CITY OF KERRVILLE

Water and Wastewater Master Plan Update

VOLUME I

PREPARED FOR:
City of Kerrville

OCTOBER 2022

PREPARED BY:
Freese and Nichols, Inc.
10431 Morado Circle, Suite 300
Austin, Texas 78759
(512) 617-3100

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10/26/2022

FREESE AND NICHOLS, INC.
TEXAS REGISTERED
ENGINEERING FIRM
F-2144

City of Kerrville
701 Main Street
Kerrville, TX 78028

FREESE AND NICHOLS, INC.
10431 Morado Circle, Suite 300
Austin, Texas 78759
(512) 617-3100

FNI Project Number: KER21213

TABLE OF CONTENTS

List of Figures

List of Tables

APPENDICES (VOLUME II)

EXECUTIVE SUMMARY

The City of Kerrville (City) contracted with Freese and Nichols, Inc. (FNI) in 2021 to provide an update to the Water Master Plan and Wastewater Master Plan. The goals of this study were to evaluate the capacity of the existing water distribution and wastewater collection systems and to recommend water and wastewater capital improvement plans (CIP). The recommended improvements will serve as a basis for the design, construction, and financing of facilities required to meet the City's water and wastewater capacity needs as a result of the projected population growth and commercial development.

Growth projections were developed through collaboration between FNI and the City, utilizing information from the 2018 Kerrville Long Range Water Supply Plan (LRWSP) and the Kerrville 2050 Comprehensive Plan. The annual growth rate was estimated using data from the 2018 Long Range Water Supply Plan. This overall growth rate was used to project the growth in connections throughout all planning periods and as a guide to allocate developments to projected planning periods. Four growth scenarios were developed based on different locations of potential development. Projected connection counts were developed for the planning years of 2022, 2027, 2032, and 2047. **Table ES-1** and **Table ES-2** present the growth projections by planning year for the City's Water and Wastewater Service Areas.

Table ES-1: Water Growth Projections

Scenario	Connections				Growth Rate
	Existing	5-year	10-year	25-year	
A	11,338	12,145	13,007	15,965	1.38%
B	11,338	12,045	12,796	15,330	1.21%
C	11,338	11,958	12,611	14,771	1.06%
D	11,338	12,076	12,872	15,574	1.28%
System Wide	**11,338**	**11,969**	**12,635**	**14,863**	**1.09%**

Table ES-2: Wastewater Growth Projections

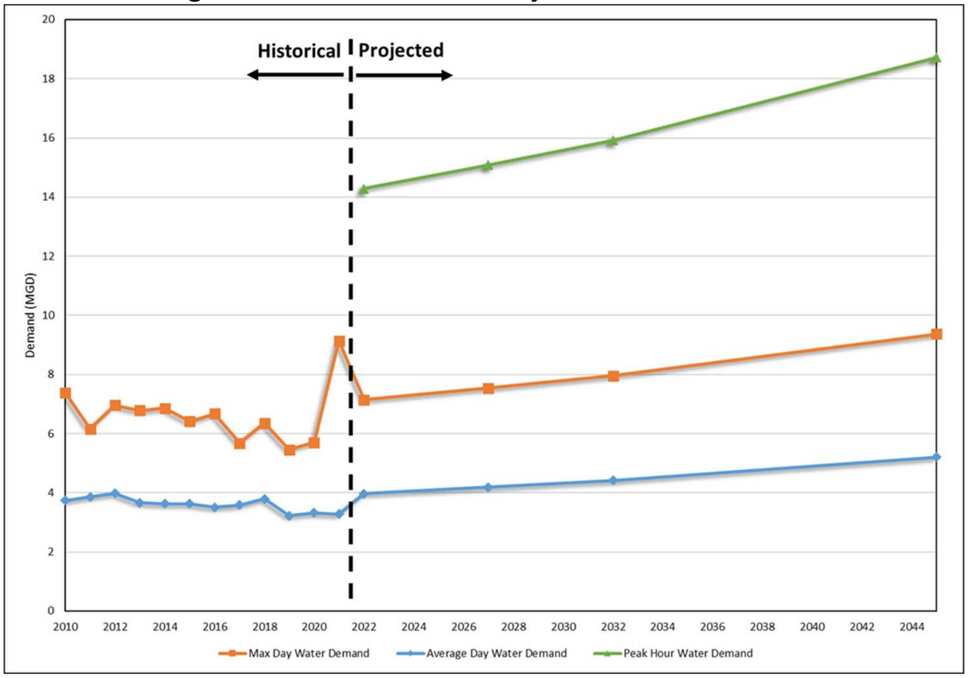
Scenario	Connections				Growth Rate
	Existing	5-year	10-year	25-year	
A	10,701	11,508	12,370	15,328	1.45%
B	10,701	11,408	12,159	14,693	1.28%
C	10,701	11,321	11,974	14,134	1.12%
D	10,701	11,439	12,235	14,937	1.34%
System Wide	**10,701**	**11,332**	**11,998**	**14,226**	**1.15%**

The City of Kerrville owns and maintains a hydraulic water model in the InfoWater® software package from Innovyze that was created during the 2013 Water Master Plan. FNI updated the existing water model with current GIS data. Field pressure testing was conducted from August 10th to August 20th, 2021, gathering data used to calibrate the updated model. During the calibration, adjustments were made to the model to match the observed conditions of August 14th, 2020. Results suggest a good correlation between recorded and modeled values. This degree of calibration provides confidence in the accuracy of the model and is considered suitable for the development of a water system CIP and master planning purposes.

FNI developed a hydraulic model to be used as a tool for evaluating the wastewater collection system using InfoSewer® software by Innovyze. FNI retained ADS Environmental Services (ADS) to conduct temporary flow monitoring within selected portions of the existing sanitary sewer system. The temporary flow monitors were deployed for a total of 58 days from May 11, 2021 to July 8, 2021. Data gathered during flow monitoring was used to calibrate the hydraulic wastewater model. On average, the hydraulic model is calibrated to observed dry weather conditions within 1% and calibrated to observed wet weather conditions within 3%.

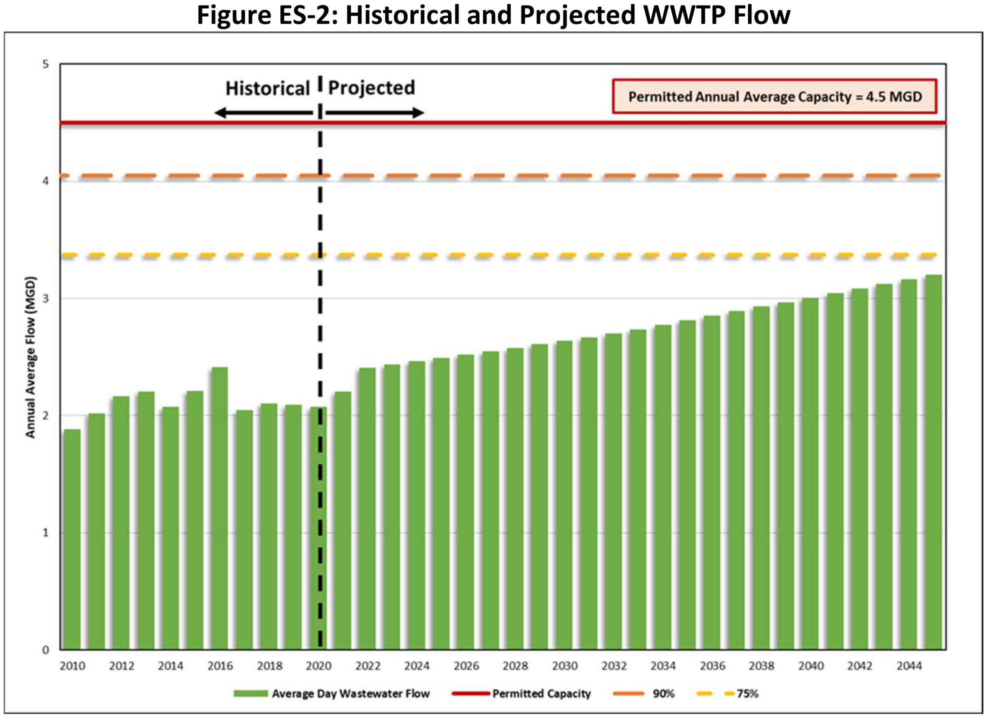
Historical water demand data was used to select planning criteria for projected water demand. These planning criteria were applied to the projected growth scenarios to estimate future demand in the water system. Historical water demand and projected water demand are shown in **Figure ES-1**.

Figure ES-1: Historical and Projected Water Demand



Historical wastewater flow data was used to select planning criteria for projected future wastewater flow. These planning criteria were applied to the projected growth scenarios to estimate future flow in the wastewater system. Historical WWTP flow and projected WWTP flow are shown in **Figure ES-2**.

Figure ES-2: Historical and Projected WWTP Flow



As a public water utility, the City must comply with the rules and regulations for public water systems set forth by the Texas Commission on Environmental Quality (TCEQ) in TAC §290.45. The TCEQ sets minimum system requirements pertaining to production, pumping, and storage capacity. TCEQ minimum system requirements and the recommended planning criteria were utilized to evaluate the 2022 water system. The evaluation indicates that the 2022 water system is meeting all TCEQ minimum system requirements.

In addition to the pumping and storage evaluations, the calibrated model was used to conduct a hydraulic analysis under 2022 maximum day and peak hour demand conditions to evaluate system operations and residual pressure throughout the distribution system. The TCEQ minimum required pressure within a distribution system is 35 pounds per square inch (psi) under non-emergency demand conditions. Most of the system meets the minimum requirement, but small areas in the Stadium Pressure Plane, along the boundary between Stadium and College Cove Pressure Plane, show modeled pressure below 35 psi.

TCEQ minimum system requirements and the recommended planning criteria were utilized to evaluate the future water system and develop system improvements. Additionally, available fire flow and water age model simulations were conducted to identify any deficiencies and needed improvements.

Hydraulic analyses were also conducted to identify deficiencies in the City's existing wastewater collection system and to establish a capital improvement plan to improve the existing system and accommodate projected wastewater flows through the 2047 planning period. First, collection system infrastructure identified for improvement includes pipes where the existing or projected flow (q) exceeds the full capacity of the pipe (Q) calculated by Manning's equation based on the diameter, slope, and Manning's roughness coefficient (typically 0.013). Second, the system is allowed to utilize the system storage capacity by permitting a degree of surcharging while still preventing potential sanitary sewer overflows (SSOs). Evaluation of lift stations is primarily based on the firm pumping capacity of the lift station. Force mains with projected velocities greater than 8 fps are identified for improvement.

The hydraulic wastewater model was utilized to apply the existing and future projected peak wet weather flows to the wastewater collection system to identify potential system capacity restrictions.

In addition to the capacity analysis, FNI performed a risk-based assessment (RBA) of the water and wastewater systems. In March 2022, FNI and City staff conducted field assessments of 16 of the water system pump stations and wells, and 6 of the wastewater system lift stations. The assessment included site visits and visual inspection of the facilities to determine the overall condition of the station and specific condition of major components including pumps and motors, electrical equipment, instrumentation, piping and valves, structure, and site.

General water system recommendations include replacing gaseous chlorine with liquid bleach, constructing appropriate buildings to protect production well pumps and piping similar to Methodist Encampment, provide lighted canopies over exposed electrical boxes, conducting electrical power system study, and re-coating exposed piping and valves.

Airport Commerce Park and Al Mooney Lift Stations are identified for rehabilitation projects intended to address corrosion issues and poor site conditions. Quinlan Lift Station is identified for a rehabilitation project intended to address debris issues. General recommendations for lift stations include implementing backup power and backup controls for critical, high flow, or remote sites.

Pipeline material, diameter, and work order history was available for the waterline assessment. FNI utilized the available data to identify and prioritize an annual waterline replacement program. FNI and City Staff identified cast-iron water mains to be the category of pipeline most at risk of critical failure in the water distribution system. FNI developed 25 replacement project areas, containing approximately 6,300 linear feet of cast-iron main, prioritized based on work order density and pipeline diameter.

Pipeline material and diameter data was also available for the wastewater collection system. Due to the lack of detailed condition data on the wastewater pipelines, FNI does not recommend an annual pipeline replacement program for the wastewater collection system. FNI recommends utilizing CCTV inspection within the wastewater system to identify specific points of failure and populate more information for use in future analysis. Analysis of observed flow monitoring data identifies areas of high RDII. Relatively high rates of RDII can be a good indicator of general condition issues in a given flow monitoring basin as broken manholes and cracks in gravity mains are direct contributors to inflow and infiltration. FNI recommends additional annual budget for use in general wastewater renewal projects and inspection.

Capital improvement plans were developed to address existing system deficiencies and near-term needs, regardless of population growth. Additional projects were developed for each of the growth scenarios to identify infrastructure needed to serve new customers. These additional development-driven projects would not be needed until growth occurs and would likely be part of a cost-sharing agreement with developers. **Table ES-3** and **ES-4** show a summary of the water and wastewater capital improvements, respectively.

Table ES-3: Water CIP Project Cost Summary

Project Number	Project Name			Project Cost
Capital Improvement Plan				
1	High Service Pump Station Expansion			$661,800
2	Targeted Pipeline Replacement			$5,254,900
3	12-inch Legion Dr Water Line			$3,454,400
4	12-inch Veterans Highway Water Line Guadalupe River Crossing			$511,900
5	Ridgewood Fire Flow Improvements			$923,900
6	College Cove Fire Flow Improvements			$688,000
7	H Street Well Renewal			$385,100
8	Methodist Encampment Well Renewal			$491,400
9	Comanche Trace PRV Replacement			$982,800
10	Hydropneumatic Zone Replacement			$4,266,000
			Capital Improvement Plan Total	**$17,620,200**
Annual Renewal Plan				
		Annual Cost	**Years**	**Total Cost**
11	General Water Distribution System Renewal	$100,000	10	$1,000,000
12	Annual Pipeline Replacement Program	$2,009,948	25	$50,248,700
			Annual Renewal Plan Total	**$51,248,700**

Table ES-4: Wastewater CIP Project Cost Summary

Project Number	Project Name			Project Cost
Capital Improvement Plan				
1	Knapp FM and Interceptor			$6,374,692
2	New Knapp Lift Station			$3,049,500
3	Al Mooney Lift Station Renewal			$765,600
4	Airport Commerce Lift Station Renewal			$464,600
5	Quinlan Manhole Replacement			$894,400
6	Quinlan Lift Station Renewal			$206,400
7	Quinlan Interceptor Reroute			$4,790,400
8	Comanche Trace Lift Station Expansion			$499,000
9	Ingram Interceptor Expansion			$19,068,700
			Capital Improvement Plan Total	**$36,113,292**
Annual Renewal Plan				
		Annual Cost	**Years**	**Total Cost**
10	Annual Wastewater System Renewal	$100,000	10	$1,000,000
			Annual Renewal Plan Total	**$1,000,000**

1.0 INTRODUCTION

The City of Kerrville (City) contracted with Freese and Nichols, Inc. (FNI) in 2021 to provide an update to the Water Master Plan and Wastewater Master Plan. The goals of this study were to evaluate the capacity of the existing water distribution and wastewater collection systems and to recommend water and wastewater capital improvement plans (CIP). Projected development assumptions were determined based on the Kerrville 2050 Comprehensive Plan and in coordination with City staff and were utilized to project water demand and wastewater flow. Hydraulic models of the water distribution system and wastewater collection systems were updated using a combination of geographic information system (GIS) data and as-built records. The recommended improvements will serve as a basis for the design, construction, and financing of facilities required to meet the City's water and wastewater capacity needs as a result of the projected population growth and commercial development.

1.1 SCOPE OF WORK

The major elements of the scope of this project include:

- Water Model Update and Calibration
- Wastewater Flow Monitoring
- Wastewater Model Update and Calibration
- Water Demand and Wastewater Flow Projections
- Existing and Future Water Distribution and Wastewater Collection Systems Hydraulic Analysis
- Risk Based Condition Assessment
- Water and Wastewater System Capital Improvement Plan

1.2 ACRONYMS

Table 1-1 presents a list of acronyms that appear throughout the report.

Table 1-1: Acronyms

Acronym	Definition
AACE	American Association of Cost Engineers
AADF	Annual Average Daily Flow
AD	Average Day
ADS	ADS Environmental Services
ASR	Aquifer Storage and Recovery
CCN	Certificate of Convenience and Necessity
CIP	Capital Improvement Plan
d/D	Depth to Diameter Ratio
EPS	Extended Period Simulation
EST	Elevated Storage Tank
ETJ	Extraterritorial Jurisdiction
FNI	Freese and Nichols, Inc.
fps	Feet per Second
gpcd	Gallons per connection per day
gpm	Gallons per Minute
GIS	Geographic Information System
GST	Ground Storage Tank
HGL	Hydraulic Grade Line
I/I	Inflow and Infiltration
LF	Linear Feet
LS	Lift Station
LUE	Living Unit Equivalent
MD	Maximum Day
MD:PH	Maximum Day to Peak Hour
MG	Million Gallons
MGD	Million Gallons Per Day
NOAA	National Oceanic and Atmospheric Administration
PH	Peak Hour
PRV	Pressure Reducing Valve
PS	Pump Station
psi	pounds per square inch
PZ	Pressure Zone
q/Q	Modeled flow divided by full flow capacity
RBA	Risk Based Analysis
RDII	Rainfall Derived Inflow and Infiltration
SCADA	Supervisory Control and Data Acquisition
SSO	Sanitary Sewer Overflow
TAC	Texas Administrative Code
TCEQ	Texas Commission on Environmental Quality
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

1.3 KEY DEFINITIONS

The following is a list of key definitions utilized in the Report.

- Annual Average Flow – the total cumulative flow through a WWTP over any given year divided by the number of days in the year. It is considered the average flow condition for a given year.
- Average Day Demand – the total annual water use divided by the number of days in the year.
- Capacity Improvement – general project to improve the ability of the water/wastewater system to convey projected water demands/wastewater flows.
- Capacity Utilization (q/Q) – the maximum observed flow through a gravity pipe divided by the calculated full pipe capacity. It is a unitless ratio or percentage and can be greater than one under surcharged conditions.
- Maximum Day Demand – the maximum quantity of water used system-wide on any one day of the year.
- Depth to Diameter Ratio (d/D) – the maximum observed depth of flow within a gravity pipe divided by the diameter of the pipe. It is a unitless ratio or percentage and can be greater than one under surcharged conditions.
- Diurnal Pattern – a typical pattern of water demand/wastewater flow that occurs every 24 hours, dependent on the demographics of the area.
- Elevated Storage Tank – a water storage tank that sets the hydraulic grade of the pressure zone in which it operates. Tank can be either a true elevated tank (i.e., raised on a structure, typically 50-200 feet in the air) or a ground tank located at an elevation high enough to provide adequate pressure.
- Firm Pumping Capacity – the pumping capacity of a pump station or lift station with the largest pump offline and all other pumps running, based on recorded flows at the best efficiency point or rated capacity of each pump.
- Inflow and Infiltration (I/I) – the primary means of stormwater or groundwater entering into the wastewater collection system. Inflow is a fast response such as an open manhole lid while infiltration is a slow response such as cracks along the pipeline.
- Peak Hour Demand – the peak rate at which water is required during any one hour of the year.

- Peaking Factor – recorded or projected peak flow divided by the average flow.
- Pressure – a measure of energy at a given location and time in the water/wastewater system, typically measured in pounds per square inch (psi).
- Pressure Zone – an area of the water system that operates at a particular hydraulic grade.
- Hydraulic Grade Line (HGL) – Energy of water or wastewater flow expressed in feet above mean sea level (i.e., head). Wastewater surface elevation in a partially full pipe or manhole. Under pressure (full pipe) conditions the HGL is what the water surface elevation would be if unconfined.
- Rainfall Derived Inflow and Infiltration – similar to I/I, RDII is the increase in flow in a wastewater collection system specifically in response to a wet weather storm event.
- Risk Based Analysis – An analysis method utilizing results of condition assessments to score water and wastewater facilities and linear assets. Data from facility site visits and recorded linear asset maintenance history are used to determine condition.
- Sanitary Sewer Overflow (SSO) – A type of unauthorized discharge of untreated or partially treated wastewater from a collection system or its components (e.g., a manhole, lift station, or cleanout) before reaching a treatment facility. (See also Texas Water Code 26.049(e)(4).)
- Surcharging – when the HGL is greater than the top of pipe (d/D greater than 1), often due to a capacity restriction downstream or within the pipe itself. It can lead to SSOs.

2.0 EXISTING WATER SYSTEM

The existing water distribution system consists of a network of lines ranging in size from 1 inch to 16 inches in diameter, seven pump stations (PS), three elevated storage tanks (EST), and eight ground storage tanks (GST), six of which act as elevated storage. The existing water distribution system is shown on **Figure 2-1**. **Figure 2-2** shows the water system in schematic form.

2.1 PRESSURE ZONES

The City currently operates six main pressure zones: Stadium, Methodist, Kerrville North, Ridgewood, College Cove, and Summit. With these six main pressure zones, the water system serves customers at ground elevations ranging from approximately 1,560 feet to 1960 feet. **Table 2-1** shows the overflow hydraulic grade line (HGL) of the pressure zones. Kerrville North, Ridgewood, and College Cove PZ all serve areas that include service elevations below the recommended minimum allowable service elevation. These areas are generally served either by mainline or individual pressure reducing valves (PRV).

Table 2-1: Existing Pressure Zone Overview

Pressure Zone	Overflow HGL (feet)
Stadium	1,814
Methodist	1,966
Kerrville North	1,966
Ridgewood	2,002
College Cove	1,942
Summit	2,076

The Stadium Pressure Plane consists of the center of the City and is the largest pressure plane, accounting for approximately 62% of the City's total water usage. The Stadium Pressure Plane operates at a static hydraulic gradient of 1,814 feet.

The Methodist Pressure Plane consists of the northwestern portion of the City and is the second largest pressure plane, accounting for approximately 19% of the City's total water usage. The Methodist Pressure Plane operates at a static hydraulic gradient of 1,966 feet. West Bluff, Hilltop and The Heights hydropneumatic systems serve three high elevation areas of the Methodist Pressure Plane.

The Kerrville North Pressure Plane is in the central north portion of the City. The Kerrville North Pressure Plane operates at a static hydraulic gradient of 1,966 feet. Though the Kerrville North and Methodist Pressure Planes have the same hydraulic gradient and are connected hydraulically by a 12-inch water line, a closed valve on the water line maintains separate pressure planes. The Keystone hydropneumatic system serves the high elevation area of the Kerrville North Pressure Plane. The Hillcrest PRV zone serves the low elevation area of the Kerrville North Pressure Plane.

The Ridgewood Pressure Plane is a relatively small area in the southern portion of the City. The Ridgewood Pressure Plane is operated at a static hydraulic gradient of 2,002 feet. Comanche Trace, Comanche Trace Lower and Stone Ridge PRV zones serve three low elevation areas of the Ridgewood Pressure Plane.

The College Cove Pressure Plane is a relatively small area on the eastern side of the City. The College Cove Pressure Plane is operated at a static hydraulic gradient of 1,942 feet. The College Cove PRV zone serves the low elevation area of the College Cove Pressure Plane.

The Summit Pressure Plane includes the area in the northeastern portion of the City. A static hydraulic gradient of 2,076 feet is established by one elevated storage tank.

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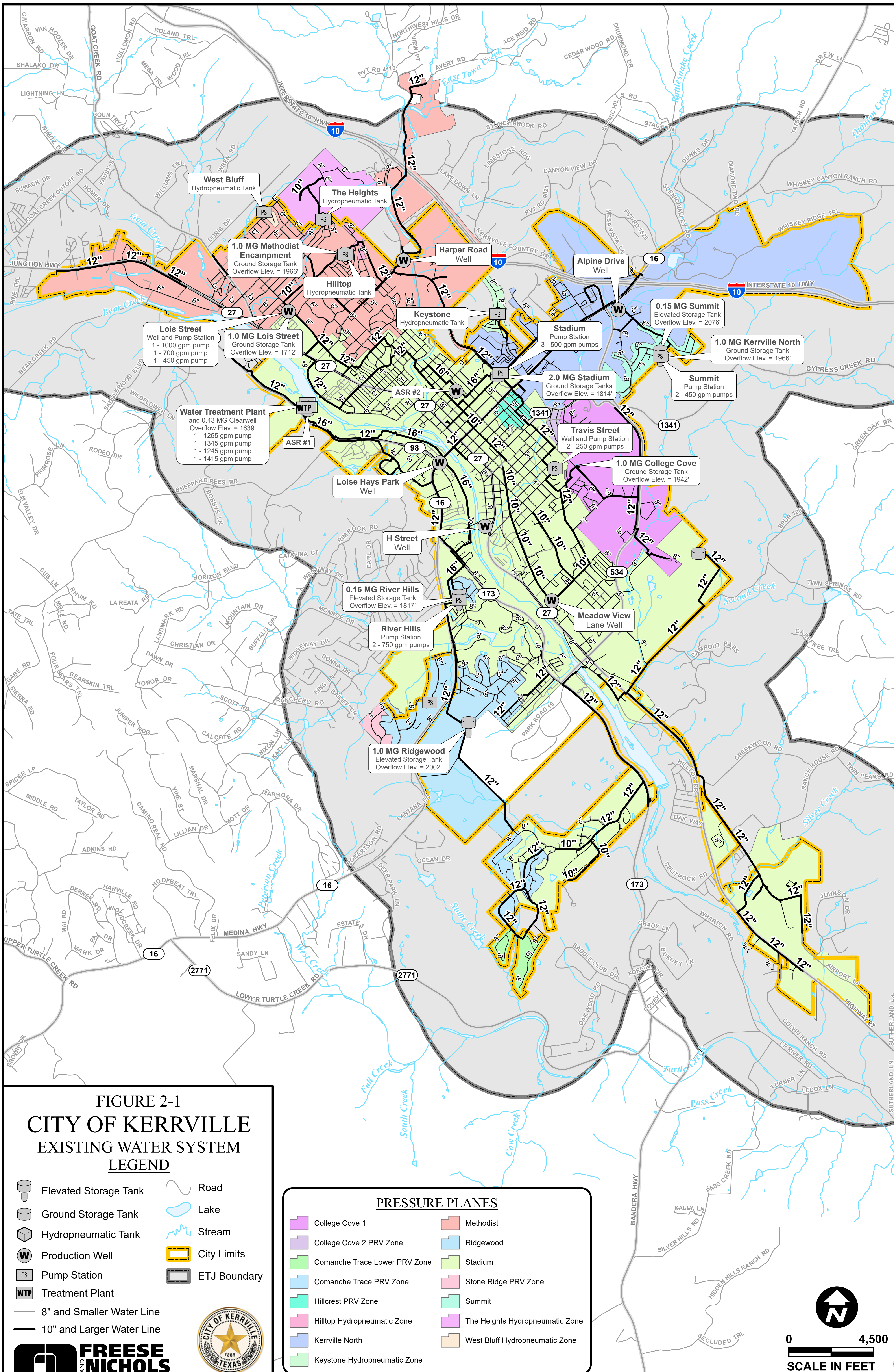
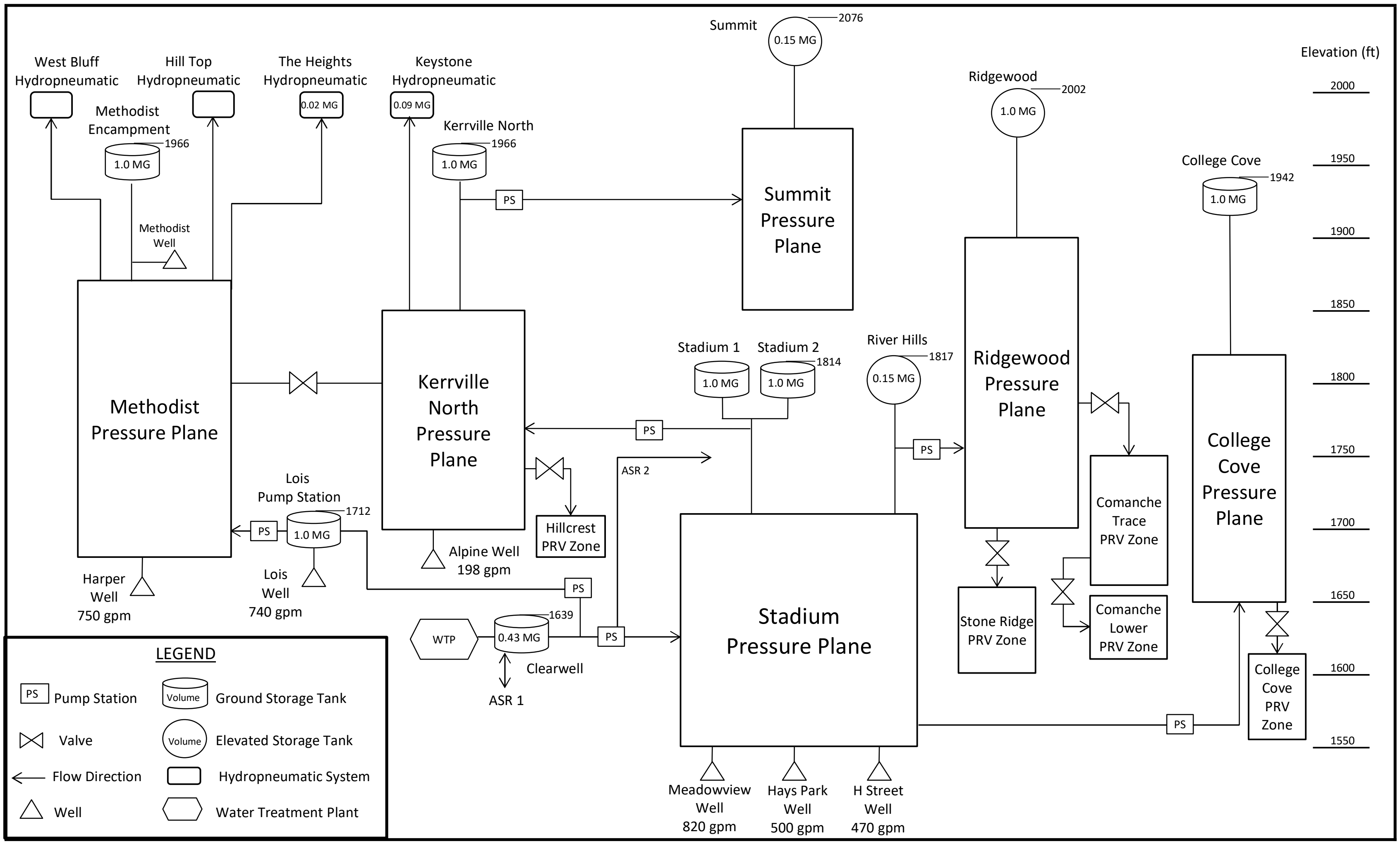


FIGURE 2-1
CITY OF KERRVILLE
EXISTING WATER SYSTEM

Figure 2-2 Existing Water System Schematic



2.2 PRODUCTION FACILITIES

The City's water supply consists of surface water from the Guadalupe River, native groundwater, and water from Aquifer Storage and Recovery (ASR) wells. The surface water is treated at the Water Treatment Plant (WTP), which has a treatment capacity of 5.54 MGD, and the Membrane Treatment Plant (MTP), which has a treatment capacity of 1.15 MGD. The City is currently permitted to use 5.40 MGD annually of surface water from the Guadalupe River. However, during low flow conditions in the river the City may not be allowed to use the full permitted supply. To help offset the surface water seasonal restrictions, the City injects treated water into a groundwater aquifer at the ASR facility during low demands or high river flows to store water that can be used when needed throughout the year. A high service pump station at the plant supplies water to the distribution system. The WTP site includes an ASR well, ASR 1. An additional ASR well, ASR 2, is located within the Stadium Pressure Plane. The remaining groundwater wells pump directly into the distribution system. Stadium Pressure Plane, Methodist Pressure Plane, and Kerrville North Pressure Plane all contain groundwater wells. **Table 2-2** shows the capacity of each production facility in the City's water system.

Table 2-2: Existing Production Facilities

Name	Type	Capacity (gpm)	Capacity (MGD)	Pressure Zone Served
Water Treatment Plant	WTP	3,845	5.54	--
Membrane Treatment Plant	WTP	800	1.15	--
ASR 1	ASR	750	1.08	--
ASR 2	ASR	1,000	1.44	Stadium
Alpine	Well	133	0.19	Kerrville North
H Street*	Well	540	0.78	Stadium
Harper Road	Well	340	0.49	Methodist
Lois Street	Well	560	0.81	Methodist
Hays Park	Well	350	0.50	Stadium
Meadow View Lane	Well	730	1.05	Stadium
Methodist*	Well	900	1.30	Methodist
Loop 534 Ellenburger	Well	700	1.01	Stadium
Travis Street*	Well	400	0.57	Stadium
Total	**--**	**9,208**	**13.26**	**Systemwide**

*Well is offline and does not contribute to system production capacity

2.3 DISTRIBUTION PUMPING FACILITIES

The City utilizes pump stations to convey water to pressure zones at higher hydraulic grades. Most of the water supply enters the system in the Stadium PZ, except for the Alpine, Harper Road, Lois Street, and Methodist wells. The Stadium PZ is the largest in the system, both in terms of demand and area, but operates at the lowest hydraulic grade. Therefore, pump stations are used to supply water to the subsequent zones in the system. In addition, pump stations are also utilized when a ground storage tank is used to store water from a supply source rather than directly entering the distribution system. **Table 2-3** presents a summary of pump stations by pressure zone.

Table 2-3: Existing Distribution Pumping Facilities

Pump Station	Pump Number	Rated Capacity (gpm)	Rated Capacity (MGD)	Firm Capacity (gpm)	Pressure Plane
High Service	1	1,200	1.73	1,200	Lois GST
	2	1,200	1.73		Lois GST
	3	1,500	2.16	2,900	Central Stadium
	4	1,500	2.16		Central Stadium
	5	1,400	2.02		South Stadium
Lois Street	1	450	0.65	1,150	Methodist
	2	700	1.01		Methodist
	3	1,000	1.44		Methodist
Stadium	1	500	0.72	1,000	Kerrville North
	2	500	0.72		Kerrville North
	3	500	0.72		Kerrville North
Summit	1	450	0.65	450	Summit
	2	450	0.65		Summit
Travis Street	1	250	0.36	250	College Cove
	2	250	0.36		College Cove
River Hills	1	750	1.08	750	Ridgewood
	2	750	1.08		Ridgewood
Hilltop	1	600	0.86	600	Hilltop
	2	600	0.86		Hilltop
Keystone	1	500	0.72	500	Keystone
	2	500	0.72		Keystone
The Heights	1	530	0.76	530	The Heights
	2	530	0.76		The Heights
West Bluff	1	50	0.07	50	West Bluff
	2	70	0.10		West Bluff
Total	**--**	**16,730**	**24**	**9,380**	**Systemwide**

2.4 STORAGE TANKS

Elevated storage of water generally refers to a volume of water stored at an elevation such that gravity can be utilized to maintain appropriate system pressure. Elevated storage tanks can be constructed with legs or a pedestal to achieve the desired elevation, or they can be ground storage tanks constructed at elevations higher than the area they serve. TAC §290.38(25) defines elevated storage as "that portion of water which can be stored at least 80 feet above the highest service connection in the pressure [zone] served by the storage tank." Ground storage of water is generally utilized at pump stations to allow for consistent suction head and to provide storage of an additional volume of water that can supply the pump station. Hydropneumatic tanks provide storage for smaller hydropneumatic pressure zones and maintain a supply of pressurized water.

Currently, the City operates eight ground storage tanks (GSTs) within the distribution system, two of which provide traditional ground storage for the High Service and Lois Street Pump Stations. The remaining six GSTs are located at higher ground elevations and provide elevated storage for Methodist, Kerrville North, Stadium, and College Cove Pressure Planes. The City also operates three elevated storage tanks (ESTs). The Summit EST serves the Summit Pressure Plane, River Hills EST is located in the Stadium Pressure Plane, and the Ridgewood EST serves the Ridgewood Pressure Plane. Additionally, the City maintains four hydropneumatic tanks that serve customers in the four corresponding hydropneumatic zones. **Table 2-4** summarizes the City's existing ground storage tanks. **Table 2-5** summarizes the City's existing elevated storage tanks, and **Table 2-6** summarizes the City's hydropneumatic tanks.

Table 2-4: Existing Ground Storage Tanks

Tank	Type	Storage Capacity (MG)	Pressure Plane
Clear Well	Clear Well	0.43	Stadium
Lois Street	Ground	1.00	Methodist

Table 2-5: Existing Elevated Storage Tanks

Tank	Type	Overflow Elevation (ft)	Sidewater Depth (ft)	Storage Capacity (MG)	Pressure Plane
Methodist	Ground as Elevated	1,966	56	1.00	Methodist
Stadium 1	Ground as Elevated	1,814	38	1.00	Stadium
Stadium 2	Ground as Elevated	1,814	48	1.00	Stadium
Kerrville North	Ground as Elevated	1,966	20	1.00	Kerrville North
College Cove	Ground as Elevated	1,942	38	1.00	College Cove
River Hills	Elevated	1,817	30	0.15	Stadium
Ridgewood	Elevated	2,002	40	1.00	Ridgewood
Summit	Elevated	2,076	30	0.15	Summit
Total	--	--	--	**6.30**	**Systemwide**

Table 2-6: Existing Hydropneumatic Tanks

Tank	Type	Storage Capacity (gal)	Pressure Plane
Hilltop	Hydropneumatic	10,000	Hilltop
Keystone	Hydropneumatic	90,000	Keystone
The Heights	Hydropneumatic	20,000	The Heights
West Bluff	Hydropneumatic	500	West Bluff

2.5 WATER DISTRIBUTION SYSTEM

The City's existing water distribution system consists of approximately 220 miles of water lines. Pipeline diameters range in size from 0.75-inch to 16-inches. **Table 2-7** provides a breakdown of the linear footage (LF) by diameter of pipe. The majority of the pipes are 6-inch in diameter.

Table 2-7: Water Line Summary

Diameter (inches)	GIS Length (feet)	GIS Length (miles)	Percent of Total (%)
0.75	353	0.1	0.0%
1	1,648	0.3	0.1%
1.25	1,475	0.3	0.1%
1.5	1,014	0.2	0.1%
2	47,301	9.0	4.1%
3	34,871	6.6	3.0%
4	22,250	4.2	1.9%
6	472,764	89.5	40.8%
8	269,536	51.0	23.2%
10	60,754	11.5	5.2%
12	208,912	39.6	18.0%
16	38,617	7.3	3.3%
Total	**1,159,494**	**220**	**100.0%**

3.0 EXISTING WASTEWATER SYSTEM

The existing wastewater collection system consists of a network of gravity mains, lift stations, force mains, and a wastewater treatment plant that provides wastewater service to the City's customers. **Figure 3-1** displays the existing wastewater collection system.

3.1 WASTEWATER TREATMENT PLANTS

The wastewater collection system is served by one wastewater treatment plant. The Wastewater Treatment Plant (WWTP) is located along Loop 534 on the east side of the city, with a total permitted treatment capacity of 4.5 MGD.

3.2 LIFT STATIONS

Lift stations are necessary when wastewater needs to be pumped to a higher elevation where the flow can resume flowing by gravity to the outfall of the system. Due to the varying topography, Kerrville operates 26 lift stations throughout the service area. **Table 3-1** provides a summary of each lift station. The firm pumping capacity is the pumping capacity of the lift station with the largest pump out of service. The lift stations vary in size from small development lift stations near the city limits to the four large lift stations in the center of the City.

The WWTP currently receives flow from three major basins, Legion, Quinlan, and Birkdale. The Legion Lift Station serves the Legion Basin and also currently receives flow from the Airport, Broadway, and Jefferson Lift Stations. The Birkdale Lift Station receives flow from the Comanche Trace, Birkdale, and Jefferson basins. The Quinlan Lift Station receives flow from the Quinlan Basin. One force main carries flow from both Quinlan and Loop 534 Lift Stations to the WWTP, with the majority of the flow in the force main coming from the Quinlan Lift Station. The Loop 534 Lift Station is designed to handle future flows along Loop 534 north of the WWTP and currently does not contribute significant flow. The Jefferson Lift Station currently can pump to Quinlan, Legion, and Birkdale Lift Stations. The majority of the flow has historically been pumped to Legion, however in the future flow can also be pumped to Birkdale. The Quinlan connection operates as an emergency bypass and little to no flow will be pumped to Quinlan from Jefferson in the future.

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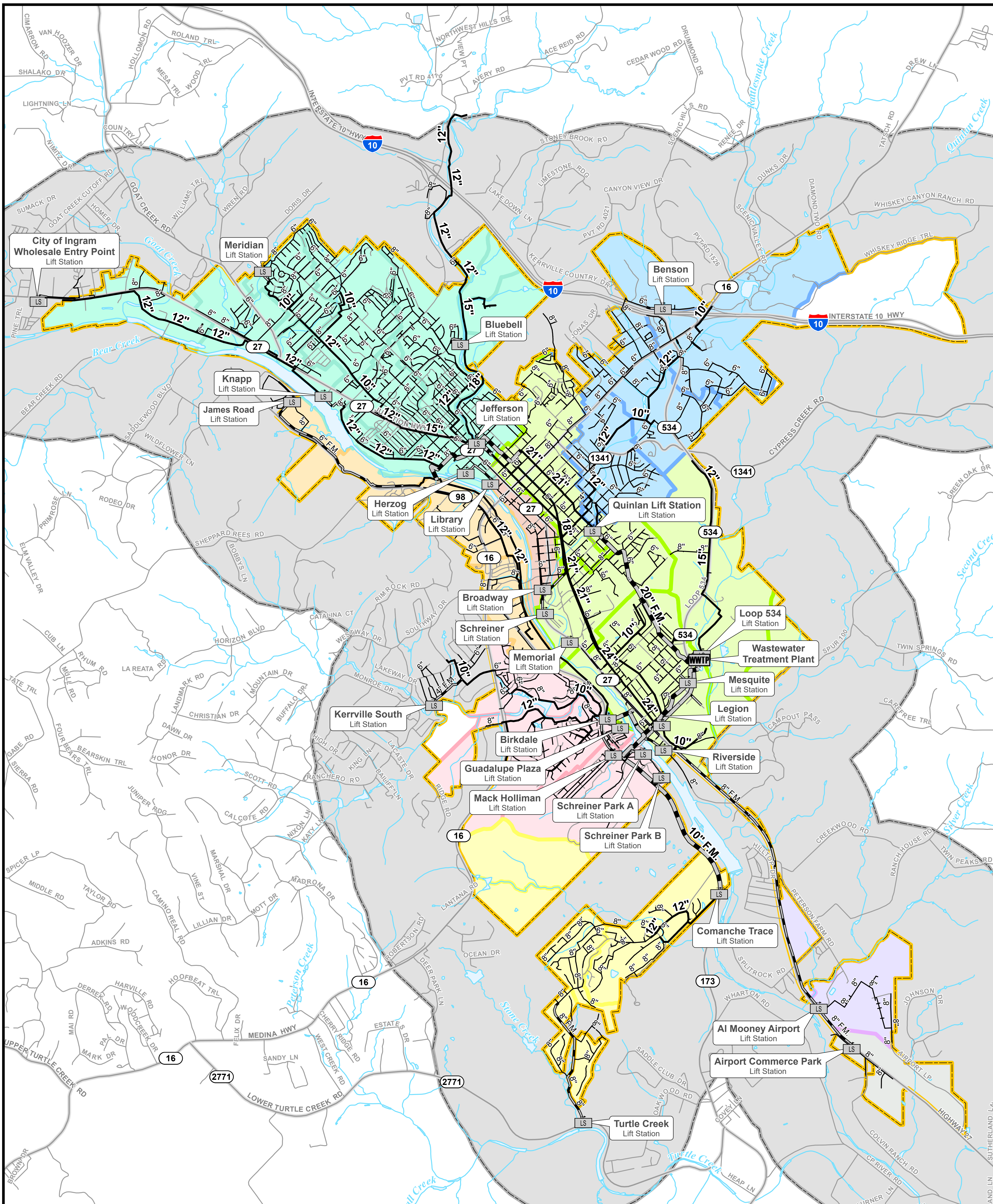


FIGURE 3-1

CITY OF KERRVILLE

WASTEWATER COLLECTION SYSTEM

LEGEND

- LS Existing Lift Station
- WWTP Wastewater Treatment Plant
- 8" and Smaller Wastewater Line
- 10" and Larger Wastewater Line
- 8" and Smaller Force Main
- 10" and Larger Force Main
- Road
- Stream
- Lake
- City Limit
- ETJ Boundary

SEWER BASINS

- Airport/Commerce Basin
- Airport/Al Mooney Basin
- Birkdale Basin
- Broadway Basin
- Comanche Trace Basin
- G-Street Basin
- Jefferson Basin
- Legion Basin
- Quinlan Basin

0 4,500
SCALE IN FEET

FREESE AND NICHOLS

CITY OF KERRVILLE TEXAS 1889

Table 3-1: Lift Station Summary

Lift Station	Firm Capacity (gpm)	Force Main Diameter (in)
Airport Commerce	150	8
Al Mooney	210	8
Benson	--	4
Birkdale	6,300	20
Bluebell	60	8
Broadway	500	8
Comanche Trace	600	10
Guadalupe Plaza	--	4
Herzog	70	6
James Road	500	6
Jefferson	6,000	16
Kerrville South	180	4
Knapp	560	6
Legion	6,300	20
Library	--	4
Loop 534	1,700	12
Mack Holliman	--	6
Memorial	1,000	--
Meridian	175	4
Mesquite	--	2
Quinlan	2,400	12
Riverside	50	8
Schreiner	150	6
Schreiner Park A	27	4
Schreiner Park B	27	4
Turtle Creek	400	8

3.3 COLLECTION SYSTEM

The City's existing wastewater collection system consists of approximately 200 miles of collector mains, interceptors and force mains. Pipeline diameters range in size from 2 inches to 30 inches. **Table 3-2** provides a breakdown of the linear footage (LF) by diameter of pipe. The majority of the system is comprised of 6-inch and 8-inch wastewater lines that commonly serve subdivisions, neighborhoods, and small commercial areas throughout the system. Larger interceptors collect the smaller mains and convey flow to the major lift stations.

Table 3-2: Wastewater Line Summary

Diameter (inches)	GIS Length (feet)	GIS Length (miles)	Percent of Total (%)
Unknown	1,994	0.4	0.2%
2	996	0.2	0.1%
4	10,762	2.0	1.0%
6	469,391	88.9	44.7%
8	314,787	59.6	30.0%
10	51,574	9.8	4.9%
12	86,371	16.4	8.2%
14	4,065	0.8	0.4%
15	19,369	3.7	1.8%
16	1,086	0.2	0.1%
18	25,758	4.9	2.5%
20	20,943	4.0	2.0%
21	22,980	4.4	2.2%
24	10,119	1.9	1.0%
27	8,127	1.5	0.8%
30	837	0.2	0.1%
Total	**1,049,159**	**199**	**100.0%**

4.0 GROWTH PROJECTIONS

Growth projections are an important element in the analysis of water and wastewater systems. Water demands and wastewater flows depend on the residential population and commercial development served by the systems and determine the sizing and location of system infrastructure.

4.1 HISTORICAL POPULATION

Historical population data was used to analyze past growth rates within the City. **Table 4-1** shows the historical population of Kerrville from 2011 to 2021 from the United States Census Bureau. The average annual growth rate over the last 10 years was 0.91%.

Table 4-1: Historical City of Kerrville Population

Year	Census Population	Annual Growth Rate
2011	22,369	--
2012	22,412	0.19%
2013	22,427	0.07%
2014	22,643	0.96%
2015	22,928	1.26%
2016	23,169	1.05%
2017	23,339	0.73%
2018	23,622	1.21%
2019	23,754	0.56%
2020	24,278	2.21%
2021	24,477	0.82%
Average	**--**	**0.91%**

Billing data for years 2020 and 2021 was provided by City staff and used to determine the existing connection count. The existing connection count and known population were utilized to estimate the average number of people served by one connection for each system. There are approximately 600 less connections in the wastewater system than the water system. This is potentially due to water customers utilizing alternate wastewater services such as septic systems. The ratio of people per connection is approximately 2.2 people per connection.

The people per connection ratios were used to estimate the number of connections in each system for years prior to 2020, based on the historical population data. Resulting connection estimates are shown in **Table 4-2**.

Table 4-2: Historical Water and Wastewater Connections

Year	Water Connections	Wastewater Connections
2011	10,175	9,581
2012	10,194	9,601
2013	10,201	9,608
2014	10,299	9,706
2015	10,429	9,835
2016	10,538	9,945
2017	10,616	10,022
2018	10,744	10,151
2019	10,805	10,211
2020	11,043	10,449
2021	11,148	10,554

4.2 GROWTH PROJECTIONS

Growth projections were developed through collaboration between FNI and the City, utilizing information from the 2018 Kerrville Long Range Water Supply Plan (LRWSP) and the Kerrville 2050 Comprehensive Plan. The unit of growth used for this study is a "connection," as defined in §290.38(16) of the *Texas Administrative Code (TAC).* Growth projections were calculated for the planning years 2022, 2027, 2032, and 2047.

The annual growth rate was estimated using data from the 2018 Long Range Water Supply Plan. **Table 4-3** summarizes the annual growth rates by account use type. The overall growth rate in connections was calculated by weighting the growth rates by the number of existing accounts in each category. This overall growth rate was used to project the growth in connections throughout all planning periods and as a guide to allocate developments to projected planning periods.

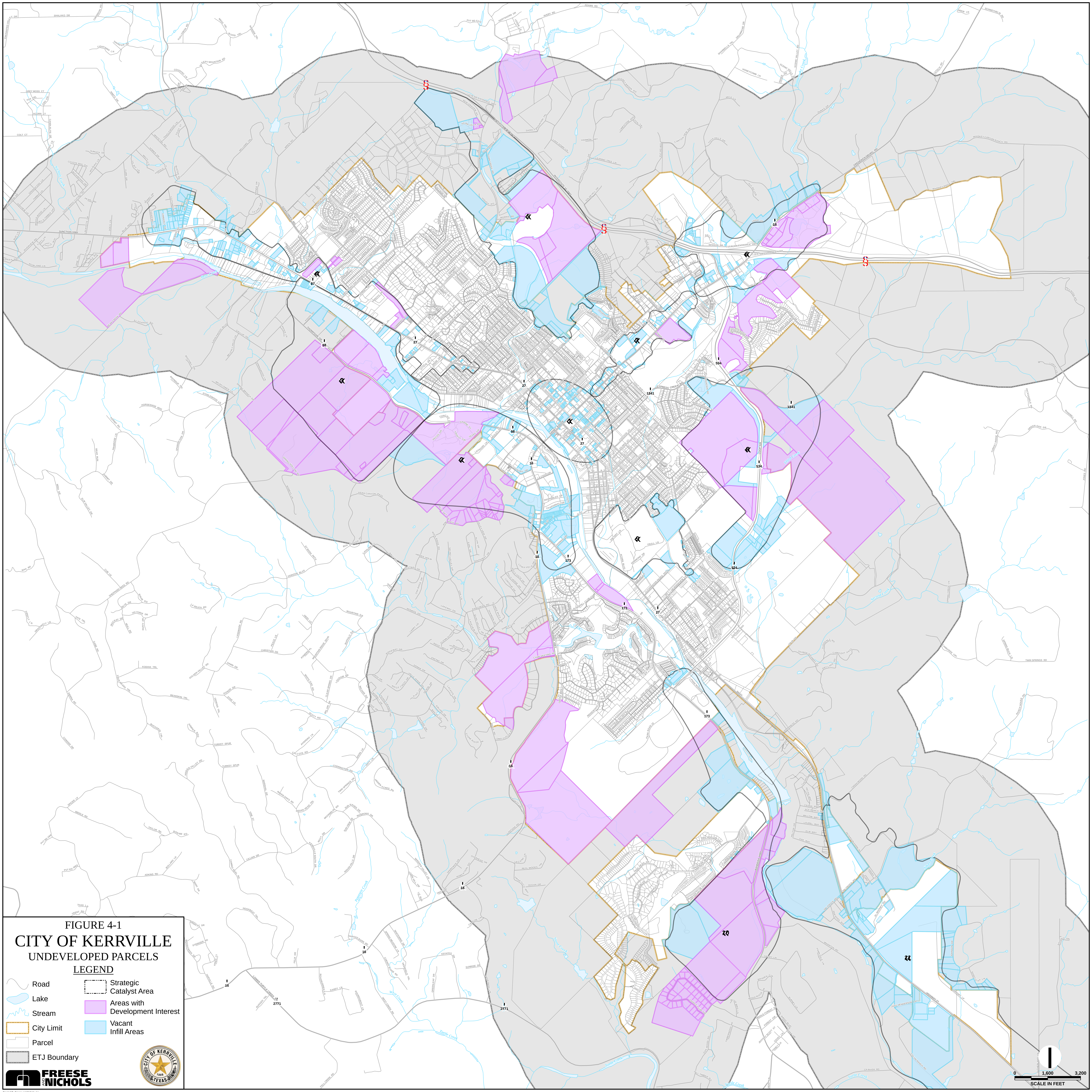
Table 4-3: Annual Growth Rate by Account Type from 2018 LRWSP

Account Use Type	Projected Annual Growth in Number of Connections
Residential	1.00%
Commercial	1.50%
Irrigation	1.50%
Municipal	1.50%
Weighted Overall Growth Rate	**1.09%**

To spatially distribute the projected growth, FNI utilized the Kerrville 2050 Comprehensive Plan to identify undeveloped parcels within the catalyst growth areas outlines in the plan. A parcel was considered undeveloped if it did not have an associated water meter and had an existing land use of "vacant". Additional undeveloped parcels outside of catalyst areas were also identified for future growth based on development interest the City has received. The catalyst areas and undeveloped parcels are shown on **Figure 4-1.** Future land use data established by the Kerrville 2050 Comprehensive Plan was used to estimate the type of development that would occur within the undeveloped areas. Typical meter densities were developed for residential and non-residential land use types by reviewing existing developments. **Table 4-4** shows the selected meter densities, in meters per acre, by land use type. The selected meter densities were then applied to the developable area of each undeveloped parcel to estimate the future number of connections anticipated for that parcel. Developable area is identified as the area of the parcel that does not fall within the floodplain.

Table 4-4: Connections per Acre and Customer Group by Land Use

Land Use Type	Connections per Acre
Residential	3.00
Non-Residential	0.50



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4.3 DEVELOPMENT SCENARIOS

The timing of each development was based on proximity to existing infrastructure and existing development, with the goal of establishing a growth rate within the range of the overall growth rate estimated from the 2018 LRWSP. If all of the identified developments occurred across the City, an additional 16,288 connections would be added to the system for a total connection count of 26,842. Using the weighted overall growth rate from the LRWSP of 1.09%, this amount of growth would take more than 80 years to occur. For this master plan study, a long-term planning period of 25 years was selected to appropriately size system improvements. If a planning period too far into the future is selected, there is an increased risk of building over-sized or unnecessary infrastructure. Because the majority of the remaining developable land will be difficult to develop from a water and wastewater standpoint, it is anticipated that once an area starts to develop it would trigger additional nearby development to also develop. Therefore, four growth scenarios were identified to account for different growth patterns and locations of development. The scenarios were generated based on general geographical location relative to existing pressure zone and sewer basin boundaries. Parcels were assigned a planning year and development scenario such that the resulting total annual growth in connections aligned with the projected growth rates utilized in the 2018 LRWSP and 2050 Kerrville Comprehensive Plan. The development scenarios were utilized to spatially distribute the projected growth across the water distribution and wastewater collection system to help assess system capacity and size recommended system improvements.

Table 4-5 and **Table 4-6** present the growth projections by planning year for the City's Water and Wastewater Service Areas. **Figure 4-2** shows the development parcels categorized by their development scenarios. These growth projections were entered into the hydraulic models of the water and wastewater systems to simulate future demands, which helped develop the location, capacity, and phasing of the proposed system improvements.

Table 4-5: Water Growth Projections

Scenario	Connections				Growth Rate
	Existing	5-year	10-year	25-year	
A	11,338	12,145	13,007	15,965	1.38%
B	11,338	12,045	12,796	15,330	1.21%
C	11,338	11,958	12,611	14,771	1.06%
D	11,338	12,076	12,872	15,574	1.28%
System Wide	**11,338**	**11,969**	**12,635**	**14,863**	**1.09%**

Table 4-6: Wastewater Growth Projections

Scenario	Connections				Growth Rate
	Existing	5-year	10-year	25-year	
A	10,701	11,508	12,370	15,328	1.45%
B	10,701	11,408	12,159	14,693	1.28%
C	10,701	11,321	11,974	14,134	1.12%
D	10,701	11,439	12,235	14,937	1.34%
System Wide	**10,701**	**11,332**	**11,998**	**14,226**	**1.15%**

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FIGURE 4-2
CITY OF KERRVILLE
DEVELOPMENT SCENARIOS

5.0 HYDRAULIC MODEL UPDATE AND CALIBRATION

Model calibration involves the process of rectifying parameters within the hydraulic model until the model generates and conveys flow in a similar manner as observed in the system. A properly calibrated model serves as the foundation for any future modeling scenarios.

5.1 WATER MODEL UPDATE

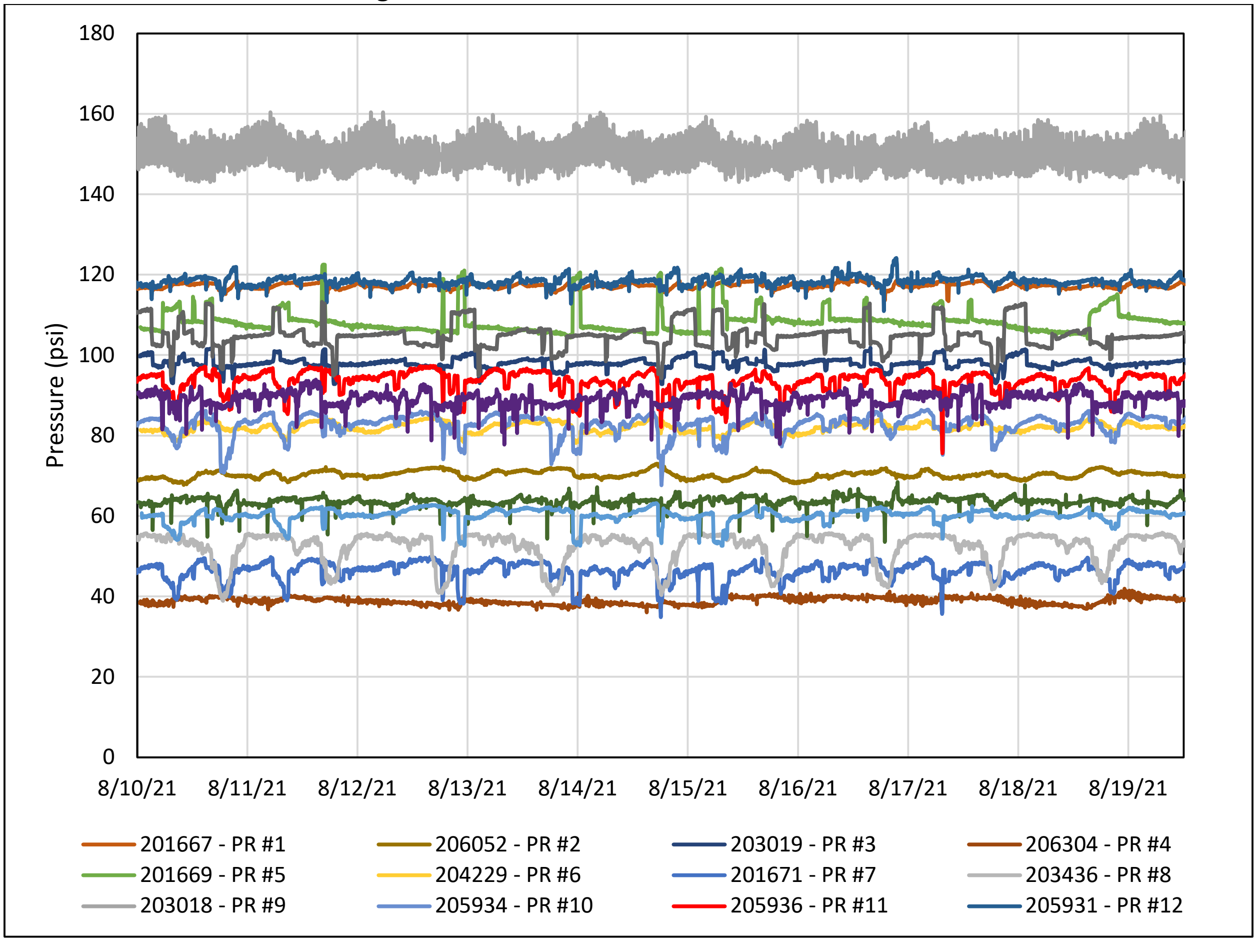
The City of Kerrville owns and maintains a hydraulic water model in the InfoWater® software package from Innovyze that was created during the 2013 Water Master Plan. FNI updated the existing water model with current GIS data. All water lines were included in the hydraulic model. This section discusses the model update process, including model calibration to observed system conditions.

5.2 WATER PRESSURE TESTING AND MODEL CALIBRATION

5.2.1 Pressure Testing

Field pressure testing was conducted from August 10th to August 20th, 2021. 16 pressure recorders were installed throughout the distribution system. Locations of the pressure recorders are illustrated on **Figure 5-2**. The City also provided Supervisory Control and Data Acquisition (SCADA) records during the pressure testing period. Hourly readings of flow, tank level, and pump status were recorded for all monitored points in the system. A summary of the observed pressure data is included on **Figure 5-1** with detailed graphs included in **Appendix A**.

Figure 5-1: Observed Pressure Recorder Data



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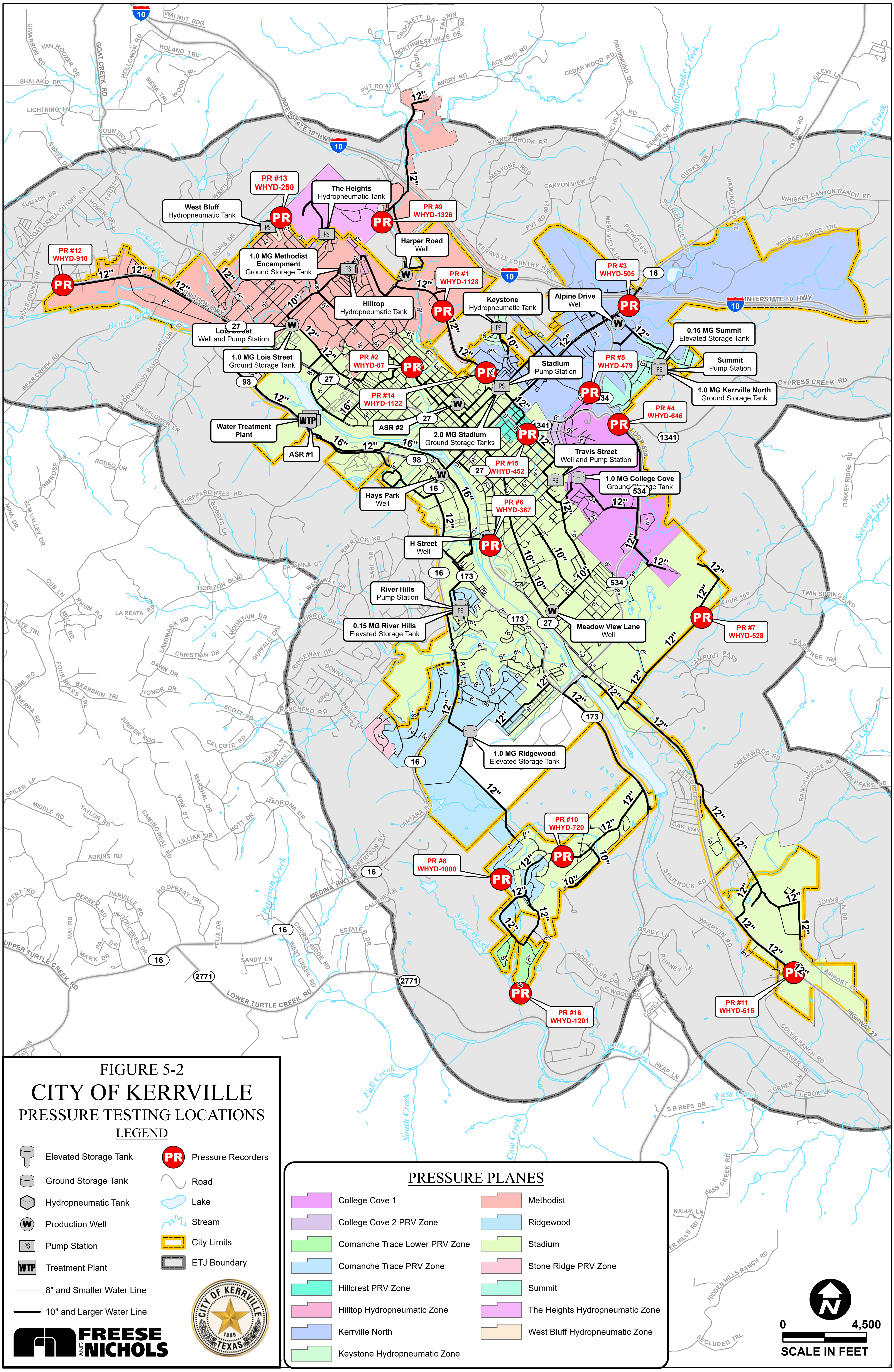


5.2.2 Model Calibration

An extended period simulation (EPS) model calibration was performed to verify that the hydraulic model accurately represents real-world distribution system operations. The calibration process involved adjusting system operation, C-values, demand allocation, and peaking factors to match known conditions. August 14th, 2021 was selected for calibration as the diurnal curve indicated no irregularities and the total demand was relatively high compared to other days during field testing. Model calibration during high demand conditions is preferable as operations are captured when the system is stressed, and most facilities are in use.

Demands were allocated to the model from averaged metered billing data provided by the City for the period from August 2020 to August 2021. The demands were then scaled to match the demands experienced on August 14th, 2021 for calibration. Based on diurnal curves calculated using the SCADA records provided by the City, the total demand was 4.56 MGD. Flow and tank level data were utilized to calculate diurnal demand curves by examining water going into (supply) and out of (demand) each pressure plane in the distribution system. The demand pattern, in 1-hour increments, was input to the model and assigned to the appropriate demand nodes. This pattern adjusts the demand for each pressure plane to match fluctuations over the course of the 24-hour calibration period. **Figure 5-3** presents the diurnal curves for the calibration period of August 14th, 2021. Operational controls were used to simulate the conditions that occurred during the calibration period. Initial tank levels, at 12:00 am on August 14th, were input to the model from the provided SCADA records. The “Pump Status” provided in SCADA was used to develop control statements in the model that turned pumps on and off throughout the day.

During the calibration, adjustments were made to the model to match the observed conditions of August 14th, 2020. Pump run times were adjusted slightly to better match recorded tank levels. The results of the EPS calibration are summarized on the graphs included in **Appendix B**. An example of the calibration graphs is shown in **Figure 5-4**. The graphs show modeled pressures and tank levels versus recorded data at tanks, and pressure recorder locations. The results suggest a good correlation between recorded and modeled values. This degree of calibration provides confidence in the accuracy of the model and is considered suitable for the development of a water system CIP and master planning purposes.

Figure 5-3: Calibration Day Diurnal Curves

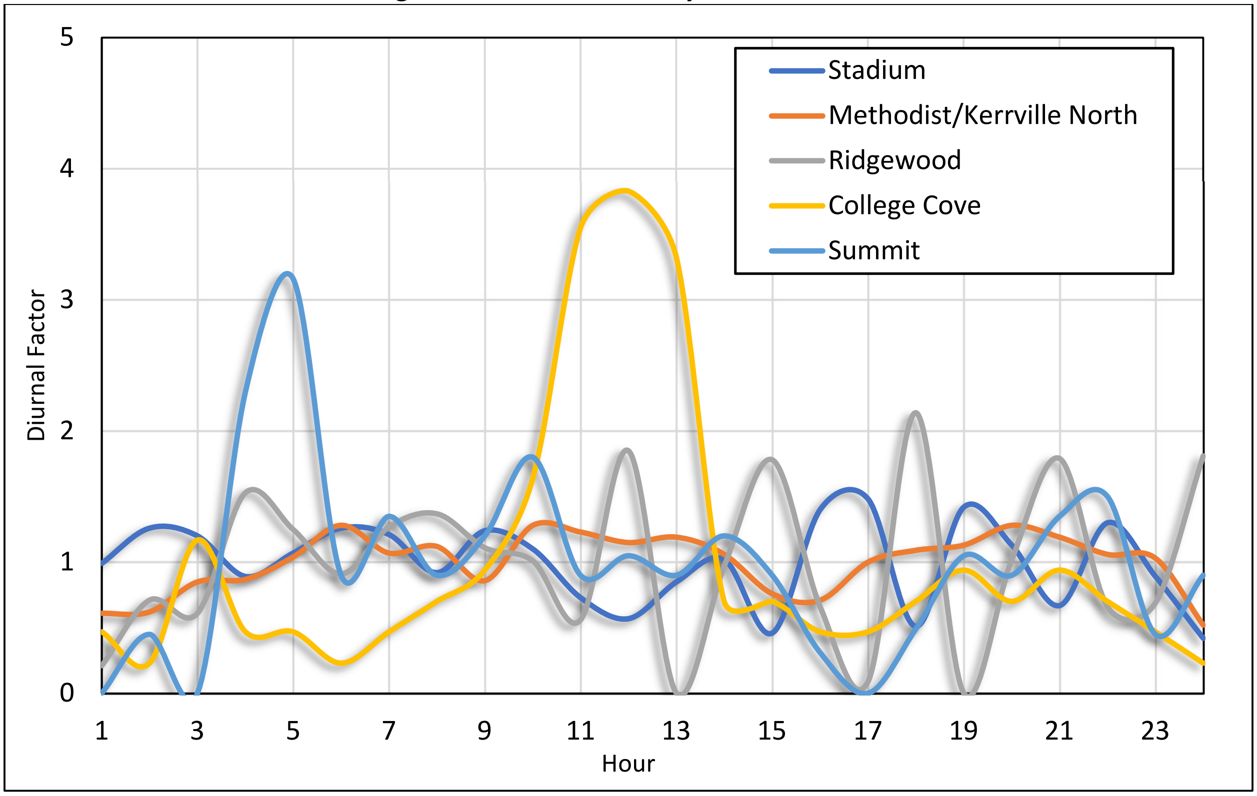
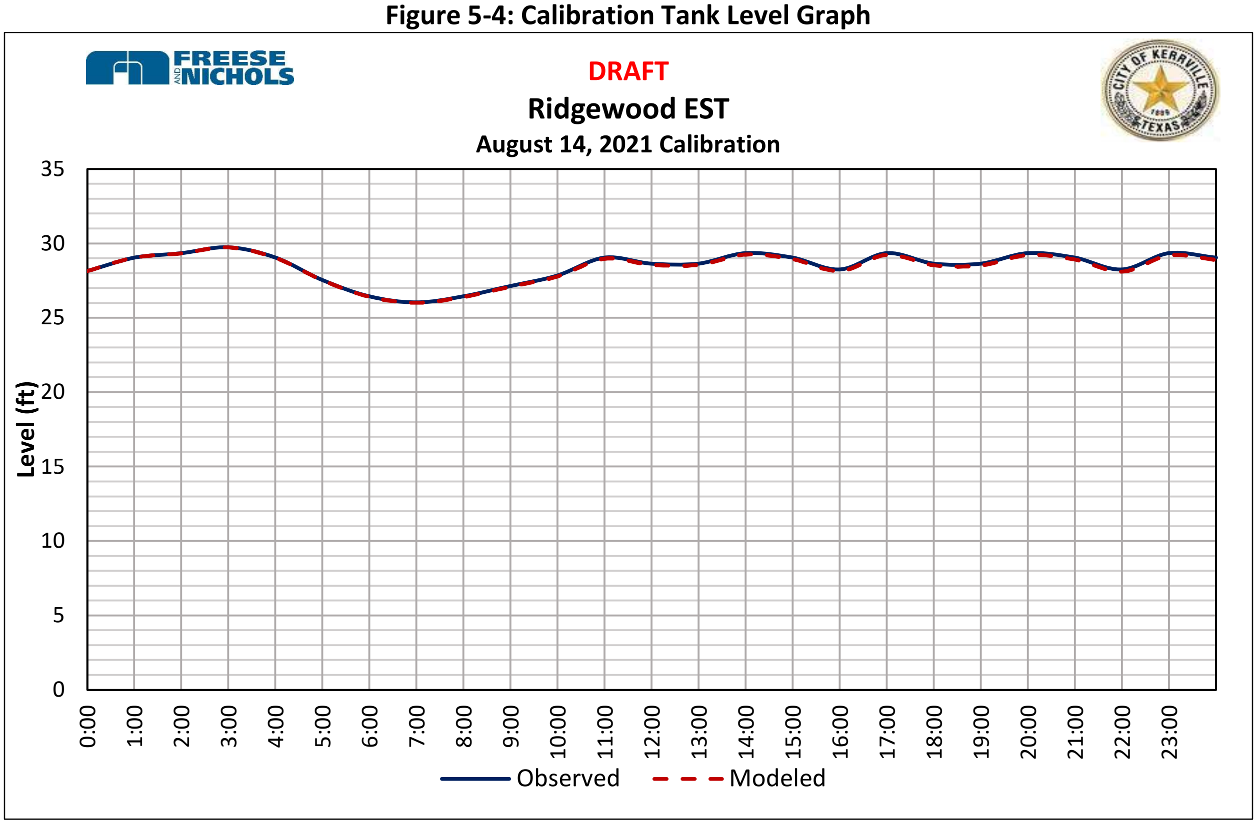


Figure 5-4: Calibration Tank Level Graph



5.3 WASTEWATER MODEL UPDATE

FNI developed a hydraulic model to be used as a tool for evaluating the wastewater collection system using InfoSewer® software by Innovyze. The City provided a GIS geodatabase, which was the basis for building the hydraulic model. The provided geodatabase contained spatial and attribute information for the wastewater gravity lines, manholes, force mains, and lift stations as well as parcel data. The GIS data for these assets was imported into the hydraulic model and processed to validate attribute data and allow for proper network topology. Missing information was populated with the results of field inspections and as-build records when available. Some missing facility information that is critical for inclusion in the hydraulic model could not be rectified. To account for this missing information, the hydraulic model was skeletonized, or assets with missing information were removed from the model network to allow the hydraulic model to operate properly. Critical system infrastructure is included in the hydraulic assessment, but the majority of small diameter lines were removed from the model during the skeletonization process. A summary of the modeled network is shown in **Table 5-2** and **Figure 5-5**. On average, 85% of the wastewater collection system was included in the hydraulic model, with 99% of lines 10-inches or greater.

Table 5-1: Modeled Wastewater Collection System

Diameter (inches)	Gravity Main Length (LF)	Modeled Gravity Main Length (LF)	Percent Modeled
Unknown	1,100	0	0.00%
4	3,410	2,059	60.37%
6	458,343	331,159	72.25%
8	281,705	272,248	96.64%
10	40,839	40,727	99.73%
12	85,323	85,238	99.90%
15	19,369	19,325	99.77%
16	199	199	100.00%
18	22,925	22,280	97.19%
20	100	0	0.00%
21	22,980	22,980	100.00%
24	10,099	10,093	99.93%
27	8127.4	8127.4	100.00%
30	837	718	85.80%
Total	**955,359**	**815,153**	**85.32%**
≥ 10"	**210,801**	**209,687**	**99.47%**

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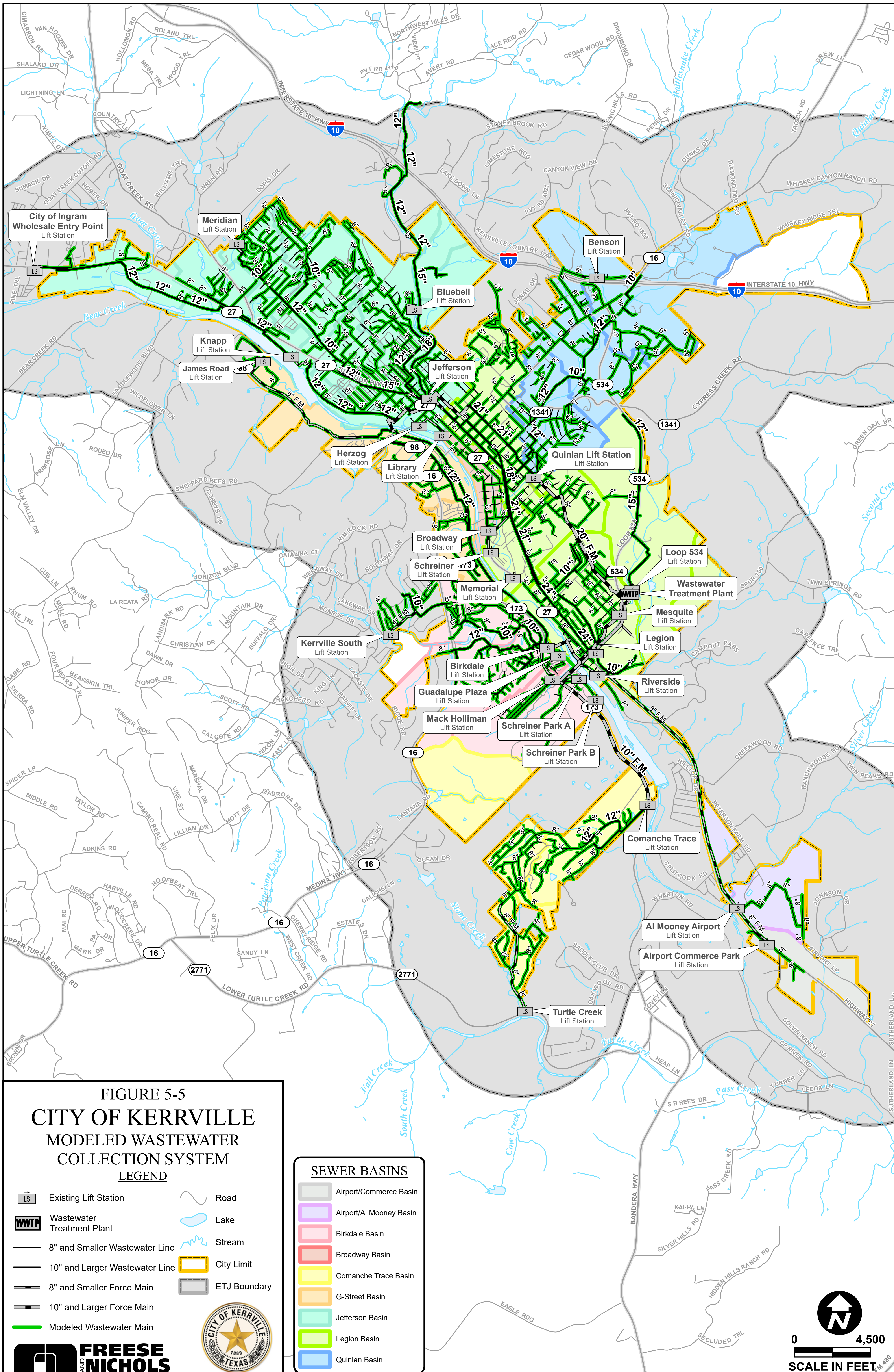

5.4 WASTEWATER FLOW MONITORING AND MODEL CALIBRATION

Flow monitoring is an important step in evaluating a sanitary sewer collection system. The flow monitoring data is used to examine the existing dry and wet weather flows, evaluate the effects of rainfall on the wastewater collection system, and calibrate a hydraulic model to evaluate the capacity of the system to transport peak flows.

FNI retained ADS Environmental Services (ADS) to conduct temporary flow monitoring within selected portions of the existing sanitary sewer system. Evaluation of the results of the temporary flow monitoring allows for the characterization of dry weather and wet weather flows within the wastewater system, the ranking of relative severity of observed infiltration and inflow (I/I), and the evaluation of key performance indicators to direct subsequent condition assessment and rehabilitation activities. The flow monitoring data was also used in the hydraulic model calibration. Flow monitoring was performed using area-velocity flow monitors manufactured, installed, and maintained by ADS. Each flow monitor was mounted near the top of a manhole and was connected to flow, depth, and velocity sensors positioned in the incoming sewer line. Each flow monitor was equipped with an ultrasonic depth sensor mounted at the crown of the sewer line and a velocity sensor mounted at or near the invert of the sewer line. A pressure depth sensor was also mounted at or near the invert to measure surcharge depths. Depth, velocity, and flow rate data from each flow monitor were collected and evaluated to provide insight into sewer performance, revealing important information about how the existing wastewater system accommodates observed flow rates.

Dry weather and wet weather performance within the existing wastewater system were evaluated by installing flow monitors within the collection system to observe and document existing flow conditions. FNI coordinated with City staff to identify locations for the temporary flow monitors. The locations were strategically selected to allow the temporary flow monitors to capture the system's performance during rain events and provide data for hydraulic model calibration. Three rain gauges were also installed within the system to quantify the observed rain events. **Figure 5-6** shows the locations of the 12 temporary flow monitors and the three rain gauges. A schematic of the flow monitor locations and collection system operations is shown on **Figure 5-7**. The temporary flow monitors were deployed for a total of 58 days from May 11, 2021 to July 8, 2021. Flow monitoring data during observed wet weather events is used to identify areas with high rainfall-dependent infiltration and inflow (RDII) and to calibrate the hydraulic model to wet weather conditions. During the flow monitoring period, multiple rain events occurred within the study area, with the maximum rainfall observed being 2.72 inches on May 28, 2021.

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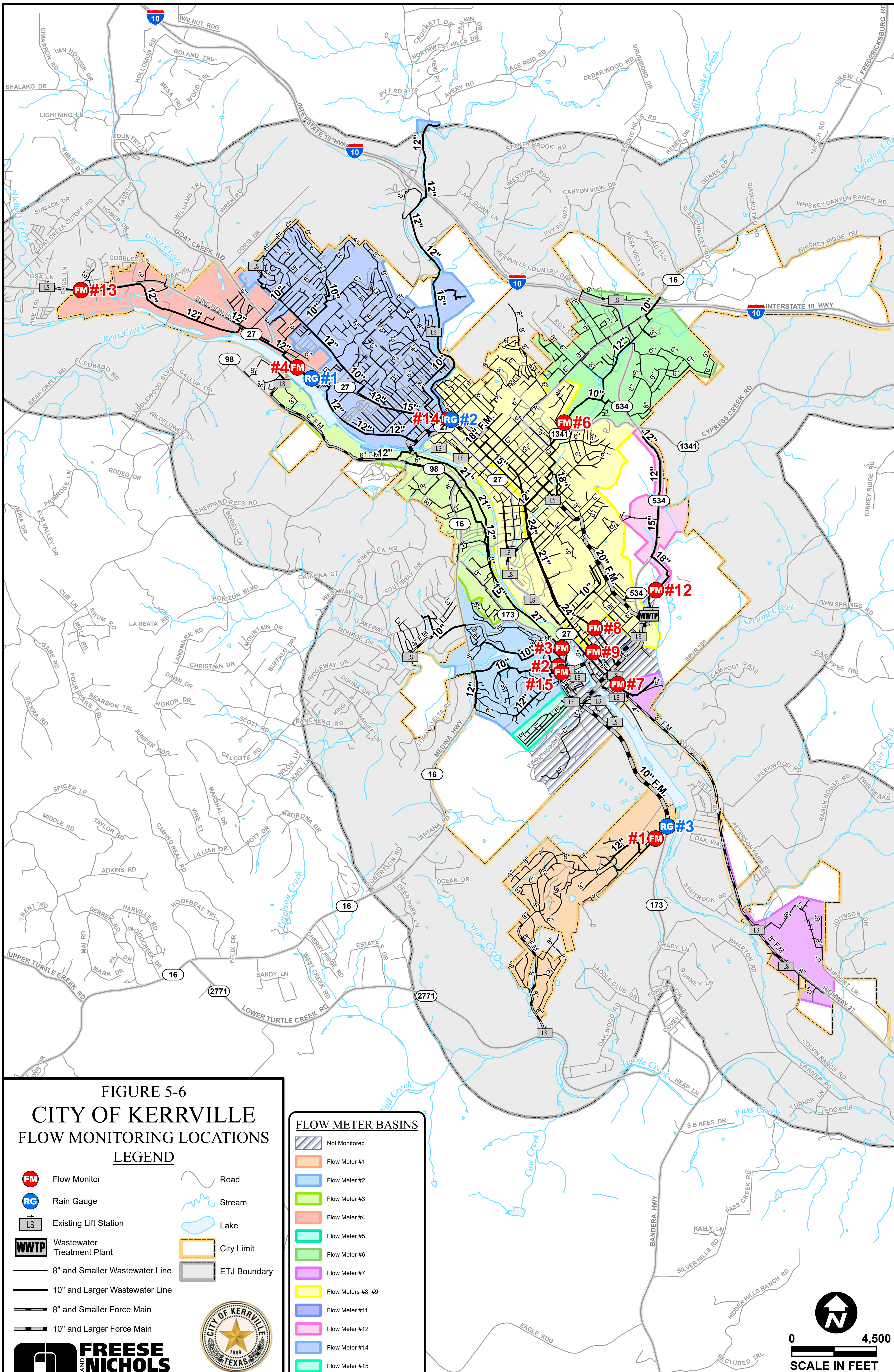
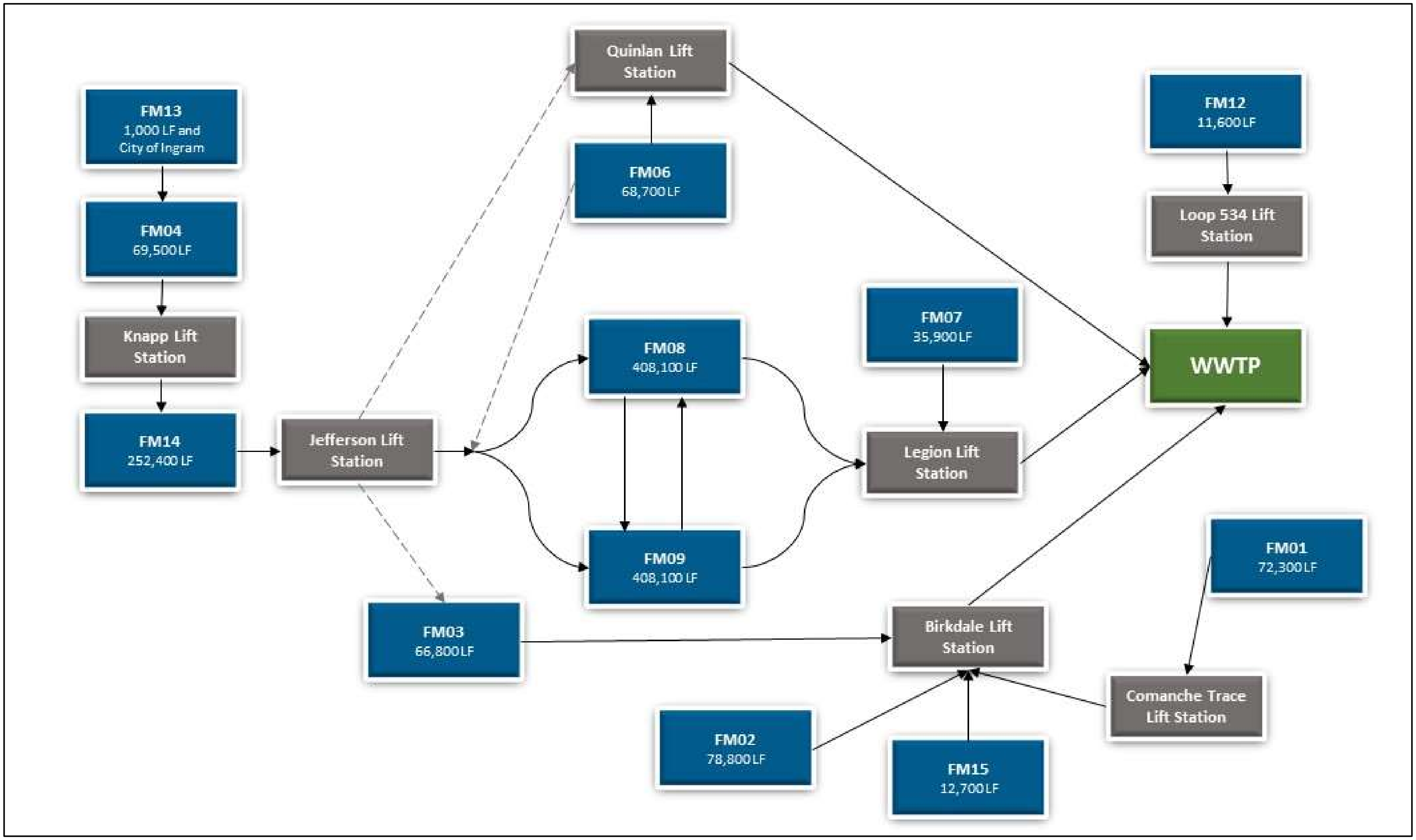


FIGURE 5-6
CITY OF KERRVILLE
FLOW MONITORING LOCATIONS

Figure 5-7: Temporary Wastewater Flow Monitoring Schematic



5.4.1 Flow Monitoring Analysis

A summary of the average dry and peak wet weather flows at each temporary flow monitoring location can be found in **Table 5-2**. The ADS flow monitoring report can be found in its entirety in **Appendix C**.

Table 5-2: Flow Monitoring Summary

Flow Monitor	Observed Dry Weather Flow (MGD)	Observed Peak Wet Weather Flow (MGD)	Peaking Factor
FM 01	0.09	1.20	13.33
FM 02	0.81	1.38	1.70
FM 03	0.14	0.94	6.71
FM 04	0.28	1.32	4.71
FM 06	0.23	1.16	5.04
FM 07	0.10	0.50	5.00
FM 08	0.36	1.45	4.03
FM 09	1.14	3.63	3.18
FM 12	0.01	0.23	23.00
FM 13	0.14	1.15	8.21
FM 14	0.76	2.32	3.05
FM 15	0.05	0.23	4.60
Total	**2.88**	**10.49**	**3.64**

Peaking factor is the ratio of observed peak flow to average flow and plays a key role in wastewater analysis and design. Peaking factors are inversely proportional to the population served and generally decrease as average dry weather flow increases. The largest peaking factors observed during the study period were a value of 23.00 at site FM 12 and a value of 13.33 at site FM 01. These high peaking factors are likely due to low or irregular average dry weather flow and are not a concern at this time. Wet weather peaking factors at the remaining flow monitor locations ranged from 1.70 to 8.21 with an average peaking factor of 3.64. A peaking factor of 4.00 is typical for a wastewater collection system and observed peaking factors greater than 4.00 indicate high levels of inflow and infiltration upstream of the flow monitoring location.

5.4.2 Dry Weather Calibration

FNI utilized the flow monitoring data for dry and wet weather events to calibrate the wastewater model. FNI first calibrated the model to the average dry weather flow, which represents a week of dry weather conditions. During dry weather model calibration, the per-capita flow rates were adjusted to match observed flow volumes. Iterations were performed until the model results closely reflected the observed dry weather flow data at each flow meter site within a tolerance of +/- 5%. Results are provided for the average flow rate over the dry weather calibration period in **Table 5-3**. On average, the hydraulic model is calibrated to observed dry weather conditions within 1%.

Table 5-3: Dry Weather Calibration Summary

Flow Monitor	Observed Dry Weather Flow (gpm)	Modeled Dry Weather Flow (gpm)	Dry Weather Calibration
FM 01	73	73	100.0%
FM 02	553	553	100.0%
FM 03	77	77	100.0%
FM 04	199	199	100.0%
FM 06	160	160	100.0%
FM 07	23	23	100.0%
FM 08	250	251	100.3%
FM 09	805	801	99.4%
FM 12	6	6	100.0%
FM 13	100	100	100.0%
FM 14	536	536	100.0%
FM 15	34	34	100.0%
Total	**1,948**	**1,944**	**99.8%**

5.4.3 Wet Weather Calibration

The calibrated dry weather flow was utilized during wet weather calibration to represent the dry weather component of the observed flow hydrograph and derive the I/I response during wet weather calibration. Criteria for selecting wet weather calibration events typically includes no preceding or subsequent rain events and consistency throughout the study area. Multiple significant wet weather events were observed during the flow monitoring period. The primary wet weather calibration event occurred on May 28, 2021 with a total volume of 2.72 inches over approximately 12 hours.

These values were adjusted until the modeled wet weather flows closely matched the observed wet weather flows. Iterations were performed until the model results closely reflected the observed wet weather flow data at each flow meter site within a tolerance of +/- 10%. Results of the wet weather calibration are summarized in **Table 5-4.** On average, the hydraulic model is calibrated to observed wet weather conditions within 3%.

Table 5-4: Wet Weather Calibration Summary

Flow Monitor	Observed Peak Wet Weather Flow (gpm)	Modeled Peak Wet Weather Flow (gpm)	Peak Wet Weather Calibration
FM 01	833	833	100.0%
FM 02	958	957	99.9%
FM 03	653	653	100.0%
FM 04	917	917	100.0%
FM 06	806	805	99.9%
FM 07	347	347	100.0%
FM 08	1,007	944	93.7%
FM 09	2,521	2,503	99.3%
FM 12	160	160	100.0%
FM 13	799	799	100.0%
FM 14	1,611	1,611	100.0%
FM 15	160	160	100.0%
Total	**7,285**	**7,203**	**98.9%**

6.0 WATER DEMAND

A water utility must be able to supply water at rates that fluctuate over time. Yearly, monthly, daily, and hourly variations in water use occur, with higher use during dry years and in hot months. Water use typically follows a diurnal pattern, being low at night and peaking in the early morning and evening. Flow rates most important to the hydraulic design and operation of a water treatment plant and distribution system are average day (AD), maximum day (MD), and peak hour (PH) demands.

6.1 HISTORICAL WATER DEMANDS

Reviewing historical water demands provides insight into selecting design criteria used to project future water demands. Historical monthly water usage was analyzed from 2010 through 2021. The City provided water production data consisting of amount of water pumped or produced daily at each groundwater well and treatment plant. Historical annual average day demand, and maximum day to average day peaking factors are summarized in **Table 6-1**.

Table 6-1: Historical Water Demand

Year	Connections	WTP Annual Average Production (MGD)	Average Per Connection Flow Rate (gpCd)	WTP Maximum Daily Production (MGD)	Maximum Day Peaking Factor
2010	10,165	3.74	368	7.37	1.97
2011	10,175	3.86	379	6.15	1.60
2012	10,194	3.98	391	6.96	1.75
2013	10,201	3.66	359	6.79	1.85
2014	10,299	3.63	352	6.86	1.89
2015	10,429	3.63	348	6.41	1.77
2016	10,538	3.51	333	6.68	1.90
2017	10,616	3.58	337	5.67	1.58
2018	10,744	3.79	353	6.36	1.68
2019	10,805	3.22	298	5.46	1.69
2020	11,043	3.32	301	5.70	1.71
2021	11,148	3.28	294	9.14	2.79
Average	**-**	**3.60**	**343**	**6.63**	**1.84**
Maximum	**-**	**3.98**	**391**	**9.14**	**2.30**

6.2 PROJECTED WATER DEMANDS

Water demands were projected for 2022, 2027, 2032, and 2047 conditions. The evaluation of historical trends in per capita data provided a basis for determining the planning criteria used to project average day water demands. The average day demand rate was used as a basis for estimating maximum day and peak hour demands, which are more relevant values to locating and sizing recommended improvements.

In selecting a peaking factor to project maximum day demands, FNI reviewed historical peaking factors and the years in which those peaking factors occurred, as just using the highest values for per-capitas and peaking factors may not be representative. Historical water usage data indicated the maximum day to average day peaking factor ranged from 1.60 to 2.79, with an average of 1.84 over the last 12 years. Therefore, a maximum day peaking factor of 1.8 was selected for the calculation of projected maximum day demands. Since hourly flows are not measured at every water facility, historical peak hour demands could not be calculated. In the absence of data specific to the City of Kerrville, a peaking factor of 2.0 was selected to project peak hour demands.

The recommended planning criteria for the water system are summarized in **Table 6-2**. These planning criteria were applied to the projected growth scenarios to estimate future demand in the water system. **Table 6-3**, **Table 6-4**, and **Table 6-5** summarize the average day, maximum day, and peak hour demands for each development scenario, respectively. **Figure 6-1** illustrates the historical and projected water demands for the City of Kerrville.

Table 6-2: Water Demand Criteria

Demand per Connection (gpCd)	Maximum Day to Average Day Peaking Factor	Peak Hour to Maximum Day Peaking Factor
350	1.80	2.00

Table 6-3: Average Day Demand Projections Per Development Scenario

Scenario	Average Day Demand (MGD)			
	Existing	5-year	10-year	25-year
A	3.97	4.25	4.55	5.59
B	3.97	4.22	4.48	5.37
C	3.97	4.19	4.41	5.17
D	3.97	4.23	4.51	5.45
System Wide	**3.97**	**4.19**	**4.42**	**5.20**

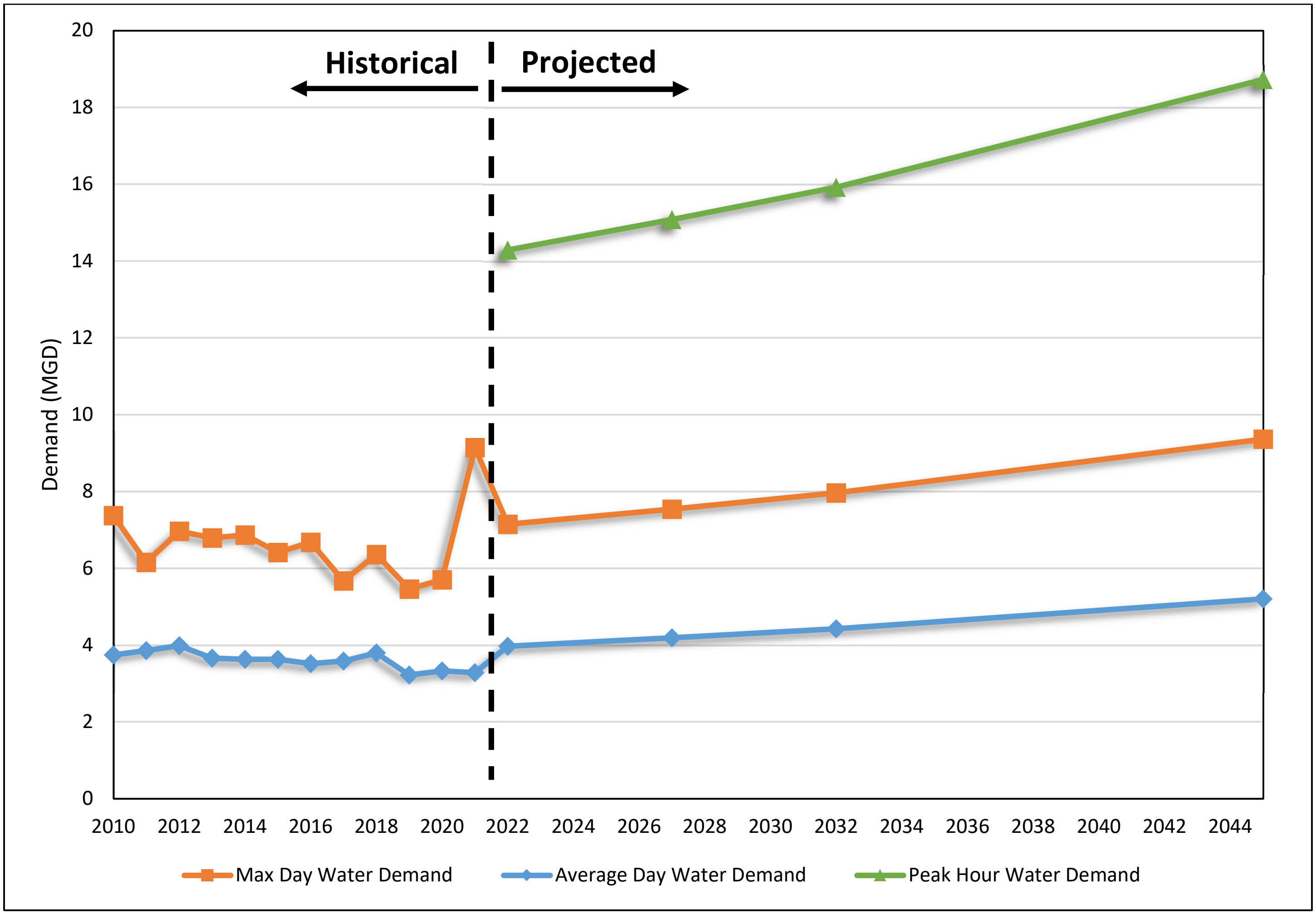
Table 6-4: Maximum Day Demand Projections Per Development Scenario

Scenario	Max Day Demand (MGD)			
	Existing	5-year	10-year	25-year
A	7.14	7.65	8.19	10.06
B	7.14	7.59	8.06	9.66
C	7.14	7.53	7.94	9.31
D	7.14	7.61	8.11	9.81
System Wide	**7.14**	**7.54**	**7.96**	**9.36**

Table 6-5: Peak Hour Demand Projections Per Development Scenario

Scenario	Peak Hour Demand (MGD)			
	Existing	5-year	10-year	25-year
A	14.29	15.30	16.39	20.12
B	14.29	15.18	16.12	19.32
C	14.29	15.07	15.89	18.61
D	14.29	15.22	16.22	19.62
System Wide	**14.29**	**15.08**	**15.92**	**18.73**

Figure 6-1: Historical and Projected Water Demand



7.0 WASTEWATER FLOW

The performance of the City's wastewater system is dependent on the amount of flow being conveyed through the system currently and in the future. To determine locations where future wastewater system improvements are necessary, FNI developed existing and future wastewater load projections using historical flow data and existing and future development projections.

Wastewater flows in a municipal collection system vary by time of day, wastewater discharge source, and weather conditions. Annual average daily flow (AADF) is defined as the total wastewater flow over a one-year period divided by the number of days in that year. WWTPs are typically sized in terms of average daily flow, while the collection system is sized to convey peak wastewater flows. Peak wastewater flow is comprised of three components: the maximum daily dry weather flow, infiltration, and inflow. Infiltration is the seepage of groundwater into the sewer pipe and appurtenances. Inflow is the rainwater that enters the collection system, both directly and indirectly, during and immediately following a storm event. The inflow represents stormwater runoff from paved and non-paved areas throughout the service area.

7.1 HISTORICAL WASTEWATER TREATMENT PLANT FLOWS

To project future wastewater flows, historical flow data was analyzed to determine the historical trends in system-wide average daily flow and per-connection flow. The City provided average plant discharge flow data from 2010 to 2021. Historical connections from water billing data were used to determine the historical per-connection flow rate. **Table 7-1** provides a summary of the historical annual average daily flows at the wastewater treatment plant.

Table 7-1: Historical WWTP Flow

Year	WWTP Annual Average Flow (MGD)	Connections	Per Connection Flow Rate (gpcd)
2010	1.88	9,571	197
2011	2.02	9,581	211
2012	2.17	9,601	226
2013	2.21	9,608	230
2014	2.07	9,706	214
2015	2.21	9,835	225
2016	2.41	9,945	243
2017	2.05	10,022	204
2018	2.10	10,151	207
2019	2.09	10,211	205
2020	2.08	10,449	199
2021	2.21	10,554	209
Average	**2.13**	**-**	**215**
Maximum	**2.41**	**-**	**243**

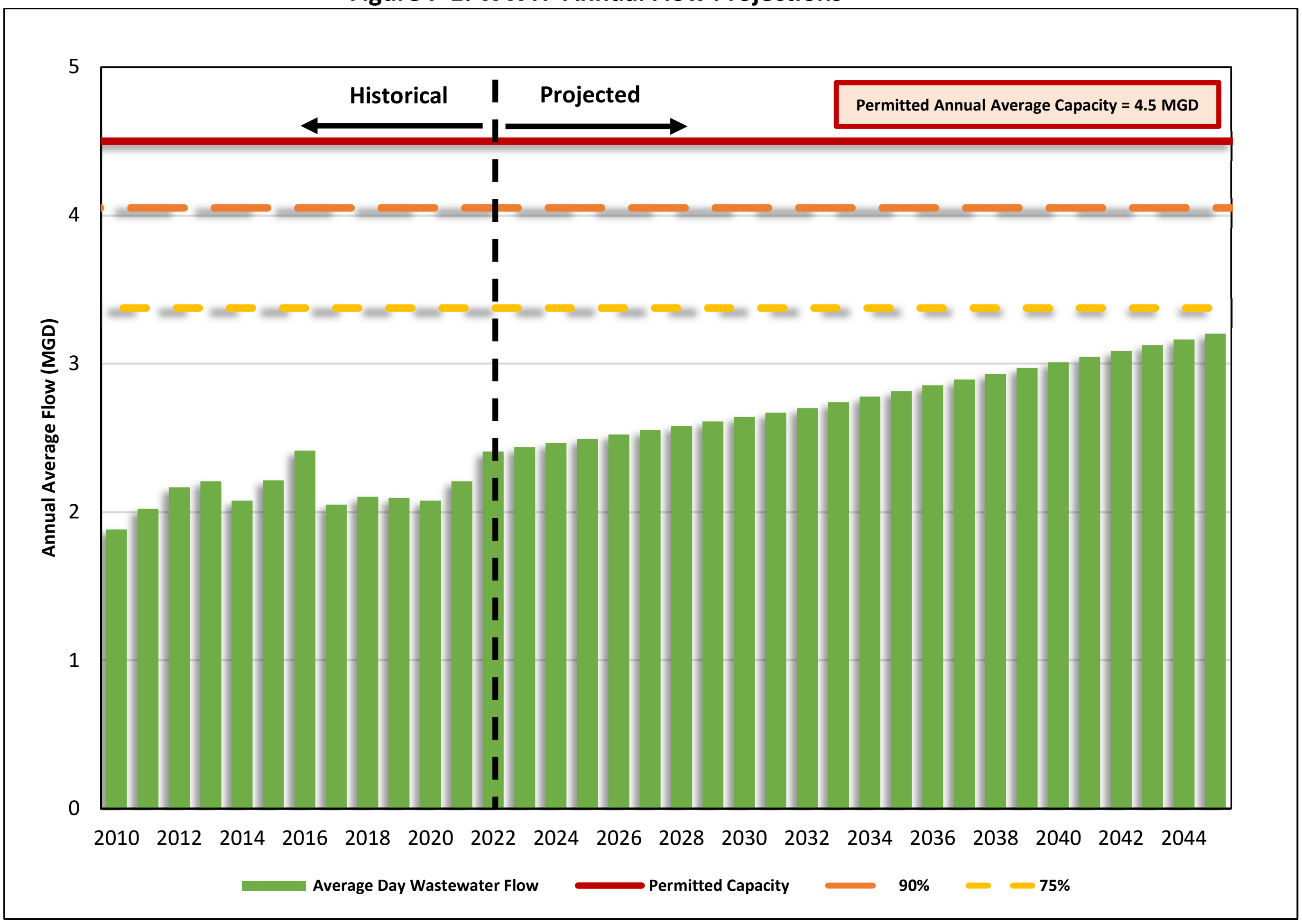
7.2 AVERAGE FLOW PROJECTIONS

FNI utilized the historical WWTP flows, the temporary wastewater flow monitoring data, and the calibrated wastewater model as the basis to develop the existing and future average day flows. Future growth for each planning period was added to the 2022 wastewater flow using a future per-capita flow of 225 gallons per connection per day (gpcd) multiplied by the number of new connections projected for each planning period. While the historical rates vary, the projected 225 gpcd is slightly more conservative than the historical average and anticipates that the per-capita will be maintained through all future planning periods. **Table 7-2** shows the projected annual average daily flow for each planning scenario. Additionally, **Figure 7-1** shows the projected average annual flows compared to the existing WWTP permitted capacity. This chart also shows the projected average annual flows compared to 75% and 90% of the permitted capacity, to indicate if and when the WWTP would trigger the TCEQ “75/90 rule”. The 75/90 rules states that when a plant exceeds 75% of its permitted annual average flow for three consecutive months, the facility must begin planning for its next expansion. In addition, the rule states that when a facility exceeds 90% of its permitted annual average flow, the facility must be in construction of its next expansion. The WWTP is not projected to exceed 75% of the existing permitted capacity through the 2047 planning period.

Table 7-2: Projected Annual Average WWTP Flow

Scenario	Average Day Flow (MGD)			
	Existing	5-year	10-year	25-year
A	2.41	2.59	2.78	3.45
B	2.41	2.57	2.74	3.31
C	2.41	2.55	2.69	3.18
D	2.41	2.57	2.75	3.36
System Wide	**2.41**	**2.55**	**2.70**	**3.20**

Figure 7-1: WWTP Annual Flow Projections



7.3 PEAK WET WEATHER FLOW PROJECTIONS

Peak wet weather flow projections are the primary driver of collection system hydraulic analysis and ultimately determine the projected sizing for future improvements. Inflow and infiltration (I/I) can lead to hydraulic bottlenecks in the system and can potentially cause surcharging and sanitary sewer overflows.

To evaluate the collection system, a storm event should be selected as the basis for evaluation. Peak wet weather flows are determined by evaluating the collection system's response to the applied storm. The peak wet weather flow was analyzed by utilizing the observed wet weather responses during the flow monitoring period. The wet weather response rates within each flow monitor basin were analyzed and scaled to estimate responses to a storm, including projecting peaking factors and maximum flow rates. The peaking factors from wet weather calibration were used to select the projected peak flow rate. A peaking factor of 4.50 was selected based on scaling up the observed peaking factor to a conservative estimate. This analysis assumes that existing I/I rates are held constant through all future planning periods and appropriate rehabilitation measures are taken to mitigate future system degradation and prevent an increase in I/I rates as the collection system ages. The projected peak flows are shown in **Table 7-3** for each planning scenario.

Table 7-3: Projected Peak Wet Weather WWTP Flow

Scenario	Peak Wet Weather Flow (MGD)			
	Existing	5-year	10-year	25-year
A	10.83	11.65	12.53	15.52
B	10.83	11.55	12.31	14.88
C	10.83	11.46	12.12	14.31
D	10.83	11.58	12.39	15.12
System Wide	**10.83**	**11.47**	**12.15**	**14.40**

8.0 WATER SYSTEM HYDRAULIC ANALYSIS

The existing distribution system was evaluated to assess current production, pumping, and storage capacity. This analysis is performed to determine if there are any existing system deficiencies and to provide a baseline for the current level of service. The water distribution system was also evaluated with consideration for future water service. Various combinations of improvements and modifications were investigated to determine the most appropriate approach for meeting projected demands. Parameters used in developing the improvements plan included increasing system reliability, simplifying system operations, and maintaining proper pressure throughout the system.

8.1 TCEQ REQUIREMENTS AND PLANNING CRITERIA

As a public water utility, the City must comply with the rules and regulations for public water systems set forth by the Texas Commission on Environmental Quality (TCEQ) in TAC §290.45. The TCEQ sets minimum system requirements pertaining to production, pumping, and storage capacity.

The TCEQ evaluates the City's water distribution system by pressure zone, except for water production and total storage, which are evaluated on a system-wide basis. The TCEQ production requirement is 0.6 gpm/connection, unless the system has less than 50 connections, in which case the requirement increases to 1.5 gpm/connection.

For service pumping, if a pressure zone has at least 200 gallons per connection of elevated storage, the resulting TCEQ pumping requirement is 0.6 gpm/connection. If the PZ has less than 200 gallons per connection of elevated storage, the TCEQ service pumping requirement increases to 2.0 gpm/connection or at least 1,000 gpm total with the ability to meet peak hour demands in the pressure zone with the largest pump out of service (i.e., firm capacity).

The City is also required to meet the TCEQ elevated storage capacity requirement of 100 gallons per connection and total storage capacity requirement of 200 gallons per connection. As described in **Section 2.4**, only the storage volume 80 feet or more above the highest service elevation in each pressure zone may be considered elevated storage. Minimum production, pumping, and storage requirements per TCEQ are summarized in **Table 8-1**.

Table 8-1: TCEQ Minimum Requirements and Recommended Planning Criteria Summary

System Component	TCEQ Minimum Requirement	Evaluation Area
Production	0.6 gpm/connection	Systemwide
Distribution Pumping	0.6 gpm/connection (≥ 200 gal/connection elevated storage) 2.0 gpm/connection (< 200 gal/connection elevated storage)	Pressure Plane
Elevated Storage	100 gallons/connection	Pressure Plane
Total Storage	200 gallons/connection	Systemwide

8.2 HYDRAULIC ANALYSIS

8.2.1 Existing System Analysis

TCEQ minimum system requirements and the recommended planning criteria were utilized to evaluate the 2022 water system. **Table 8-2**, **Table 8-3**, and **Table 8-4** summarize the evaluation against the recommended planning criteria for production capacity, distribution pumping capacity, and elevated storage capacity, respectively. Projected connections for 2022 (i.e., 2021 existing connections plus 1 year of projected growth) were used to evaluate the existing system against the minimum requirements.

Table 8-2: Recommended Production Capacity for Existing System

Planning Period	Connections	Required Production (gpm)	Surface Water Production (gpm)	Ground Water Production (gpm)	Total Available Production (gpm)	Meets TCEQ Requirement?
2022	11,338	6,802.8	4,645.0	4,563.0	9,208	Yes

Table 8-3: Recommended Distribution Pumping Capacity for Existing System

Planning Period	Pressure Plane	Connections	Required Pumping Capacity (gpm)	Available Pumping Capacity (gpm)	Meets TCEQ Requirement?
2022	Stadium	8,817	5,290	5,680	Yes
	Methodist	2,521	1,513	2,050	Yes
	Kerrville North	1,328	797	1,133	Yes
	Summit	208	125	450	Yes
	College Cove	379	228	250	Yes
	Ridgewood	692	415	1,350	Yes
	Hilltop	25	49	600	Yes
	Keystone	106	212	500	Yes
	The Heights	71	141	530	Yes
	West Bluff	12	24	50	Yes

Table 8-4: Recommended Elevated Storage Capacity for Existing System

Planning Period	Pressure Plane	Connections	Required Elevated Storage (MG)	Required Total Storage (MG)	Available Elevated Storage (MG)	Available Total Storage (MG)	Meets TCEQ Requirement?
2022	Stadium	6,418	1.28	1.28	2.15	2.41	Yes
	Methodist	2,521	0.50	0.50	1.00	2.00	Yes
	Kerrville North	1,120	0.22	0.22	1.00	1.00	Yes
	Summit	208	0.04	0.04	0.15	0.15	Yes
	College Cove	379	0.08	0.08	1.00	1.00	Yes
	Ridgewood	692	0.14	0.14	1.00	1.00	Yes

In addition to the pumping and storage evaluations, the calibrated model was used to conduct a hydraulic analysis under 2022 maximum day and peak hour demand conditions to evaluate system operations and pressure throughout the distribution system. A 24-hour EPS analysis was performed, which provides a means to evaluate the system over time to assess response to hourly changes in demand, pump cycling, and tanks filling or draining. A maximum day EPS model run evaluates the ability of the system to provide adequate supply to meet demands while maintaining levels in storage facilities.

During a maximum day EPS analysis, the peak hour demand condition is also simulated by setting the highest hourly peaking factor in the diurnal demand pattern of each PZ equal to the maximum day to peak hour peaking factor. Peak hour demand represents the single hour of the year with the highest system demand. Peak hour simulations are used to assess the ability of the distribution system to maintain residual pressure, because the highest demand period typically induces the lowest pressure due to increased headloss throughout the system. Lower demand periods throughout the day are simulated in EPS modeling as well, allowing the system's ability to replenish storage tanks to be evaluated. **Figure 8-1** shows the minimum pressure observed in the model under 2022 demand conditions. Minimum pressures shown on the map represent the lowest pressure experienced at each node during the 24-hour simulation, usually occurring during the peak hour demand. This map helped identify potential problem areas in the system and was used as a tool to assess expected pressure ranges throughout the system. The TCEQ minimum required pressure within a distribution system is 35 pounds per square inch (psi) under non-emergency demand conditions. As seen on **Figure 8-1**, most of the system meets the minimum requirement, but small areas in the Stadium Pressure Plane, along the boundary between Stadium and College Cove Pressure Plane, show modeled pressure below 35 psi. There are improvements identified in **Section 11.1.1** to remedy these areas of low modeled pressure.

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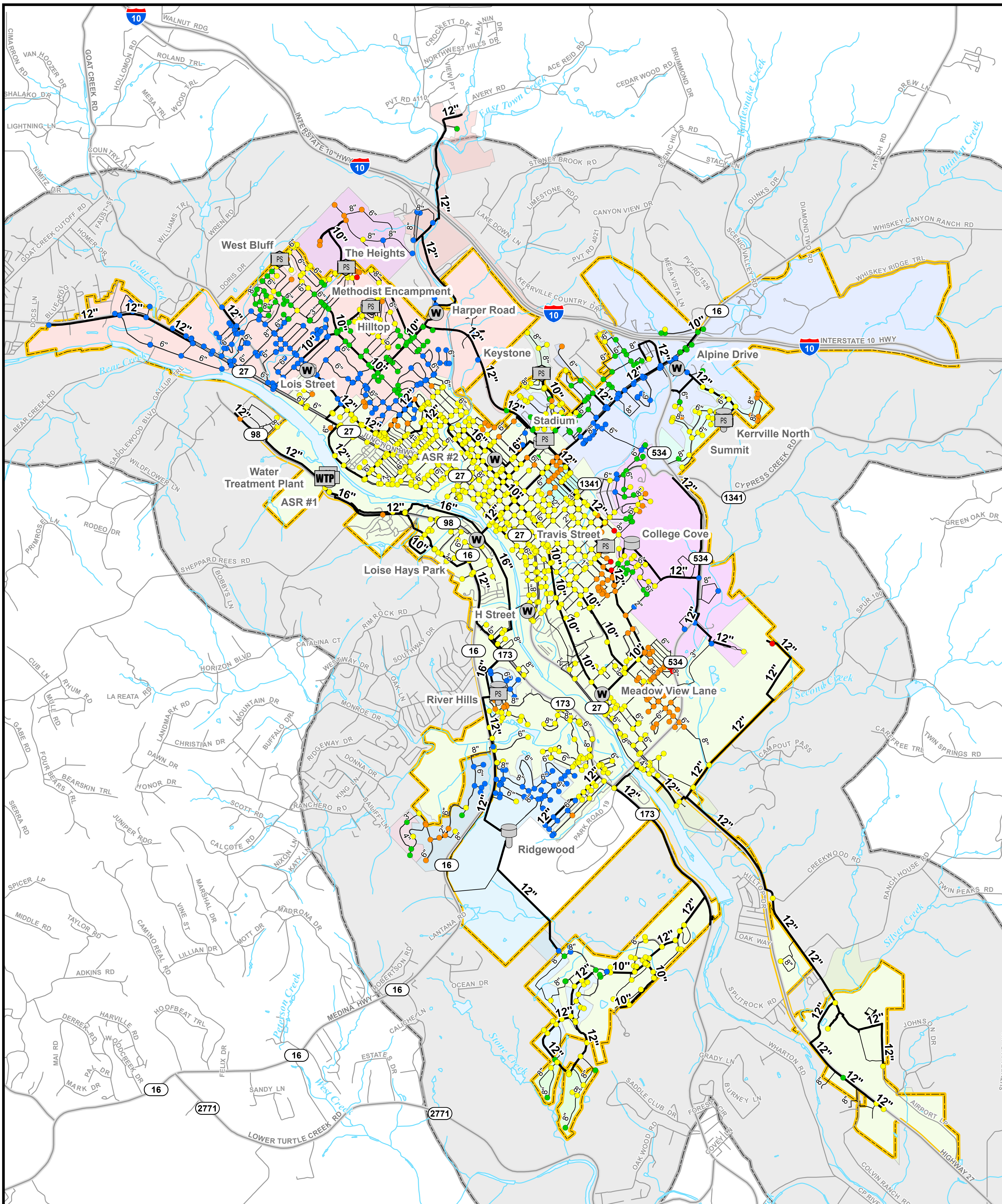

FIGURE 8-1
CITY OF KERRVILLE
EXISTING WATER SYSTEM MINIMUM MODELED PRESSURE

LEGEND

- Elevated Storage Tank
- Ground Storage Tank
- Hydropneumatic Tank
- Production Well
- Pump Station
- Treatment Plant
- 8" and Smaller Water Line
- 10" and Larger Water Line
- Road
- Lakes
- Streams
- City Limits
- ETJ Boundary

MINIMUM PRESSURE

- Less than 35 psi
- 35 - 50 psi
- 50 - 80 psi
- 80 - 100 psi
- Greater than 100 psi

PRESSURE PLANES

- College Cove 1
- College Cove 2 PRV Zone
- Comanche Trace Lower PRV Zone
- Comanche Trace PRV Zone
- Hillcrest PRV Zone
- Hilltop Hydropneumatic Zone
- Kerrville North
- Keystone Hydropneumatic Zone
- Methodist
- Ridgewood
- Stadium
- Stone Ridge PRV Zone
- Summit
- The Heights Hydropneumatic Zone
- West Bluff Hydropneumatic Zone

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8.2.2 Future System Analysis

The recommended planning criteria were utilized to evaluate the water system in future planning years. **Table 8-5**, **Table 8-6**, and **Table 8-7** summarize the recommended production, distribution pumping, and elevated storage capacity, respectively, for each planning year. **Table 8-5** also provides a calculation of total recommended production capacity in terms of gpm per connection for comparison with TCEQ requirements. The recommended production capacity increases each year due to the increasing percentage of new connections, which are assigned the existing system-wide maximum day demand of 0.44 gpm/connection.

Table 8-5: Projected Production Capacity Recommendations

Planning Period	Connections	Required Production (gpm)	Surface Water Production (gpm)	Ground Water Production (gpm)	Total Available Production (gpm)	Meets TCEQ Requirements?
2022	11,338	6,802.8	4,645.0	4,563.0	9,208	Yes
2027	11,969	7,181.3	4,645.0	4,563.0	9,208	Yes
2032	12,635	7,580.8	4,645.0	4,563.0	9,208	Yes
2047	14,863	8,917.8	4,645.0	4,563.0	9,208	Yes

Table 8-6: Projected Distribution Pumping Capacity Recommendations

Planning Period	Pressure Plane	Connections	Minimum Requirement (gpm/ connection)	Required Pumping Capacity (gpm)	Available Pumping Capacity (gpm)	Meets TCEQ Requirement?	Controlling Development Scenario
2027	Stadium	9,555	0.6	5,733	5,680	**No**	D
	Methodist	3,221	0.6	1,932	2,050	Yes	B
	Kerrville North	1,946	0.6	1,167	1,133	**No**	C
	Summit	215	0.6	129	450	Yes	C
	College Cove	382	0.6	229	250	Yes	C
	Ridgewood	692	0.6	415	1,350	Yes	-
	Hilltop	25	2.0	49	600	Yes	-
	Keystone	106	2.0	212	500	Yes	-
	The Heights	71	2.0	141	530	Yes	-
	West Bluff	12	2.0	24	50	Yes	-
2032	Stadium	10,351	0.6	6,210	5,680	**No**	D
	Methodist	3,552	0.6	2,131	2,050	**No**	B
	Kerrville North	2,522	0.6	1,513	1,133	**No**	C
	Summit	624	0.6	374	450	Yes	C
	College Cove	458	0.6	275	250	**No**	C
	Ridgewood	692	0.6	415	1,350	Yes	-
	Hilltop	25	2.0	49	600	Yes	-
	Keystone	106	2.0	212	500	Yes	-
	The Heights	71	2.0	141	530	Yes	-
	West Bluff	12	2.0	24	50	Yes	-
2047	Stadium	13,053	0.6	7,832	5,680	**No**	D
	Methodist	4,815	0.6	2,889	2,050	**No**	B
	Kerrville North	4,153	0.6	2,492	1,133	**No**	C
	Summit	1,814	0.6	1,088	450	**No**	C
	College Cove	946	0.6	567	250	**No**	C
	Ridgewood	1,913	0.6	1,148	1,350	Yes	D
	Hilltop	25	2.0	49	600	Yes	-
	Keystone	288	2.0	576	500	**No**	B
	The Heights	387	2.0	773	530	**No**	B
	West Bluff	12	2.0	24	50	Yes	-

Table 8-7: Projected Elevated Storage Capacity Recommendations

Planning Period	Pressure Plane	Connections	Required Elevated Storage (MG)	Required Total Storage (MG)	Available Elevated Storage (MG)	Available Total Storage (MG)	Meets TCEQ Requirement?	Controlling Development Scenario
2027	Stadium	7,156	1.43	1.43	2.15	2.15	Yes	D
	Methodist	3,221	0.64	0.64	1.00	1.00	Yes	B
	Kerrville North	1,731	0.35	0.35	1.00	1.00	Yes	C
	Summit	215	0.04	0.04	0.15	0.15	Yes	C
	College Cove	382	0.08	0.08	1.00	1.00	Yes	C
	Ridgewood	692	0.14	0.14	1.00	1.00	Yes	-
2032	Stadium	7,952	1.59	1.59	2.15	2.15	Yes	D
	Methodist	3,552	0.71	0.71	1.00	1.00	Yes	B
	Kerrville North	1,898	0.38	0.38	1.00	1.00	Yes	C
	Summit	624	0.12	0.12	0.15	0.15	Yes	C
	College Cove	458	0.09	0.09	1.00	1.00	Yes	C
	Ridgewood	692	0.14	0.14	1.00	1.00	Yes	-
2047	Stadium	9,432	1.89	1.89	2.15	2.15	Yes	D
	Methodist	4,815	0.96	0.96	1.00	1.00	Yes	B
	Kerrville North	2,340	0.47	0.47	1.00	1.00	Yes	C
	Summit	1,814	0.36	0.36	0.15	0.15	**No**	C
	College Cove	946	0.19	0.19	1.00	1.00	Yes	C
	Ridgewood	1,913	0.38	0.38	1.00	1.00	Yes	D

Water system improvements were developed to accommodate the anticipated growth in the water service area through 2047 based on the recommended planning criteria.

Other challenges facing the water system include addressing existing deficiencies in pumping capacity and providing adequate infrastructure to serve growth. The following key observations and recommendations resulted from modeling and evaluating the distribution system.

- **Development Scenario A**

  The Stadium Pressure Plane requires more high service pumping to serve future growth. To serve projected developments, FNI recommends establishing a new pressure plane to the West of Stadium Pressure Plane. This will require additional elevated storage and high service pump capacity.

- **Development Scenario B**

  The Keystone Hydropneumatic Zone will require additional pumping capacity to serve projected future growth. The Methodist Pressure Zone will have insufficient pumping capacity if Methodist Well remains offline.

- **Development Scenario C**

  The Kerrville North, Summit, and College Cove Pressure Zones are all projected to be deficient in pumping capacity to serve future growth. Summit Pressure Plane will also require additional storage to serve future growth and meet TCEQ minimum requirements.

- **Development Scenario D**

  The Stadium Pressure Plane requires more high service pumping to serve future growth in the lower Stadium area.

Once these improvements were added to the existing water system, hydraulic analyses were performed under 2047 maximum day and peak hour demand conditions. A 24-hour EPS simulation was performed under maximum day and peak hour demand conditions for each future planning scenario to develop system operations and verify effectiveness of system improvements. Any areas of high headloss, high velocity (i.e., greater than 5 feet per second), or low pressure were adjusted accordingly to verify sizing of system improvements. With proposed improvements, projected minimum pressures in the system remain above 36 psi. Improvements are further discussed in **Section 11.2.1.**

8.3 FIRE FLOW ANALYSIS

Fire flow is a reserved volume in the water distribution system for emergency use. To evaluate the fire suppression capabilities of the system, a fire flow analysis was conducted under existing maximum day demand conditions. TCEQ requires a minimum pressure of 20 psi be maintained while delivering the fire flow demand. For this analysis, a steady-state model run was utilized to calculate the available fire flow at each node in the system with a pressure of 20 psi and maximum velocity of 10 feet per second in the water main. **Figure 8-2** shows the results of the fire flow simulation. The majority of the water system can provide at least 1,000 gpm, which is a typical residential fire flow demand. Available fire flows below 1,000 gpm are due to small diameter lines in isolated areas. Upsizing of smaller lines and looping are two methods to improve low fire flow. These improvements were incorporated into the CIP project recommendations discussed in further detail in **Section 11.1.1**. Results of the fire flow analysis under existing maximum day demand conditions with the proposed improvements is shown on **Figure 8-3**. Results are for planning purposes only and are not a substitute for detailed design. Available fire flow is calculated at model junctions and interpolated between points, actual fire flow at hydrants may vary. Available fire flow for commercial, multi-family, and industrial land uses should be evaluated on a case-by-case basis.

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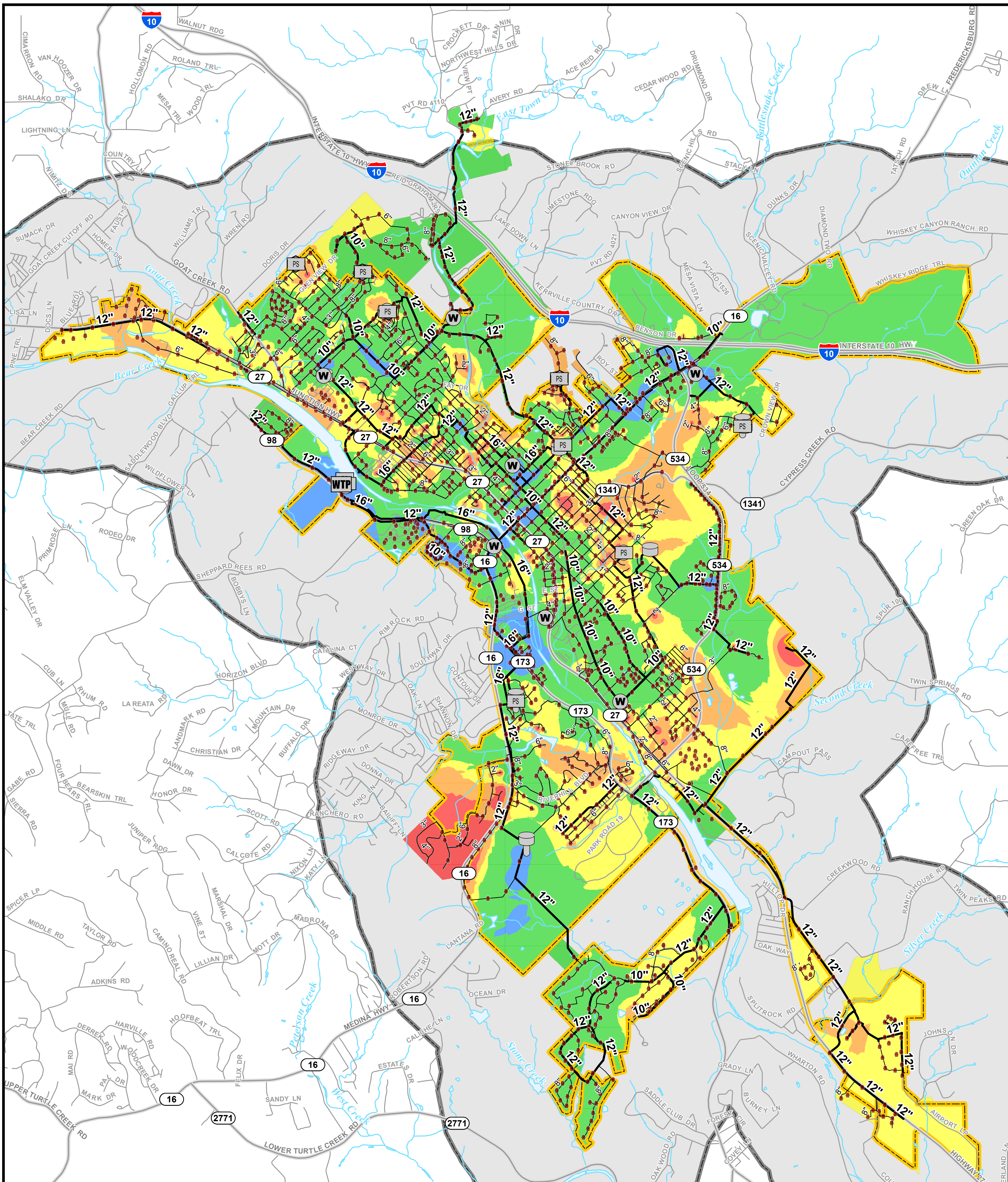

FIGURE 8-2

CITY OF KERRVILLE

EXISTING WATER SYSTEM AVAILABLE FIRE FLOW

LEGEND

- Elevated Storage Tank
- Ground Storage Tank
- Hydropneumatic Tank
- Production Well
- Pump Station
- Treatment Plant
- 8" and Smaller Water Line
- 10" and Larger Water Line
- Fire Hydrant
- Road
- Lakes
- Streams
- City Limits
- ETJ Boundary

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AVAILABLE FIRE FLOW

- Less than 500 gpm
- 500 - 1,000 gpm
- 1,000 - 1,500 gpm
- 1,500 - 3,500 gpm
- Greater than 3,500 gpm

Planning Purposes Only
Results are for planning purposes only and are not to be used as a substitute for detailed design. Available fire flow for potential developments should be evaluated on a case-by-case basis. Available fire flow is calculated at model junctions and interpolated between points, actual fire flow at hydrants may vary.

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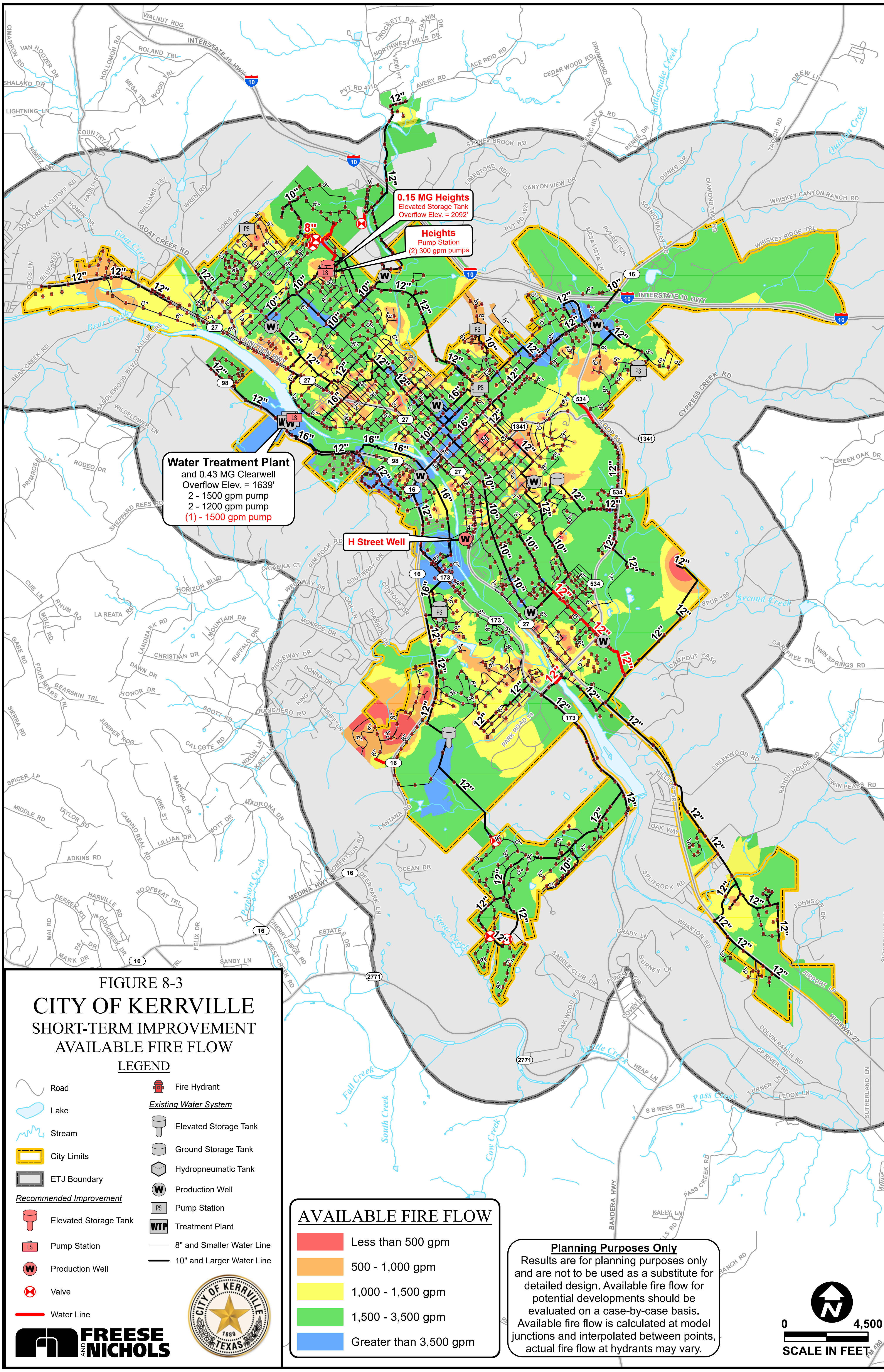

FIGURE 8-3
CITY OF KERRVILLE
SHORT-TERM IMPROVEMENT
AVAILABLE FIRE FLOW

Planning Purposes Only
Results are for planning purposes only and are not to be used as a substitute for detailed design. Available fire flow for potential developments should be evaluated on a case-by-case basis. Available fire flow is calculated at model junctions and interpolated between points, actual fire flow at hydrants may vary.

8.4 WATER AGE ANALYSIS

Water age modeling was conducted under existing average day demand conditions to identify areas in the distribution system with high water age. While water age does not directly cause poor water quality, it is known that chlorine degrades over time and disinfection byproducts levels increase over time. Therefore, decreasing the water age can help reduce the formation of disinfection byproducts. A 30-day simulation was performed to ensure a consistent pattern of water age had been established in the model. The results of existing water age analysis are shown on **Figure 8-4**.

The majority of Kerrville North, Methodist, and Stadium Pressure Planes have a relatively low water age of less than 4 days old. The outlying sections of these pressure planes and the College Cove, Ridgewood and Summit Pressure Planes see much higher water age. Dead end lines, low demands and low tank turnover contribute to high water age in these areas. In addition to modeling the existing system water age, a water age analysis was performed on the system with proposed short-term CIP projects incorporated. The results of the improved system water age analysis are shown on **Figure 8-5**. The Ridgewood Pressure Plane, Summit Pressure Plane, and outlying areas of the pressure planes continue to experience high water age.

A combination of operational changes and system improvements are generally recommended to help improve water age. The simplest approach to water age reduction is to change existing system operations at storage tanks to help facilitate tank turnover and mixing. Pump stations that fill storage tanks are typically set to turn on and off at certain tank levels. By lowering the level at which pumps turn on and increasing the level at which pumps turn off, a larger volume of water is entering and subsequently leaving the tank; therefore, increasing the overall tank turnover. There are limits on the lowest and highest water levels inside tanks that must be adhered to when adjusting storage tank operations, such as overflow elevations and minimum levels needed to sustain system pressures. A tank mixing system may also be implemented to help reduce water age in the system. A tank mixing system prevents old water from sitting in the tank and becoming stagnant. Lastly, regular system flushing in specific areas with high water age can adequately address water age issues. Water age improvements may be implemented at the discretion of City Staff. General water system improvements, including improvements to water age, are detailed in **Section 11.1.1**.

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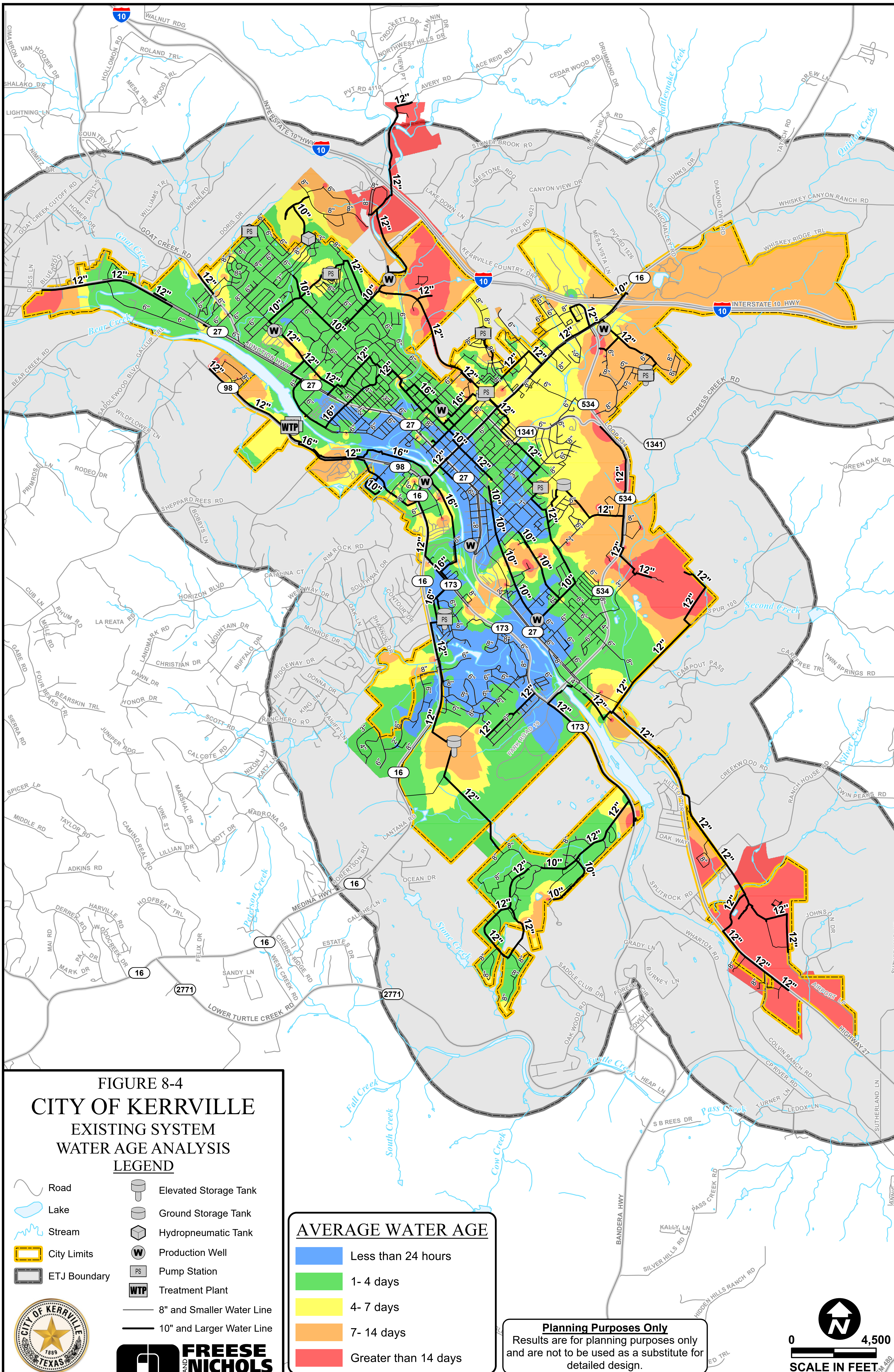

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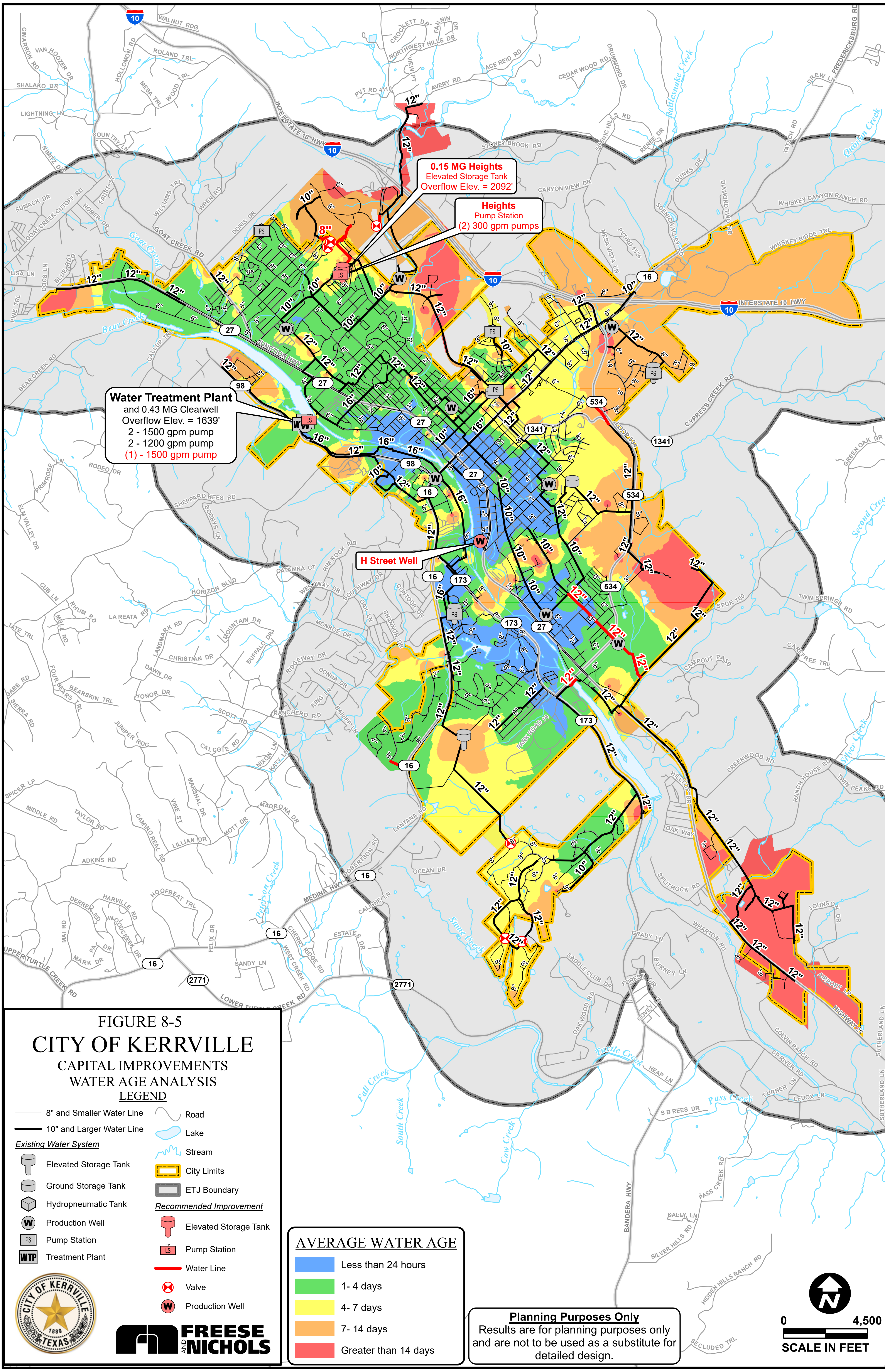

9.0 WASTEWATER SYSTEM HYDRAULIC ANALYSIS

Following model calibration for both dry and wet weather storm events, the next step in the wastewater master planning process is to use the hydraulic model to evaluate the existing system and provide recommendations to address existing deficiencies and future projections. Hydraulic analyses were conducted to identify deficiencies in the City's existing wastewater collection system and to establish a capital improvement plan to improve the existing system and accommodate projected wastewater flows through the 2047 planning period. Various combinations of improvements and modifications were investigated to determine the approach for conveying projected flows. Parameters considered when developing the improvements plan included conveying peak wet weather flows, maintaining proper velocities, reducing surcharging, and eliminating projected sanitary sewer overflows.

9.1 PLANNING CRITERIA

The determination of the planning criteria is an important process as it guides the identification of potential CIP projects. The following sections summarize the planning criteria used in the hydraulic analysis, system evaluation, and capacity CIP determination.

Planning criteria for analyzing existing, new, and replacement facilities were developed in accordance with minimum TCEQ standards as outlined in *Chapter 217: Design Criteria for Domestic Wastewater Systems, Subchapter C: Conventional Collection Systems*.

The capacity improvement trigger utilized in this evaluation consists of two parts. First, collection system infrastructure identified for improvement includes pipes where the existing or projected flow (q) exceeds the full capacity of the pipe (Q) calculated by Manning's equation based on the diameter, slope, and Manning's roughness coefficient (typically 0.013). This capacity utilization of a pipe is also known as the q/Q ratio. Lines identified as capacity restrictions have a $q/Q \geq 1$. Second, the system is allowed to utilize the system storage capacity by permitting a degree of surcharging while still preventing potential sanitary sewer overflows (SSOs). FNI recommends allowing surcharging to within three feet of the manhole rim elevation. Maximizing the use of existing system storage affords the collection system the ability to attenuate and dampen the effects of wet weather peak flow conditions. Utilizing storage within the existing collection system is in accordance with TCEQ regulations (217C) provided the causes of projected SSOs are remedied. New proposed CIP improvements are required to be sized to handle the peak projected flow without surcharging.

Planning criteria for the size of gravity sewer lines are based on maintaining a minimum a velocity range of 2 feet per second (fps) to 10 fps during full pipe flow, according to TCEQ regulations. New wastewater line sizes are based on selecting the smallest diameter to convey the projected peak instantaneous flow within 90% of pipe capacity ($q/Q \leq 0.9$). This allows for an additional factor of safety and flexibility to prevent newly constructed pipes from surcharging if development projections and/or peak wet weather flows are greater than assumed in this study. Additionally, slopes for new lines serving undeveloped areas meet minimum slope requirements as set by TCEQ.

Evaluation of lift stations is primarily based on the firm pumping capacity of the lift station. The firm capacity of a lift station is defined as the pumping capacity with the largest available pump out of service. Lift Station improvements are evaluated based on the projected peak wet weather flow. This assumes that the lift station wet well provides adequate storage to attenuate the instantaneous peak flow and pumps are sized to handle peak wet-weather flow. Lift stations with projected peak wet weather flows greater than the firm capacity are identified for improvement to increase the firm capacity at or above the projected peak flow.

Force mains are evaluated based on the velocity within the pipeline. Force mains with velocities greater than 8 fps are identified for improvement. Force mains are sized based on the lift station pumping capacity with a maximum velocity of 8 fps at firm capacity, a minimum velocity of 2 fps with the smallest pump in service, and a once daily flushing velocity of 5 fps per TCEQ regulations.

Projected peak flows for existing and future planning periods are determined by applying the selected peaking factors to the calibrated wastewater hydraulic model. The dry and wet weather parameters determined during calibration are utilized to project the collection system’s response to the applied storm and determine the RDII. It is assumed that the existing RDII rate remains constant through all future planning periods and no I/I reduction was applied to future peak flow projections.

9.2 COLLECTION SYSTEM ANALYSIS

A hydraulic analysis and system evaluation was conducted on the existing and future wastewater collection system to identify capacity deficiencies. The results of the existing system evaluation provide the basis for near-term capacity improvements while future evaluation drives long-term improvement recommendations.

9.2.1 Existing System Analysis

The hydraulic wastewater model was utilized to apply the existing (2022) projected peak wet weather flows to the current wastewater collection system to identify potential existing system capacity restrictions. Results of the existing system analysis are shown on **Figure 9-1**. Wastewater lines in red are overloaded during existing peak wet weather events, and the projected flow exceeds the capacity of the wastewater line (q/Q > 1). Manholes in red are model predicted SSOs. Manholes in yellow indicate locations where the model predicts surcharging within three feet of the manhole rim.

Hydraulic analysis indicates that there are several areas with projected capacity restrictions under existing system conditions. These capacity restrictions primarily correspond to areas with planned capital improvements that are currently under design or construction. Capacity restrictions in the existing system analysis include the following:

Knapp Force Main

A capacity restriction is identified downstream of the Knapp Lift Station. The existing 12-inch gravity main receives flow that exceeds the full pipe capacity. Manholes along the interceptor are also identified as surcharging within three feet of the manhole rim.

Ingram Interceptor

A significant capacity restriction is identified downstream of the Ingram Lift Station. The existing 12-inch gravity main receives flow that exceeds the full pipe capacity when the second lift station pump turns on. Manholes along the interceptor are also identified as surcharging within three feet of the manhole rim.

Quinlan Interceptor

A capacity restriction is identified upstream from the Quinlan Lift Station, in the interceptor parallel to Quinlan Creek. The 12- and 10-inch gravity main receives peak flow that exceeds the full pipe capacity. Manholes along the interceptor also are identified as surcharging within three feet of the manhole rim.

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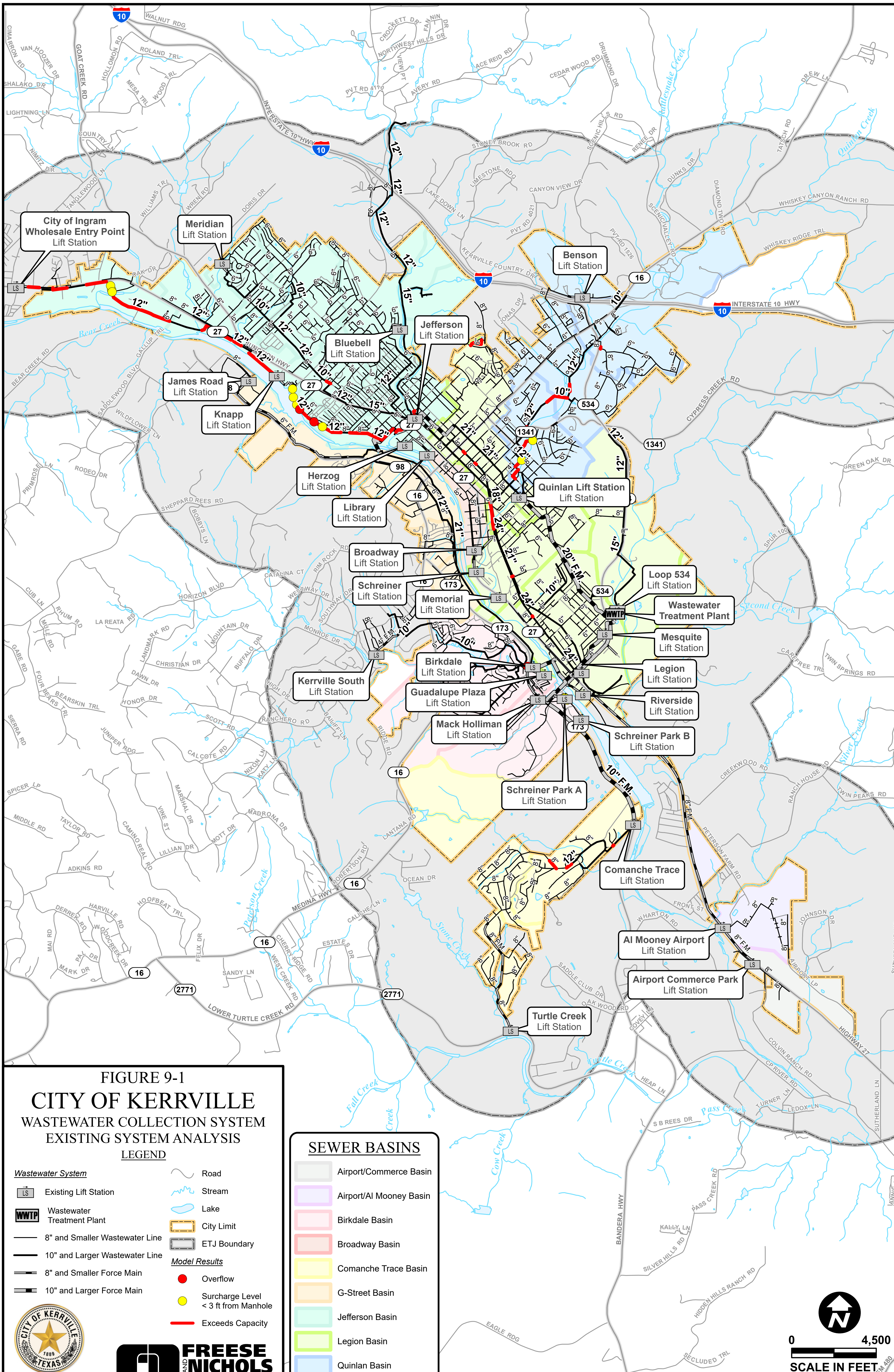

FIGURE 9-1

CITY OF KERRVILLE

WASTEWATER COLLECTION SYSTEM
EXISTING SYSTEM ANALYSIS

LEGEND

Wastewater System

- Existing Lift Station
- Wastewater Treatment Plant
- 8" and Smaller Wastewater Line
- 10" and Larger Wastewater Line
- 8" and Smaller Force Main
- 10" and Larger Force Main
- Road
- Stream
- Lake
- City Limit
- ETJ Boundary

Model Results

- Overflow
- Surcharge Level < 3 ft from Manhole
- Exceeds Capacity

SEWER BASINS

- Airport/Commerce Basin
- Airport/Al Mooney Basin
- Birkdale Basin
- Broadway Basin
- Comanche Trace Basin
- G-Street Basin
- Jefferson Basin
- Legion Basin
- Quinlan Basin

FREESE AND NICHOLS

9.2.2 Future System Analyses

The hydraulic wastewater model was utilized to apply future projected peak wet weather flows to the current wastewater collection system to identify potential capacity restrictions for all future planning periods. As the system develops, existing capacity deficiencies are compounded, and new capacity restrictions are identified. The following key observations and recommendations resulted from modeling and evaluating the collection system.

Development Scenario A

- No additional capacity restrictions are identified in Development Scenario A.

Development Scenario B

- Development Scenario B creates a capacity restriction at the James Road Lift Station. A detailed lift station analysis is included in **Section 9.3**.

Development Scenario C

- No additional capacity restrictions are identified in Development Scenario C.

Development Scenario D

- Development Scenario D creates a capacity restriction at the Turtle Creek, Al Mooney, Airport Commerce, and Comanche Trace Lift Stations. A detailed lift station analysis is included in **Section 9.3**. A capacity restriction also occurs in the gravity main, downstream of the Comanche Trace Force Main.

Recommended improvements intended to address the capacity restrictions are further discussed in **Section 11.2.2**.

9.3 LIFT STATION EVALUATION

The City owns and operates 26 collection system lift stations. 26 lift stations were included in the wastewater hydraulic model and evaluated for hydraulic capacity under existing and future conditions. Available Living Unit Equivalents (LUE) of each lift station were determined as part of this evaluation. An LUE is a unit equivalent to the flow from a single-family residence. Current available LUEs were determined based on the difference between existing peak flow rate and existing firm capacity, converted to a connection count based on the design criteria discussed in **Section 7.** One existing connection was determined to be equivalent to one existing LUE. In general, a single-family residence has one meter, corresponding to one LUE. Other land-use types may produce more wastewater flow per meter, resulting in a higher LUE per meter. Future connections were assigned a land use based on their location and then converted to LUEs based on land use type. The results of the lift station hydraulic evaluation are shown in **Table 9-1**. The maximum projected future LUEs are included for each lift station per the planning criteria discussed in **Section 9.1**. These maximum peak LUEs are based on the maximum projected growth from the four development scenarios. Projected LUEs in excess of the existing lift station firm capacity are highlighted in **red** text.

Table 9-1: Lift Station Hydraulic Analysis

Lift Station	Existing Capacity (gpm)	Force Main Diameter (in)	Force Main Capacity at 8 ft/s (gpm)	Existing Peak Flow Rate (gpm)	Existing LUEs	Available LUEs
Airport Commerce	150	8	1,253	102	14	7
Al Mooney	210	8	1,253	**266**	37	**-80**
Benson	--	4	313	10	8	-
Birkdale	6,300	20	7,834	2,675	1,012	5,156
Bluebell	60	8	1,253	10	35	71
Broadway	500	8	1,253	256	679	347
Comanche Trace	600	10	1,958	**835**	606	**-334**
Guadalupe Plaza	--	4	313	67	69	-
Herzog	70	6	705	53	118	24
James Road	*500*	6	705	37	27	658
Jefferson	6,000	16	5,013	1,950	4,952	5,760
Kerrville South	180	4	313	111	55	98
Knapp	560	6	705	**918**	1,059	**-509**
Legion	6,300	20	7,834	4,021	5,776	3,241
Library	--	4	313	0	6	-
Loop 534	1,700	12	2,820	161	24	2,189
Mack Holliman	--	6	705	10	243	-
Memorial	--	--	--	0	0	0
Meridian	175	4	313	25	125	213
Mesquite	--	2	78	0	0	0
Quinlan	2,400	12	2,820	1,454	3,083	1,345
Riverside	50	8	1,253	12	21	54
Schreiner	150	6	705	0	124	213
Schreiner Park A	27	4	313	10	12	24
Schreiner Park B	27	4	313	0	0	38
Turtle Creek	400	8	1,253	248	192	216

10.0 RISK BASED CONDITION ASSESSMENT

FNI was tasked with performing a risk-based condition assessment (RBA) for the water distribution system and wastewater collection system. The risk assessment consisted of an evaluation of the condition and criticality of the current facilities and linear assets based on available data and field inspections. The results of the condition assessments were used to generate improvement projects and recommendations included in the capital improvement plan.

10.1 FACILITY FIELD ASSESSMENT

In March 2022, FNI and City staff conducted field assessments of 16 water system pump stations and wells, and 6 wastewater system lift stations. Water and wastewater system facility inspection locations are shown on **Figure 10-1** and **Figure 10-2**, respectively. The assessment included site visits and visual inspection of the facilities to determine the overall condition of the station and specific condition of major components including pumps and motors, electrical equipment, instrumentation, piping and valves, structure, and site. Detailed condition scoring parameters and weighting were determined in coordination with City staff and are shown in **Table 10-1**. Notes and photographs were taken for each inspection site to document existing condition and develop a condition score for all major components on a 1 to 5 scale as shown in **Table 10-2**. A score of 1 is considered “new” condition with no steps recommended to maintain condition, while a score of 5 is considered “very poor” condition and the asset has probable or eminent failure unless rehabilitation steps are taken. General facility information was also noted as well as comments from City staff. Inspection sheets for all inspected facilities can be found in **Appendix D**. The results of the facility condition assessment were incorporated into recommended improvements that help mitigate the risk of asset failure and proactively manage system degradation.

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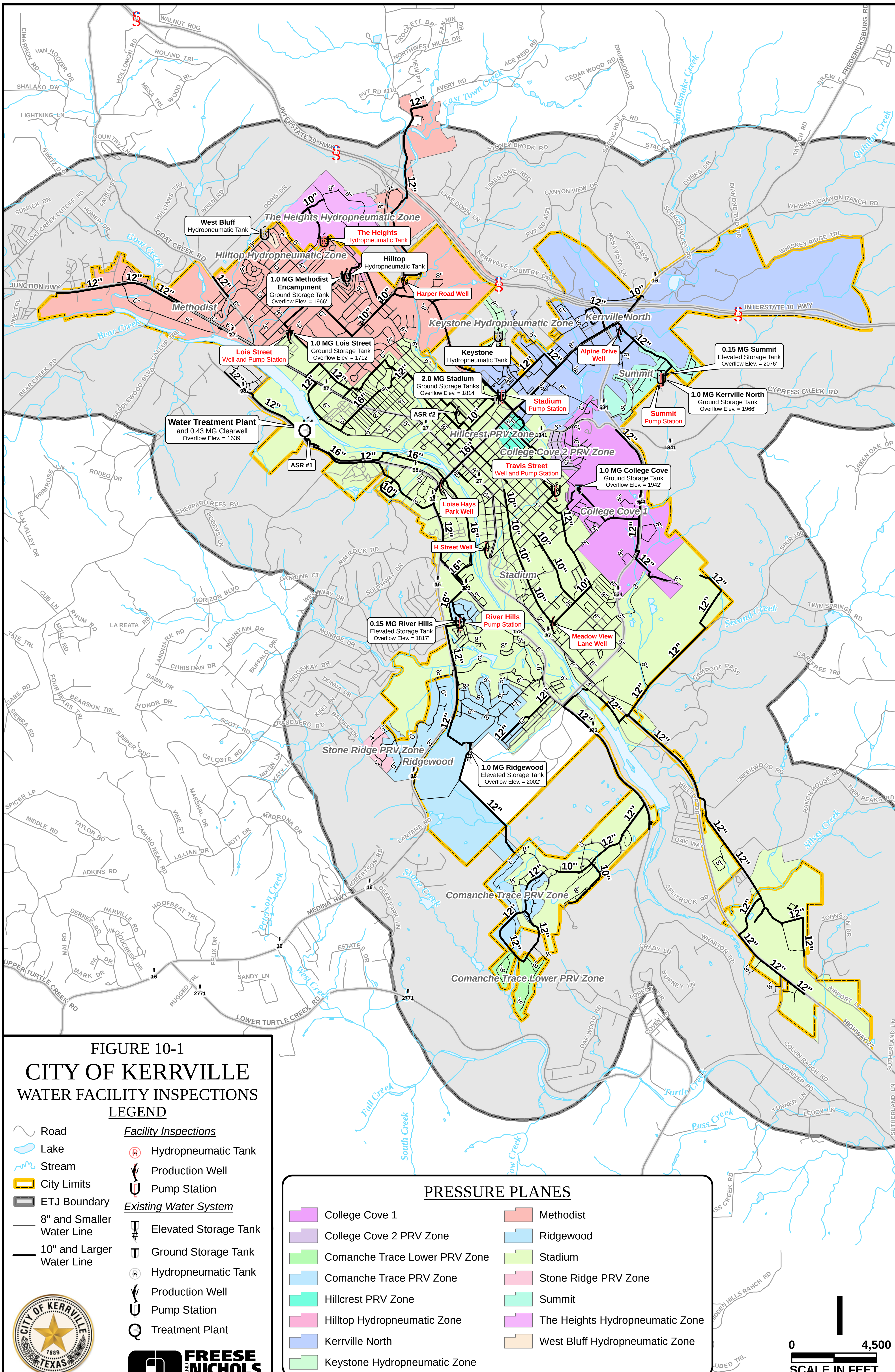

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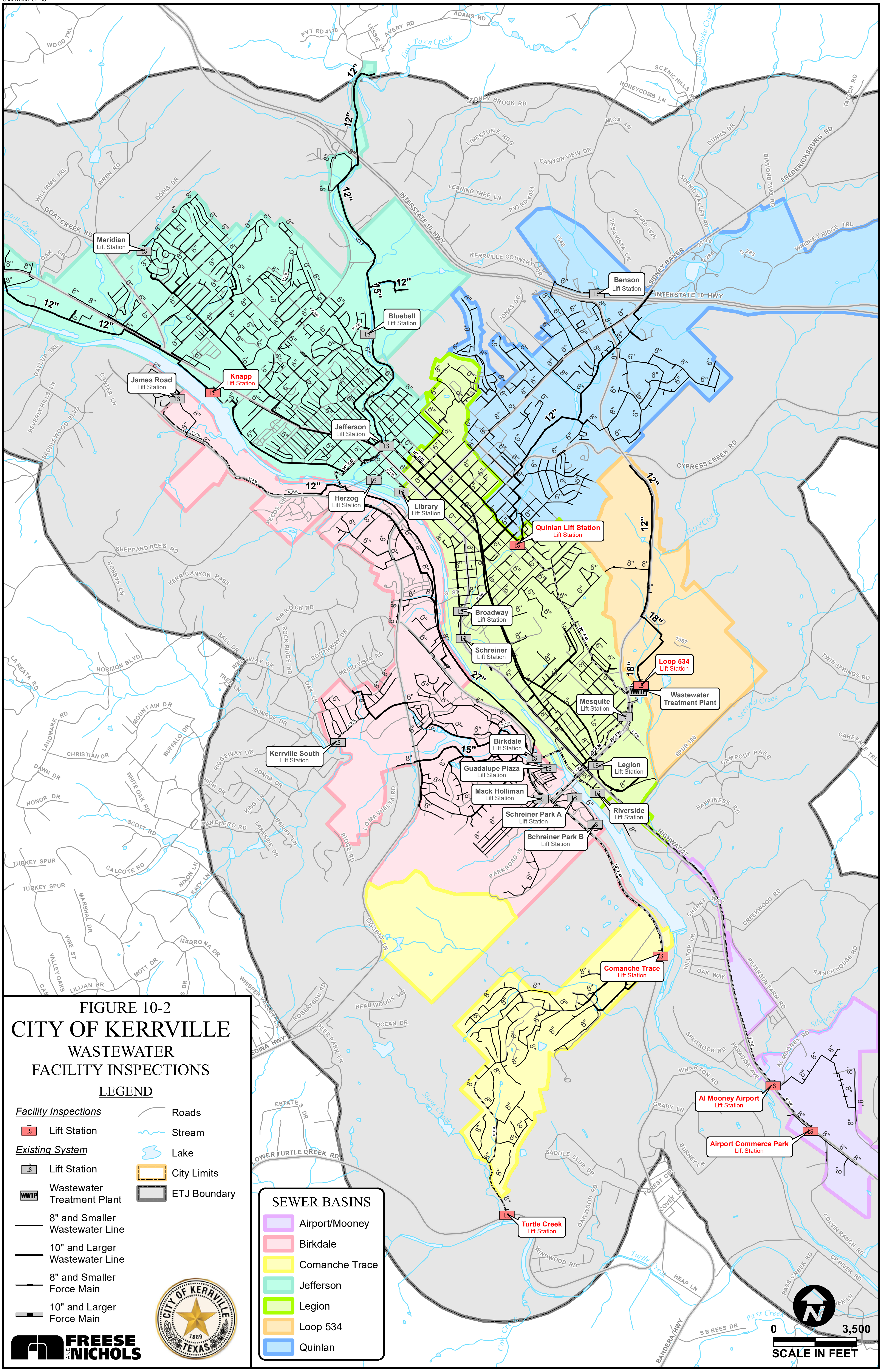

FIGURE 10-2
CITY OF KERRVILLE
WASTEWATER
FACILITY INSPECTIONS

Table 10-1: Facility Inspection Scoring Parameters

Parameter	Details	Weighting	Point Range
Pumps and Motors	Pumps, motors, and general components, including pumping capacity.	20%	1 - 5
Electrical	Electrical equipment including service, disconnect, starters, back-up power, and other general components.	25%	1 - 5
Instrumentation	Instrumentation including floats, levels, SCADA, alarms, and controls.	15%	1 - 5
Structure	Facility structure including foundation, storage, hatches, corrosion, cracks, and general structural condition.	15%	1 - 5
Piping and Valves	Piping and valves including risers, mains, check valves, and bypass connections.	10%	1 - 5
Mechanical*	Mechanical components including ventilation, chemical dosing, and odor control.	5%	1 - 5
Site	General site condition including drainage, access, security, fencing, and cover.	10%	1 - 5

*Mechanical was not included in water system facilities, 5% weight was added to pumps and motors.

Table 10-2: Facility Inspection Scoring Guidelines

Score	Guidelines
1	New condition, no improvements recommended to maintain function
2	Good condition, minor improvements recommended to enhance performance
3	Fair condition, improvements recommended to improve performance or efficiency
4	Poor condition, improvements recommended to maintain reliability
5	Very poor condition, rehabilitation or replacement required

10.1.1 Water System Facilities

The results of the water system field assessments were compiled, and the major component scores were weighted to generate an overall condition score for each facility. A summary of the inspection results is shown in **Table 10-3**. The majority of inspected pump stations and wells were found to be in "good" to "fair" condition with scores ranging from 1.25 to 2.95. H Street Well was determined to be in the worst condition of the water facilities and is currently offline. Higher condition scores typically resulted from corroded valves and pipes, lack of electrical equipment protection, and site conditions. Detailed recommendations for rehabilitation are discussed in detail in **Section 11.0**. General recommendations include replacing gaseous chlorine with liquid bleach, constructing appropriate buildings to protect production well pumps and piping similar to Methodist Encampment, provide lighted canopies over exposed electrical boxes, conducting electrical power system study, and re-coating exposed piping and valves.

Table 10-3: Water Facility Scoring Results

Facility Name	Pumps and Motors	Electrical	Instrumentation	Structure	Piping and Valves	Site	Condition Rating
Lois Street PS	3	4	3	2	3	1	2.90
River Hills	2	3	2	3	2	1	2.30
Stadium	3	2	2	2	3	3	2.45
Summit	3	1	2	2	3	1	2.00
Travis Street PS	2	2	3	2	2	1	2.05
High Service	2	1	1	1	2	1	1.35
The Heights	1	2	2	1	1	1	1.40
Hilltop	2	2	2	1	1	1	1.65
West Bluff	3	2	3	2	2	3	2.50
Alpine Drive	2	2	1	2	2	1	1.75
H Street	5	2	3	3	2	1	2.95
Harper Road	2	2	1	2	2	2	1.85
Hays Park	2	2	1	2	2	1	1.75
Lois Street Well	3	2	1	2	1	1	1.90
Meadow View Lane	1	2	1	2	2	1	1.50
Methodist Encampment	1	2	1	1	1	1	1.25
Weight	**25%**	**25%**	**15%**	**15%**	**10%**	**10%**	**100%**
Average	**2.31**	**2.06**	**1.81**	**1.88**	**1.94**	**1.31**	**1.97**

10.1.2 Wastewater System Facilities

The results of the field assessments were compiled, and the major component scores were weighted to generate an overall condition score for each lift station. A summary of the inspection results is shown in **Table 10-4**. The majority of inspected lift stations were found to be in "good" condition with scores ranging from 1.5 to 2.4. Quinlan was determined to be in “fair” condition with a score of 2.90. Airport Commerce Park and Al Mooney Lift Stations were determined to be in “poor” condition with scores of 3.10 and 3.45, respectively. Higher condition scores typically resulted from corrosion, lack of back-up power, and poor site conditions. Detailed recommendations for rehabilitation are discussed in detail in **Section 11.0**. Airport Commerce Park and Al Mooney Lift Stations are identified for rehabilitation projects intended to address corrosion issues and poor site conditions. Quinlan Lift Station is identified for a rehabilitation project intended to address debris issues. General recommendations for lift stations include implementing backup power and backup controls for critical, high flow, or remote sites.

Table 10-4: Wastewater Facility Scoring Results

Facility Name	Pumps and Motors	Electrical	Instrumentation	Structure	Piping and Valves	Mechanical	Site	Condition Rating
Airport Commerce Park	3	4	3	4	1	3	2	3.10
Al Mooney	3	3	4	5	4	3	2	3.45
Comanche Trace	2	2	2	4	1	2	1	2.10
Knapp	2	2	2	4	2	2	3	2.40
Loop 534	1	1	1	4	1	2	1	1.50
Quinlan	3	2	4	4	2	2	3	2.90
Turtle Creek	2	3	2	1	1	2	1	1.90
Weight	**20%**	**25%**	**15%**	**15%**	**10%**	**5%**	**10%**	**100%**
Average	**2.29**	**2.43**	**2.57**	**3.71**	**1.71**	**2.29**	**1.86**	**2.48**

10.2 PIPELINE ASSESSMENT

In addition to facility inspection, available linear asset data was analyzed to develop renewal recommendations. Field inspection data such as closed caption television (CCTV) was not available for use in the condition assessment of linear assets. FNI utilized available GIS data and work order history to evaluate the condition and criticality of water and wastewater pipelines.

10.2.1 Water System Pipelines

Pipeline material, diameter, and work order history was available for the waterline assessment. Waterline material is summarized on **Table 10-5** and **Figure 10-3**. Waterline diameter is summarized on **Table 10-6** and **Figure 10-4**. The City also provided distribution system work order history for the five-year period from 2017 through April of 2022. Using GIS, the address information in the work order history can be spatially located in a process called geocoding. FNI geocoded the work order data to identify locations with a high density of consistent issues and repeated remediation efforts. A heat map of the geocoded work order history is shown on **Figure 10-5**.

Table 10-5: Waterline Material Summary

Material	Count	Length (ft)	Percent
Asbestos Cement	944	430,353.8	37.12%
Cast Iron	543	189,522.0	16.35%
Copper	17	5,485.4	0.47%
Ductile Iron	90	40,426.6	3.49%
Galvenized	41	13,183.5	1.14%
PVC	933	480,028.1	41.40%
Stainless	1	190.0	0.02%
Unknown	2	305.0	0.03%
Total	**2,571**	**1,159,494.4**	**100.00%**

Table 10-6: Waterline Diameter Summary

Diameter	Count	Length (ft)	Percent
Less than 4-inches	277	108,911.7	9.39%
6 to 10-inches	1,929	803,054.1	69.26%
Greater than 12-inches	365	247,528.5	21.35%
Total	**2,571**	**1,159,494.4**	**100.00%**

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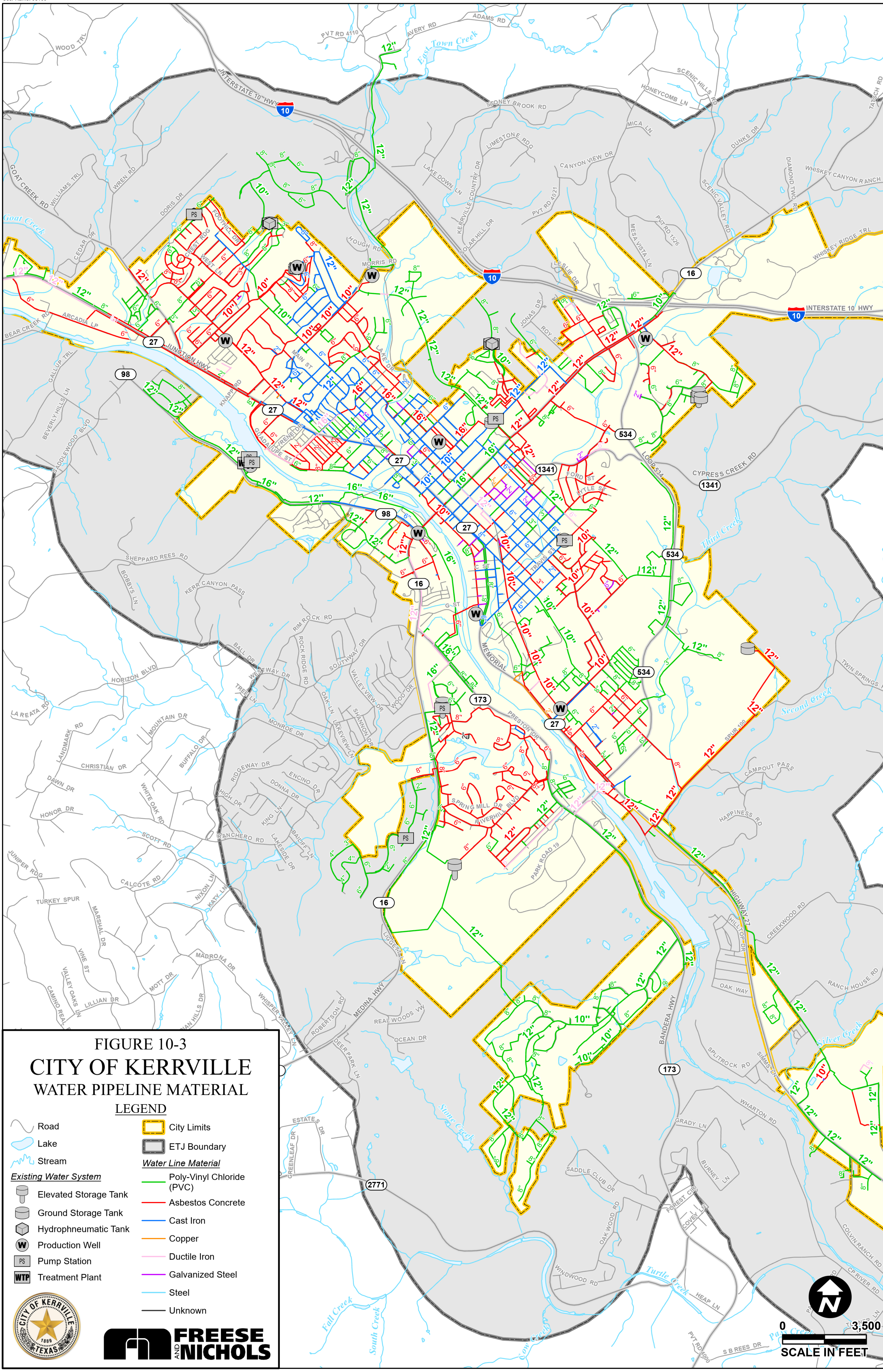

FIGURE 10-3
CITY OF KERRVILLE
WATER PIPELINE MATERIAL

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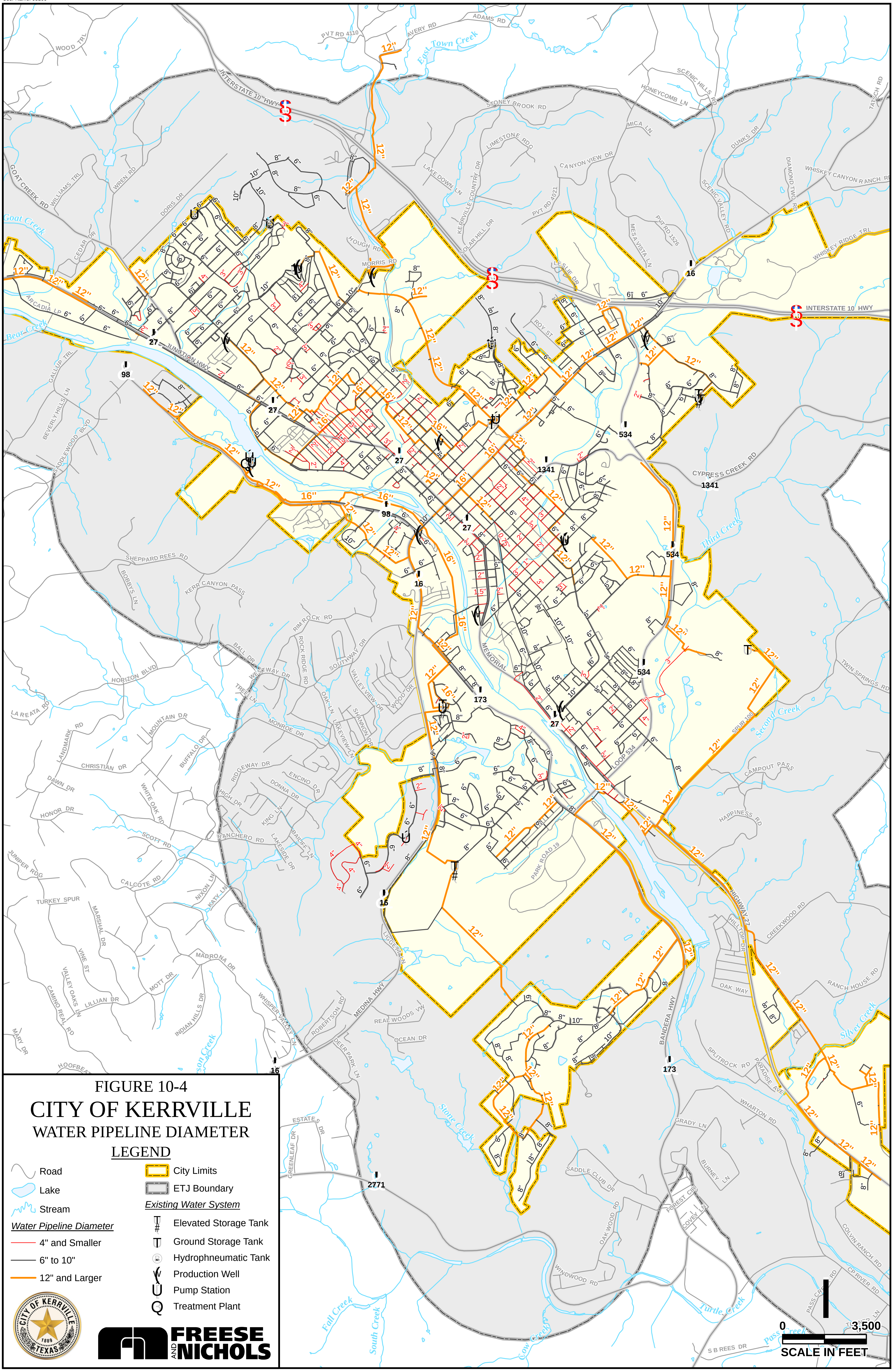

FIGURE 10-4
CITY OF KERRVILLE
WATER PIPELINE DIAMETER

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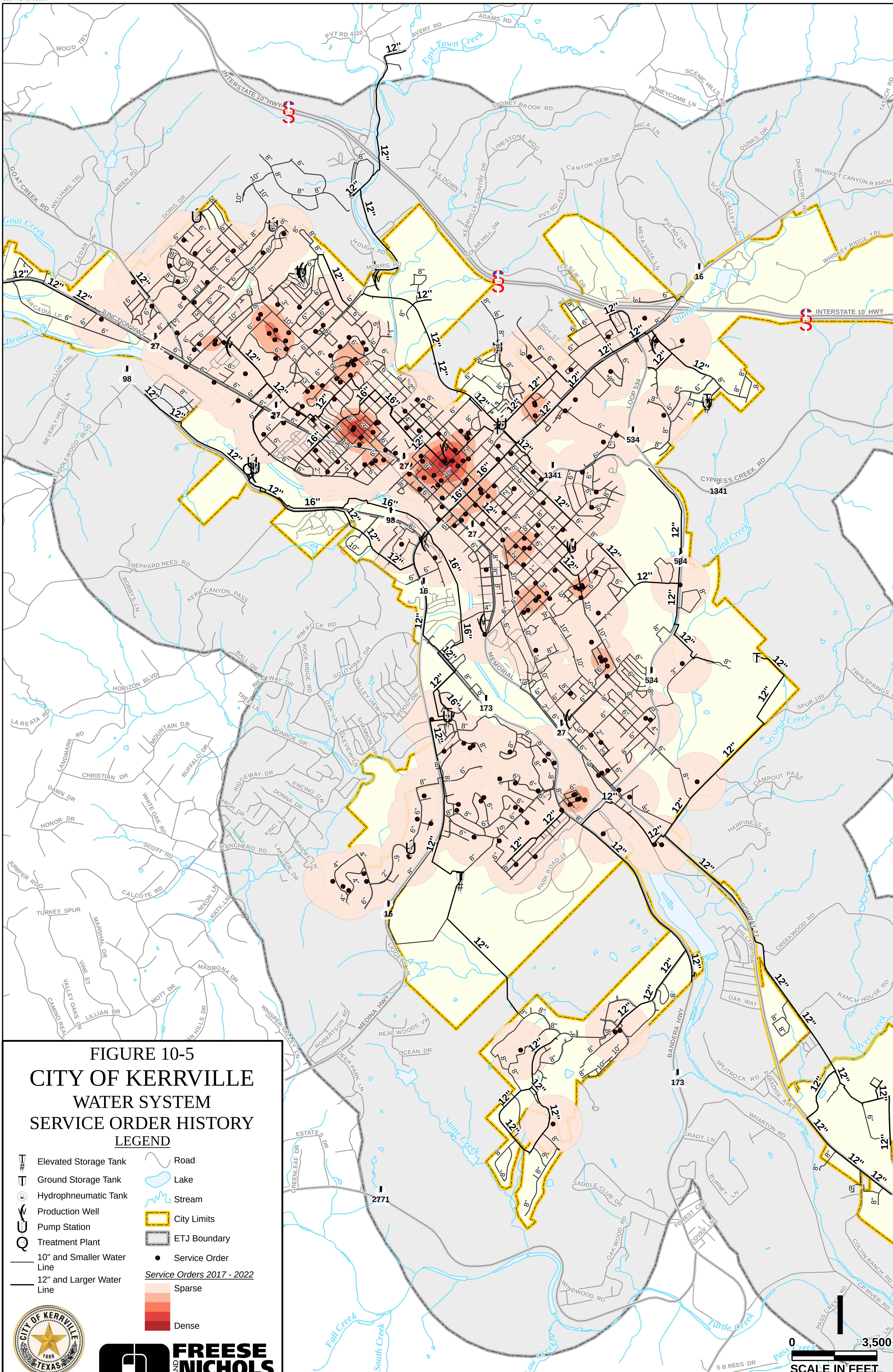


FNI utilized the available data to identify and prioritize an annual waterline replacement program. FNI and City Staff identified cast-iron water mains to be the category of pipeline most at risk of critical failure in the water distribution system. Cast-iron was the material of choice for new waterline construction in Kerrville approximately 70 years ago and is past the end of the typical design life of 50-years. As cast-iron pipes age, they can experience increased corrosion leading to water quality issues, capacity restrictions, and an increased likelihood of failure. City Staff has recently experienced several instances of critical failure of cast-iron pipes due to pipeline age and general degradation. Recent critical failures are the primary driver to proactively address this potential condition issue. In total, there are approximately 189,522 linear feet of cast-iron mains documented within the existing water distribution system. Approximately 14,565 linear feet of cast-iron main has previously been selected for replacement as part of the City's January 2021 Water Main Installation and Replacement Report. This is included as a recommended CIP project, detailed in **Section 11.1.1**. This leaves approximately 174,957 linear feet of cast-iron main within the water system.

Replacing approximately 7,000 linear feet of cast-iron main per year, the City can remove all cast-iron main from their system within a 25-year period. FNI developed 25 project areas, containing approximately 6,300 to 7,700 linear feet of cast-iron main, prioritized based on work order density and pipeline diameter. Some of the cast-iron pipelines are located under roadways that have recently been re-paved as part of the City's annual pavement replacement program. This information was also used to prioritize projects, where projects with longer lengths of pipeline contained under newly paved roadways being ranked at a lower priority. The prioritized project areas are illustrated on **Figure 10-6** and in **Table 10-7**.

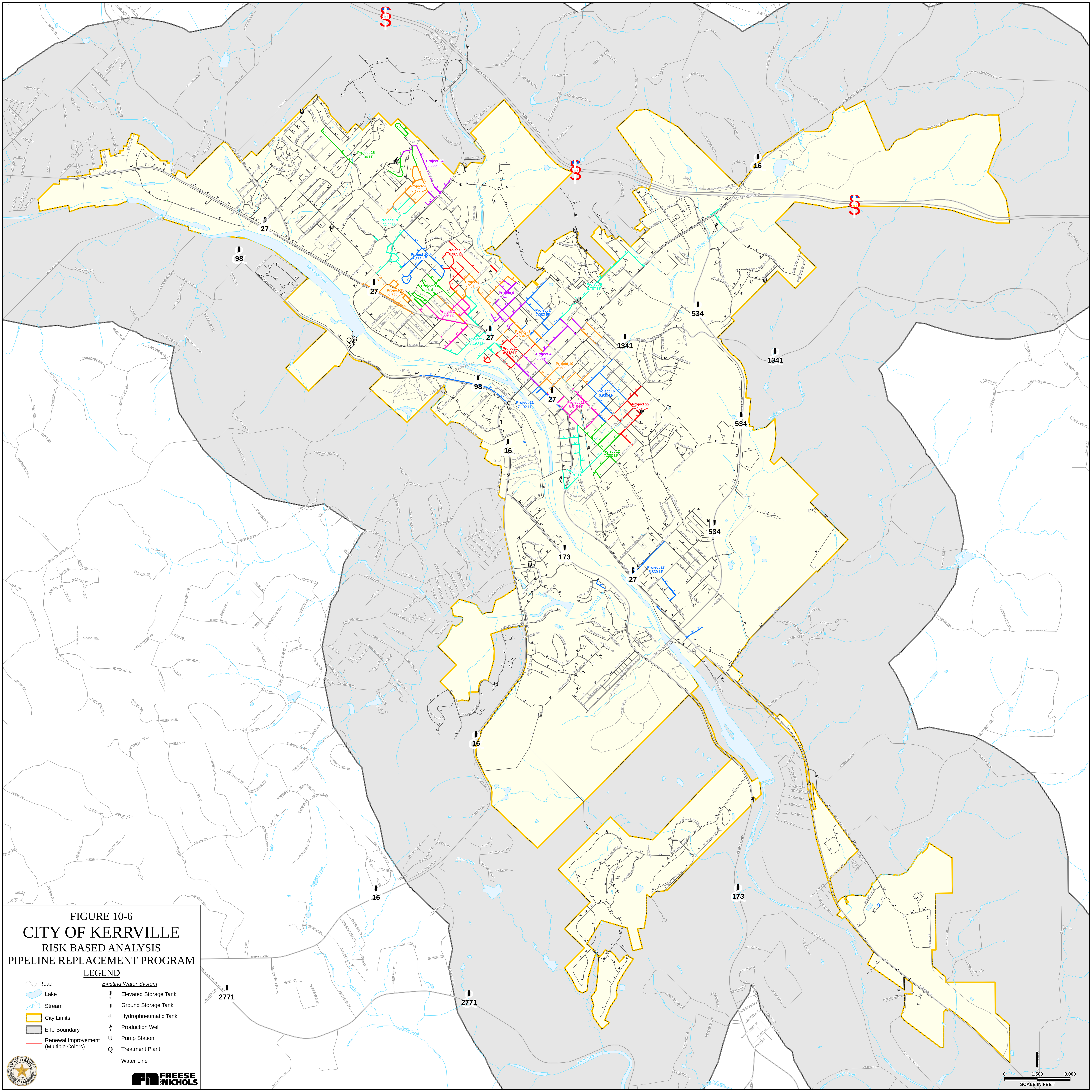
Table 10-7: Pipeline Replacement Program Summary

Project	Length (lf)
1	6,342.4
2	7,081.5
3	7,224.7
4	6,659.3
5	6,787.2
6	7,460.6
7	7,192.8
8	7,634.1
9	7,147.7
10	7,373.5
11	7,147.7
12	7,380.2
13	6,515.2
14	7,248.9
15	6,755.6
16	6,929.7
17	6,965.4
18	7,187.4
19	7,177.3
20	6,788.3
21	7,191.5
22	7,467.4
23	5,838.6
24	6,356.4
25	7,103.9
Total	**174,957.3**

FIGURE 10-6

CITY OF KERRVILLE

RISK BASED ANALYSIS
PIPELINE REPLACEMENT PROGRAM



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10.2.2 Wastewater System Pipelines

Pipeline material and diameter data was also available for the wastewater collection system. Wastewater main material is summarized on **Table 10-8** and **Figure 10-7**. Wastewater line diameter is summarized on **Table 10-9** and **Figure 10-8**. The City also provided collection system work order history from 2018 to 2021. However, location data and asset identifiers are not included in historical work order data for the wastewater system. Additionally, the majority of documented work orders were routine lift station maintenance and line cleaning, both of which have a negligible impact on linear asset condition. Due to the lack of detailed condition data on the wastewater pipelines, FNI does not recommend an annual pipeline replacement program for the wastewater collection system. FNI recommends utilizing CCTV inspection within the wastewater system to identify specific points of failure and populate more information for use in future analysis. As detailed in **Section 5.4.1**, analysis of observed flow monitoring data identifies areas of high RDII. Relatively high rates of RDII can be a good indicator of general condition issues in a given flow monitoring basin as broken manholes and cracks in gravity mains are direct contributors to inflow and infiltration. Flow monitoring cannot identify specific pipelines in poor conditions, but comparatively high rates of RDII can help target areas of the collection system for addition investigation. Observed Net RDII rates for each flow monitoring basin are shown in **Table 10-10** and on **Figure 10-9**. FNI recommends prioritizing CCTV inspection in basins with the highest RDII. FNI recommends additional annual budget for use in general wastewater renewal projects and inspection. More detail on the additional annual budget is provided in **Section 11.1**.

Table 10-8: Wastewater Main Material Summary

Material	Count	Length (ft)	Percent
Asbestos Cement	4	817.5	0.08%
Cast Iron	3	824.4	0.09%
Ductile Iron	3	211.7	0.02%
HDPE	44	14,558.7	1.51%
PVC	2,367	511,122.9	52.86%
Stainless	1	163.9	0.02%
Unknown	31	5,924.2	0.61%
VCP	1,664	433,271.7	44.81%
Total	**4,117**	**966,895.0**	**100.00%**

Table 10-9: Wastewater Main Diameter Summary

Diameter	Count	Length (ft)	Percent
Less than 4-inches	20	4,542.1	0.47%
6 to 12-inches	3,797	874,746.1	90.47%
Greater than 15-inches	300	87,606.9	9.06%
Total	**4,117**	**966,895.0**	**100.00%**

Table 10-10: Observed Net RDII rates

Flow Monitor Basin	Basin Length (lf)	Basin Area (acre)	Normalized Max Net RDII (gal/lf)	Normalized Max Net RDII (gal/ac)
FM01	65,258.7	841.5	2.084	161.614
FM02	81,993.9	819.9	6.232	623.278
FM03	62,049.0	879.0	2.031	143.344
FM04	44,362.6	838.1	3.269	173.003
FM06	75,102.2	1,009.2	2.157	160.525
FM07	20,158.5	470.9	1.488	63.704
FM08/09	273,734.6	2,223.8	4.062	500.042
FM12	11,608.0	236.9	4.480	219.458
FM13*	1,771.4	30.0	124.758	7,359.062
FM14	255,164.5	2,073.4	1.289	158.678
FM15	13,116.0	89.0	1.144	168.623

*Basin FM13 includes flow from Ingram Lift Station and does not include the contributing Ingram basin length resulting in high Net RDII.

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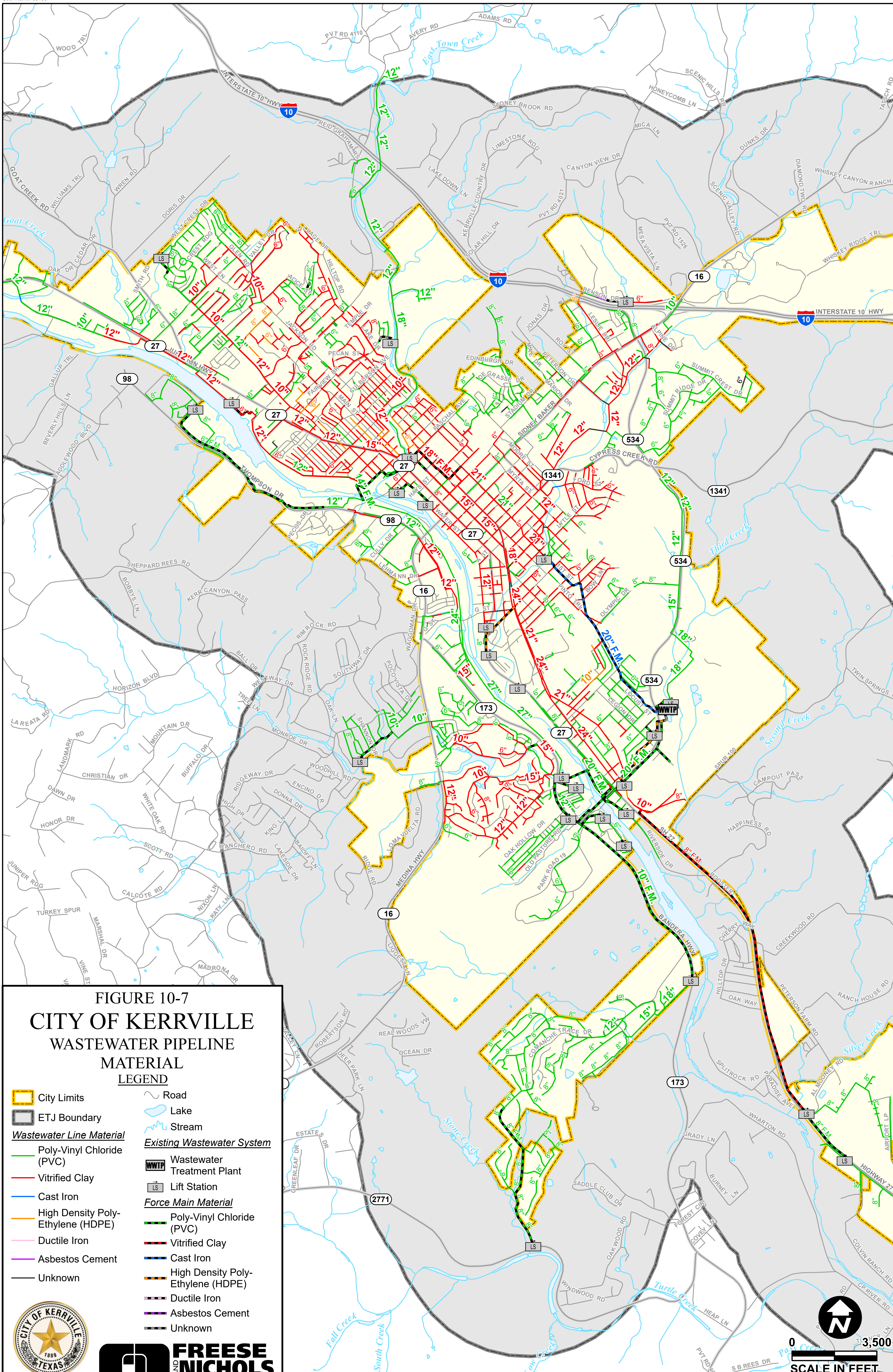

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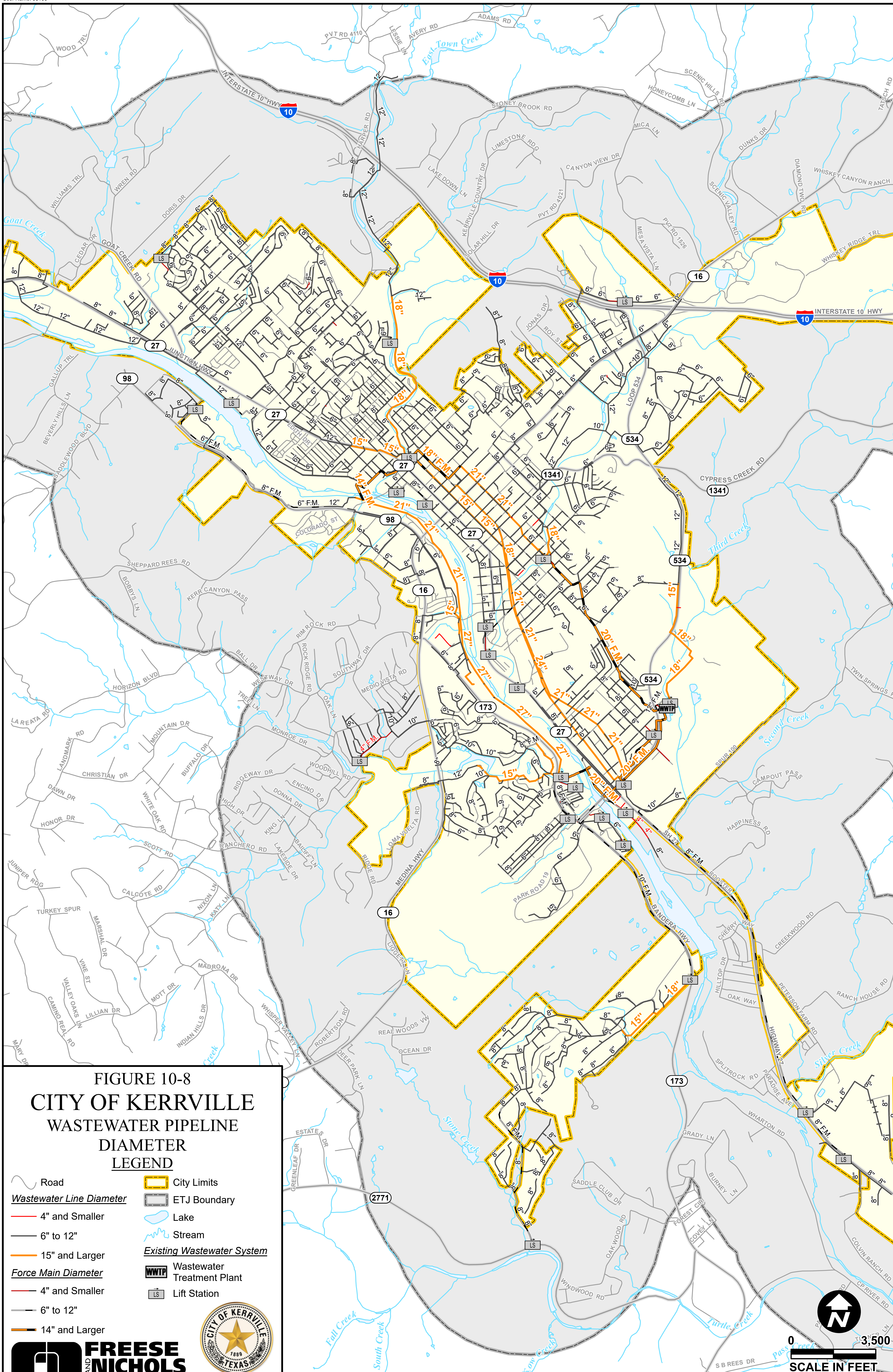

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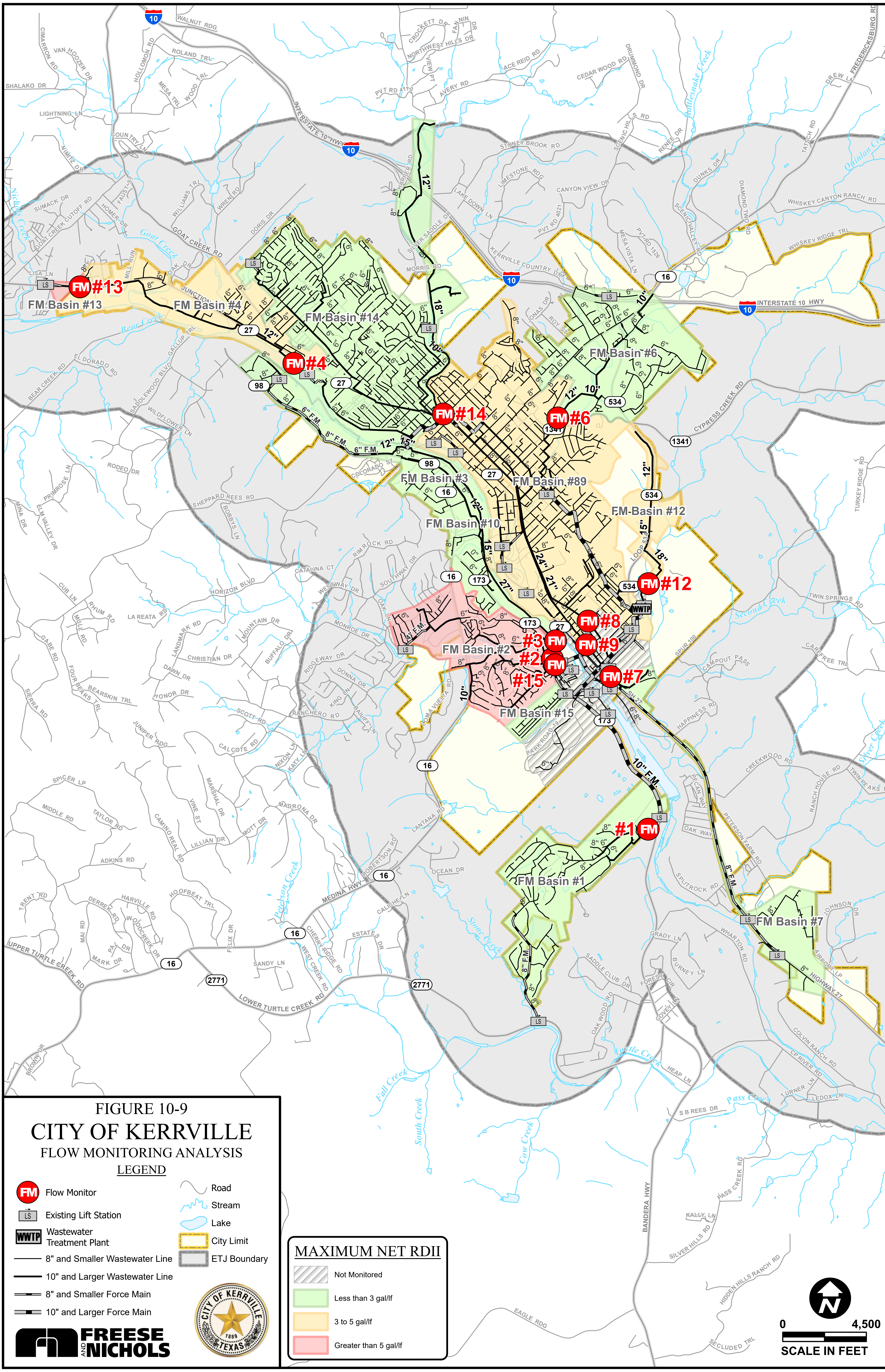


FIGURE 10-9
CITY OF KERRVILLE
FLOW MONITORING ANALYSIS

11.0 CAPITAL IMPROVEMENT PLANS

The goal of the capital improvement plans is to address existing deficiencies in the water distribution and wastewater collection systems, as well as provide capacity for future development. The recommended system improvements, estimated project costs, and project implementation triggers for both systems are discussed in this section. Two improvement plans were developed: one for short-term improvements needed to address existing deficiencies and one for infrastructure needed to serve projected growth.

11.1 RECOMMENDED EXISTING CAPACITY AND CONDITION IMPROVEMENTS

Short-term improvements were developed to address existing system deficiencies. The City requested two improvement plans, one that addressed the existing system and one that addressed future growth The two separate plans assist in delineating the purpose of the projects. The water and wastewater treatment plants were not evaluated as part of this study so previously identified improvements to these facilities from the 2014 Water Master Plan and the 2012 Wastewater Master Plans should be incorporated into the project list as needed.

11.1.1 Water Distribution System

All projects were developed in accordance with TCEQ minimum system requirements and the planning criteria as outlined in **Section 8.1**. Projects are prioritized by hydraulic requirements and criticality of the facility.

Locations for new water lines and other recommended improvements shown were investigated for feasibility but generalized for hydraulic analysis and planning purposes. Specific alignments and sites will be determined as part of the design process. Unless specified, the recommended diameters are for full pipe replacement and include decommissioning the existing line. In-depth analysis is recommended as part of the design process to determine the condition of the existing line and the cost effectiveness of full replacement or rehabilitation and parallel for each project.

Figure 11-1 shows the proposed CIP projects to remedy all identified existing system deficiencies. Brief descriptions of the identified projects are included in this section with detailed project descriptions in **Appendix E**.

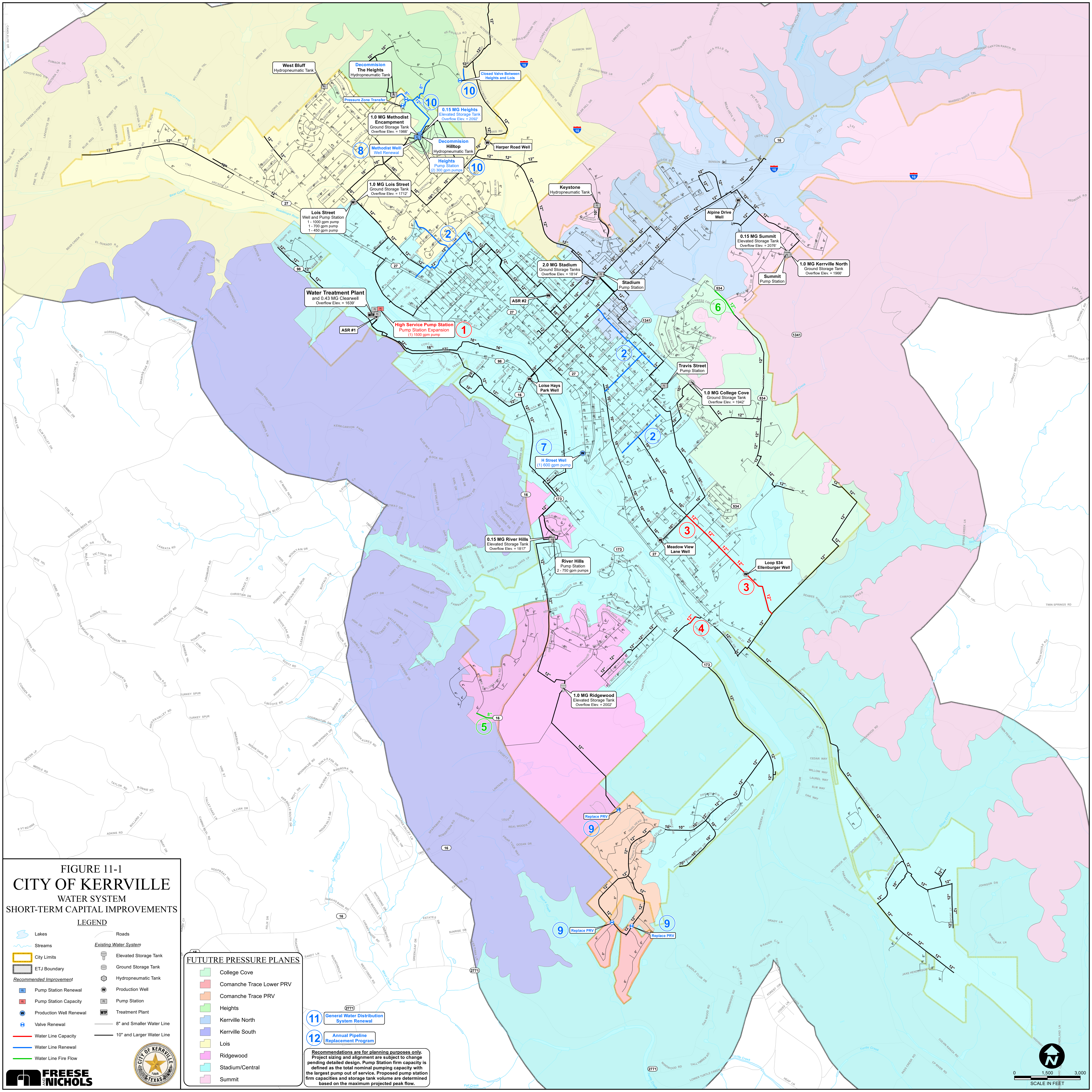


FIGURE 11-1
CITY OF KERRVILLE
WATER SYSTEM
SHORT-TERM CAPITAL IMPROVEMENTS
LEGEND
Lakes
Streams
City Limits
ETJ Boundary
Recommended Improvement
Pump Station Renewal
Pump Station Capacity
Production Well Renewal
Valve Renewal
Water Line Capacity
Water Line Renewal
Water Line Fire Flow
Roads
Existing Water System
Elevated Storage Tank
Ground Storage Tank
Hydropneumatic Tank
Production Well
Pump Station
Treatment Plant
8" and Smaller Water Line
10" and Larger Water Line
FUTUTRE PRESSURE PLANES
College Cove
Comanche Trace Lower PRV
Comanche Trace PRV
Heights
Kerrville North
Kerrville South
Lois
Ridgewood
Stadium/Central
Summit
FREESE AND NICHOLS
CITY OF KERRVILLE TEXAS 1889
11 General Water Distribution System Renewal
12 Annual Pipeline Replacement Program
Recommendations are for planning purposes only. Project sizing and alignment are subject to change pending detailed design. Pump Station firm capacity is defined as the total nominal pumping capacity with the largest pump out of service. Proposed pump station firm capacities and storage tank volume are determined based on the maximum projected peak flow.
West Bluff Hydropneumatic Tank
Decommission The Heights Hydropneumatic Tank
Closed Valve Between Heights and Lois
Pressure Zone Transfer
1.0 MG Methodist Encampment Ground Storage Tank Overflow Elev. = 1966'
0.15 MG Heights Elevated Storage Tank Overflow Elev. = 2092'
Decommission Hilltop Hydropneumatic Tank
Harper Road Well
Methodist Well Well Renewal
Heights Pump Station (2) 300 gpm pumps
1.0 MG Lois Street Ground Storage Tank Overflow Elev. = 1712'
Keystone Hydropneumatic Tank
Lois Street Well and Pump Station 1 - 1000 gpm pump 1 - 700 gpm pump 1 - 450 gpm pump
Alpine Drive Well
0.15 MG Summit Elevated Storage Tank Overflow Elev. = 2076'
1.0 MG Kerrville North Ground Storage Tank Overflow Elev. = 1966'
Summit Pump Station
2.0 MG Stadium Ground Storage Tanks Overflow Elev. = 1814'
Stadium Pump Station
Water Treatment Plant and 0.43 MG Clearwell Overflow Elev. = 1639'
ASR #2
ASR #1
High Service Pump Station Pump Station Expansion (1) 1500 gpm pump
Travis Street Pump Station
Loise Hays Park Well
1.0 MG College Cove Ground Storage Tank Overflow Elev. = 1942'
H Street Well (1) 600 gpm pump
0.15 MG River Hills Elevated Storage Tank Overflow Elev. = 1817'
River Hills Pump Station 2 - 750 gpm pumps
Meadow View Lane Well
Loop 534 Ellenburger Well
1.0 MG Ridgewood Elevated Storage Tank Overflow Elev. = 2002'
Replace PRV
0 1,500 3,000
SCALE IN FEET

Project 1: *High Service Pump Station Expansion*

This project includes a new 1,500 gpm pump and appurtenances in the available pump slot of the existing high service pump station. The new pump will expand firm pumping capacity to the Stadium Pressure Plane. This project also includes additional piping to connect to the existing Stadium and Riverhills pump headers. This project provides firm pumping capacity for the Stadium High Service Pump Station.

Project 2: *Targeted Pipeline Replacement*

This project includes replacement of existing cast-iron water mains identified by the City in their January 2021 Water Main Installation and Replacement Report. Cast-iron pipes are at a higher risk of failure due to corrosion and chemical weathering. Replacement of these pipes is recommended to maintain functionality and reduce the risk of emergency repairs. This project would allow for replacement of approximately 14,600 linear feet of pipeline.

Project 3: *12-inch Legion Dr Water Line*

This project includes a new 12-inch water line to replace the existing 6-inch and 8-inch water line along Legion Drive from Meadowview Lane to Spur 100. This project is intended to increase the capacity of the existing water line, improving service from the new Loop 534 groundwater well to be distributed throughout the pressure plane.

Project 4: *12-inch Veterans Highway Water Line Guadalupe River Crossing*

This project includes a new 12-inch water line to replace the existing 8-inch water line across the Guadalupe River at Veterans Highway. This project is intended to increase transmission capacity across the river, allowing for better connectivity within the Stadium Pressure Plane.

Project 5: *Ridgewood Fire Flow Improvements*

This project includes a new 8-inch water line near Medina Highway and Ridge Road to create looping within the Ridgewood distribution system. This project increases available fire flow within the Ridgewood Pressure Plane.

Project 6: *College Cove Fire Flow Improvements*

This project includes a new 12-inch water line along Loop 534 near Paragon Place to create looping within the College Cove Pressure Plane. This project increases available fire flow within the College Cove Pressure Plane.

Project 7: *H Street Well Renewal*

This project restores service to the existing H Street Well. This project includes a new pump and motor and equipping the facility with SCADA. This project provides additional production pumping capacity to the Stadium Pressure Plane.

Project 8: *Methodist Encampment Well Renewal*

This project restores service to the existing Methodist Encampment Well. Currently, the well is only approved for emergency use because of water quality issues. This project is for the re-routing of the well discharge piping from the outlet piping of the Methodist ground storage tank to the top of the GST. This re-routing is intended to blend the groundwater with the stored treated surface water and help maintain water quality in the system.

Project 9: *Comanche Trace PRV Replacement*

This project involves the replacement of the existing PRVs in the Comanche Trace PRV zones. The project involves replacement of valve and valve box. This project enhances service reliability and ease of maintenance and operation of the Comanche Trace PRVs.

Project 10: *Hydropneumatic Zone Replacement*

This project creates a new pressure plane to replace the existing Hilltop and The Heights Hydropneumatic Zones. This project includes a new 300 gpm pump station and 0.15 MG EST with an overflow elevation of 2,092 feet at the existing Methodist Encampment site. This project also includes a new 12-inch transmission line and new 8-inch lines to connect the existing Hilltop and The Heights Zones. Once the new pump station, EST, and water lines are in service, the existing hydropneumatic facilities may be decommissioned. This project enhances service reliability and capacity to The Heights and Hilltop Zones. This project also addresses maintenance and operation issues with the existing hydropneumatic facilities. This project is sized for existing demand but may also provide additional capacity for potential future development in the area that may participate in project implementation.

Project 11: *General Water Distribution System Renewal*

This project consists of general water system renewal projects. General water system renewal improvements are needed to maintain functionality and reduce the risk of emergency repairs. Recommended water renewal projects may include, but are not limited to, water main inspections, a water loss study, tank mixing systems, replacing gaseous chlorine with liquid bleach at distribution system facilities, production well buildings/enclosures, lighted canopies over electrical boxes, recoating exposed piping and valves, or other water distribution system improvements best implemented as capital projects. The purpose of this project is to allocate additional annual budget for potential renewal projects to allow the City's existing water distribution system to continue to operate at a high level of service.

Project 12: *Annual Pipeline Replacement Program*

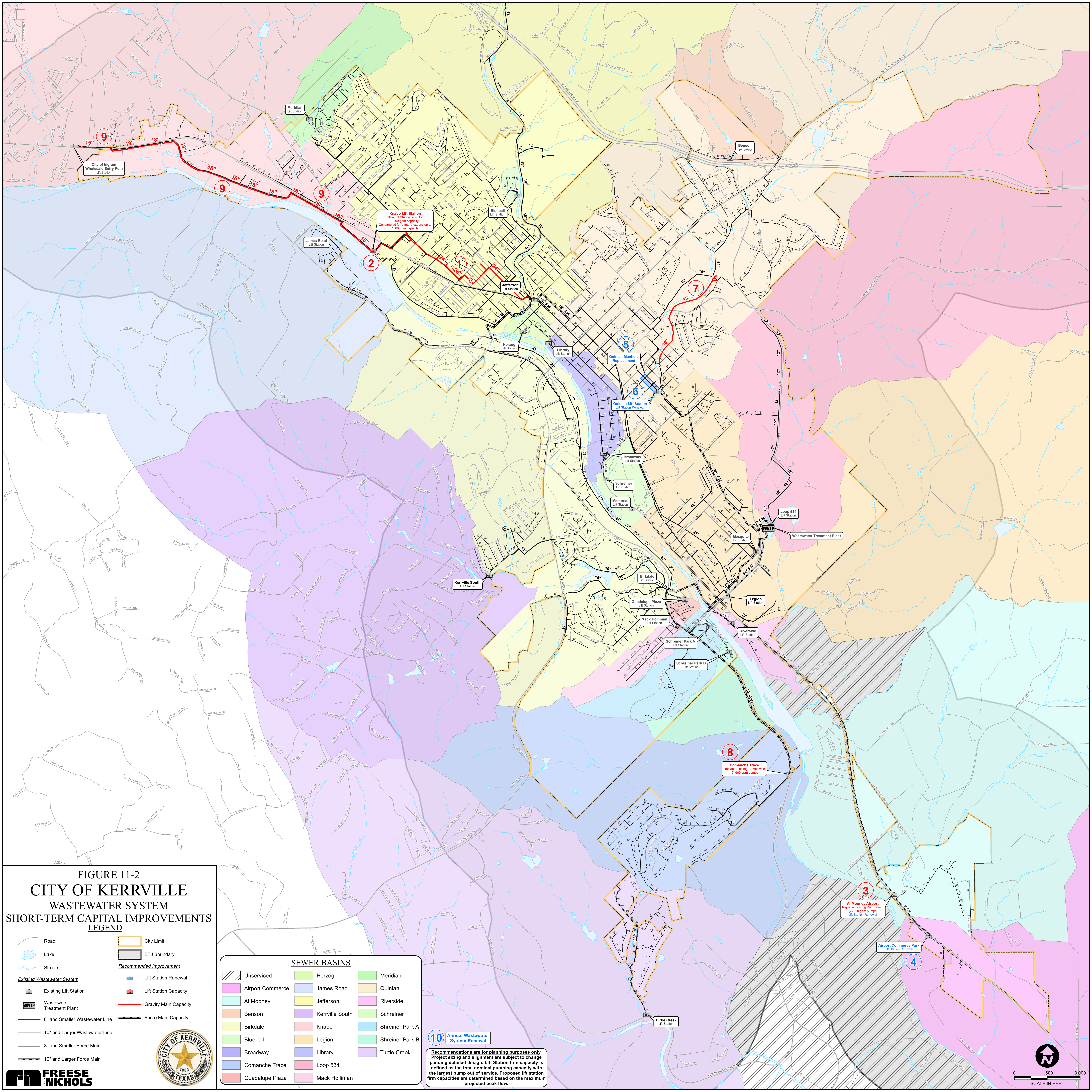
This project includes replacement of existing cast-iron water mains. Cast-iron pipes are at a higher risk of failure due to corrosion and chemical weathering. Replacement of these pipes is recommended to maintain functionality and reduce the risk of emergency repairs. According to the City's GIS database, approximately 189,522 linear feet of cast-iron water mains are documented within the existing water distribution system. Where existing cast-iron mains are less than 8-inch in diameter, pipes will be replaced with minimum 8-inch diameter mains. 14,565 linear feet will be replaced as part of the Targeted Pipeline Replacement Project. Accounting for the Targeted Pipeline Replacement Project and the pavement replacement program leaves approximately 174,957 linear feet of cast-iron main that is recommended to be replaced. FNI developed 25 project areas, containing approximately 7,000 linear feet of cast-iron main, prioritized based on work order density and pipeline diameter.

11.1.2 Wastewater Collection System

All projects were developed in accordance with TCEQ minimum system requirements and the planning criteria as outlined in **Section 9.1**. The sizing of these projects was determined by the required capacity as projected by the hydraulic model. Projects are prioritized and phased by hydraulic requirements and criticality of the asset.

Locations for new collector mains and other recommended improvements shown were investigated for feasibility but generalized for hydraulic analysis and planning purposes. Specific alignments and sites will be determined as part of the design process. Unless specified, the recommended diameters are for full pipe replacement and include decommissioning the existing line. In-depth analysis is recommended as part of the design process to determine the condition of the existing line and the cost effectiveness of full replacement or rehabilitation and parallel for each project.

Figure 11-2 shows the proposed CIP projects to address all identified existing system deficiencies as determined in the system analysis. Brief descriptions of the identified projects are included in this section with detail project descriptions in **Appendix F**.



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Project 1: *Knapp Force Main and Interceptor*

This project involves construction of a new force main and interceptor between the existing Knapp and Jefferson Lift Stations. The design includes approximately 2,000 feet of 12-inch force main and approximately 8,000 feet of 18-, and 24-inch gravity sewer. The proposed interceptor is triggered by an existing capacity restriction downstream of the Knapp Lift Station. The proposed force main is intended to meet projected future peak capacity once improvements to the lift station have been made. The force main will not be put into service until the lift station pump capacity is increased, to meet minimum velocity conditions in the new force main.

Project 2: *Knapp Lift Station*

This project involves construction of a new 1.75 MGD Knapp Lift Station on or adjacent to the existing site. The new lift station includes three 600 gpm pumps with an additional slot for a fourth pump to be installed during a future expansion. This project is triggered by an existing capacity restriction at the lift station. This project is intended to increase capacity for existing peak flows but also provides additional capacity for potential future development in the area that may participate in project implementation. This project increases wastewater service capacity for both City of Kerrville and City of Ingram customers.

Project 3: *Al Mooney Lift Station Renewal*

This project replaces the existing pumps at the Al Mooney Lift Station with two new 300 gpm pumps. This project also includes upgrades to enhance the condition of the facility, such as wet well, pump control, and electrical improvements. This project is triggered by an existing capacity restriction and by existing condition issues noted during the lift station site visit. This project is intended to increase capacity for existing peak flows but also provides additional capacity for potential future development in the area that may participate in project implementation.

Project 4: *Airport Commerce Lift Station Renewal*

This project includes condition upgrades to the Airport Commerce Lift Station. Upgrades include improvements to the wet well, electrical system, and control panels. This project is triggered by existing condition issues noted during the lift station site visit.

Project 5: *Quinlan Manhole Replacement*

This project includes in-place replacement of approximately 16 fiberglass manholes within the 1st Street roadway, upstream of the Quinlan Lift Station. According to City Staff, these manholes are not adequate to sustain repeated dynamic loads from vehicular traffic on the roadway. New manholes should be installed with appropriate structural conditions to support heavy vehicle loading. This project also includes repaving approximately 1,000 lf of 1st Street once the new manholes are in place. According to City Staff, poor structural conditions of the existing manholes are causing the roadway to deteriorate and fail.

Project 6: *Quinlan Lift Station Renewal*

This project includes addressing debris issues at the Quinlan Lift Station. Debris control may include wet well recirculation, grinder pumps, or screening to be determined by the design engineer. This project also includes a new wet well hatch and safety grate, new pump controls, and enhanced wet well ventilation. Results from the Lift Station Site Visits noted severe debris and ragging issues of the pumps at Quinlan Lift Station, verified by City Staff. Debris causes maintenance and operational issues but lift station capacity is not identified as deficient.

Project 7: *Quinlan Interceptor Reroute*

This project includes a new 18-inch Quinlan Interceptor along a new alignment on the east side of Quinlan Creek. This project will connect to the existing 12-inch line near Quinlan Creek Drive and the existing 18-inch line at 3rd Street. This project will connect to existing the 6-inch and 8-inch lines along the alignment where feasible. Once this project is in service, the existing 10-inch line under Quinlan Creek may be cut and capped or abandoned in place. This project is triggered by a projected capacity restriction in the existing Quinlan Interceptor. This project is intended to increase capacity for existing peak flows but also provides additional capacity for potential future development in the area that may participate in project implementation.

Project 8: *Comanche Trace Lift Station Expansion*

This project expands the capacity of the existing Comanche Trace Lift Station. The existing pumps will be replaced with two 900 gpm pumps. This project also replaces the existing floats and controls. Pumps are sized according to existing peak flows through the pump station and the development agreement required capacity of 1,300 LUEs. Approximately 544 connections currently contribute to existing peak flows. Future growth will drive additional lift station expansion. This project is triggered by an existing capacity restriction and is sized to convey existing peak flows. As the Comanche Trace Lift Station solely serves the Comanche Trace development, it is assumed that the developer will be primarily responsible for the implementation of this project.

Project 9: *Ingram Interceptor Expansion*

This project is intended to increase the capacity of the existing interceptor downstream of the Ingram Lift Station. This project replaces the existing 12-inch gravity main with a new 15-inch and 18-inch line from the Ingram Lift Station to the Knapp Lift Station. This project is triggered by a projected existing capacity restriction. This project is intended to increase capacity for existing peak flows but also provides additional capacity for potential future development in the area that may participate in project implementation. This project increases wastewater service capacity for both City of Kerrville and City of Ingram customers.

Project 10: *Annual Wastewater System Renewal*

This project allocates additional annual budget for the renewal of the City's wastewater system. General wastewater system renewal improvements are needed to maintain functionality and reduce the risk of emergency repairs. Budget may be used towards general lift station renewal, gravity main CCTV inspection, main replacement, or any additional operation and maintenance concern. General lift station renewal may include, but is not limited to, cleaning and coating wet wells or valve vaults, general electrical improvements, addressing corrosion or debris issues, accommodations for back-up power supply, or other lift station improvements. CCTV inspection may be used to assess the condition of a gravity main of interest, collect invert elevation data, and pipe material data. The purpose of this project is to allocate additional annual budget to allow the City's existing facilities to continue to operate at a high level of service.

11.1.3 Planning Level Cost Estimates

Planning level cost estimates were developed for the recommended improvements. The cost estimating process was developed according to the American Association of Cost Engineers (AACE) Estimate Class 5 specifications. This corresponds to a maturity level of project design deliverables of approximately five percent. Estimates are developed to be conservative for budgeting purposes, but actual project costs may vary. The costs are provided as estimates based on previous similar engineering experience in 2022 dollars and include an allowance for engineering, surveying, and contingencies. The project cost estimates do not include an allowance for land or right of way acquisition, adjacent distribution and collection lines impacted by the project, individual service connections, permitting, construction allowances, or other unique project specific costs beyond “typical" project requirements. Unit costs were developed based on engineering experience and analysis of recent, local bid tabs. These unit costs account for various appurtenances included with each item and are higher than the simple cost of the material. Additionally, unit costs incorporate existing market conditions, and future changes to material supply and construction demand will affect project costs. These costs are for planning and budgeting purposes only and are not to be considered as a detailed opinion of probable construction cost.

Table 11-1 and **Table 11-2** summarize the estimated project costs by phase for the water and wastewater system improvements, respectively. Projects are listed in order of priority from high to low. Each project was also assigned a unique project number that is meant to stay consistent into future studies, whether the relative priority of each project changes or not. Detailed and itemized descriptions of all water and wastewater CIP projects and associated costs are shown in **Appendix E** and **Appendix F**, respectively.

Table 11-1: Water CIP Project Cost Summary

Project Number	Project Name			Project Cost
Capital Improvement Plan				
1	High Service Pump Station Expansion			$661,800
2	Targeted Pipeline Replacement			$5,254,900
3	12-inch Legion Dr Water Line			$3,454,400
4	12-inch Veterans Highway Water Line Guadalupe River Crossing			$511,900
5	Ridgewood Fire Flow Improvements			$923,900
6	College Cove Fire Flow Improvements			$688,000
7	H Street Well Renewal			$385,100
8	Methodist Encampment Well Renewal			$491,400
9	Comanche Trace PRV Replacement			$982,800
10	Hydropneumatic Zone Replacement			$4,266,000
	Capital Improvement Plan Total			**$17,620,200**
Annual Renewal Plan				
		Annual Cost	**Years**	**Total Cost**
11	General Water Distribution System Renewal	$100,000	10	$1,000,000
12	Annual Pipeline Replacement Program	$2,009,948	25	$50,248,700
	Annual Renewal Plan Total			**$51,248,700**

Table 11-2: Wastewater CIP Project Cost Summary

Project Number	Project Name			Project Cost
Capital Improvement Plan				
1	Knapp FM and Interceptor			$6,374,692
2	New Knapp Lift Station			$3,049,500
3	Al Mooney Lift Station Renewal			$765,600
4	Airport Commerce Lift Station Renewal			$464,600
5	Quinlan Manhole Replacement			$894,400
6	Quinlan Lift Station Renewal			$206,400
7	Quinlan Interceptor Reroute			$4,790,400
8	Comanche Trace Lift Station Expansion			$499,000
9	Ingram Interceptor Expansion			$19,068,700
	Capital Improvement Plan Total			**$36,113,292**
Annual Renewal Plan				
		Annual Cost	**Years**	**Total Cost**
10	Annual Wastewater System Renewal	$100,000	10	$1,000,000
	Annual Renewal Plan Total			**$1,000,000**

11.2 RECOMMENDED GROWTH DRIVEN IMPROVEMENTS

Growth driven improvements were recommended to provide service to future developments. These improvements are not recommended until growth occurs and are intended to be used as a roadmap to generally plan for improvements needed to extend service into currently undeveloped areas. It is likely these improvements would be fully or partially funded by land developers through cost participation agreements.

11.2.1 Water Distribution System

All projects were developed in accordance with TCEQ minimum system requirements and the planning criteria as outlined in **Section 8.1.** Projects are prioritized by impact to the water distribution system and constructability. However, project implementation may vary due to changes in development.

Locations for new water lines and other recommended improvements shown were investigated for feasibility but generalized for hydraulic analysis and planning purposes. Specific alignments and sites will be determined as part of the design process. Unless specified, the recommended diameters are for full pipe replacement and include decommissioning the existing line. In-depth analysis is recommended as part of the design process to determine the condition of the existing line and the cost effectiveness of full replacement or rehabilitation and parallel for each project.

Figure 11-3 shows the proposed improvement projects to provide service to all projected development areas. Brief descriptions of the identified projects in each individual development area are included in this section with detail project descriptions in **Appendix G**.

<u>Development Scenario A</u>:

Projects recommended to serve development scenario A include three water main projects. These water main projects extend the distribution system within the Stadium and Ridgewood Pressure Planes. Establishing a new pressure plane is recommended to serve development further west, which cannot be served by an existing pressure plane due to elevation constraints. A new high service pump and elevated storage tank are recommended as part of the new pressure plane. New water mains are also recommended to serve developments in this new pressure plane.

<u>Development Scenario B</u>: Projects recommended to serve development scenario B include an extension of the water distribution system within the Methodist Pressure Plane and an extension of the water

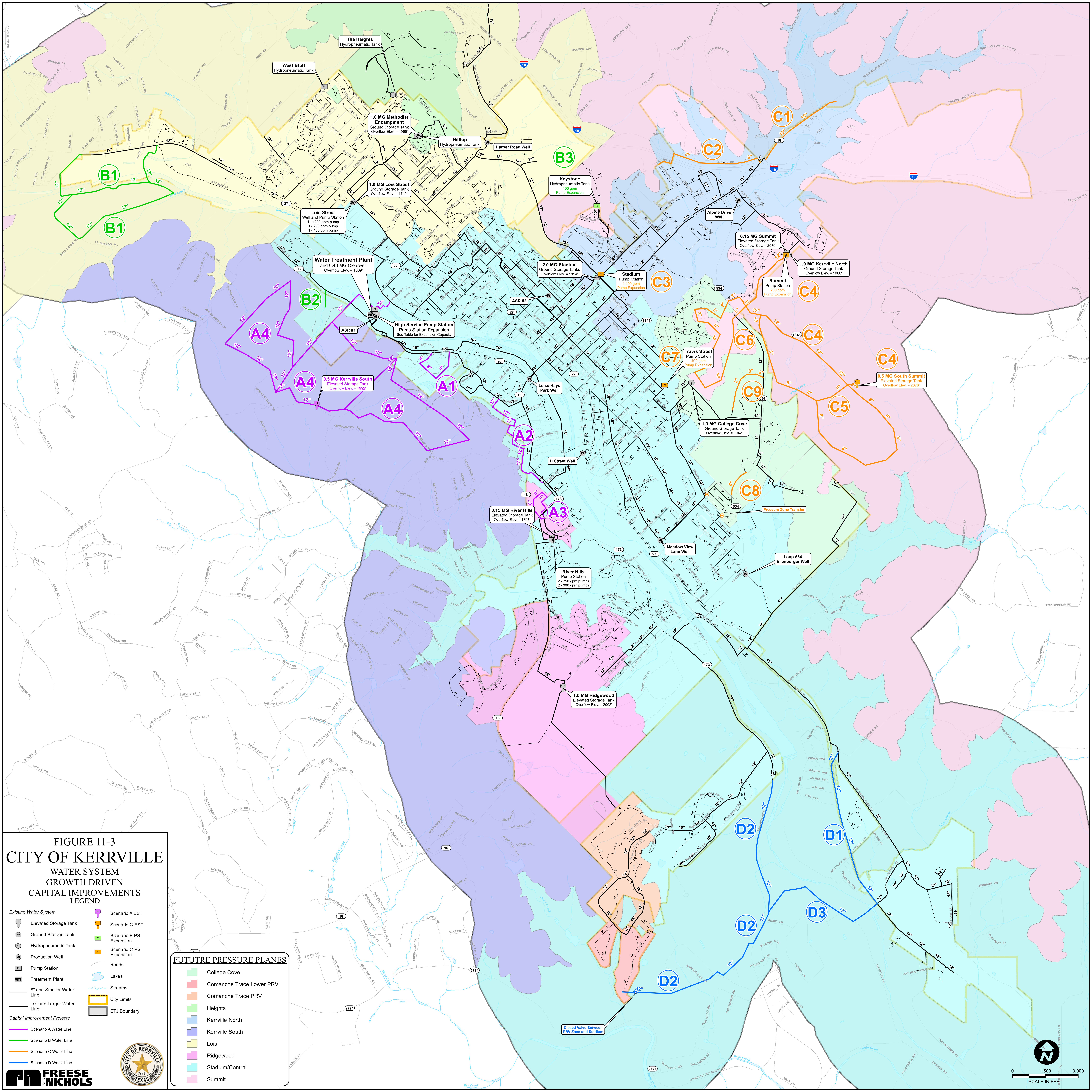
distribution system within the Stadium Pressure Plane. A pump expansion at Keystone Hydropneumatic Tank is also recommended to serve projected development in that area.

Development Scenario C:

Projects recommended to serve development scenario C include water main projects to extend the water distribution system within the Kerrville North, College Cove, and Summit Pressure Planes. To serve the projected developments in these pressure planes, pump station expansions are recommended to meet minimum TCEQ distribution pumping capacity requirements. Stadium Pump Station, Travis Street Pump Station, and Summit Pump Station are recommended to expand their capacity. To extend service from the Summit Pressure Plane, additional elevated storage is recommended to meet TCEQ storage capacity requirements. A new elevated storage tank is recommended within the Summit Pressure Plane to serve projected development. An existing development is also recommended to be transferred from the Stadium Pressure Plane to the College Cove Pressure Zone. The valving and water main construction is anticipated to be completed by the developer.

Development Scenario D:

Projects recommended to serve development scenario D include three water main projects to extend the water distribution system within the lower Stadium Pressure Plane.



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11.2.2 Wastewater Distribution System

All projects were developed in accordance with TCEQ minimum system requirements and the planning criteria as outlined in **Section 9.1**. The sizing of these projects was determined by the required capacity as projected by the hydraulic model. Projects are prioritized and phased by hydraulic requirements and criticality of the asset. However, project implementation may vary due to changes in development.

Locations for new collector mains and other recommended improvements shown were investigated for feasibility but generalized for hydraulic analysis and planning purposes. Specific alignments and sites will be determined as part of the design process. Unless specified, the recommended diameters are for full pipe replacement and include decommissioning the existing line. In-depth analysis is recommended as part of the design process to determine the condition of the existing line and the cost effectiveness of full replacement or rehabilitation and parallel for each project.

Figure 11-4 shows the proposed improvement projects to provide service to all projected development areas. Brief descriptions of the identified projects are included in this section with detail project descriptions in **Appendix H**.

Development Scenario A:

Projects recommended to serve development scenario A include extension of the gravity main system within the Birkdale Lift Station basin. The gravity mains are recommended to convey flow from projected developments to the WWTP.

Development Scenario B:

Projects recommended to serve development scenario B include extension of the gravity main system within the James Road Lift Station basin. With projected increased wastewater flow within the James Road Lift Station basin, a lift station expansion is recommended. Additionally, due to ground elevations, a new lift station and force main are recommended to serve development in the northwestern portion of the basin.

Development Scenario C:

Projects recommended to serve development scenario C include extension of the gravity main system within the Quinlan and Loop 534 Lift Station basins. The gravity mains are recommended to convey flow from projected developments to the WWTP.

Development Scenario D:

Projects recommended to serve development scenario D include extension of the gravity main system within the Comanche Trace, Turtle Creek, Birkdale, and Al Mooney Lift Station basins. Comanche Trace Lift Station and Turtle Creek Lift Station capacity expansions are recommended to provide service to the projected developments. Peak flows from the Turtle Creek Lift Station are projected to overwhelm the existing Comanche Trace gravity main system, requiring the capacity to be increased. Two options are proposed to remedy this: a project involving upsizing the existing gravity main through the Comanche Trace Neighborhood, from the outfall of the Turtle Creek force main to the existing 12-inch line near Pinnacle Club Drive or a project rerouting the existing 8-inch Turtle Creek Force Main with a new 8-inch force main along a new alignment around the existing Comanche Trace neighborhood. To serve developments near the airport, a new regional lift station is recommended, to replace the Al Mooney and Airport Commerce Lift Stations. These lift stations are recommended for decommissioning. Two additional lift stations are recommended to serve projected developments North of the Comanche Trace neighborhood.

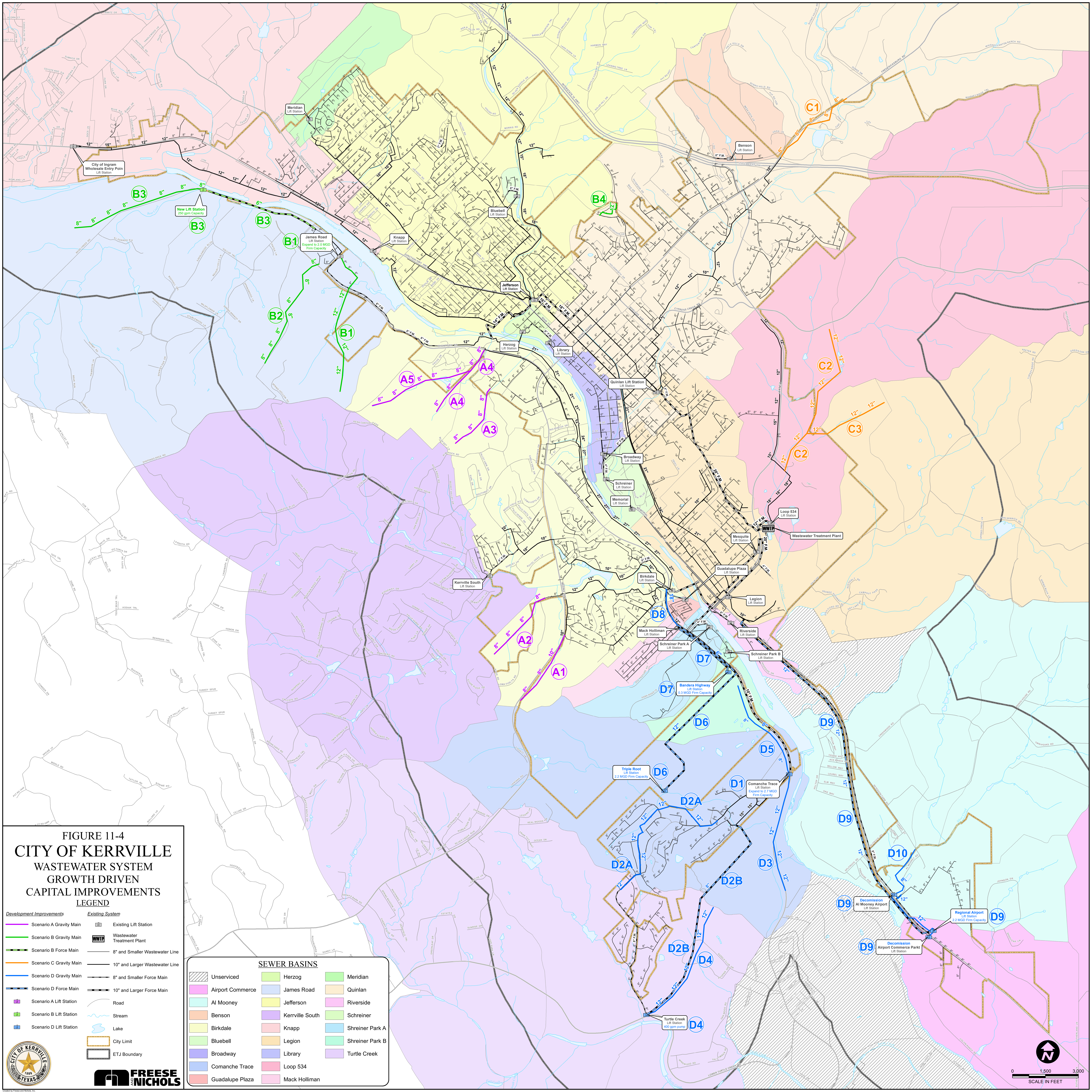

FIGURE 11-4
CITY OF KERRVILLE
WASTEWATER SYSTEM
GROWTH DRIVEN
CAPITAL IMPROVEMENTS
LEGEND
Development Improvements
Scenario A Gravity Main
Scenario B Gravity Main
Scenario B Force Main
Scenario C Gravity Main
Scenario D Gravity Main
Scenario D Force Main
Scenario A Lift Station
Scenario B Lift Station
Scenario D Lift Station
Existing System
Existing Lift Station
WWTP
Wastewater Treatment Plant
8" and Smaller Wastewater Line
10" and Larger Wastewater Line
8" and Smaller Force Main
10" and Larger Force Main
Road
Stream
Lake
City Limit
ETJ Boundary
SEWER BASINS
Unserviced
Airport Commerce
Al Mooney
Benson
Birkdale
Bluebell
Broadway
Comanche Trace
Guadalupe Plaza
Herzog
James Road
Jefferson
Kerrville South
Knapp
Legion
Library
Loop 534
Mack Holliman
Meridian
Quinlan
Riverside
Schreiner
Shreiner Park A
Shreiner Park B
Turtle Creek
CITY OF KERRVILLE 1889 TEXAS
FREESE AND NICHOLS
Meridian Lift Station
City of Ingram Wholesale Entry Poin Lift Station
New Lift Station 250 gpm Capacity
James Road Lift Station Expand to 2.0 MGD Firm Capacity
Knapp Lift Station
Bluebell Lift Station
Benson Lift Station
Jefferson Lift Station
Herzog Lift Station
Library Lift Station
Quinlan Lift Station Lift Station
Broadway Lift Station
Schreiner Lift Station
Memorial Lift Station
Loop 534 Lift Station
Wastewater Treatment Plant
Mesquite Lift Station
Guadalupe Plaza Lift Station
Birkdale Lift Station
Kerrville South Lift Station
Legion Lift Station
Riverside Lift Station
Mack Holliman Lift Station
Schreiner Park A Lift Station
Schreiner Park B Lift Station
Bandera Highway Lift Station 0.3 MGD Firm Capacity
Triple Root Lift Station 2.2 MGD Firm Capacity
Comanche Trace Lift Station Expand to 2.7 MGD Firm Capacity
Decomission Al Mooney Airport Lift Station
Regional Airport Lift Station 2.2 MGD Firm Capacity
Decomission Airport Commerce Parkt Lift Station
Turtle Creek Lift Station 400 gpm pump
A1 A2 A3 A4 A5 B1 B2 B3 B4 C1 C2 C3 D1 D2A D2B D3 D4 D5 D6 D7 D8 D9 D10
0 1,500 3,000
SCALE IN FEET
Created by Freese and Nichols, Inc.
Job No.: KER11495
Location: H:\W_WW_PLANNING\01_DELIVERABLES\00_FINAL_REPORT\(Figure_11-4)-WW_Dev_Improvements.mxd
Updated: Tuesday, October 25, 2022

11.2.3 Planning Level Cost Estimates

Planning level cost estimates were developed for the recommended improvements. The cost estimating process was developed according to the American Association of Cost Engineers (AACE) Estimate Class 5 specifications. This corresponds to a maturity level of project design deliverables of approximately five percent. Estimates are developed to be conservative for budgeting purposes, but actual project costs may vary. The costs are provided as estimates based on previous similar engineering experience in 2022 dollars and include an allowance for engineering, surveying, and contingencies. The project cost estimates do not include an allowance for land or right of way acquisition, adjacent distribution and collection lines impacted by the project, individual service connections, permitting, construction allowances, or other unique project specific costs beyond “typical" project requirements. Unit costs were developed based on engineering experience and analysis of recent, local bid tabs. These unit costs account for various appurtenances included with each item and are higher than the simple cost of the material. Additionally, unit costs incorporate existing market conditions, and future changes to material supply and construction demand will affect project costs. These costs are for planning and budgeting purposes only and are not to be considered as a detailed opinion of probable construction cost.

Table 11-3 and **Table 11-4** summarize the estimated project costs by development scenario for the water and wastewater system improvements, respectively. Projects are listed in order of priority from high to low. Each project was also assigned a unique project number that is meant to stay consistent into future studies, whether the relative priority of each project changes or not. Detailed and itemized descriptions of all water and wastewater growth driven CIP projects and associated costs are shown in **Appendix G** and **Appendix H**, respectively.

Table 11-3: Water Development Improvements Project Cost Summary

Project Number	Project Name	Project Cost
Development Scenario A		
A1	8-inch Trinity Circle Water Main	$1,676,600
A2	12-inch Sidney Baker St S Water Main	$3,202,300
A3	8-inch Overlook Dr E Water Main	$1,193,100
A4	Kerrville South Pressure Plane	$21,329,900
	Development Scenario A Total	**$27,401,900**
Development Scenario B		
B1	12-inch Bear Creek Rd Water Main	$10,560,600
B2	8-inch Jade Loop Water Main	$596,600
B3	Keystone Hydropneumatic Tank	$557,000
	Development Scenario B Total	**$11,714,200**
Development Scenario C		
C1	10-inch Fredericksburg Rd N Water Main	$1,660,200
C2	8-inch Benson Dr N Water Main	$1,821,000
C3	Stadium Pump Station Expansion	$982,800
C4	South Summit Transmission and EST	$7,601,800
C5	South Summit South-eastern Water Main	$6,624,400
C6	South Summit South-western Water Main	$5,808,100
C7	Travis Street Pump Station	$557,000
C8	College Cove Pressure Transfer	$1,146,600
C9	8-inch Hal Peterson Middle School Water Main	$910,500
	Development Scenario C Total	**$27,112,400**
Development Scenario D		
D1	12-inch Highway 27 Water Main	$3,084,700
D2	12-inch Bandera Highway Water Main	$6,353,700
D3	12-inch Wharton Rd E Water Main	$2,990,500
	Development Scenario D Total	**$12,428,900**
	Growth Driven Projects Total	**$78,657,400**

Table 11-4: Wastewater Development Improvements Project Cost Summary

Project Number	Project Name	Project Cost
Development Scenario A		
A1	8-inch Medina Highway Gravity Main	$1,310,400
A2	8-inch Camp Meeting Rd Gravity Main	$1,310,400
A3	12-inch Birkdale Ln Gravity Main	$707,700
A4	8-inch Cully Drive Gravity Main	$1,146,600
A5	8-inch Texas Drive Gravity Main	$2,919,800
	Development Scenario A Total	**$7,394,900**
Development Scenario B		
B1	12-inch James Road Gravity Main	$5,136,300
B2	8-inch Landing Lane Gravity Main	$1,915,100
B3	Bear Creek Rd Gravity Main and LS	$4,621,400
B4	12-inch Yorktown Blvd Gravity Main	$1,620,100
	Development Scenario B Total	**$13,292,900**
Development Scenario C		
C1	8-inch Fredericksburg Rd N	$1,539,800
C2	12-inch Veterans Highway Gravity Main	$3,089,400
C3	12-inch Third Creek Gravity Main	$1,620,100
	Development Scenario C Total	**$6,249,300**
Development Scenario D		
D1	Comanche Trace Lift Station	$163,800
D2A	Comanche Trace Gravity Main	$5,454,600
D3	12-inch Bandera Highway Gravity Main	$2,298,200
D4	12-inch Turtle Creek Gravity Main & LS	$2,602,700
D5	8-inch Guadalupe River Gravity Main	$1,664,000
D6	Triple Root Lift Station	$6,994,900
D7	Bandera Highway Lift Station	$1,445,900
D8	18-inch Bandera Highway Gravity Main	$1,772,800
D9	Airport Regional Lift Station	$11,564,200
D10	8-inch Silver Creek Gravity Main	$1,251,500
	Development Scenario D Total	**$35,212,600**
	Growth Driven Projects Total	**$62,149,700**