

## Water Model Update Existing Water Distribution System

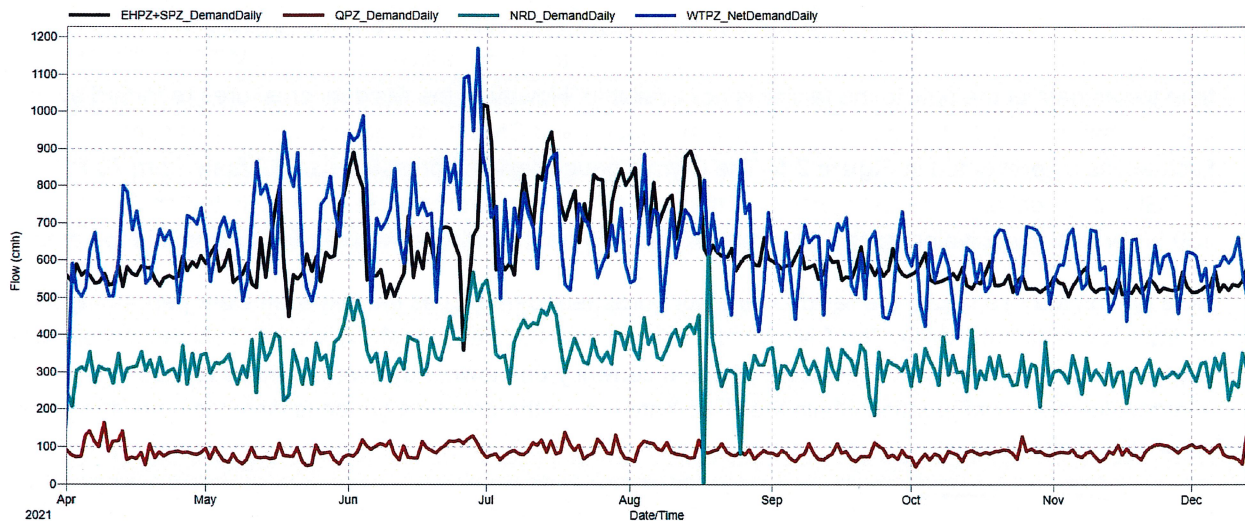


Figure 2-4: Calculated Local Daily Demand Flows

### 2.4.2.2 Peaking Factors

The flow statistics, i.e., the ADD, MDD and PHD were calculated on the processed time series flow data. Table 2-5 presents the flow statistics and the calculated peaking factors.

Table 2-5: Statistics and Peaking Factors from Time Series Flow Data

Parameter	WTPPZ	EHPZ+SPZ	NRD	QPZ	WTP HLPS
ADD (m3/h)	650	613	334	87	1,684
MDD (m3/h)	1,171	1,019	662	165	2,766
PHD (m3/h)	1,793	1,763	784	415	3,201
MDD/ADD	1.80	1.66	1.98	1.90	1.64
PHD/ADD	2.76	2.88	2.35	4.77	1.90
PHD/MDD	1.53	1.73	1.18	2.52	1.16

As indicated in the above table, the peaking factors derived from the WTP HLPS metered flow are lower than the factors derived from the demand flows in the pressure zones, which is considered reasonable as the filling flows from the WTP HLPS to other facilities skewed the flow patterns. The MDD/ADD factors are higher than the City's residential flow peaking factor which is 1.6 (MDD/ADD = 600 Lcpd/375 Lcpd) as there are mixed land use types in the pressure zones.

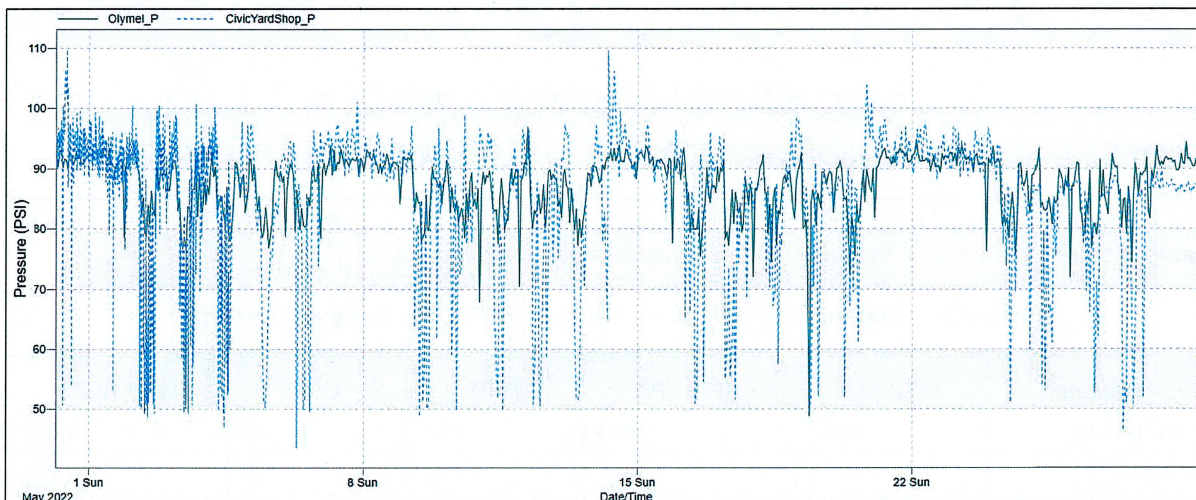
In Queens Business Park, the PHD/ADD = 4.77 is higher than the rest of the pressure zones. It is perceived that the high value results from the high flow car wash operation and low base demands in the Queens Business Park. Eventually, the factor value will decrease to around 4 as more water users move into this industrial park.



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In WTPPZ, the biggest water user is the Olymel facility, which consumed an ADD of 88 m<sup>3</sup>/hr, accounting for approximate 13% of the ADD in WTPPZ or 5% of the total flow delivered from the WTP HLPS. The time series data of the flow to the facility is not available. However, the residual pressures recorded by the gauges that the City installed in the area (Olymel gauge and Civic Yards gauge) have significant fluctuations as presented in **Figure 2-5**. The Olymel gauge residual pressure can fluctuate from 70 PSI to 92 PSI in a typical day. The estimated flow that can cause this level of pressure drop is 550 m<sup>3</sup>/hr or 150 L/s, which is equivalent to the flow from a typical hydrant connected with a 150 mm main in the area.

Note that on May 19, 2022, the residual pressure dropped to 48 PSI as the City staff were performing the hydrant flow test in the area. The Civic Yards residual pressure records show a larger fluctuation than the Olymel gauge records show. The cause of this is currently unknown, however it may be related to the internal plumbing configuration within the Civic Yards, e.g., the pressure gauge is installed downstream of a fast action check valve.



**Figure 2-5: Residual Pressure Records from the Olymel and Civic Yards Gauges**

### 2.4.2.3 Diurnal Curves

The hourly time series flow data from each pressure zone are processed to develop the diurnal patterns and to preserve the calculated peaking factor values in **Table 2-5**. The demand diurnal patterns were adjusted as presented diurnal curves in the following **Figure 2-6**. These demand patterns can be used in the Extended Period Simulation (EPS).



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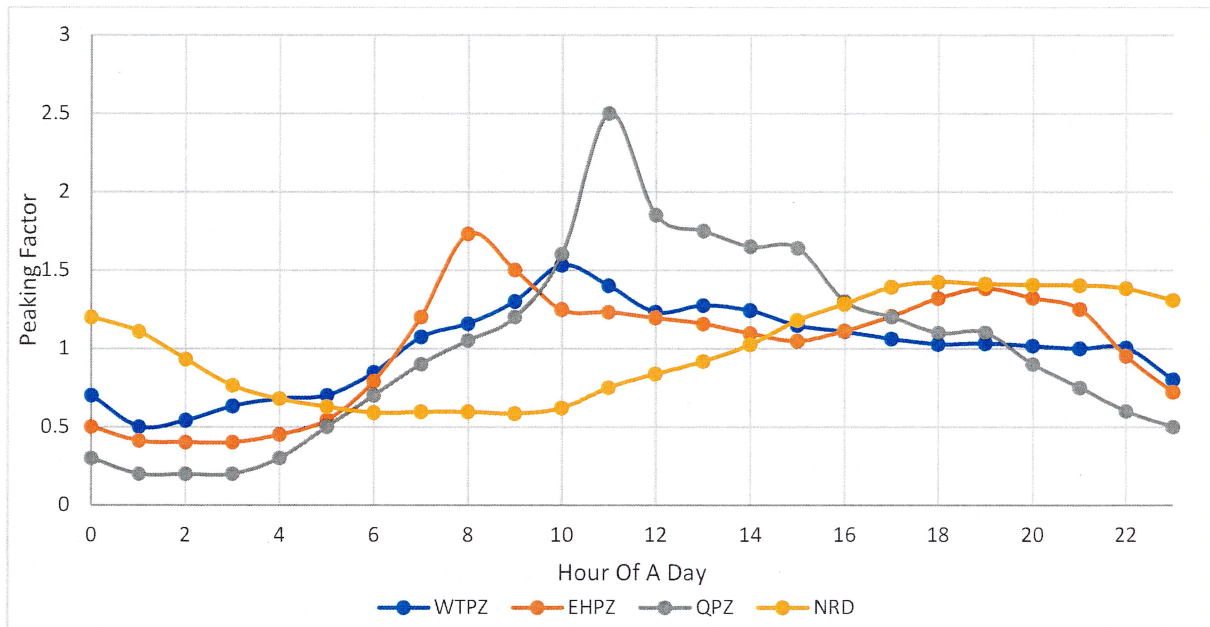


Figure 2-6: Demand Diurnal Curves in the Pressure Zones

### 2.5 Design Guidelines and Level of Service Review

To update the Water Master Plan and the WaterCAD hydraulic model, the relevant water design criteria in the current City of Red Deer Engineering Services Design Guidelines (2020 Edition), referred to as the Design Guidelines hereafter, were reviewed with respect to the level of service (LOS) on the following elements:

- Water Demands
- Peaking Factors
- Fire Flows
- Operating Pressures
- Pipe Flow Velocities
- Pipe Friction Loss Coefficient (Hazen-Williams C)
- Distribution Main Sizes

As indicated in the 'Introduction' section of the Design Guidelines, the Guidelines are the technical information for the new lots, subdivision development or rebuilding of existing infrastructure. The developer might propose alternative designs due to unusual or complicated design situations. The City might initiate the request for the alternative criteria. In this Water Master Plan Update, the City would like to consider alternations in the short term to maximize the use of existing water infrastructure to service the proposed development north of Highway 11A such that the high capital investment of key infrastructure can be



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postponed. The subsequent sections will describe the criteria and the potential deviations, and the impacts from the deviations on the LOS for the elements listed above.

### 2.5.1 Water Demands

The water demand projections in the Design Guidelines are based on the land use types with the criteria shown below.

#### Residential Water Demands

For residential water demands, the following consumption rates are used per the Design Guidelines:

- Average day demand (ADD): 375 Lcpd
- Maximum day demand (MDD): 600 Lcpd
- Peak hour demand (PHD): 1200 Lcpd

For a typical density of 30 units per Ha and a 2.5 people per unit within the City, the typical 64 Ha quarter section will accommodate 4,800 people. The projected ADD in the typical residential quarter section development is 1,800 m<sup>3</sup>/day.

As indicated in Section 2.4.1, the current composite per capita ADD is 306 Lcpd, including all the residential and non-residential water use in the City. The calculated metered year by year composite per capita flow rate has a downward trend.

#### Non-Residential Water Demands

The Design Guidelines sets the minimum water demands of the non-residential (Commercial / Industrial / Institutional) to be 0.15 L/s/ha or 12,960 L/day/ha. For a typical quarter section development with a 64 Ha gross area, the minimum demand is 829 m<sup>3</sup>/day, which is equivalent to 2,211 people average demand in the quarter section.

The communities in the Hwy 2 corridor have similar minimum water consumption rates. E.g., the City of Lacombe Design Guidelines request a minimum 0.2 L/s/ha. However, the observed water consumption rates are normally lower than the minimum consumption rates in these communities as to most of the non-residential development in Hwy 2 corridor communities are light highway commercial or industrial types, e.g., machinery, retail, heavy equipment garages or parking lots with extremely limited water use. In the City, with the estimated developed area of 190 Ha in the Queens Business Park and a 24 L/s demand in 2021, the unit water consumption rate is at 0.126 L/s/ha.

The City might consider reducing the water consumption rate in the Hwy 11A development area as the area will mainly attract the light highway commercial or industrial developments. However, using too low a rate, say 0.05 L/s/ha, would not accommodate the potential larger industrial developments.

In the following existing water system analysis and future system planning, the existing water demands are assumed unchanged. For future developments, the minimum 0.15 L/s/ha can be used to size the



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arterial water trunks or the ones with difficult construction conditions, e.g., highway crossing or railway crossing to meet the ultimate development plan. A lower consumption rate, say 0.10 L/s/ha can be used to size the local watermains. Note that the fire flow might be the determining factor in the watermain sizing.

### 2.5.2 Peaking Factor Guidelines

The Design Guidelines suggest a MDD to ADD factor of 1.6 and a PHD to ADD factor of 3.2 for residential demands. In non-residential development, the peaking factor is  $P_f = 10Q^{-0.45}$  with a minimum of 2.5 and maximum of 25 range, where Q is the average demand in L/s.

Compared to the peaking factors as presented in **Table 2-5**, the MDD to ADD factors are less than the observed values in the pressure zones. However, the observed PHD to ADD factors in the WTPPZ, EHPZ+SPZ and NRD zones are less than 3.2 as defined in the Design Guidelines. Note that the WTPPZ consists of residential and non-residential developments (mixed land use type) while the developments EHPZ+SPZ are predominately residential and light commercial users.

In the QPZ, which is an industrial development area, the calculated  $P_f = 2.4$ . The minimum value of 2.5 should be used. A  $P_f$  of 2.5 is lower than the observed value of 4.7 in **Table 2-5**. Note that the Design Guidelines do not specify the MDD to ADD factor value. It is suggested an observed QPZ MDD to ADD factor of 1.9 can be adopted in the industrial development areas.

In the existing system analysis, it is suggested to use the observed peaking factors in **Table 2-5** in the hydraulic models. For the future developments, the peaking factors (MDD/ADD = 1.7 and PHD/ADD = 2.9) in the EHPZ and SPZ zones can be used in the residential zones.

### 2.5.3 Firefighting Flow Requirements

The Design Guidelines require a minimum 4500 L/minute (75 L/s) flow for single family residential subdivisions. The requirements for Fire Fighting (FF) flows are not clearly specified but should be determined by the architectural design elements. In most municipalities in Alberta, the firefighting flows are determined based on land use type and typical standard values, which are in accordance with the Water Supply for Public Fire Protection published by Fire Underwriters Survey (FUS 1999). **Table 2-6** presents the typical fire flow rates for residential land use type in Alberta.

**Table 2-6: Fire Fighting Flow Requirements**

Land Use Type	Required Fire Fighting Flow (L/s)
Single Family Residential	75
Multi-family Residential	180
Commercial, Industrial	233



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### 2.5.4 Pressure Requirements

**Table 2-7** presents the minimum and maximum allowable pressures at ground level during the various operating conditions as specified in the Design Guideline.

**Table 2-7: Normal Operating Pressures**

Operating Scenario	Minimum Pressure	Maximum Pressures
ADD, MDD, PHD	300 kPa (43 PSI)	700 kPa (101 PSI)
MDD + FF	150 kPa (21 PSI)	700 kPa (101 PSI)

\*MDD+FF = Maximum Day Demand plus Fire Flow scenario

The residual pressures required in daily operation (ADD, MDD and PHD) are set to ensure that the water fixtures in the buildings can operate effectively. Lower residual pressures lead to lower fixture flow and user complaints. However, slightly less than the 300 kPa pressure, say 250 kPa, in a low demand period, e.g., late night to early morning hours, is normally tolerated as it does not affect the LOS. The reservoirs can be filled during this period at a higher filling rate through the distribution system.

In a fire fighting scenario, the residual pressure of 150 kPa is critical to protect the distribution pipes from being overdrawn by the fire truck causing pipe collapse. Some municipalities use a slightly lower value such as 140 kPa for the firefighting scenario.

For maximum pressures, some municipalities with significant topographic elevation changes, e.g., City of Kelowna, allow the maximum pressure of 793 kPa (120 PSI) with a required service PRV to protect the plumbing system in the houses. City of Red Deer can use the 700 kPa (101 PSI) as the maximum pressure limit and use service PRV's to protect the building plumbing systems.

### 2.5.5 Flow Velocities

The Design Guideline specifies that the flow velocities should not exceed 1.5 m/s during peak hour flow conditions and 2.5 m/s during maximum day demand plus fire flow (MDD+FF) conditions in the distribution mains. The peak hour flow velocity limit is set to ensure that the watermains are sized appropriately to avoid excessive head loss such that the energy consumption in daily operation is efficient. In some communities, e.g., Strathcona County, they have a high flow velocity limit of 3.0 m/s under all peak flow scenarios.

The velocity constraint in the MDD+FF scenario is to mainly reduce the potential pressure surges that might cause damage to the pipe network, thrust blocks and other appurtenances in the distribution system. However, in a fire fighting event, the firefighters do not have the capacity to monitor the flow velocity but monitor the residual pressure at the fire pump suction. Therefore, it is recommended that the evaluation of the fire flows in the existing system or the short term planned network extension from the existing network do not have a velocity constraint. The velocity constraint should apply to the new pipes for the long-term planning of the watermains. A higher flow velocity of 3.0 m/s can be considered to meet the short-term PHD in the near future developments.



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Note that in the Queens Business Park servicing study reports completed by WSP in 2016-17, a relaxed MDD+FF flow velocity 3.0 m/s was accepted by the City.

### 2.5.6 Friction Loss Coefficient

In the hydraulic simulation required in the subdivision water servicing network planning and design, the specified Hazen-Williams roughness coefficient (C) for various pipe types are:

- HDPE = 150
- PVC = 140
- Asbestos Cement (AC) = 130
- Ductile Iron (DI) or Cast Iron (CI) = 80 to 100

Note that the DI or CI pipe can have a greater C value exceeding 100 if they are cement mortar lined. As for the most common material PVC pipe, although PVC pipe manufacturers indicated the C value can reach 155-165 from their inhouse experiments, AWWA (American Water Work Association) recommended a C=150 in friction loss calculation. For near term future planning (10 years), slightly higher C=150 values can be used for the PVC pipe.

### 2.5.7 Distribution Main Sizes

The minimum size of distribution mains shall be as follows:

- Residential = 150 mm diameter
- Industrial = 200 mm diameter
- 200 mm diameter or larger for an un-looped main with two hydrants

No more than 30 dwelling units shall be permitted to be serviced on an un-looped (dead end) section of watermain. Although the City had relaxed the 30-dwelling servicing off a single feed as an interim solution in the past, e.g., Garden Heights and Evergreen subdivisions, it is an exception and typically does not occur.

### 2.5.8 Design Criteria and Proposed Deviations Summary

The water design standards from the City of Red Deer Design Guidelines (2020) formed the basis for the analysis of the City's water distribution system and should be followed for the long-term servicing planning for new developments, including the future Highway 11A development. To maximize the use of the existing infrastructure capacity, some relaxations or deviations can be applied as discussed in the above **Sections 2.5.1 to 2.5.7**. The following table summarizes the key design criteria and the proposed deviations for providing adequate service levels to the new developments for the near-term future.



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**Table 2-8: Design Criteria and Proposed Deviation Summary**

Parameter	Guideline Value	Proposed Deviation (For Growth Areas)	Unit	Note
Average Daily Demand (ADD)	375	300	Lpcd	Deviations are for short term servicing only
Maximum Daily Demand (MDD)	600	480	Lpcd	MDD/ADD=1.6
Peak Hour Demand (PHD)	1200	870	Lpcd	PHD/ADD = 2.9
Non-Residential Unit ADD	0.15	0.10	L/s/ha	Deviation is for short-term planning without large water users
Max. Velocity in Pipe at MDD+ fire	2.5*	no limit	m/s	The near future available fire flow will be calculated without velocity constraint
Max. Velocity in Pipe at PHD	1.5	3.0	m/s	3.0 m/s is the limit for the near future flow
Hazen-William C for PVC pipes	140	150	-	Deviation for short term servicing only
Minimum pressure during daily flows	300	250	kPa	
Minimum residual pressure in the system during fire flow	150	150	kPa	Critical safety value, no deviation is proposed
Design fire flow for single family area	75	75	L/s	No deviation proposed
Design fire flow for medium density residential area	150	150	L/s	No deviation proposed
Design fire flow for industrial/commercial area	233	233	L/s	No deviation proposed

Note: \* City accepted 3.0 m/s as a relaxed standard in the Queens Business Park development servicing studies completed by WSP in 2016.



### 3 Existing Model Updates and Calibration

#### 3.1 Model Updates

The hydraulic model network constructed in the 2013 Water System Study was updated based on the as-built drawings or GIS data supplied by the City. The as-built data covers the new watermain additions and the pump station upgrades completed from 2013 to 2021 including the following components:

- New watermains in the new subdivisions or infill development areas in Evergreen, Garden Heights, Clearview, Timberlands, Timberstone, Timber Ridge, Laredo, Vanier, Capstone and Queens Business Park;
- Pumping system upgrades in Mountview Reservoir and Glendale Reservoir;
- The 19 Street trunk and PRV addition;
- The installation of the 600 mm HDPE DR11 pipe across the Red Deer River, replacing the aged 200 mm and 400 mm river crossing pipes; and
- The addition of the newly purchased portion of the NRDRWSC regional line up to the new pressure sustaining valve vault and the mains in the Central Park area.

The 2013 and 2022 existing systems are both presented in **Figure 3-1** for comparison.





## **3.2 Calibration Data Review**

### **3.2.1 Hydrant Flow Testing**

City staff conducted numerous hydrant flow tests in 2021 and 2022. For each of the tests, three hydrants were used: one static hydrant and two flow hydrants. The flow hydrants were connected to the BigBoy Hose Monster hydrant flow diffusers through the 4" port. The BigBoy Hose Monster assembly was equipped with a pressure-flow gauge to measure the flow discharged out from the flow hydrant. The test staff also installed a Pollard water pressure gauge to the 2.5" port of the static hydrant to measure the residual pressures during the flow tests. The hydrant flow tests were carried out in the following steps:

- Install the gauges and Hose Monster to the static and flow hydrants;
- Read the residual pressure in the static hydrant and record the time;
- Open the first flow hydrant and record the pressure-flow gauge and the time; read and record the residual pressure at the static hydrant;
- Open the second flow hydrant and record the pressure-flow gauges and the time at both hydrants; read and record the residual pressure at the static hydrant; and
- Close the first flow hydrant and continue the second flow hydrant flow. Record the pressure-flow gauge and the time at the second flow hydrant; read and record the residual pressure at the static hydrant.

### **3.2.2 Model Boundary Conditions**

The existing distribution system consists of five reservoir pump stations and one booster pump station. Although most of the pumps are driven by Variable Frequency Drives (VFD) and can be programed to automatic operating mode with certain operation set points, some of the reservoir pump stations are operated manually, e.g., Glendale pump station, Lancaster pump station, and Mountview pump station. The manually operated pumps in the stations are remotely selected to run at a certain speed or stopped by the operators from the WTP Supervisory Control and Data Acquisition (SCADA) system to meet the peak demands in the distribution system. Running the pumps in these pump stations is at the operators' discretion.

The operations schematic increases the complexity in defining the model boundary condition. The flow rate being discharged to the North Red Deer River Water Service Commission (NRDRWSC) regional line is also fluctuating from time to time. The WTP SCADA system records the flow rates, pressures, pump status and pump speeds in all the pumping facilities and in the NRDRWSC meter chamber. With the fine resolution (min. five-second interval) recording capacity, the flow rates, pressures, pump speeds can be quantified in the pumping facilities and in the NRDRWSC meter chamber if the hydrant flow test time stamp is recorded in field. The modeling boundary conditions for each of the hydrant flow tests can be then defined based on the SCADA records.

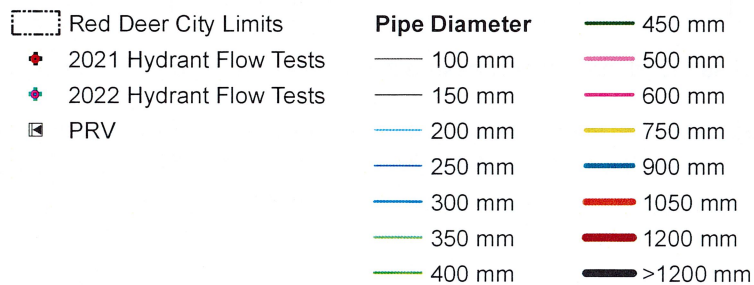
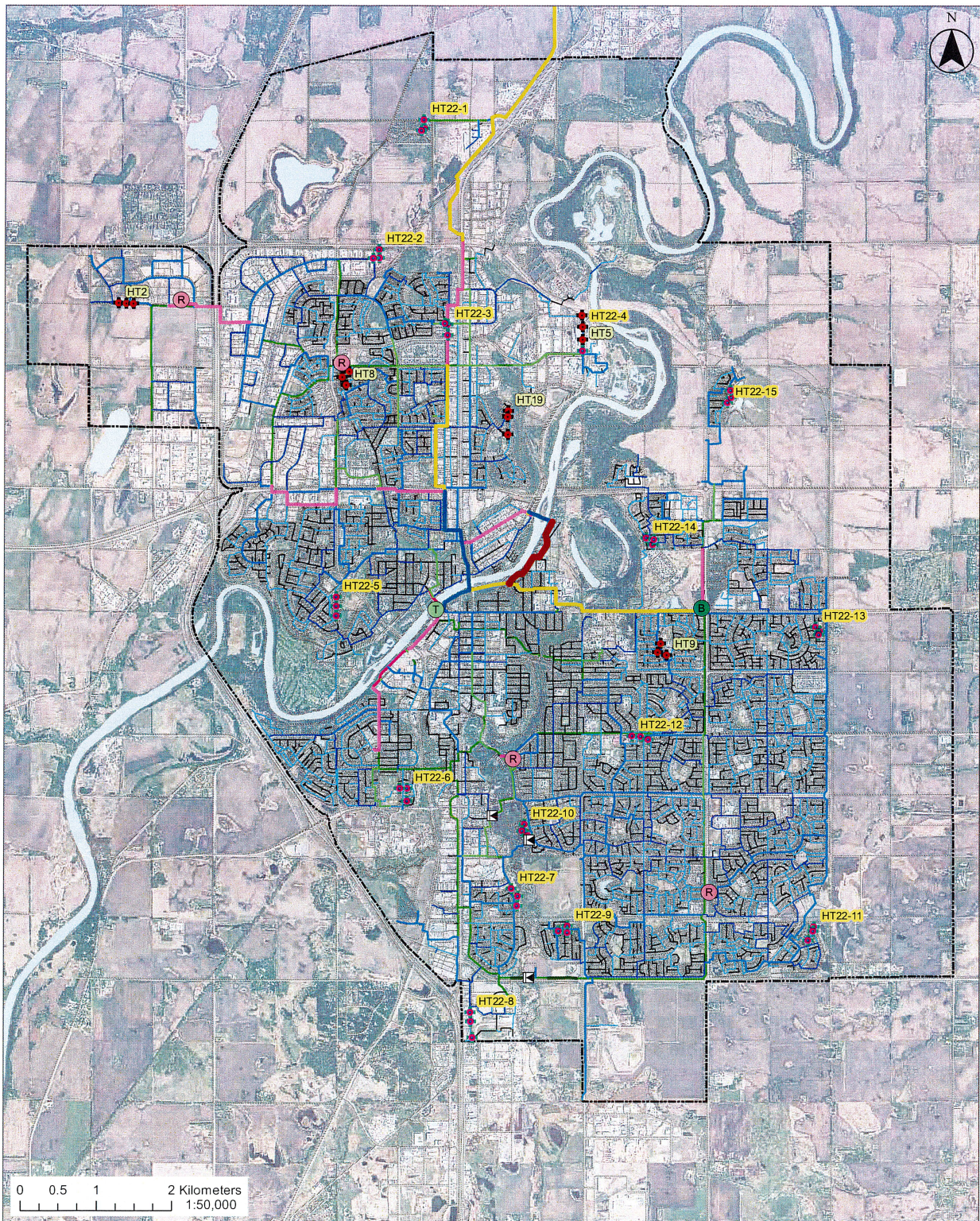
For hydrant tests without the test time recorded on the day of test, uncertainty is brought in the boundary condition settings. E.g., for a test in the EHPZ, the flows can come from Clearview pump station and from



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Mountview pump station or from Clearview and Lancaster pump station. The two different combinations will lead to different boundary conditions and different calibration results. On April 28, 29, May 19 and Aug 17, 2022, fifteen (15) hydrant flow tests were conducted and recorded with the time stamps. Another five (5) hydrant test results from 2021 are considered suitable in the model calibration as their conditions can be determined without the time stamp. The locations of the hydrant flow tests that are utilized in the model calibration are presented in **Figure 3-2**.





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### **3.3 Model Calibration**

#### **3.3.1 Calibration Methodology**

As the first step of the model calibration, the static hydrant pressure readings were utilized to verify the HGL elevation settings in the pressure zones. The field measured HGL elevation of the static hydrant was calculated by adding the static hydrants ground elevation to the static pressure recorded before and after the hydrant flows. In the hydraulic model, the demands in the pressure zones were adjusted so that flows from pump stations matched (+/- 10%) the flow data recorded in SCADA before the flow hydrants were opened. Ideally, the static pressure difference, defined as the modeled value subtracting field measured value, should be within 5% of the field measured value. This step is necessary to verify the location and elevation of the static hydrant and the HGL elevation set point in the pressure zone.

The verified static condition was then utilized to calibrate the network with both hydrants opened. The field measured flow at each hydrant was input into the model. The boundary conditions in the static scenario were updated according to the SCADA data, including the pump running status and speed, flows, and pressure at the discharge headers and in the NRDRWSC Pressure Sustaining Valve (PSV) chamber. In addition, the modeled demands in the pressure zones were also adjusted so that the flows passing through the headers and the valve chamber can match (+/- 10%) the SCADA data. The modeled residual pressure at the static hydrant was compared to the field measured residual pressure at the static hydrant. Adjustments were made to the pipes, including the pipe diameters where applicable and C values, to minimize the difference between the two pressure values (modeled value vs field measured value). The goal for the calibration is to be within a maximum difference of 10% between the two values.

#### **3.3.2 Model Verification and Calibration Results**

In the HGL verification, it was found that the SPZ HGL was set at 920 m instead of the nominal 930 m based on the HT22-7 and HT22-8 field measured static readings. This was confirmed with the pressure set points at the three pressure reducing valves (PRV) noted in the following table.

**Table 3-1: South Pressure Zone PRV Set Points**

<b>Location</b>	<b>Pressure Out Set Point (PSI)</b>	<b>Pressure Set Point (kPa)</b>	<b>PRV Elevation (m)</b>	<b>HGL Elevation (m)</b>
19 <sup>th</sup> Street PRV	60	413.7	882	921.2
47 <sup>th</sup> Street Avenue PRV	62	427.49	879	919.6
Selkirk PRV	60	413.7	880	919.2

The verification process found that other pressure zones are operating at their nominal HGL set points.

With the verified HGL setpoints, scenarios of the hydrant flow conditions are added into the hydraulic model to calibrate the model. The results of the calibration, i.e., after the necessary adjustment to the C values in the existing watermains, are presented in the following table.



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**Table 3-2: Model Calibration Results Field Measurements vs Hydraulic Model**

Test No.	Test Date	Subdivision	Pressure Zone	Static Pressure Difference	Residual Pressure Difference	Model Adjustments in Calibration and Comments
HT2	2021 Oct 1	Queens Park	QPZ	-3%	-9%	No adjustment is needed; three pumps are running in QBP pump station.
HT5	2021 April 12 or 13	Riverside Heavy Industrial Park	WTPPZ	0%	10%	No adjustment is needed
HT8	2021 April 12 or 13	Glendale Park Estates	EHPZ	-7%	0%	No adjustment is needed
HT9	2021 April 12 or 13	Clearview Meadows	EHPZ	-3%	2%	No adjustment is needed; assuming two pumps running in CLV
HT19	2021 April 12 or 13	Pines	WTPPZ	-6%	6%	No adjustment is needed; assuming one pump running in GLD
HT22-1	2022 April 28	Central Park	WTPPZ	1%	7%	Changed the 400 mm dia. HDPE DR11 main to have a 327 mm I.D.
HT22-2	2022 April 28	Edgar Industrial Park	WTPPZ	0%	-9%	Increased the C values in the pipe set "HT22-2pipeschanges"
HT22-3	2022 April 28	Kenwood East	WTPPZ	3%	2%	No adjustment is needed.
HT22-4	2022 May 19	Riverside Heavy Industrial Park	WTPPZ	-2%	0%	No adjustment is needed.
HT22-5	2022 April 28	Oriole Park	WTPPZ	2%	-10%	Increased the C value in the "Fairview+Riverview+OldOriole" selection set in the model.
HT22-6	2022 April 28	West Park	WTPPZ	-4%	-6%	Slightly increased the "HT22-6pipeschanged" selection set AC pipe C value to 110.
HT22-7	2022 April 28	Bower	SPZ	1%	-3%	Corrected the SPZ HGL from 930 to 920. No adjustment is needed.
HT22-8	2022 May 19	Westerner Park	SPZ	-4%	-4%	Corrected the SPZ HGL from 930 m to 920 m. No adjustment is needed.
HT22-9	2022 April 28	Sunnybrook South	EHPZ	0%	-9%	Increased the HT22-9pipeschanged C values to normal PVC 140, some pipes are at 150. The Lancaster pump1 ran briefly as per the SCADA data.
HT22-10	2022 April 28	Sunnybrook	EHPZ	3%	8%	Increased the "HT22-10pipeschanged" selection set C value to 130.



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Test No.	Test Date	Subdivision	Pressure Zone	Static Pressure Difference	Residual Pressure Difference	Model Adjustments in Calibration and Comments
HT22-11	2022 Aug 17	Vanier Woods	EHPZ	3%	-5%	Increased the C to 140 in the "HT22-11 pipes changed.
HT22-12	2022 April 29	Eastview	EHPZ	2%	-4%	No adjustment is needed.
HT22-13	2022 April 29	Rosedale Meadows	EHPZ	-6%	7%	No adjustment is needed.
HT22-14	2022 May 19	Clearview Ridge	EHPZ	-3%	-4%	No adjustment is needed.
HT22-15	2022 Aug 17	Evergreen	EHPZ	7%	-7%	Assigned C=155 in the 300 mm PVC main. The PVC pipes in Evergreen and Timberland North C values are set at 150.

**Notes:**

- Hydrant Flow Tests with prefix "HT" refer to the tests conducted in 2021; Hydrant Flow Tests with prefix "HT22-" refer to the tests conducted in 2022.
- Static pressure refers to the pressure recorded in the static hydrant before and after the hydrant flow.
- Residual pressure refers to the pressure recorded in the static hydrant gauge when the two flow hydrants are both opened.
- Pressure difference refers to Model to the Field Difference, i.e. (Modeled Value - Field Measured Value)/Field Measured Value. A negative difference value implies that the modeled pressure in the WaterCAD hydraulic model is lower than the pressure readings recorded by the test crew in field. A positive difference value implies the modeled pressure is higher than the measured value in field.

As indicated in **Table 3-2**, 16 of the 20 static pressure differences are less than 5%, however the difference of the H8, H19, HT22-13 and HT22-15 static pressure are more than +/-5% which are not ideal but within the +/- 10% range.

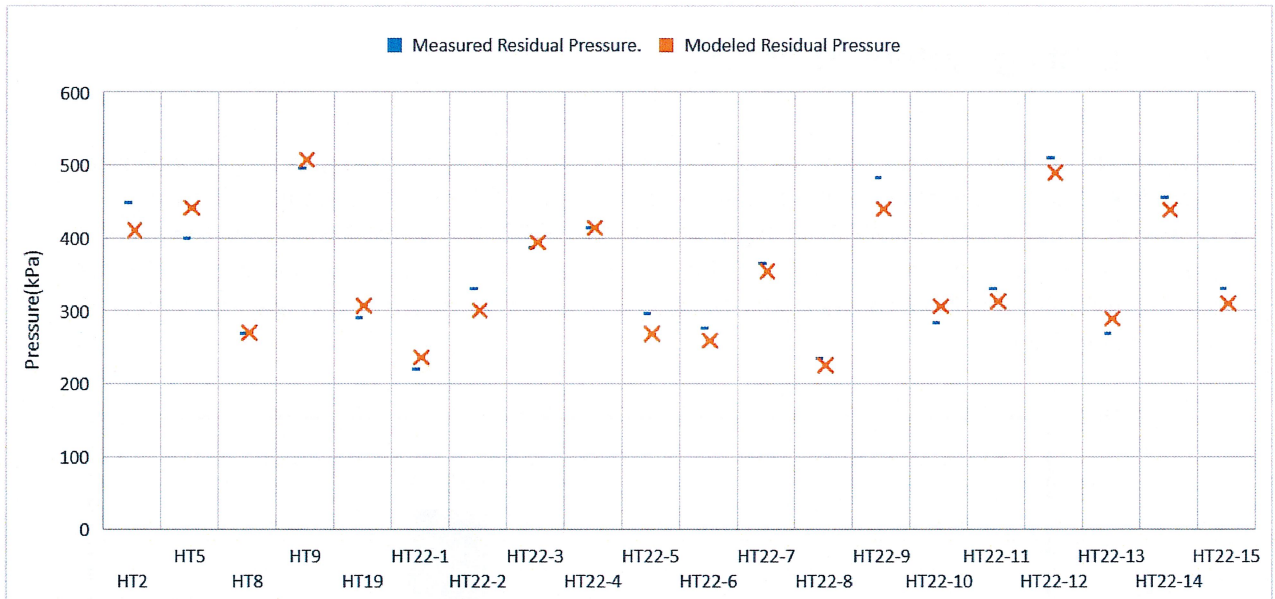
**Table 3-2** also presents the residual pressure difference for the 20 locations for the two hydrants flow scenarios. With the adjustments to the pipe properties, most of the residual pressure differences can meet the calibration goal to be within +/-10%.

The field measured records of the hydrant flow tests, calibration results, and the detailed boundary condition set up for the calibration tests are presented in **Appendix A**.

The hydrant test results and the model values with two open flow hydrants are presented in the following figure.



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**Figure 3-3: Hydrant Test and Model Calibration Results**

With the calibration results meeting the goal of a maximum 10% difference, the calibrated hydraulic model can be utilized for existing distribution system hydraulic performance evaluation and future distribution system planning.



## 4 Existing System Analysis

This chapter presents the results of the existing water distribution system analysis and the identified recommendations for improvements. The analysis included a review of the system's hydraulic performance during ADD, MDD, PHD, Night Fill (NF), and MDD with fire flow conditions based on the 2021 water demands outlined in **Table 2-5**. Each of these flow scenarios were simulated in WaterCAD utilizing the calibrated model developed in **Section 3**. The total water demand for each scenario is presented in **Figure 4-1**.

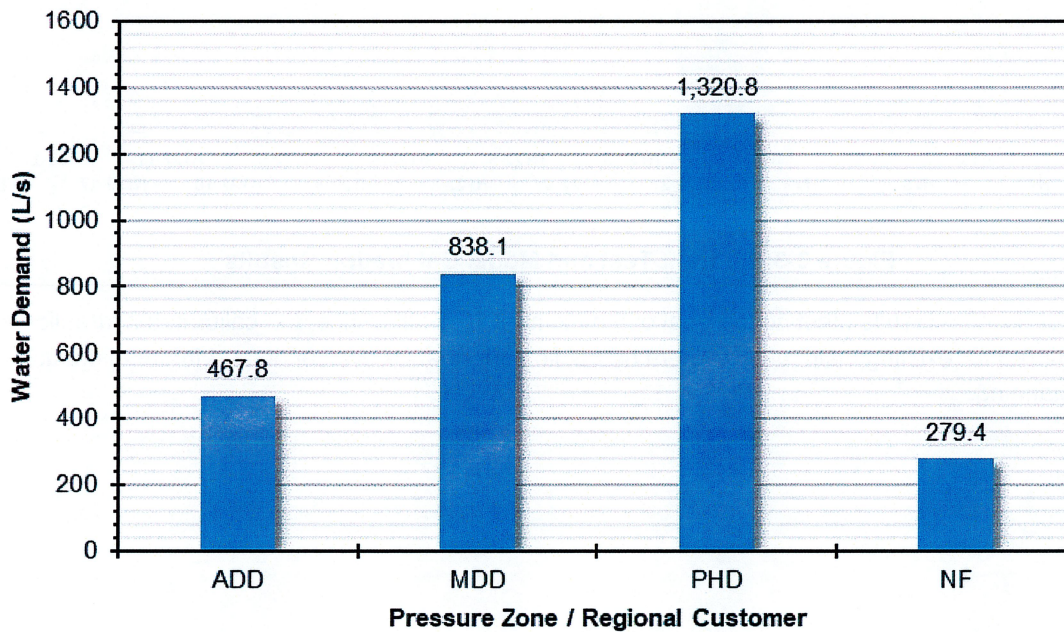


Figure 4-1: Total 2021 Water Demands for Existing System

Figure 4-2 shows the distribution of demands in each of the pressure zones for all the demand scenarios for the existing system analysis.



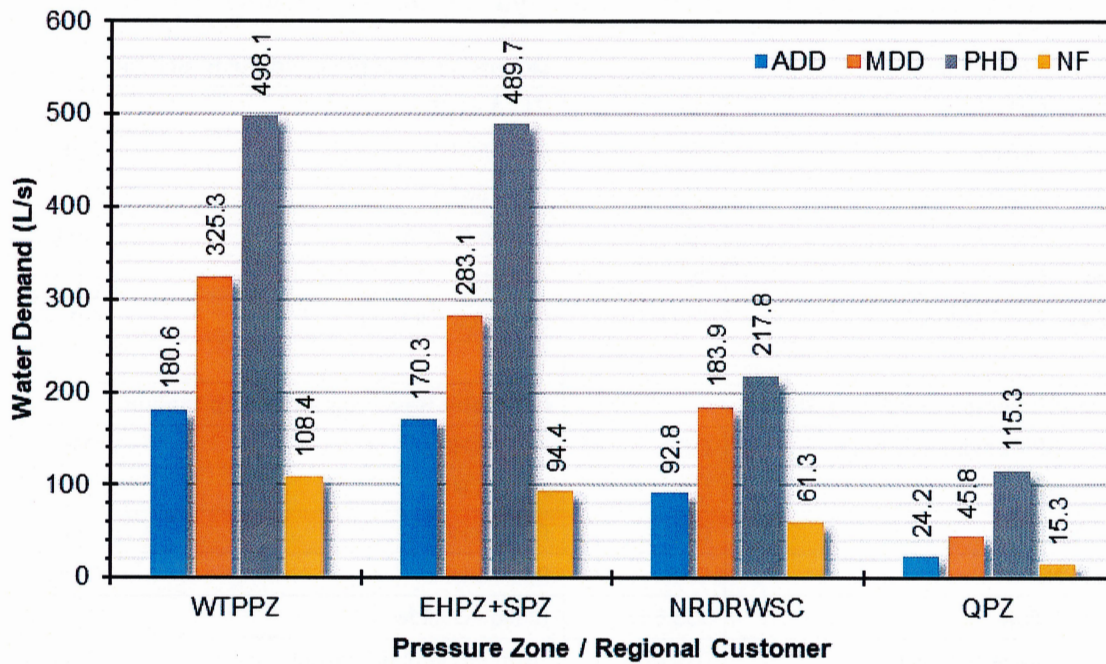


Figure 4-2: Existing System 2021 Water Demand Summary

Summaries for the analysis of each of these scenarios are provided in the following sections.

#### 4.1 Existing System Average Day Demand Analysis

A summary of the boundary conditions used in the WaterCAD hydraulic analysis for each facility in the water distribution system for the average day demand scenario is shown in **Table 4-1**.



**Water Model Update  
Existing System Analysis**

**Table 4-1: Existing System ADD – WaterCAD Hydraulic Analysis Boundary Conditions**

<b>Water Treatment Plant</b>		<b>Mountview Reservoir and Pump Station</b>	
HLP 101 - 900 HP	OFF	P1 - 150 HP	OFF
HLP 102 - 700 HP	ON	P2 - 150 HP	OFF
HLP 103 - 350 HP	OFF	P3 - 150 HP	OFF
HLP 104 - 900 HP	OFF	Station Outflow	0.0 L/s
Station Outflow	360.1 L/s	Hydraulic Grade Setpoint	940 m
Hydraulic Grade Setpoint	919 m <sup>1</sup>	Reservoir Fill Rate	0.0 L/s
<b>Glendale Reservoir and Pump Station</b>		<b>Lancaster Reservoir and Pump Station</b>	
P1 - 200 HP - Emergency Pump	OFF	P1 - 75 HP	OFF
P2 - 125 HP	ON	P2 - 75 HP	OFF
P3 - 125 HP	OFF	P3 - 75 HP	OFF
P4 - 125 HP	OFF	P4 - 75 HP	OFF
Station Outflow	133.5 L/s	Station Outflow	0.0 L/s
Hydraulic Grade Setpoint	919 m	Hydraulic Grade Setpoint	940 m
Reservoir Fill Rate	0.0 L/s	Reservoir Fill Rate	0.0 L/s
<b>Clearview Booster Station</b>		<b>Queens Business Park Reservoir and Pump Station</b>	
P1 - 125 HP	ON	P1 - 75 HP	ON
P2 - 125 HP	OFF	P2 - 100 HP	OFF
P3 - 125 HP	OFF	P3 - 150 HP	OFF
P4 - 125 HP	OFF	P4 - 150 HP	OFF
Station Outflow	170.3 L/s	Station Outflow	24.2 L/s
Hydraulic Grade Setpoint	940 m	Hydraulic Grade Setpoint	950 m
		Reservoir Fill Rate	50.0 L/s <sup>2</sup>

<sup>1</sup> The original design of the WTPPZ was based on a HGL setpoint of 919 m. The setpoint was recently set to 917 m to reduce the non-revenue water demand (i.e., water losses). Both set points were simulated in the WaterCAD hydraulic modeling, but the original setpoint (i.e., 919 m) showed better performance in meeting the LOS described in **Table 2-8**.

<sup>2</sup> The City indicated in their "2021 Updates to the Existing Water System Model" technical memorandum that the Queens Business Park has been filling continuously.

A summary of the system pressures and velocity in the existing system during the ADD scenario is presented in **Figure 4-3** and **Figure 4-4**.

