1759	12	1200	$ 268,000
P-1760	12	250	$ 55,000
P-1761	8	60	$ 11,000
P-1888	12	140	$ 30,000
P-1889	12	980	$ 219,000
		Subtotal	$1,884,000
		Contingency (20%)	$ 377,000
		Engineering	$ 339,000
		2015 OPCC	**$2,599,000**

Project 11 Aerials



Project 12: Water Group F Capital Improvements

Project Description

The project includes significant water line additions and replacements along Highway 26 in the high elevation area, near the proposed location of the new elevated storage tank.

Justification

The City has implemented a policy that all future transmission lines along Colleyville Boulevard be 10-inch or larger. This project will upsize several 4-inch pipelines along the Boulevard to ensure adequate fire flow and future capacity. A dead end also exists in this area along Colleyville Boulevard to a commercial section near a Braum's restaurant. This project will loop that dead end, along with pipe replacement and looping in residential and commercial areas in that region to improve water pressures, quality, and fire flow.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

If the EST is not installed near the proposed location, modifications will likely be required to this Water Group to increase the conveyance capacity to handle peak flow rates.

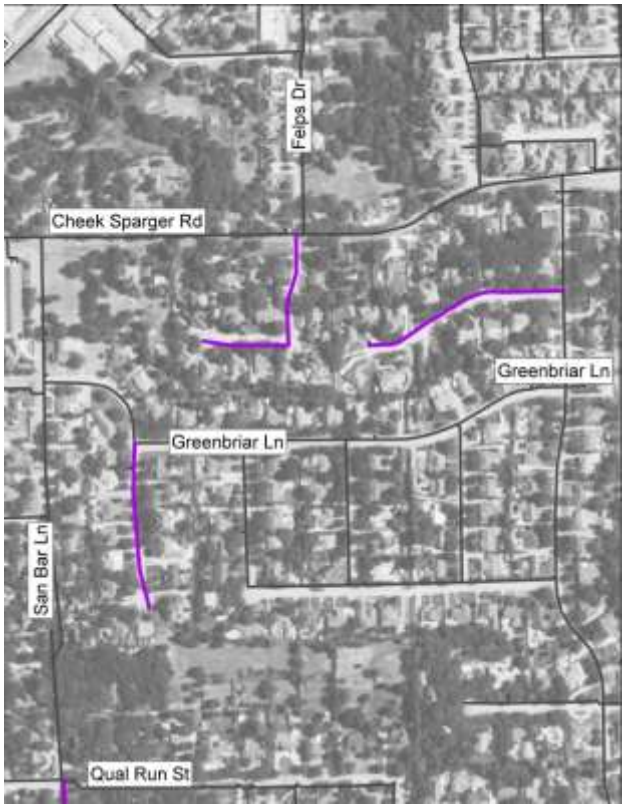
Potential Alternatives

No known alternatives exist.

Project Identification		
Number:	12	
Location:	Low Plane	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Condition	
Secondary Trigger:	Operational	
Trigger Date:	Oct 2020	
Project Complete:	April 2022	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	9	
Total Project Duration:	**18**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$2.03	$2.42
Engineering	$0.30	$0.36
OPCC	**$2.33**	**$2.78**

Group F			
Label	Diameter (in.)	Length (ft.)	Total Cost
P-1617	8	650	$ 130,000
P-1618	8	410	$ 82,000
P-1619	8	380	$ 75,000
P-1620	8	380	$ 76,000
P-1621	8	250	$ 50,000
P-1622	8	380	$ 76,000
P-1623	8	650	$ 129,000
P-1624	8	600	$ 120,000
P-1625	8	230	$ 46,000
P-1627	8	50	$ 9,000
P-1628	8	760	$ 151,000
P-1629	8	360	$ 71,000
P-1630	8	80	$ 15,000
P-1631	8	340	$ 67,000
P-1826	8	310	$ 61,000
P-1827	8	420	$ 83,000
P-1829	8	100	$ 20,000
P-1830	8	80	$ 15,000
P-1831	8	470	$ 93,000
P-1832	8	440	$ 87,000
P-1833	8	300	$ 59,000
P-1834	8	460	$ 91,000
P-1835	8	330	$ 65,000
P-2099	10	130	$ 28,000
		Subtotal	$1,688,000
		Contingency (20%)	$ 338,000
		Engineering	$ 304,000
		2015 OPCC	**$2,329,000**

Project 12 Aerials



Project 13: Water Group L Capital Improvements

Project Description

This project incorporates new 8, 10, and 12-inch along Glade Road from Old Oak Lane to an existing 12-inch east of Martin Parkway. This connection will provide increased fire flows throughout the area and improve looping.
Additional improvements to the institutional/retail area adjacent to SH-121 are also included to provide minimum 3,000 gpm fire flows.

Project Identification		
Number:	13	
Location:	Low Plane	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Fire Flow	
Secondary Trigger:	City-Identified	
Trigger Date:	Oct 2021	
Project Complete:	April 2023	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	9	
Total Project Duration:	**18**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$1.16	$1.43
Engineering	$0.17	$0.21
OPCC	**$1.33**	**$1.64**

Justification

The recommended improvements area has a mix of residential and large commercial development. Increasing the Glade Road line to a 12" will improve system hydraulics and provide required fire flow at nearby connections. The improvements along SH-121 will ensure adequate fire flow is readily available to that area's commercial developments.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

Individual service lines within the commercial development are recommended for assessment to ensure adequate fire flow coverage off the main lines.

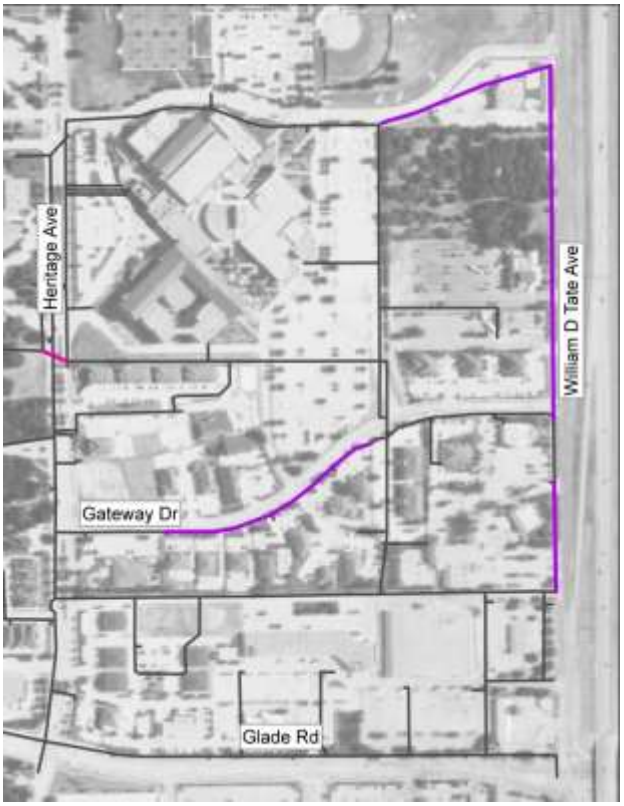
Special consideration should also be given to existing landscaping during detailed design, to limit the impacts of construction.

Potential Alternatives

Alternative pipeline paths and looping scenarios may exist. However, the proposed alignments are believed to present the most feasible alignments.

Group L			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-1796	12	330	$ 72,000
P-1797	12	30	$ 7,000
P-1798	12	440	$ 98,000
P-1799	12	180	$ 39,000
P-1800	12	100	$ 23,000
P-1801	12	470	$ 106,000
P-1802	12	20	$ 4,000
P-1813	10	40	$ 9,000
P-1814	10	60	$ 12,000
P-1817	8	380	$ 76,000
P-1818	8	1450	$ 289,000
P-1819	8	380	$ 75,000
P-1820	8	270	$ 54,000
P-1821	8	100	$ 20,000
P-1822	8	60	$ 12,000
P-1823	8	390	$ 77,000
		Subtotal	$ 966,000
		Contingency (20%)	$ 194,000
		Engineering	$ 174,000
		2015 OPCC	**$1,334,000**

Project 13 Aerials



Project 14: Water Group I Capital Improvements

Project Identification		
Number:	14	
Location:	High Plane	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Fire Flow	
Secondary Trigger:	Operational	
Trigger Date:	Oct 2021	
Project Complete:	April 2023	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	9	
Total Project Duration:	**18**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$1.20	$1.47
Engineering	$0.18	$0.22
OPCC	**$1.38**	**$1.69**

Project Description

The project includes the addition of multiple 8-inch pipes to address fire flow requirements throughout mixed use areas within the high pressure plane. Addition of a line along Precinct Line Road is also intended to loop a dead end line near LD Locket.

Justification

The proposed improvements will greatly enhance fire flow availability in the residential and retail portions of the high pressure plane. The addition of looped lines will also improve water quality and pressures within that area.

Unintended Consequences

The increase in line size throughout these service areas may slightly impact water age. However, the increase in age is not anticipated to be significant.

Special Considerations

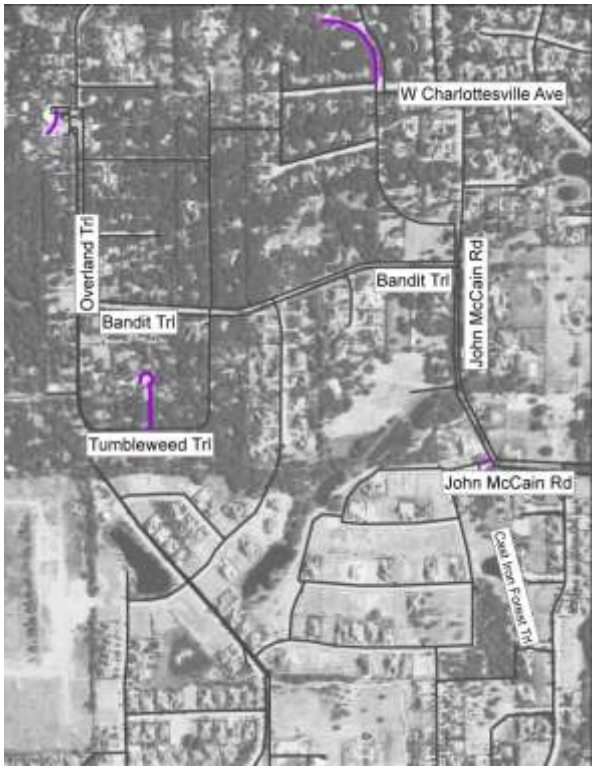
As the area is primarily residential and served by single lines, special consideration must be given to keep users in service during construction.

Potential Alternatives

Minor alternative realignments may exist. However, the proposed routes indicate the most ideal layout to address deficient fire flows and looping.

Group I			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-1643	8	120	$ 24,000
P-1651	8	72	$ 15,000
P-1652	8	372	$ 75,000
P-1653	8	513	$ 103,000
P-1654	8	505	$ 101,000
P-1655	8	502	$ 101,000
P-1656	8	444	$ 89,000
P-1661	8	56	$ 11,000
P-1690	8	537	$ 108,000
P-1692	8	47	$ 10,000
P-1693	8	353	$ 71,000
P-1694	8	319	$ 64,000
P-1695	8	69	$ 14,000
P-1698	8	342	$ 69,000
P-1702	8	149	$ 30,000
P-1703	8	589	$ 118,000
P-1707	8	7	$ 2,000
P-2055	8	878	$ 176,000
		Subtotal	$ 996,000
		Contingency (20%)	$ 200,000
		Engineering	$ 180,000
		2015 OPCC	**$1,375,000**

Project 14 Aerials



Project 15: Water Group G Capital Improvements

Project Description

The project includes replacement of glue joint pipe to address condition concerns, as well as significant fire flow upgrades. In total, 14,613 LF of 8-inch PVC pipeline is proposed to service residential customers with adequate fire flow in that area.

Justification

The area cannot currently provide the required 1,500 gpm of fire flow, and the existing pipe is glue joint, which the City has identified as an ongoing maintenance issue. This combination of factors justifies the improvements needed to replace the glue joint pipe and provide adequate fire flow.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

Special considerations for connections to existing pipelines and service connections must be given, as these pipeline segments are in dense residential areas.

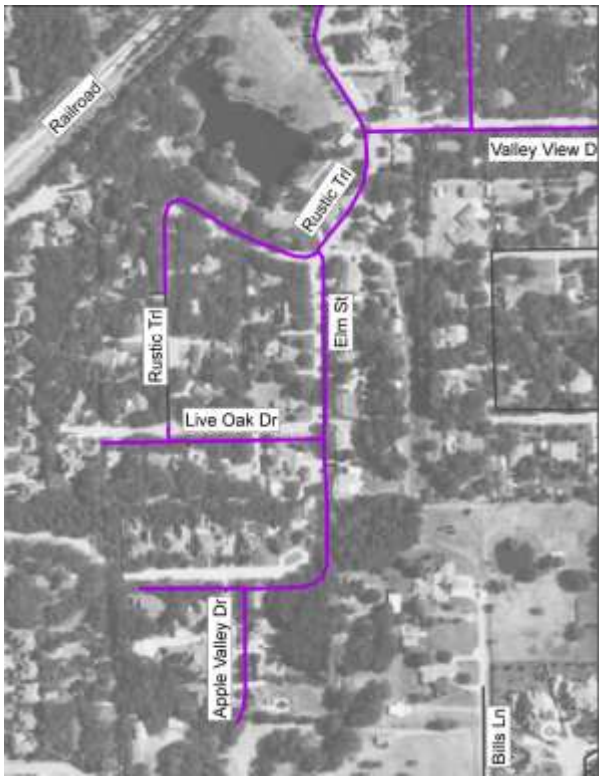
Project Identification		
Number:	15	
Location:	Low Plane	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Fire Flow	
Secondary Trigger:	Condition	
Trigger Date:	Oct 2023	
Project Complete:	Oct 2025	
Project Implementation (Months)		
Engineering/Design:	12	
Bid/Construction:	12	
Total Project Duration:	**24**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$3.50	$4.70
Engineering	$0.53	$0.69
OPCC	**$4.03**	**$5.39**

Potential Alternatives

Potential alternatives could include alternative connections or looping through other parts of the system in certain areas, but a majority of the improvements would remain as designed. The proposed alignments are believed to be the lowest cost option to the City to achieve fire flow and replace pipes of concern.

Group G			
Label	Diameter (in.)	Length (ft.)	Total Cost
P-1540	8	350	$ 70,000
P-1541	8	920	$ 182,000
P-1542	8	470	$ 93,000
P-1543	8	540	$ 107,000
P-1555	8	30	$ 5,000
P-1556	8	40	$ 7,000
P-1557	8	30	$ 5,000
P-1558	8	810	$ 161,000
P-1559	8	20	$ 4,000
P-1560	8	270	$ 53,000
P-1561	8	490	$ 98,000
P-1562	8	70	$ 14,000
P-1563	8	1430	$ 284,000
P-1564	8	30	$ 6,000
P-1565	8	20	$ 4,000
P-1566	8	1080	$ 215,000
P-1567	8	590	$ 117,000
P-1568	8	150	$ 29,000
P-1569	8	480	$ 95,000
P-1570	8	80	$ 15,000
P-1571	8	330	$ 66,000
P-1572	8	330	$ 66,000
P-1573	8	760	$ 151,000
P-1574	8	400	$ 79,000
P-1575	8	410	$ 80,000
P-1576	8	730	$ 146,000
P-1577	8	420	$ 82,000
P-1578	8	550	$ 109,000
P-1579	8	60	$ 12,000
P-1580	8	90	$ 17,000
P-1581	8	140	$ 28,000
P-1582	8	450	$ 89,000
P-1583	8	60	$ 12,000
P-1584	8	260	$ 52,000
P-1585	8	420	$ 83,000
P-1586	8	60	$ 12,000
P-1587	8	200	$ 40,000
P-1588	8	50	$ 10,000
P-1590	8	780	$ 154,000
P-1593	8	410	$ 82,000
		Subtotal	$2,917,000
		Contingency (20%)	$ 584,000
		Engineering	$ 526,000
		2015 OPCC	**$4,026,000**

Project 15 Aerials



Project 16: Water Group H Capital Improvements

Project Identification		
Number:	16	
Location:	Low Plane	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Condition	
Secondary Trigger:	Operational	
Trigger Date:	Oct 2025	
Project Complete:	April 2027	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	9	
Total Project Duration:	**18**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$1.42	$1.97
Engineering	$0.21	$0.30
OPCC	**$1.64**	**$2.27**

Project Description

The project includes 5,938 LF of AC and 8-inch line replacement along Timberline Drive. The improvements are intended to replace pipes of condition concern and address fire flow issues in that area.

Justification

The residential neighborhood in this area is currently served by a looped 6" AC line, which the City has identified as a condition concern and cannot supply adequate fire flow. Replacing this line with larger PVC pipe will alleviate the fire flow and condition issues.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

None identified.

Potential Alternatives

No potential alternative is known, as this project is a remove and replace of the existing line.

Group H			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-1544	8	610	$ 121,000
P-1545	8	880	$ 174,000
P-1546	8	1030	$ 206,000
P-1547	8	470	$ 93,000
P-1548	8	280	$ 56,000
P-1549	8	30	$ 6,000
P-1550	8	380	$ 76,000
P-1551	8	530	$ 105,000
P-1552	8	910	$ 181,000
P-1553	8	860	$ 171,000
		Subtotal	$1,186,000
		Contingency (20%)	$ 238,000
		Engineering	$ 214,000
		2015 OPCC	**$1,636,000**

Project 16 Aerial

Project 17: Water Group J Capital Improvements

Project Description

The project includes 3,578 LF of 8-inch PVC piping improvements in the northwest portion of the low plane to improve fire flow issues. This project, in conjunction with Group B, will alleviate fire flows in this area.

Justification

The mostly residential area is unable to provide minimum fire flows without the improvements.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

None identified.

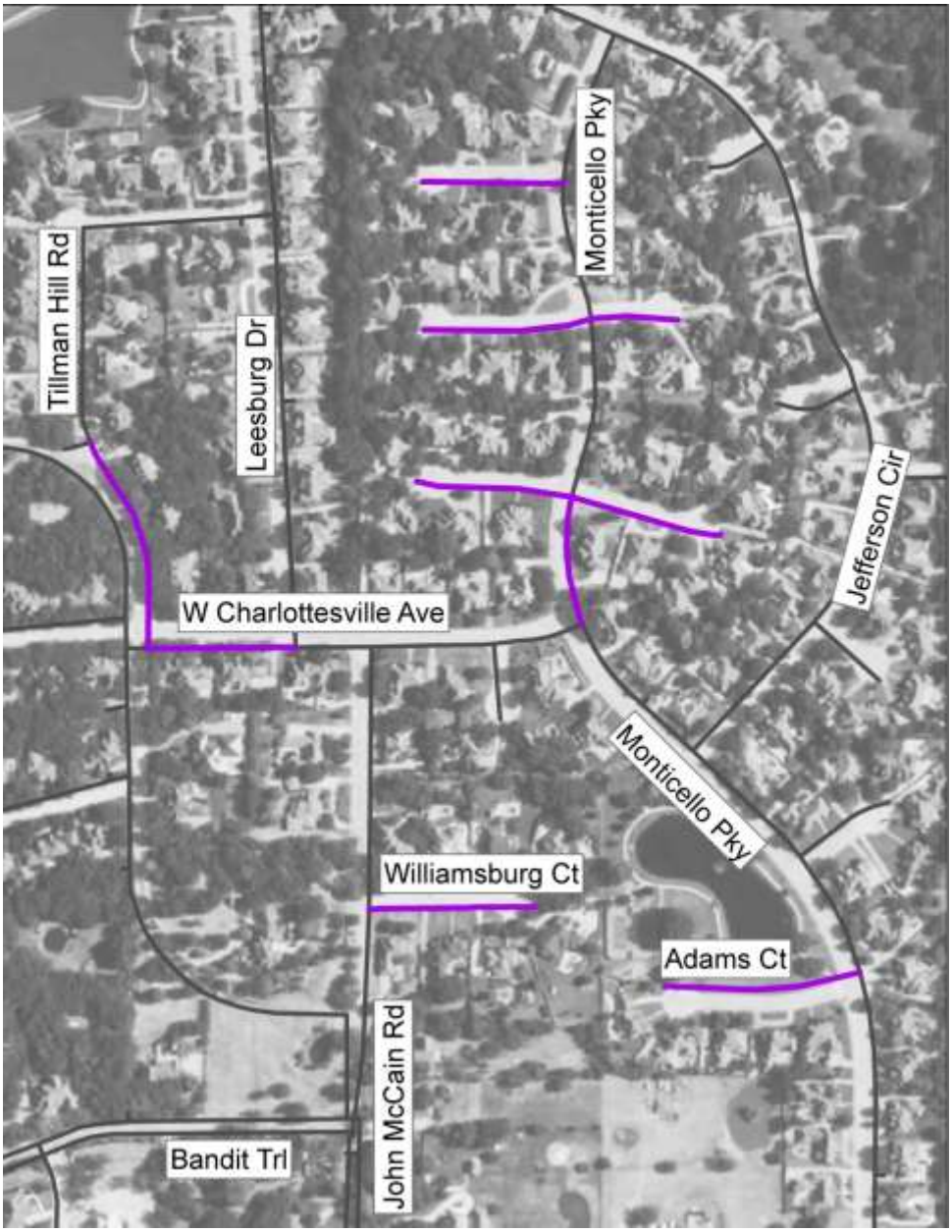
Potential Alternatives

Alternative connections or piping improvements in this area are possible. However, this represents the minimal amount of improvements to ensure this area is available to meet fire flow.

Project Identification		
Number:	17	
Location:	Low Plane	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Fire Flow	
Secondary Trigger:	None	
Trigger Date:	Oct 2026	
Project Complete:	April 2028	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	9	
Total Project Duration:	**18**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.86	$1.22
Engineering	$0.13	$0.18
OPCC	**$0.99**	**$1.41**

Group J			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-1708	8	20	$ 4,000
P-1709	8	220	$ 44,000
P-1710	8	150	$ 30,000
P-1711	8	450	$ 90,000
P-1712	8	360	$ 71,000
P-1713	8	360	$ 71,000
P-1714	8	200	$ 40,000
P-1715	8	390	$ 77,000
P-1716	8	330	$ 65,000
P-1717	8	70	$ 14,000
P-1718	8	270	$ 54,000
P-1719	8	500	$ 100,000
P-1722	8	300	$ 60,000
		Subtotal	$ 714,000
		Contingency (20%)	$ 143,000
		Engineering	$ 129,000
		2015 OPCC	**$ 986,000**

Project 17 Aerial



Project 18: Water Group K Capital Improvements

Project Identification	
Number:	18
Location:	Low Plane
Flexibility:	Medium
Schedule	
Primary Trigger:	Fire Flow
Secondary Trigger:	None
Trigger Date:	Oct 2027
Project Complete:	Oct 2029
Project Implementation (Months)	
Engineering/Design:	12
Bid/Construction:	12
Total Project Duration:	**24**

	2015 Costs ($ Millions)	Forecasted Costs ($ Millions)
Construction	$4.71	$7.12
Engineering	$0.71	$1.04
Total Project Cost	**$5.42**	**$8.16**

Project Description

The project includes close to 20,000 LF of 8-inch and 10-inch pipe improvements in the northern parts of the low plane to address inadequate fire flows.

Justification

There are significant existing lines throughout this area displaying less than 1,500 gpm fire flow in the model, due to small line size or inadequate connectivity. This project will address those issues and provide adequate fire flow to these areas.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

None identified.

Potential Alternatives

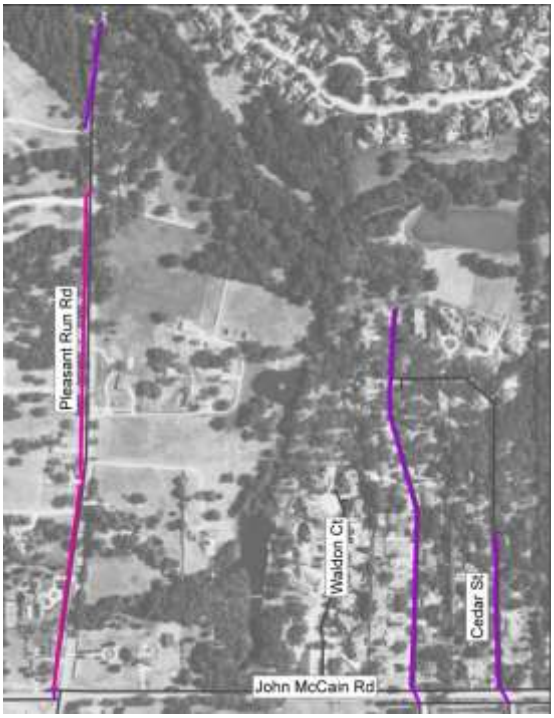
Alternative connections, piping improvements, elevated storage, or pressure increase through a booster station in this area are all possible alternatives. However, many of these improvements would likely remain to be needed, and this approach represents the minimal amount of improvements to ensure this area is available to meet fire flow.

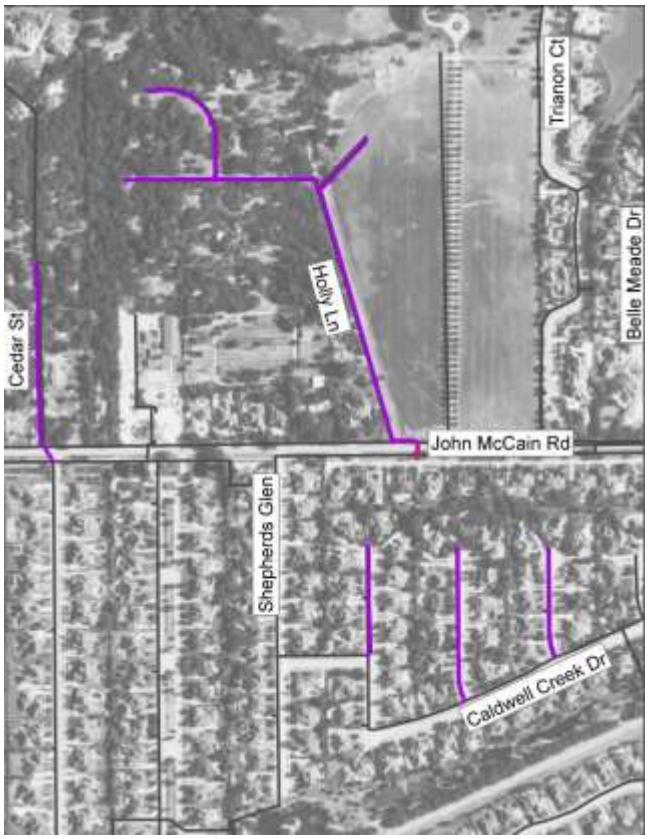
Group K			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-1728	8	140	$ 27,000
P-1729	8	150	$ 30,000
P-1730	8	210	$ 41,000
P-1731	8	280	$ 56,000
P-1732	8	30	$ 5,000
P-1733	8	130	$ 26,000
P-1734	8	160	$ 32,000
P-1735	8	180	$ 36,000
P-1736	8	170	$ 34,000
P-1737	8	430	$ 85,000
P-1739	8	40	$ 8,000
P-1740	8	170	$ 34,000
P-1741	8	510	$ 102,000
P-1742	8	210	$ 41,000
P-1743	8	400	$ 79,000
P-1745	8	320	$ 63,000
P-1746	8	760	$ 152,000

Group K (Cont.)			
Group K (Cont.)			
Label	Diameter (in.)	Length (ft.)	Total Cost
P-1747	8	10	$ 2,000
P-1748	8	480	$ 95,000
P-1749	8	460	$ 92,000
P-1750	8	30	$ 6,000
P-1751	8	430	$ 86,000
P-1752	8	220	$ 44,000
P-1754	8	50	$ 9,000
P-1755	8	20	$ 3,000
P-1756	10	950	$ 201,000
P-1757	10	1330	$ 283,000
P-1758	8	190	$ 38,000
P-1762	8	70	$ 13,000
P-1763	8	70	$ 14,000
P-1764	8	720	$ 143,000
P-1765	8	1440	$ 288,000
P-1766	8	180	$ 35,000
P-1767	8	160	$ 31,000
P-1768	10	60	$ 12,000
P-1769	8	100	$ 20,000
P-1770	8	1030	$ 205,000
P-1771	8	290	$ 57,000
P-1773	8	350	$ 69,000
P-1774	8	90	$ 17,000
P-1775	8	370	$ 74,000
P-1776	8	180	$ 35,000
P-1777	8	360	$ 72,000
P-1780	8	230	$ 46,000
P-1781	8	210	$ 42,000
P-1782	8	220	$ 44,000
P-1783	8	390	$ 77,000
P-1784	8	190	$ 37,000
P-1785	8	250	$ 50,000
P-1790	8	20	$ 4,000
P-1791	8	60	$ 11,000
P-1792	8	360	$ 72,000
P-1793	8	190	$ 38,000
P-1874	8	50	$ 10,000
Group K (Cont.)			

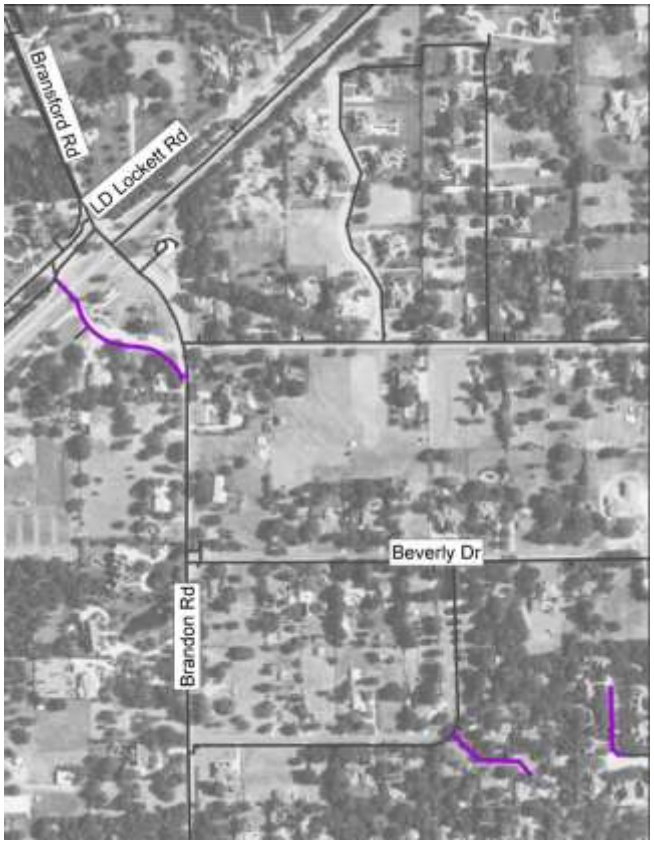
Group K (Cont.)			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-1875	8	470	$ 94,000
P-1876	8	390	$ 78,000
P-1877	8	210	$ 42,000
P-1878	8	510	$ 100,000
P-1879	8	350	$ 69,000
P-1880	8	440	$ 87,000
P-1881	8	20	$ 4,000
P-1882	8	250	$ 49,000
P-1883	8	130	$ 25,000
P-1884	8	40	$ 8,000
P-1885	8	80	$ 15,000
P-1886	8	180	$ 35,000
P-1890	8	340	$ 67,000
P-2234	8	300	$ 60,000
		Subtotal	$3,924,000
		Contingency (20%)	$ 785,000
		Engineering	$ 707,000
		2015 OPCC	**$5,414,000**

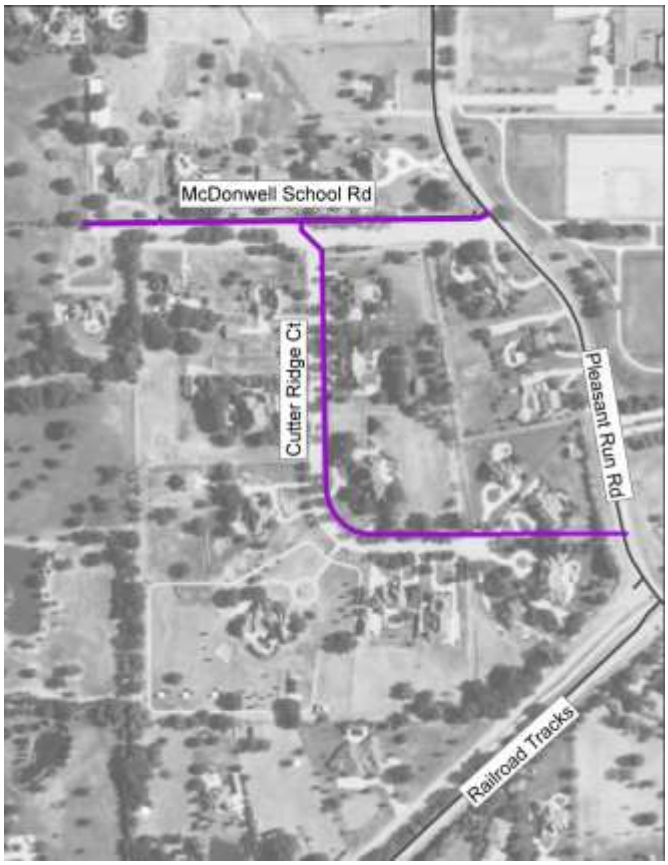
Project 18 Aerials





Additional Project 18 Aerials





Project 19: Water Group M Capital Improvements

Project Description

This project will implement 9,578 LF of 8-inch piping improvements in the southern part of the low pressure plane to improve the available fire flow.

Justification

The model revealed available fire flow issues throughout the south portion of the low pressure plane. This project will address those issues in one grouping with upsized and interconnected piping.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

None identified.

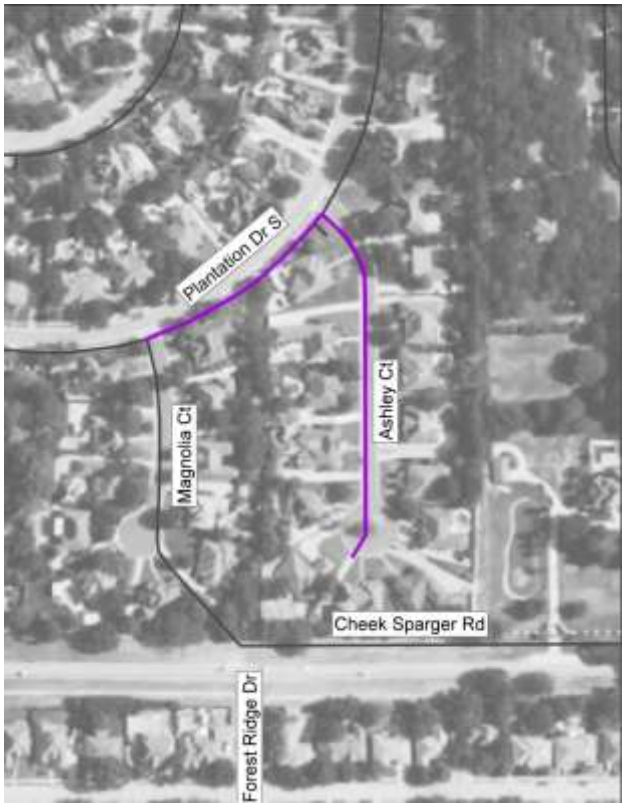
Potential Alternatives

Alternative connections, piping improvements, elevated storage, or pressure increase through a booster station in this area are all possible alternatives. However, many of these improvements would likely remain to be needed, and this approach represents the minimal amount of improvements to ensure this area is available to meet fire flow.

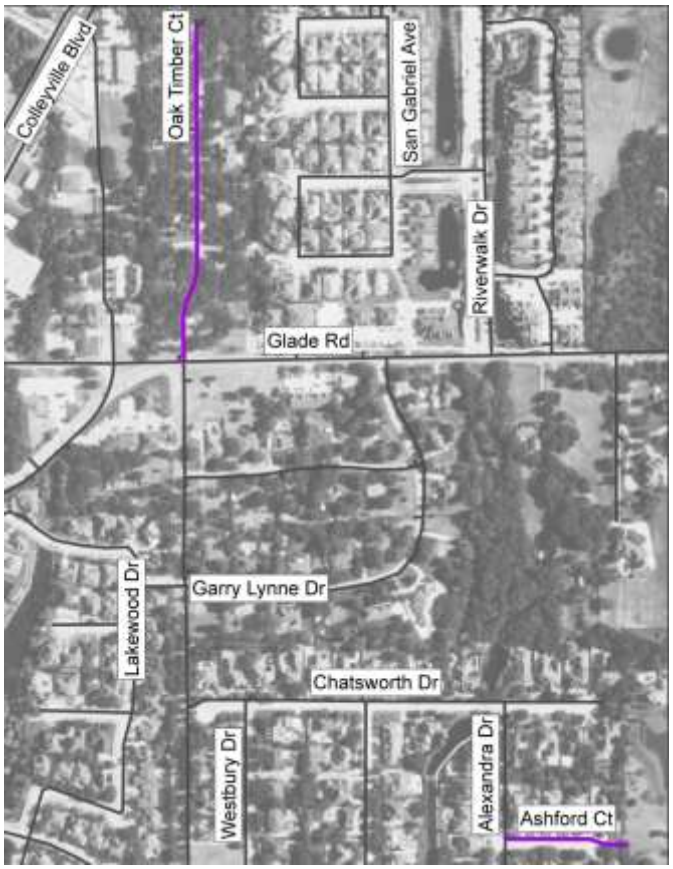
Project Identification		
Number:	19	
Location:	Low Plane	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Fire Flow	
Secondary Trigger:	None	
Trigger Date:	Oct 2028	
Project Complete:	July 2030	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	12	
Total Project Duration:	**21**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$2.29	$3.57
Engineering	$0.34	$0.52
OPCC	**$2.64**	**$4.10**

Group M			
Label	Diameter (in.)	Length (ft.)	Total Cost
P-1836	8	360	$ 72,000
P-1837	8	30	$ 5,000
P-1838	8	640	$ 127,000
P-1839	8	20	$ 3,000
P-1840	8	360	$ 71,000
P-1841	8	240	$ 46,000
P-1842	8	120	$ 24,000
P-1843	8	200	$ 40,000
P-1844	8	280	$ 54,000
P-1845	8	70	$ 14,000
P-1846	8	260	$ 52,000
P-1848	8	200	$ 39,000
P-1849	8	270	$ 54,000
P-1850	8	440	$ 88,000
P-1863	8	840	$ 167,000
P-1864	8	370	$ 73,000
P-1865	8	270	$ 54,000
P-1866	8	310	$ 62,000
P-1867	8	430	$ 85,000
P-1868	8	390	$ 78,000
P-1869	8	300	$ 60,000
P-1870	8	360	$ 70,000
P-1871	8	380	$ 76,000
P-1872	8	480	$ 95,000
P-1873	8	420	$ 83,000
P-1887	8	150	$ 30,000
P-1894	8	320	$ 64,000
P-1917	8	60	$ 12,000
P-1918	8	230	$ 46,000
P-2115	8	270	$ 53,000
P-2116	8	620	$ 123,000
P-2117	8	30	$ 5,000
		Subtotal	$1,912,000
		Contingency (20%)	$ 383,000
		Engineering	$ 345,000
		2015 OPCC	**$2,639,000**

Project 19 Aerials



Additional Project 19 Aerials



Project 20: Water Group W Capital Improvements

Project Description

This project will replace a 12-inch AC Class 200 pipe along Cheek-Sparger from Buckingham Place to Martin Parkway.

Justification

This line was identified as a conditional concern by the City. Replacing the line will decrease maintenance needs for the existing pipeline, and eliminate the integrity concerns.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

None identified.

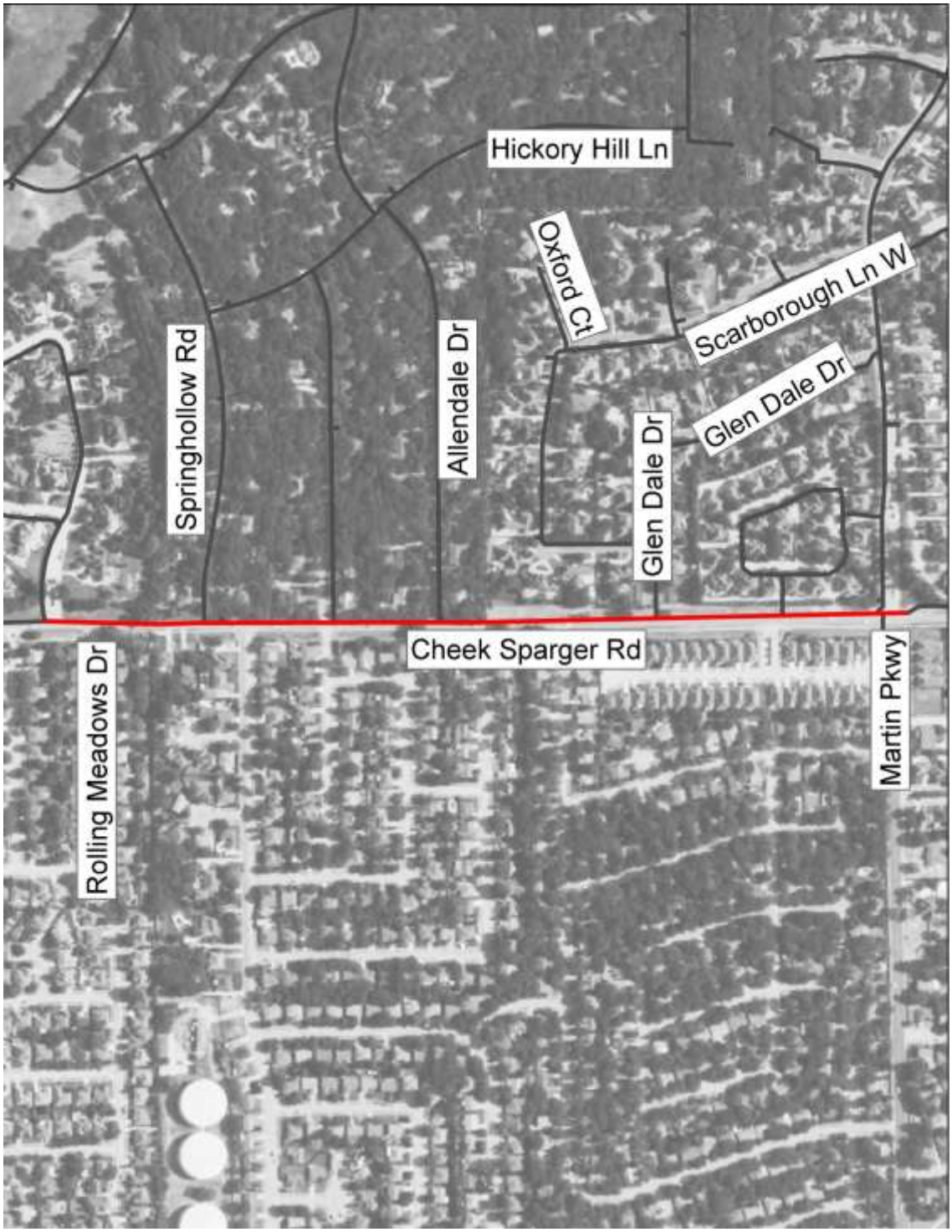
Potential Alternatives

None identified.

Project Identification		
Number:	20	
Location:	Low Plane	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Condition	
Secondary Trigger:	City-Identified	
Trigger Date:	Oct 2029	
Project Complete:	April 2031	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	9	
Total Project Duration:	**18**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.82	$1.28
Engineering	$0.12	$0.19
OPCC	**$0.95**	**$1.48**

Group W			
Label	**Diameter (in.)**	**Length (ft.)**	**Cost**
614	12	9	$ 3,000
608	12	16	$ 4,000
613	12	63	$ 15,000
609	12	78	$ 18,000
1954	12	172	$ 39,000
606	12	342	$ 77,000
611	12	379	$ 85,000
612	12	395	$ 89,000
607	12	450	$ 101,000
1957	12	567	$ 128,000
1953	12	594	$ 134,000
		Subtotal	$ 687,000
		Contingency (20%)	$ 138,000
		Engineering	$ 124,000
		OPCC	**$ 948,000**

Project 20 Aerial



Project 21: Water Group X Capital Improvements

Project Identification	
Number:	21
Location:	Low Plane
Flexibility:	High
Schedule	
Primary Trigger:	Capacity
Secondary Trigger:	City-Identified
Trigger Date:	Oct 2030
Project Complete:	Feb 2032
Project Implementation (Months)	
Engineering/Design:	7
Bid/Construction:	9
Total Project Duration:	**16**

	Forecasted Costs ($ Millions)	Forecasted Costs ($ Millions)
Construction	$0.35	$0.56
Engineering	$0.05	$0.08
OPCC	**$0.40**	**$0.65**

Project Description

This project installs 10-inch along Pool Road as directed by City preference, as well as minor improvements necessary to mitigate velocities greater than 10 fps in the distribution system.

Justification

This project continues a previous project which installed 10-inch pipeline along Pool Road. The project will address velocity-related concerns with the replacement of a 6-inch line along Summertree Lane, as well as connection of a 12-inch line along Hall-Johnson Road to a 10-inch at Meadowhill Drive.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

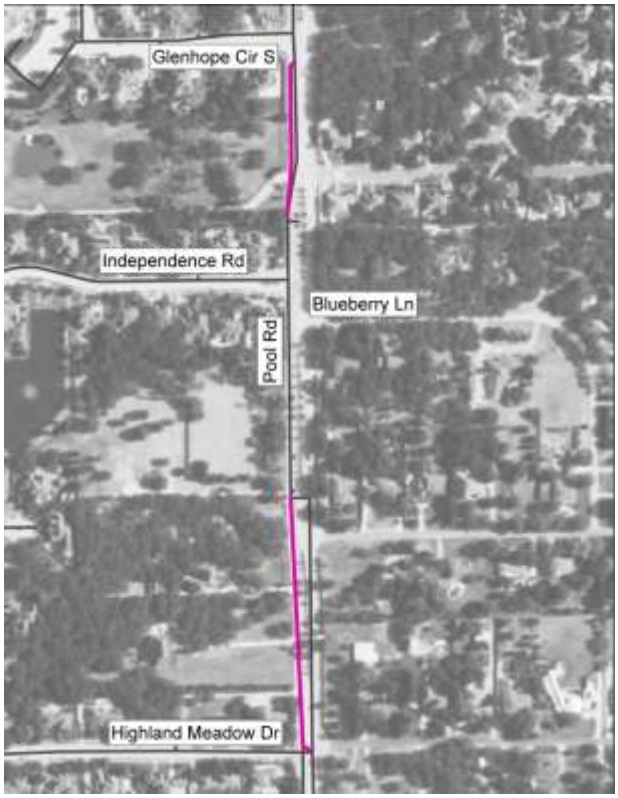
None identified.

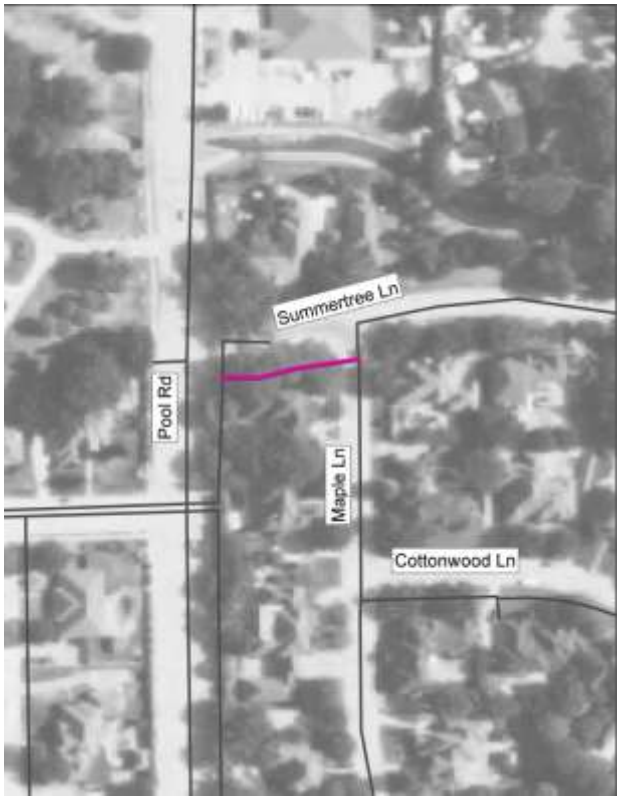
Potential Alternatives

None identified.

Group X			
Label	**Diameter (in.)**	**Length (ft.)**	**Cost**
P-1526	10	720	$ 154,000
P-1527	10	460	$ 96,000
P-2140	10	170	$ 35,000
P-2145	12	10	$ 2,000
P-2146	10	40	$ 8,000
		Subtotal	$ 292,000
		Contingency (20%)	$ 59,000
		Engineering	$ 53,000
		OPCC	**$ 403,000**

Project 21 Aerials





Project 22: Water Group C Capital Improvements

Project Identification	
Number:	22
Location:	Low Plane
Flexibility:	Medium
Schedule	
Primary Trigger:	Capacity
Secondary Trigger:	Operational
Trigger Date:	Oct 2030
Project Complete:	April 2032
Project Implementation (Months)	
Engineering/Design:	9
Bid/Construction:	9
Total Project Duration:	**18**

	2015 Costs ($ Millions)	Forecasted Costs ($ Millions)
Construction	$0.66	$1.07
Engineering	$0.10	$0.16
OPCC	**$0.76**	**$1.22**

Project Description

This project will implement 16-inch piping along Cotton Belt Trail to Pleasant Run Road (from Bransford Tank).

Justification

This project provides adequate conveyance capacity to balance levels in Bransford and Hall-Johnson. This project is necessary to maintain adequate levels in Hall-Johnson for 2034, and it is needed to maintain adequate system pressures in the area around Tinker Road and Emerald Drive.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

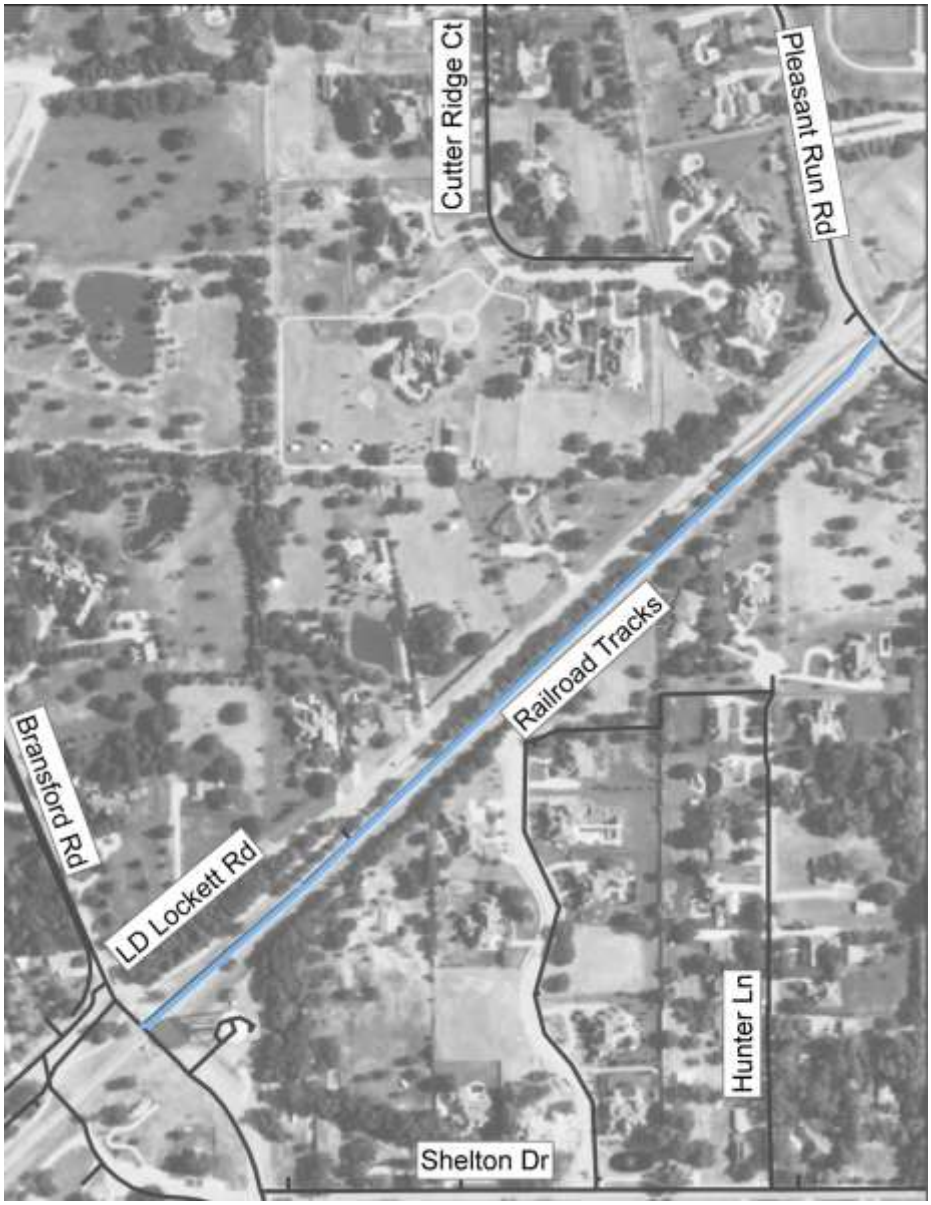
None identified.

Potential Alternatives

Alternative connections, piping improvements, elevated storage, or pressure increase through a booster station in this area are all possible alternatives. However, many of these improvements would likely remain to be needed, and this approach represents the minimal amount of improvements to ensure this area is available to meet fire flow.

Group C			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-1515	16	490	$ 117,000
P-1516	16	170	$ 39,000
P-1895	16	1280	$ 303,000
P-1896	16	410	$ 97,000
		Subtotal	$ 553,000
		Contingency (20%)	$ 111,000
		Engineering	$ 100,000
		2015 OPCC	**$ 763,000**

Project 22 Aerial



Project 23: Water Group Q Capital Improvements

Project Identification	
Number:	23
Location:	Low Plane
Flexibility:	Medium
Schedule	
Primary Trigger:	City-Identified
Secondary Trigger:	Operational
Trigger Date:	Oct 2030
Project Complete:	April 2032
Project Implementation (Months)	
Engineering/Design:	9
Bid/Construction:	9
Total Project Duration:	**18**

	2015 Costs ($ Millions)	Forecasted Costs ($ Millions)
Construction	$0.99	$1.59
Engineering	$0.15	$0.24
OPCC	**$1.14**	**$1.83**

Project Description

This project will implement 8-inch and 10-inch lines in the low plane to provide looping as required in the design criteria.

Justification

Dead end lines greater than 1,200 LF and looped lines greater than 3,000 LF will be looped as specified in the design criteria. Looping limits the number of customers who will lose service in the event of a line outage (e.g., break, repair, replacement), and improves water age conditions.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

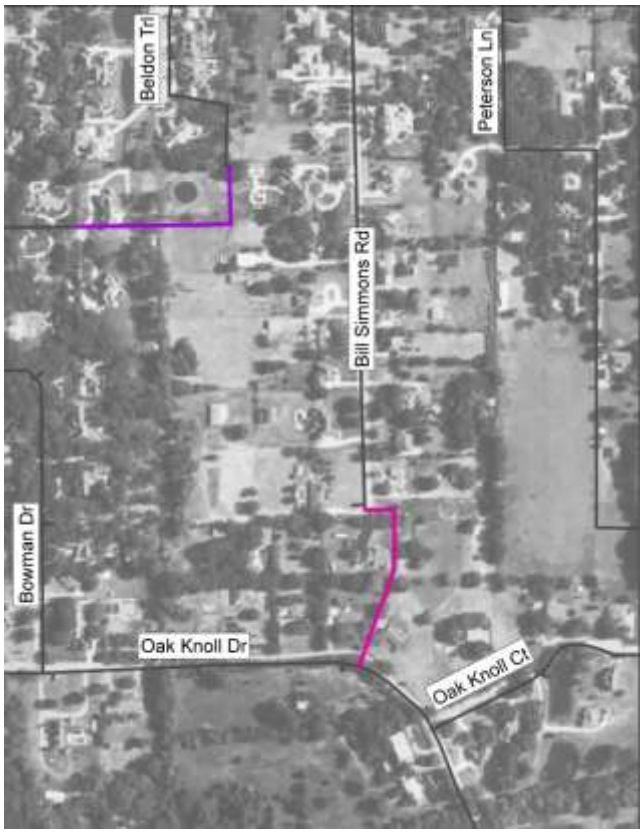
Multiple easements will likely be required along the proposed alignments.

Potential Alternatives

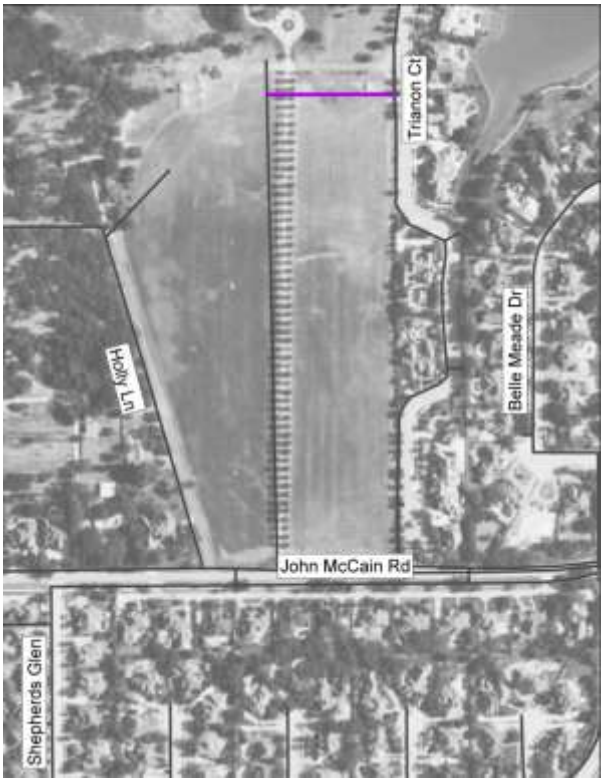
Potential alternate routes for the line along Emerald Drive are north to cul-de-sac at south end of David Lane (requires crossing railroad tracks) or south to connect to the 8" line serving Good Shepherd Catholic Church.

Group Q			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-2058	8	1,170	$ 234,000
P-2059	8	530	$ 105,000
P-2062	8	380	$ 76,000
P-2080	10	650	$ 137,000
P-2082	8	270	$ 54,000
P-2083	8	410	$ 80,000
P-2084	8	710	$ 142,000
		Subtotal	$ 825,000
		Contingency (20%)	$ 165,000
		Engineering	$ 149,000
		2015 OPCC	**$ 1,139,000**

Project 23 Aerials



Project 23 Additional Aerials



Project 24: Water Group N Capital Improvements

Project Description

This project will install a combination of 8-inch, 10-inch, and 12-inch east of the new EST in the SW portion of the low plane.

Justification

Existing pipe in this area was identified by the City as asbestos cement, and a condition concern. Additionally, these improvements will address fire flow concerns in that area.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

The section along Glade Road from Garry Lynne Drive to Riverwalk Drive should be completed with any identified roadwork projects.

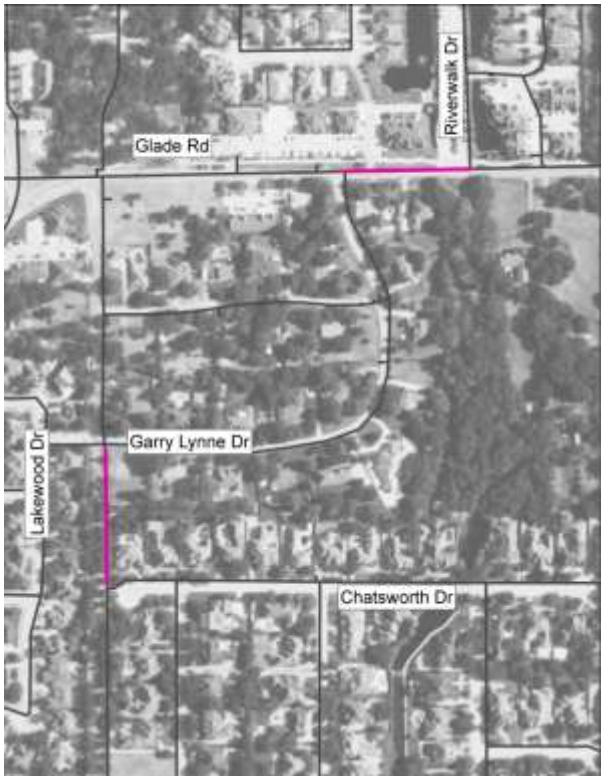
Potential Alternatives

None identified.

Project Identification		
Number:	24	
Location:	Low Plane	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Fire Flow	
Secondary Trigger:	Condition	
Trigger Date:	Oct 2031	
Project Complete:	April 2033	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	9	
Total Project Duration:	**18**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$1.35	$2.23
Engineering	$0.20	$0.34
OPCC	**$1.55**	**$2.57**

Group N			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-1599	10	140	$ 28,000
P-1600	12	910	$ 204,000
P-1601	12	50	$ 10,000
P-1602	12	40	$ 8,000
P-1603	12	20	$ 5,000
P-1604	12	700	$ 155,000
P-1605	8	840	$ 168,000
P-1828	8	480	$ 96,000
P-1900	12	700	$ 157,000
P-1901	12	30	$ 6,000
P-1902	12	30	$ 7,000
P-1903	12	160	$ 36,000
P-1904	12	370	$ 82,000
P-1905	10	220	$ 46,000
P-1906	10	200	$ 41,000
P-1908	10	30	$ 5,000
P-1909	10	330	$ 70,000
P-1910	10	40	$ 8,000
		Subtotal	$1,126,000
		Contingency (20%)	$ 226,000
		Engineering	$ 203,000
		2015 OPCC	**$1,553,000**

Project 24 Aerials



Project 25: Water Group R Capital Improvements

Project Description
This project replaces multiple sections of glue joint pipe in the area around Cheshire Drive and Dorset Drive.

Justification
The City has identified these portions of pipe as glue joint pipe and eventually necessitating replacement.

Unintended Consequences
No unintended consequences are foreseen by the implementation of this project.

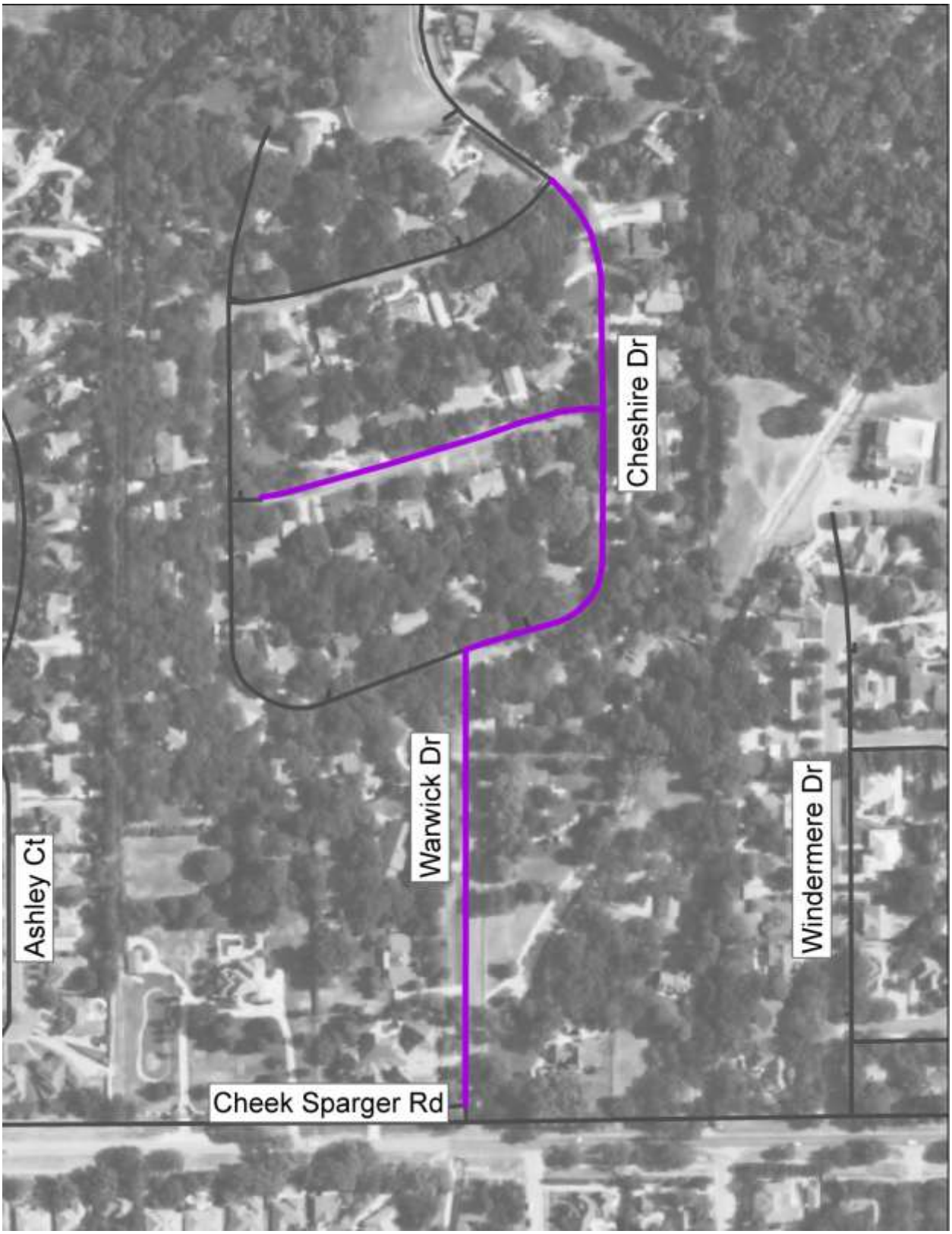
Special Considerations
None identified.

Potential Alternatives
None identified.

Project Identification		
Number:	25	
Location:	Low Plane	
Flexibility:	High	
Schedule		
Primary Trigger:	City-Identified	
Secondary Trigger:	Condition	
Trigger Date:	Oct 2032	
Project Complete:	April 2034	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	9	
Total Project Duration:	**18**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.62	$1.05
Engineering	$0.09	$0.16
OPCC	**$0.71**	**$1.21**

Group R			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
2962	8	90	$ 18,000
2957	8	130	$ 25,000
2960	8	250	$ 50,000
2959	8	410	$ 81,000
2961	8	420	$ 83,000
2963	8	460	$ 91,000
2955	8	860	$ 170,000
		Subtotal	$ 516,000
		Contingency (20%)	$ 104,000
		Engineering	$ 93,000
		2015 OPCC	**$ 712,000**

Project 25 Aerial



Project 26: Water Group S Capital Improvements

Project Description

This project will decommission a 6-inch pipeline along Bandit Trail and John McCain Road in the high plane, and connect any existing service connections to the parallel 12-inch.

Justification

The 6-inch does not provide a significant benefit with the 12-inch parallel pipeline already in place. Decommissioning will improve water age and eliminate maintenance of the 6-inch pipeline.

Unintended Consequences

Decommissioning the 6-inch may result in several unknown active line feeds being decommissioned as well.

Special Considerations

It is not known if existing hydrants have already been connected to the parallel 12-inch line. If active fire hydrants still existing on the 6-inch line, fire flow would also be a trigger, and the project may move up in priority.

Potential Alternatives

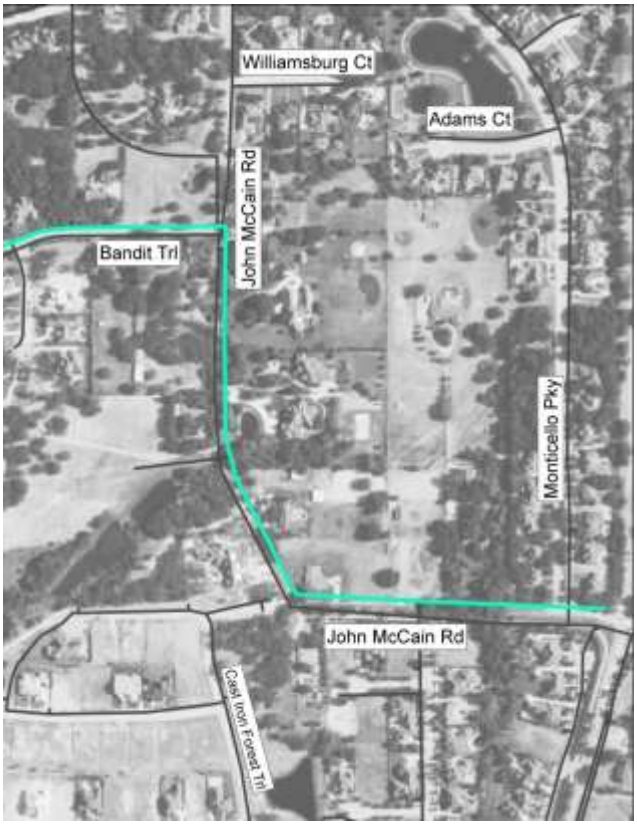
None identified.

Project Identification	
Number:	26
Location:	High Plane
Flexibility:	High
Schedule	
Primary Trigger:	City-Identified
Secondary Trigger:	Operational
Trigger Date:	Oct 2032
Project Complete:	Feb 2034
Project Implementation (Months)	
Engineering/Design:	7
Bid/Construction:	9
Total Project Duration:	**16**

	2015 Costs ($ Millions)	Forecasted Costs ($ Millions)
Construction	$0.09	$0.15
Engineering	$0.01	$0.02
OPCC	**$0.10**	**$0.18**

Group S			
Label	**Connections**	**Cost Each**	**Total Cost**
2090	30	2500	$ 75,000
2094			
2589			
2080			
P-1046			
P-1048			
P-1049			
P-1269			
P-1389			
P-1268			
		Subtotal	$ 75,000
		Contingency (20%)	$ 15,000
		Engineering	$ 14,000
		2015 OPCC	**$ 104,000**

Project 26 Aerials



Project 27: Water Group T Capital Improvements

Project Description

This project replaces all lines smaller than 10-inch along Colleyville Boulevard with 10-inchand 12-inch pipe, per City directive.

Justification

The City has identified Colleyville Boulevard as a significant retail corridor, and thereby desires a minimum 10-inch pipeline through this segment.

Unintended Consequences

No unintended consequences are foreseen by the implementation of this project.

Special Considerations

None identified.

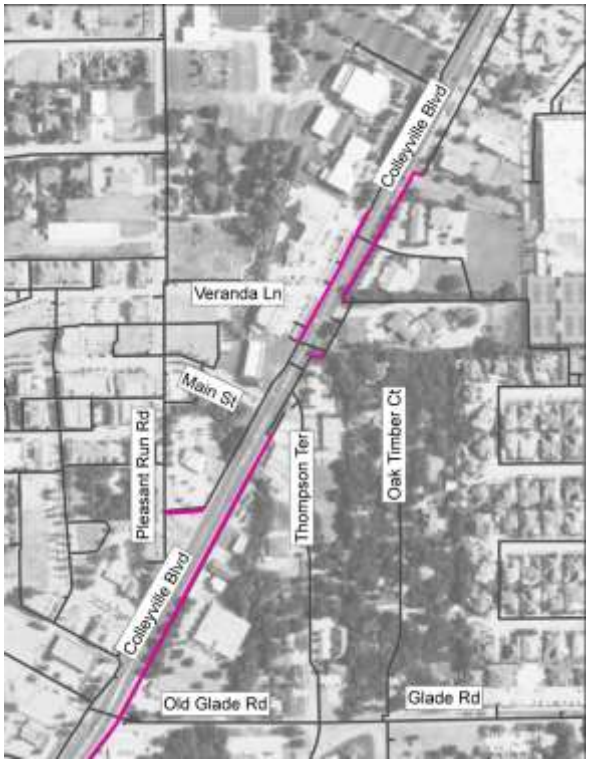
Potential Alternatives

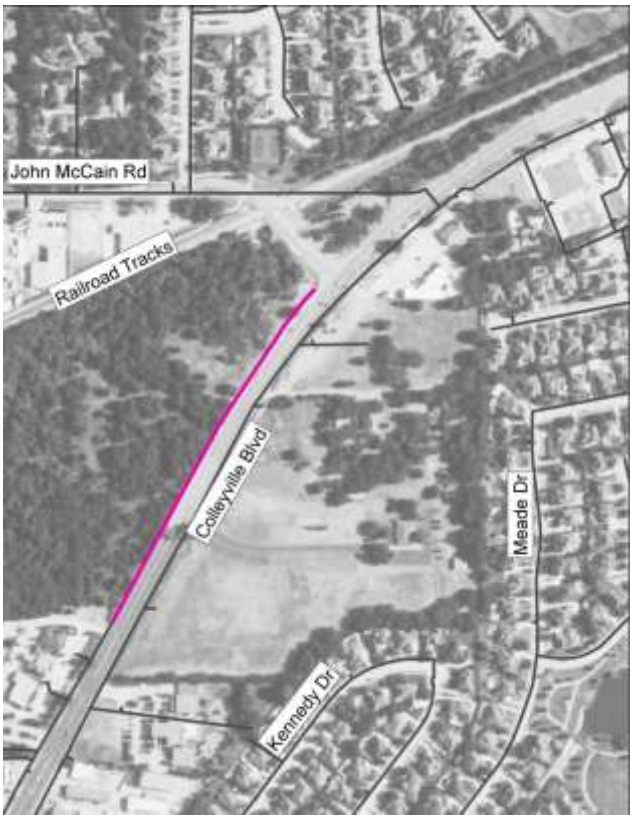
This project is identified by the City in order to provide a higher level of service along Colleyville Boulevard. However, current usage patterns do not dictate the need for upsizing piping. Therefore, if future growth is not realized along this corridor, upsizing portions of the line may not be necessary.

Project Identification		
Number:	27	
Location:	Low Plane	
Flexibility:	High	
Schedule		
Primary Trigger:	City-Identified	
Secondary Trigger:	None	
Trigger Date:	Oct 2033	
Project Complete:	July 2035	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	12	
Total Project Duration:	**21**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$2.21	$3.99
Engineering	$0.33	$0.58
OPCC	**$2.54**	**$4.57**

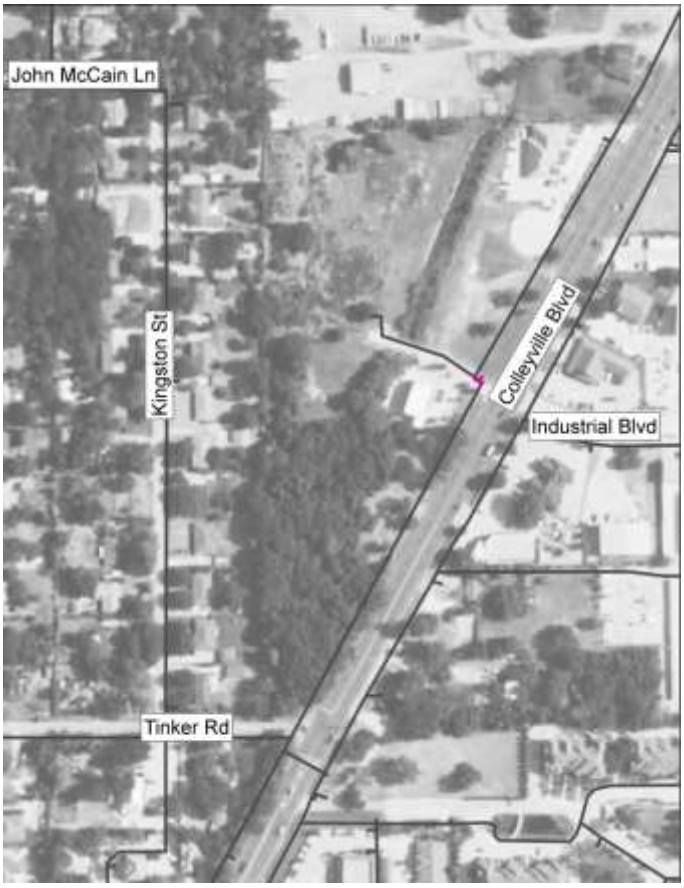
Group T			
Label	**Diameter (in.)**	**Length (ft.)**	**Total Cost**
P-2087	10	940	$ 199,000
P-2088	10	400	$ 84,000
P-2089	10	20	$ 4,000
P-2090	10	350	$ 75,000
P-2091	10	30	$ 6,000
P-2092	10	1,170	$ 248,000
P-2093	10	50	$ 9,000
P-2094	10	120	$ 25,000
P-2095	10	230	$ 49,000
P-2096	10	220	$ 47,000
P-2102	10	1,330	$ 282,000
P-2103	10	20	$ 3,000
P-2104	10	20	$ 3,000
P-2105	10	110	$ 22,000
P-2106	10	130	$ 26,000
P-2107	10	90	$ 19,000
P-2108	10	190	$ 39,000
P-2109	10	60	$ 13,000
P-2110	10	160	$ 33,000
P-2113	10	80	$ 17,000
P-2114	10	40	$ 7,000
P-2121	12	230	$ 50,000
P-2122	12	300	$ 66,000
P-2123	12	40	$ 9,000
P-2124	12	340	$ 75,000
P-2125	12	320	$ 71,000
P-2126	12	690	$ 155,000
P-2127	12	40	$ 9,000
P-2128	12	930	$ 208,000
		Subtotal	$1,841,000
		Contingency (20%)	$ 369,000
		Engineering	$ 332,000
		2015 OPCC	**$2,540,000**

Project 27 Aerials





Additional Project 27 Aerials



Appendix E

Wastewater Capital Improvements Plan

1.0 Executive Summary

This wastewater Capital Improvements Plan utilizes the model criteria and simulations detailed in the Master Plan to develop proposed projects from the recommended improvements. These improvements are identified for a 20-year planning horizon.

Shorter CIP programs can be developed by limiting the timeframe for the proposed projects as desired. For instance, a current 5-year CIP can be developed by utilizing all projects from the 2015-2020 planning horizon. A summary of all proposed project costs and schedules is displayed on Page 4, while a proposed schedule is located on Page 5. Project descriptions are in order based on priority ranking following this summary.

1.1 Identification and Ranking

Each project is identified by up to two of the four known triggers (capacity, condition, operational, or City-identified). As these triggers activated a need, alphabetical project groupings were developed in localized areas to address that need. These were then ranked numerically based on the criticality of the project and service area impacted. Higher priority was given to areas which exhibited greater than 95% capacity during peak flow conditions, or elevated condition concerns.

1.1.1 Timeline

This priority was then applied to an overall timeline, in order to meet a 20-year planning horizon. Each project has also been assigned a flexibility rating of low, medium, or high. Projects with higher flexibility can be extended later in the planning horizon, depending on the City's available funding or changing system conditions which may impact the need for the project (such as unexpected delayed City-wide growth delaying the need for capacity improvements).

The project priority dictates the trigger date. Projects with higher priority will trigger sooner than those with lower priority. The trigger date is then utilized to capture anticipated costs for the life of the project, by escalating the total estimated 2015 costs at a rate of 3% to the trigger date for the engineering and construction items.

1.2 Cost Development

Costs estimates were prepared for each individual project, based on industry standards and the 2014 bidding environment. These costs are an estimate, and should be re-evaluated as each project nears the trigger date. Each project has the following costs associated with the total OPCC:

- Opinion of Probable Construction Cost (OPCC)

  The OPCC is the report level estimate of Contractor's bid price once the project has been designed and is ready for the bid phase to begin. It represents a combination of the estimated total construction costs, engineering, and a 20% estimating contingency.

- Engineering

  Engineering includes estimates of professional services needed to bid each project, including survey, geotechnical, deed research (as needed), preliminary, and final design of all improvements. This cost represents 15% of the OPCC. Construction engineering is not included, since those services are assumed to be provided by City staff.

- Forecasted Project Costs

  Forecasted project costs are the Opinion of Construction Costs (OPCC) with a 3 percent escalation for inflation to the Trigger Date month and year.

Costs for manhole rehabilitation were developed in conjunction with Pipeline Analysis' assessment and cost estimations. Pipeline installation costs were developed from a combination of Dallas Water Utilities' Average Costs Manual, recent neighboring city water and sewer project bid tabulations, and 2014 water and sewer bid tabulations provided by the City of Colleyville. The developed cost estimates are presented in Table 1, and individual item descriptions follow.

Table 1: Sanitary Sewer Cost Development

PVC Sewer Line Cost Estimates ($ per LF)						
Size (in.)	General Improvements	Connections & Manholes	Line BOTOC	Open Cut	Total Pipe Install	Extra Depth +20' Unit Price
6	$42	$48	$180	$96	$202	$304
8	$45	$51	$190	$102	$206	$309
10	$47	$54	$195	$107	$212	$318
12	$49	$56	$200	$111	$218	$327

Total Pipe Install Assumes 10% BOTOC

Bid Item Descriptions:

General Improvements: All sitework, backfill, erosion control, rehabilitation of existing structures, testing, and contractor overhead costs.

Connections and manholes: Cost for all manholes, assuming an even distribution of 4-foot and 5-foot diameter installations. Also includes all sewer service wye connections and connections to existing pipes or manholes.

Line BOTOC: (By Other Than Open Cut): The cost for installation of SDR-26 where boring is required, such as sensitive utility areas or across highways or railroads.

Open Cut: This line item contains the estimated cost of materials and labor to install SDR-26 PVC in an open trench

Total Pipe Install: This is the summary of all previous line items, and utilized on a linear footage basis for the cost estimates, depending on line size. As noted in Table 1, it is assumed that 10% of project linear footage will require line install BOTOC.

Extra Depth +20' Unit Price: This cost reflects the additional cost to excavate and backfill pipes greater than 20 feet deep, as well as the additional manhole costs for this depth. It also reflects additional trench safety and site improvements needed by the larger trenches which will be required.

1.3 Project Descriptions and Legend

Individual descriptions are included for each project, along with associated aerials. Figure 1 to the right displays the legend for each of these aerials. For line projects, summary sheets displaying the total linear footage of pipe and the individual pipe segments identified for improvements are included. These line segments correspond to the line segment labels within the updated model provided with the Master Plan.

1.4 Cost and Schedule Summary

A summary of all proposed projects, costs, and schedules follows on the next page, with a proposed project schedule on the following page.

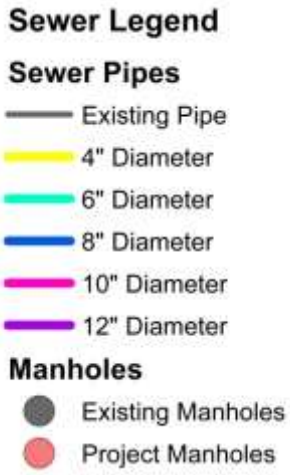
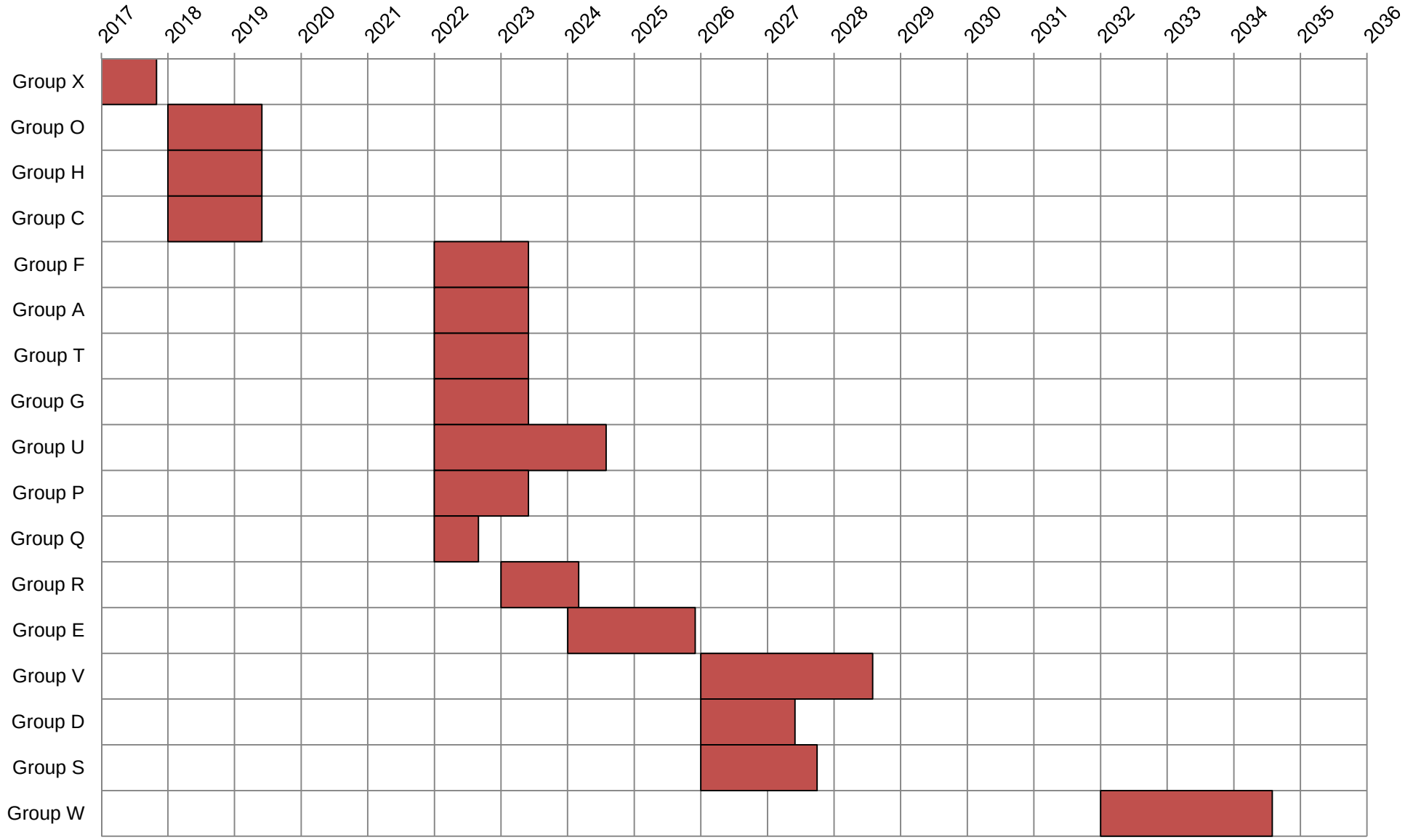


Figure 1 – Aerial Legend

Wastewater CIP Project Summary

Project Identification				Schedule							2015 Cost ($1,000)	Forecasted Cost ($1,000)		
Project	Description	Location	Flexibility	Primary Trigger	Secondary Trigger	Trigger Date	Project Complete	Engineering /Design	Bid/ Construction	Total Project Duration	OPCC	Construction	Engineering	OPCC
1	Group X	Area 3	Low	City-directed		Oct-17	Jul-18		9	9	$1,556	$1,701		$1,701
2	Group O	City-Wide	Low	Condition	None	Oct-18	Feb-20	7	9	16	$185	$182	$27	$209
3	Group H	City-Wide	Low	City-directed	Condition	Oct-18	Feb-20	7	9	16	$497	$486	$73	$559
4	Group C	North	Low	Capacity	None	Oct-18	Feb-20	7	9	16	$193	$189	$28	$217
5	Group F	North	Low	Capacity	Condition	Oct-22	Feb-24	7	9	16	$616	$679	$102	$781
6	Group A	West	Low	Capacity	None	Oct-22	Feb-24	7	9	16	$632	$696	$104	$801
7	Group T	South	Low	City-directed	Condition	Oct-22	Feb-24	7	9	16	$624	$687	$103	$790
8	Group G	East	Low	Capacity	None	Oct-22	Feb-24	7	9	16	$325	$358	$54	$412
9	Group U	City-Wide	High	Condition	None	Oct-22	Apr-25	15	15	30	$2,259	$2,563	$373	$2,936
10	Group P	City-Wide	Medium	Condition	None	Oct-22	Feb-24	7	9	16	$560	$617	$93	$709
11	Group Q	City-Wide	Medium	Condition	None	Oct-22	May-23	7	0	7	$218	$	$277	$277
12	Group R	City-Wide	Medium	Condition	None	Oct-23	Nov-24	7	6	13	$318	$361	$54	$415
13	Group E	East	Medium	Capacity	None	Oct-24	Aug-26	9	13	22	$1,020	$1,228	$179	$1,406
14	Group V	City-Wide	High	Condition	None	Oct-26	Apr-29	15	15	30	$2,259	$2,884	$420	$3,304
15	Group D	South	Medium	Capacity	None	Oct-26	Feb-28	7	9	16	$531	$659	$99	$758
16	Group S	City-Wide	High	Capacity	None	Oct-26	Jun-28	7	13	20	$993	$1,231	$185	$1,415
17	Group W	City-Wide	High	Condition	None	Oct-32	Apr-35	15	15	30	$2,259	$3,444	$502	$3,945
										Total 2015 OPCC:	**$15,044,200**		**Total Forecasted OPCC:**	**$20,635,500**

Proposed 20 Year Wastewater CIP Schedule



A proposed spending schedule follows. Further refinement of this spending schedule and associated project trigger dates is available, depending upon City funding timeframes.

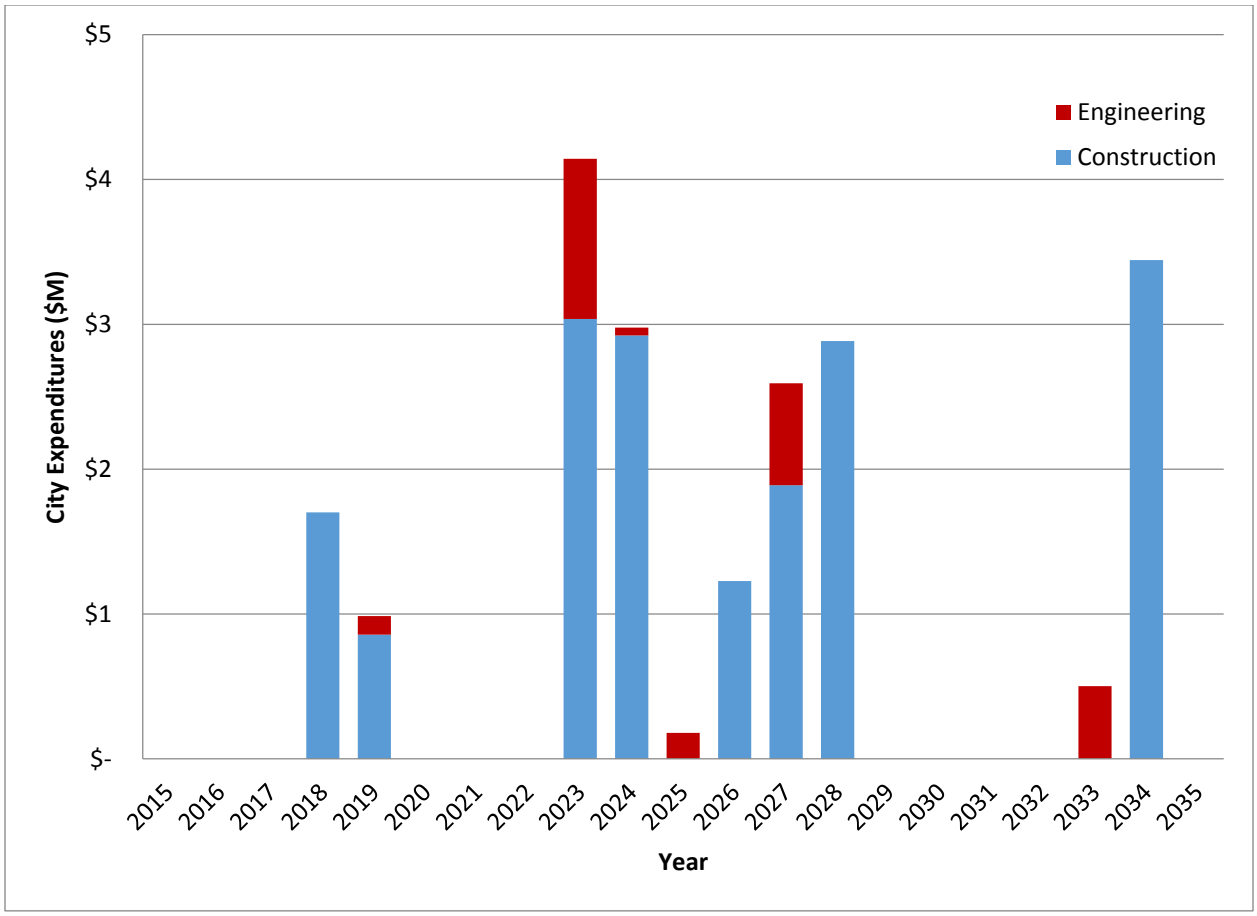


Figure 2 - Proposed Spending Schedule for Sewer CIP

Individual project descriptions are detailed in the following pages. A further description on project grouping is located in the Master Plan.

Project 1: Sewer Group X Improvements

Project Description

The project involves the sewer line renewals of Area C per City direction of the City's 2013 CIP. Projects include the replacement of 6, 8, and 10 inch pipelines. Approximately, 4,885 LF, 1,947 LF, and 262 LF of 6, 8, and 10 inch pipeline will be replaced respectively. Project 1 also includes rehabilitation and installation of new manholes throughout Area C. In addition, Group X improvements include the rehabilitation of various curbs, sidewalks, and gutters.

Justification

These improvements were previously outlined by the City from 2013 CIP, and are therefore included in this CIP to reflect all current and recommended capital improvement project.

Unintended Consequences

None identified.

Special Considerations

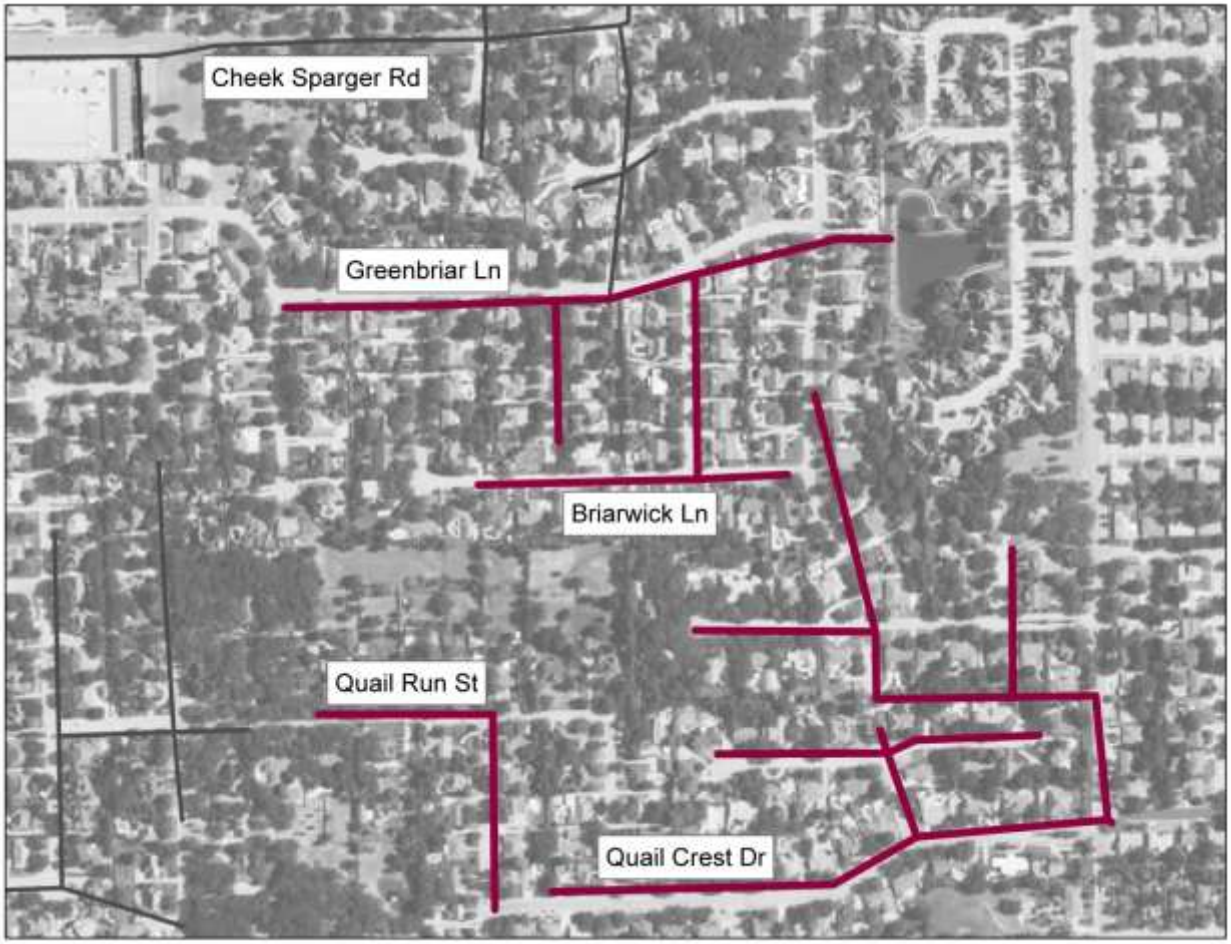
None identified.

Potential Alternatives

None identified.

Project Identification		
Number:	1	
Location:	Area 3	
Flexibility:	Low	
Schedule		
Primary Trigger:	City-identified	
Secondary Trigger:	None	
Trigger Date:	Oct 2017	
Project Complete:	July 2018	
Project Implementation (Months)		
Engineering/Design:	-	
Bid/Construction:	9	
Total Project Duration:	**9**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$1.56	$1.70
Engineering	$ -	$ -
OPCC	**$1.56**	**$1.70**

Project 1 Aerials



Project 2: Sewer Group O Improvements

Project Description

The project involves the rehabilitation of Priority 1 manholes throughout the City identified in the manhole assessment. The assessment was conducted in conjunction with the formation of this Master Plan.

Methods for repair will vary depending on the type and severity of the defect. Manholes located in groups that have been proposed to be addressed within five years have been removed from this group and these estimated costs, as those manholes will be replaced with the separate projects.

An overall location map for the proposed manholes is located on the following page, along with a breakout of project costs. Further cost information is located in the manhole condition assessment report, completed in June 2014 and accompanying this Master Plan.

Project Identification		
Number:	2	
Location:	City-wide	
Flexibility:	Low	
Schedule		
Primary Trigger:	Condition	
Secondary Trigger:	None	
Trigger Date:	Oct 2018	
Project Complete:	Feb 2020	
Project Implementation (Months)		
Engineering/Design:	7	
Bid/Construction:	9	
Total Project Duration:	**16**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.16	$0.18
Engineering	$0.02	$0.03
OPCC	**$0.19**	**$0.21**

Justification

Through the manhole assessment study, it was determined that these manholes presented severe defects that, if left untreated, could eventually contribute to manhole failure. Additionally, infiltration was witnessed during the inspections and would be lessened by incorporating these improvements. It is intended that the base wastewater flow would be decreased as the inflow and infiltration into the system is improved, and the manhole useful life would be extended by these repairs.

Unintended Consequences

Reducing infiltration in certain areas may cause groundwater tables to rise slightly, however this is not expected to be significant.

Special Considerations

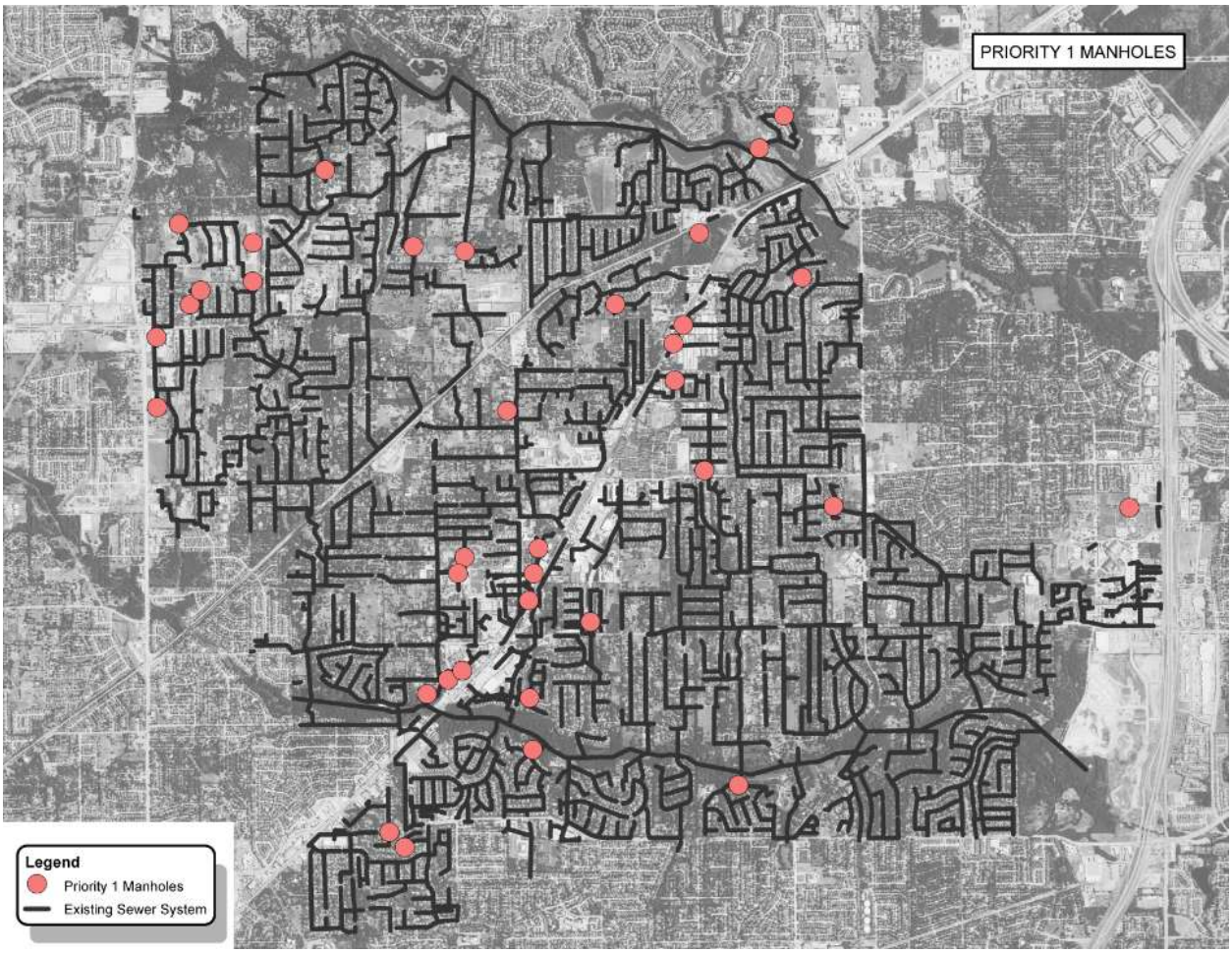
Consideration should be given to the type of rehabilitation technology employed. Waterproofing manholes in high groundwater tables may have a tendency to cause the manholes to float, and some manhole rehabilitation types would prevent future modifications to the manhole inverts (thereby requiring full manhole replacement if future lines are added or modified).

Potential Alternatives

While various alternatives exist for each of the recommended rehabilitation methods, the only alternative to manhole rehabilitation in most cases is total replacement, which is anticipated to be more expensive.

Group O	
Label	**2015 Total Cost**
Priority 1 MH Improvements (minus grouped replacements)	$ 135,000
Subtotal	$ 135,000
Contingency (20%)	$ 27,000
Engineering	$ 25,000
2015 OPCC	**$ 186,000**

Project 2 Aerial



Priority 1 Assessed Manhole locations Throughout City

Project 3: Sewer Group H Line Improvements

Project Description

The City staff have identified several pipelines for replacement due to condition and operational issues associated with each line segment. Two of the lines are 8-inch, while two are 6-inch.

Justification

City staff have reported the following issues with these portions of line:

- The Quail's Path line has been cut and patched numerous times
- The line near the soccer fields is a bottleneck and causing septic sewage
- The 6" across Colleyville needs upsized due to condition
- The Manning line segment needs replaced due to condition

Unintended Consequences

None identified.

Special Considerations

Alternative methods to open trench may prove more cost effective and should be evaluated during detailed design, such as adding larger diameters in place of increased slopes.

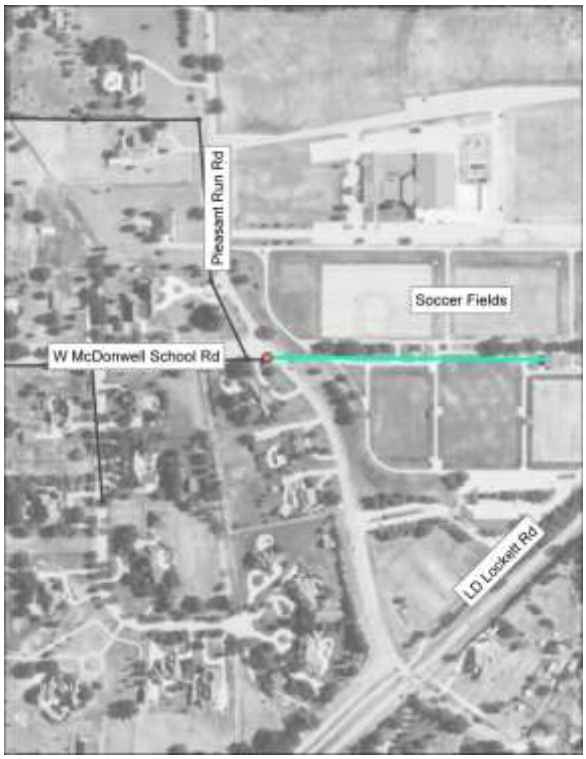
Potential Alternatives

None identified.

Project Identification		
Number:	3	
Location:	City-wide	
Flexibility:	Low	
Schedule		
Primary Trigger:	City-identified	
Secondary Trigger:	Condition	
Trigger Date:	Oct 2018	
Project Complete:	Feb 2020	
Project Implementation (Months)		
Engineering/Design	7	
Bid/Construction	9	
Total Project Duration	**16**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.43	$0.49
Engineering	$0.07	$0.07
OPCC	**$0.50**	**$0.56**

Group H				
Label	**Diameter (in.)**	**Length (ft.)**	**Avg. US & DS Depth (ft.)**	**Pipe Cost**
204-1522	8	150	12	$ 30,000
1522-1519	8	140	11	$ 29,000
Soccer Fields-2151	6	860	15	$ 173,000
1931-1930	6	640	-	$ 129,000
			Subtotal	$ 361,000
			Contingency (20%)	$ 73,000
			Engineering	$ 65,000
			2015 OPCC	**$ 498,000**

Project 3 Aerials



Project 4: Sewer Group C Line Improvements

Project Identification	
Number:	4
Location:	North
Flexibility:	Low
Schedule	
Primary Trigger:	Capacity
Secondary Trigger:	None
Trigger Date:	Oct 2018
Project Complete:	Feb 2020
Project Implementation (Months)	
Engineering/Design	7
Bid/Construction	9
Total Project Duration	**16**

	2015 Costs ($ Millions)	Forecasted Costs ($ Millions)
Construction	$0.17	$0.19
Engineering	$0.03	$0.03
OPCC	**$0.19**	**$0.22**

Project Description

This project includes replacement of 4 10-inch pipelines with 10-inch PVC at improved slope design to provide adequate capacity at existing and design flow. It is anticipated that installation will be via open trench, due to the change in slope for the designed pipelines.

Justification

These pipes show to be severely surcharged during the design flow rate, due to the inadequate slope that they exhibit (ranging from 0.01 to -0.00014).

Unintended Consequences

None identified.

Special Considerations

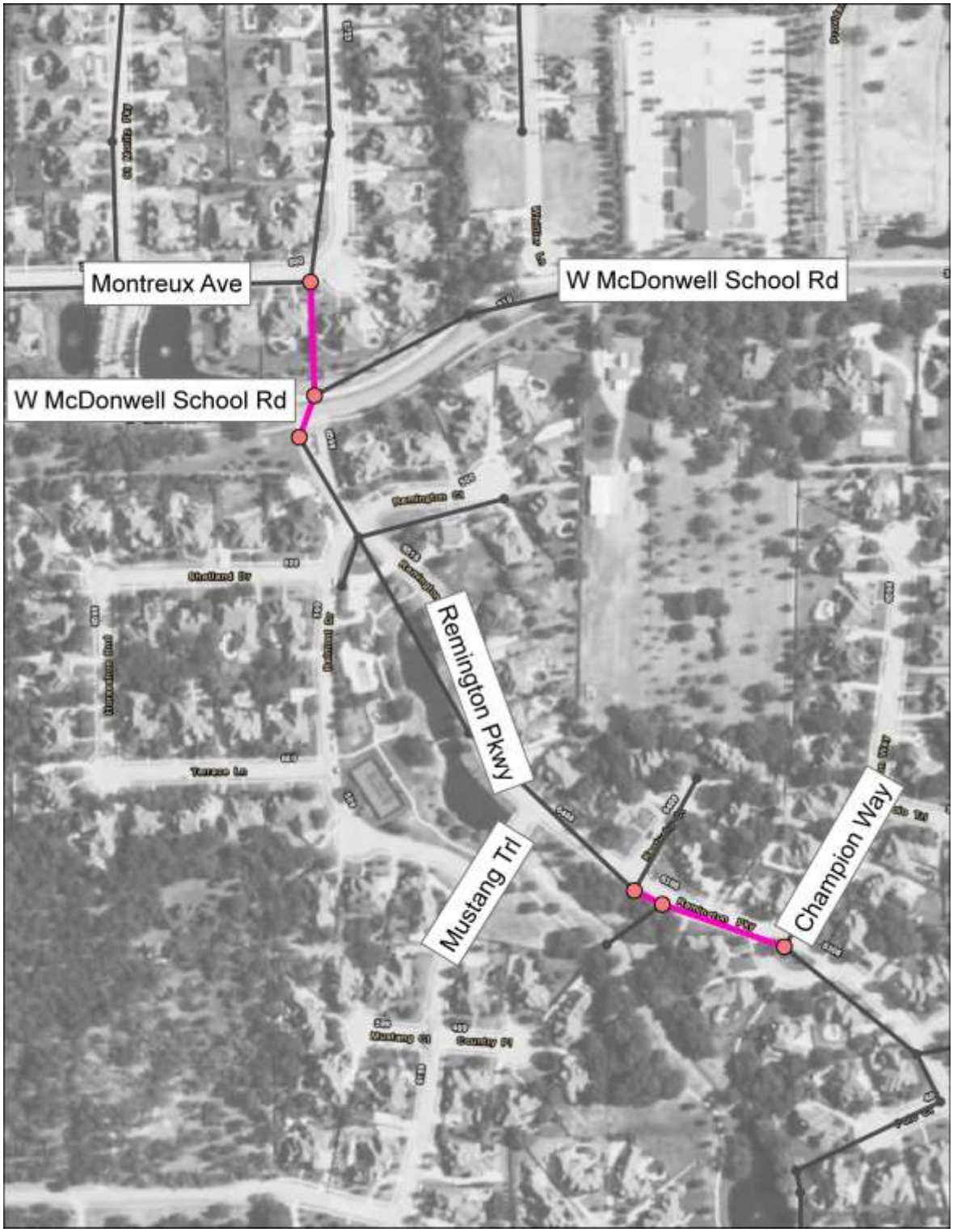
Consideration should be given to the type of pipeline installation method chosen during construction, to ensure that proper slopes are met and the existing pipeline at either end of the improvement is maintained. Slopes should be verified during detailed design.

Potential Alternatives

If The Reserve Lift Station is placed off-line and flow is diverted to another drainage basin, this project reduces in priority. However, due to the minimum slope issues and potential for surcharge, it is recommended that these sections be replaced. An alternative to the 10-inch design may be the installation of 12-inch pipeline, depending on the results of detailed design survey.

Group C				
Label	**Diameter (in.)**	**Length (ft.)**	**Avg. US & DS Depth (ft.)**	**Pipe Cost**
779-1263	10	270	10.00	$ 57,000
1261-779	10	70	10.00	$ 14,000
2478-1277	10	100	10.00	$ 20,000
2477-2478	10	240	11.00	$ 50,000
			Subtotal	$ 140,000
			Contingency (20%)	$ 28,000
			Engineering	$ 26,000
			2015 OPCC	**$ 193,000**

Project 4 Aerial



Project 5: Sewer Group F Line Improvements

Project Description

The project includes replacement of 8-inch and 10-inch sanitary sewer line with 10-inch PVC along Quail's Path and LD Lockett. Improvements are intended to address capacity concerns in the model, as inadequate pipe size and slopes have the potential to cause surcharge within the manholes.

Justification

The model shows this section of line to surcharge when faced with current and future system demands, due to a reduction in pipe size and inadequate slopes. Therefore, this project will address surcharge issues that may occur.

Unintended Consequences

None identified.

Special Considerations

None identified.

Potential Alternatives

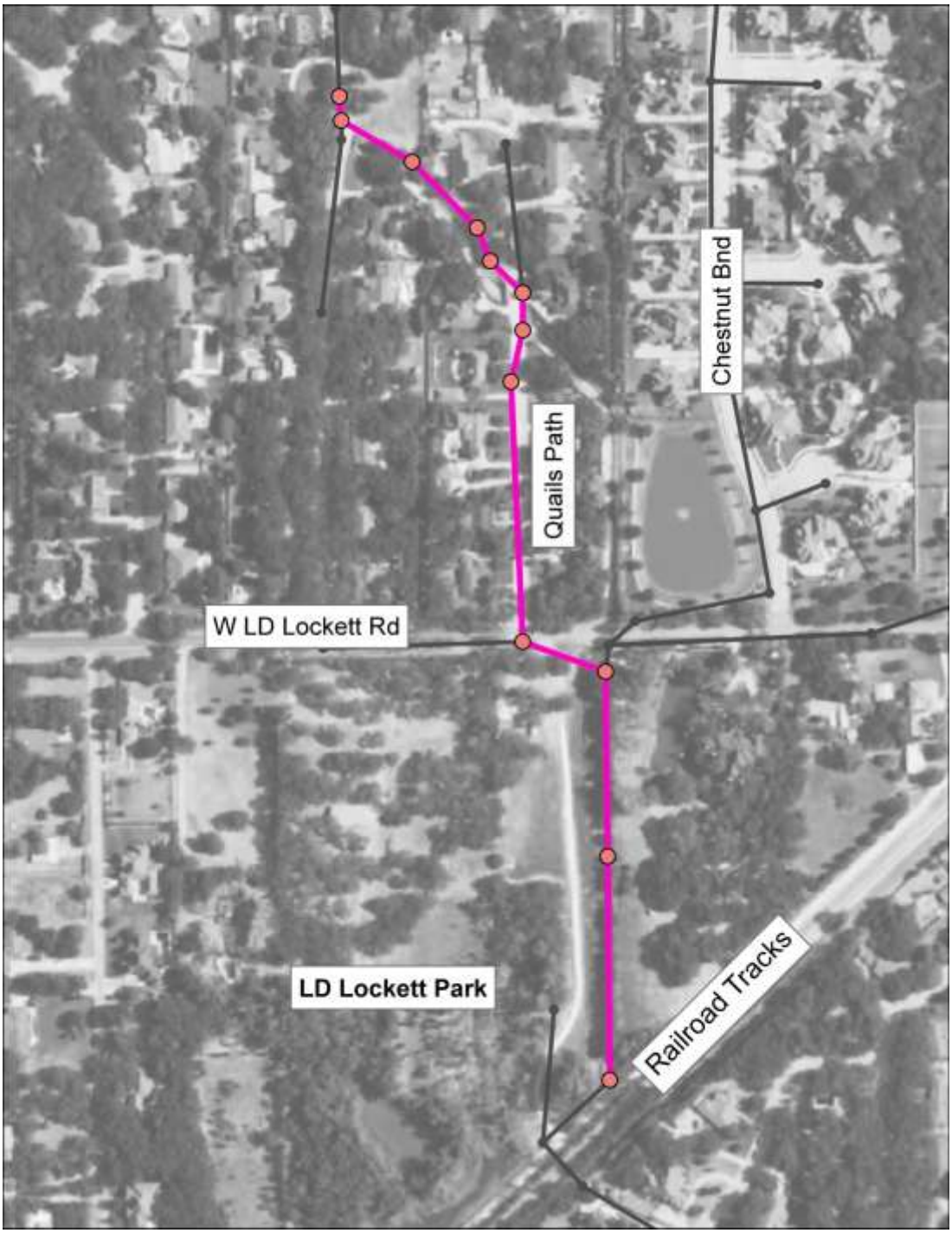
None identified, as pipe slopes must change. Therefore, all new line and manholes must be installed.

Project Identification	
Number:	5
Location:	North
Flexibility:	Low
Schedule	
Primary Trigger:	Capacity
Secondary Trigger:	Condition
Trigger Date:	Oct 2022
Project Complete:	Feb 2024
Project Implementation (Months)	
Engineering/Design	7
Bid/Construction	9
Total Project Duration	**16**

	2015 Costs ($ Millions)	Forecasted Costs ($ Millions)
Construction	$0.54	$0.68
Engineering	$0.08	$0.10
OPCC	**$0.62**	**$0.78**

Group F				
Label	**Diameter (in.)**	**Length (ft.)**	**Avg. US & DS Depth (ft.)**	**Pipe Cost**
1671-1672	10	420	10	$ 89,000
2291-1671	10	350	10	$ 74,000
1669-2291	10	170	11	$ 35,000
2831-1669	10	490	11	$ 104,000
1661-2831	10	100	10	$ 21,000
3021-1661	10	80	10	$ 15,000
3020-3021	10	90	9	$ 18,000
3019-3020	10	70	9	$ 15,000
3018-3019	10	180	10	$ 37,000
2541-3018	10	160	10	$ 33,000
3017-2541	10	50	11	$ 10,000
			Subtotal	$ 447,000
			Contingency (20%)	$ 90,000
			Engineering	$ 81,000
			OPCC	**$ 617,000**

Project 5 Aerial



Project 6: Sewer Group A Line Improvements

Project Identification		
Number:	6	
Location:	West	
Flexibility:	Low	
Schedule		
Primary Trigger:	Capacity	
Secondary Trigger:	None	
Trigger Date:	Oct 2022	
Project Complete:	Feb 2024	
Project Implementation (Months)		
Engineering/Design:	7	
Bid/Construction:	9	
Total Project Duration:	**16**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.55	$0.70
Engineering	$0.08	$0.10
OPCC	**$0.63**	**$0.80**

Project Description

The project involves the replacement of multiple 8" sanitary sewer line segments with a combination of 10" and 12" pipeline. This project is critical, as most pipelines in this line segment exceed 100% of capacity, and do not have minimal required slopes. This project assumes existing manholes will be replaced, and new PVC pipeline will be trenched.

Justification

This section of the collection system is undersized, displays capacity issues in the model, and does not meet minimum slope requirements. Additionally, upstream capacity will be improved with the implementation of these improvements.

Unintended Consequences

The proposed line is through highly residential neighborhoods, and may impact homeowners. Additional effort may be needed to coordinate with the pipeline replacement.

Special Considerations

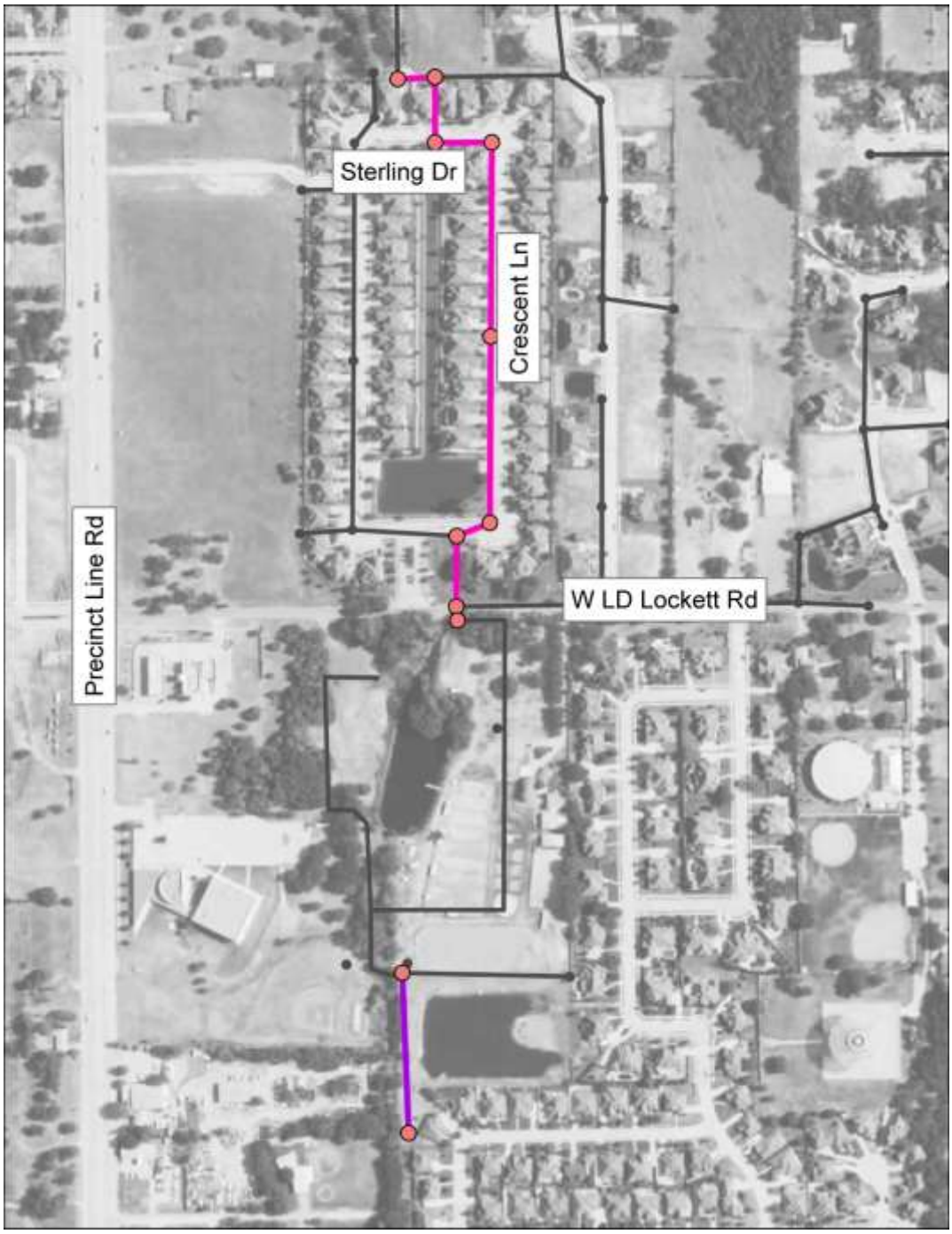
Limited impact through the residential area must be a priority. Pipe inverts were missing for some of this group. Therefore, some pipes slopes were assumed and must be verified during detailed design. Additionally, future buildout was applied and projected increased flow to this basin due to a new development and lift station. If that development is not constructed, this project reduces in priority.

Potential Alternatives

Alternatives would be to require reduction of existing flow through this pipeline segment, which would require a parallel or separate line. This is not considered feasible. Therefore, no potential alternatives exist.

Group A				
Label	**Diameter (in.)**	**Length (ft.)**	**Avg. Depth (ft.)**	**Pipe Cost**
2403-2402	12	420	13	$ 92,000
1480-1677	12	40	12	$ 8,000
2423-1480	10	190	12	$ 40,000
2425-2423	10	100	11	$ 20,000
2939-2425	10	490	12	$ 104,000
2426-2939	10	510	12	$ 108,000
2427-2426	10	150	11	$ 32,000
2430-2427	10	180	11	$ 37,000
2419-2430	10	100	10	$ 21,000
			Subtotal	$ 459,000
			Contingency (20%)	$ 92,000
			Engineering	$ 83,000
			2015 OPCC	**$ 633,000**

Project 6 Aerial



Project 7: Sewer Group T Line Improvements

Project Description

The project consists of pipelines identified for improvements to remove dips and bellies in the lines to alleviate issues with grease clogging. Improvements are for 8-inch pipes along the interceptor in the vicinity of Saddlebrook Drive. Final determination of pipe sizes for this group of improvements should be made during the design phase for the project.

Justification

Implementation of this project will alleviate sags and areas where grease accumulates within the existing pipelines.

Unintended Consequences

None identified.

Special Considerations

These pipes were not surveyed or included in the flow monitoring study. Therefore additional flow studies and survey should be taken during design to confirm pipe sizing.

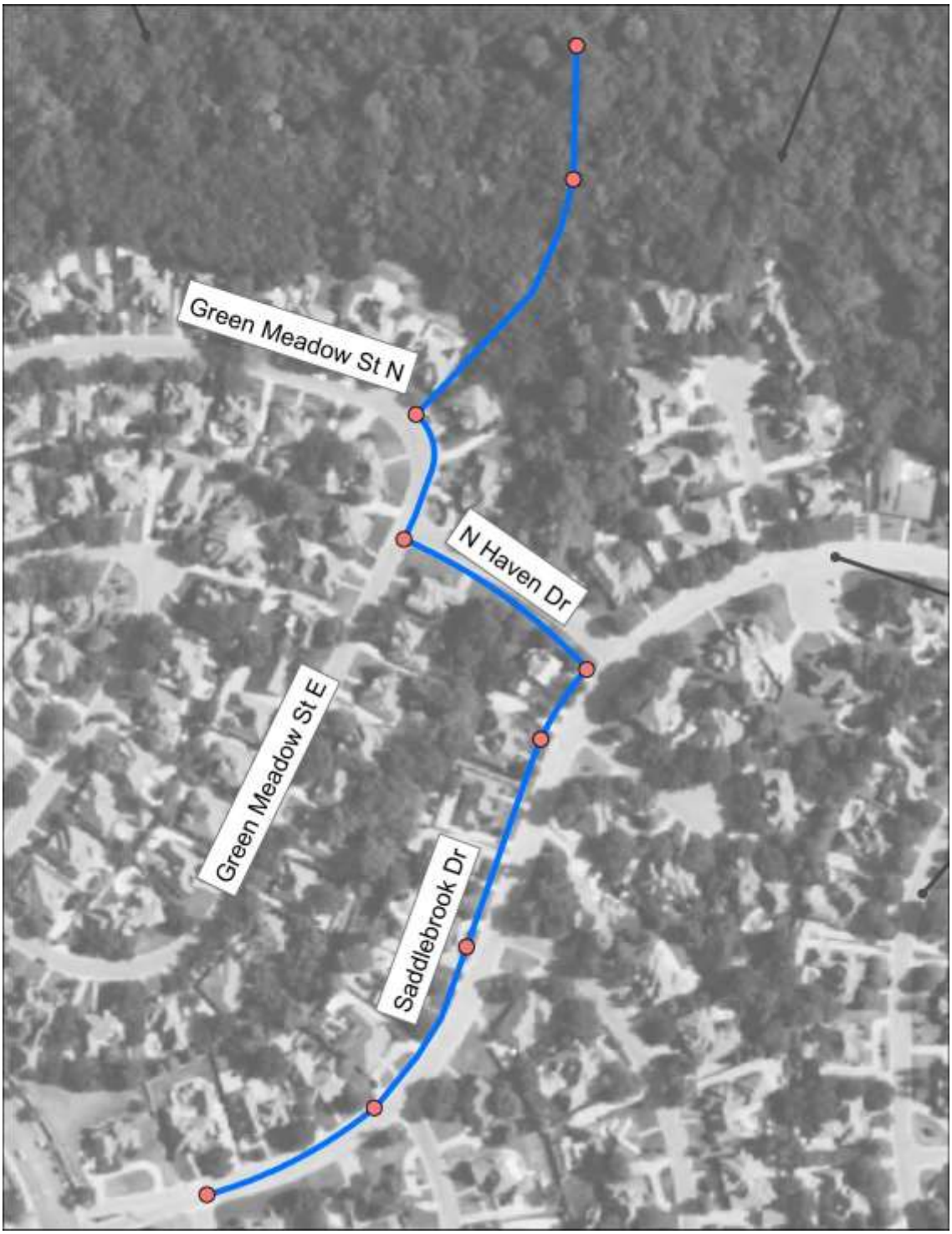
Potential Alternatives

None identified.

Project Identification		
Number:	7	
Location:	South	
Flexibility:	Low	
Schedule		
Primary Trigger:	City-identified	
Secondary Trigger:	Condition	
Trigger Date:	Oct 2022	
Project Complete:	Feb 2024	
Project Implementation (Months)		
Engineering/Design:	7	
Bid/Construction:	9	
Total Project Duration:	**16**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.54	$0.69
Engineering	$0.08	$0.10
OPCC	**$0.62**	**$0.79**

Group T				
Label	**Diameter (in.)**	**Length (ft.)**	**Avg. US & DS Depth (ft.)**	**Pipe Cost**
1310-1940	8	200	<20' assumed	$ 42,000
1309-1310	8	440	<20' assumed	$ 89,000
1299-1309	8	210	<20' assumed	$ 42,000
1298-1299	8	340	<20' assumed	$ 70,000
1297-1298	8	130	<20' assumed	$ 26,000
1296-1297	8	330	<20' assumed	$ 68,000
1295-1296	8	290	<20' assumed	$ 58,000
1291-1295	8	290	<20' assumed	$ 59,000
			Subtotal	$ 452,000
			Contingency (20%)	$ 91,000
			Engineering	$ 82,000
			OPCC	**$ 624,000**

Project 7 Aerials



Project 8: Sewer Group G Line Improvements

Project Description

The project includes replacement of 1,113 LF of 8-inch pipe with 10-inch pipe and associated manholes along Glade Road and Camelot Drive, due to modeled surcharge and near-full conditions.

Justification

Several of the modeled pipe sections do not meet minimum slope, and the model shows manhole surcharge during simulation. The proposed project will increase capacity by increasing pipe slope and pipe diameter.

Unintended Consequences

None identified.

Special Considerations

Special consideration should be given to landscaping to limit the impact on trees during detailed design.

Potential Alternatives

None identified.

Project Identification		
Number:	8	
Location:	East	
Flexibility:	Low	
Schedule		
Primary Trigger:	Capacity	
Secondary Trigger:	None	
Trigger Date:	Oct 2022	
Project Complete:	Feb 2024	
Project Implementation (Months)		
Engineering/Design:	7	
Bid/Construction:	9	
Total Project Duration:	**16**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.28	$0.36
Engineering	$0.04	$0.05
OPCC	**$0.33**	**$0.41**

Group G				
Label	**Diameter (in.)**	**Length (ft.)**	**Avg. US & DS Depth (ft.)**	**Pipe Cost**
1836-1835	10	130	13	$ 26,000
1837-1836	10	170	12	$ 35,000
596-1837	10	180	13	$ 37,000
581-575	10	360	13	$ 75,000
1829-581	10	310	12	$ 65,000
			Subtotal	$ 236,000
			Contingency (20%)	$ 48,000
			Engineering	$ 43,000
			2015 OPCC	**$ 326,000**

Project 8 Aerial

Project 9: Sewer Group U Improvements

Project Description

The project anticipates a renewal and replacement program in place for the City, to adequately prepare for future condition concerns of existing pipe. This project addresses a second of four phases for those renewal and replacements.

Justification

The City has identified 23,190 LF of pipe to be replaced due to condition concerns. This represents 1% of the City's total pipeline inventory, and it is anticipated that this addresses needs that have developed in the previous 5 years. Therefore, this level of renewal and replacement is expected to occur on a 5-year basis. The first group of renewal and replacement is represented in the previously identified projects with condition triggers. This second phase identifies the same percentage of replacement, distributed by pipe size according to the City's current inventory.

Unintended Consequences

This project may not be implemented if condition concerns are not identified. It is recommended that if no condition concerns are present, an investigative technique such as CCTV or smoke testing be conducted at 5 year intervals to ensure the system is maintained and limited on RDII.

Project Identification		
Number:	9	
Location:	City-wide	
Flexibility:	High	
Schedule		
Primary Trigger:	Condition	
Secondary Trigger:	None	
Trigger Date:	Oct 2022	
Project Complete:	April 2025	
Project Implementation (Months)		
Engineering/Design:	15	
Bid/Construction:	15	
Total Project Duration:	**30**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$1.96	$2.56
Engineering	$0.30	$0.37
OPCC	**$2.26**	**$2.94**

Special Considerations

Consideration should be given to the location and scope of this project. Additionally, condition concerns that appear between the five year milestones may need immediate attention.

Potential Alternatives

None identified.

Group U			
Diameter (in.)	**Length (ft.)**	**Avg. US & DS Depth (ft.)**	**Pipe Cost**
6	5350	Assumed <20'	$ 1,083,000
8	2130	Assumed <20'	$ 439,000
10	370	Assumed <20'	$ 79,000
12	170	Assumed <20'	$ 38,000
		Subtotal	$ 1,637,000
		Contingency (20%)	$ 328,000
		Engineering	$ 295,000
		OPCC	**$ 2,259,000**

Project 10: Sewer Group P Improvements

Project Identification	
Number:	10
Location:	City-wide
Flexibility:	Medium
Schedule	
Primary Trigger:	Condition
Secondary Trigger:	None
Trigger Date:	Oct 2022
Project Complete:	Feb 2024
Project Implementation (Months)	
Engineering/Design:	7
Bid/Construction:	9
Total Project Duration:	**16**

	2015 Costs ($ Millions)	Forecasted Costs ($ Millions)
Construction	$0.49	$0.62
Engineering	$0.07	$0.09
OPCC	**$0.56**	**$0.71**

Project Description

The project involves the rehabilitation of Priority 2 manholes throughout the City identified in the manhole assessment. The assessment was conducted in conjunction with the formation of this Master Plan.
Methods for repair will vary depending on the type and severity of the defect. Any manholes identified for improvements but included in groups within the first five years of this CIP have been removed from this group.

Justification

Through the manhole assessment study, it was determined that these manholes presented severe defects that, if left untreated, could eventually contribute to manhole failure. Additionally, infiltration was witnessed during the inspections and would be lessened by incorporating these improvements. It is intended that the base wastewater flow would be decreased as the inflow and infiltration into the system is improved, and the manhole useful life would be extended by these repairs.

Unintended Consequences

Reducing infiltration in certain areas may cause groundwater tables to rise slightly, however this is not expected to be significant.

Special Considerations

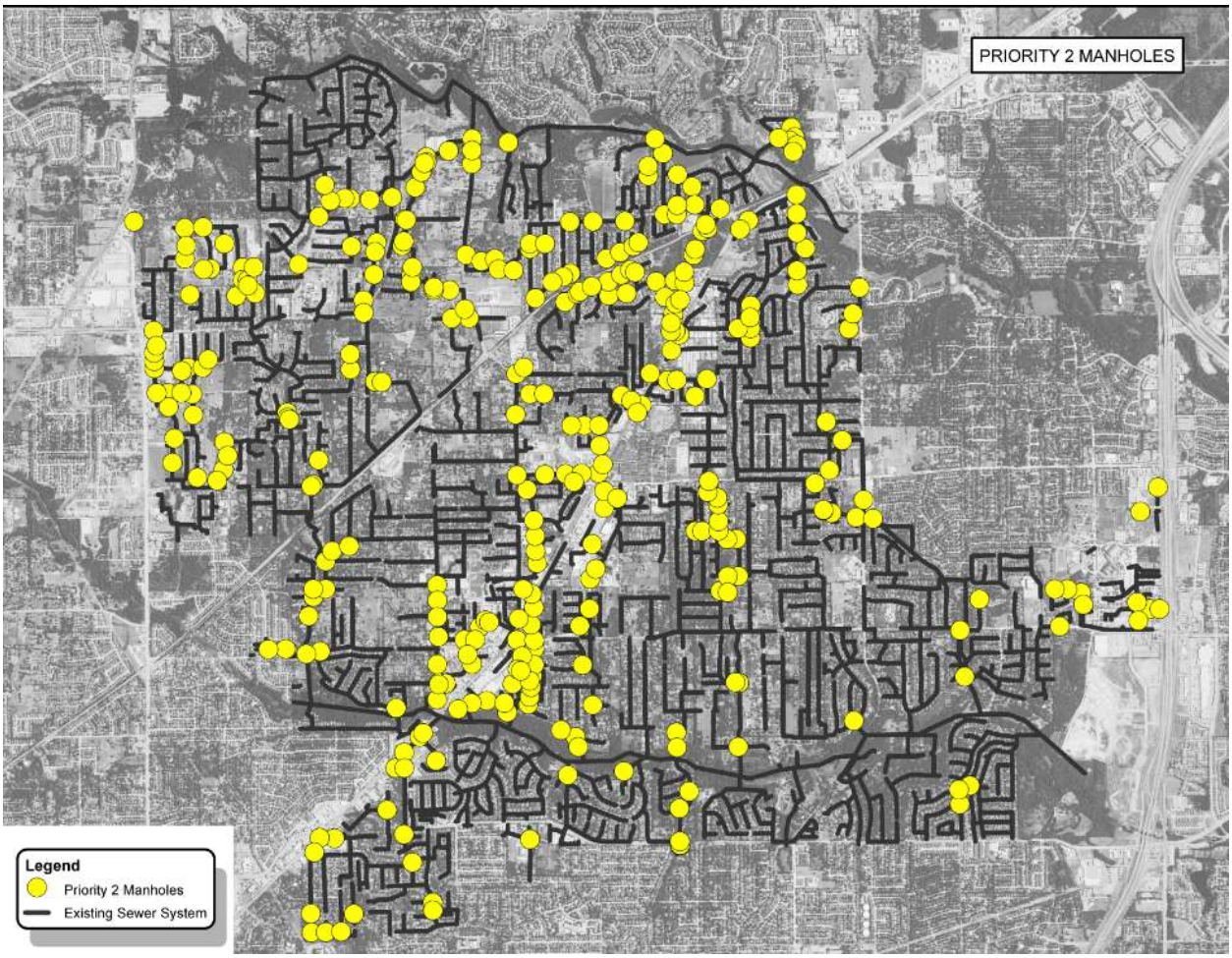
Consideration should be given to the type of rehabilitation technology employed. Waterproofing manholes in high groundwater tables may have a tendency to cause the manholes to float, and some manhole rehabilitation types would prevent future modifications to the manhole inverts (thereby requiring full manhole replacement if future lines are added or modified).

Potential Alternatives

While various alternatives exist for each of the recommended rehabilitation methods, the only alternative to manhole rehabilitation in most cases is total replacement, which is anticipated to be more expensive.

Group P	
Label	**Total Cost**
Priority 2 MH Improvements	$ 406,000
Subtotal	$ 406,000
Contingency (20%)	$ 82,000
Engineering	$ 74,000
2015 OPCC	**$ 560,000**

Project 10 Aerial



Priority 2 Assessed Manhole locations Throughout City

Project 11: Sewer Group Q Improvements

Project Description

The project involves the assessment of the remaining manholes in the City's wastewater collection system to identify potential issues and categorize two priorities of improvements needed to defective manholes.

Justification

The original assessment was conducted on 1,134 of the City's 3,029 total manholes, leaving 1,895 left to be assessed. Though these manholes may have been on smaller line sizes, the manhole will likely contribute similar rates of I/I, regardless of line size. In addition, the report determined that infiltration may be occurring from 0-13% throughout the system. Therefore, it is critical that the remaining manholes be assessed.

Unintended Consequences

None identified.

Special Considerations

Access to manholes on private landowners will be a key coordination item that was not coordinated with the previous assessment.

Potential Alternatives

None identified.

Project Identification		
Number:	11	
Location:	City-wide	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Condition	
Secondary Trigger:	None	
Trigger Date:	Oct 2022	
Project Complete:	May 2023	
Project Implementation (Months)		
Engineering/Design:	7	
Bid/Construction:	0	
Total Project Duration:	**7**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$ -	$ -
Engineering	$0.22	$0.28
OPCC	**$0.22**	**$0.28**

Group Q	
Label	**Total Cost**
Inspect Remaining MH	$ 182,000
Subtotal	$ 182,000
Contingency (20%)	$ 37,000
2015 OPCC	**$ 219,000**

Project 12: Sewer Group R Improvements

Project Identification		
Number:	12	
Location:	City-wide	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Condition	
Secondary Trigger:	None	
Trigger Date:	Oct 2023	
Project Complete:	Nov 2024	
Project Implementation (Months)		
Engineering/Design:	7	
Bid/Construction:	6	
Total Project Duration:	**13**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.28	$0.36
Engineering	$0.04	$0.05
OPCC	**$0.32**	**$0.42**

Project Description

The project involves the rehabilitation of the highest priority manholes throughout the City that are anticipated to be identified in the manhole assessment from Project 11. It has been projected from the assessed manholes Priority 1 ratios that approximately 142 manholes will be included in this project.

Justification

Through the previous manhole assessment study, it was determined that these type of severe defects, if left untreated, could eventually contribute to manhole failure. Additionally, infiltration is expected to be witnessed during the inspections and would be lessened by incorporating these improvements. It is intended that the base wastewater flow would be decreased as the inflow and infiltration into the system is improved, and the manhole useful life would be extended by these repairs.

Unintended Consequences

Reducing infiltration in certain areas may cause groundwater tables to rise slightly, however this is not expected to be significant.

Special Considerations

Consideration should be given to the type of rehabilitation technology employed. Waterproofing manholes in high groundwater tables may have a tendency to cause the manholes to float, and some manhole rehabilitation types would prevent future modifications to the manhole inverts (thereby requiring full manhole replacement if future lines are added or modified).

Potential Alternatives

While various alternatives exist for each of the recommended rehabilitation methods, the only alternative to manhole rehabilitation in most cases is total replacement, which is anticipated to be more expensive.

Group R	
Label	**Total Cost**
High Priority Unidentified MH Improvements	$ 231,000
Subtotal	$ 231,000
Contingency (20%)	$ 47,000
Engineering	$ 42,000
2015 OPCC	**$ 319,000**

Project 13: Sewer Group E Line Improvements

Project Description

The project includes 3,488 LF of 10-inch PVC pipe and manholes to replace a combination of 8-inch and 10-inch existing pipeline. The existing pipe segments are consistently near surcharge during the design flow rates and approximately half do not meet minimum slope requirements.

Justification

The lack of minimum slope and pipe sizing combines to approach surcharge during modeled conditions. Implementing these improvements will reduce the chance that surcharge will occur, and will more correctly size and install the pipe so that the pipes will properly self-scour.

Unintended Consequences

None identified.

Special Considerations

None identified.

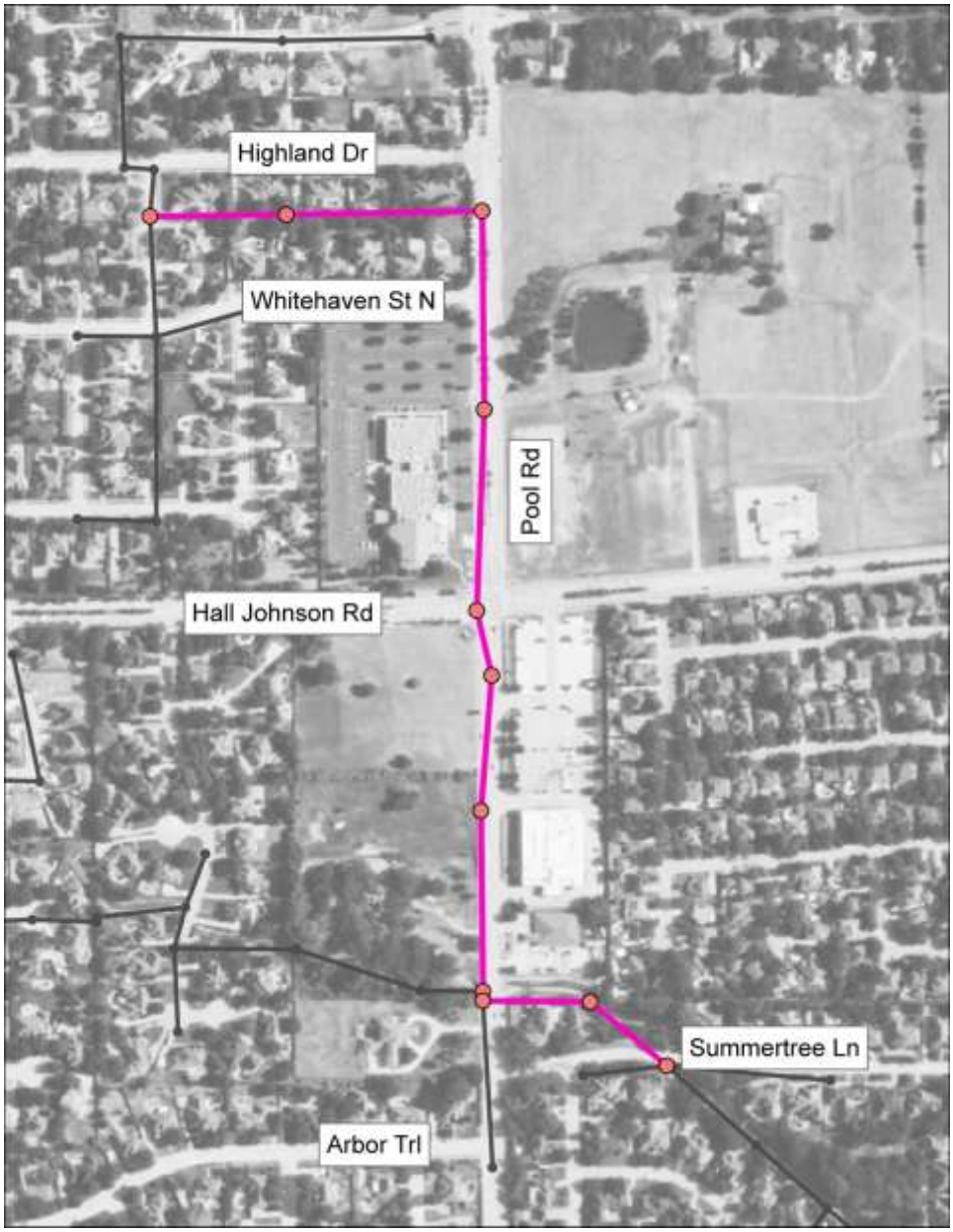
Potential Alternatives

None identified.

Project Identification		
Number:	13	
Location:	East	
Flexibility:	Medium	
Schedule		
Primary Trigger:	Capacity	
Secondary Trigger:	None	
Trigger Date:	Oct 2024	
Project Complete:	Aug 2026	
Project Implementation (Months)		
Engineering/Design:	9	
Bid/Construction:	13	
Total Project Duration:	**22**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.89	$1.23
Engineering	$0.13	$0.18
OPCC	**$1.02**	**$1.41**

Group E				
Label	**Diameter (in.)**	**Length (ft.)**	**Avg. US & DS Depth (ft.)**	**Pipe Cost**
817-815	10	270	10	$ 56,000
459-817	10	290	11	$ 60,000
458-459	10	30	13	$ 6,000
1004-458	10	480	14	$ 101,000
1005-1004	10	360	15	$ 76,000
1006-1005	10	180	18	$ 37,000
1008-1006	10	530	17	$ 112,000
1009-1008	10	530	17	$ 111,000
1786-1009	10	520	17	$ 109,000
1785-1786	10	360	15	$ 76,000
			Subtotal	$ 740,000
			Contingency (20%)	$ 148,000
			Engineering	$ 134,000
			2015 OPCC	**$ 1,020,000**

Project 13 Aerial



Project 14: Sewer Group V Improvements

Project Identification	
Number:	14
Location:	City-wide
Flexibility:	High
Schedule	
Primary Trigger:	Condition
Secondary Trigger:	None
Trigger Date:	Oct 2026
Project Complete:	Apr 2029
Project Implementation (Months)	
Engineering/Design:	15
Bid/Construction:	15
Total Project Duration:	**30**

	2015 Costs ($ Millions)	Forecasted Costs ($ Millions)
Construction	$1.96	$2.88
Engineering	$0.30	$0.42
OPCC	**$2.26**	**$3.30**

Project Description

The project anticipates a renewal and replacement program in place for the City, to adequately prepare for future condition concerns of existing pipe. This project addresses a third of four phases for those renewal and replacements.

Justification

The City has identified 23,190 LF of pipe to be replaced due to condition concerns. This represents 1% of the City's total pipeline inventory, and it is anticipated that this addresses needs that have developed in the previous 5 years. Therefore, this level of renewal and replacement is expected to occur on a 5-year basis. The first group of renewal and replacement is represented in the previously identified projects with condition triggers. This third phase identifies the same percentage of replacement, distributed by pipe size according to the City's current inventory.

Unintended Consequences

This project may not be implemented if condition concerns are not identified. It is recommended that if no condition concerns are present, an investigative technique such as CCTV or smoke testing be conducted at 5 year intervals to ensure the system is maintained and limited on RDII.

Special Considerations

Consideration should be given to the location and scope of this project. Additionally, condition concerns that appear between the five year milestones may need immediate attention.

Potential Alternatives

None identified.

Group V			
Diameter (in.)	**Length (ft.)**	**Avg. US & DS Depth (ft.)**	**Pipe Cost**
6	5350	Assumed <20'	$ 1,083,000
8	2130	Assumed <20'	$ 439,000
10	370	Assumed <20'	$ 79,000
12	170	Assumed <20'	$ 38,000
		Subtotal	$ 1,637,000
		Contingency (20%)	$ 328,000
		Engineering	$ 295,000
		OPCC	**$ 2,259,000**

Project 15: Sewer Group D Line Improvements

Project Identification	
Number:	15
Location:	South
Flexibility:	Medium
Schedule	
Primary Trigger:	Capacity
Secondary Trigger:	None
Trigger Date:	Oct 2026
Project Complete:	Feb 2028
Project Implementation (Months)	
Engineering/Design:	7
Bid/Construction:	9
Total Project Duration:	**16**

	2015 Costs ($ Millions)	Forecasted Costs ($ Millions)
Construction	$0.46	$0.66
Engineering	$0.07	$0.10
OPCC	**$0.53**	**$0.76**

Project Description

The project includes upstream piping improvements to existing 6 and 8-inch that feeds the TRA main interceptor in the areas south of Meandering Way and Shady Creek Lane. Under limited modeling information, the system surcharges due to capacity issues. A combination of 8, 10, and 12-inch pipeline improvements has been recommended.

Justification

The current model raises capacity concerns in this area, which appears to have significant 6 and 8-inch piping. Detailed design is recommended on these line segments to determine the exact elevations.

Unintended Consequences

If the lines are not properly sized according to the anticipated flow, manholes could surcharge under high flow conditions, and pipes could deposit material during low flow conditions.

Special Considerations

Additional survey and study is necessary to verify these improvements are required, as the majority of this sewershed is collected by 6-inch lines, which were not surveyed as part of this Master Plan.

Potential Alternatives

The scope of this project may be increased or decreased, depending on survey and flow study findings for this drainage basin.

Group D				
Label	**Diameter (in.)**	**Length (ft.)**	**Avg. US & DS Depth (ft.)**	**Pipe Cost**
101-150	12	500	10	$ 109,000
106-101	8	320	10	$ 65,000
95-96	10	610	14	$ 128,000
94-95	8	420	14	$ 85,000
			Subtotal	$ 386,000
			Contingency (20%)	$ 78,000
			Engineering	$ 70,000
			2015 OPCC	**$ 532,000**

Project 15 Aerial

Project 16: Sewer Group S Improvements

Project Identification		
Number:	16	
Location:	City-wide	
Flexibility:	High	
Schedule		
Primary Trigger:	Capacity	
Secondary Trigger:	None	
Trigger Date:	Oct 2026	
Project Complete:	June 2028	
Project Implementation (Months)		
Engineering/Design:	7	
Bid/Construction:	13	
Total Project Duration:	**20**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$0.86	$1.23
Engineering	$0.13	$0.19
OPCC	**$0.99**	**$1.42**

Project Description

The project involves the rehabilitation of medium priority manholes throughout the City identified in the manhole assessment from Project 11. It is estimated that 521 manholes will be included in this project, based upon the ratio of Priority 2 identified manholes in the field assessment.

Justification

Through the previous manhole assessment study, it was determined that medium priority manholes present severe defects that, if left untreated, could eventually contribute to manhole failure. Additionally, infiltration is expected to be witnessed during the inspections and would be lessened by incorporating these improvements. It is intended that the base wastewater flow would be decreased as the inflow and infiltration into the system is improved, and the manhole useful life would be extended by these repairs.

Unintended Consequences

Reducing infiltration in certain areas may cause groundwater tables to rise slightly, however this is not expected to be significant.

Special Considerations

Consideration should be given to the type of rehabilitation technology employed. Waterproofing manholes in high groundwater tables may have a tendency to cause the manholes to float, and some manhole rehabilitation types would prevent future modifications to the manhole inverts (thereby requiring full manhole replacement if future lines are added or modified).

Potential Alternatives

While various alternatives exist for each of the recommended rehabilitation methods, the only alternative to manhole rehabilitation in most cases is total replacement, which is anticipated to be more expensive.

Group S	
Label	**Total Cost**
Medium Priority Unidentified MH Improvements	$ 720,000
Subtotal	$ 720,000
Contingency (20%)	$ 144,000
Engineering	$ 130,000
2015 OPCC	**$ 993,000**

Project 17: Sewer Group W Improvements

Project Identification		
Number:	17	
Location:	City-wide	
Flexibility:	High	
Schedule		
Primary Trigger:	Condition	
Secondary Trigger:	None	
Trigger Date:	Oct 2032	
Project Complete:	April 2035	
Project Implementation (Months)		
Engineering/Design:	15	
Bid/Construction:	15	
Total Project Duration:	**30**	
	2015 Costs ($ Millions)	**Forecasted Costs ($ Millions)**
Construction	$1.96	$3.44
Engineering	$0.30	$0.50
OPCC	**$2.26**	**$3.95**

Project Description

The project anticipates a renewal and replacement program in place for the City, to adequately prepare for future condition concerns of existing pipe. This project addresses a fourth of four phases for those renewal and replacements.

Justification

The City has identified 23,190 LF of pipe to be replaced due to condition concerns. This represents 1% of the City's total pipeline inventory, and it is anticipated that this addresses needs that have developed in the previous 5 years. Therefore, this level of renewal and replacement is expected to occur on a 5-year basis. The first group of renewal and replacement is represented in the previously identified projects with condition triggers. This fourth phase identifies the same percentage of replacement, distributed by pipe size according to the City's current inventory.

Unintended Consequences

This project may not be implemented if condition concerns are not identified. It is recommended that if no condition concerns are present, an investigative technique such as CCTV or smoke testing be conducted at 5 year intervals to ensure the system is maintained and limited on RDII.

Special Considerations

Consideration should be given to the location and scope of this project. Additionally, condition concerns that appear between the five year milestones may need immediate attention.

Potential Alternatives

None identified.

Group W			
Diameter (in.)	**Length (ft.)**	**Avg. US & DS Depth (ft.)**	**Pipe Cost**
6	5350	Assumed <20'	$ 1,083,000
8	2130	Assumed <20'	$ 439,000
10	370	Assumed <20'	$ 79,000
12	170	Assumed <20'	$ 38,000
		Subtotal	$ 1,637,000
		Contingency (20%)	$ 328,000
		Engineering	$ 295,000
		OPCC	**$ 2,259,000**

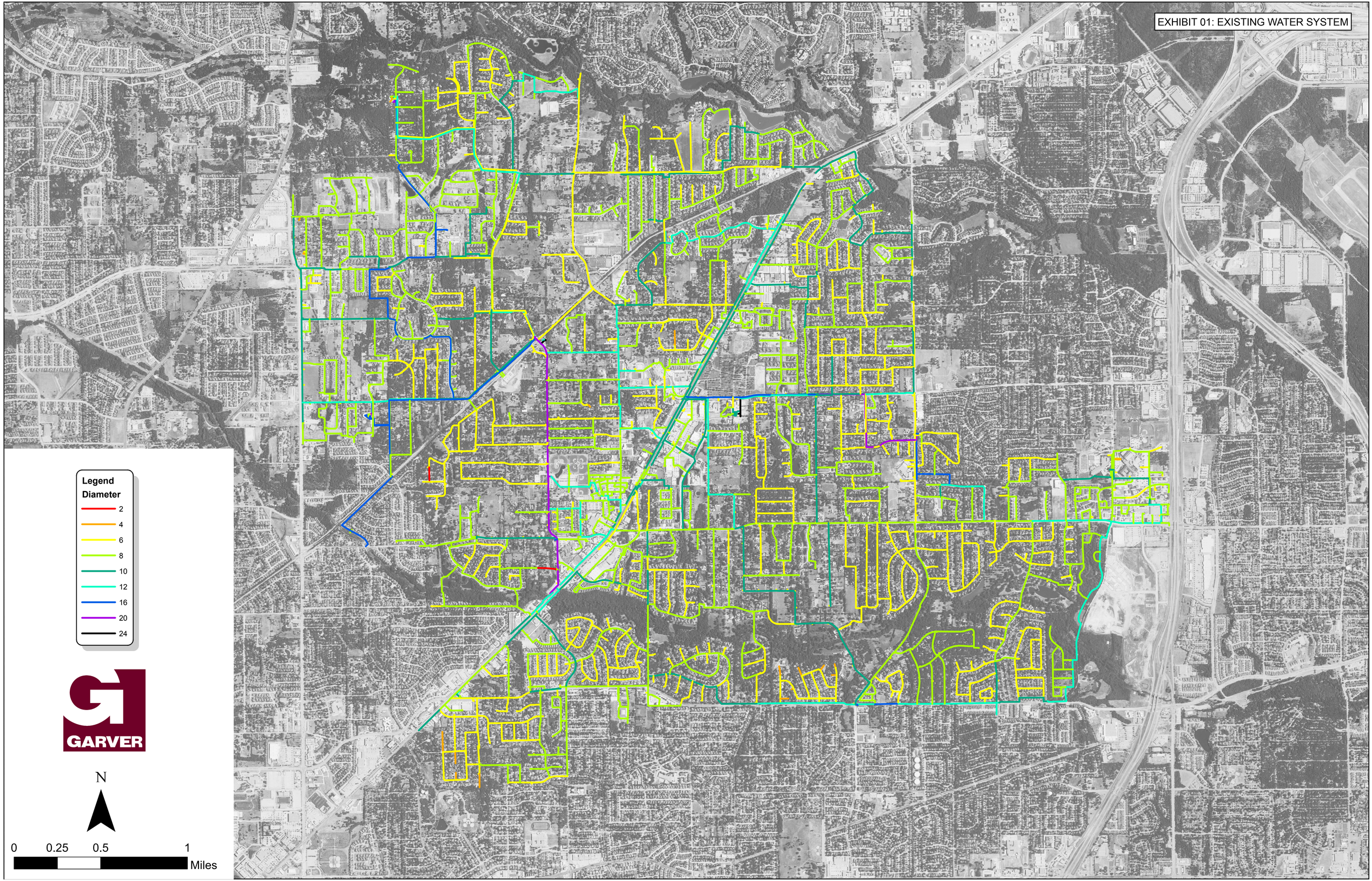


EXHIBIT 01: EXISTING WATER SYSTEM
Legend
Diameter
2
4
6
8
10
12
16
20
24
GARVER
N
0
0.25
0.5
1
Miles

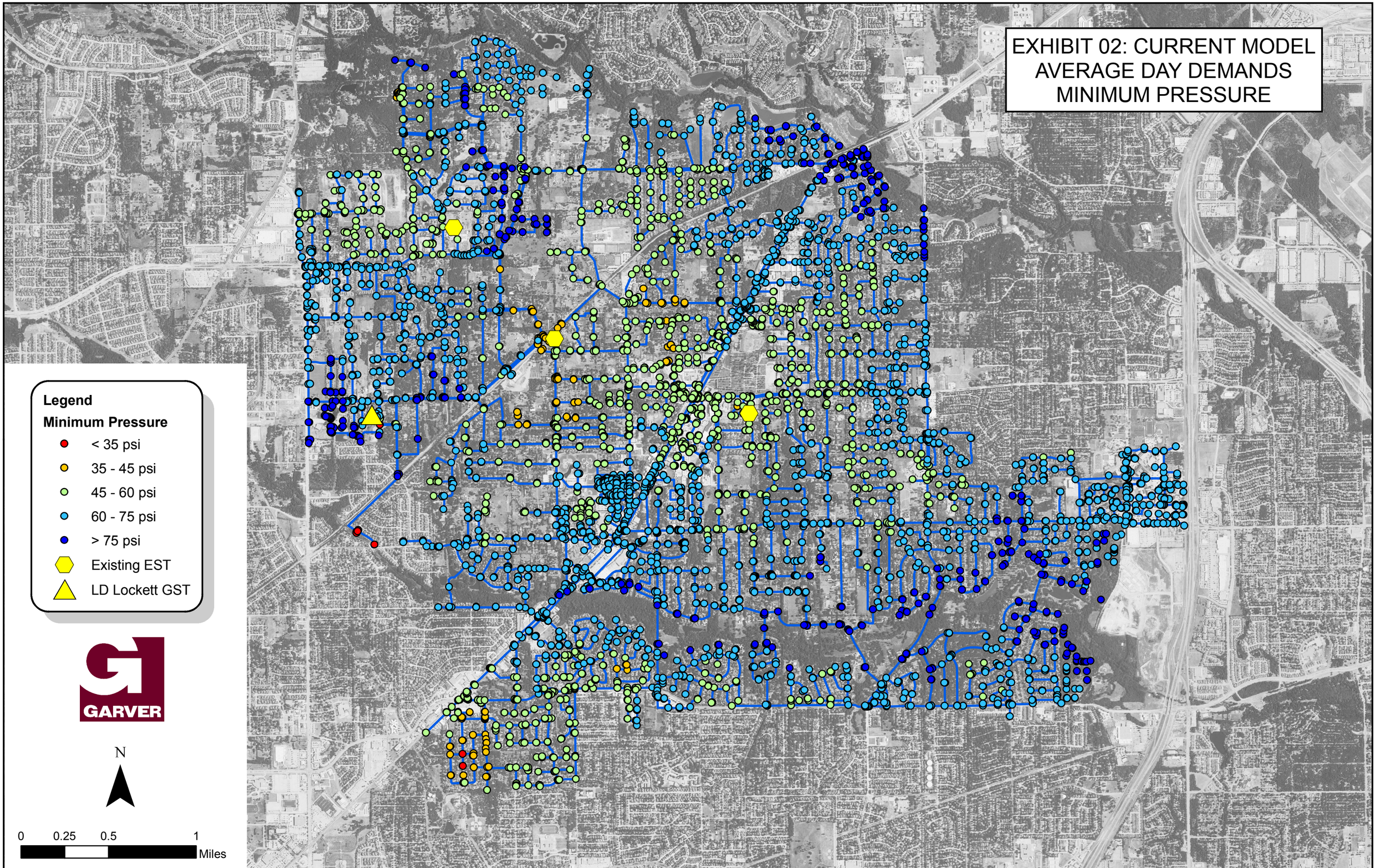


EXHIBIT 02: CURRENT MODEL
AVERAGE DAY DEMANDS
MINIMUM PRESSURE
Legend
Minimum Pressure
< 35 psi
35 - 45 psi
45 - 60 psi
60 - 75 psi
> 75 psi
Existing EST
LD Lockett GST
GARVER
N
0
0.25
0.5
1
Miles

EXHIBIT 03: CURRENT MODEL AVERAGE DAY DEMANDS MAXIMUM VELOCITY

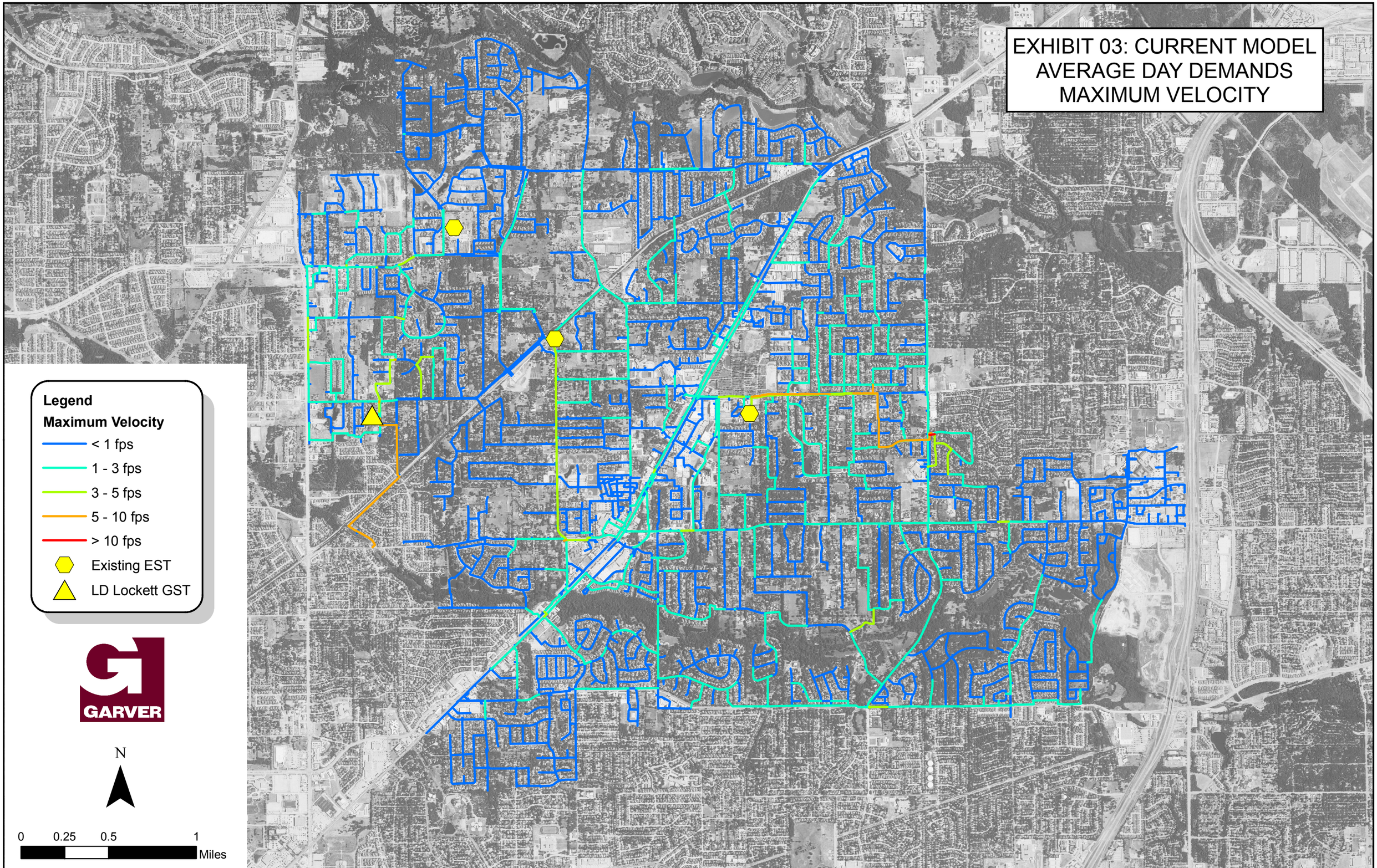
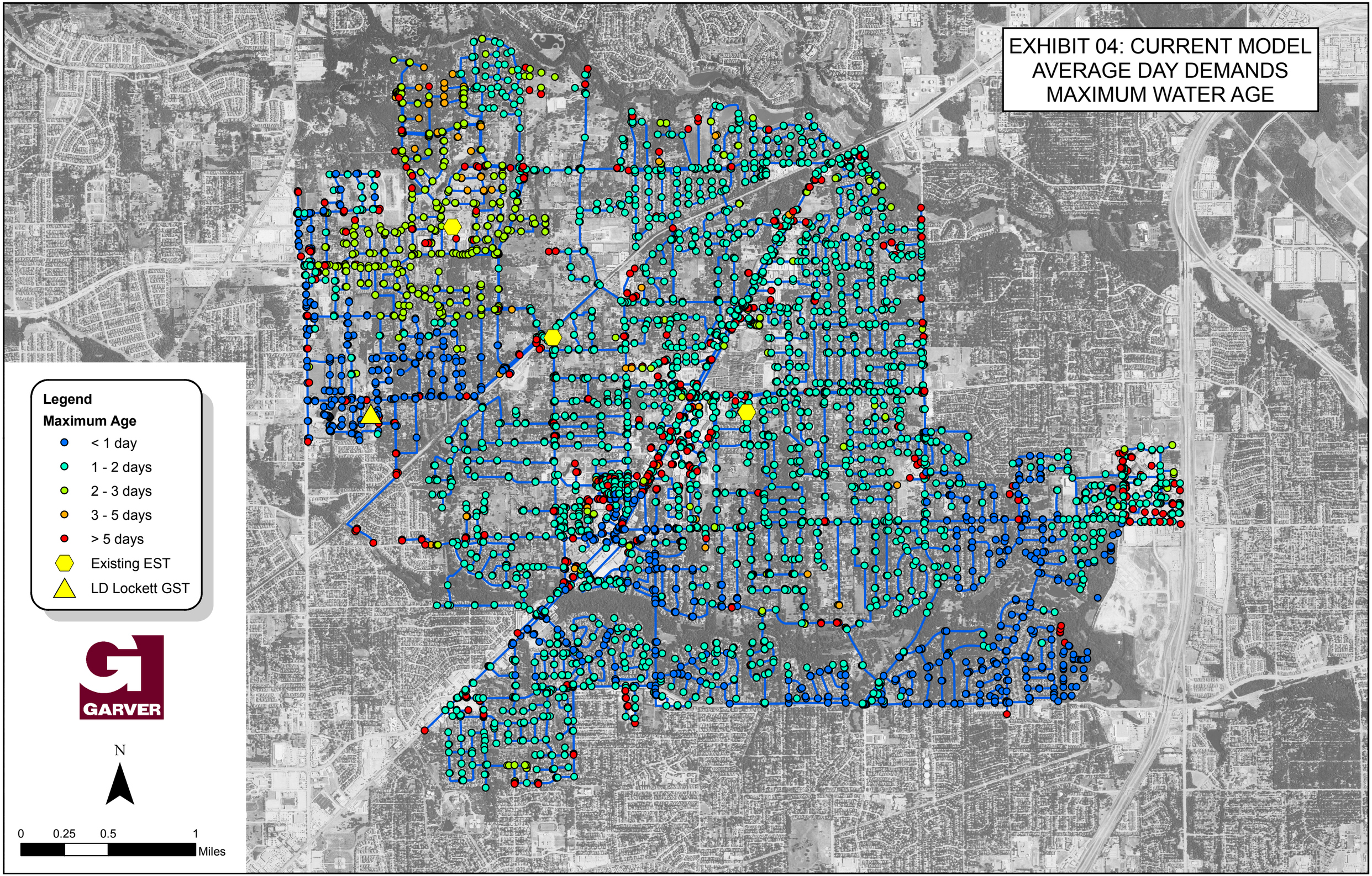


EXHIBIT 04: CURRENT MODEL AVERAGE DAY DEMANDS MAXIMUM WATER AGE



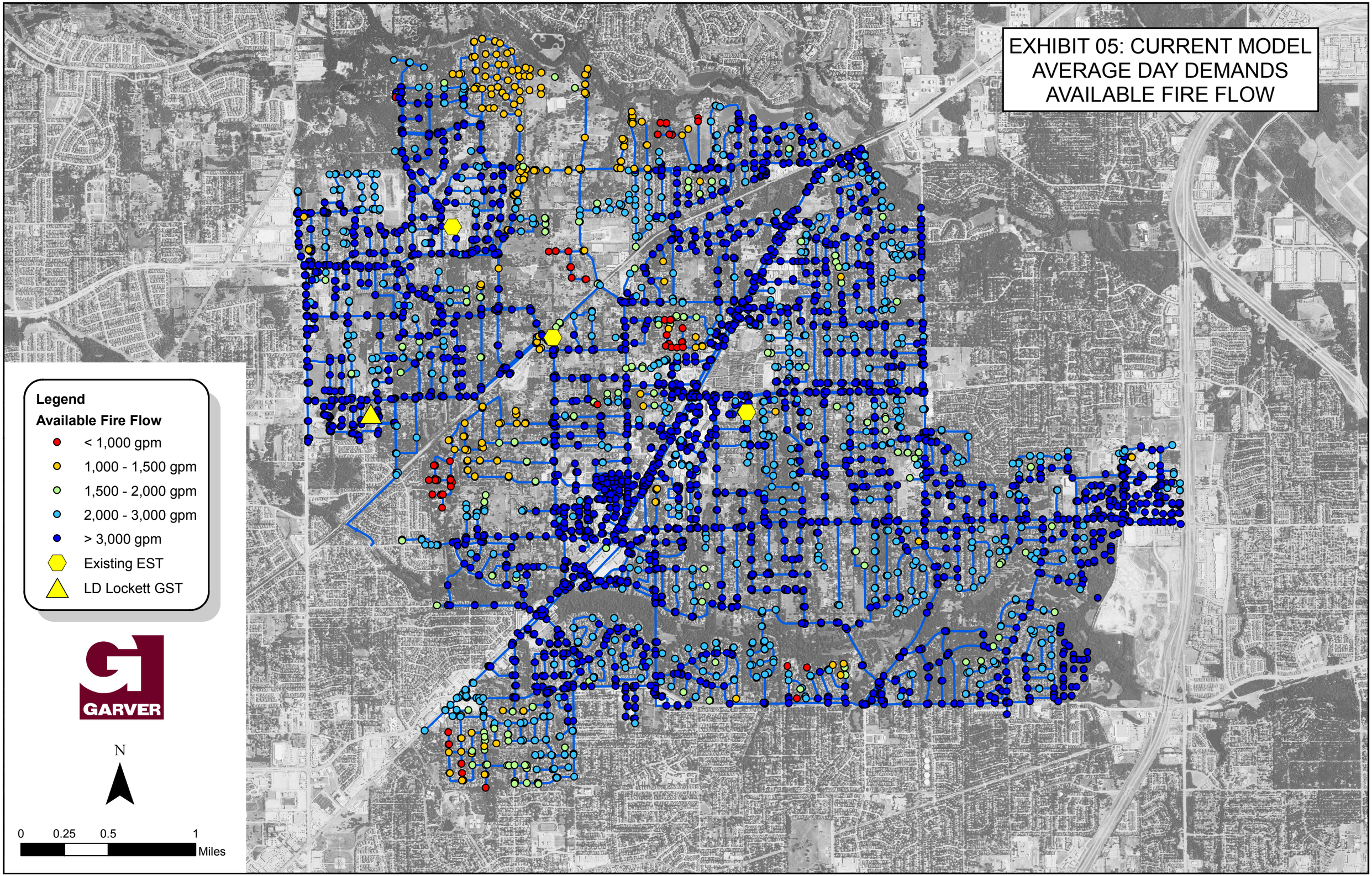


EXHIBIT 05: CURRENT MODEL
AVERAGE DAY DEMANDS
AVAILABLE FIRE FLOW
Legend
Available Fire Flow
< 1,000 gpm
1,000 - 1,500 gpm
1,500 - 2,000 gpm
2,000 - 3,000 gpm
> 3,000 gpm
Existing EST
LD Lockett GST
GARVER
N
0
0.25
0.5
1
Miles

EXHIBIT 06: CURRENT MODEL MAXIMUM DAY DEMANDS MINIMUM PRESSURE

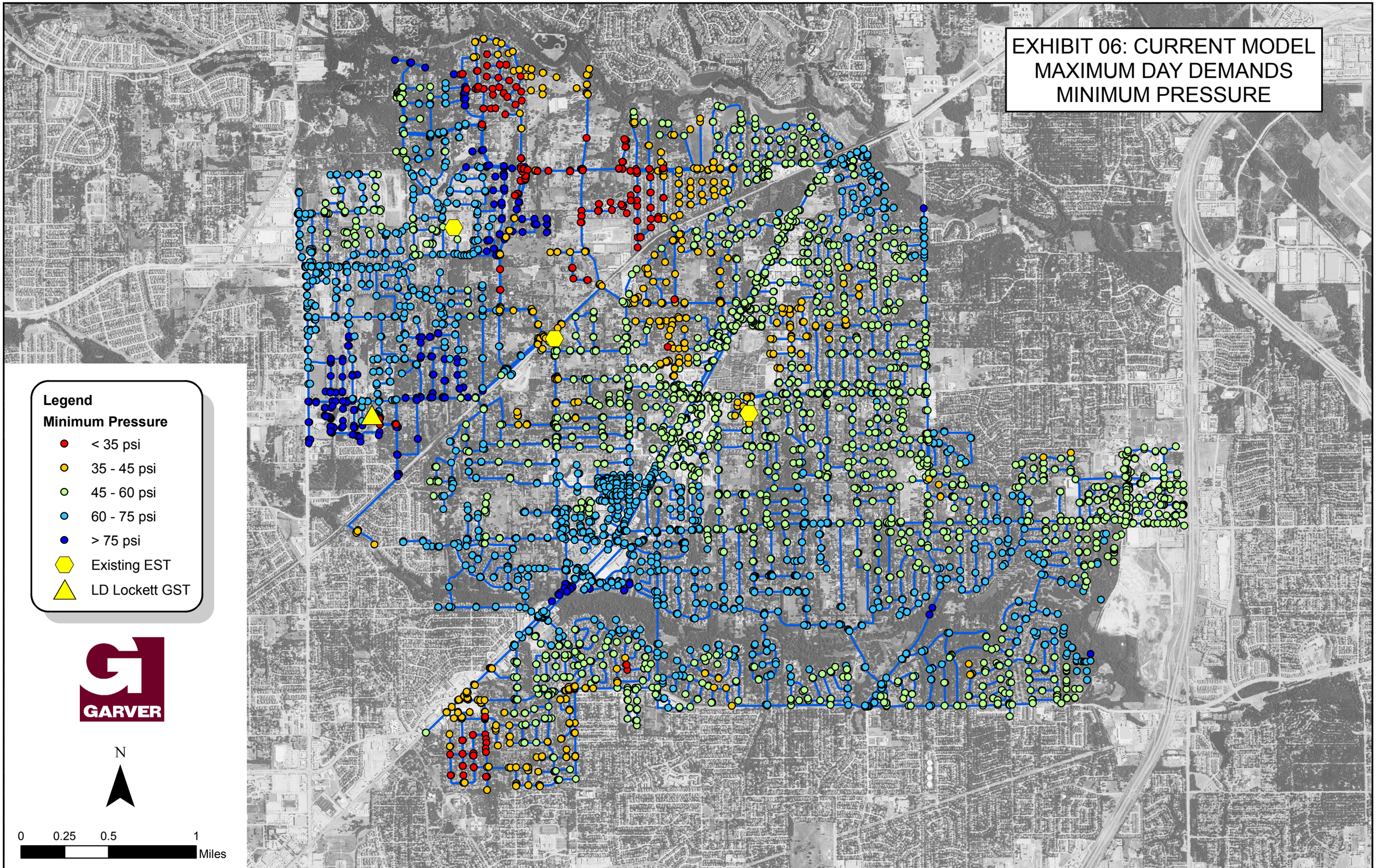
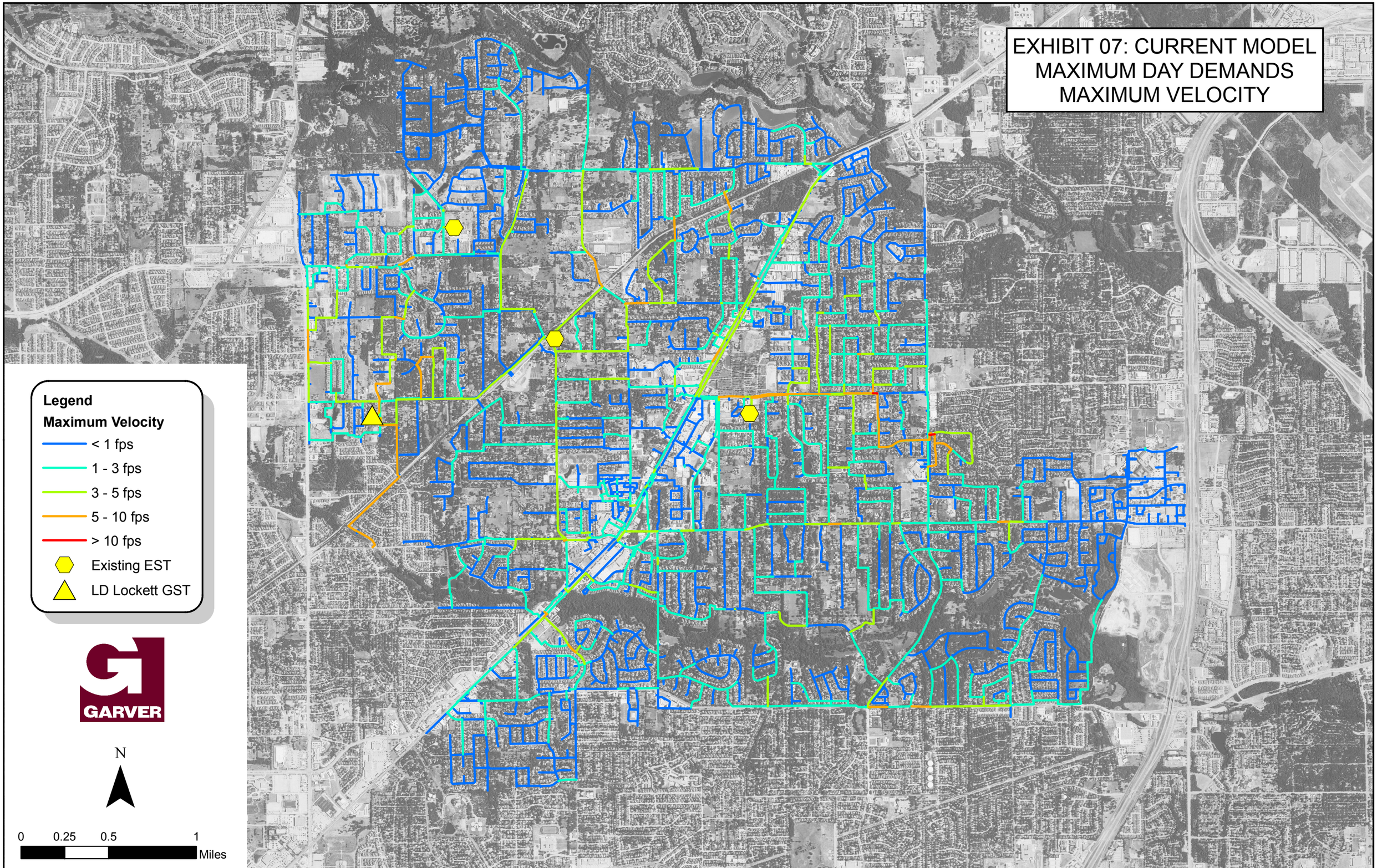


EXHIBIT 07: CURRENT MODEL MAXIMUM DAY DEMANDS MAXIMUM VELOCITY



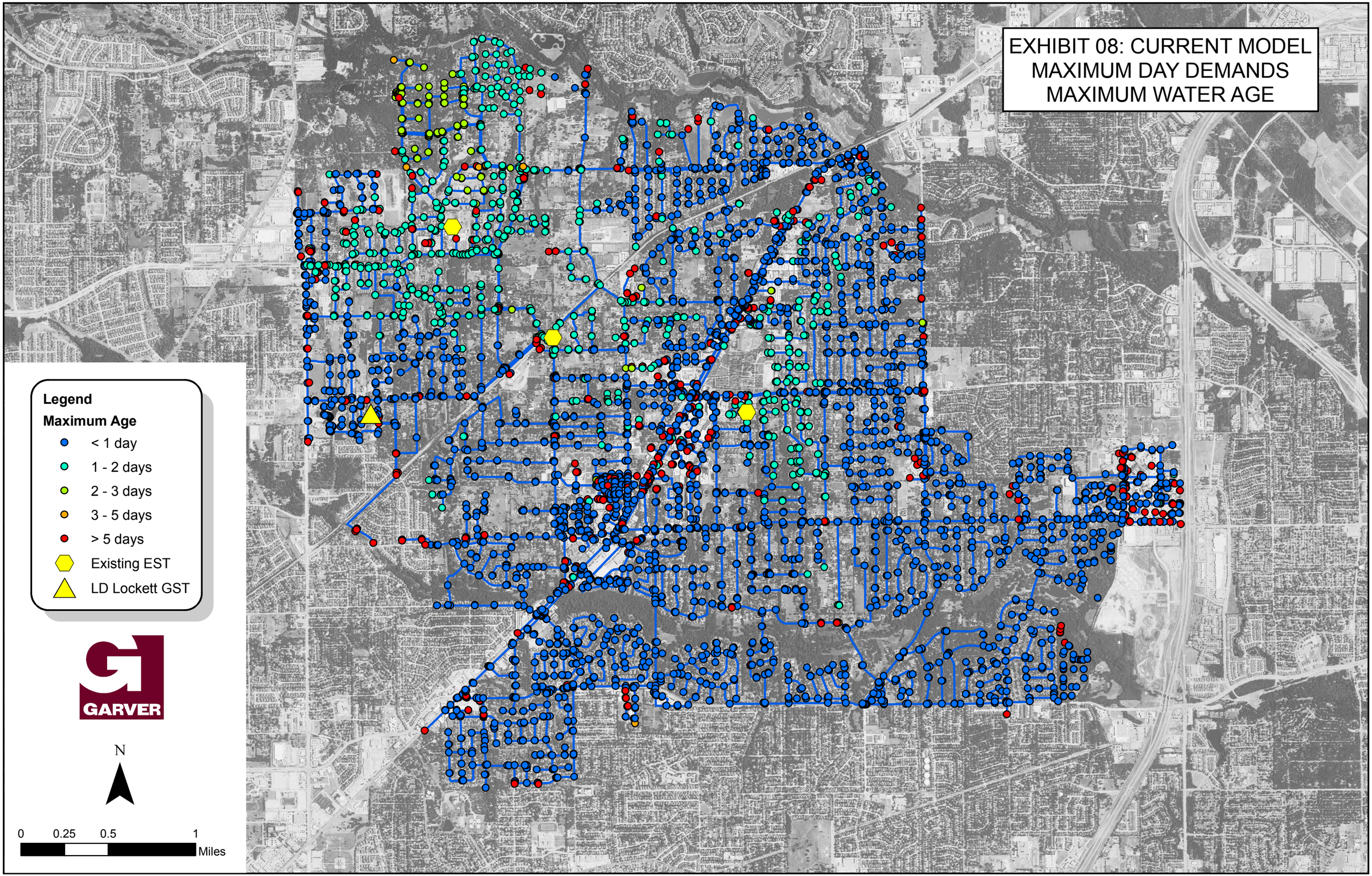
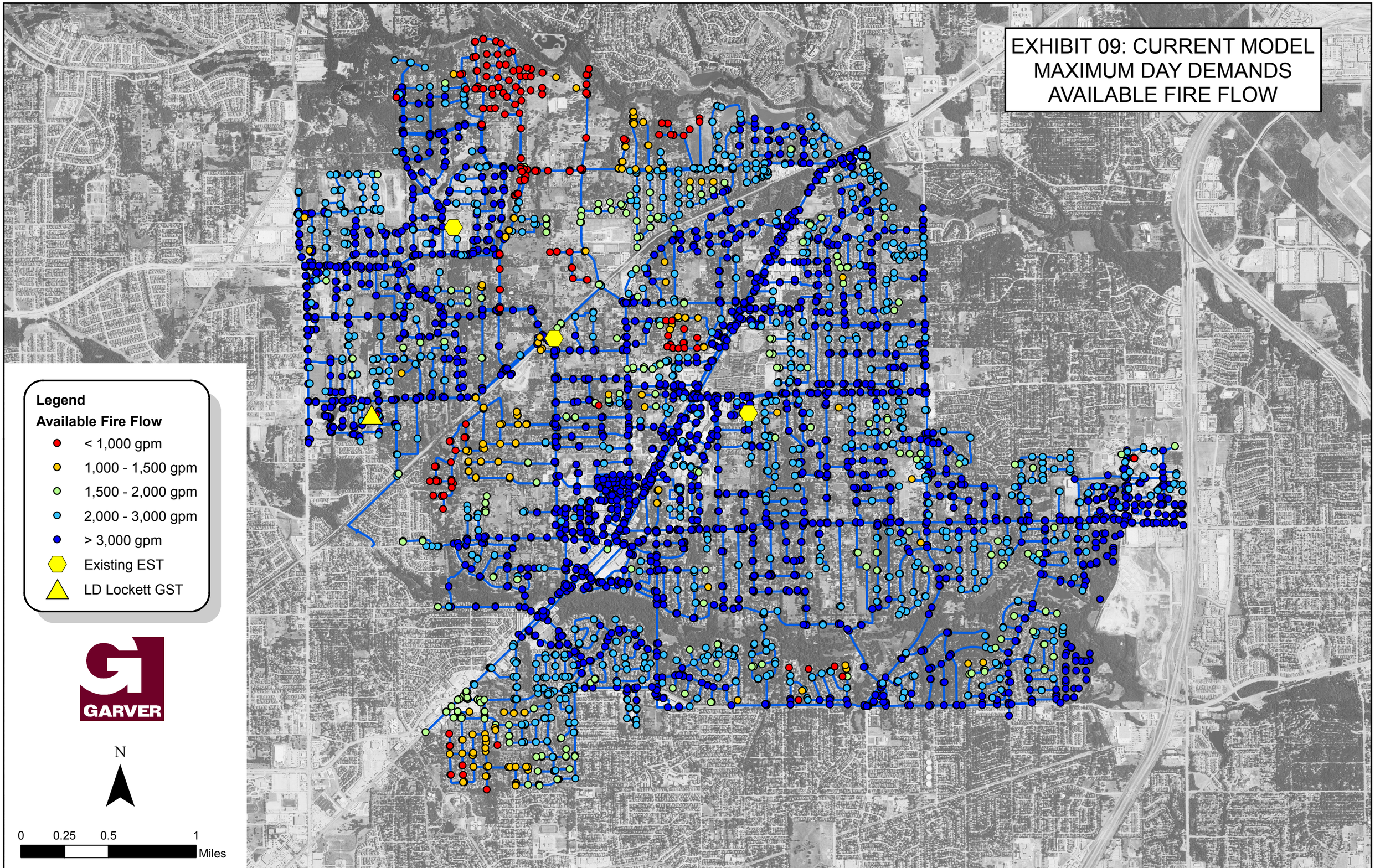


EXHIBIT 08: CURRENT MODEL
MAXIMUM DAY DEMANDS
MAXIMUM WATER AGE
Legend
Maximum Age
< 1 day
1 - 2 days
2 - 3 days
3 - 5 days
> 5 days
Existing EST
LD Lockett GST
GARVER
N
0
0.25
0.5
1
Miles

EXHIBIT 09: CURRENT MODEL MAXIMUM DAY DEMANDS AVAILABLE FIRE FLOW



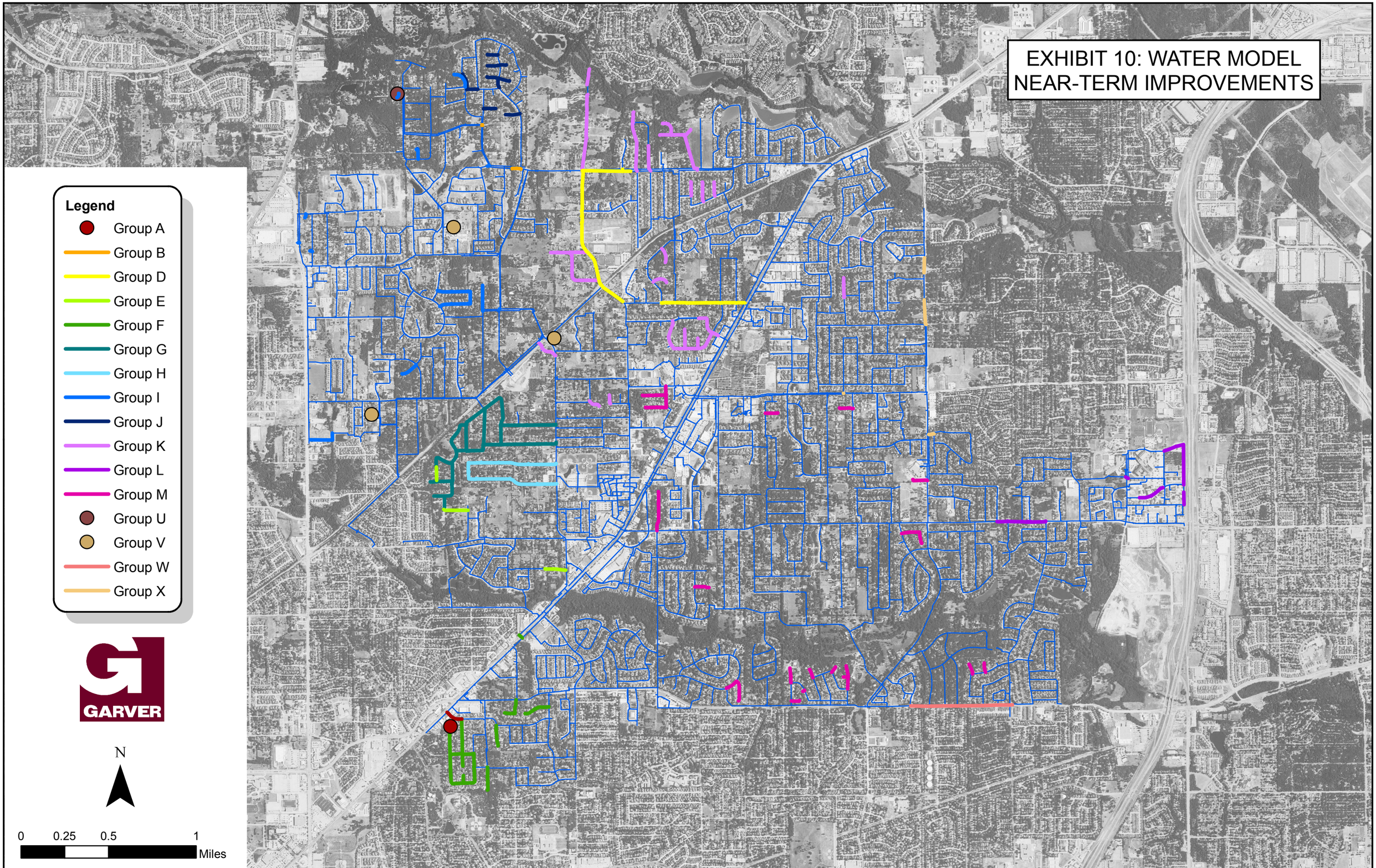


EXHIBIT 10: WATER MODEL
NEAR-TERM IMPROVEMENTS
Legend
Group A
Group B
Group D
Group E
Group F
Group G
Group H
Group I
Group J
Group K
Group L
Group M
Group U
Group V
Group W
Group X
GARVER
N
0
0.25
0.5
1
Miles

EXHIBIT 11: CURRENT MODEL WITH IMPROVEMENTS
MAXIMUM DAY DEMANDS
MINIMUM PRESSURE

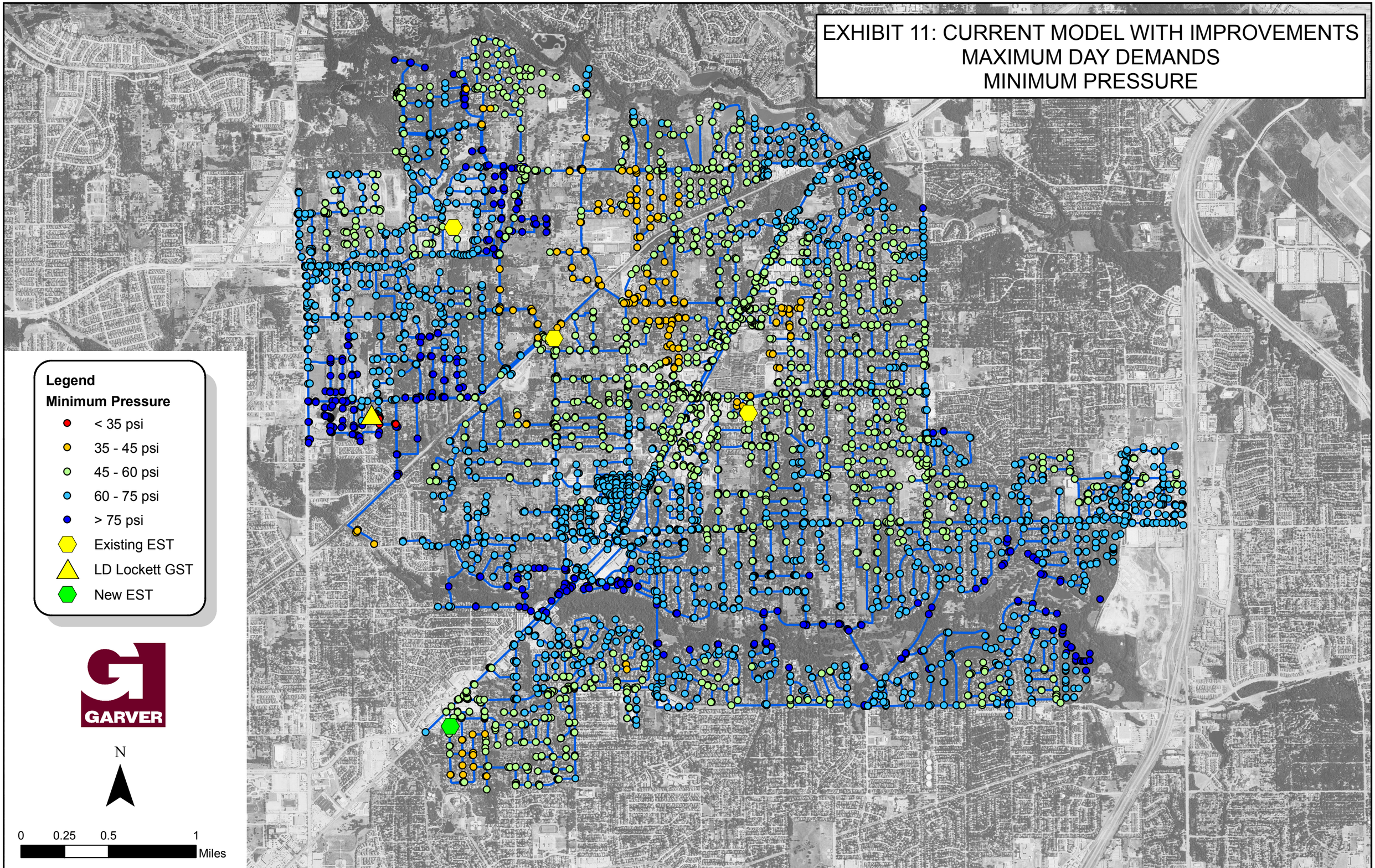


EXHIBIT 12: CURRENT MODEL WITH IMPROVEMENTS
MAXIMUM DAY DEMANDS
MAXIMUM VELOCITY

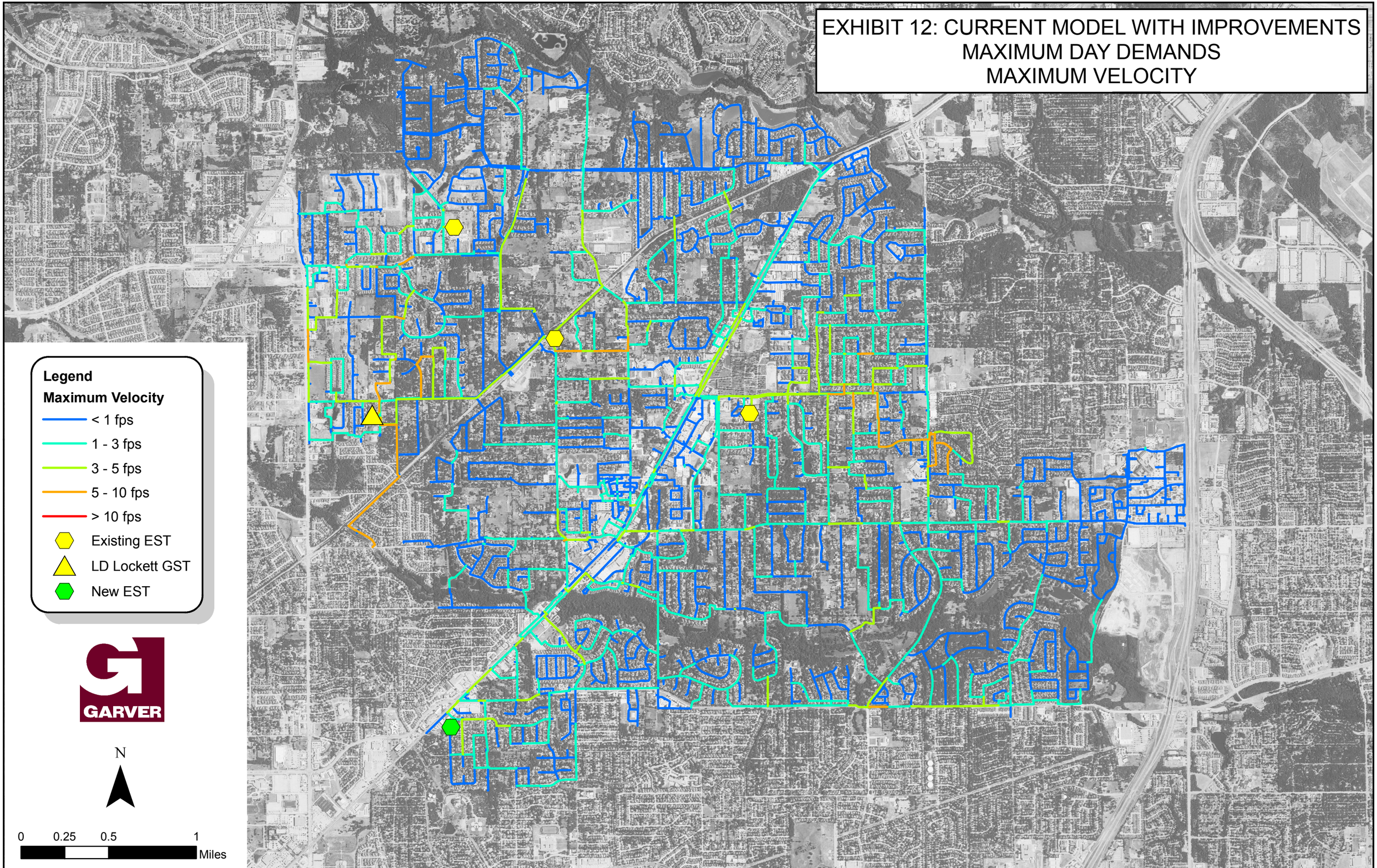


EXHIBIT 13: CURRENT MODEL WITH IMPROVEMENTS
MAXIMUM DAY DEMANDS
MAXIMUM WATER AGE

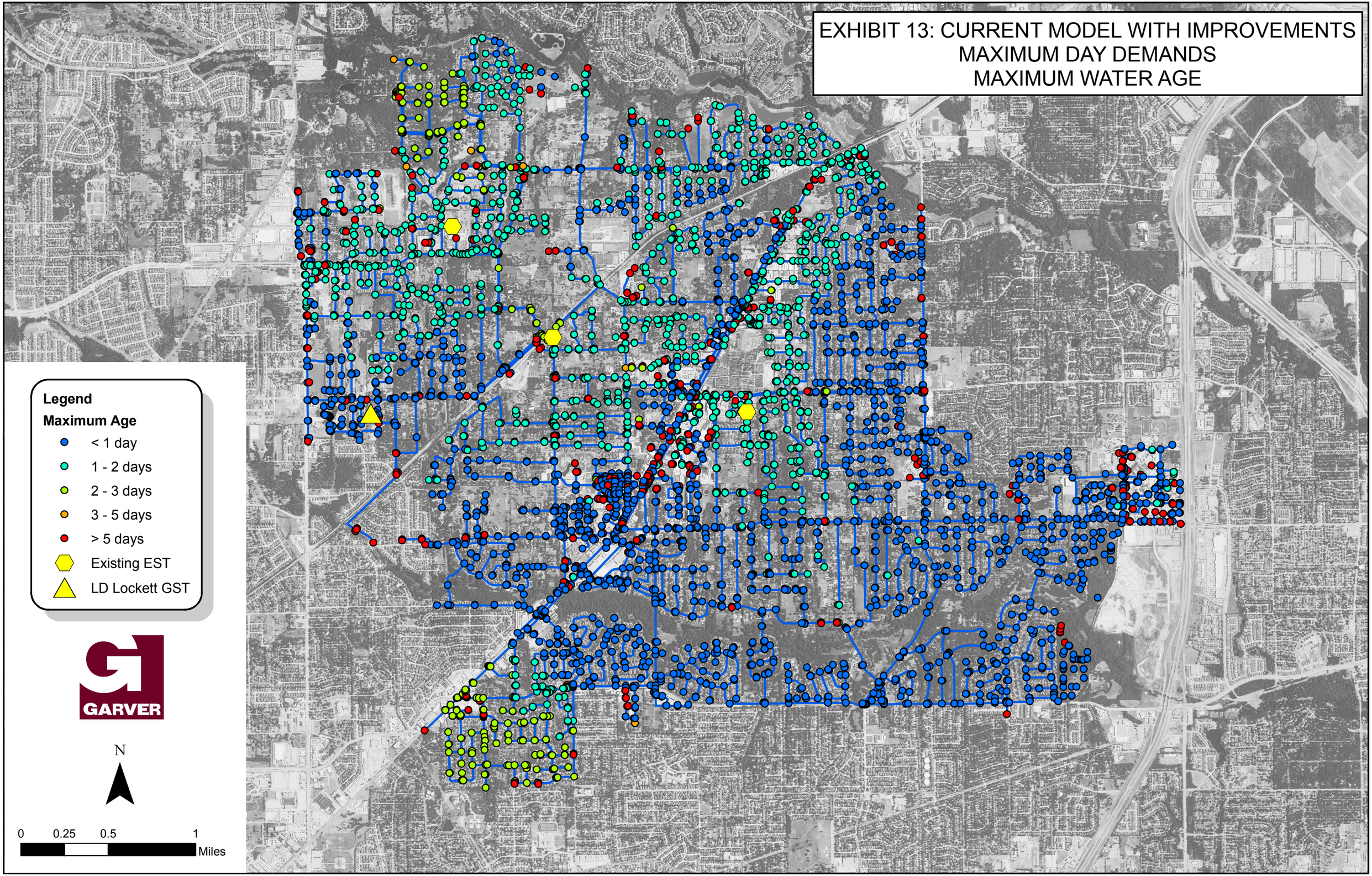


EXHIBIT 14: CURRENT MODEL WITH IMPROVEMENTS
MAXIMUM DAY DEMANDS
AVAILABLE FIRE FLOW

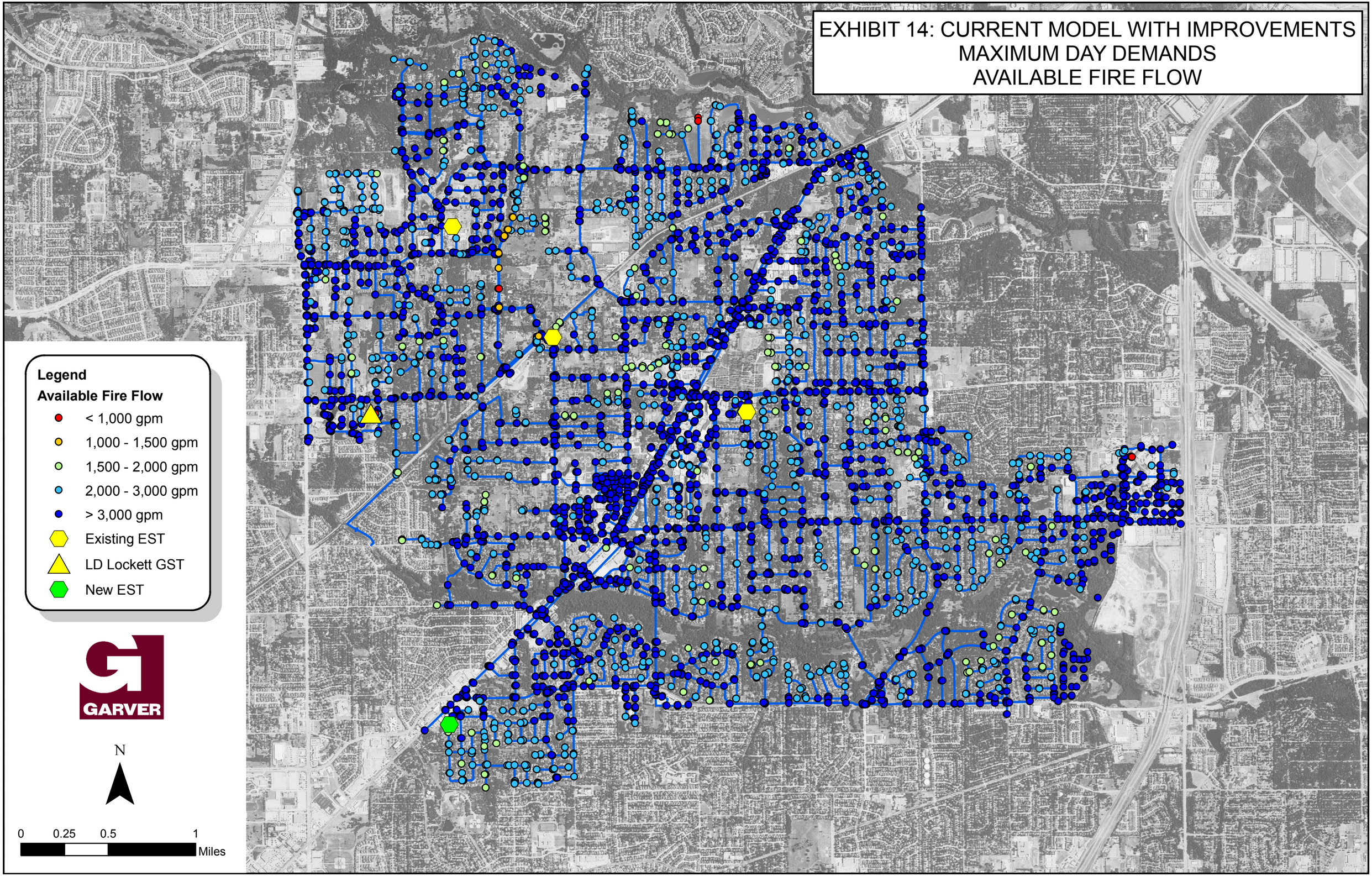
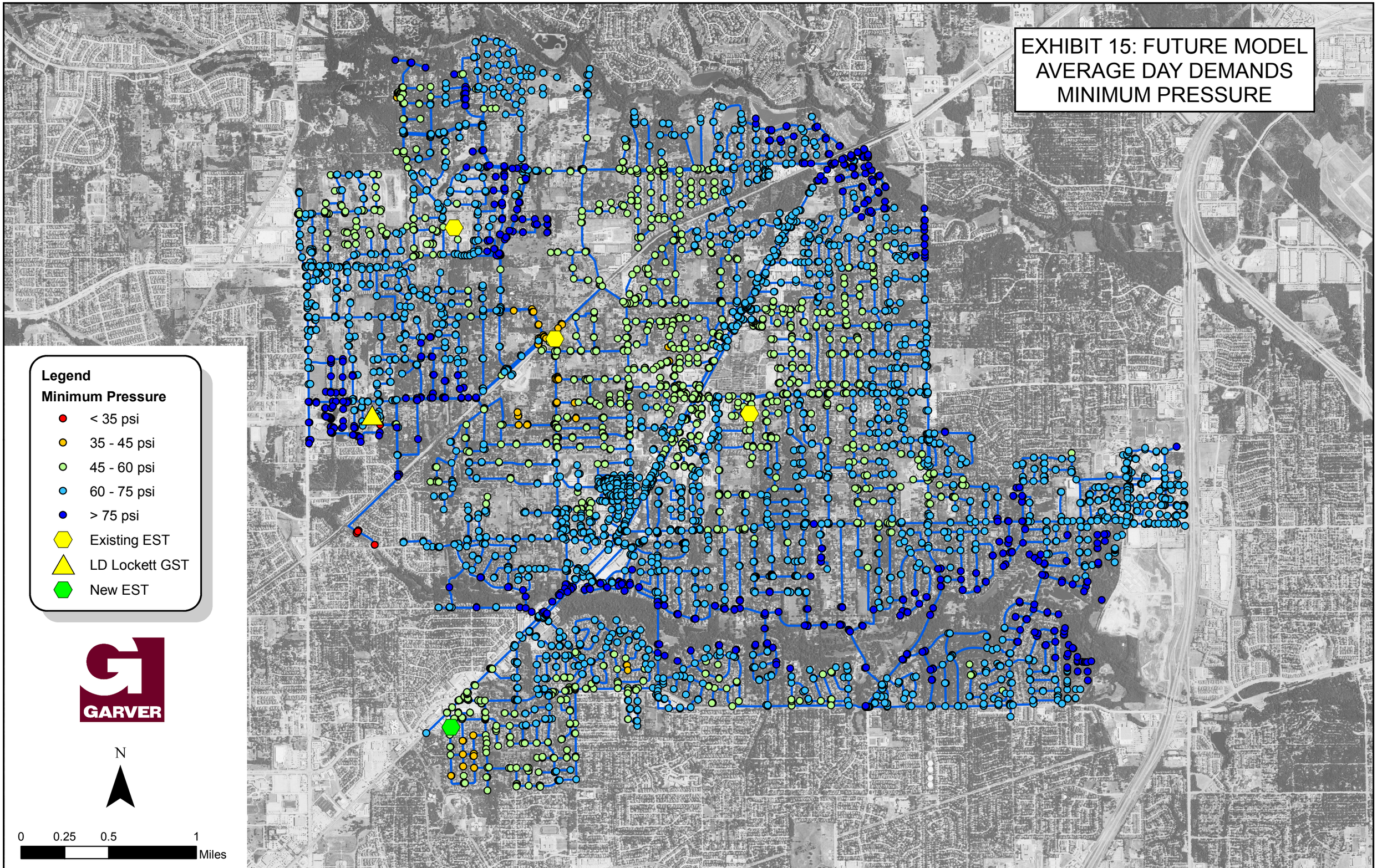


EXHIBIT 15: FUTURE MODEL AVERAGE DAY DEMANDS MINIMUM PRESSURE



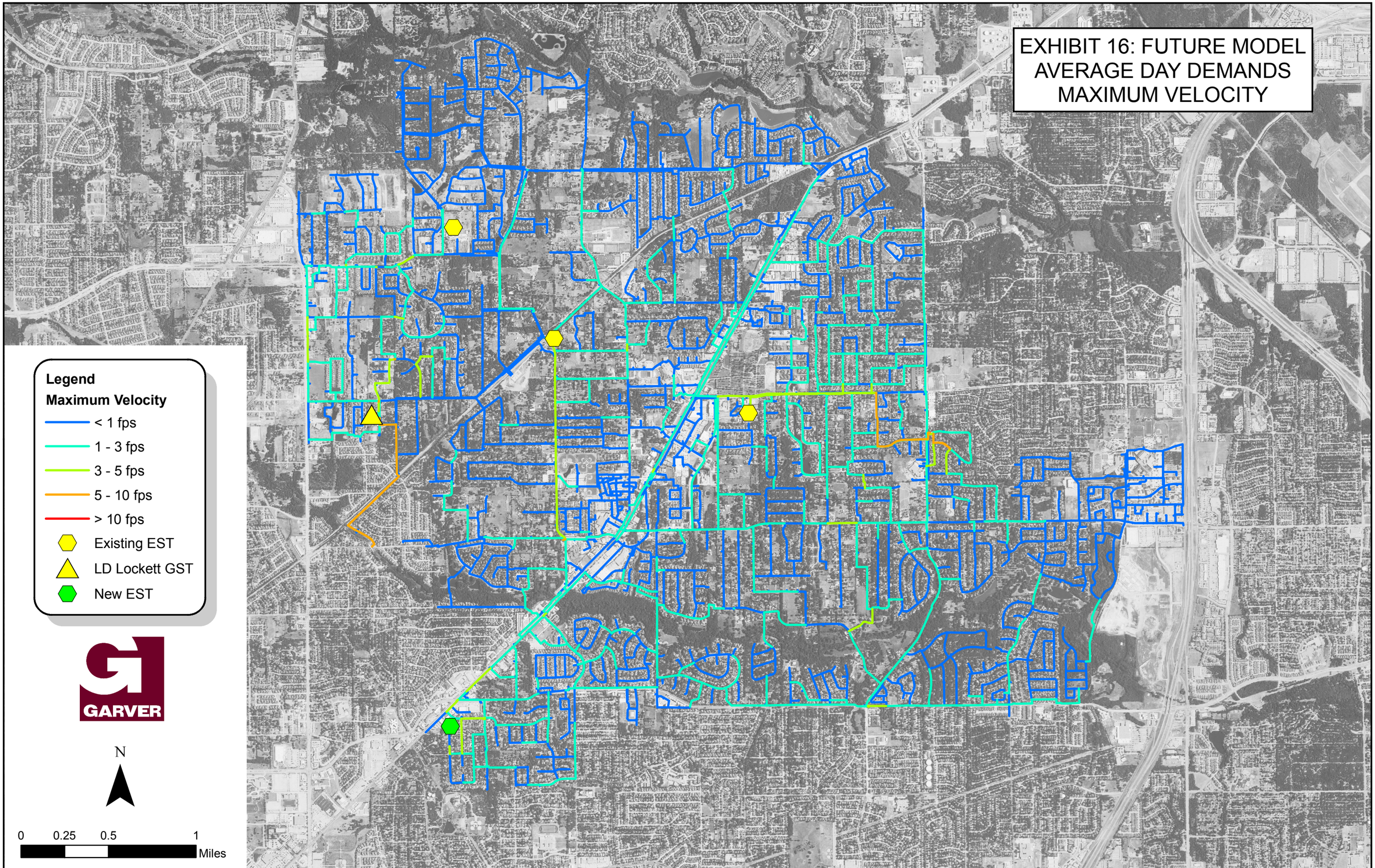


EXHIBIT 16: FUTURE MODEL
AVERAGE DAY DEMANDS
MAXIMUM VELOCITY
Legend
Maximum Velocity
< 1 fps
1 - 3 fps
3 - 5 fps
5 - 10 fps
> 10 fps
Existing EST
LD Lockett GST
New EST
GARVER
N
0
0.25
0.5
1
Miles

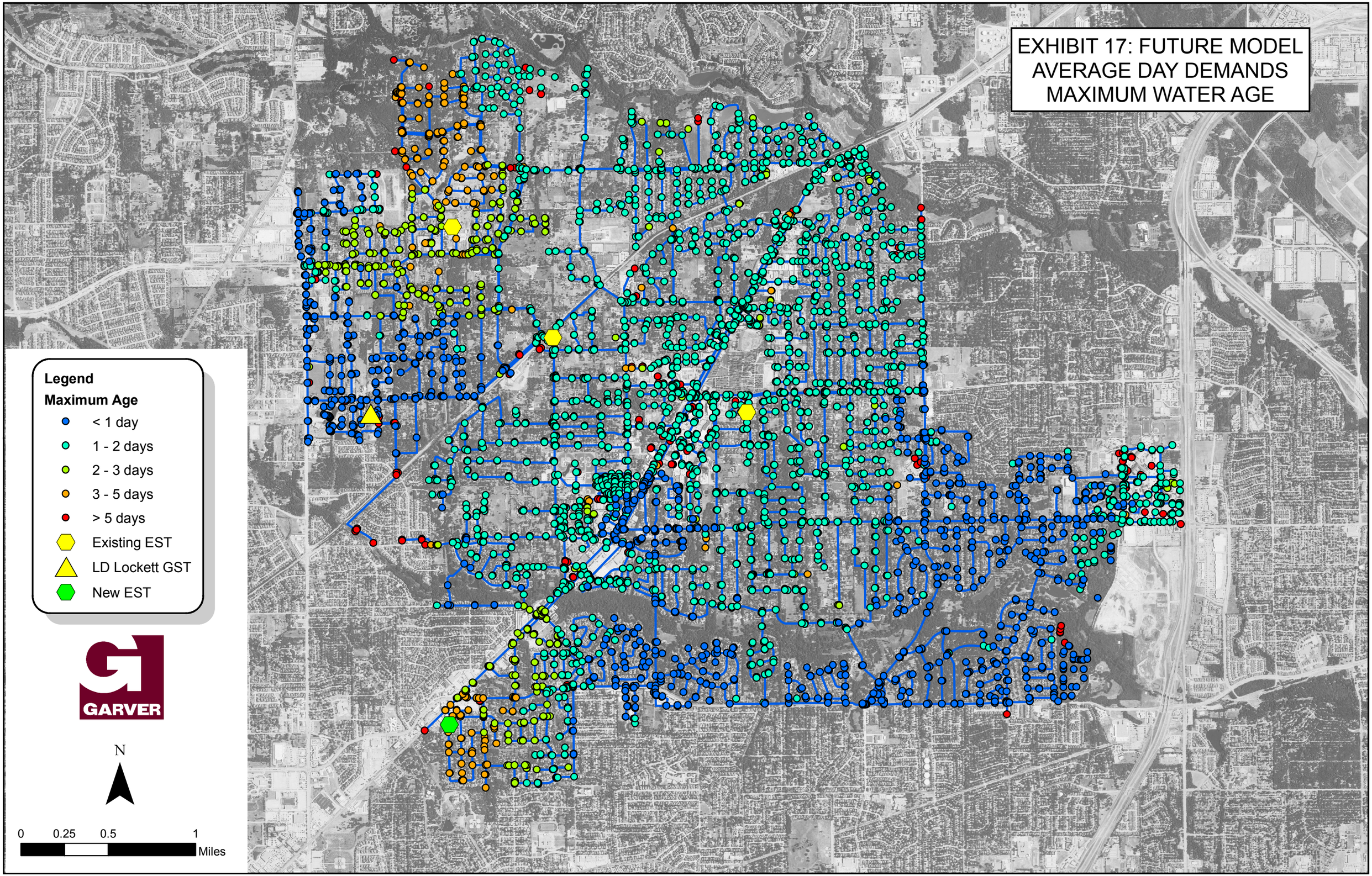


EXHIBIT 17: FUTURE MODEL
AVERAGE DAY DEMANDS
MAXIMUM WATER AGE
Legend
Maximum Age
< 1 day
1 - 2 days
2 - 3 days
3 - 5 days
> 5 days
Existing EST
LD Lockett GST
New EST
GARVER
N
0
0.25
0.5
1
Miles

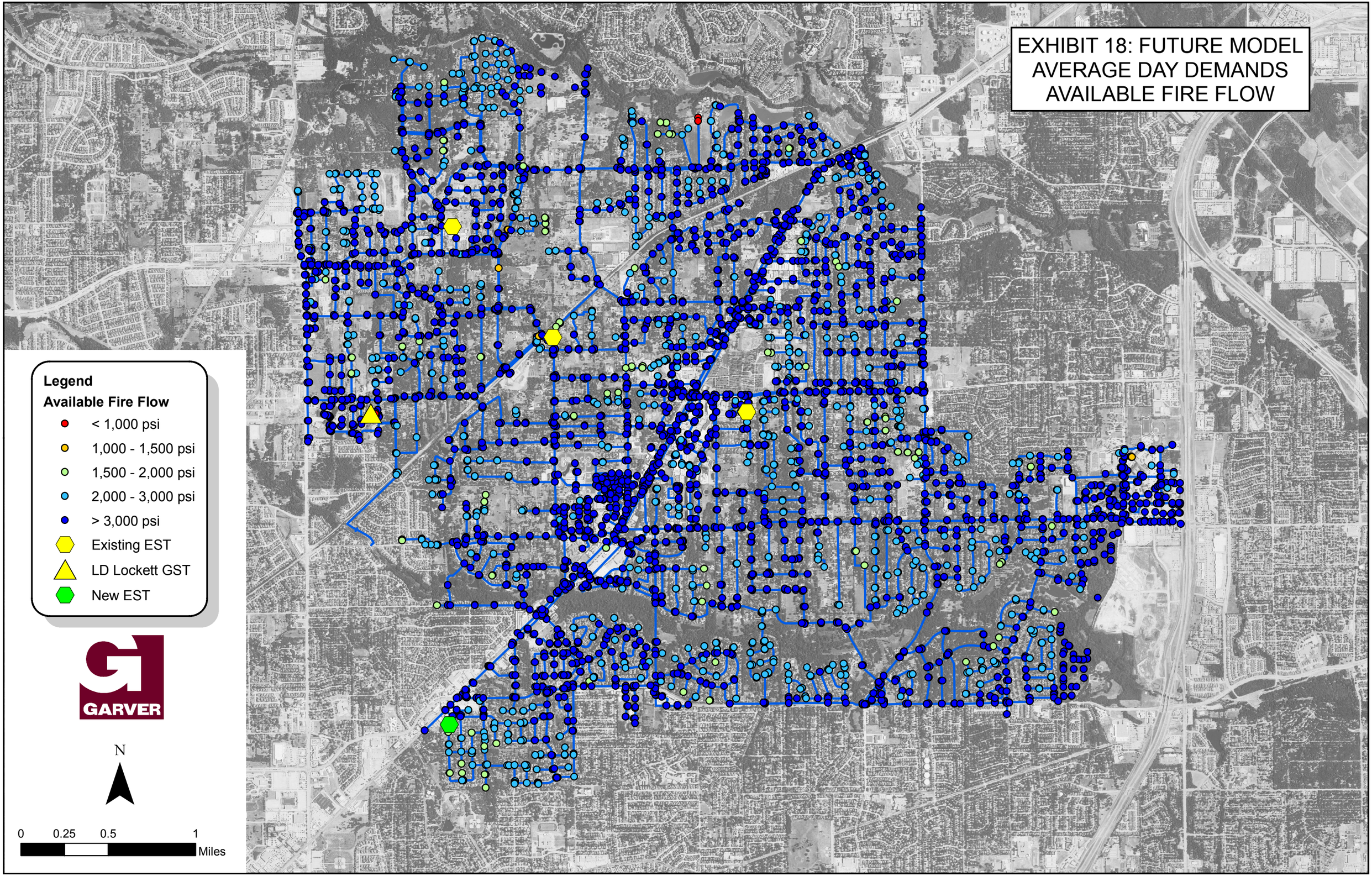
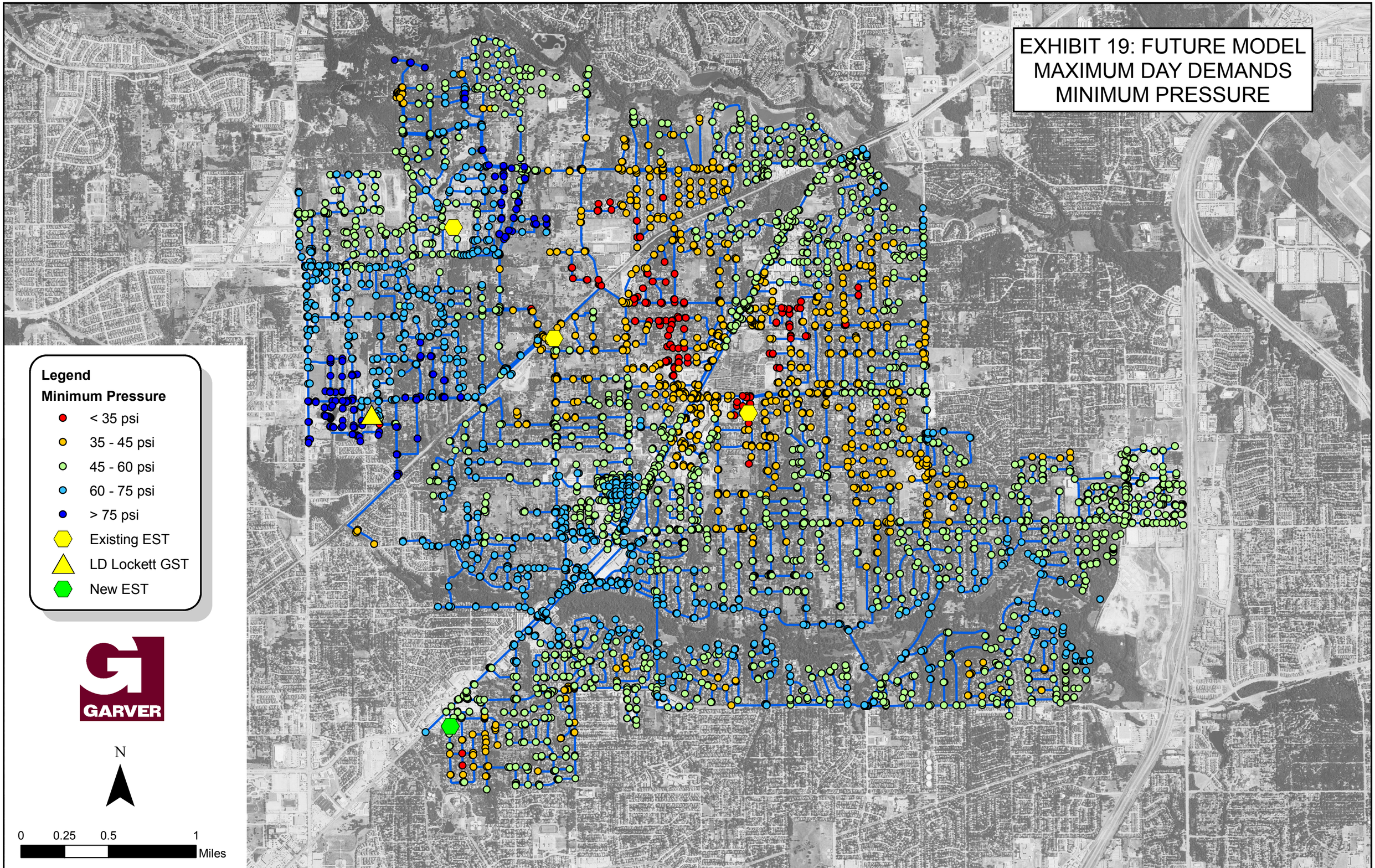


EXHIBIT 18: FUTURE MODEL
AVERAGE DAY DEMANDS
AVAILABLE FIRE FLOW
Legend
Available Fire Flow
< 1,000 psi
1,000 - 1,500 psi
1,500 - 2,000 psi
2,000 - 3,000 psi
> 3,000 psi
Existing EST
LD Lockett GST
New EST
GARVER
N
0
0.25
0.5
1
Miles

EXHIBIT 19: FUTURE MODEL MAXIMUM DAY DEMANDS MINIMUM PRESSURE



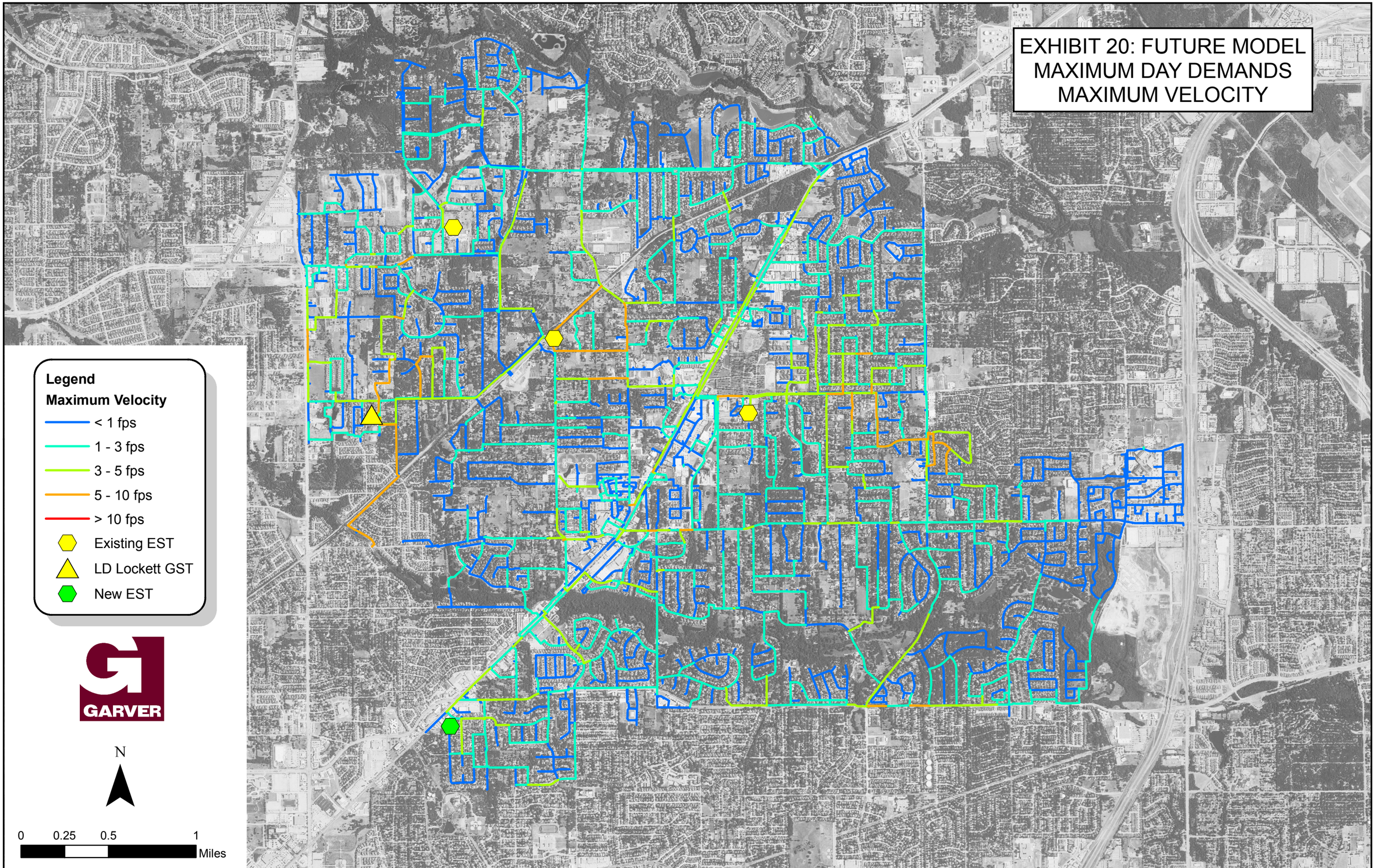


EXHIBIT 20: FUTURE MODEL
MAXIMUM DAY DEMANDS
MAXIMUM VELOCITY
Legend
Maximum Velocity
< 1 fps
1 - 3 fps
3 - 5 fps
5 - 10 fps
> 10 fps
Existing EST
LD Lockett GST
New EST
GARVER
N
0
0.25
0.5
1
Miles

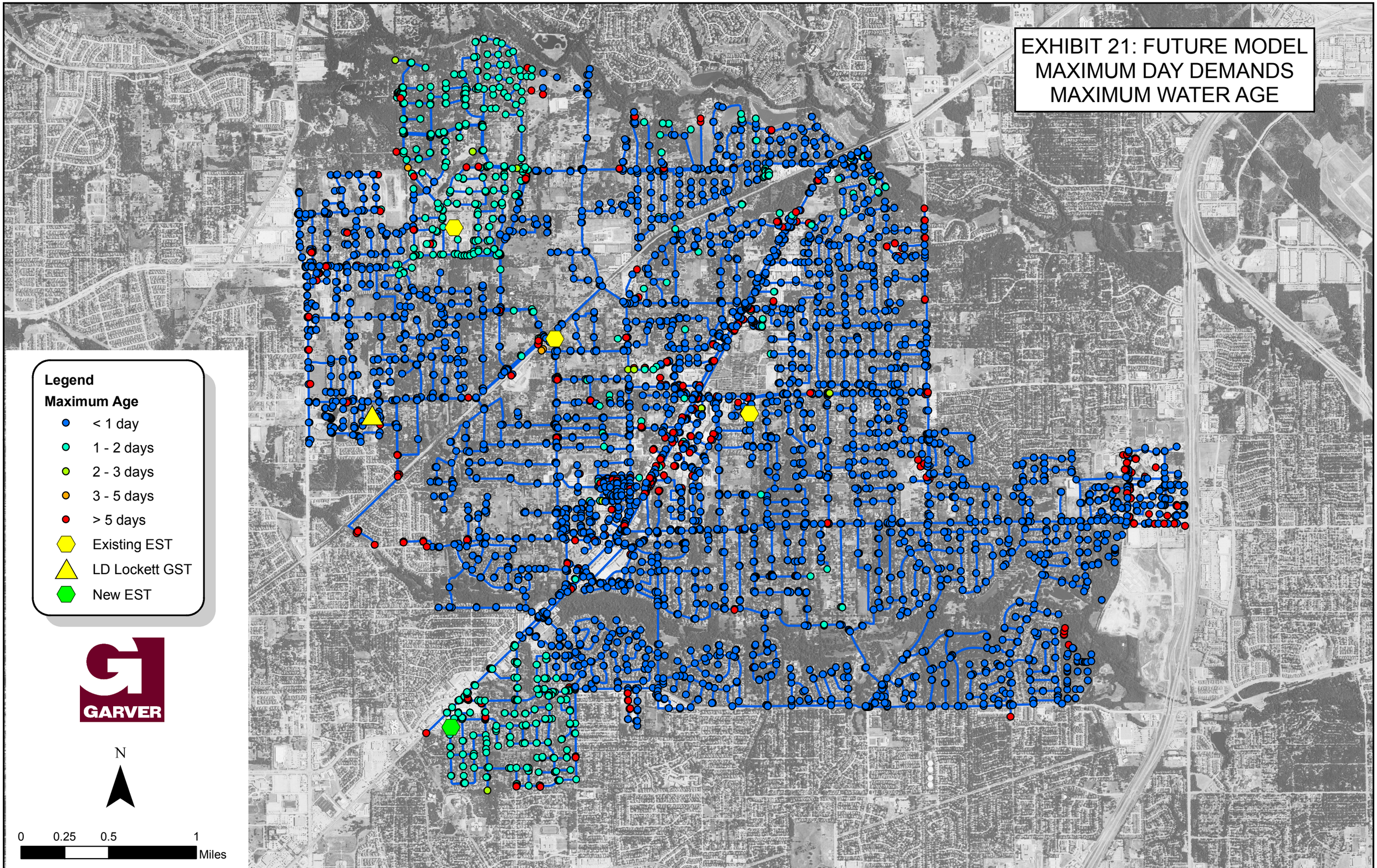
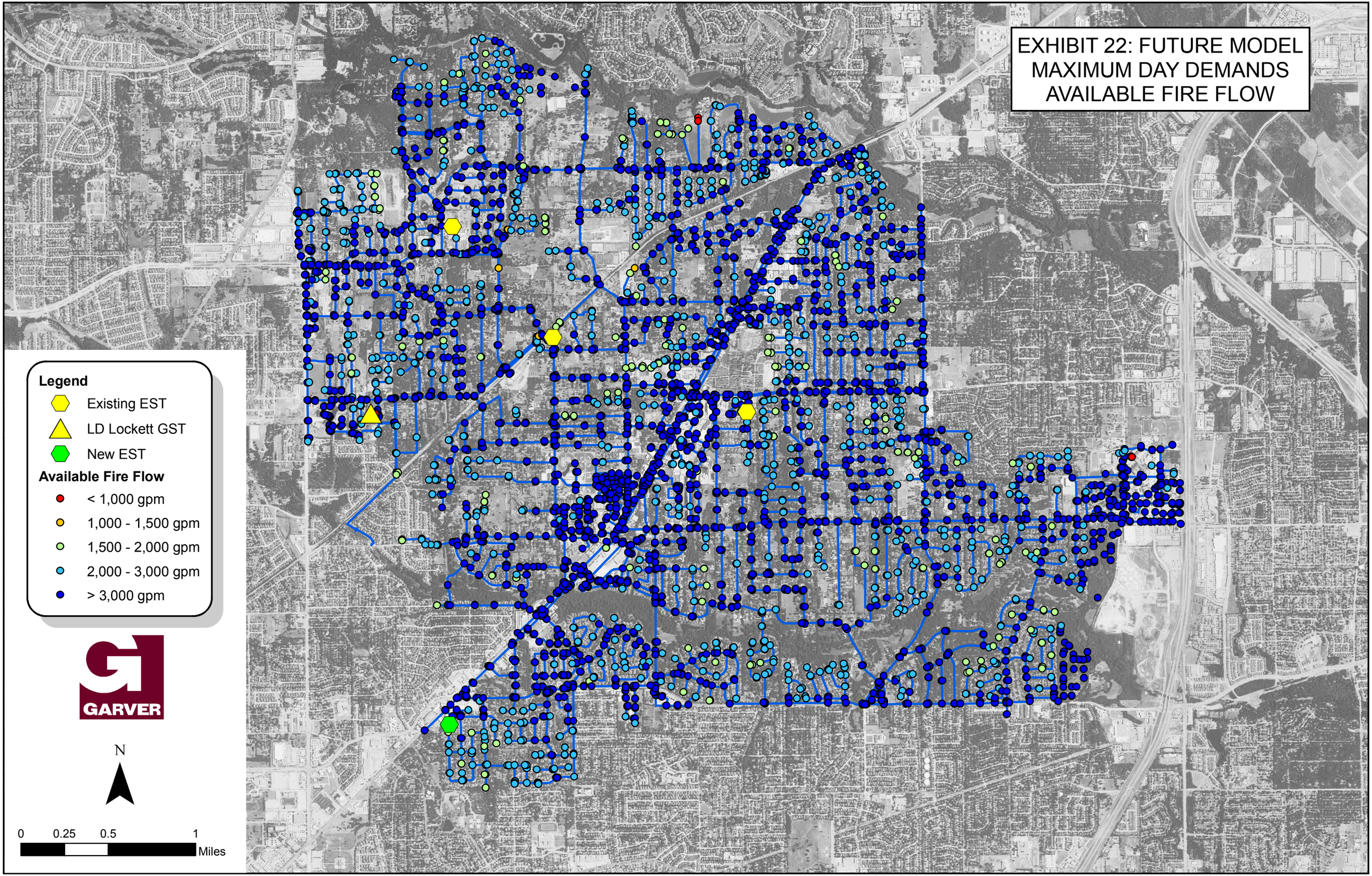


EXHIBIT 21: FUTURE MODEL
MAXIMUM DAY DEMANDS
MAXIMUM WATER AGE
Legend
Maximum Age
< 1 day
1 - 2 days
2 - 3 days
3 - 5 days
> 5 days
Existing EST
LD Lockett GST
New EST
GARVER
N
0
0.25
0.5
1
Miles

EXHIBIT 22: FUTURE MODEL MAXIMUM DAY DEMANDS AVAILABLE FIRE FLOW



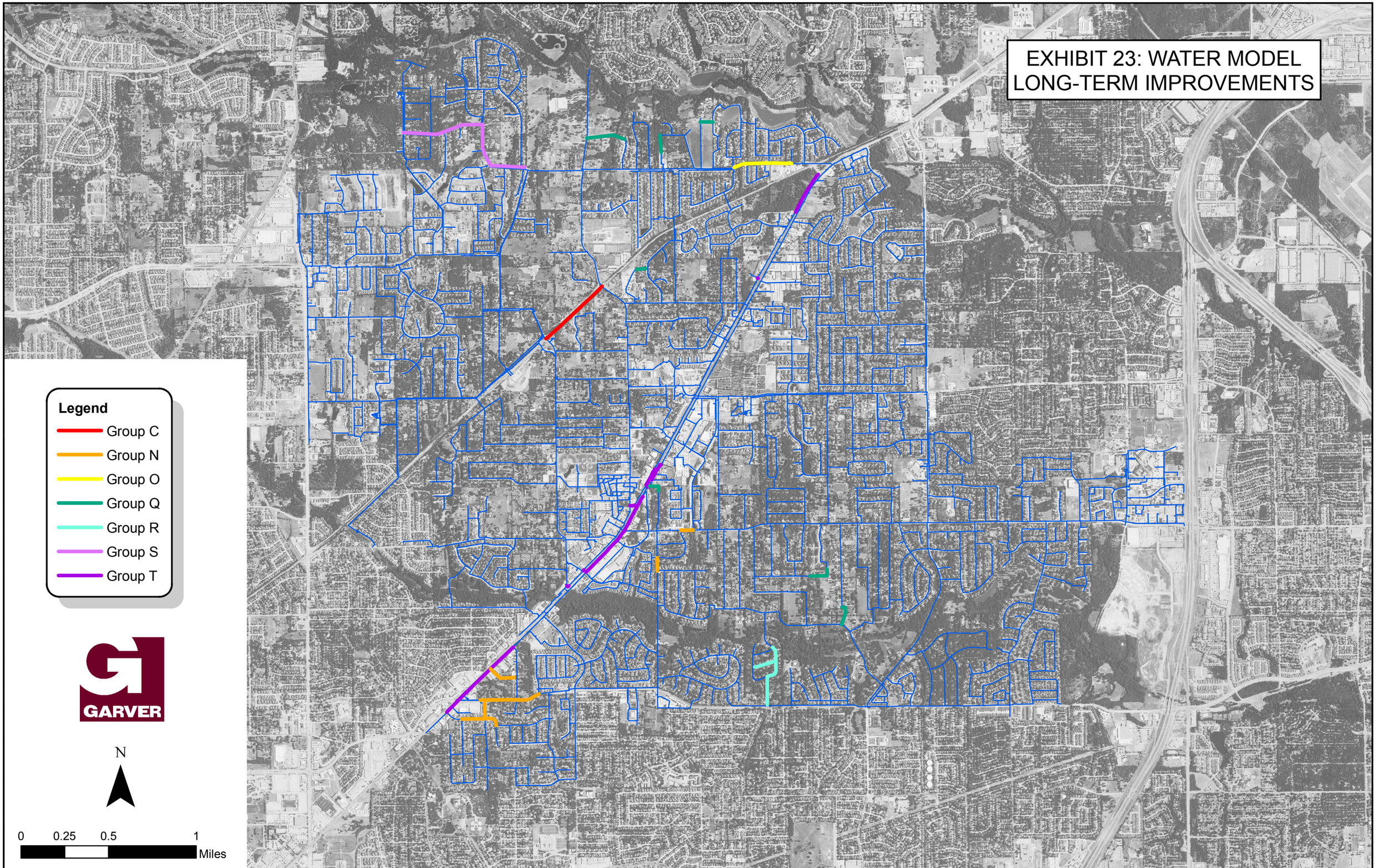


EXHIBIT 23: WATER MODEL
LONG-TERM IMPROVEMENTS
Legend
Group C
Group N
Group O
Group Q
Group R
Group S
Group T
GARVER
N
0
0.25
0.5
1
Miles

EXHIBIT 24: FUTURE MODEL WITH IMPROVEMENTS
MAXIMUM DAY DEMANDS
MINIMUM PRESSURE

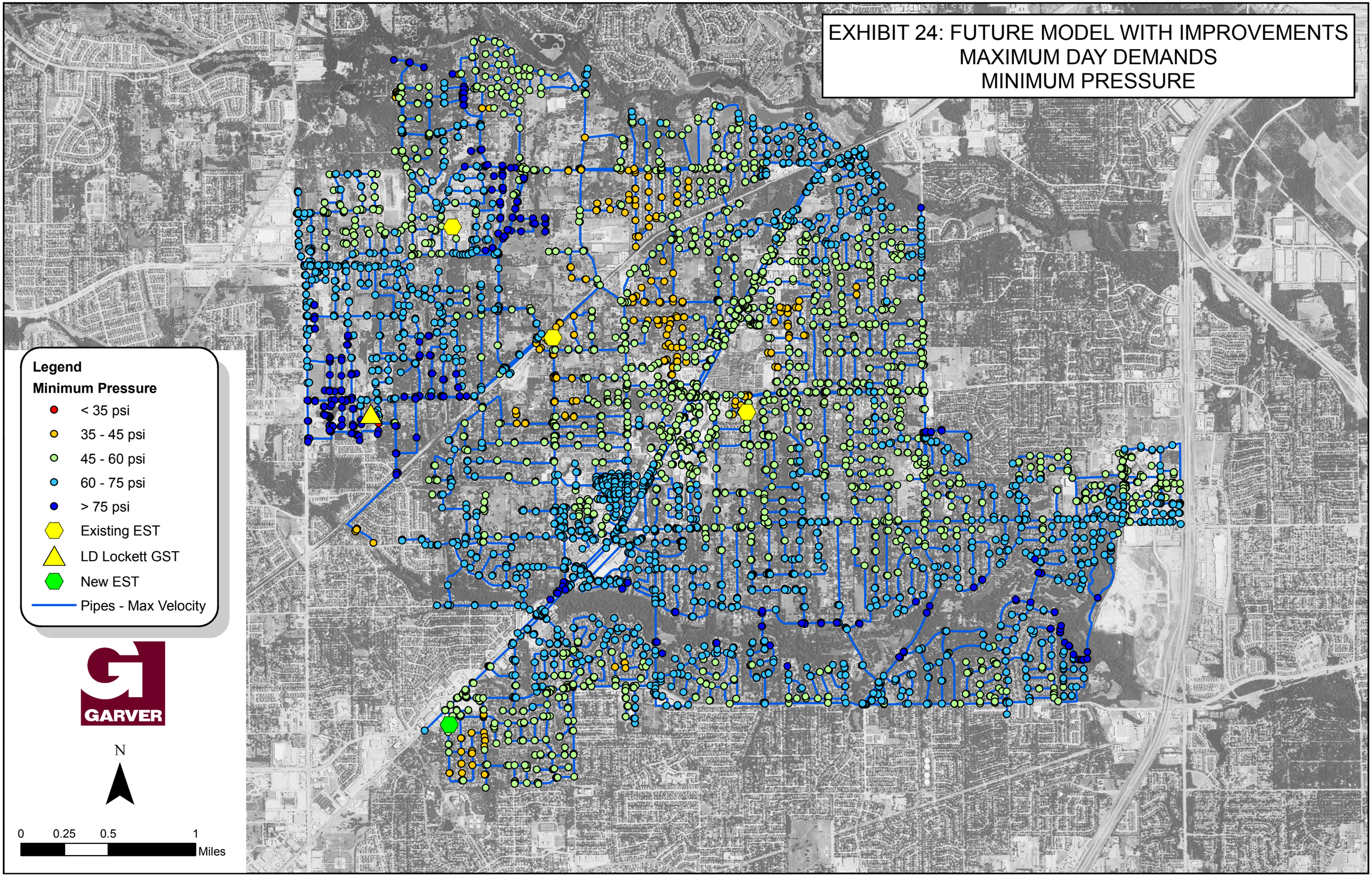


EXHIBIT 25: FUTURE MODEL WITH IMPROVEMENTS
MAXIMUM DAY DEMANDS
MAXIMUM VELOCITY

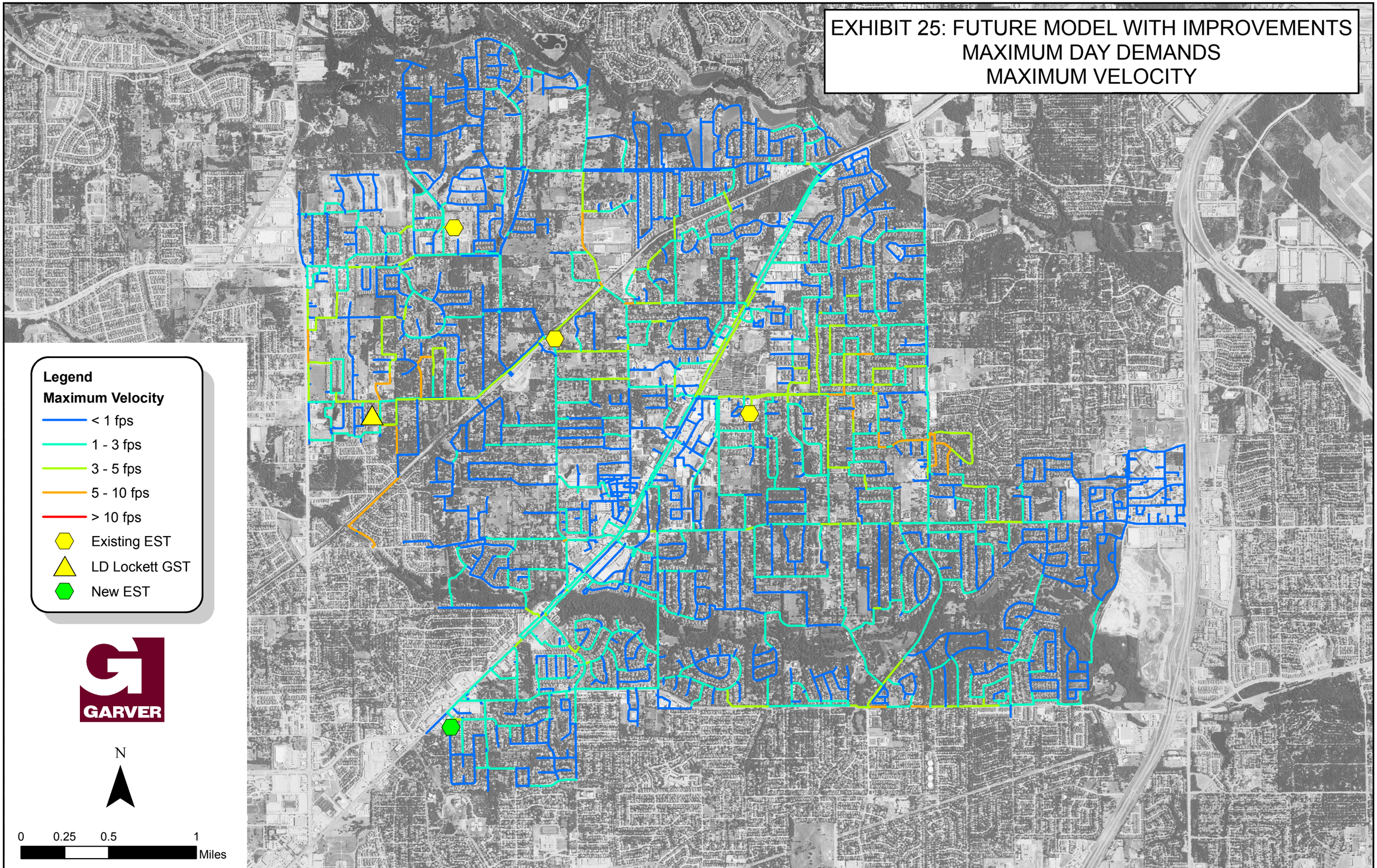
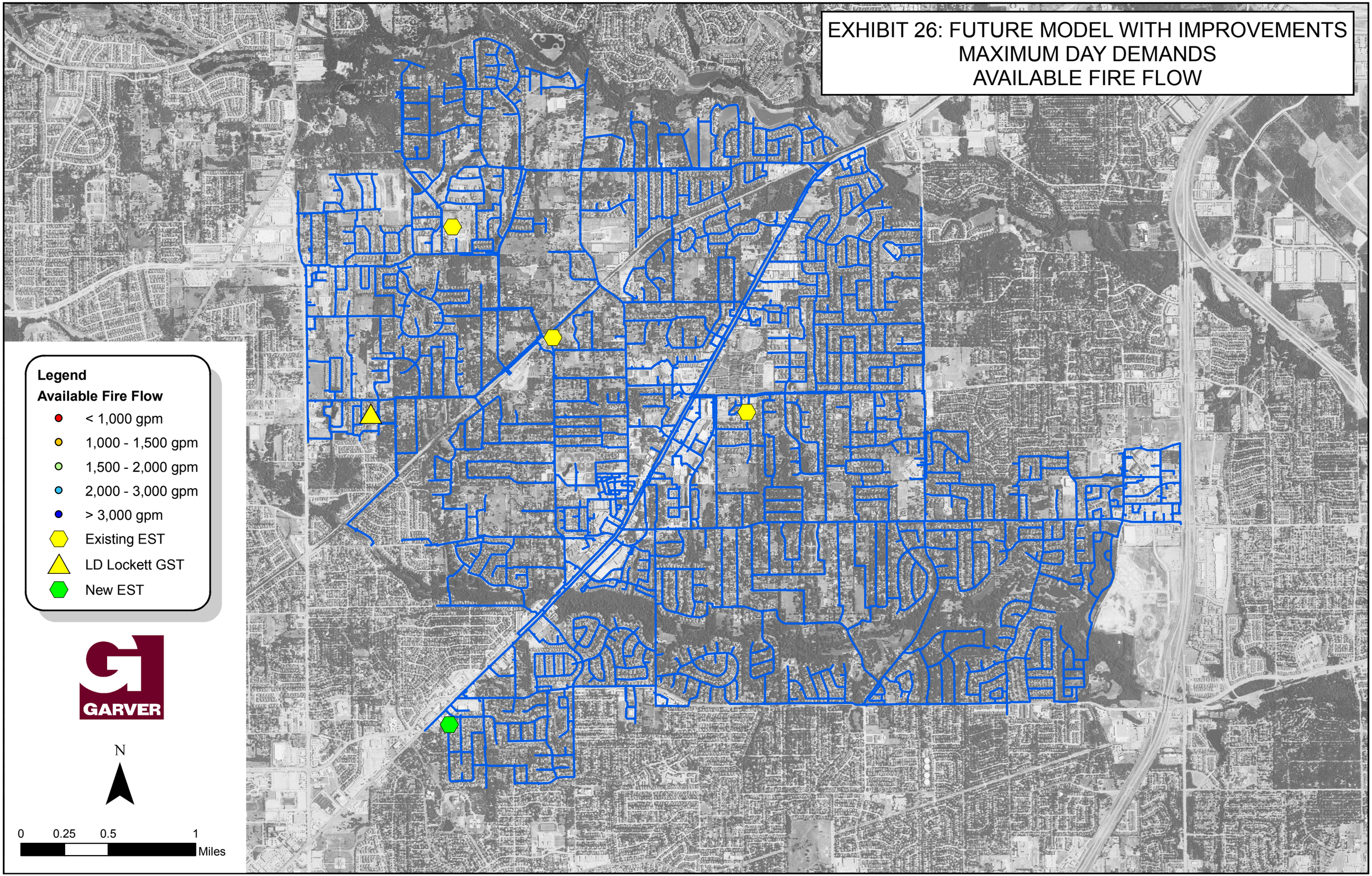


EXHIBIT 26: FUTURE MODEL WITH IMPROVEMENTS
MAXIMUM DAY DEMANDS
AVAILABLE FIRE FLOW



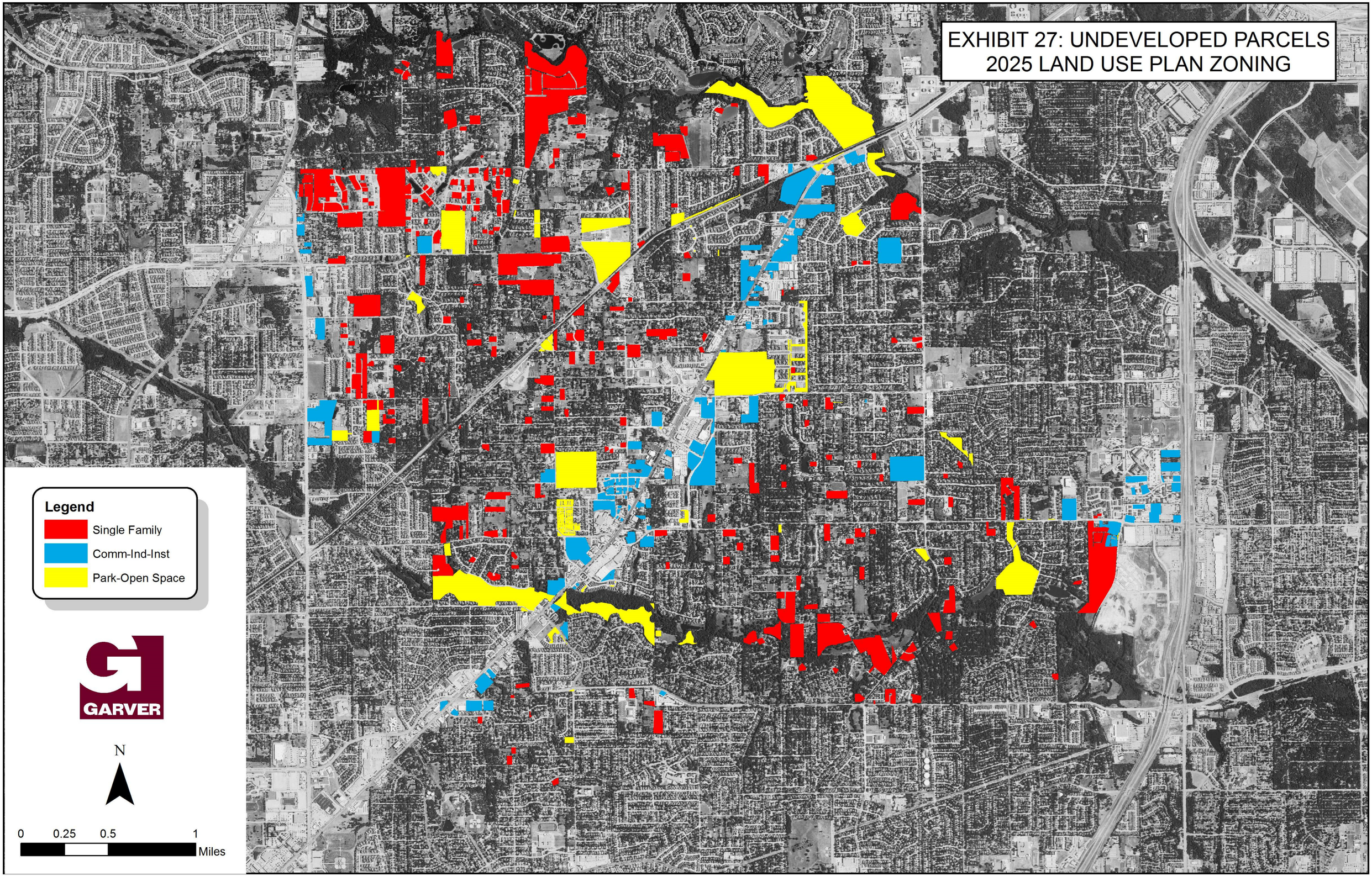


EXHIBIT 27: UNDEVELOPED PARCELS
2025 LAND USE PLAN ZONING
Legend
Single Family
Comm-Ind-Inst
Park-Open Space
GARVER
N
0
0.25
0.5
1
Miles

EXHIBIT 28: EXISTING WASTEWATER SYSTEM

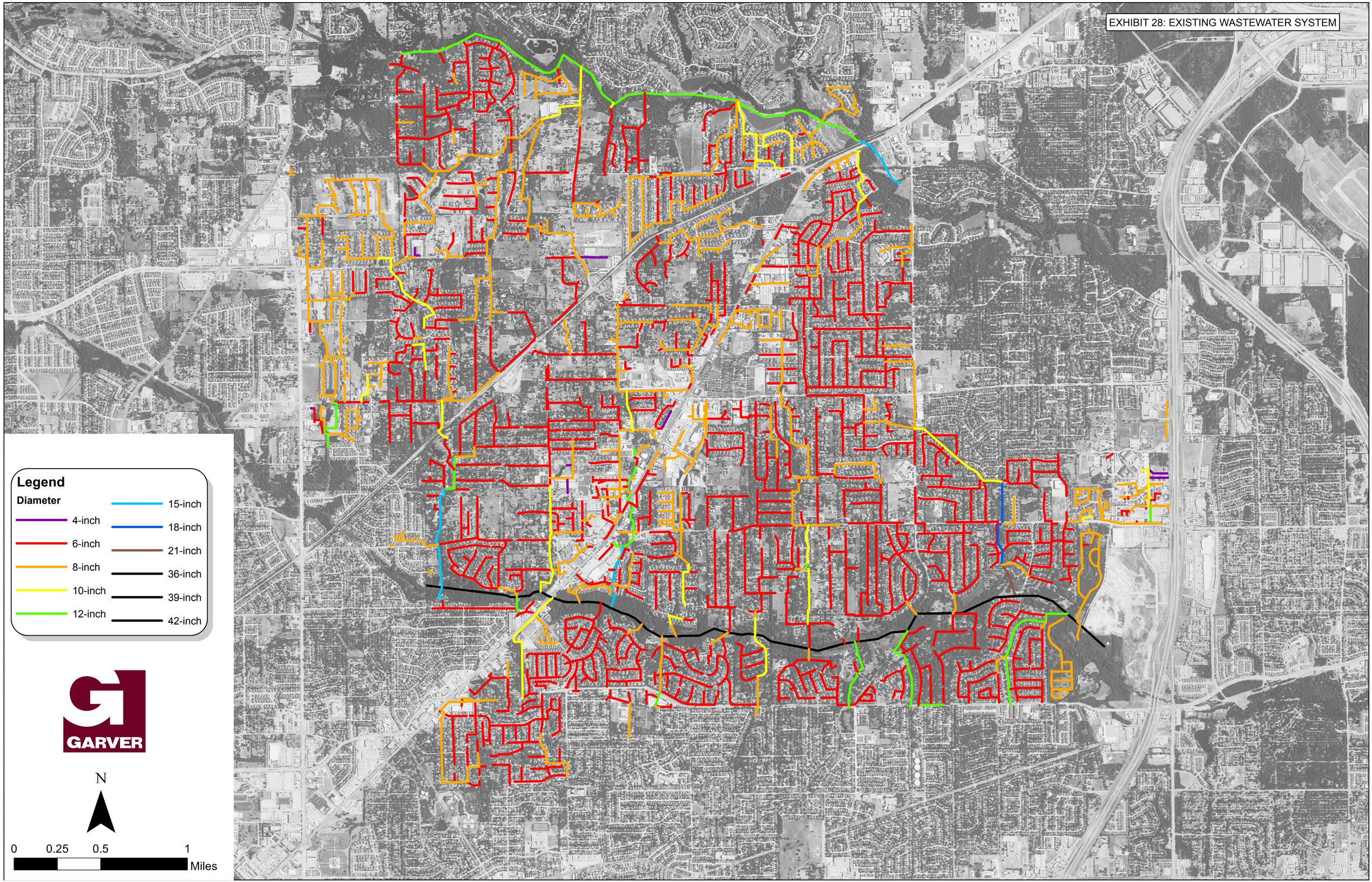
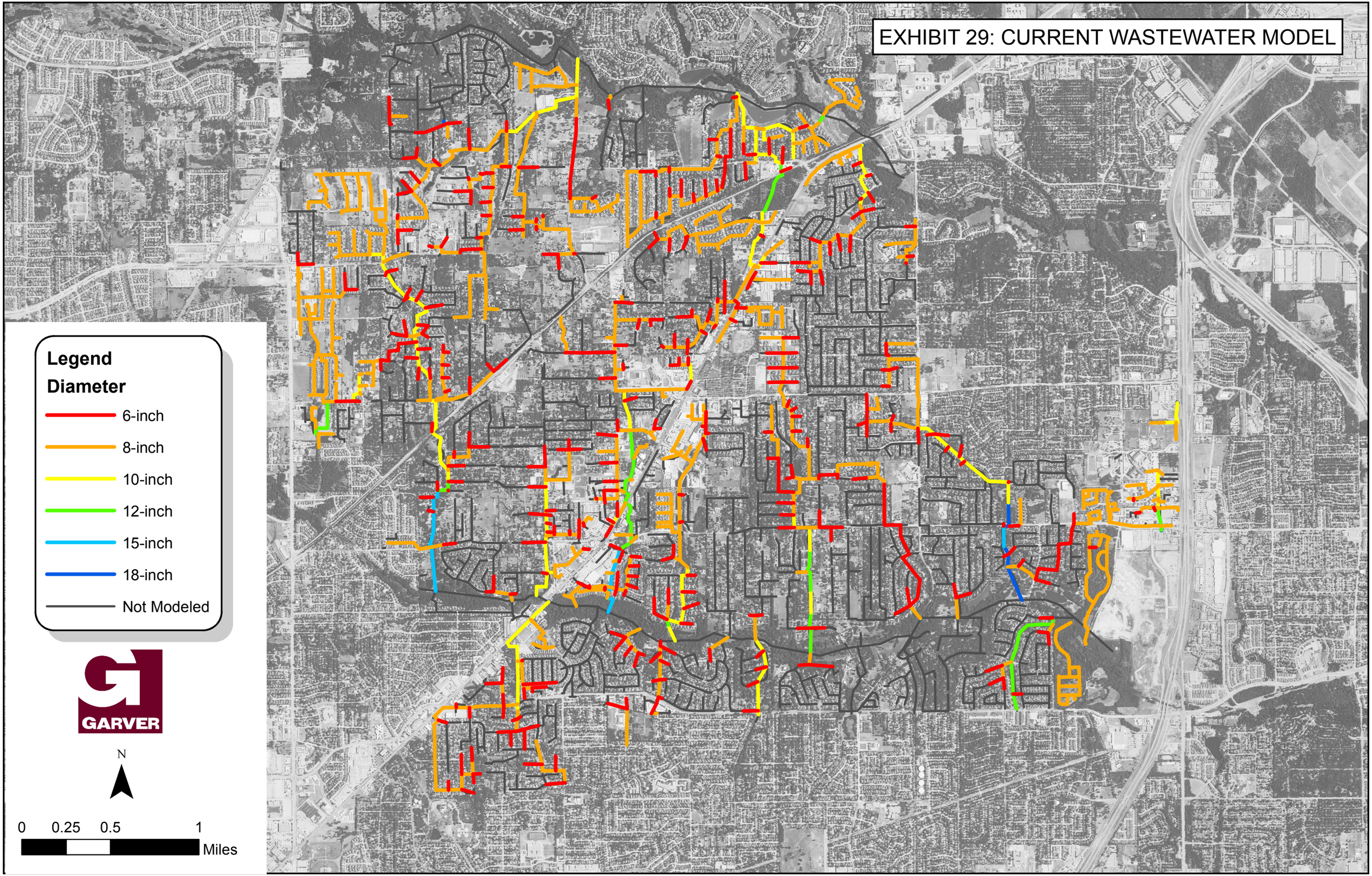


EXHIBIT 29: CURRENT WASTEWATER MODEL



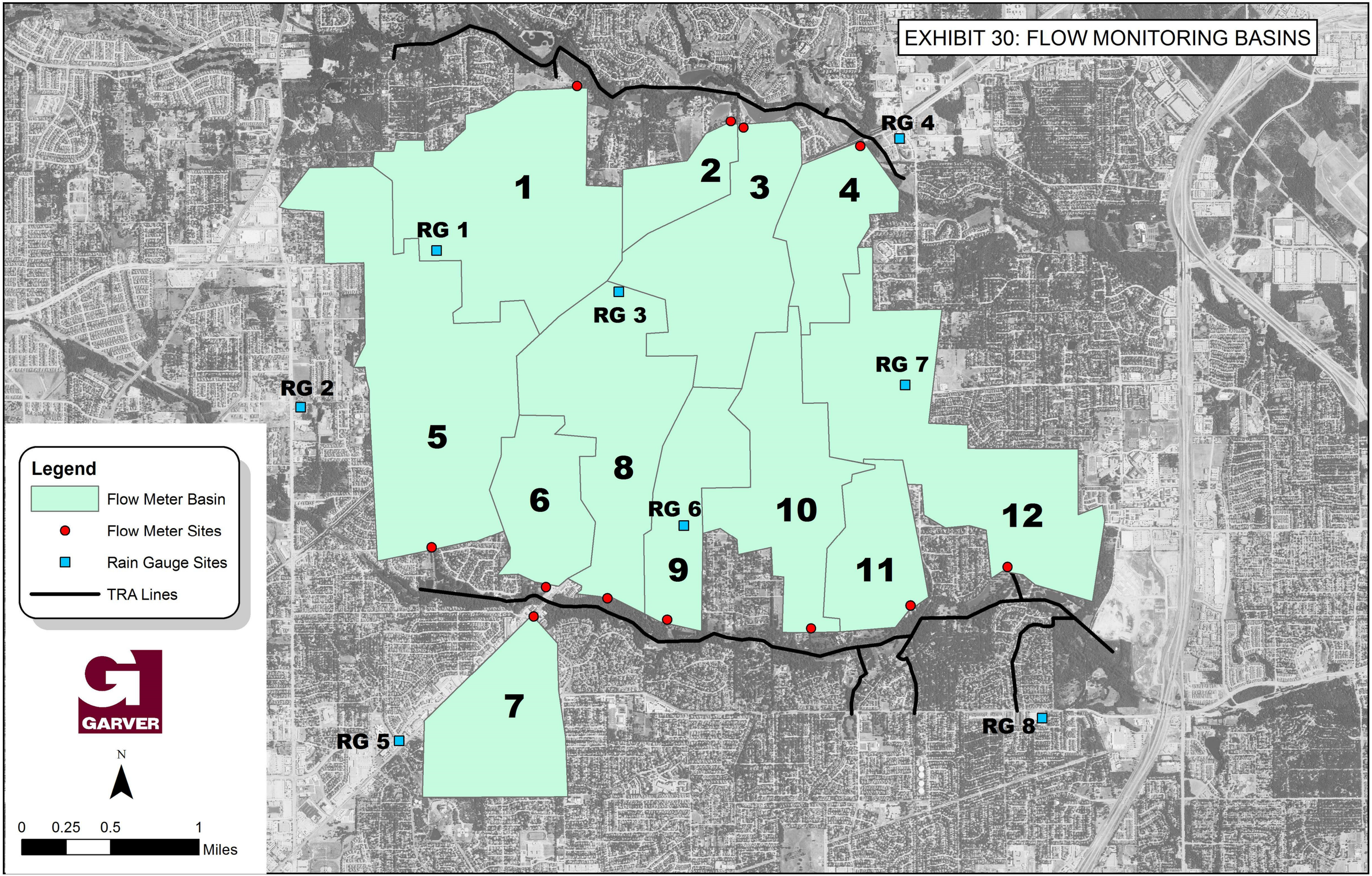


EXHIBIT 30: FLOW MONITORING BASINS
1
2
3
4
5
6
7
8
9
10
11
12
RG 1
RG 2
RG 3
RG 4
RG 5
RG 6
RG 7
RG 8
Legend
Flow Meter Basin
Flow Meter Sites
Rain Gauge Sites
TRA Lines
GARVER
N
0
0.25
0.5
1
Miles

EXHIBIT 31: CURRENT SYSTEM PIPES WITH INADEQUATE SLOPES

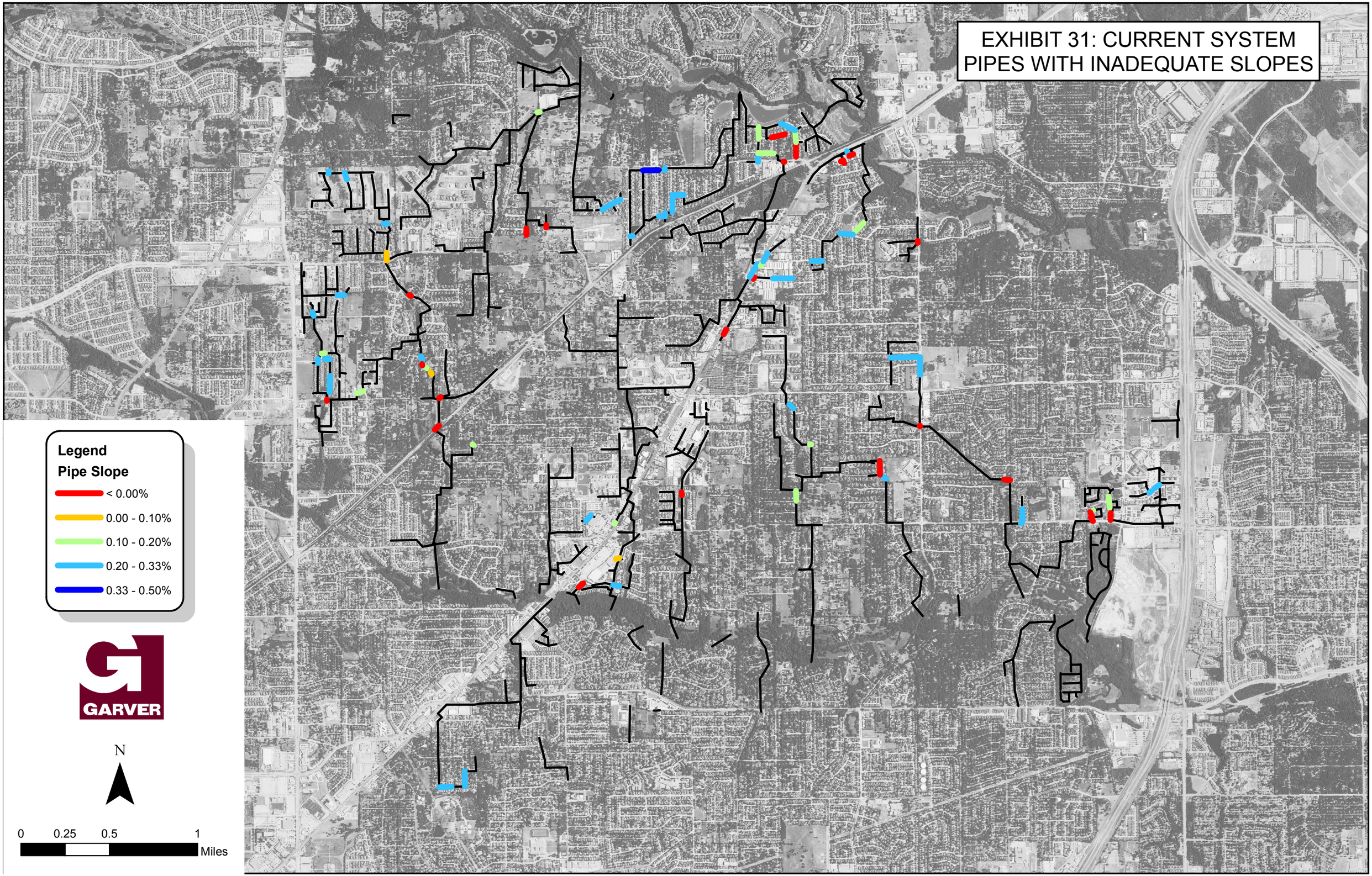


EXHIBIT 32: CURRENT SYSTEM DRY WEATHER FLOW PIPE CAPACITY

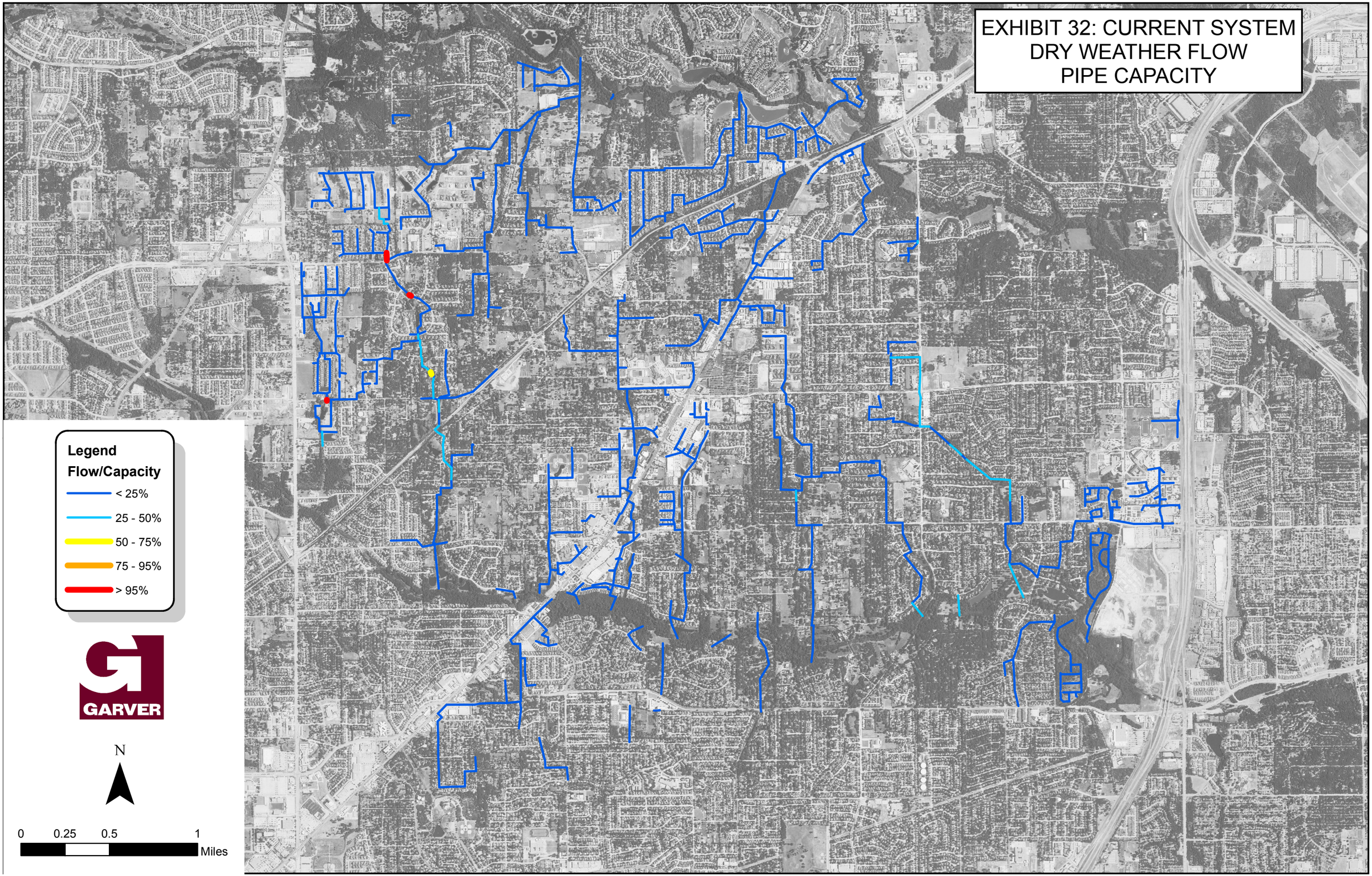
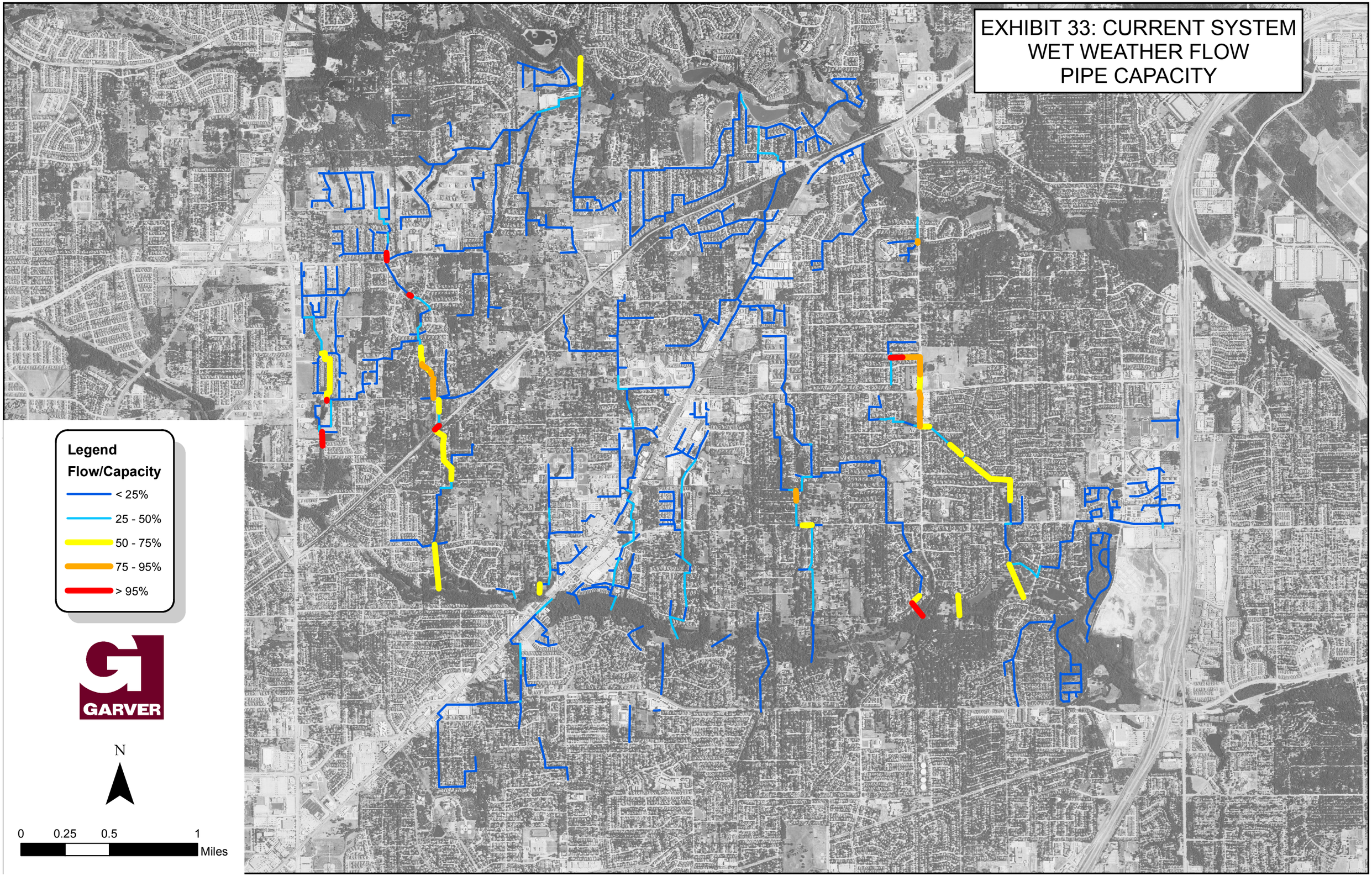


EXHIBIT 33: CURRENT SYSTEM WET WEATHER FLOW PIPE CAPACITY



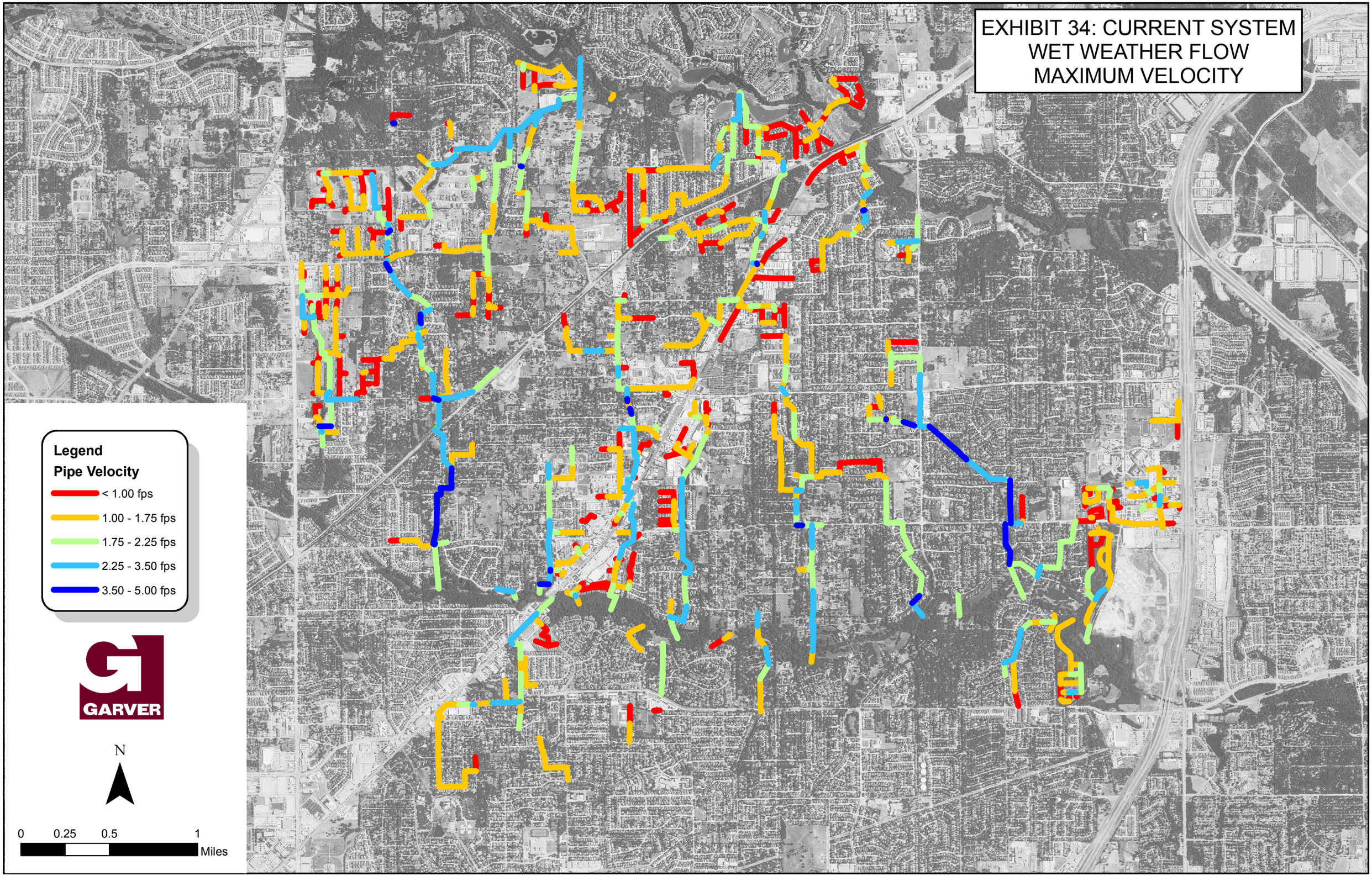


EXHIBIT 34: CURRENT SYSTEM
WET WEATHER FLOW
MAXIMUM VELOCITY
Legend
Pipe Velocity
< 1.00 fps
1.00 - 1.75 fps
1.75 - 2.25 fps
2.25 - 3.50 fps
3.50 - 5.00 fps
GARVER
N
0
0.25
0.5
1
Miles

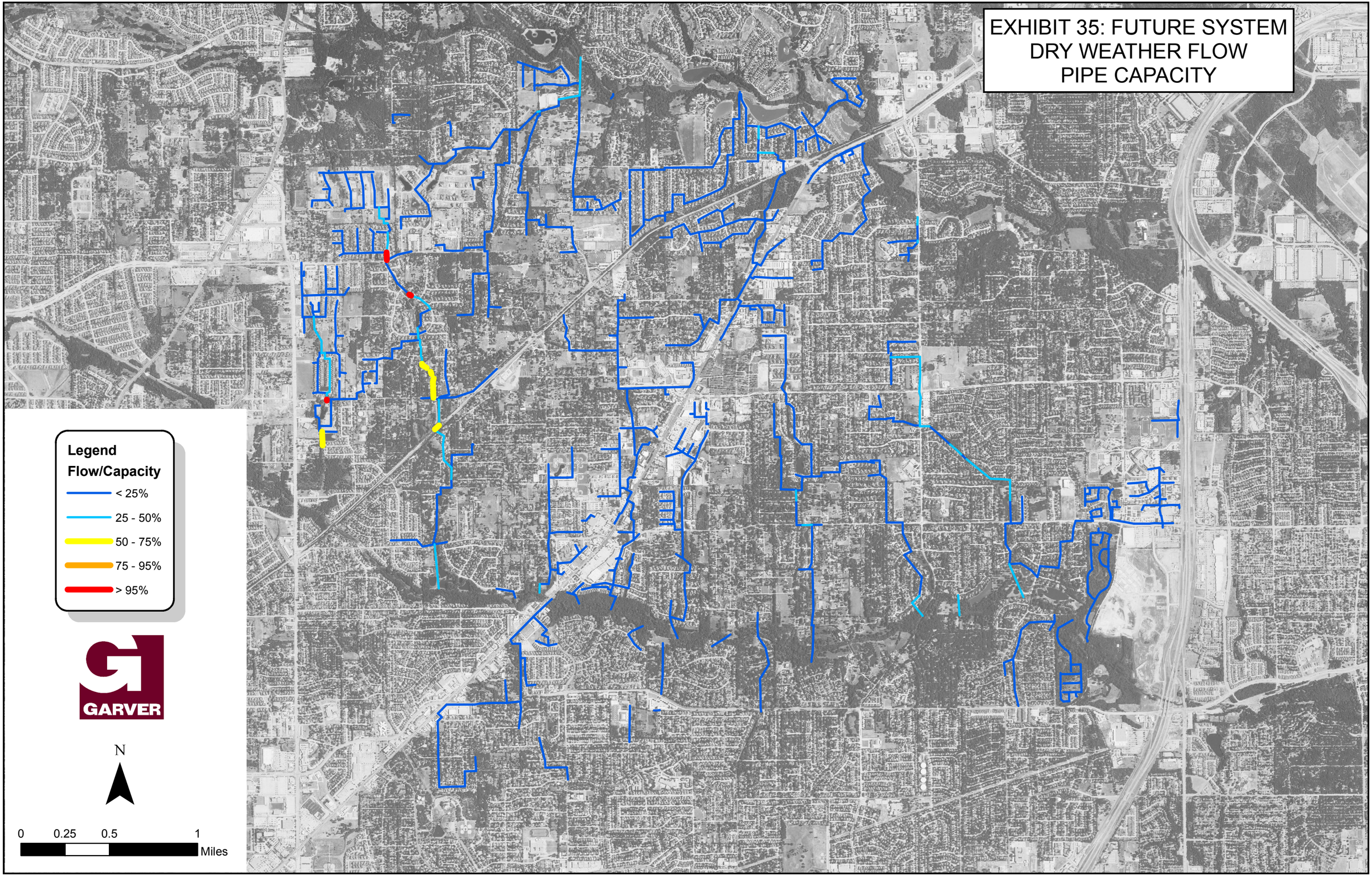


EXHIBIT 35: FUTURE SYSTEM
DRY WEATHER FLOW
PIPE CAPACITY
Legend
Flow/Capacity
< 25%
25 - 50%
50 - 75%
75 - 95%
> 95%
GARVER
N
0
0.25
0.5
1
Miles

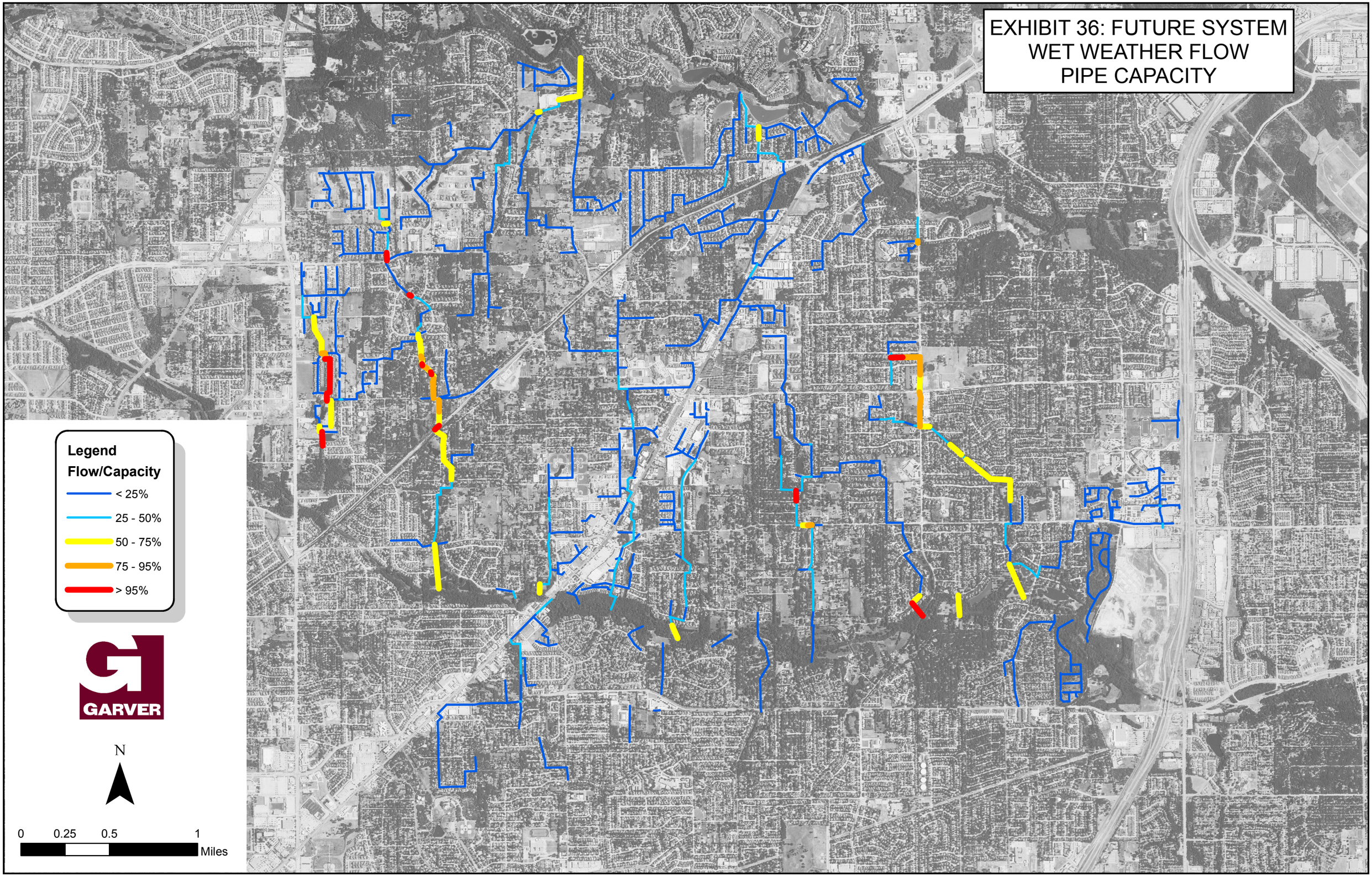


EXHIBIT 36: FUTURE SYSTEM
WET WEATHER FLOW
PIPE CAPACITY
Legend
Flow/Capacity
< 25%
25 - 50%
50 - 75%
75 - 95%
> 95%
GARVER
N
0
0.25
0.5
1
Miles

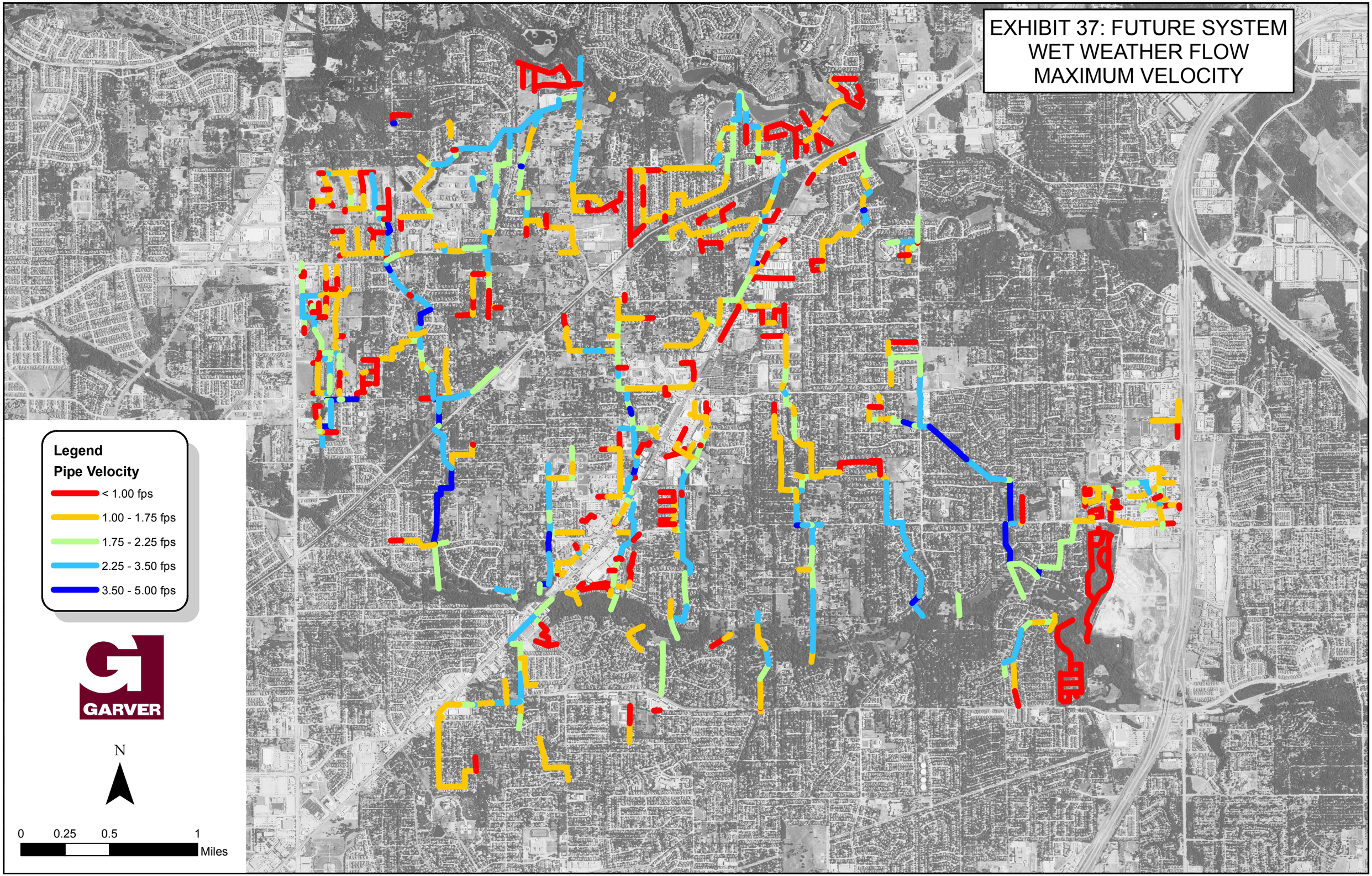
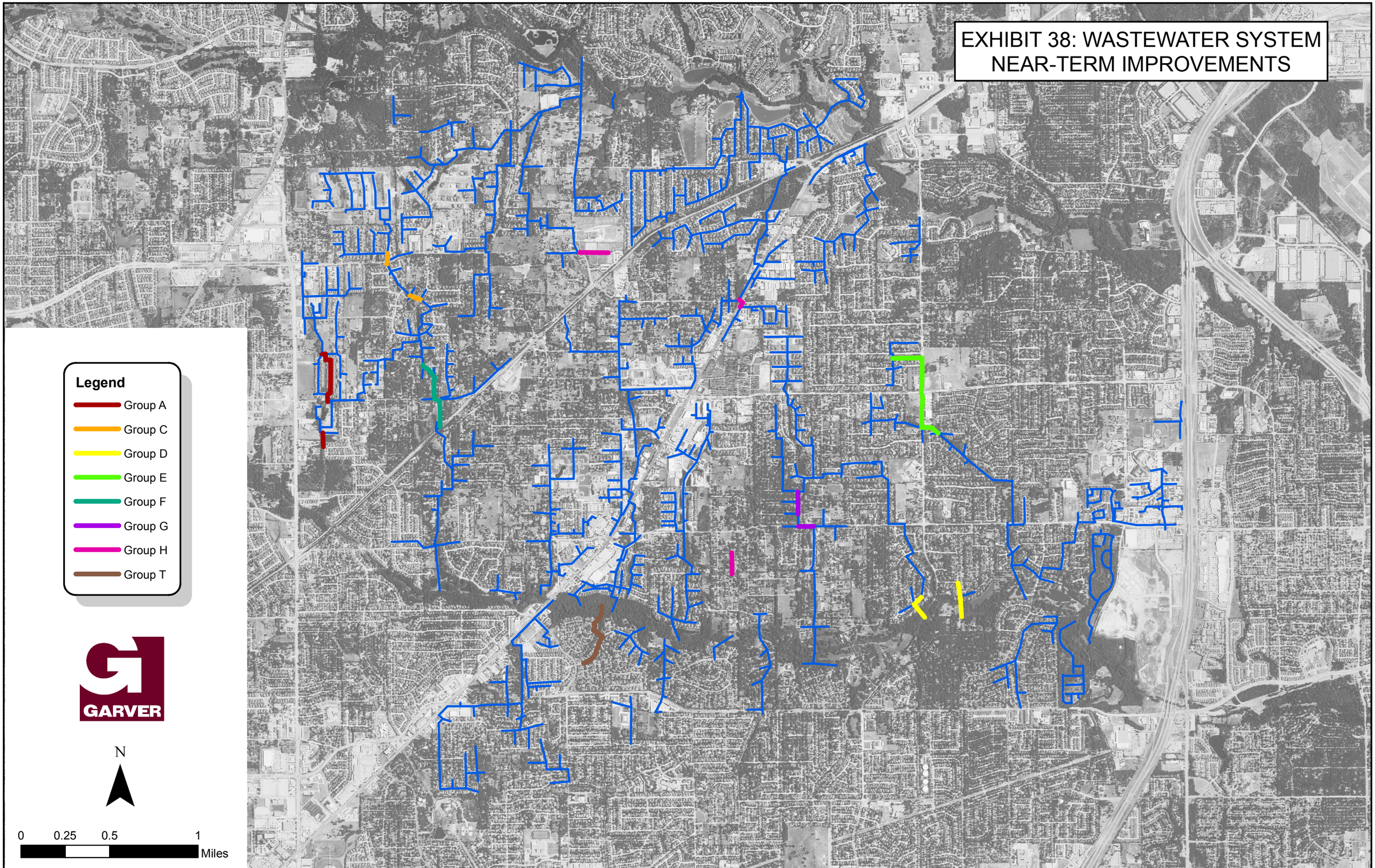


EXHIBIT 37: FUTURE SYSTEM
WET WEATHER FLOW
MAXIMUM VELOCITY
Legend
Pipe Velocity
< 1.00 fps
1.00 - 1.75 fps
1.75 - 2.25 fps
2.25 - 3.50 fps
3.50 - 5.00 fps
GARVER
N
0
0.25
0.5
1
Miles

EXHIBIT 38: WASTEWATER SYSTEM NEAR-TERM IMPROVEMENTS



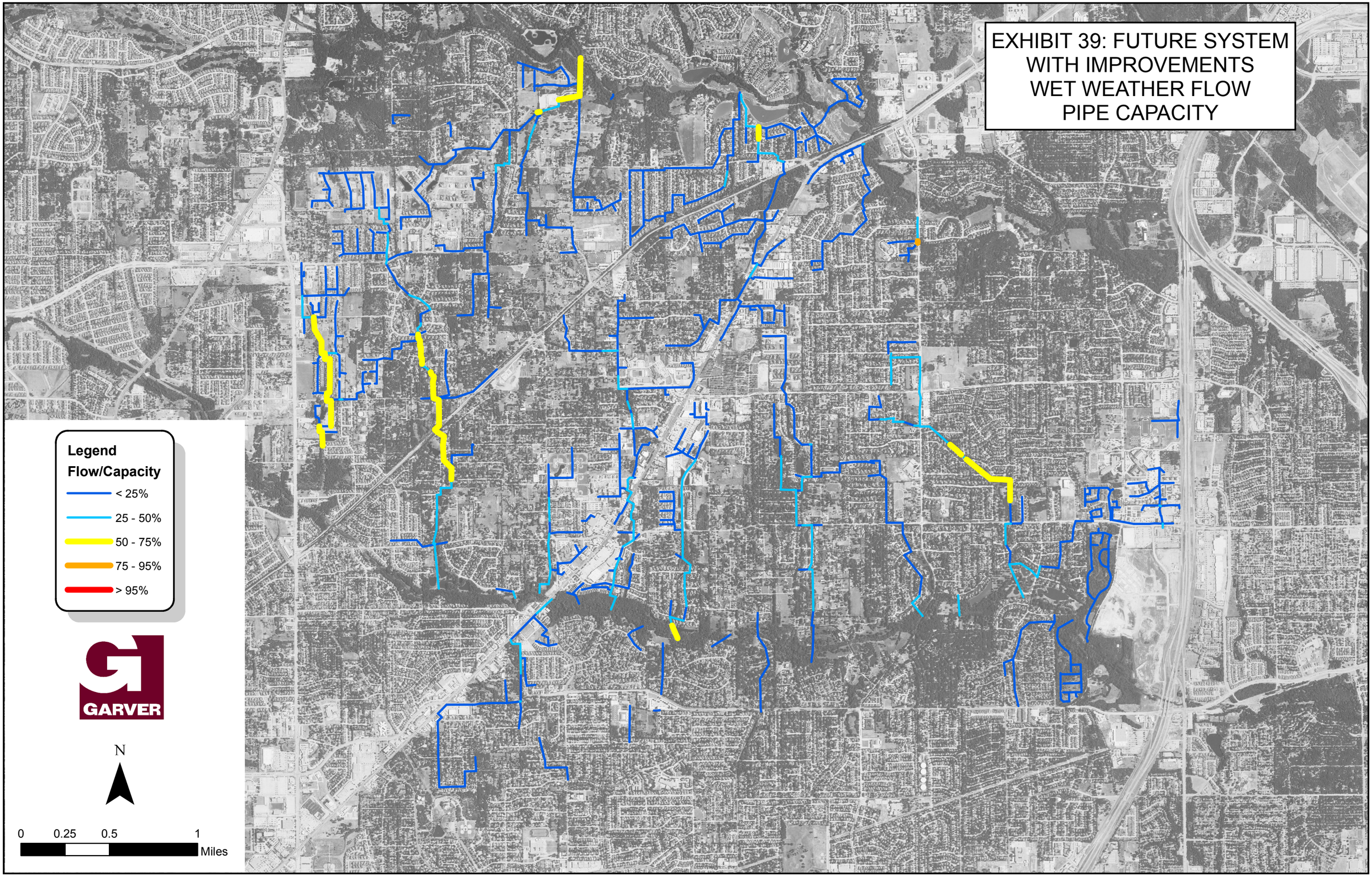


EXHIBIT 39: FUTURE SYSTEM
WITH IMPROVEMENTS
WET WEATHER FLOW
PIPE CAPACITY
Legend
Flow/Capacity
< 25%
25 - 50%
50 - 75%
75 - 95%
> 95%
GARVER
N
0
0.25
0.5
1
Miles

EXHIBIT 40: OVERLAND TRAIL LIFT STATION
GRAVITY BYPASS ALTERNATIVE
BYPASS ALTERNATIVE
APPROX. 1,900 LF 6-INCH PVC
CONNECT TO EXISTING MH 1616
OVERLAND TRAIL
LIFT STATION
9 ft
14 ft
38 ft
22 ft
10 ft
15 ft
Legend
Gravity Bypass
GARVER
N
0 0.025 0.05 0.1
Miles

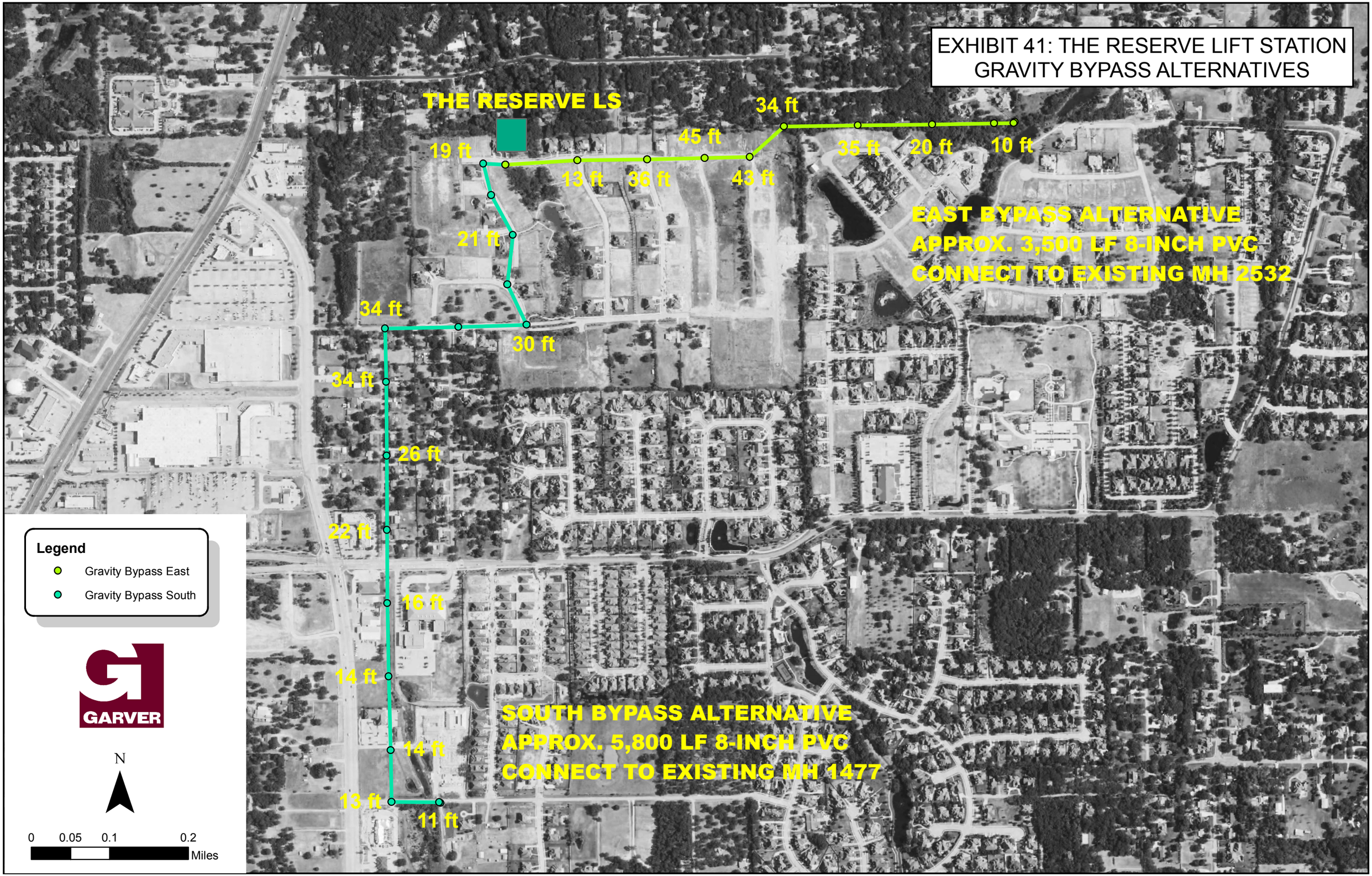


EXHIBIT 41: THE RESERVE LIFT STATION
GRAVITY BYPASS ALTERNATIVES
THE RESERVE LS
19 ft
13 ft
36 ft
45 ft
43 ft
34 ft
35 ft
20 ft
10 ft
21 ft
34 ft
30 ft
34 ft
26 ft
22 ft
16 ft
14 ft
14 ft
13 ft
11 ft
EAST BYPASS ALTERNATIVE
APPROX. 3,500 LF 8-INCH PVC
CONNECT TO EXISTING MH 2532
SOUTH BYPASS ALTERNATIVE
APPROX. 5,800 LF 8-INCH PVC
CONNECT TO EXISTING MH 1477
Legend
Gravity Bypass East
Gravity Bypass South
GARVER
N
0 0.05 0.1 0.2
Miles