WATER SYSTEM ANALYSIS

WATER MASTER PLAN

FOR

TRAVIS COUNTY W.C.&I.D. POINT VENTURE

2-5-2024

STATE OF TEXAS
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EXECUTIVE SUMMARY

A Water Master Plan was completed for Travis County Water Control and Improvement District (WCID) Point Venture (WCID Point Venture) to analyze the existing system, project the needs of future growth on the system, identify any deficiencies in the system, and recommend improvements needed to remedy these deficiencies. The master plan included the development of a hydraulic model of WCID Point Venture's water distribution system. This hydraulic model was used to assist in evaluating the existing system, as well as additional growth on the system, and any proposed improvements to the system.

WCID Point Venture has an existing contract with the Lower Colorado River Authority (LCRA) to withdrawal raw water from Lake Travis. WCID Point Venture pumps raw water from Lake Travis to its existing 1.0 Million Gallons per Day (MGD) Water Treatment Plant (WTP), and the treated water is conveyed into the water distribution system.

The water distribution system is made up of two service areas or pressure planes; these include the Upper and Lower Pressure Planes. Pressure planes are isolated areas of the distribution system that maintain a specific hydraulic grade line, which dictates the pressure to an area based on its relative elevation. These pressure planes have a single hydraulic grade line that provides a reasonable water pressure to all areas within the pressure plane.

A hydraulic model, or water model, was created to represent the existing water distribution system. This model was created by inputting physical data furnished by WCID Point Venture. Existing waterlines, pump stations, control valves and storage tanks were input into the model. The existing water demands were then placed on the system to give a representation of how the system operates under different conditions. The model was then calibrated in order to ensure that the model accurately represents what is actually happening in the field. This was accomplished by running the fire flow scenario and comparing results to the fire flow testing data that was performed by Capital Hydrant, LLC.

Historical data, from January 2018 through December 2022, was provided by WCID Point Venture and included water usage, pumping records, and water connection records. Using this data, the average day, maximum day, and peak hour flow rates were established for the system. Minimum fire flow requirements are established by the Insurance Services Office (ISO). These flow requirements were then used to evaluate both the existing and future systems for their ability to meet the demands of supplying water during a fire.

The Texas Commission on Environmental Quality (TCEQ) provides minimum requirements for public water systems per Sections §290.44, Water Distribution, and §290.45, Minimum Water System Capacity Design Requirements for

Public Water Systems. The existing water distribution system was evaluated using the minimum requirements. Some deficiencies were found in the existing system, which included service pump capacity and elevated storage capacity.

Next, the water distribution system was evaluated for the projected future demand. This analysis was based on 7-years of expected growth within the Village of Point Venture. The total number of Living Unit Equivalents (LUEs) in the Village of Point Venture through December 2022 was estimated at 1,000. An annual growth rate of 29 LUEs per year was estimated over the next 7 years, resulting in a total of 1,203 LUEs for the year 2029. Based on the number of property lots existing and available in the Village of Point Venture, the full-build out number of LUEs was capped at 1,190. Based on future projections, the water system was evaluated using the minimum requirements set forth by TCEQ. Some deficiencies were found in the existing system, which included service pump capacity and elevated storage capacity.

After the water system was evaluated for existing and future demands, it was then modeled using Bentley WaterGEMS software. For both existing and future conditions, simulations were performed for the average day, maximum day, peak hour, and fire flow demand scenario. Deficiencies occurred in the system, which included low pressures, mainly in the Lower Pressure Plane, and six hydrants not meeting fire flow requirements.

A list of recommended improvements was developed to address these deficiencies in the existing water distribution system and to meet the anticipated growth and demand. These improvements included: replacing the Augusta Standpipe; rehabilitating the Augusta Pump Station; rehabilitating the Augusta Elevated Storage Tank; installing a 6-inch waterline from Nicklaus Drive to Champions Circle; installing a Pressure Reducing Valve (PRV) assembly; installing a 2-inch waterline to transfer three lots to the Upper Pressure Plane; replacing 2-inch waterlines with 8-inch waterlines along Lakehead Circle and Lakeland Circle; and installing 6-inch waterlines along Valley Hill Drive and Valley Hill Lane to reallocate 35 LUEs to the Lower Pressure Plane.

Hydraulic modeling and evaluation of WCID Point Venture’s water system shows several deficiencies in the system when compared to TCEQ minimum requirements. If the recommended improvements are implemented, WCID Point Venture’s water system will adhere to the TCEQ minimum requirements and address operational and aging infrastructure issues at build-out.

1.0 INTRODUCTION

1.1 PROJECT SCOPE

The scope of this master plan is to evaluate and model WCID Point Venture's water distribution system. This master plan evaluated WCID Point Venture's treatment, pumping, storage, and distribution capacity and is an update to the Water System Study that was originally prepared by River City Engineering (RCE) and completed in October of 2013. The water master plan evaluated the ability of the existing system to meet TCEQ requirements for existing and projected future demands. Future demands were determined based on historical data and growth projections in Point Venture. Utilizing Bentley WaterGEMS, the water system was evaluated to determine if deficiencies exist. This was performed for existing and future conditions. A fire flow simulation for the existing and future conditions was also performed.

The report outlines and describes the results from evaluating WCID Point Venture's water system per TCEQ requirements for both existing and future conditions of the system. A summary of the water model results is also included. The report also provides recommendations to address system deficiencies encountered and prioritizes the importance and effect of these solutions on the overall system.

1.2 PROJECT DESCRIPTION

The first step of the water master plan was to evaluate the existing water system facilities and its current operation. Information pertaining to water lines, storage tanks, pumps, water meter connections, and water usage data was collected and used to evaluate individual pressure planes as well as the water distribution system as a whole. Field tests such as pressure testing selected homeowners' hose bibbs were performed where possible to verify record information. Existing treatment, pumping, and storage capacities were then compared to TCEQ's Chapter 290 minimum design requirements for public water systems. The existing water system was then modeled and calibrated using Bentley WaterGEMS software and any deficiencies were noted.

Following the evaluation of the existing system, an evaluation of the system based on future conditions was performed.

The future conditions evaluation was performed using a full build-out projection of the WCID Point Venture's service area. The full build-out projection was for the next 7 years. WCID Point Venture serves mainly single family homes, several townhomes, and a few commercial developments. The majority of existing vacant lots are projected to become single-family homes and a couple of remaining vacant lots are projected to be townhome development. The additional system demand expected from future growth was allocated to the water model and the system was again modeled using Bentley WaterGEMS.

A list of recommended improvements was generated based on the evaluation of the water system per TCEQ requirements and the results of the water model. This list identifies the major projects that are recommended to ensure the water distribution system can provide an adequate level of service and meet all TCEQ design requirements for both the existing system and the projected future growth.

2.0 WATER SYSTEM OVERVIEW

2.1 WCID POINT VENTURE JURISDICTION

WCID Point Venture is a registered public water system in the State of Texas and is listed as Public Water System (PWS) ID No. 2270038. WCID Point Venture provides water to customers in the Village of Point Venture in Travis County, Texas by Certificate of Convenience and Necessity (CCN) No. 10296. WCID Point Venture's Service Area is shown in Exhibit A.

2.2 WATER SOURCE

WCID Point Venture withdraws raw water from Lake Travis. WCID Point Venture has an existing contract, effective May 23, 2013, with LCRA to purchase and use LCRA's firm raw water for municipal use. This contract allows for a maximum annual authorized diversion of 285 acre-feet per year.

2.3 WATER SYSTEM FACILITIES

WCID Point Venture's Raw Water Intake Barge Pump Station withdraws raw water from Lake Travis and pumps to the existing WTP. The Raw Water Intake Barge Pump Station has 3 existing vertical turbine pumps (VTP), each rated at 350 gallons per minute (gpm), at a firm capacity of 700 gpm. The WTP treatment capacity is rated at 1.0 MGD (700 gpm). The WTP consists of two different plants: Plant A is a 0.5 MGD conventional clarifier and gravity filter media system, and Plant B is a 0.5 MGD Trident packaged adsorption clarifier and mixed media filter treatment system. Treated water is then pumped from each Plant A and Plant B Transfer Pump Stations to the existing 96,687 (100,000 nominal) gallon bolted steel Clearwell #1 and the existing 119,590 (120,000 nominal) gallon bolted steel Clearwell #2. Plant A Transfer Pump Station has 2 existing VTPs, each rated 350 gpm, at a firm capacity of 350 gpm. Plant B Transfer Pump Station has 2 existing end-suction pumps, each rated 350 gpm, at a firm capacity of 350 gpm. The WTP High Service Pump Station withdraws water from both Clearwells and pumps into the Lower Pressure Plane and into the existing 296,089 (300,000 nominal) gallon bolted steel Augusta Standpipe. The WTP High Service Pump Station has 2 existing VTPs, each rated 660 gpm, at a firm capacity of 660 gpm. It is to be noted the WTP's treatment capacity is restricted by the WTP High Service Pump Station. Therefore, the WTP is de-rated from 700 gpm (1.0 MGD) to 660 gpm (0.95 MGD). The Augusta Standpipe distributes water to the distribution system in the Lower Pressure Plane. The Augusta Pump Station withdraws water from the Augusta Standpipe and pumps into the Upper Pressure Plane and into the existing 50,000 gallon steel spheroid Augusta Elevated Storage Tank (EST). The Augusta Pump Station has 2 existing end-suction pumps, each rated 469 gpm, with a firm capacity of 469 gpm. The Augusta EST distributes water to the distribution system in the Upper Pressure Plane. A summary of the existing facilities is shown in Table 2-1. A summary of the existing water storage capacities is shown in Table 2-2.

Pressure planes are isolated areas of a distribution system that maintain a specified hydraulic grade for a particular service area. The hydraulic grade of a pressure plane is equal to water service elevation of the tank servicing the area during static conditions (i.e., no water being pumped through the system). Pressure planes are arranged so that a reasonable range of operating pressure exists in each area. Per TCEQ, reasonable operating pressures range from 35 to 80 pounds per square inch (psi). Pressure plane boundaries are usually dependent on topography and the geometric configuration or layout of the system.

WCID Point Venture's service area consists of two pressure planes: the Lower Pressure Plane and the Upper Pressure Plane. The Lower Pressure Plane is served by the Augusta Standpipe, which provides both ground and elevated storage. The Upper Pressure Plane is served by the Augusta Pump Station and the Augusta EST, which provides only elevated storage. Clearwells #1 and #2 and the WTP High Service Pump Station provides system storage and pumping capacity, respectively, for both Lower and Upper Pressure Planes. See Exhibit B for WCID Point Venture's Existing Pressure Plane Schematic.

TABLE 2-1. SUMMARY OF EXISTING FACILITIES

Facility	Supply Source	Capacity (gal)	Supply/Service Pumps		Service Area
			Individual (gpm)	Firm Capacity (gpm)	
Raw Water Barge Intake Pump Station	Lake Travis	-	350 350 350	700	-
WTP	Raw Water Barge Intake Pump Station	0.95 MGD	-	-	-
Plant A Transfer Pump Station	Plant A	-	350 350	350	-
Plant B Transfer Pump Station	Plant B	-	350 350	350	-
Clearwell #1	Plant A / B Transfer Pump Stations	96,687	-	-	System
Clearwell #2	Plant A / B Transfer Pump Stations	119,590	-	-	System
WTP High Service Pump Station	Clearwells #1 and #2	-	660 660	660	System
Augusta Standpipe	WTP High Service Pump Station	296,089	-	-	Lower Pressure Plane
Augusta Pump Station	Augusta Standpipe	-	469 469	469	Upper Pressure Plane
Augusta EST	Augusta Pump Station	50,000	-	-	Upper Pressure Plane

TABLE 2-2. SUMMARY OF EXISTING WATER STORAGE CAPACITY

Tank	I.D. (ft)	FFE (ft AMSL)	Overflow (ft AMSL)	Height, FFE to Overflow (ft)	System Capacity (gal)	Lower Pressure Plane[1]		Upper Pressure Plane[2]	
						Ground Storage (gal)	Elevated Storage (gal)	Ground Storage (gal)	Elevated Storage (gal)
Clearwell #1	26.75	745.6	768.6	23.0	96,687	-	-	-	-
Clearwell #2	29.75	745.6	768.6	23.0	119,590	-	-	-	-
Augusta Standpipe	30.0	822.0	878.0	56.0	296,089	280,227	15,862	-	-
Augusta EST	24.0	824.5	959.5	135.0	50,000	-	-	-	50,000
Subtotal	**-**	**-**	**-**	**-**	**562,366**	**280,227**	**15,862**	**0**	**50,000**

[1]Lower Pressure Plane serves a maximum elevation of 795.00 ft AMSL. Elevated storage is 875.00 ft AMSL and above.
[2]Upper Pressure Plane serves a maximum elevation of 836.50 ft AMSL. Elevated storage is 940.50 ft AMSL and above.

3.0 DESIGN CRITERIA

3.1 LIVING UNIT EQUIVALENTS (LUES)

The quantification and projecting of water use in terms of Living Unit Equivalents (LUEs) provides a useful way of quantifying the different water usage in a system. A LUE is defined as the typical flow that would be produced by a single-family residence. Previously water usage was quantified in terms of the number of connections in a system. This becomes an issue when a connection serves development that has a water usage different than a single-family residence. For example, a multi-family or commercial development will typically use more water than a single-family residence. A particular number of LUEs are assigned to each development based on its projected water use. The number of LUEs that are to be assigned are based on the size of the water meter to be used. The size of the water meter needed is based on the fixture unit count for the development.

Individual plumbing fixtures are assigned a value based on the typical flow used, and the cumulative total of the service is then used to determine the size of the meter needed. These plumbing fixtures values and meter sizing are taken from the American Water Works Association's (AWWA) Manual M22, Sizing Water Service Lines and Meters. LUEs are determined in accordance with the following meter equivalent recommendations, as shown below in Table 3-1, per AWWA Standards C700, C701, and C702.

TABLE 3-1. METER EQUIVALENCIES BASED ON RESPECTIVE WATER METER SIZE AND CAPACITY

Meter Size	Meter Type	Maximum Rate for Continuous Duty (gpm)	Meter Equivalents (LUEs)
5/8" x 3/4"	Displacement	10	1.0
3/4"	Displacement	15	1.5
1"	Displacement	25	2.5
1-1/2"	Displacement	50	5.0
2"	Displacement	80	8.0
2"	Compound, Class II	80	8.0
2"	Turbine, Class I	100	10.0
3"	Compound, Class II	175	17.5
3"	Turbine, Class I	220	22.0
4"	Compound, Class II	300	30.0
4"	Turbine, Class I	420	42.0
6"	Compound, Class II	675	67.5
6"	Turbine, Class I	865	86.5
8"	Compound, Class II	900	90.0
8"	Turbine, Class II	2,400	240.0
10"	Turbine, Class II	3,500	350.0

Currently, the makeup of the Village of Point Venture is predominantly single family homes with approximately 237 townhomes and a few commercial developments such as the retail center and floating barge restaurant. The single family homes, townhomes, and retail center have 5/8" x 3/4" size displacement meters. There are approximately twenty-five 5/8" x 3/4" displacement irrigation meters utilized by the townhomes and the Point Venture Property Owners Association (POA). There are approximately four 1-inch displacement irrigation meters utilized by the townhomes and docks. For future projections, there are plans to develop additional single family homes and townhomes. Quantifying water use in terms of LUEs and not connections correlates with industry standards when projecting future water demands in the service area.

3.2 HISTORICAL WATER USE

WCID Point Venture records monthly water use through a master meter located at the WTP. This provides record information for the District's monthly LCRA water diversion. The District's water use as recorded over the past 5 years is shown below in Table 3-2. .

TABLE 3-2. WCID POINT VENTURE HISTORICAL WATER USE SUMMARY

Year	Yearly Avg. LUEs	Total Yearly Water Use (MG)	Yearly Avg. Daily Water Use (gpd)	Yearly Avg. Daily Water Use per LUE (gpd/LUE)	Yearly Max. Daily Water Use (gpd)	Yearly Max. Daily Water Use per LUE (gpd/LUE)	Yearly Max. Daily Water Use per LUE (gpm/LUE)
2018	873	56.757	155,390	178	372,937	427	0.30
2019	903	54.532	149,154	165	357,970	396	0.28
2020	938	63.016	171,913	183	412,591	439	0.30
2021	951	66.314	181,936	191	436,646	459	0.32
2022	989	66.674	182,548	184	448,889	442	0.31

The preceding table shows the average and maximum daily water usage. WCID Point Venture provided meter data, which was converted to LUEs utilizing the meter equivalencies in Table 3-1 and averaged for each year.
WCID Point Venture provided both total and average daily water usage data, which the total water usage was summed for each year and the average daily water usage was averaged for each year. WCID Point Venture did not have available data for maximum daily water usage. TCEQ §290.38(46) mentions in the absence of verified historical data to multiply the average daily demand by 2.4 to calculate for the maximum daily demand. The average daily water usage data was multiplied by the 2.4 factor and was averaged for each year. The historical and calculated water usage, meter, and LUE data are provided in Exhibit C.

3.3 DESIGN REQUIREMENTS

WCID Point Venture's water system was evaluated using TCEQ §290.44 water distribution and §290.45 minimum water system capacity design requirements for public water systems.

TCEQ provides guidelines when determining average and maximum daily flows in the absence of historical data. However, for this project, WCID Point Venture historical data was used to determine the average and maximum daily flows, and the peak hour flow. Water loss in the system was not evaluated as a part of this master plan.

3.3.1 TCEQ DESIGN REQUIREMENTS

Since WCID Point Venture withdraws raw water from Lake Travis for treatment and distribution, the water system was evaluated using TCEQ §290.45(b)(2) for community surface water systems.

Raw water pump capacity must have a minimum capacity of 0.6 gpm (864 gpd) per LUE with the largest pump out of service. Treatment plant capacity must have a minimum capacity of 0.6 gpm (864 gpd) per LUE under normal rated design flow. Transfer pumps must have a minimum capacity of 0.6 gpm (864 gpd) per LUE with the largest pump out of service. TCEQ §290.38(83) defines transfer pump as any pump which conveys water from one point to another within the water treatment process or which conveys water to storage facilities prior to distribution. On October 30, 2012, TCEQ authorized WCID Point Venture an alternative production capacity of 0.46 gpm (662 gpd) per LUE, which includes the raw water pumps, transfer pumps, and the WTP.

Community surface water systems must also have a total storage capacity of 200 gallons per LUE, and an elevated storage capacity of 100 gallons per LUE. According to TCEQ §290.38(25), elevated storage is defined as the portion of water that can be stored at least 80 feet above the highest service connection in the pressure plane served by the storage tank. Covered clearwell storage at the WTP must have a minimum capacity of 50 gallons per LUE or for systems serving more than 250 LUEs, 5.0% of daily plant capacity.

Service pump capacity should provide each pump station or pressure plane with two or more pumps that have a total capacity of 2.0 gpm (2,880 gpd) per LUE, or that have a total capacity of at least 1,000 gpm and the ability to meet peak hourly demands with the largest pump out of service, whichever is less. TCEQ §290.38(79) defines service pump as any pump that takes treated water from storage and discharges to the distribution system. According to TCEQ §290.45(b)(2)(F), if a system provides elevated storage capacity of 200 gallons per LUE, two service pumps with a minimum combined capacity of 0.6 gpm per connection are required at each pump station or pressure plane.

TCEQ §290.44(d) provides minimum system pressure requirements. TCEQ §290.44(d) requires that the system must be designed to maintain a minimum pressure of 35 psi at all points within the distribution network at a flow rate of at least 1.5 gpm (2,160 gpd) per LUE. When the system is intended to provide firefighting capability, it must be able to maintain a minimum pressure of 20 psi under combined fire and drinking water flow conditions.

3.3.2 HISTORICAL WATER USAGE

Historical water usage and connection records were provided to Trihydro Corporation (Trihydro) by WCID Point Venture. Historical data for water usage and number of meters is presented in Exhibit C. Since Village of Point Venture is currently a second family home and vacation destination, only the months of June through August of each historical year were analyzed. Based upon this analysis, August 2022 was used to determine the average daily demand, maximum daily demand, and the peak hour demand as it represented the highest historical water demands in the system. The maximum daily demand represents the highest anticipated water usage for a 24-hour period during any given year. In the absence of verified historical data, TCEQ §290.38(60) defines peak hourly demand as 1.25 times the maximum daily demand (prorated to an hourly rate) if a public water system meets TCEQ minimum requirements for elevated storage capacity. A hydraulic pattern was also calculated based on minimum and maximum average daily water usage. The low demand hour multiplier was found to be 0.416 and this corresponds to hours where people are typically asleep or away at work. The high demand hour multiplier was found to be 1.458 and this corresponds to the hours where people would typically be at home and using water. The high demand multiplier was also used to determine peak hourly demand. Peak hourly demand represents the highest anticipated water usage for any given instant in the water system.

Water usage is based on metered water treated at the WTP. Exhibit C indicates that the average daily demand is 267 gpd per LUE or 0.185 gpm per LUE in August 2022. A maximum daily water usage of 386,000 gallons with 993 LUEs occurred on August 15, 2022. This equates to a maximum daily demand of 933 gpd per LUE, or 0.648 gpm per LUE. Using the TCEQ peaking factor of 1.25 and a maximum demand hour multiplier of 1.458, the peak hour demand was determined to be 1,701 gpd per LUE, or 1.181 gpm per LUE. These results are summarized below in Table 3-3.

TABLE 3-3. WATER USAGE DEMAND SUMMARY

Average Daily Demand	267 gpd/LUE	0.185 gpm/LUE
Maximum Daily Demand	933 gpd/LUE	0.648 gpm/LUE
Peak Hourly Demand	1,701 gpd/LUE	1.181 gpm/LUE

3.3.3 FIRE PROTECTION REQUIREMENTS

Fire flow demands can represent a large fraction of the total demand for a water distribution system. The effects of fire demands are difficult to derive precisely since fires occur with random frequency in different areas, with each having unique fire protection requirements. Generally, the amount of water needed to adequately fight a fire depends on the size of the structure, materials, combustibility of the contents, and the proximity to adjacent buildings.

Providing adequate water for fire protection is beneficial to residents and businesses in community because it lowers insurance rates. The Insurance Services Office (ISO) audits and rates a community's fire department and the capacity of the water distribution system using the Fire Suppression Rating Schedule. The ISO evaluation process is summarized in AWWA Manual M31 (2008), Distribution System Requirements for Fire Protection.

Since systems will be evaluated using ISO methods, WCID Point Venture may consider requiring the design of the fire protection system be based on the ISO rating system. However, there are other systems commonly used in Texas such as the National Fire Protection Association (NFPA) and the International Fire Code (IFC) standards. For this Report, the ISO method was used. This includes determining fire flow demands according to the ISO approach. The ISO method yields a Needed Fire Flow (NFF) that can be used for design and evaluation of the system. There are different calculation methods that are used for different building types, such as residential, commercial, or industrial. Table 3-4 shows the NFF for one- and two-family residences.

TABLE 3-4. REQUIRED RESIDENTIAL FIRE FLOW

Distance Between Buildings (ft)	Fire Flow (gpm)
More than 100	500
31 – 100	750
11 – 31	1,000
Less than 11	1,500

The NFF for commercial and industrial structures is based on building area, construction class (frame or masonry construction), occupancy (e.g., department store or chemical manufacturing plant), exposure (distance to and type of nearest building), and communication (types and locations of doors and walls). The formula for determining NFF is summarized below:

$$\mathrm{NFF} = 18 \times \mathrm{F} \times (\mathrm{A}^{0.5}) \times \mathrm{O}\ (1 + \mathrm{X} + \mathrm{P})$$

Where:

NFF = Needed Fire Flow (gpm)
F = Class of Construction Coefficient
A = Effective Area (ft^2)
O = Occupancy Factor
X = Exposure Factor
P = Communication Factor

The procedure for determining NFF is documented in the ISO Fire Suppression Rating Schedule (2013) and AWWA Manual M31 (2008). The minimum need fire flow should not be less than 500 gpm, and the maximum should not exceed 12,000 gpm.

In addition to a flow rate requirement for fire protection, a requirement exists for the duration over which the flow can be supplied. According to ISO (2013), fires requiring 3,500 gpm or less are referred to as receiving "Public Fire Suppression," and those requiring greater than 3,500 gpm are classified as receiving "Individual Property Fire Suppression." For fires requiring 2,500 gpm or less, a 2-hour duration is sufficient; for fires requiring 3,000 to 3,500 gpm, a 3-hour duration is used; and for fires requiring more than 3,500 gpm, a 4-hour duration is used.

For the purpose of this master plan, a minimum required fire flow rate of 1,000 gpm was used to evaluate the water system. This value was chosen because WCID Point Venture serves primarily single-family residential connections, with building separation typically in the range of 11 to 31.

4.0 EXISTING SYSTEM REVIEW

WCID Point Venture's existing water system was evaluated using TCEQ design requirements for public water systems and summarized in Table 4-1. Meter data was provided to Trihydro and converted to LUEs utilizing the meter equivalencies as shown in Exhibit C. As of December 2022, there are a total of 1,005 LUEs in the water system. For the purpose of evaluating the existing system, an approximate value of 1,000 LUEs were used. It is approximated that 600 of the 1,000 LUEs exist in the Lower Pressure Plane and the remaining 400 LUEs exist in the Upper Pressure Plane.

Initial review of the existing water system facilities shows that the WTP Raw Water Pump Station, the Treatment Plant itself, the WTP Transfer Pump Stations, the WTP High Service Pump Station, and the Clearwells meet capacity requirements. Within the Lower Pressure Plane, the Augusta Standpipe provides adequate total storage capacity. However, it is apparent that there is deficient elevated storage capacity in the Augusta Standpipe to serve the pressure plane. The Augusta Pump Station is deficient in pumping capacity. In the Upper Pressure Plane, the Augusta EST provides adequate elevated storage capacity. However, it is to be noted that there is not too much available LUE capacity in the Upper Pressure Plane as the Village of Point Venture further develops. Subsequent sections in the report will discuss options for improvements to the Augusta Standpipe and the Augusta Pump Station to meet TCEQ compliance.

Additionally, water model simulation was used to evaluate the system's ability to maintain adequate pressure. The computer modeling indicates there are areas where pressure problems exist. These results are presented in subsequent sections of this report.

TABLE 4-1. EXISTING SYSTEM CAPACITY REQUIREMENTS

Facility	Minimum Requirements	Existing Capacity
WTP		
Raw Water Pump Capacity	0.46 gpm/LUE	700 gpm
		1,522 LUEs
Treatment Plant Capacity	0.46 gpm/LUE	660 gpm
		1,435 LUEs
Transfer Pump Capacity		
Plant A Transfer Pump Station	0.46 gpm/LUE	350 gpm
		761 LUEs
Plant B Transfer Pump Station	0.46 gpm/LUE	350 gpm
		761 LUEs
Subtotal		*700 gpm*
		1,522 LUEs
Clearwell Storage Capacity		
Clearwells #1 #2	5% daily plant capacity	216,277 gal
		22.76%
Service Pump Capacity		
WTP High Service Pump Station	2.00 gpm/LUE	1,320 gpm
		660 LUEs
Lower Pressure Plane		
Ground Storage Capacity		
Augusta Standpipe	200 gal/LUE	280,227 gal
		1,401 LUEs
Elevated Storage Capacity		
Augusta Standpipe	100 gal/LUE	15,862 gal
		159 LUEs
Upper Pressure Plane		
Elevated Storage Capacity		
Augusta EST	100 gal/LUE	50,000 gal
		500 LUEs
Service Pump Capacity		
Augusta Pump Station	2.00 gpm/LUE	469 gpm
		235 LUEs

5.0 FUTURE CONDITIONS SYSTEM REVIEW

5.1 POPULATION & LUE PROJECTION

In order to properly evaluate the future conditions of the system, growth projections of the system were developed. Many different factors affect growth and development within the area. These include, but are not limited to, the local and regional economy, development restrictions, environmental constraints, current housing inventory, and existing and proposed roadways. For the purpose of this master plan, the average LUEs added to the system over the past five years, 29 LUEs per year, was used to predict future growth. These projections are only intended to serve as a guide.

WCID Point Venture serves customers that live in the Village of Point Venture and the amount and type of growth that is available in the Village of Point Venture is limited. WCID Point Venture serves a majority of single family residences with considerable amount of townhomes, which overtime is slowly transitioning from second family home and vacation destination to permanent residential and retirement community. The majority of existing vacant lots are projected to become single-family homes and a couple of remaining vacant lots are projected to become townhome development. It is projected at full build out, the number of LUEs in the WCID Point Venture service area would be approximately 1,190 LUEs. This is based on approximately 1,155 property lots at 1 LUE each and an additional 35 LUEs from irrigation meters. Using an average rate of 29 LUEs per year, it was determined the service area would be at full build out in Year 2029. Growth projections are shown in Table 5-1.

TABLE 5-1. GROWTH PROJECTIONS

Year	Avg. # of LUEs per Year	Avg. LUEs Added per Year	% Growth Rate per Year
2018	877		
2019	907	30	3.4
2020	942	35	3.9
2021	955	13	1.4
2022	990	35	3.7
Avg. Growth Rate		**29**	**3.1**

Using the average growth rate of 29 LUEs per year, yearly projections were made until the system reached its full build out capacity of approximately 1,190 LUEs. Table 5-2 shows future yearly projections.

TABLE 5-2. FUTURE GROWTH PROJECTIONS

Year	Avg. # of Projected LUEs (Using 29 LUEs/Year Growth Rate)
2023	1,029
2024	1,058
2025	1,087
2026	1,116
2027	1,145
2028	1,174
2029	1,203

5.2 TREATMENT CAPACITY

Initial review of the existing WTP shows it does meet the minimum TCEQ §290.45(b)(2)(B) requirements with respect to treatment capacities. In a letter dated October 30, 2012, TCEQ granted WCID Point Venture a new minimum alternative capacity of 0.46 gpm/LUE that is still current. WCID Point Venture's WTP will be at 85% capacity at the alternative capacity when the system serves 1,220 LUEs and will be at 100% capacity at 1,435 LUEs. Full build-out in Point Venture should occur at the projected 1,190 LUEs, therefore the WTP will meet TCEQ requirements at full build-out with no further capacity required.

5.3 PUMPING CAPACITY

As discussed in Section 4.0, WCID Point Venture's pumping capacities meet the minimum TCEQ requirements with the exception of the Augusta Pump Station under both existing and future conditions. The TCEQ §290.45(b)(2)(F) requirement for a service pump station is to provide each pump station or pressure plane with two or more pumps that have a total capacity of 2.0 gpm per LUE or a total capacity at least 1,000 gpm and the ability to meet peak hourly demands with the largest pump out of service. The existing Augusta Pump Station has 2 pumps with a firm pumping capacity of 469 gpm, which is only a total capacity of 1.17 gpm/LUE based on the existing 400 LUE's in the Upper Pressure Plane. To meet both existing and future demands, the Augusta Pump Station needs to be upgraded to have a minimum firm pumping capacity of 1,000 gpm with the largest pump out of service (2.0 gpm per LUE @ 500 projected LUEs). Section 7.2 will further discuss these improvement options that will allow the Augusta Pump Station to meet TCEQ requirements at both existing and future conditions.

For the WTP High Service Pump Station, the TCEQ §290.45(b)(2)(F) requirement is two service pumps with a minimum combined capacity of 0.6 gpm per LUE at each pressure plane or pump station for systems providing an elevated storage capacity of 200 gallons per LUE. If the system does not provide an elevated storage capacity of 200 gallons per LUE, the total capacity required is 2.0 gpm per connection. Currently, the WTP High Service Pump Station has a total capacity of 1,320 gpm (1.3 gpm per LUE at 1,000 LUEs) which will not meet the 2.0 gpm per connection requirement and in the future would only have a capacity of 1.1 gpm per LUE at 1,190 LUEs. Improvements to the Augusta Standpipe will entail utilizing the 200 gallons per LUE for elevated storage capacity, which will be explained in Section 5.4. The combined capacity required at full build-out is 714 gpm (0.6 gpm per LUE at 1,190 projected LUEs). There are 2 existing pumps with a combined capacity of 1,320 gpm, yielding a 1.11 gpm per LUE value. Therefore, the WTP High Service Pump Station meets the TCEQ combined capacity requirement at full build-out.

5.4 STORAGE CAPACITY

Review of the existing WCID Point Venture storage capacities shows that the total existing storage capacity is able to meet minimum TCEQ requirements under existing and future conditions. The TCEQ required minimum total storage for the entire system at full build-out is 238,000 gallons (200 gallons per LUE @ 1,190 projected LUEs). The existing storage capacity for the entire system is 562,366 gallons. Although there is enough storage capacity in the entire system to meet requirements, there is not enough elevated storage capacity in the Lower Pressure Plane to meet requirements at full build-out.

For the Lower Pressure Plane, the TCEQ minimum elevated storage capacity is 138,000 gallons (200 gallons per LUE @ 690 projected LUEs). The Lower Pressure Plane has an elevated storage capacity of 15,862 gallons and a ground storage capacity of 280,227 gallons, all from the existing Augusta Standpipe. As noted, the existing Augusta Standpipe is deficient in elevated storage to serve the Lower Pressure Plane. As detailed out in Section 7.3, the existing Augusta Standpipe will be replaced with a new standpipe incorporating the minimum 138,000 gallons of elevated storage. Ground storage capacity in the new standpipe will be approximately 407,122 gallons.

For the Upper Pressure Plane, the TCEQ minimum elevated storage capacity is 50,000 gallons (100 gallons per LUE @ 500 projected LUEs). The current Upper Pressure Plane has an elevated storage capacity of 50,000 gallons, all from the existing Augusta EST. Therefore, the Upper Pressure Plane will meet the TCEQ elevated storage capacity at full build-out. However, the way the existing distribution system is configured, there will be a need to transfer some LUEs from the Upper Pressure Plane into the Lower Pressure Plane to prevent exceedance of LUEs within the Upper Pressure Plane.

WCID Point Venture's future water system was evaluated using TCEQ design requirements for public water systems and summarized in Table 5-3.

TABLE 5-3. FUTURE SYSTEM CAPACITY REQUIREMENTS

Facility	Minimum Requirements	Future Capacity
WTP		
Raw Water Pump Capacity	0.46 gpm/LUE	700 gpm
		1,522 LUEs
Treatment Plant Capacity	0.46 gpm/LUE	660 gpm
		1,435 LUEs
Transfer Pump Capacity		
Plant A Transfer Pump Station	0.46 gpm/LUE	350 gpm
		761 LUEs
Plant B Transfer Pump Station	0.46 gpm/LUE	350 gpm
		761 LUEs
Subtotal		*700 gpm*
		1,522 LUEs
Clearwell Storage Capacity		
Clearwells #1 & #2	5% daily plant capacity	216,277 gal
		22.76%
Combined Service Pump Capacity		
WTP High Service Pump Station	0.60 gpm/LUE	1,320 gpm
		1.11 gpm/LUE
Lower Pressure Plane		
Ground Storage Capacity		
Augusta Standpipe	200 gal/LUE	407,122 gal
		2,036 LUEs
Elevated Storage Capacity		
Augusta Standpipe	200 gal/LUE	138,000 gal
		690 LUEs
Upper Pressure Plane		
Elevated Storage Capacity		
Augusta EST	100 gal/LUE	50,000 gal
		500 LUEs
Service Pump Capacity		
Augusta Pump Station	2.00 gpm/LUE	1,000 gpm
		500 LUEs

A summary of the future water storage capacities and the future facilities are shown in Table 5-4 and Table 5-5, respectively. A Proposed Pressure Plane Schematic, provided in Exhibit I, illustrates the future facilities for the water distribution system. Exhibit N shows the proposed extents of the pressure planes. A water model simulation was used to evaluate the system's ability to maintain adequate pressure under existing and future conditions. The results are presented in Section 6.

TABLE 5-4. SUMMARY OF FUTURE WATER STORAGE CAPACITY

Tank	I.D.	FFE	Overflow	Height, FFE to Overflow	System Capacity	Lower Pressure Plane[1]		Upper Pressure Plane[2]	
						Ground Storage	Elevated Storage	Ground Storage	Elevated Storage
	(ft)	(ft AMSL)	(ft AMSL)	(ft)	(gal)	(gal)	(gal)	(gal)	(gal)
Clearwell #1	26.75	745.60	768.60	23.00	96,687	-	-	-	-
Clearwell #2	29.75	745.60	768.60	23.00	119,590	-	-	-	-
Augusta Standpipe	30.00	822.00	926.50	104.50	545,122	407,122	138,000	-	-
Augusta EST	24.00	824.50	959.50	135.00	50,000	-	-	-	50,000
Subtotal	**-**	**-**	**-**	**-**	**811,399**	**407,122**	**138,000**	**0**	**50,000**

[1]Lower Pressure Plane serves a maximum elevation of 795.00 ft AMSL (feet above mean sea level). Elevated storage is 899.00 ft AMSL and above.

[2]Upper Pressure Plane serves a maximum elevation of 836.50 ft AMSL. Elevated storage is 940.50 ft AMSL and above.

TABLE 5-5. SUMMARY OF FUTURE FACILITIES

Facility	Supply Source	Capacity	Supply/Service Pumps		Service Area
			Individual	Firm Capacity	
		(gal)	(gpm)	(gpm)	
Raw Water Barge Intake Pump Station	Lake Travis	-	350 350 350	700	-
WTP	Raw Water Barge Intake Pump Station	0.95 MGD	-	-	-
Plant A Transfer Pump Station	Plant A	-	350 350	350	-
Plant B Transfer Pump Station	Plant B	-	350 350	350	-
Clearwell #1	Plant A / B Transfer Pump Stations	96,687	-	-	System
Clearwell #2	Plant A / B Transfer Pump Stations	119,590	-	-	System
WTP High Service Pump Station	Clearwells #1 and #2	-	660 660	660	System
Augusta Standpipe	WTP High Service Pump Station	545,122	-	-	Lower Pressure Plane
Augusta Pump Station	Augusta Standpipe	-	500 500 500	1,000	Upper Pressure Plane
Augusta EST	Augusta Pump Station	50,000	-	-	Upper Pressure Plane

6.0 WATER MODEL DEVELOPMENT

6.1 WATER MODEL DEVELOPMENT OVERVIEW

Hydraulic analysis of the distribution system was performed using Bentley WaterGEMS water modeling software. The water system was analyzed for both the existing and future conditions. For each condition, simulations were performed for the average day, maximum day, peak hour, and fire flow demand scenario. The average day and maximum day scenarios were performed using a 24-hour extended period simulation. The peak hour and fire flow demand scenarios were analyzed using a steady-state simulation.

6.1.1 PHYSICAL DATA

Physical data input into the water model includes water storage facilities, pump stations and pump characteristics, and pipe networks. The following sources of data were used to determine the physical data input:

- WCID Point Venture Utility Maps
- Construction Plans and drawings of existing facilities
- Topographic data from ArcGIS
- Historical Operational data obtained from WCID Point Venture

Pipes entered into the model ranged from 2-inch to 8-inch diameter. Pipes smaller than 6-inch diameter were entered into the model to accurately analyze problem areas at locations being served by smaller waterlines ranging from 2- to 4-inch diameter. Existing AutoCAD files from previous WCID Point Venture projects provided street and lot layouts. The street and lot layouts were imported as a background file into the water model. The general layout of the pipe network was obtained from the previous water model performed by RCE and was adjusted based on updated WCID Point Venture utility maps, as-built plans of improvement projects that were completed since 2013, and survey data provided by WCID Point Venture for the main line valves and fire hydrants. The pipe network was imported into ArcGIS to obtain the surface elevations of nodes, hydrants, tanks, and pumps. The topographical data was assembled from the surface elevations and incorporated into the model. Pipe lengths were automatically calculated by WaterGEMS when setting up the pipe layout. Additionally, Trihydro utilized the updated pipe network to develop a water system map to aid in the water model efforts. The water system map is shown in Exhibit D.

When pipes are added to the pipe network in WaterGEMS, the software allows the user to select the pipe material. Based on the data obtained, the pipes used in the system were asbestos cement, cast iron, ductile iron, and PVC. The Manning's roughness coefficients (C-values) used for various pipe materials are listed below in Table 6-1.

TABLE 6-1. TYPICAL C-VALUES

Pipe Material	C-Value
Asbestos Cement	140.0
Cast Iron	130.0
Ductile Iron	130.0
PVC	150.0

Pumps, tanks, and valves were also modeled based on the data sources listed above. Pump controls were modeled based on information provided by WCID Point Venture. Pumps were modeled in WaterGEMS using simple controls; pump ON and pump OFF were based on predetermined tank levels. Simple controls do not account for pump timers or delays.

6.1.2 DEMAND ALLOCATION

The general methodology for developing water demands for the system was to first develop a per LUE demand, and then determine the total number of LUEs within the system. As discussed in Section 3.3, the average day, maximum day, and peak hour demands per LUE were 0.185, 0.648, and 1.181, respectively.

To allocate the total demand, individual service areas were developed throughout the entire service area. The individual service areas were chosen based on pressure plane, similar location, topography, and water line size. Demand nodes for each individual service area were entered into WaterGEMS. An overlay of the demand nodes and aerial map was generated to determine the number of LUEs per demand node. For each demand node, the demand was calculated by multiplying the number of LUEs in the individual service area by the per LUE demand as discussed above. For example, if the individual service area had 10 LUEs, the average day demand in the model would be 1.85 gpm (0.185 gpm/LUE at 10 LUEs). Exhibit E provides a map depicting each service area boundary.

6.1.3 MODEL CALIBRATION

The existing conditions model was calibrated by running the fire flow scenario and then comparing the results to fire flow testing that was performed by Capital Hydrant, LLC on December 15, 2022. Fire flow tests are conducted to determine pressure and flow-producing capabilities within the distribution system. The primary function of fire flow test is to determine how much water is available for fighting fires, but the tests also serve as a means of determining the general condition of the distribution system (AWWA M17, 2006). Each test requires one hydrant designated as the Residual Hydrant to measure both static and residual pressures, and one or more hydrants designated as the Flow Hydrants to measure the flow rate. Static pressure represents the pressure at a given point under normal distribution system conditions and is measured at the Residual Hydrant with no Flow Hydrants opened. Residual pressure is the pressure that exists in the distribution system, measured at the Residual Hydrant at the same time flow measurements are taken at the Flow Hydrants. Fire hydrants should maintain a residual pressure of 20 psi for effective firefighting. Available fire flow rate is then calculated using the field measurements to determine the flow available at a residual pressure of 20 psi. The residual pressures and fire flow available at 20 psi at the test hydrants were compared to the modeled results at each test location. The results of the fire flow test are provided in Exhibit F.

A certain level of system error always exists in modeling due to many variables modeled, and the precision of information available to be input into the model. Due to some of the fire flow testing results having residuals and available fire flows that were lower than what should typically be seen, only four test locations were used for calibrating. The testing results that were not used had residual fire flows less than 20 psi and available fire flow less than 1,000 gpm at 20 psi. The modeled results at these locations had above 100% differences from the actual results for fire flow availability and large differences on the residual pressure. Also, the hydrant testing was performed in December 2022 when most of the residents in Point Venture are not present. This may have affected the hydrant testing results because the system was not experiencing the higher demands that typically occur in the summer. Based on the comparisons of fire flow testing at these four test locations and modeled fire results, we felt that the water model had been calibrated to an acceptable level.

6.2 EXISTING CONDITIONS MODEL

Performance and adequacy of the WCID Point Venture water system under existing conditions was analyzed for the four demand scenarios. The four demand scenarios included 24-hour extended period simulation for average day and maximum day scenarios, and a steady-state simulation for the peak hour and fire flow scenarios. The existing conditions model reflects the typical operations of the system as of December 2022. The results from the existing conditions simulation are presented in the following sections.

6.2.1 AVERAGE DAY SIMULATION

The average day simulation is a 24-hour extended period simulation based on an average day demand of 0.185 gpm per LUE. The results of the simulation are presented in Exhibit G-1.

The results of the average day simulation shows that most of the system is able to maintain the minimum required pressure of 35 psi. There were sixteen (16) nodes in total in the system with a minimum pressure at or below 35 psi during the 24-hour extended period simulation. Seven (7) of these nodes were near the standpipe, which is an area expected to have low pressure due to the node elevations and GST elevation being similar. The remaining nine (9) nodes that experienced low pressures were located in high elevation portions of the lower pressure plane. The corresponding nodes with low or marginal pressures during the average day simulation are shown in Table 6-2.

TABLE 6-2. AREAS OF LOW PRESSURE (EXISTING CONDITIONS, AVERAGE DAY)

Node	Approximate Location	Pressure Plane	Minimum Pressure (psi)
J-101	18605 Venture Dr 78645	Lower	23.19
J-102	18605 Venture Dr 78645	Lower	23.35
J-116	18606 Venture Dr 78645	Lower	22.71
J-135	19003 Venture Dr 78645	Lower	34.39
J-42	18511 Venture Dr 78645	Lower	24.54
J-55	18501 Venture Dr 78645	Lower	30.98
J-73	19042 Venture Dr 78645	Lower	34.04
LD17	19041 Venture Dr 78645	Lower	33.67
LD21	19003 Venture Dr 78645	Lower	34.33
LD28	18505 Venture Dr 78645	Lower	28.18
MLV-054	18603 Venture Dr 78645	Lower	23.30
MLV-073	19005 Venture Dr 78645	Lower	34.41
MLV-074	19003 Venture Dr 78645	Lower	34.78
MLV-086	19041 Venture Dr 78645	Lower	33.83
MLV-087	19041 Venture Dr 78645	Lower	33.49
MLV-088	19042 Venture Dr 78645	Lower	33.84

Areas of high pressure are defined as areas where the static pressure exceeds 85 psi. The maximum pressure during the EPS simulation for average day demand was greater than or equal to 85 psi at eight (8) nodes. The locations of high pressure are in the Upper Pressure Plane. These areas have high pressure due to being located at the lower elevations of the pressure plane. The corresponding nodes in the water model at these locations are shown in Table 6-3.

TABLE 6-3. AREAS OF HIGH PRESSURE (EXISTING CONDITIONS, AVERAGE DAY)

Node	Approximate Location	Pressure Plane	Maximum Pressure (psi)
J-123	18674 Champions Cir 78645	Upper	86.05
J-159	311 Southwind Rd 78645	Upper	86.36
J-201	18678 Champions Cir 78645	Upper	86.94
J-206	214 Southwind Rd 78645	Upper	85.73
J-4	212 S Venture Blvd 78645	Upper	96.39
MLV-046	18674 Champions Cir 78645	Upper	85.97
MLV-065	19009 Mariners Point 78645	Upper	95.29
UD17	18680 Champions Cir 78645	Upper	86.13

6.2.2 MAXIMUM DAY SIMULATION

The Maximum Day Simulation is a 24-hour extended period simulation based on a maximum day demand of 0.648 gpm per LUE. This simulation represents the highest anticipated water usage for a 24-hour period in a given year. The results of the simulation are presented in Exhibit G-2. The corresponding nodes in the water model with low pressures and high pressures are shown in Tables 6-4 and 6-5, respectively.

TABLE 6-4. AREAS OF LOW PRESSURE (EXISTING CONDITIONS, MAXIMUM DAY)

Node	Approximate Location	Pressure Plane	Minimum Pressure (psi)
J-101	18605 Venture Dr 78645	Lower	23.01
J-102	18605 Venture Dr 78645	Lower	23.35
J-116	18606 Venture Dr 78645	Lower	22.07
J-135	19003 Venture Dr 78645	Lower	34.39
J-42	18511 Venture Dr 78645	Lower	24.54
J-55	18501 Venture Dr 78645	Lower	30.98
J-73	19042 Venture Dr 78645	Lower	34.04
LD17	19041 Venture Dr 78645	Lower	33.68
LD21	19003 Venture Dr 78645	Lower	34.33
LD28	18505 Venture Dr 78645	Lower	28.18
MLV-054	18603 Venture Dr 78645	Lower	23.30
MLV-073	19005 Venture Dr 78645	Lower	34.41

Node	Approximate Location	Pressure Plane	Minimum Pressure (psi)
MLV-074	19003 Venture Dr 78645	Lower	34.78
MLV-086	19041 Venture Dr 78645	Lower	33.84
MLV-087	19041 Venture Dr 78645	Lower	33.49
MLV-088	19042 Venture Dr 78645	Lower	33.84

TABLE 6-5. AREAS OF HIGH PRESSURE (EXISTING CONDITIONS, MAXIMUM DAY)

Node	Approximate Location	Pressure Plane	Maximum Pressure (psi)
J-123	18674 Champions Cir 78645	Upper	85.99
J-159	311 Southwind Rd 78645	Upper	86.31
J-201	18678 Champions Cir 78645	Upper	86.88
J-206	214 Southwind Rd 78645	Upper	85.68
J-4	212 S Venture Blvd 78645	Upper	96.33
MLV-046	18674 Champions Cir 78645	Upper	85.91
MLV-065	19009 Mariners Point 78645	Upper	95.20
UD17	18680 Champions Cir 78645	Upper	86.07

6.2.3 PEAK HOUR SIMULATION

The Peak Hour Simulation is a steady-state simulation based on a peak hourly demand of 1.181 gpm per LUE. This simulation represents the highest anticipated water usage at any given instant in the water system. The results of the simulation are presented in Exhibit G-3.

Since the Peak Hour Simulation represents the highest anticipated water usage at any given instant in the system, this scenario could occur with tanks at any level. The Peak Hour Simulation was modeled with tanks at average level, as previously performed in the average and maximum day scenarios. It is anticipated that this scenario will accurately identify areas with low pressure during peak hour and have similar results to what would be seen in Point Venture.

The results of the Peak Hour Simulation show low pressure at fifteen (15) locations. Seven (7) of these nodes are located just before the standpipe so they would be expected to have low pressure. The other eight (8) nodes are located at the highest elevations of the Lower Pressure Plane. The corresponding nodes are shown in Table 6-6.

TABLE 6-6. AREAS OF LOW PRESSURE (EXISTING CONDITIONS, PEAK HOUR)

Node	Approximate Location	Pressure Plane	Minimum Pressure (psi)
J-101	18605 Venture Dr 78645	Lower	23.35
J-102	18605 Venture Dr 78645	Lower	23.35
J-116	18606 Venture Dr 78645	Lower	23.29
J-135	19003 Venture Dr 78645	Lower	34.81
J-42	18511 Venture Dr 78645	Lower	24.54
J-55	18501 Venture Dr 78645	Lower	30.98
J-73	19042 Venture Dr 78645	Lower	34.64
LD17	19041 Venture Dr 78645	Lower	34.28
LD21	19003 Venture Dr 78645	Lower	34.75
LD28	18505 Venture Dr 78645	Lower	28.18
MLV-054	18603 Venture Dr 78645	Lower	23.30
MLV-073	19005 Venture Dr 78645	Lower	34.83
MLV-086	19041 Venture Dr 78645	Lower	34.44
MLV-087	19041 Venture Dr 78645	Lower	34.09
MLV-088	19042 Venture Dr 78645	Lower	34.44

The results of the Peak Hour Simulation show high pressures at 2 nodes. The corresponding nodes are shown in Table 6-7.

TABLE 6-7. AREAS OF HIGH PRESSURE (EXISTING CONDITIONS, PEAK HOUR)

Node	Approximate Location	Pressure Plane	Maximum Pressure (psi)
J-4	212 S Venture Blvd 78645	Upper	94.32
MLV-065	19009 Mariners Point 78645	Upper	92.58

6.2.4 FIRE FLOW SIMULATION

A Fire Flow Simulation was performed using the fire protection requirements previously discussed in section 3.3.3. WaterGEMS calculates the fire flow available at each node in the system through an iterative process for a steady state simulation.

Although required fire flow for any node in the system is dependent on an individual's structure use, type of construction, and size, a minimum required fire flow of 1,000 gpm was specified at each node at a residual pressure of 20 psi. This minimum required fire flow was selected because the Village of Point Venture is mainly single-family homes with an average of 11-31 feet between buildings. A minimum required pressure in the system was set to 20 psi. Therefore, WaterGEMS will perform an analysis at each node to determine if that specific node can meet both the 1,000 gpm and 20 psi requirements while maintaining 20 psi in the rest of the system. WaterGEMS will calculate the maximum flow available while maintaining the pressure requirements. The existing conditions fire flow analysis is presented in Exhibit G-4.

The fire flow analysis shows that 9% (6 out of 68) of the fire hydrants modeled do not meet fire flow requirements of 1,000 gpm. These hydrants that do not meet the 1,000 gpm requirement are summarized below in Table 6-8. Exhibit G-4 provides a map of the fire hydrants that do and do not meet fire flow requirements.

TABLE 6-8. AREAS NOT MEETING FIRE FLOW REQUIREMENTS (EXISTING CONDITIONS)

Hydrant	Approximate Location	Pressure Plane	Fire Flow Available (gpm)	Pressure (Calculated Residual @ Total Flow Needed) (psi)
HL-24	200 Lakefront Dr 78645	Lower	428.373	-24.37
HL-25	304 Lakefront Dr 78645	Lower	525.492	-3.26
HL-31	405 Lakefront Dr 78645	Lower	676.78	17.27
HL-51	Hogan Cir 78641	Upper	785.649	-5.26
HL-52	518 Demarett Dr 78645	Upper	777.737	-4.77
HL-58	19001 Venture Dr 78645	Upper	914.172	18.36

Three (3) of the hydrant locations are in the Lower Pressure Plane furthest from the existing WTP site and pump station in the northwestern area of the Village of Point Venture. Three (3) of the hydrant locations are in the Upper Pressure Plane in or near the Demarett Drive/Palmer Drive neighborhood. The layout of the current system appears to be affecting the availability of fire flow in these areas.

6.3 FUTURE CONDITIONS MODEL

Performance and adequacy of the WCID Point Venture water system under future conditions was analyzed for the four demand scenarios. The four demand scenarios included 24-hour extended period simulation for average day and maximum day scenarios, and steady-state simulations for peak hour and for the fire flow scenario. The future conditions model represents what is to be believed typical operations of the future system at full build-out of the water system (approximately December 2032) based on the growth projections discussed in Section 5.1. The results from the future conditions simulation are presented in the following sections.

6.3.1 AVERAGE DAY SIMULATION

The Average Day Simulation is a 24-hour extended period simulation based on an average day demand of 0.185 gpm/LUE. The results of the simulation are presented in Exhibit H-1. There are only minor differences in pressure with future conditions compared to existing conditions and no changes in the number or locations of nodes that experienced low or high pressures. The locations of the low pressure nodes are all in the Lower Pressure Plane and the locations of all high pressure nodes for this simulation are in the Upper Pressure Plane. Results for the average day simulation at future conditions are shown below in Table 6-9 and 6-10 for areas of low pressure and high pressure, respectively.

TABLE 6-9. AREAS OF LOW PRESSURE (FUTURE CONDITIONS, AVERAGE DAY)

Node	Approximate Location	Pressure Plane	Minimum Pressure (psi)
J-101	18605 Venture Dr 78645	Lower	23.18
J-102	18605 Venture Dr 78645	Lower	23.35
J-116	18606 Venture Dr 78645	Lower	22.70
J-135	19003 Venture Dr 78645	Lower	34.38
J-42	18511 Venture Dr 78645	Lower	24.54
J-55	18501 Venture Dr 78645	Lower	30.98
J-73	19042 Venture Dr 78645	Lower	34.03
LD17	19041 Venture Dr 78645	Lower	33.67
LD21	19003 Venture Dr 78645	Lower	34.32
LD28	18505 Venture Dr 78645	Lower	28.18
MLV-054	18603 Venture Dr 78645	Lower	23.30
MLV-073	19005 Venture Dr 78645	Lower	34.40
MLV-074	19003 Venture Dr 78645	Lower	34.77
MLV-086	19041 Venture Dr 78645	Lower	33.83
MLV-087	19041 Venture Dr 78645	Lower	33.48
MLV-088	19042 Venture Dr 78645	Lower	33.83

TABLE 6-10. AREAS OF HIGH PRESSURE (FUTURE CONDITIONS, AVERAGE DAY)

Node	Approximate Location	Pressure Plane	Maximum Pressure (psi)
J-123	18674 Champions Cir 78645	Upper	86.04
J-159	311 Southwind Rd 78645	Upper	86.36
J-201	18678 Champions Cir 78645	Upper	86.94
J-206	214 Southwind Rd 78645	Upper	85.72
J-4	212 S Venture Blvd 78645	Upper	96.39
MLV-046	18674 Champions Cir 78645	Upper	85.96
MLV-065	19009 Mariners Point 78645	Upper	95.29
UD17	18680 Champions Cir 78645	Upper	86.13

The low pressure at these locations is caused by these areas being located in high elevations of the lower pressure plane that are less than 85 feet below the standpipe elevation. As discussed in Section 5, the standpipe will not have adequate elevated storage capacity at full build-out and will need to be replaced to compensate. The proposed improvement to the standpipe will increase the pressures in these areas and is detailed further in Section 7.3.

6.3.2 MAXIMUM DAY SIMULATION

The Maximum Day Simulation is a 24-hour extended period simulation based on a maximum day demand of 0.648 gpm/LUE. This simulation represents the highest anticipated water usage for a 24-hour period in a given year. The results of this simulation are presented in Exhibit H-2.

The results of the Maximum Day Simulation show that the same areas identified as having low and high pressures for the Average Day Simulation also have low pressures during the Maximum Day Simulation. The corresponding nodes with low and high pressures are shown below in Tables 6-11 and 6-12, respectively.

TABLE 6-11. AREAS OF LOW PRESSURE (FUTURE CONDITIONS, MAXIMUM DAY)

Node	Approximate Location	Pressure Plane	Minimum Pressure (psi)
J-101	18605 Venture Dr 78645	Lower	22.81
J-102	18605 Venture Dr 78645	Lower	23.35
J-116	18606 Venture Dr 78645	Lower	21.34

Node	Approximate Location	Pressure Plane	Minimum Pressure (psi)
J-135	19003 Venture Dr 78645	Lower	34.38
J-42	18511 Venture Dr 78645	Lower	24.54
J-55	18501 Venture Dr 78645	Lower	30.98
J-73	19042 Venture Dr 78645	Lower	34.04
LD17	19041 Venture Dr 78645	Lower	33.68
LD21	19003 Venture Dr 78645	Lower	34.32
LD28	18505 Venture Dr 78645	Lower	28.18
MLV-054	18603 Venture Dr 78645	Lower	23.30
MLV-073	19005 Venture Dr 78645	Lower	34.40
MLV-074	19003 Venture Dr 78645	Lower	34.77
MLV-086	19041 Venture Dr 78645	Lower	33.84
MLV-087	19041 Venture Dr 78645	Lower	33.49
MLV-088	19042 Venture Dr 78645	Lower	33.84

TABLE 6-12. AREAS OF HIGH PRESSURE (FUTURE CONDITIONS, MAXIMUM DAY)

Node	Approximate Location	Pressure Plane	Maximum Pressure (psi)
J-123	18674 Champions Cir 78645	Upper	85.95
J-159	311 Southwind Rd 78645	Upper	86.27
J-201	18678 Champions Cir 78645	Upper	86.84
J-206	214 Southwind Rd 78645	Upper	85.63
J-4	212 S Venture Blvd 78645	Upper	96.29
MLV-046	18674 Champions Cir 78645	Upper	85.87
MLV-065	19009 Mariners Point 78645	Upper	95.14
UD17	18680 Champions Cir 78645	Upper	86.03

6.3.3 PEAK HOUR SIMULATION

The Peak Hour Simulation is a steady-state simulation based on a peak hourly demand of 1.181 gpm/LUE. This simulation represents the highest anticipated water usage at any given instant in the water system. Exhibit H-3 shows the location of nodes that experienced low and high pressures in this scenario. Tables 6-13 and 6-14 show the corresponding nodes that experienced low and high pressures, respectively.

TABLE 6-13. AREAS OF LOW PRESSURE (FUTURE CONDITIONS, PEAK HOUR)

Node	Approximate Location	Pressure Plane	Minimum Pressure (psi)
J-101	18605 Venture Dr 78645	Lower	23.35
J-102	18605 Venture Dr 78645	Lower	23.35
J-116	18606 Venture Dr 78645	Lower	23.29
J-135	19003 Venture Dr 78645	Lower	34.70
J-42	18511 Venture Dr 78645	Lower	24.54
J-55	18501 Venture Dr 78645	Lower	30.98
J-73	19042 Venture Dr 78645	Lower	34.67
LD17	19041 Venture Dr 78645	Lower	34.31
LD21	19003 Venture Dr 78645	Lower	34.64
LD28	18505 Venture Dr 78645	Lower	28.18
MLV-054	18603 Venture Dr 78645	Lower	23.30
MLV-073	19005 Venture Dr 78645	Lower	34.72
MLV-086	19041 Venture Dr 78645	Lower	34.46
MLV-087	19041 Venture Dr 78645	Lower	34.12
MLV-088	19042 Venture Dr 78645	Lower	34.47

TABLE 6-14. AREAS OF HIGH PRESSURE (FUTURE CONDITIONS, PEAK HOUR)

Node	Approximate Location	Pressure Plane	Maximum Pressure (psi)
J-4	212 S Venture Blvd 78645	Upper	93.69
MLV-065	19009 Mariners Point 78645	Upper	91.66

6.3.4 FIRE FLOW SIMULATION

A Fire Flow Simulation was performed using the fire protection requirements outlined in Section 3.3. WaterGEMS calculates the fire flow available at each node in the system through an iterative process for a steady state simulation. A minimum required fire flow of 1,000 gpm was specified for each hydrant at a residual pressure of 20 psi. The fire flow analysis shows that the same 9% (6 of 68) of the fire hydrants modeled do not meet the fire flow requirements in the future scenario. These hydrants that do not meet the fire flow requirements are summarized below in Table 6-15. Exhibit H-4 provides a map of the hydrants that do and do not meet requirements.

TABLE 6.15. AREAS NOT MEETING FIRE FLOW REQUIREMENTS (FUTURE CONDITIONS)

Hydrant	Approximate Location	Pressure Plane	Fire Flow Available (gpm)	Pressure (Calculated Residual @ Total Flow Needed) (psi)
HL-24	200 Lakefront Dr 78645	Lower	425.959	-24.74
HL-25	304 Lakefront Dr 78645	Lower	522.381	-3.59
HL-31	405 Lakefront Dr 78645	Lower	672.731	17.01
HL-51	Hogan Cir 78641	Upper	780.453	-5.93
HL-52	518 Demarett Dr 78645	Upper	772.390	-5.43
HL-58	19001 Venture Dr 78645	Upper	907.850	17.79

Three (3) of the hydrant locations are in the Lower Pressure Plane furthest from the existing WTP site and pump station in the northwestern area of the Village of Point Venture. Three (3) of the hydrant locations are in the Upper Pressure Plane in or near the Demarett Drive/Palmer Drive neighborhood. The layout of the system could be affecting the availability of fire flow in these areas. A project connecting the Demarett Drive/Palmer Drive neighborhood to Champions Circle is recommended to solve the fire flow issues in the Upper Pressure Plane. To improve the fire flow in the Lower Pressure Plane, adding a PRV assembly on Lakefront Drive and opening the isolation valve on the same road is recommended. These improvement projects are further discussed in Section 7.4.

7.0 PROPOSED IMPROVEMENTS

7.1 PROPOSED IMPROVEMENTS OVERVIEW

After evaluating WCID Point Venture's water system against TCEQ requirements for existing and future conditions and performing a water system simulation, a list of proposed improvement projects was generated. These proposed improvements will allow WCID Point Venture's water system to meet TCEQ requirements for existing and future scenarios and meet the projected future demand of the system. A complete engineer's opinion of probable construction cost (OPCC) for each proposed improvement project is provided below. Each Engineer's OPCC includes engineering costs such as but not limited to: design, construction administration, and/or inspection.

7.2 PUMPING IMPROVEMENTS

One pumping improvement project is proposed. The Augusta Pump Station will need to be upgraded to meet TCEQ requirements for both existing and future conditions. The WTP Raw Water Pump Station, the WTP Plant A & Plant B Transfer Pump Stations, and the WTP High Service Pump Station do not need to be upgraded since they currently meet TCEQ requirements for both existing and future conditions.

7.2.1 AUGUSTA PUMP STATION REHABILITATION

The Augusta Pump Station does not meet TCEQ pumping requirements for both the existing and future conditions. The service pumping capacity needs to increase from 469 gpm to 1,000 gpm to be in compliance with TCEQ pumping requirements. The pump station conveys potable water from the Augusta Standpipe to the Augusta EST and the Upper Pressure Plane. The minimum Total Dynamic Head (TDH) required for each pump is 184 feet (79.65 psi). At a minimum, rehabilitation shall include: demolition of the two existing centrifugal pumps and associated piping and valving, electrical, and controls; and installation of three, 500 gpm centrifugal pumps and associated piping and valving including coatings, a new generator and automatic transfer switch (ATS), electrical, and controls. Also included at WCID Point Venture's discretion are modifications to the existing pump station metal frame building consisting of replacing the insulation, light fixtures, and HVAC equipment. The Engineer's OPCC to rehabilitate the Augusta Pump Station is provided below in Table 7-1.

TABLE 7-1. AUGUSTA PUMP STATION REHABILITATION OPCC

ITEM DESCRIPTION	UNIT	QTY	UNIT PRICE	TOTAL COST
MOBILIZATION, BONDING, INSURANCE	LS	1	$ 254,750.00	$ 254,750.00
DEMOLITION				
PUMPING AND VALVING EQUIPMENT	LS	1	$ 17,250.00	$ 17,250.00
ELECTRICAL AND CONTROL EQUIPMENT	LS	1	$ 34,500.00	$ 34,500.00
METAL FRAME BUILDING INSULATION	LS	1	$ 2,875.00	$ 2,875.00
PUMP STATION IMPROVEMENTS				
PERMITS	LS	1	$ 1,150.00	$ 1,150.00
CENTRIFUGAL PUMPS	EA	3	$ 63,000.00	$ 189,000.00
CONCRETE EQUIPMENT PADS	EA	3	$ 2,300.00	$ 6,900.00
10" DUCTILE IRON PIPE	LF	10	$ 839.00	$ 8,390.00
8" DUCTILE IRON PIPE	LF	25	$ 712.00	$ 17,800.00
6" DUCTILE IRON PIPE	LF	24	$ 644.00	$ 15,456.00
10" DUCTILE IRON GATE VALVE	EA	1	$ 3,510.00	$ 3,510.00
6" DUCTILE IRON GATE VALVE	EA	6	$ 1,560.00	$ 9,360.00
6" DUCTILE IRON SILIENT GLOBE CHECK VALVE	EA	3	$ 3,627.00	$ 10,881.00
DUCTILE IRON FITTINGS	TON	1.12	$ 25,300.00	$ 28,336.00
COATINGS	LS	1	$ 34,500.00	$ 34,500.00
BLANKET TYPE METAL FRAME BUILDING INSULATION	SF	900	$ 30.00	$ 27,000.00
UNIT HEATER	EA	1	$ 2,940.00	$ 2,940.00
MISCELLANEOUS METALS	LS	1	$ 23,000.00	$ 23,000.00
GENERATOR, EQUIPMENT PAD, AND ATS	LS	1	$ 115,000.00	$ 115,000.00
ELECTRICAL AND INSTRUMENTATION	LS	1	$ 301,317.00	$ 301,317.00
CONSTRUCTION SUBTOTAL				$ 1,103,915.00
DESIGN				$ 187,666.00
CONSTRUCTION ADMINISTRATION				$ 33,118.00
CONTINGENCY				$ 331,175.00
TOTAL				**$ 1,655,874.00**

7.3 STORAGE IMPROVEMENTS

Two storage improvements are proposed. The Augusta Standpipe will be replaced with a taller and larger capacity tank to address pressure issues within the Lower Pressure Plane, meet TCEQ storage requirements for future conditions, and address deficiencies related to the standpipe's condition and age. The Augusta EST will be rehabilitated to meet TCEQ and AWWA standards and address deficiencies related to the EST's condition and age. Clearwells #1 and #2 do not need to be upgraded since they currently meet TCEQ requirements for both existing and future conditions.

7.3.1 AUGUSTA STANDPIPE REPLACEMENT

The existing bolted steel Augusta Standpipe will need to be replaced to address deficiencies in elevated storage requirements for the Lower Pressure Plane, and to alleviate problem areas within the Lower Pressure Plane related to low pressures at various locations. Additionally, replacing the tank will address operational deficiencies related to the tank's age and condition since it is the original tank when the Village of Point Venture was first developed. Over time, this tank has experienced severe degradation and corrosion with areas of leakage between the gaskets and the side wall panels. It is recommended that the Augusta Standpipe be replaced with a 30'-0" diameter, 104'-6" tall, welded steel standpipe with a volume of 545,122 (550,000 nominal) gallons. The new standpipe will meet future elevated storage capacity requirements in the Lower Pressure Plane and increase both available storage and static pressures in the Lower Pressure Plane. To maintain adequate pressures throughout the Lower Pressure Plane, the new standpipe will be raised to at least 105 feet above the highest elevation in the Lower Pressure Plane to provide a minimum 45 psi to each homeowner's meter box. This improvement will address low pressure issues for homes along Hole #4 between Venture Drive/Demarett Drive and Venture Drive/Nicklaus Drive. By increasing the available storage capacity within the Lower Pressure Plane, 35 LUEs from the Upper Pressure Plane can be transferred to the Lower Pressure Plane to allow for both pressure planes to maintain TCEQ storage. This can be accomplished by installing 6-inch waterlines and isolation gate valves, which is further detailed in Section 7.4.4. The project would entail at a minimum: demolition of the existing bolted steel standpipe and foundation; temporary storage tanks and bypass pumping equipment; temporary easements for the temporary storage tanks and bypass pumping equipment within the Waterford property next to the Augusta tank site; installation of a new welded steel standpipe including a new ring wall foundation, associated tank appurtenances, piping, valving, and coatings, and electrical and controls modifications; and inspection services for the tank welding and interior and exterior coatings. Also included at WCID Point Venture's discretion are site improvements such as fencing replacement, yard piping modifications, and revegetation. The Engineer's OPCC to replace the Augusta Standpipe is provided below in Table 7-2.

TABLE 7-2. AUGUSTA STANDPIPE REPLACEMENT OPCC

ITEM DESCRIPTION	UNIT	QTY	UNIT PRICE	TOTAL COST
MOBILIZATION, BONDING, INSURANCE	LS	1	$ 1,107,592.00	$ 1,107,592.00
EROSION AND SEDIMENTATION CONTROL PLAN AND IMPLEMENTATION				
PERMITS	LS	1	$ 1,150.00	$ 1,150.00
DEVELOP SWPPP PLAN	LS	1	$ 2,300.00	$ 2,300.00
SILT FENCE	LF	250	$ 6.00	$ 1,500.00
STABILIZED CONSTRUCTION ENTRANCE	EA	1	$ 1,950.00	$ 1,950.00
MAINTAIN SWPPP	MO	13	$ 230.00	$ 2,990.00
DEMOLITION				
0.296 MG AUGUSTA STANDPIPE & FOUNDATION	LS	1	$ 143,750.00	$ 143,750.00
WOOD FENCING	LF	400	$ 7.00	$ 2,800.00
WOOD GATE	EA	2	$ 127.00	$ 254.00
TEMPORARY STORAGE TANKS AND BYPASS PUMPING	LS	1	$ 287,500.00	$ 287,500.00
TANK IMPROVEMENTS				
545,122 GALLON WELDED STEEL STANDPIPE	GAL	545,122	$ 5.00	$ 2,725,610.00
TANK RING WALL FOUNDATION	CY	48	$ 2,300.00	$ 110,400.00
BACKFILL (COMMON)	CY	71	$ 58.00	$ 4,118.00
STRUCTURAL BACKFILL	CY	196	$ 127.00	$ 24,892.00
LEVELING COURSE	CY	18	$ 368.00	$ 6,624.00
EXCAVATION SAFETY PLAN	LS	1	$ 1,150.00	$ 1,150.00
EXCAVATION SAFETY	PF	126	$ 115.00	$ 14,490.00
6" DUCTILE IRON CHECK VALVE	EA	1	$ 5,655.00	$ 5,655.00
6" DUCTILE IRON GATE VALVE	EA	2	$ 1,560.00	$ 3,120.00
8" DUCTILE IRON OVERFLOW FLAP VALVE	EA	1	$ 4,875.00	$ 4,875.00
DUCTILE IRON FITTINGS	TON	0.25	$ 25,300.00	$ 6,325.00
YARD PIPING	LS	1	$ 115,000.00	$ 115,000.00
HYDROSTATIC & DISINFECTION TESTING	LS	1	$ 17,250.00	$ 17,250.00
TRENCH SAFETY PLAN	LS	1	$ 1,150.00	$ 1,150.00
TRENCH SAFETY	LF	500	$ 58.00	$ 29,000.00
8' TALL WOOD FENCE	LF	400	$ 92.00	$ 36,800.00
8' TALL, 12' WIDE DOUBLE SWING WOOD GATE	EA	2	$ 8,625.00	$ 17,250.00
CLEAR AND GRUBB	SF	6,200	$ 2.00	$ 12,400.00
REVEGETATION (HYDROMULCH-TOPSOIL-WATERING)	SY	689	$ 6.00	$ 4,134.00
ELECTRICAL AND INSTRUMENTATION	LS	1	$ 107,534.00	$ 107,534.00
CONSTRUCTION SUBTOTAL				$ 4,799,563.00
DESIGN				$ 815,926.00
GEOTECHNICAL				$ 71,994.00
CONSTRUCTION MATERIALS TESTING				$ 95,992.00
CONSTRUCTION ADMINISTRATION				$ 143,987.00
TANK WELDING & COATING INSPECTION				$ 167,985.00
CONTINGENCY				$ 1,439,869.00
TOTAL				**$ 7,535,316.00**

7.3.2 AUGUSTA EST REHABILITATION

The existing steel spheroid Augusta EST will be rehabilitated to be in compliance with TCEQ and AWWA standards and to address operational deficiencies related to the tank's age and condition. Rehabilitation would involve: sandblasting and coating the tank interior and exterior; removing and replacing various tank appurtenances; rehabilitating access tube and roof manways; replacing the entire riser pipe; rehabilitating pipe supports on riser pipes; installing new level gauge and sample connection; installing new pressure transmitter assembly; and inspection services for the tank welding and interior and exterior coatings. During rehabilitation, piping to the Augusta EST will be valved off and the Augusta pumps will pump directly into the Upper Pressure Plane to maintain continuous water service. Additionally, the three cell providers will need to remove their equipment from the Augusta EST and install their antennas on temporary poles; there should be no cost associated the removal and reinstallation. The Engineer's OPCC to rehabilitate the Augusta EST is provided below in Table 7-3.

TABLE 7-3. AUGUSTA EST REHABILITATION OPCC

ITEM DESCRIPTION	UNIT	QTY	UNIT PRICE	TOTAL COST
MOBILIZATION, BONDING, INSURANCE	LS	1	$ 144,442.00	$ 144,442.00
TANK INTERIOR				
SANDBLASTING AND COATING	SF	4,900	$ 35.00	$ 171,500.00
10% ADDER FOR ACCESS TUBE, PLATFORMS, RISER-PIPES	LS	1	$ 19,723.00	$ 19,723.00
ADDER FOR CAULKING	LS	1	$ 5,750.00	$ 5,750.00
TANK EXTERIOR				
SANDBLASTING AND COATING	SF	3,950	$ 46.00	$ 181,700.00
ADDER FOR SHROUD	LS	1	$ 17,250.00	$ 17,250.00
ADDER FOR COATING APPURTENANCES	LS	1	$ 5,750.00	$ 5,750.00
APPURTENANCES				
REMOVE AND REPLACE 24" FLANGED ROOF VENT	LS	1	$ 8,625.00	$ 8,625.00
REMOVE AND REPLACE INTERIOR WET LADDER (+/- 4' LENGTH)	LS	1	$ 2,300.00	$ 2,300.00
REMOVE 24" ACCESS HATCH AND REPLACE WITH 30" (BOTTOM PLATFORM)	LS	1	$ 11,500.00	$ 11,500.00
REMOVE 24" ACCESS HATCH AND REPLACE WITH 30" (TOP PLATFORM)	LS	1	$ 11,500.00	$ 11,500.00
REHAB EXISTING 30" ACCESS TUBE AND ROOF MANWAYS	LS	1	$ 2,875.00	$ 2,875.00
REHAB PIPE SUPPORTS ON EXISTING 8" INLET/OUTLET PIPE	LS	1	$ 5,750.00	$ 5,750.00
REMOVE AND REPLACE 8" INLET/OUTLET PIPE	LS	1	$ 20,000.00	$ 20,000.00
LEVEL GAUGE AND SAMPLE CONNECTION	LS	1	$ 5,750.00	$ 5,750.00
PRESSURE TRANSMITTER ASSEMBLY	LS	1	$ 11,500.00	$ 11,500.00
CONSTRUCTION SUBTOTAL				$ 625,915.00
DESIGN				$ 106,406.00
CONSTRUCTION ADMINISTRATION				$ 18,778.00
TANK WELDING AND COATING INSPECTION				$ 21,908.00
CONTINGENCY				$ 187,775.00
TOTAL				**$ 960,782.00**

A proposed process flow diagram, provided in Exhibit J, illustrates the conceptual layout at the Augusta Tank Site to include the proposed Augusta Standpipe and proposed Augusta Pump Station. Design sizing and calculations for the proposed Augusta Standpipe, the proposed Augusta Pump Station, and evaluation of the existing WTP High Service Pump Station is included in Exhibit M.

7.4 DISTRIBUTION IMPROVEMENTS

After running the water model for existing and future conditions, five distribution improvement projects were identified that would solve the problem areas discovered in the model. These improvements will help meet the anticipated future demand, provide adequate water pressures, and increase fire flows in areas that do not meet the 1,000 gpm requirement as discussed in Section 6.3.

7.4.1 PRV ASSEMBLY

The first proposed distribution improvement is to install a PRV assembly to the north side of the intersection of Peckham Drive and Lakefront Drive. This improvement would increase the fire flow availability within the Lower Pressure Plane at three hydrant locations between the 200 and 405 address of Lakefront Drive, as mentioned in Section 6.3.4. Subsequently, the existing isolation valve at the intersection would be open to allow flow through the PRV assembly. The Engineer's OPCC for the PRV Assembly is provided below in Table 7-4.

TABLE 7-4. PRV ASSEMBLY OPCC

ITEM DESCRIPTION	UNIT	QTY	UNIT PRICE		TOTAL COST	
MOBILIZATION, BONDING, INSURANCE	LS	1	$	35,985.00	$	35,985.00
EROSION AND SEDIMENTATION CONTROL PLAN AND IMPLEMENTATION						
PERMITS	LS	1	$	1,150.00	$	1,150.00
DEVELOP SWPPP PLAN	LS	1	$	2,300.00	$	2,300.00
TREE PROTECTION	LF	75	$	6.00	$	450.00
SILT FENCE	LF	60	$	6.00	$	360.00
STABILIZED CONSTRUCTION ENTRANCE	EA	1	$	1,950.00	$	1,950.00
MAINTAIN SWPPP	MO	7	$	230.00	$	1,610.00
WATERLINE IMPROVEMENTS						
CLEAR AND GRUBB	SF	35	$	12.00	$	420.00
4" DUCTILE IRON PIPE	LF	15	$	342.00	$	5,130.00
4" DUCTILE IRON GATE VALVE	EA	3	$	1,024.00	$	3,072.00
4" DUCTILE IRON PRESSURE REDUCING VALVE	EA	1	$	7,215.00	$	7,215.00
DUCTILE IRON FITTINGS	TON	0.16	$	25,300.00	$	4,048.00
7'-6" X 4'-6" X 5'-0" PRE-CAST CONCRETE VALVE VAULT	LS	1	$	39,000.00	$	39,000.00
72" x 48" VALVE VAULT ACCESS DOOR	EA	1	$	19,500.00	$	19,500.00
CONNECTION TO EXISTING 6" WATERLINE	EA	3	$	4,600.00	$	13,800.00
2" COMBINATION AIR/VACUUM VALVE ASSEMBLY	EA	1	$	9,200.00	$	9,200.00
REVEGETATION (HYDROMULCH-TOPSOIL-WATERING)	SY	4	$	98.00	$	392.00
ASPHALT PAVEMENT REPAIR	SY	10	$	575.00	$	5,750.00
TRENCH SAFETY SYSTEM						
DEVELOP SAFETY PLAN	LS	1	$	1,150.00	$	1,150.00
TRENCH SAFETY	LF	15	$	230.00	$	3,450.00
CONSTRUCTION SUBTOTAL					$	155,932.00
DESIGN					$	26,509.00
CONSTRUCTION ADMINISTRATION					$	4,678.00
CONTINGENCY					$	46,780.00
TOTAL					**$**	**233,899.00**

7.4.2 6-INCH" WATERLINE FROM NICKLAUS DRIVE TO CHAMPIONS CIRCLE

The second proposed distribution improvement is to install a 6-inch waterline from Nicklaus Drive at the intersection of Palmer Drive to Champions Circle. This improvement would increase fire flow to an acceptable level for the townhomes within the Demarett Drive and Palmer Drive neighborhood. With the waterline alignment crossing the golf course, a permanent waterline/utility easement and temporary construction easement will have to be established. The Engineer's OPCC for 6-Inch Waterline from Nicklaus Drive to Champions Circle is provided below in Table 7-5.

TABLE 7-5. 6-INCH WATERLINE FROM NICKLAUS DRIVE TO CHAMPIONS CIRCLE OPCC

ITEM DESCRIPTION	UNIT	QTY	UNIT PRICE	TOTAL COST
MOBILIZATION, BONDING, INSURANCE	LS	1	$ 132,521.00	$ 132,521.00
EROSION AND SEDIMENTATION CONTROL PLAN AND IMPLEMENTATION				
PERMITS	LS	1	$ 1,150.00	$ 1,150.00
DEVELOP SWPPP PLAN	LS	1	$ 2,300.00	$ 2,300.00
TREE PROTECTION	LF	200	$ 6.00	$ 1,200.00
SILT FENCE	LF	1,555	$ 6.00	$ 9,330.00
STABILIZED CONSTRUCTION ENTRANCE	EA	1	$ 1,950.00	$ 1,950.00
MAINTAIN SWPPP	MO	7	$ 230.00	$ 1,610.00
WATERLINE IMPROVEMENTS				
CLEAR AND GRUBB	SF	27,600	$ 2.00	$ 55,200.00
6" PVC WATERLINE	LF	1,555	$ 127.00	$ 197,485.00
6" DUCTILE IRON GATE VALVE W/ VALVE BOX & COVER	EA	6	$ 3,450.00	$ 20,700.00
DUCTILE IRON FITTINGS	TON	0.23	$ 25,300.00	$ 5,819.00
CONNECTION TO EXISTING 6" WATERLINE	EA	2	$ 4,600.00	$ 9,200.00
2" COMBINATION AIR/VACUUM VALVE ASSEMBLY	EA	1	$ 9,200.00	$ 9,200.00
REVEGETATION (HYDROMULCH-TOPSOIL-WATERING)	SY	3,067	$ 6.00	$ 18,402.00
ASPHALT PAVEMENT REPAIR	SY	117	$ 144.00	$ 16,848.00
TRENCH SAFETY SYSTEM				
DEVELOP SAFETY PLAN	LS	1	$ 1,150.00	$ 1,150.00
TRENCH SAFETY	LF	1,555	$ 58.00	$ 90,190.00
CONSTRUCTION SUBTOTAL				$ 574,255.00
DESIGN				$ 97,624.00
CONSTRUCTION ADMINISTRATION				$ 17,228.00
CONTINGENCY				$ 172,277.00
TOTAL				**$ 861,384.00**

7.4.3 2-INCH WATERLINE FOR LOTS 571, 572, AND 573

The third proposed distribution improvement is to install a 2-inch waterline from Lot 571 at the intersection of Venture Drive and Summit Ridge Drive to the end of Lot 573. This improvement would transfer these 3 lots from the Lower Pressure Plane into the Upper Pressure Plane. The Engineer's OPCC for 2-Inch Waterline for Lots 571, 572, and 573 is provided below in Table 7-6. Developer/Builder incurring the cost for this improvement shall be considered.

TABLE 7-6. 2-INCH WATERLINE FOR LOTS 571, 572, & 573 OPCC

ITEM DESCRIPTION	UNIT	QTY	UNIT PRICE	TOTAL COST
MOBILIZATION, BONDING, INSURANCE	LS	1	$ 14,494.00	$ 14,494.00
EROSION AND SEDIMENTATION CONTROL PLAN AND IMPLEMENTATION				
PERMITS	LS	1	$ 1,150.00	$ 1,150.00
DEVELOP SWPPP PLAN	LS	1	$ 2,300.00	$ 2,300.00
SILT FENCE	LF	370	$ 6.00	$ 2,220.00
STABILIZED CONSTRUCTION ENTRANCE	EA	1	$ 1,950.00	$ 1,950.00
MAINTAIN SWPPP	MO	2	$ 230.00	$ 460.00
WATERLINE IMPROVEMENTS				
CLEAR AND GRUBB	SF	640	$ 2.00	$ 1,280.00
2" HDPE TUBING & METER CONNECTIONS	LF	320	$ 75.00	$ 24,000.00
HORIZONTAL DIRECTIONAL DRILL	LF	50	$ 259.00	$ 12,950.00
6" SERVICE SADDLE WITH 2" TAP	EA	1	$ 303.00	$ 303.00
2" CORPORATION STOP	EA	1	$ 1,268.00	$ 1,268.00
REVEGETATION (HYDROMULCH-TOPSOIL-WATERING)	SY	72	$ 6.00	$ 432.00
CONSTRUCTION SUBTOTAL				$ 62,807.00
DESIGN				$ 10,678.00
CONTINGENCY				$ 18,843.00
TOTAL				**$ 92,328.00**

7.4.4 2-INCH WATERLINE REPLACEMENTS

The fourth proposed distribution improvement is replacing 2-inch waterlines with 8-inch waterlines. The existing 2-inch waterlines are located at the following: Lakehead Circle; and Lakeland Circle. These two (2) locations are along cul-de-sacs and are situated within the Lower Pressure Plane. In the past, there were instances of residents experiencing low water pressure at the following addresses: 507-513 Lakeland Circle, and 18500-18505 Lakehead Circle. As discussed above in Section 7.3.1, the new Standpipe will establish a minimum pressure of 45-psi for the Lower Pressure Plane at the customer's meter box. However, when the time and funding is appropriate, replacing the existing 2-inch waterlines with 8-inch waterlines at both locations/cul-de-sacs will provide additional capacity and fire flow, and improve the overall system. The Engineer's OPCC for 2-Inch Waterline Replacements is provided below in Table 7-7.

TABLE 7-7. 2-INCH WATERLINE REPLACEMENTS OPCC

ITEM DESCRIPTION	UNIT	QTY	UNIT PRICE	TOTAL COST
MOBILIZATION, BONDING, INSURANCE	LS	1	$ 116,129.00	$ 116,129.00
EROSION AND SEDIMENTATION CONTROL PLAN AND IMPLEMENTATION				
PERMITS	LS	1	$ 1,150.00	$ 1,150.00
DEVELOP SWPPP PLAN	LS	1	$ 2,300.00	$ 2,300.00
SILT FENCE	LF	1,000	$ 6.00	$ 6,000.00
STABILIZED CONSTRUCTION ENTRANCE	EA	1	$ 1,950.00	$ 1,950.00
MAINTAIN SWPPP	MO	7	$ 230.00	$ 1,610.00
WATERLINE IMPROVEMENTS				
LAKELAND CIRCLE AND LAKEHEAD CIRCLE				
ABANDON 2" PVC WATERLINE	LF	915	$ 12.00	$ 10,980.00
ABANDON 6" PVC WATERLINE	LF	85	$ 12.00	$ 1,020.00
8" PVC WATERLINE	LF	1,000	$ 156.00	$ 156,000.00
DUCTILE IRON FITTINGS	TON	0.46	$ 25,300.00	$ 11,638.00
8" SERVICE SADDLE WITH 2" TAP	EA	8	$ 332.00	$ 2,656.00
2" CORPORATION STOP	EA	8	$ 1,268.00	$ 10,144.00
8" DUCTILE IRON GATE VALVE W/ VALVE BOX & COVER	EA	3	$ 6,900.00	$ 20,700.00
CONNECTION TO EXISTING 8" WATERLINE	EA	1	$ 5,750.00	$ 5,750.00
PAVEMENT REPAIR	SY	667	$ 144.00	$ 96,048.00
TRENCH SAFETY SYSTEM				
DEVELOP SAFETY PLAN	LS	1	$ 1,150.00	$ 1,150.00
TRENCH SAFETY	LF	1,000	$ 58.00	$ 58,000.00
CONSTRUCTION SUBTOTAL				$ 503,225.00
DESIGN				$ 85,549.00
CONSTRUCTION ADMINISTRATION				$ 15,097.00
CONTINGENCY				$ 150,968.00
TOTAL				**$ 754,839.00**

7.4.5 6-INCH WATERLINES FOR LUE REALLOCATION

The fifth proposed distribution improvement is to install 6-inch waterlines along Valley Hill Drive between Southwind Road and Venture Drive, and along Valley Hill Lane between Southwind Road and Valley Hill Drive. This improvement would reallocate 35 LUEs along Southwind Road from the Upper Pressure Plane to the Lower Pressure Plane. The Augusta EST provides a maximum 500 LUEs to the Upper Pressure Plane. When Point Venture reaches full build-out, approximately 532 LUEs are projected to be within the Upper Pressure Plane, which will exceed LUE capacity. Installing the proposed 6-inch waterlines, including pressure plane isolation gate valves, will maintain TCEQ storage requirements and prevent exceeding LUE capacity within the Upper Pressure Plane. The highest elevation within the reallocation of the LUEs is 775.00 ft AMSL. Using the Lower Pressure Plane HGL of 899.00 ft AMSL yields a pressure of 54 psi. Also, an additional 9 LUEs will be reallocated to the Lower Pressure Plane by closing the gate valve at the top of hill off Mariners Point & Peckham Drive and opening the valve at the bottom of the hill off Mariners Point & Lakefront Drive. The Engineer's OPCC for 6-Inch Waterlines for LUE Reallocation is provided below in Table 7-8.

TABLE 7-8. 6-INCH WATERLINES FOR LUE REALLOCATION OPCC

ITEM DESCRIPTION	UNIT	QTY	UNIT PRICE	TOTAL COST
MOBILIZATION, BONDING, INSURANCE	LS	1	$ 76,972.00	$ 76,972.00
EROSION AND SEDIMENTATION CONTROL PLAN AND IMPLEMENTATION				
PERMITS	LS	1	$ 1,150.00	$ 1,150.00
DEVELOP SWPPP PLAN	LS	1	$ 2,300.00	$ 2,300.00
SILT FENCE	LF	580	$ 6.00	$ 3,480.00
STABILIZED CONSTRUCTION ENTRANCE	EA	1	$ 1,950.00	$ 1,950.00
MAINTAIN SWPPP	MO	4	$ 230.00	$ 920.00
WATERLINE IMPROVEMENTS				
6" PVC WATERLINE	LF	580	$ 127.00	$ 73,660.00
8" DUCTILE IRON GATE VALVE W/ VALVE BOX & COVER	EA	3	$ 6,900.00	$ 20,700.00
6" DUCTILE IRON GATE VALVE W/ VALVE BOX & COVER	EA	10	$ 3,450.00	$ 34,500.00
DUCTILE IRON FITTINGS	TON	0.31	$ 25,300.00	$ 7,843.00
CONNECTION TO EXISTING 8" WATERLINE	EA	1	$ 5,750.00	$ 5,750.00
CONNECTION TO EXISTING 6" WATERLINE	EA	3	$ 4,600.00	$ 13,800.00
ASPHALT PAVEMENT REPAIR	SY	387	$ 144.00	$ 55,728.00
TRENCH SAFETY SYSTEM				
DEVELOP SAFETY PLAN	LS	1	$ 1,150.00	$ 1,150.00
TRENCH SAFETY	LF	580	$ 58.00	$ 33,640.00
CONSTRUCTION SUBTOTAL				$ 333,543.00
DESIGN				$ 56,703.00
CONSTRUCTION ADMINISTRATION				$ 10,007.00
CONTINGENCY				$ 100,063.00
TOTAL				**$ 500,316.00**

A summary of all the proposed improvements, cost, and priority listing is provided below in Table 7-9.

TABLE 7-9. SUMMARY OF PROPOSED IMPROVEMENTS

PRIORITY	PROPOSED IMPROVEMENT	COST
1	***AUGUSTA STANDPIPE REPLACEMENT***	
	Construction	$ 4,799,563.00
	Design	$ 815,926.00
	Geotechnical	$ 71,994.00
	Construction Materials Testing	$ 95,992.00
	Construction Administration	$ 143,987.00
	Tank Welding & Coating Inspection	$ 167,985.00
	Contingency	$ 1,439,869.00
	Total	$ 7,535,316.00
2	***AUGUSTA PUMP STATION REHABILITATION***	
	Construction	$ 1,103,915.00
	Design	$ 187,666.00
	Construction Administration	$ 33,118.00
	Contingency	$ 331,175.00
	Total	$ 1,655,874.00
3	***6-INCH WATERLINE FROM NICKLAUS DR. TO CHAMPIONS CIR.***	
	Construction	$ 574,255.00
	Design	$ 97,624.00
	Construction Administration	$ 17,228.00
	Contingency	$ 172,277.00
	Total	$ 861,384.00
4	***PRV ASSEMBLY***	
	Construction	$ 155,932.00
	Design	$ 26,509.00
	Construction Administration	$ 4,678.00
	Contingency	$ 46,780.00
	Total	$ 233,899.00
5	***AUGUSTA EST REHABILITATION***	
	Construction	$ 625,915.00
	Design	$ 106,406.00
	Construction Administration	$ 18,778.00
	Tank Welding & Coating Inspection	$ 21,908.00
	Contingency	$ 187,775.00
	Total	$ 960,782.00
6	***6-INCH WATERLINES FOR LUE REALLOCATION***	
	Construction	$ 333,543.00
	Design	$ 56,703.00
	Construction Administration	$ 10,007.00
	Contingency	$ 100,063.00
	Total	$ 500,316.00

PRIORITY	PROPOSED IMPROVEMENT	COST
7	***2-INCH WATERLINE REPLACEMENTS***	
	Construction	$ 503,225.00
	Design	$ 85,549.00
	Construction Administration	$ 15,097.00
	Contingency	$ 150,968.00
	Total	$ 754,839.00
	SUMMATION TOTAL	**$ 12,502,410.00**

A proposed water system map showing the locations of the proposed improvements is provided in Exhibit K. Performance and adequacy of the WCID Point Venture water system with the proposed improvements under future conditions was analyzed for the four demand scenarios to ensure that TCEQ and fire flow requirements were met. The results of the fire flow scenario are provided in Exhibit L. The minimum foundation floor slab elevation in Point Venture is 724.00 ft AMSL, which is at the minimum required 2-feet above the 100-year floodplain elevation per Travis County Code Chapter 464; the 100-year floodplain elevation in Point Venture is 722.00 ft AMSL. At the Augusta Standpipe overflow elevation of 926.50 ft AMSL, the absolute maximum pressure during no demand that would be experienced within the Lower Pressure Plane is 87.7 psi.

8.0 CONCLUSION

Hydraulic modeling and evaluation of WCID Point Venture's water system in comparison to TCEQ minimum requirements for public water systems indicates there are several deficiencies in the system. These include insufficient storage capacity at the existing Augusta Standpipe, insufficient pumping capacity for the Augusta Pump Station, and several areas in the system that do not meet TCEQ minimum pressures. Projected growth in WCID Point Venture's service area is expected to increase the number of LUEs in the water system to 1,190 LUEs in the next 7-years at full build-out using a 29 LUE per year growth forecast. The existing system deficiencies and future growth in the system will require several improvements to be implemented.

Proposed improvements have been recommended to address the immediate concerns and deficiencies in the water system. These include replacing the Augusta Standpipe and rehabilitating the Augusta Pump Station. Additional projects were identified to address aging infrastructure, fire flow availability, and operation issues which included: rehabilitating the Augusta EST; installing a 6-inch waterline from Nicklaus Drive to Champions Circle; installing a PRV assembly; installing a 2-inch waterline to transfer 3 lots to the Upper Pressure Plane; replacing 2-inch waterlines with 8-inch waterlines at Lakeland Circle and Lakehead Circle; and installing 6-inch waterlines along Valley Hill Drive and Valley Hill Lane to reallocate 35 LUEs to the Lower Pressure Plane. If the recommended improvements are implemented, WCID Point Venture's water system will adhere to the TCEQ minimum requirements and address operational and aging infrastructure issues at build-out.

A complete listing of all recommended improvements are presented in Table 7-9. The summary table includes a general prioritization and cost of the projects. Tables 7-1 through 7-8 depict a detailed itemized cost estimate for each improvement project. It is to be noted that scope and funding will be dependent upon final project costs of the proposed wastewater treatment plant if WCID Point Venture elects to allocate 2020 Bond Issuance funding to any of the proposed improvements.

EXHIBIT A

SERVICE AREA

C:\USERS\KPETERSEN\TRIHYDRO\TRAVIS COUNTY W.C.&I.D. POINT VENTURE - DOCUMENTS\CADD\WATERSYSTEMANALYSIS\SINGLEFIGURES\701-WCIDPV_SERVICEAREA

EXPLANATION

- VILLAGE OF POINT VENTURE FULL PURPOSE JURISDICTION
- WCID PV SERVICE BOUNDARY
- EXISTING PARCELS
- LAKE TRAVIS

N

0 600'

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EXHIBIT A

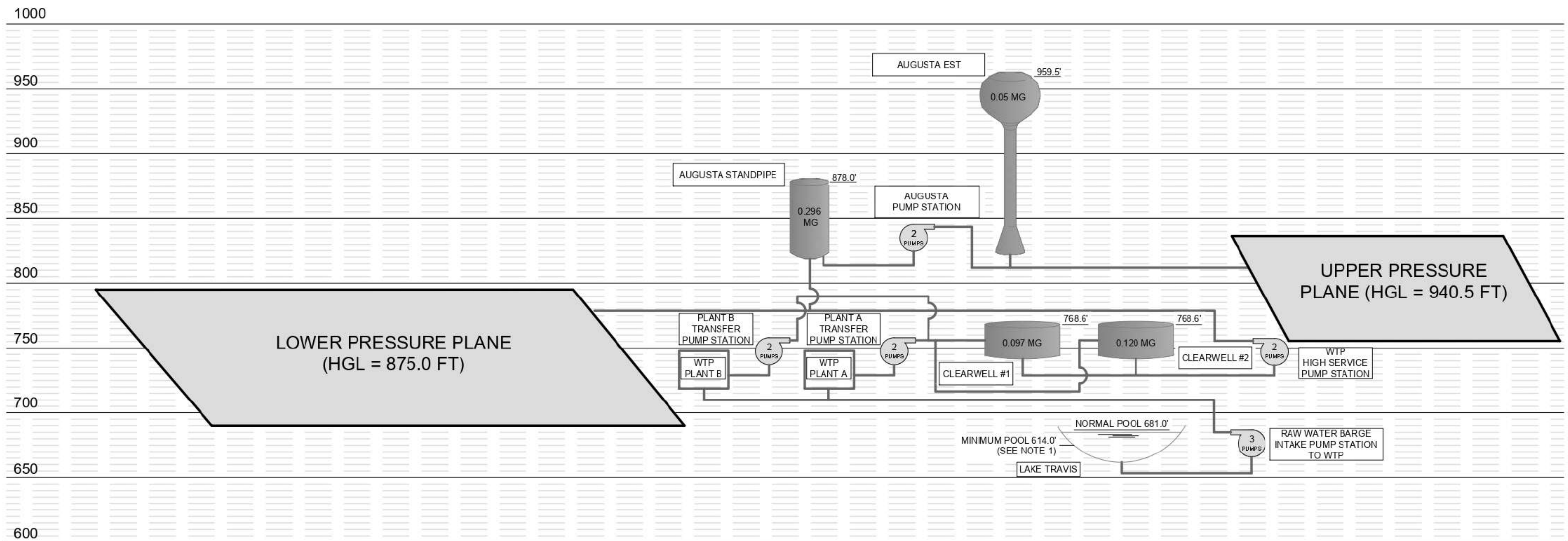
SERVICE AREA

WATER SYSTEM ANALYSIS
TRAVIS COUNTY W.C.I.D. POINT VENTURE
POINT VENTURE, TEXAS

Drawn By: KP	Checked By: DV	Scale: 1" = 600'	Date: 1/30/2023

EXHIBIT B

EXISTING PRESSURE PLANE SCHEMATIC



FACILITY NAME	INSIDE DIAMETER (FT)	FINISH FLOOR ELEVATION (MSL)	OVERFLOW ELEVATION (MSL)	HEIGHT (FT)	CAPACITY (GAL)	FUNCTION
CLEARWELL #1	26.75	745.60	768.60	23.00	96,687	CLEARWELL STORAGE FEEDING HIGH SERVICE PUMPS
CLEARWELL #2	29.75	745.60	768.60	23.00	119,590	CLEARWELL STORAGE FEEDING HIGH SERVICE PUMPS
AUGUSTA STANDPIPE	30.00	822.00	878.00	56.00	296,089	GROUND AND ELEVATED STORAGE FOR LOWER PRESSURE PLANE
AUGUSTA EST	24.00	824.50	959.50	135.00	50,000	ELEVATED STORAGE FOR UPPER PRESSURE PLANE

ABBREVIATIONS

EST	ELEVATED STORAGE TANK
FT	FEET
GAL	GALLON
HGL	HYDRAULIC GRADE LINE (FT)
MG	MILLION GALLON
MSL	MEAN SEA LEVEL (FT)
WTP	WATER TREATMENT PLANT

NOTES:

1. THE 614.0' MINIMUM POOL IS THE ELEVATION TO MEET FIRM PUMP CAPACITY OF THE RAW WATER BARGE INTAKE PUMP STATION.

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EXHIBIT B

EXISTING PRESSURE PLANE SCHEMATIC

WATER SYSTEM ANALYSIS
TRAVIS COUNTY W.C.& I.D. POINT VENTURE
POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: N.T.S.	Date: 6/2/2023

EXHIBIT C

HISTORICAL WATER USAGE DATA

EXHIBIT C. HISTORICAL WATER USAGE DATA

Active Meters (Raw Data)					
Month	**2018**	**2019**	**2020**	**2021**	**2022**
January	810	852	887	887	933
February	823	854	891	888	935
March	825	854	891	925	941
April	847	867	898	898	949
May	835	865	899	914	949
June	839	867	910	919	952
July	842	871	903	920	954
August	845	879	907	921	958
September	843	870	907	925	965
October	847	874	918	928	971
November	848	876	912	931	971
December	855	881	914	931	970
Yearly Avg. Meters	**838**	**868**	**903**	**916**	**954**

Total Water Usage (Gallons Per Month) (Raw Data)					
Month	**2018**	**2019**	**2020**	**2021**	**2022**
January	3,245,336	2,485,268	3,146,494	3,689,000	3,718,000
February	3,227,939	2,301,278	2,959,091	4,652,000	3,875,000
March	3,333,076	2,577,195	2,947,530	3,259,000	3,844,000
April	5,149,380	4,605,194	3,918,365	4,577,000	5,758,000
May	4,892,022	3,981,996	5,705,479	5,270,000	5,254,000
June	6,960,654	5,510,575	6,235,736	9,645,000	6,899,000
July	7,056,084	5,143,178	7,587,214	6,243,000	7,644,000
August	7,479,998	7,626,608	8,944,909	7,512,000	7,682,000
September	5,556,491	7,550,846	6,107,498	7,019,000	6,457,000
October	3,777,077	5,651,250	5,842,794	5,879,000	6,520,000
November	3,418,515	3,541,076	5,694,482	4,789,000	5,446,000
December	2,659,965	3,557,368	3,926,036	3,780,000	3,577,000
Total Yearly Water Usage (MG)	**56.757**	**54.532**	**63.016**	**66.314**	**66.674**

Meter Size	**Qty**	**LUEs/Meter**	**LUEs**	
5/8"x3/4" are 1 LUE each	25	1.0	25.0	*3x POA (irrigation); 23x Townhomes (irrigation)*
1"	4.0	2.5	10.0	*2x docks (irrigation); 2x townhomes (irrigation)*
-	-	-	**35.0**	

LUEs[1]					
Month	**2018**	**2019**	**2020**	**2021**	**2022**
January	845	887	922	922	968
February	858	889	926	923	970
March	860	889	926	960	976
April	882	902	933	933	984
May	870	900	934	949	984
June	874	902	945	954	987
July	877	906	938	955	989
August	880	914	942	956	993
September	878	905	942	960	1000
October	882	909	953	963	1006
November	883	911	947	966	1006
December	890	916	949	966	1005
Yearly Avg. LUEs	**873**	**903**	**938**	**951**	**989**

(1) Incorporates the Irrigation Meters and Floating Restaurant Meter that are not at a 1:1 LUE to Meter ratio.

(2) Green highlights correspond to the maximum daily water usage and associated LUEs for that month.

EXHIBIT C. HISTORICAL WATER USAGE DATA

August 2022 Daily Data

Day	Treated Water Pumpage (MGD)	Treated Water Pumpage (gpm)	Treated Water Pumpage (gpm/LUE)
1	0.343	238.1944444	0.240
2	0.288	200	0.201
3	0.255	177.0833333	0.178
4	0.334	231.9444444	0.234
5	0.313	217.3611111	0.219
6	0.295	204.8611111	0.206
7	0.316	219.4444444	0.221
8	0.375	260.4166667	0.262
9	0.245	170.1388889	0.171
10	0.298	206.9444444	0.208
11	0.32	222.2222222	0.224
12	0.274	190.2777778	0.192
13	0.274	190.2777778	0.192
14	0.375	260.4166667	0.262
15 (1)	0.386	268.0555556	0.270
16	0.221	153.4722222	0.155
17	0.313	217.3611111	0.219
18	0.268	186.1111111	0.187
19	0.209	145.1388889	0.146
20	0.11	76.38888889	0.077
21	0.31	215.2777778	0.217
22	0.27	187.5	0.189
23	0.145	100.6944444	0.101
24	0.182	126.3888889	0.127
25	0.222	154.1666667	0.155
26	0.233	161.8055556	0.163
27	0.194	134.7222222	0.136
28	0.214	148.6111111	0.150
29	0.276	191.6666667	0.193
30	0.175	121.5277778	0.122
31	0.173	120.1388889	0.121
Avg.			**0.185 gpm/LUE**
Min.			**0.077 gpm/LUE**
Max.			**0.270 gpm/LUE**

(1) The highlighted data corresponds to the maximum water usage in a day in the month of August.

Hydraulic Pattern Determination

Low Demand Hours (Minimum/Average)	0.416
High Demand Hours (Maximum/Average)	1.458

Average Daily Demand	**267 gpd/LUE**	**0.185 gpm/LUE**	**184 LUEs**
Maximum Daily Demand	389 gpd/LUE	0.270 gpm/LUE	268 LUEs
Maximum Daily Demand * 2.4 (TCEQ Multiplier)	**933 gpd/LUE**	**0.648 gpm/LUE**	**643 LUEs**
Peak Hour Demand (Max. Daily Demand * 1.25)	1,166 gpd/LUE	0.810 gpm/LUE	804 LUEs
Peak Hour Demand Adjusted for High Demand Hour	**1,701 gpd/LUE**	**1.181 gpm/LUE**	**1,173 LUEs**

EXHIBIT D

EXISTING WATER SYSTEM MAP

EXPLANATION

- WATER TREATMENT PLANT
- RAW WATER INTAKE
- WATER TANK
- FIRE HYDRANT
- GATE VALVE
- PRESSURE PLANE ISOLATION GATE VALVE

WATER LINES
- MAIN PIPELINE - 8"
- MAIN PIPELINE - 6"
- MAIN PIPELINE - 4"
- MAIN PIPELINE - 2"

RAW WATER LINES
- RAW WATER LINE - 8"
- RAW WATER LINE - 6"

- CCN
- TRAVIS COUNTY LAND PARCEL - SITE
- UPPER PRESSURE PLANE
- LOWER PRESSURE PLANE

ABBREVIATIONS

CCN - CERTIFICATE OF CONVENIENCE AND NECESSITY.
ISO - ISOLATION
MLV - MAIN LINE VALVE

N

0 330 660'

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EXHIBIT D

EXISTING WATER SYSTEM MAP

WATER SYSTEM ANALYSIS
TRAVIS COUNTY W.C.&I.D. POINT VENTURE
POINT VENTURE, TEXAS

Drawn By: DH	Checked By: KP	Scale: 1 " = 330 '	Date: 6/1/23	File: ModelExhibitD.mxd

EXHIBIT E

EXISTING DEMAND NODE LOCATION MAP

EXPLANATION

UPPER PRESSURE PLANE

LOWER PRESSURE PLANE

ABBREVIATIONS:

LD - LOWER PRESSURE PLANE DEMAND NODE
UD - UPPER PRESSURE PLANE DEMAND NODE

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EXHIBIT E

EXISTING DEMAND NODE LOCATION MAP

WATER SYSTEM ANALYSIS
TRAVIS COUNTY W.C.& I.D. POINT VENTURE
POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: 1" = 300'	Date: 5/31/2023

EXHIBIT F

FIRE FLOW TEST DATA

EXHIBIT F. FIRE FLOW TEST DATA

Hydrant							High Service Pump #1	High Service Pump #2	Time	
Test	Flow	Clearwell GST Levels	EST	EST Level	Standpipe	Standpipe Level			On	Off
1	2	20.6/20.2	128.3	952.8	49.5	873			10:25	10:29
		20.7/20.2	128.3	952.8	49.3	872.8				
4	3	20.8/20.3	128.2	952.7	49.3	872.8			10:34	10:36
		20.9/20.4	128.1	952.6	49.1	872.6				
69	7	21/20.5	127.3	951.8	47.8	871.3	On 10:48		10:43	10:45
		21/20.5	128.9	953.4	47.8	871.3				
21	20	20.8/20.3	127.5	952	47.2	870.7		On 10:50	10:55	10:57
		20.7/20.2	129.9	954.4	47.5	871				
28	30	20.1/19.6	130.3	954.8	47.4	870.9	Off 11:16		11:12	11:13
		20.1/19.5	130.6	955.1	47.7	871.2				
52	50	19.4/18.8	129.3	953.8	49.5	873			11:31	11:32
		19.3/18.7	129.3	953.8	49.5	873				
56	57	18.9/18.3	129.1	953.6	50	873.5		Off 11:44	11:42	11:47
		18.9/18.3	129.1	953.6	49.7	873.2				
46	47	18.9/18.3	129.1	953.6	49.6	873.1			11:56	11:58
		18.9/18.3	129.1	953.6	49.4	872.9				
44	45	18.9/18.3	128.8	953.3	49.3	872.8			12:05	12:07
		18.9/18.3	128.8	953.3	49.1	872.6				
39	40	18.9/18.3	128.7	953.2	49	872.5			12:15	12:16
		18.9/18.3	128.7	953.2	48.9	872.4				

EXHIBIT F. FIRE FLOW TEST DATA

T/F	ID No.	Location	Gallons Flowed	Pitot (p)	Available Fire Flow @ 20 psi	Total Gallons Flowing	Static (S)	Residual (R)	Year	Mfg.	Info
Test	1	210 Venture Blvd N	964		1363	964	58	38	1971	Mueller	Top oil leaking
Flow	2	18401 Venture Dr		33		0			2017	Kennedy K81D	
Test	4	18501 Venture Dr	934		934	934	35	20	2017	Kennedy K81D	
Flow	3	18405 Venture Dr		31		0			2017	Kennedy K81D	
Test	69	403 Valley Hill Dr	919		1442	919	73	50	1973	Mueller	
Flow	7	309 Summit Ridge Dr		30		0			1973	Mueller	
Test	21	303 Firestone Cir	934		1179	934	80	41	2017	Kennedy K81D	
Flow	20	18629 Champions Cir		31		0			1972	Clow Eddy	Stem o-ring leak
Test	28	18813 Venture Dr	919		1288	919	63	40	1972	Clow Eddy	Stem o-ring leak
Flow	30	18709 Venture Dr		30		0			1972	Clow Eddy	Stem o-ring leak
Test	31	405 Cascade Cir	0		0	0	68		2017	Kennedy K81D	
Flow	32	501 Lakeshore Cir		0		0			Unknown		Bagged out of service
Test	56	19000 Helmway Cir	581		545	581	60	15	2016	Kennedy K81D	
Flow	57	501 Deckhouse Dr		12		0			1971	Mueller	Top oil leaking
Test	52	518 Demarett Dr	856		-	856	60	60	1972	Clow Eddy	No drop in pressure
Flow	50	19031 Venture Dr		26		0			2017	Kennedy K81D	main closed btwn??
Test	46	503 Lakeland Cir	692		634	692	43	16	1969	Mueller	
Flow	47	19047 Venture Dr		17		0			2016	Kennedy K81D	
Test	44	18400 Lakeland Dr	692		712	692	59	22	1969	Mueller	
Flow	45	428 Venture Blvd S		17		0			2017	Kennedy K81D	
Test	39	18616 Lakeland Dr	650		623	650	57	17	1969	Mueller	
Flow	40	18513 Lakeland Dr		15		0			1969	Mueller	

EXHIBIT F. FIRE FLOW TEST DATA

T/F	ID No.	Location	Fire Flow Available @ 20 psi (gpm) (Observed)	Fire Flow Available @ 20 psi (gpm) (Modeled)	Percent Diffence of Fire Flow Available	Residual Pressure (psi) (Observed)	Residual Pressure (psi) (Modeled)	Percent Diffence of Residual Pressure
Test	1	210 Venture Blvd N	1363	1586	16.36%	38	42	10.53%
Flow	2	18401 Venture Dr						
Test	4	18501 Venture Dr	934	1871	100.32%	20	32	60.00%
Flow	3	18405 Venture Dr						
Test	69	403 Valley Hill Dr	1442	1450	0.55%	50	47	6.00%
Flow	7	309 Summit Ridge Dr						
Test	21	303 Firestone Cir	1179	1229	4.24%	41	40	2.44%
Flow	20	18629 Champions Cir						
Test	28	18813 Venture Dr	1288	1423	10.48%	40	41	2.50%
Flow	30	18709 Venture Dr						
Test	31	405 Cascade Cir	0					
Flow	32	501 Lakeshore Cir						
Test	56	19000 Helmway Cir	545	1284	135.60%	15	29	93.33%
Flow	57	501 Deckhouse Dr						
Test	52	518 Demarett Dr	-			60		
Flow	50	19031 Venture Dr						
Test	46	503 Lakeland Cir	634	1697	167.67%	16	33	106.25%
Flow	47	19047 Venture Dr						
Test	44	18400 Lakeland Dr	712	2398	236.80%	22	55	150.00%
Flow	45	428 Venture Blvd S						
Test	39	18616 Lakeland Dr	623	1431	129.70%	17	37	117.65%
Flow	40	18513 Lakeland Dr						
				Average:	**89.08%**		**Average:**	**60.97%**

(1) Because the fire flow testing had values for Available Fire Flow and Residual Pressure less than 1,000 gpm and 20 psi respectively, these test locations were ignored in calibration calculations.

EXHIBIT F. FIRE FLOW TEST DATA

Hydrant Tests Used in Model Calibration

T/F	ID No.	Location	Fire Flow Available @ 20 psi (gpm) (Observed)	Fire Flow Available @ 20 psi (gpm) (Modeled)	Percent Diffence of Fire Flow Available	Residual Pressure (psi) (Observed)	Residual Pressure (psi) (Modeled)	Percent Diffence of Residual Pressure
Test	1	210 Venture Blvd N	1363	1586	16.36%	38	42	10.53%
Flow	2	18401 Venture Dr						
Test	69	403 Valley Hill Dr	1442	1450	0.55%	50	47	6.00%
Flow	7	309 Summit Ridge Dr						
Test	21	303 Firestone Cir	1179	1229	4.24%	41	40	2.44%
Flow	20	18629 Champions Cir						
Test	28	18813 Venture Dr	1288	1423	10.48%	40	41	2.50%
Flow	30	18709 Venture Dr						
				Average:	**7.91%**		**Average:**	**5.37%**

EXHIBIT G-1

EXISTING CONDITIONS AVERAGE DAY WATER MODEL RESULTS MAP

EXPLANATION

- PRESSURE < 35 PSI (red)
- PRESSURE > 85 PSI (blue)

ABBREVIATIONS:

J - JUNCTION NODE
LD - LOWER PRESSURE PLANE DEMAND NODE
MLV - MAIN LINE VALVE
UD - UPPER PRESSURE PLANE DEMAND NODE

N

0 300' 600'

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EXHIBIT G-1

EXISTING CONDITIONS AVERAGE DAY WATER MODEL RESULTS MAP

WATER SYSTEM ANALYSIS
TRAVIS COUNTY W.C.& I.D. POINT VENTURE
POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: 1" = 300'	Date: 5/26/2023

EXHIBIT G-2

EXISTING CONDITIONS MAXIMUM DAY WATER MODEL RESULTS MAP

EXPLANATION

- PRESSURE < 35 PSI
- PRESSURE > 85 PSI

ABBREVIATIONS:

J - JUNCTION NODE
LD - LOWER PRESSURE PLANE DEMAND NODE
MLV - MAIN LINE VALVE
UD - UPPER PRESSURE PLANE DEMAND NODE

N

0 300' 600'

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EXHIBIT G-2

EXISTING CONDITIONS MAXIMUM DAY WATER MODEL RESULTS MAP

WATER SYSTEM ANALYSIS TRAVIS COUNTY W.C.& I.D. POINT VENTURE POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: 1" = 300'	Date: 5/26/2023

EXHIBIT G-3

EXISTING CONDITIONS PEAK HOUR WATER MODEL RESULTS MAP

EXPLANATION

- PRESSURE < 35 PSI
- PRESSURE > 85 PSI

ABBREVIATIONS:

J - JUNCTION NODE
LD - LOWER PRESSURE PLANE DEMAND NODE
MLV - MAIN LINE VALVE
UD - UPPER PRESSURE PLANE DEMAND NODE

N

0 300' 600'

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EXHIBIT G-3

EXISTING CONDITIONS PEAK HOUR WATER MODEL RESULTS MAP

WATER SYSTEM ANALYSIS TRAVIS COUNTY W.C.& I.D. POINT VENTURE POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: 1" = 300'	Date: 5/26/2023

EXHIBIT G-4

EXISTING CONDITIONS FIRE FLOW WATER MODEL RESULTS MAP

NFPA COLOR	NFPA CLASS	AVAILABLE FIRE FLOW AT 20 PSI RESIDUAL PRESSURE	DESCRIPTION
LIGHT BLUE	AA	1,500 GPM AND ABOVE	VERY GOOD FLOWS
GREEN	A	1,000 - 1,499 GPM	GOOD FOR RESIDENTIAL AREAS
ORANGE	B	500 - 999 GPM	MARGINALLY INADEQUATE
RED	C	LESS THAN 500 GPM	INADEQUATE

HYDRANT CLASSIFICATION AND COLOR CODING PER NFPA 291, RECOMMENDED PRACTICE FOR FIRE FLOW TESTING AND MARKING OF HYDRANTS.

MINIMUM REQUIRED AVAILABLE FIRE FLOW FOR THE WATER DISTRIBUTION SYSTEM IS 1,000 GPM.

ABBREVIATIONS:

GPM - GALLONS PER MINUTE
HL - HYDRANT NODE
NFPA - NATIONAL FIRE PROTECTION ASSOCIATION
PSI - POUNDS PER SQUARE INCH

N

0 300' 600'

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EXHIBIT G-4

EXISTING CONDITIONS FIRE FLOW WATER MODEL RESULTS MAP

WATER SYSTEM ANALYSIS TRAVIS COUNTY W.C.& I.D. POINT VENTURE POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: 1" = 300'	Date: 5/31/2023

EXHIBIT H-1

FUTURE CONDITIONS AVERAGE DAY WATER MODEL RESULTS MAP

EXPLANATION

PRESSURE < 35 PSI

PRESSURE > 85 PSI

ABBREVIATIONS:

J - JUNCTION NODE
LD - LOWER PRESSURE PLANE DEMAND NODE
MLV - MAIN LINE VALVE
UD - UPPER PRESSURE PLANE DEMAND NODE

N

0 300' 600'

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EXHIBIT H-1

FUTURE CONDITIONS AVERAGE DAY WATER MODEL RESULTS MAP

WATER SYSTEM ANALYSIS
TRAVIS COUNTY W.C.& I.D. POINT VENTURE
POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: 1" = 300'	Date: 6/7/2023

EXHIBIT H-2

FUTURE CONDITIONS MAXIMUM DAY WATER MODEL RESULTS MAP

EXPLANATION

PRESSURE < 35 PSI

PRESSURE > 85 PSI

ABBREVIATIONS:

J - JUNCTION NODE
LD - LOWER PRESSURE PLANE DEMAND NODE
MLV - MAIN LINE VALVE
UD - UPPER PRESSURE PLANE DEMAND NODE

N

0 300' 600'

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EXHIBIT H-2

FUTURE CONDITIONS MAXIMUM DAY WATER MODEL RESULTS MAP

WATER SYSTEM ANALYSIS
TRAVIS COUNTY W.C.& I.D. POINT VENTURE
POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: 1" = 300'	Date: 6/7/2023

EXHIBIT H-3

FUTURE CONDITIONS PEAK HOUR WATER MODEL RESULTS MAP

EXPLANATION

- PRESSURE < 35 PSI
- PRESSURE > 85 PSI

ABBREVIATIONS:

J - JUNCTION NODE
LD - LOWER PRESSURE PLANE DEMAND NODE
MLV - MAIN LINE VALVE
UD - UPPER PRESSURE PLANE DEMAND NODE

N

0 300' 600'

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EXHIBIT H-3

FUTURE CONDITIONS PEAK HOUR WATER MODEL RESULTS MAP

WATER SYSTEM ANALYSIS TRAVIS COUNTY W.C.& I.D. POINT VENTURE POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: 1" = 300'	Date: 6/7/2023

EXHIBIT H-4

FUTURE CONDITIONS FIRE FLOW WATER MODEL RESULTS MAP

NFPA COLOR	NFPA CLASS	AVAILABLE FIRE FLOW AT 20 PSI RESIDUAL PRESSURE	DESCRIPTION
LIGHT BLUE	AA	1,500 GPM AND ABOVE	VERY GOOD FLOWS
GREEN	A	1,000 - 1,499 GPM	GOOD FOR RESIDENTIAL AREAS
ORANGE	B	500 - 999 GPM	MARGINALLY INADEQUATE
RED	C	LESS THAN 500 GPM	INADEQUATE

HYDRANT CLASSIFICATION AND COLOR CODING PER NFPA 291, RECOMMENDED PRACTICE FOR FIRE FLOW TESTING AND MARKING OF HYDRANTS.

MINIMUM REQUIRED AVAILABLE FIRE FLOW FOR THE WATER DISTRIBUTION SYSTEM IS 1,000 GPM.

ABBREVIATIONS:

GPM - GALLONS PER MINUTE
HL - HYDRANT NODE
NFPA - NATIONAL FIRE PROTECTION ASSOCIATION
PSI - POUNDS PER SQUARE INCH

N

0 300' 600'

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EXHIBIT H-4

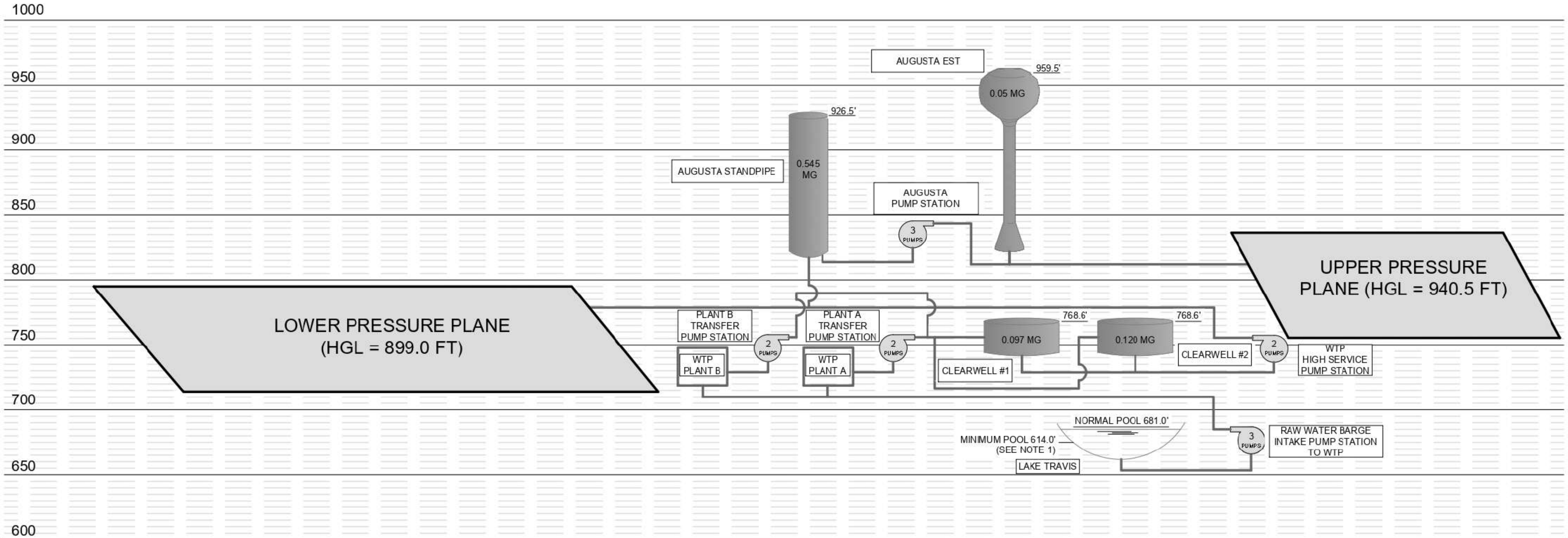
FUTURE CONDITIONS FIRE FLOW WATER MODEL RESULTS MAP

WATER SYSTEM ANALYSIS TRAVIS COUNTY W.C.& I.D. POINT VENTURE POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: 1" = 300'	Date: 6/7/2023

EXHIBIT I

PROPOSED PRESSURE PLANE SCHEMATIC



FACILITY NAME	INSIDE DIAMETER (FT)	FINISH FLOOR ELEVATION (MSL)	OVERFLOW ELEVATION (MSL)	HEIGHT (FT)	CAPACITY (GAL)	FUNCTION
CLEARWELL #1	26.75	745.60	768.60	23.00	96,687	CLEARWELL STORAGE FEEDING HIGH SERVICE PUMPS
CLEARWELL #2	29.75	745.60	768.60	23.00	119,590	CLEARWELL STORAGE FEEDING HIGH SERVICE PUMPS
AUGUSTA STANDPIPE	30.00	822.00	926.50	104.50	545,122	GROUND AND ELEVATED STORAGE FOR LOWER PRESSURE PLANE
AUGUSTA EST	24.00	824.50	959.50	135.00	50,000	ELEVATED STORAGE FOR UPPER PRESSURE PLANE

ABBREVIATIONS

EST	ELEVATED STORAGE TANK
FT	FEET
GAL	GALLON
HGL	HYDRAULIC GRADE LINE (FT)
MG	MILLION GALLON
MSL	MEAN SEA LEVEL (FT)
WTP	WATER TREATMENT PLANT

NOTES:

1. THE 614.0' MINIMUM POOL IS THE ELEVATION TO MEET FIRM PUMP CAPACITY OF THE RAW WATER BARGE INTAKE PUMP STATION.

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EXHIBIT I

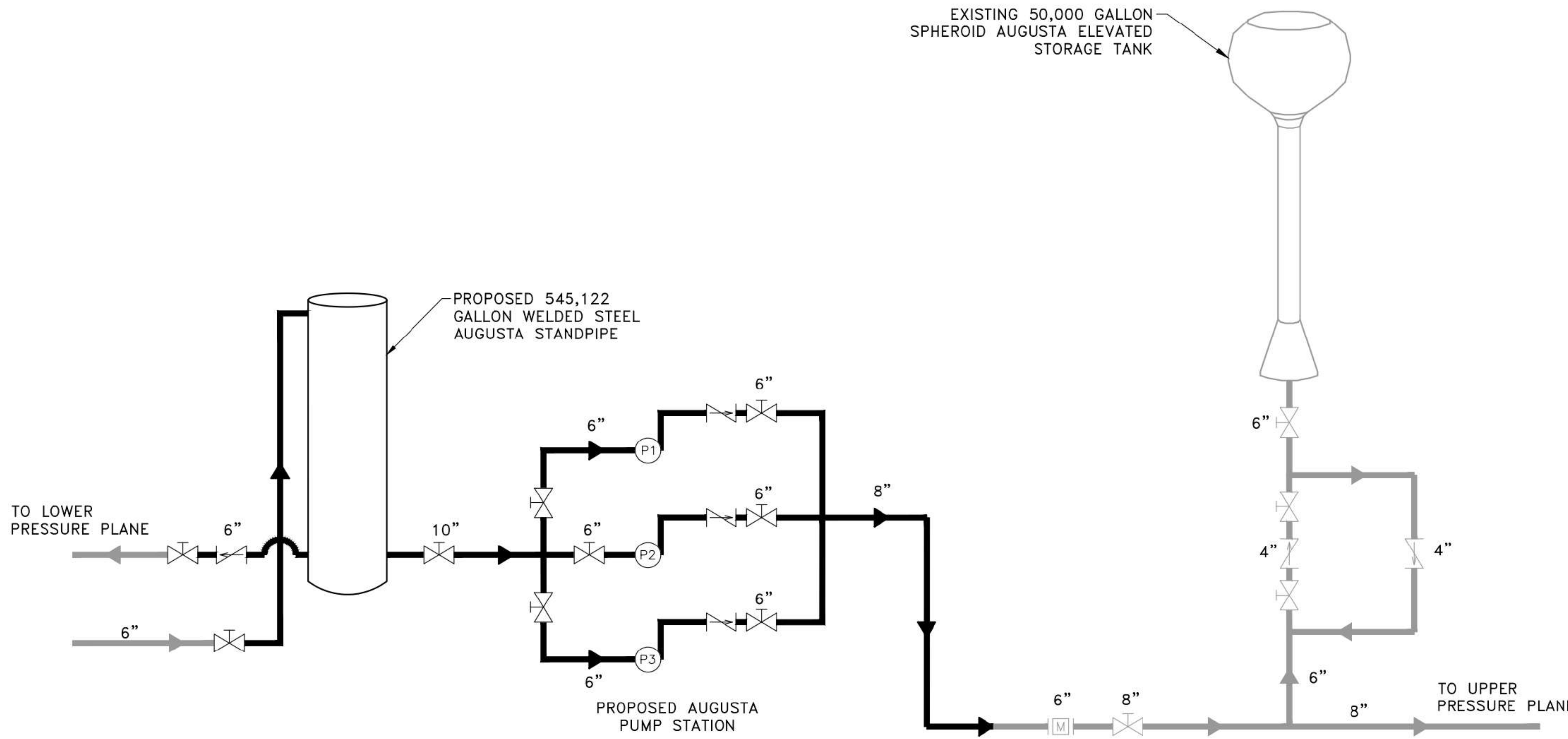
PROPOSED PRESSURE PLANE SCHEMATIC

WATER SYSTEM ANALYSIS
TRAVIS COUNTY W.C.& I.D. POINT VENTURE
POINT VENTURE, TX

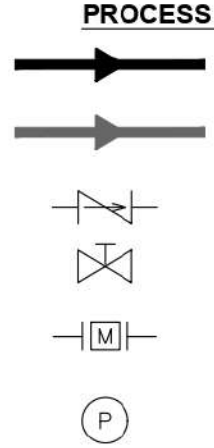
Drawn By: KP | Checked By: DV | Scale: N.T.S. | Date: 6/7/2023

EXHIBIT J

PROPOSED PROCESS FLOW DIAGRAM



PROCESS FLOW SYMBOLS



MAIN PROCESS LINE

EXISTING MAIN PROCESS LINE

SWING CHECK VALVE

HAND OPERATED VALVE

FLOW METER

PUMP

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EXHIBIT J

PROPOSED PROCESS FLOW DIAGRAM

WATER SYSTEM ANALYSIS
TRAVIS COUNTY W.C.& I.D. POINT VENTURE
POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: N.T.S.	Date: 6/8/2023

L:\TRAVIS COUNTY W.C.&I.D. POINT VENTURE – DOCUMENTS\CADD\WATERSYSTEMANALYSIS\SINGLEFIGURES\701-WCIDPV_PROCESSFLOWDIAGRAM

EXHIBIT K

PROPOSED WATER SYSTEM MAP

EXPLANATION

- WATER TREATMENT PLANT
- RAW WATER INTAKE
- WATER TANK
- FIRE HYDRANT
- GATE VALVE
- PRESSURE PLANE ISOLATION GATE VALVE
- PRESSURE REDUCING VALVE

WATER LINES

- MAIN PIPELINE - 10"
- MAIN PIPELINE - 8"
- MAIN PIPELINE - 6"
- MAIN PIPELINE - 4"
- MAIN PIPELINE - 2"

RAW WATER LINES

- RAW WATER LINE - 8"
- RAW WATER LINE - 6"
- CCN
- TRAVIS COUNTY LAND PARCEL - SITE
- UPPER PRESSURE PLANE
- LOWER PRESSURE PLANE

ABBREVIATIONS

CCN - CERTIFICATE OF CONVENIENCE AND NECESSITY.
ISO - ISOLATION
MLV - MAIN LINE VALVE

N

0 330 660'

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EXHIBIT K

PROPOSED WATER SYSTEM MAP

TRAVIS COUNTY WATER CONTROL AND IMPROVEMENT DISTRICT
POINT VENTURE, TEXAS

Drawn By: DH	Checked By: KP	Scale: 1 " = 330 '	Date: 6/14/23	File: ModelExhibitK.mxd

EXHIBIT L

FUTURE CONDITIONS W/ IMPROVEMENTS FIRE FLOW WATER MODEL RESULTS MAP

NFPA COLOR	NFPA CLASS	AVAILABLE FIRE FLOW AT 20 PSI RESIDUAL PRESSURE	DESCRIPTION
LIGHT BLUE	AA	1,500 GPM AND ABOVE	VERY GOOD FLOWS
GREEN	A	1,000 - 1,499 GPM	GOOD FOR RESIDENTIAL AREAS
ORANGE	B	500 - 999 GPM	MARGINALLY INADEQUATE
RED	C	LESS THAN 500 GPM	INADEQUATE

HYDRANT CLASSIFICATION AND COLOR CODING PER NFPA 291, RECOMMENDED PRACTICE FOR FIRE FLOW TESTING AND MARKING OF HYDRANTS.

MINIMUM REQUIRED AVAILABLE FIRE FLOW FOR THE WATER DISTRIBUTION SYSTEM IS 1,000 GPM.

ABBREVIATIONS:

GPM - GALLONS PER MINUTE
HL - HYDRANT NODE
NFPA - NATIONAL FIRE PROTECTION ASSOCIATION
PSI - POUNDS PER SQUARE INCH

N

0 300' 600'

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EXHIBIT L

FUTURE CONDITIONS W/ IMPROVEMENTS
FIRE FLOW WATER MODEL RESULTS MAP

WATER SYSTEM ANALYSIS
TRAVIS COUNTY W.C.& I.D. POINT VENTURE
POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: 1" = 300'	Date: 6/7/2023

EXHIBIT M

DESIGN CALCULATIONS

DESIGN CALCULATIONS

TRAVIS COUNTY W.C.&I.D. POINT VENTURE

WATER SYSTEM ANALYSIS

June 7, 2023

Project #: 701-023-700

PREPARED BY: Trihydro Corporation

5508 Highway, Suite 201, Austin, TX 78735

SOLUTIONS YOU CAN COUNT ON.
PEOPLE YOU CAN TRUST.

WTP High Service Pump Station

References

TCEQ Chapter 290, Subchapter D
CoA Utilities Criteria Manual
Cameron Hydraulic Data (CHD)

Pump Sizing

Number of Pumps = 2
Number of Pumps at Firm Capacity = 1
Capacity Required per Connection = 0.6 gpm/LUE
Number of LUEs in water system at Full Build-Out = 1,190 LUEs

TCEQ §290.45(b)(2)(F): For systems which provide an elevated storage capacity of 200 gallons per connection, two service pumps with a minimum combined capacity of 0.6 gpm per connection are required at each pump station or pressure plane.

$$\text{Combined Capacity Required at Full Build} - \text{Out} = 1{,}190\ \text{LUEs} \times 0.6\frac{\text{gpm}}{\text{LUE}} = 714\ \text{gpm}$$

1 Pump = 357 gpm
2 Pumps = 714 gpm

Currently, there are two (2) existing pumps at the WTP High Service Pump Station, each rated 660 gpm. The existing combined capacity is 1,320 gpm. Therefore, the existing pump station meets the TCEQ combined capacity at full build-out. However, after calculating the new TDH for pumping to the new Augusta Standpipe, each pump has been derated to 500 gpm. The derated high service pumps will have a combined capacity of 1,000 gpm, which will still meet the TCEQ combined capacity at full build-out.

Pump TDH Calculations

Suction Static Head

Clearwell Overflow =	768.60
Clearwell Max. W.S.L. =	767.60
Clearwell Min. W.S.L. =	757.60
Pump CL Elev. =	736.50 + 1.25 = 737.75
Static Head =	757.60 – 737.75 = 19.85 ft

Suction Friction Head

Exist. 12” from Clearwell Outlet to 12”x8” Reducer

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	Length of Straight Pipe (ft)	Qty	Equivalent Length (ft)
Pipe Exit[1]	76.92	1	76.92
12” Gate Valve	7.96	2	15.92
12” 90° Elbow	29.8	3	89.4
12” Tee (Thru Flow)	19.9	2	39.8
12” Tee (Branch Flow)	59.7	1	59.7
12”x8” Reducer[2]	3.69	1	3.69
Total			**285.43**

[1]Since pipe exits are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{k \times d}{f}$$

From CHD friction loss table, f = 0.013.

k value for pipe exit = 1.0

$$\text{Equiv. Length} = \frac{1.0 \times 1.0\ \text{ft}}{0.013} = 76.92\ \text{ft}$$

[2]Since reducer fittings are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{k \times d}{f}$$

From CHD friction loss table, f = 0.013.

$$k = 0.8 \times \sin\frac{\theta}{2} \times \left(1 - \frac{{d_1}^2}{{d_2}^2}\right)^2 \text{ for gradual contraction}$$

$$k = 0.8 \times \sin\frac{22.62°}{2} \times \left(1 - \frac{8^2}{12^2}\right)^2 = 0.048$$

$$\text{Equiv. Length} = \frac{0.048 \times 1.00\ \text{ft}}{0.013} = 3.69\ \text{ft}$$

Approximate pipe length from Clearwell outlet to the existing 12”x8” reducer is 188 feet.

Per CHD, friction tables for 12-inch pipe, head loss for pump flows of 357-, 714-, and 600-gpm are 0.03226-, 0.11862-, and 0.085-feet per 100-ft, respectively. Calculate friction heads.

$$h_{L,357} = \frac{0.03226\ \text{ft}}{100\ \text{ft pipe length}} \times (285.43 + 188.00)\ \text{ft} = 0.15\ \text{ft}$$

$$h_{L,714} = \frac{0.11862\ \text{ft}}{100\ \text{ft pipe length}} \times (285.43 + 188.00)\ \text{ft} = 0.56\ \text{ft}$$

$$h_{L,600} = \frac{0.085\ \text{ft}}{100\ \text{ft pipe length}} \times (285.43 + 188.00)\ \text{ft} = 0.40\ \text{ft}$$

Exist. 8” from 12”x8” Reducer to 8” Pump Can

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	**Length of Straight Pipe (ft)**	**Qty**	**Equivalent Length (ft)**
8” Tee (Branch Flow)	39.9	2	79.8
8” Gate Valve	5.32	1	5.32
8” Pipe Entrance[1]	37.33	1	37.33
Total			**122.45**

[1]Since pipe entrances are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{k \times d}{f}$$

From CHD friction loss table, f = 0.014.

k value for pipe entrance = 0.78

$$\text{Equiv. Length} = \frac{0.78 \times 0.67\ \text{ft}}{0.014} = 37.33\ \text{ft}$$

Approximate pipe length from the existing 12”x8” reducer to the existing pump can is 14 feet.

Per CHD, friction tables for 8-inch pipe, head loss for pump flows of 357-, 714-, and 600-gpm are 0.24452-, 0.92712-, and 0.661-feet per 100-ft, respectively. Calculate friction heads.

$$h_{L,357} = \frac{0.24452\ \text{ft}}{100\ \text{ft pipe length}} \times (122.45 + 14)\ \text{ft} = 0.33\ \text{ft}$$

$$h_{L,714} = \frac{0.92712\ \text{ft}}{100\ \text{ft pipe length}} \times (122.45 + 14)\ \text{ft} = 1.27\ \text{ft}$$

$$h_{L,600} = \frac{0.661\ \text{ft}}{100\ \text{ft pipe length}} \times (122.45 + 14)\ \text{ft} = 0.90\ \text{ft}$$

<u>Suction Total Dynamic Head</u>

$$\text{Suction TDH @ 357 gpm} = \text{static} - \text{friction} = 19.85\ \text{ft} - (0.15 + 0.33)\ \text{ft} = 19.37\ \text{ft} \rightarrow 19\ \text{ft}$$

$$\text{Suction TDH @ 714 gpm} = \text{static} - \text{friction} = 19.85\ \text{ft} - (0.56 + 1.27)\ \text{ft} = 18.02\ \text{ft} \rightarrow 18\ \text{ft}$$

$$\text{Suction TDH @ 600 gpm} = \text{static} - \text{friction} = 19.85\ \text{ft} - (0.40 + 0.90)\ \text{ft} = 18.55\ \text{ft} \rightarrow 19\ \text{ft}$$

<u>Discharge Static Head</u>

Bldg. FFE =	736.50
Pump CL Above FFE =	1’-3” (1.25 ft)
Pump CL Elev. =	736.50 + 1.25 = 737.75
New Standpipe Overflow Elev. =	926.50
Static Head =	926.50 – 737.75 = 188.75 ft

<u>Discharge Friction Head</u>

<u>*Exist. 8” from Pump Discharge to 8”x6” Reducer at Venture Dr. & Summit Ridge Dr. S.*</u>

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	**Length of Straight Pipe (ft)**	**Qty**	**Equivalent Length (ft)**
8”x6” Reducer[1]	2.69	4	10.76
6” Globe Check Valve (Booster Pump Control CLA-VAL)	172.0	1	172.0
8” Butterfly Valve	29.9	1	29.9
8” Tee (Branch Flow)	39.9	4	159.6
8” 90° Elbow	20.0	3	60.0
8” 45° Elbow	10.6	11	116.6
8” Turbine Meter[2]	358.93	1	358.93
8” Gate Valve	5.32	19	101.08
8” Tee (Thru Flow)	13.3	44	585.2
Total			**1,594.07**

[1]Since reducer fittings are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{k \times d}{f}$$

From CHD friction loss table, f = 0.014.

$$k = 2.6 \times \sin\frac{\theta}{2} \times \left(1 - \frac{{d_1}^2}{{d_2}^2}\right)^2 \text{ for gradual expansion}$$

$$k = 2.6 \times \sin\frac{12.85°}{2} \times \left(1 - \frac{6^2}{8^2}\right)^2 = 0.056$$

$$\text{Equiv. Length} = \frac{0.056 \times 0.67 \text{ ft}}{0.014} = 2.69 \text{ ft}$$

[2]Since meters are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{k \times d}{f}$$

From CHD friction loss table, f = 0.014.

k value for turbine meter = 7.5

$$\text{Equiv. Length} = \frac{7.5 \times 0.67 \text{ ft}}{0.014} = 358.93 \text{ ft}$$

Approximate pipe length from pump discharge to the existing 8"x6" reducer at intersection of Venture Drive and Summit Ridge Drive South is 7,317 feet.

Per CHD, friction tables for 8-inch pipe, head loss for pump flows of 357-, 714-, and 600-gpm are 0.24452-, 0.92712-, and 0.661-feet per 100-ft, respectively. Calculate friction heads.

$$h_{L,357} = \frac{0.24452 \text{ ft}}{100 \text{ ft pipe length}} \times (1{,}594.07 + 7{,}317.00) \text{ ft} = 21.79 \text{ ft}$$

$$h_{L,714} = \frac{0.92712 \text{ ft}}{100 \text{ ft pipe length}} \times (1{,}594.07 + 7{,}317.00) \text{ ft} = 82.62 \text{ ft}$$

$$h_{L,600} = \frac{0.661 \text{ ft}}{100 \text{ ft pipe length}} \times (1{,}594.07 + 7{,}317.00) \text{ ft} = 58.90 \text{ ft}$$

Exist. 6" from 8"x6" Reducer at Venture Dr. & Summit Ridge Dr. S. to Proposed Augusta Standpipe Overflow Elevation

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	Length of Straight Pipe (ft)	Qty	Equivalent Length (ft)
6" 45° Elbow	8.09	3	24.27
6" 90° Elbow	15.2	3	45.6
6" Gate Valve	4.04	1	4.04
6" Tee (Thru Flow)	10.1	1	10.1
Total			**84.01**

Approximate pipe length from existing 8"x6" reducer at intersection of Venture Drive and Summit Ridge Drive South to proposed Augusta Standpipe overflow elevation is 345 feet.

Per CHD, friction tables for 6-inch pipe, head loss for pump flows of 357-, 714-, and 600-gpm are 1.05305-, 4.0596-, and 2.89-feet per 100-ft, respectively. Calculate friction heads. Since the existing 6-inch line is over 20 years old and condition is fair, incorporate a design safety factor of 20%.

$$\mathrm{h_{L,357}} = \frac{1.05305\ \mathrm{ft}}{100\ \mathrm{ft\ pipe\ length}} \times (84.01 + 345.00)\ \mathrm{ft} = 4.52\ \mathrm{ft}\ \times 1.2 = 5.42\ \mathrm{ft}$$

$$\mathrm{h_{L,714}} = \frac{4.0956\ \mathrm{ft}}{100\ \mathrm{ft\ pipe\ length}} \times (84.01 + 345.00)\ \mathrm{ft} = 17.57\ \mathrm{ft}\ \times 1.2 = 21.08\ \mathrm{ft}$$

$$\mathrm{h_{L,600}} = \frac{2.89\ \mathrm{ft}}{100\ \mathrm{ft\ pipe\ length}} \times (84.01 + 345.00)\ \mathrm{ft} = 12.40\ \mathrm{ft}\ \times 1.2 = 14.88\ \mathrm{ft}$$

Discharge Total Dynamic Head

$$\text{Dischage TDH @ 357 gpm} = \text{static} + \text{friction} = 188.75\ \mathrm{ft} + (21.79 + 5.42)\ \mathrm{ft} = 216\ \mathrm{ft}$$

$$\text{Dischage TDH @ 714 gpm} = \text{static} + \text{friction} = 188.75\ \mathrm{ft} + (82.62 + 21.08)\ \mathrm{ft} = 292\ \mathrm{ft}$$

$$\text{Dischage TDH @ 600 gpm} = \text{static} + \text{friction} = 188.75\ \mathrm{ft} + (58.90 + 14.88)\ \mathrm{ft} = 263\ \mathrm{ft}$$

Overall Total Dynamic Head

$$\text{TDH @ 357 gpm} = \text{dischage} - \text{suction} = (216 - 19)\text{ft} = 197\text{ ft (85.28 psi)}$$

$$\text{TDH @ 714 gpm} = \text{dischage} - \text{suction} = (292 - 18)\text{ft} = 274\text{ ft (118.61 psi)}$$

$$\text{TDH @ 600 gpm} = \text{dischage} - \text{suction} = (263 - 19)\text{ft} = 244\text{ ft (105.63 psi)}$$

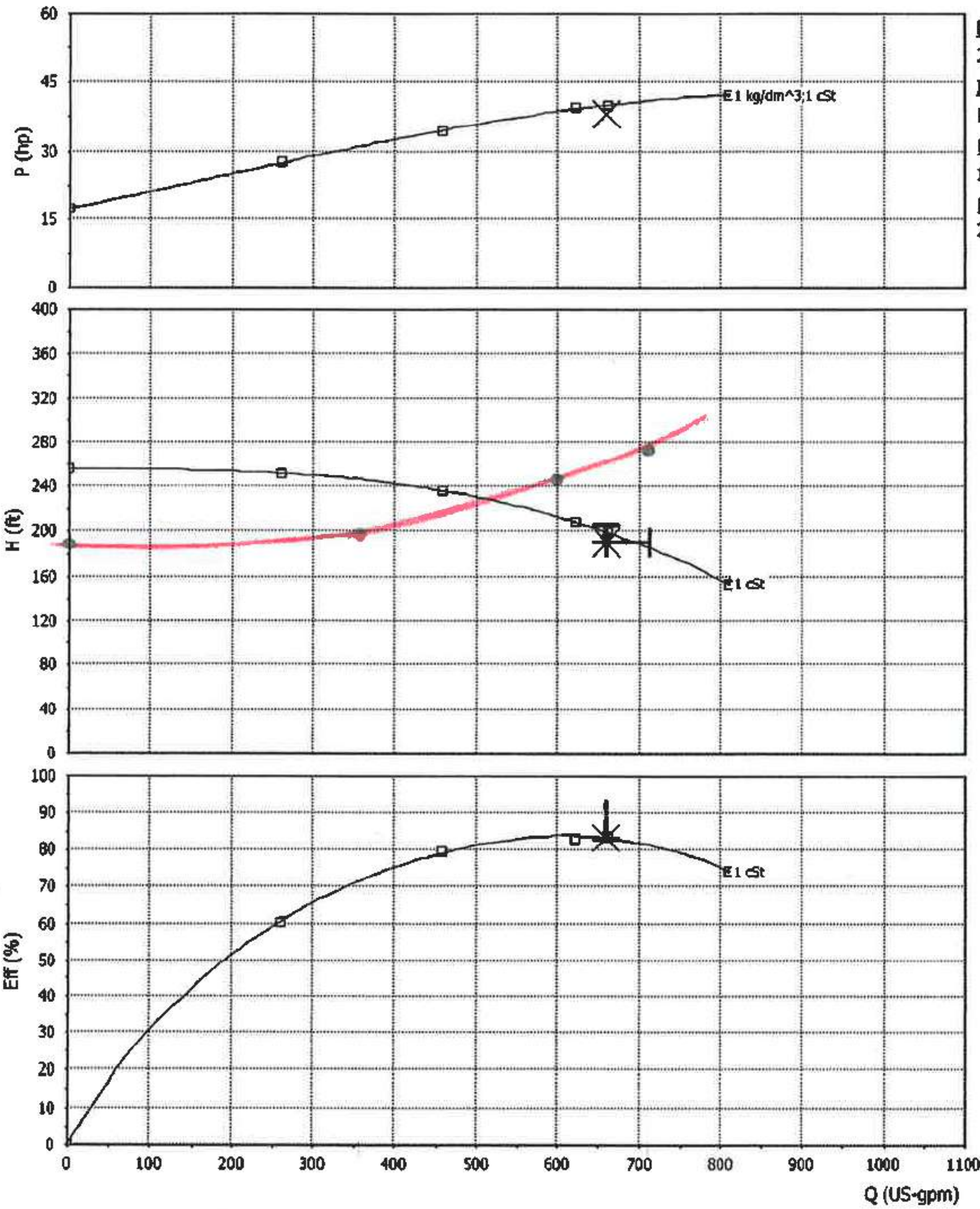
New system curve overlaps the existing pump curve at 500-gpm (0.72 MGD). Refer to Ruhrpumpen Pump Test Sheet on next page.

$$\text{Each Pump: 500 gpm} \times \frac{\text{LUE}}{0.6\text{ gpm}} = 833\text{ LUEs}$$

$$\text{Combined Capacity: 1,000 gpm} \times \frac{\text{LUE}}{0.6\text{ gpm}} = 1{,}667\text{ LUEs}$$

RUHRPUMPEN 1.2.2	Ruhrpumpen Industrial, S.A. de C.V. Monterrey , Mexico **PUMP TEST SHEET**	Date : 1/15/2019 Page : 4/4 Test Nr : 242700111-T1P2

Speed: 1770 rpm



Project Nr
242700111-T1P2
Item Nr
HSP-03 B
Pump Type
10G-H
MCSF
252.7 US-gpm

Inspected By : Company :	Test Leader : Luis M. Valdez Salazar

Augusta Pump Station

References

TCEQ Chapter 290, Subchapter D

CoA Utilities Criteria Manual

Cameron Hydraulic Data (CHD)

Pump Sizing

Number of Pumps = 3

Number of Pumps at Firm Capacity = 2

Capacity Required per Connection = 2.0 gpm/LUE

Number of LUEs in Upper Pressure Plane at Full Build-Out = 500 LUEs

TCEQ §290.45(b)(2)(F): A service pump capacity that provides each pump station or pressure plane with two or more pumps that have a total capacity of 2.0 gpm per connection or that have a total capacity of at least 1,000 gpm and the ability to meet peak hourly demands with the largest pump out of service, whichever is less.

$$\text{Firm Capacity Required at Full Build} - \text{Out} = 500\ \text{LUEs} \times 2.0\,\frac{\text{gpm}}{\text{LUE}} = 1{,}000\ \text{gpm}$$

$$\text{Flow Rate for each pump} = \frac{1{,}000\ \text{gpm}}{2} = 500\ \text{gpm}$$

Therefore, each pump is rated at 500 gpm.

Pump TDH Calculations

Suction Static Head

New Standpipe Min. W.S.L. =	899.00
Pump CL Elev. =	823.00
Static Head =	899.00 – 823.00 = 76.00 ft

Suction Friction Head

Prop. 10" from Standpipe Outlet to 10"x6" Tee

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	**Length of Straight Pipe (ft)**	**Qty**	**Equivalent Length (ft)**
10” Pipe Exit[1]	59.29	1	59.29
10” Gate Valve	6.68	1	6.68
10” Tee (Branch Flow)	50.1	1	50.1
10”x6” Tee (Branch Flow)	50.1	1	50.1
Total			**166.17**

[1]Since pipe exits are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{\mathrm{k} \times \mathrm{d}}{\mathrm{f}}$$

From CHD friction loss table, f = 0.014.

k value for pipe exit = 1.0

$$\text{Equiv. Length} = \frac{1.0 \times 0.83\ \text{ft}}{0.014} = 59.29\ \text{ft}$$

Approximate pipe length from Standpipe outlet to the 10”x6” tee is 10.00 feet.

Per CHD, friction tables for 10-inch pipe, head loss for pump flow of 1,000 gpm is 0.569 feet per 100-ft. Calculate friction head.

$$\mathrm{h_L} = \frac{0.569\ \text{ft}}{100\ \text{ft pipe length}} \times (166.17 + 10.0)\ \text{ft} = 1.00\ \text{ft}$$

Prop. 6” from 10”x6” Tee to 3” Pump Suction

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	**Length of Straight Pipe (ft)**	**Qty**	**Equivalent Length (ft)**
6” Gate Valve	4.04	1	4.04
6”x3” Reducer[1]	3.23	1	3.23
Total			**7.27**

[1]Since reducer fittings are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{\mathrm{k} \times \mathrm{d}}{\mathrm{f}}$$

From CHD friction loss table, f = 0.015.

$$\mathrm{k} = 0.8 \times \sin\frac{\theta}{2} \times \left(1 - \frac{{\mathrm{d}_1}^2}{{\mathrm{d}_2}^2}\right)^2 \text{ for gradual contraction}$$

$$\mathrm{k} = 0.8 \times \sin\frac{24.87°}{2} \times \left(1 - \frac{3^2}{6^2}\right)^2 = 0.097$$

$$\text{Equiv. Length} = \frac{0.097 \times 0.50 \text{ ft}}{0.015} = 3.23 \text{ ft}$$

Approximate pipe length from the 10”x6” tee to the 3” pump suction is 5.50 feet.

Per CHD, friction tables for 6-inch pipe, head loss for pump flow of 500 gpm is 2.02 feet per 100-ft. Calculate friction head.

$$\mathrm{h_L} = \frac{2.02 \text{ ft}}{100 \text{ ft pipe length}} \times (7.27 + 5.50) \text{ ft} = 0.26 \text{ ft}$$

<u>Suction Total Dynamic Head</u>

$$\text{Suction TDH} = \text{static} - \text{friction} = 76.00 \text{ ft} - (1.00 + 0.26) \text{ ft} = 74.74 \text{ ft} \rightarrow 75 \text{ ft}$$

<u>Discharge Static Head</u>

Bldg. FFE =	822.00
Pump CL Above FFE =	1’-0” (1.00 ft)
Pump CL Elev. =	822.00 + 1.00 = 823.00
Exist. EST H.W.L. =	959.50
Static Head =	959.50 – 823.00 = 136.50 ft

<u>Discharge Friction Head</u>

<u>*Prop. 6” from Pump Discharge to 8”x6” Tee*</u>

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	Length of Straight Pipe (ft)	Qty	Equivalent Length (ft)
6”x3” Reducer[1]	10.50	1	10.50
6” 90° Elbow	15.2	1	15.2
6” Globe Check Valve	172.0	1	172.0
6” Gate Valve	4.04	1	4.04
Total			**201.74**

[1]Since reducer fittings are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{k \times d}{f}$$

From CHD friction loss table, f = 0.015.

$$k = 2.6 \times \sin\frac{\theta}{2} \times \left(1 - \frac{{d_1}^2}{{d_2}^2}\right)^2 \text{ for gradual expansion}$$

$$k = 2.6 \times \sin\frac{24.87°}{2} \times \left(1 - \frac{3^2}{6^2}\right)^2 = 0.315$$

$$\text{Equiv. Length} = \frac{0.315 \times 0.50 \text{ ft}}{0.015} = 10.50 \text{ ft}$$

Approximate pipe length from pump discharge to 6”x8” tee is 8.00 feet.

Per CHD, friction tables for 6-inch pipe, head loss for pump flow of 500 gpm is 2.02 feet per 100-ft. Calculate friction head.

$$h_L = \frac{2.02 \text{ ft}}{100 \text{ ft pipe length}} \times (201.74 + 8.00) \text{ ft} = 4.24 \text{ ft}$$

Prop. 8” from 6”x8” Tee to Exist. 8”x6” Tee

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	Length of Straight Pipe (ft)	Qty	Equivalent Length (ft)
8”x6” Tee (Branch Flow)	39.9	2	79.8
8” 90° Elbow	20.0	3	60.0
8”x 6” Reducer[1]	2.69	2	5.38

6" Turbine Meter[2]	250.0	1	250.0
8" Gate Valve	5.32	1	5.32
Total			**400.5**

[1]Since reducer fittings are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{\mathrm{k} \times \mathrm{d}}{\mathrm{f}}$$

From CHD friction loss table, f = 0.014.

$$\mathrm{k} = 2.6 \times \sin\frac{\theta}{2} \times \left(1 - \frac{{\mathrm{d}_1}^2}{{\mathrm{d}_2}^2}\right)^2 \text{ for gradual expansion}$$

$$\mathrm{k} = 2.6 \times \sin\frac{12.85°}{2} \times \left(1 - \frac{6^2}{8^2}\right)^2 = 0.056$$

$$\text{Equiv. Length} = \frac{0.056 \times 0.67 \text{ ft}}{0.014} = 2.69 \text{ ft}$$

[2]Since meters are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{\mathrm{k} \times \mathrm{d}}{\mathrm{f}}$$

From CHD friction loss table, f = 0.015.

k value for turbine meter = 7.5

$$\text{Equiv. Length} = \frac{7.5 \times 0.50 \text{ ft}}{0.015} = 250 \text{ ft}$$

Approximate pipe length from 6"x8" tee to the existing 8"x6" tee is 37.5 feet.

Per CHD, friction tables for 8-inch pipe, head loss for pump flow of 1,000 gpm is 1.78 feet per 100-ft. Calculate friction head.

$$\mathrm{h_L} = \frac{1.78 \text{ ft}}{100 \text{ ft pipe length}} \times (400.5 + 37.5) \text{ ft} = 7.80 \text{ ft}$$

Exist. 6" from Exist. 8"x6" Tee to Exist. 6"x4" Reducer

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	**Length of Straight Pipe (ft)**	**Qty**	**Equivalent Length (ft)**
6"x4" Reducer[1]	0.9	1	0.9
Total			**0.9**

[1]Since reducer fittings are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{\text{k} \times \text{d}}{\text{f}}$$

From CHD friction loss table, f = 0.015.

$$\text{k} = 0.8 \times \sin\frac{\theta}{2} \times \left(1 - \frac{{d_1}^2}{{d_2}^2}\right)^2 \text{ for gradual contraction}$$

$$\text{k} = 0.8 \times \sin\frac{12.68°}{2} \times \left(1 - \frac{4^2}{6^2}\right)^2 = 0.027$$

$$\text{Equiv. Length} = \frac{0.027 \times 0.5 \text{ ft}}{0.015} = 0.9 \text{ ft}$$

Approximate pipe length from the existing 8"x6" tee to the existing 6"x4" reducer is 4.25 feet.

Per CHD, friction tables for 6-inch pipe, head loss for pump flow of 1,000 gpm is 7.87 feet per 100-ft. Calculate friction head. Since the existing 6-inch line is over 20 years old and condition is decent, incorporate a design safety factor of 15%.

$$h_L = \frac{7.87 \text{ ft}}{100 \text{ ft pipe length}} \times (0.9 + 4.25) \text{ ft} = 0.41 \text{ ft} \times 1.15 = 0.47 \text{ ft}$$

Exist. 4" from Exist. 6"x4" Reducer to Exist. 4"x6" Reducer

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	**Length of Straight Pipe (ft)**	**Qty**	**Equivalent Length (ft)**
4" Tee (Thru Flow)	6.71	2	13.42

4" Gate Valve	2.68	2	5.36
4" Globe Check Valve (Altitude Control CLA-VAL)	114.0	1	114.0
Total			**132.78**

Approximate pipe length from the existing 6"x4" reducer to the existing 4"x6" reducer is 5.75 feet.

Per CHD, friction tables for 4-inch pipe, head loss for pump flow of 1,000 gpm is 64.8 feet per 100-ft. Calculate friction head. Since the existing 4-inch line is over 20 years old and condition is decent, incorporate a design safety factor of 15%.

$$h_L = \frac{64.8 \text{ ft}}{100 \text{ ft pipe length}} \times (132.78 + 5.75) \text{ ft} = 89.77 \text{ ft} \times 1.15 = 103.24 \text{ ft}$$

Exist. 6" from Exist. 4"x6" Reducer to Exist. 6"x8" Reducer

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	**Length of Straight Pipe (ft)**	**Qty**	**Equivalent Length (ft)**
6"x4" Reducer[1]	2.97	1	2.97
6" Gate Valve	4.04	1	4.04
Total			**7.01**

[1]Since reducer fittings are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{k \times d}{f}$$

From CHD friction loss table, f = 0.015.

$$k = 2.6 \times \sin\frac{\theta}{2} \times \left(1 - \frac{{d_1}^2}{{d_2}^2}\right)^2 \text{ for gradual expansion}$$

$$k = 2.6 \times \sin\frac{12.68°}{2} \times \left(1 - \frac{4^2}{6^2}\right)^2 = 0.089$$

$$\text{Equiv. Length} = \frac{0.089 \times 0.5 \text{ ft}}{0.015} = 2.97 \text{ ft}$$

Approximate pipe length from the existing 4"x6" reducer to the existing 6"x8" reducer is 15.25 feet.

Per CHD, friction tables for 6-inch pipe, head loss for pump flow of 1,000 gpm is 7.87 feet per 100-ft. Calculate friction head. Since the existing 6-inch line is over 20 years old and condition is decent, incorporate a design safety factor of 15%.

$$h_L = \frac{7.87\ \text{ft}}{100\ \text{ft pipe length}} \times (7.01 + 15.25)\ \text{ft} = 1.75\ \text{ft} \times 1.15 = 2.01\ \text{ft}$$

Exist. 8" from Exist. 6"x8" Reducer to EST H.W.L.

Friction Loss in pipe fittings in terms of equivalent length. Refer to CHD friction loss table.

Description	Length of Straight Pipe (ft)	Qty	Equivalent Length (ft)
8"x6" Reducer[1]	2.69	1	2.69
8" 90° Elbow	20.0	1	20.00
8" Pipe Exit[2]	47.86	1	47.86
Total			**70.55**

[1]Since reducer fittings are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{k \times d}{f}$$

From CHD friction loss table, f = 0.014.

$$k = 2.6 \times \sin\frac{\theta}{2} \times \left(1 - \frac{{d_1}^2}{{d_2}^2}\right)^2 \text{ for gradual expansion}$$

$$k = 2.6 \times \sin\frac{12.85°}{2} \times \left(1 - \frac{6^2}{8^2}\right)^2 = 0.056$$

$$\text{Equiv. Length} = \frac{0.056 \times 0.67\ \text{ft}}{0.014} = 2.69\ \text{ft}$$

[2]Since pipe exits are not included in friction loss table, calculate equivalent length:

$$\text{Equiv. Length} = \frac{k \times d}{f}$$

From CHD friction loss table, f = 0.014.

k value for pipe exit = 1.0

$$\text{Equiv. Length} = \frac{1.0 \times 0.67\ \text{ft}}{0.014} = 47.86\ \text{ft}$$

Approximate pipe length from the existing 6”x8” reducer to EST H.W.L. is 139.25 feet.

Per CHD, friction tables for 8-inch pipe, head loss for pump flow of 1,000 gpm is 1.78 feet per 100-ft. Calculate friction head. Since the existing 8-inch line is over 20 years old and condition is decent, incorporate a design safety factor of 15%.

$$h_L = \frac{1.78\ \text{ft}}{100\ \text{ft pipe length}} \times (70.55 + 139.25)\ \text{ft} = 3.73\ \text{ft} \times 1.15 = 4.29\ \text{ft}$$

Discharge Total Dynamic Head

$$\begin{aligned}\text{Dischage TDH} &= \text{static} + \text{friction} = 136.5\ \text{ft} + (4.24 + 7.8 + 0.47 + 103.24 + 2.01 + 4.29)\ \text{ft} \\ &= 258.55\ \text{ft} \rightarrow 259\ \text{ft}\end{aligned}$$

Overall Total Dynamic Head

$$\text{TDH} = \text{dischage} - \text{suction} = (259 - 75)\text{ft} = 184\ \text{ft}\ (79.65\ \text{psi})$$

Augusta Standpipe Calculations

References

TCEQ Chapter 290, Subchapter D

Sizing Calculations

TCEQ §290.38(25): Elevated storage capacity – That portion of water which can be stored at least 80 feet (35 psi) above the highest service connection in the pressure plane served by the storage tank.

TCEQ §290.45(b)(2)(F): For systems which provide an elevated storage capacity of 200 gallons per connection, two service pumps with a minimum combined capacity of 0.6 gpm per connection are required at each pump station or pressure plane.

Set Minimum Pressure in Lower Pressure Plane to 45 psi.

$$\text{Static Pressure} = 45\ \text{psi} \times \frac{2.31\ \text{ft}}{\text{psi}} = 103.95\ \text{ft} \rightarrow \text{round to } 104\ \text{ft}$$

Maximum Elevation (High Point) of Lower Pressure Plane = 795.00

$$\text{Low Water Level (LWL)} = 795.00 + 104.00 = 899.00\ (45\ \text{psi})$$

The 899.00 elevation will be the Lower Pressure Plane Hydraulic Grade Line (HGL).

Out of the 1,190 LUEs at full build-out, 690 LUEs will be allocated to the Lower Pressure Plane.

TCEQ Required Elevated Storage Capacity (@ 690 LUEs) is:

$$690\ \text{LUEs} \times \frac{200\ \text{gal}}{\text{LUE}} = 138{,}000\ \text{gallons}$$

Standpipe Diameter = 30'-0" [per US Underwater Services 2020.6.22 Inspection Report]

Gallons per vertical foot is:

$$7.48\frac{\text{gal}}{\text{cf}} \times \frac{\pi}{4} \times (30\ \text{ft})^2 = 5{,}287.3\ \frac{\text{gal}}{\text{vf}}$$

Head range for elevated storage is:

$$\frac{138{,}000\ \text{gal}}{5{,}287.3\ \frac{\text{gal}}{\text{vf}}} = 26.1\ \text{ft} \rightarrow 26.5\ \text{ft}$$

$$\text{High Water Level (HWL)} = 899.00 + 26.5 = 925.5\ (56.5\ \text{psi})$$

Overflow elevation is typically 1 foot above max. water surface level.

$$\text{Overflow} = 925.50 + 1.00 = 926.50\ (56.9\ \text{psi})$$

Finish Floor Elevation (FFE) = 822.00

Total height of new tank from FFE to Overflow = 926.50 – 822.00 = 104.50 feet

Ground Storage Capacity from FFE to LWL is:

$$\frac{\pi}{4} \times (30\ \text{ft})^2 \times 7.48\frac{\text{gal}}{\text{cf}} \times (899 - 822)\ \text{ft}\ = 407{,}122\ \text{gal}$$

<u>Tank Summary</u>

FFE =	822.00
LWL =	899.00 (Lower Pressure Plane HGL)
HWL =	925.50
Head Range =	26’-6” (from LWL to HWL)
Overflow =	926.50 (1 ft above HWL)
Height =	104’-6” (from FFE to Overflow)
Elevated Storage =	138,000 gal
Ground Storage =	407,122 gal
Total Storage =	545,122 gal

<u>TCEQ Storage Capacity Requirements Assessment</u>

TCEQ Required Total Storage Capacity (@ 1,190 LUEs) is:

$$1{,}190\ \text{LUEs}\ \times \frac{200\ \text{gal}}{\text{LUE}} = 238{,}000\ \text{gallons}$$

Out of the 1,190 LUEs at full build-out, 690 LUEs will be allocated to the Lower Pressure Plane and 500 LUEs will be allocated to the Upper Pressure Plane.

<u>Lower Pressure Plane</u>

Total Storage:	407,122 Gallons Ground Storage
	138,000 Gallons Elevated Storage
	545,122 Gallons Total Storage
LUE Synopsis:	545,122 gal / 200 gal per LUE = 2,726 LUEs

<u>Upper Pressure Plane</u>

Total Storage:	0 Gallons Ground Storage
	50,000 Gallons Elevated Storage
	50,000 Gallons Total Storage
LUE Synopsis:	50,000 gal / 100 gal per LUE = 500 LUEs

EXHIBIT N

PROPOSED EXTENTS OF PRESSURE PLANES

EXPLANATION

UPPER PRESSURE PLANE (MIN. ELEV. = 755.00; MAX. ELEV. = 836.50)

LOWER PRESSURE PLANE (MIN. ELEV. = 715.00; MAX. ELEV. = 795.00)

------ EXTENTS OF PRESSURE PLANE BOUNDARY

N

0 300' 600'

Trihydro CORPORATION

Texas Engineering Firm F-131
Texas Survey Firm 10194320

Austin
1005 E Saint Elmo Rd Building 7
Austin, Texas 78745
(P) 512/442.3008 (F) 307/745.7729

New Braunfels
1011 West County Line Rd
New Braunfels, Texas 78130
(P) 830/626.3588 (F) 830/626.3544

www.trihydro.com

EXHIBIT N

PROPOSED EXTENTS OF PRESSURE PLANES

WATER SYSTEM ANALYSIS
TRAVIS COUNTY W.C.& I.D. POINT VENTURE
POINT VENTURE, TX

Drawn By: KP	Checked By: DV	Scale: 1" = 300'	Date: 3/10/2023