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REQUEST FOR PROPOSAL
Sale and Development of Housing on Municipal Lands
Hudson Street, Debert, Nova Scotia

ADDENDUM #3

Issued November 06, 2026

3. THE PROPERTIES

Stormwater

The following stormwater management standards must be met for all development lots in Colchester County.

For the 1:10 and 1:100 design storms, stormwater run-off from development sites must demonstrate pre and post development balancing across:

- Peak flow rate,
- Total volume, and
- Water quality.

The Hudson Street frontage multi-use path will be constructed with cross culverts at each driveway to convey the pre-development stormwater run off to the Provincial roadside drainage network. The multi-use path cross culverts should not be used as a flow control mechanism for the development sites.

Wastewater

The Hudson Street wastewater sewer extension ends at Manhole DEMH 2430, as labelled on the Hudson Street Sewer & Trail tender drawings. Manhole DEMH 2430 is located in a 20m future road reserve. The successful proponent will be responsible for running a private sewer lateral from Manhole DEMH 2430 to service Lot C internally as needed to best suit their development plans. The Municipality will not entertain extensions of the public wastewater sewer along the Hudson Street frontage of lot C but will consider proposed extensions that run in the future road reserve.

Any extensions to the public wastewater sewer system would be the responsibility of the successful proponent for design and installation. Upon satisfactory installation, the infrastructure would be turned over to the Municipality.

The Hudson Street sewer extension and Tracker Road Lift Station were designed to accommodate a development density of 15.5 units per acre for the full 41.5-acre Parent Parcel from which Lots A, B, & C have been preliminary subdivided. The design also included consideration of a less dense development pattern of 7units per acre on the 70.7-acre Parcel on the west side of Hudson. At this time, the Municipality has no plans to peruse development on the west side of Hudson, so this capacity is available for reallocation.

Proponents should refer to Section 7.1 Evaluation Criteria for guidance on development patterns and density, the items listed there will be the only metrics used to evaluate Proposals.

Fire Flow Water Pressures

The Municipality has engaged a consultant to develop a plan for improved fire flow delivery in the Debert Water Utility. This work is in the design stage, and proponents should be prepared to base their fire suppression design on the information provided in the 2023 Capacity Report provided with this addendum.

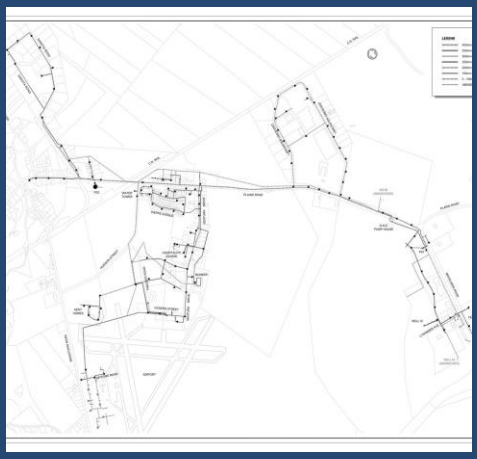
ATTACHMENTS

1. Colchester Capacity Studies – Debert Water Capacity Assessment - March 25, 2024

END OF ADDENDUM #3

FINAL REPORT









March 25, 2024

Colchester Capacity Studies – Debert Water Capacity Assessment

DesignPoint Project #23-396

SUBMITTED BY:

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Colchester Capacity Studies – Debert Water Capacity Assessment

Issued For	By	Date
Draft Report	AJS	January 22, 2024
Final Report	AJS	March 25, 2024
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This report was prepared by DesignPoint Engineering & Surveying Ltd & Dillon Consulting Ltd for the Municipality of the County of Colchester using the care and skill ordinarily exercised by members of the engineering profession currently practicing under similar circumstances on similar projects in Nova Scotia. Any use of this report by third parties is done at their own risk. DesignPoint accepts no responsibility for damages as the result of third-party use of this document or any portion thereof.

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1.0 INTRODUCTION

The Municipality of the County of Colchester (the Municipality) has retained DesignPoint Engineering and Surveying Limited (DesignPoint) along with Dillon Consulting Limited (Dillon) to review existing water and wastewater systems in order to determine current capacity. This report covers the existing water system in Debert, Nova Scotia, the extents of which are shown in Figure 1-1.

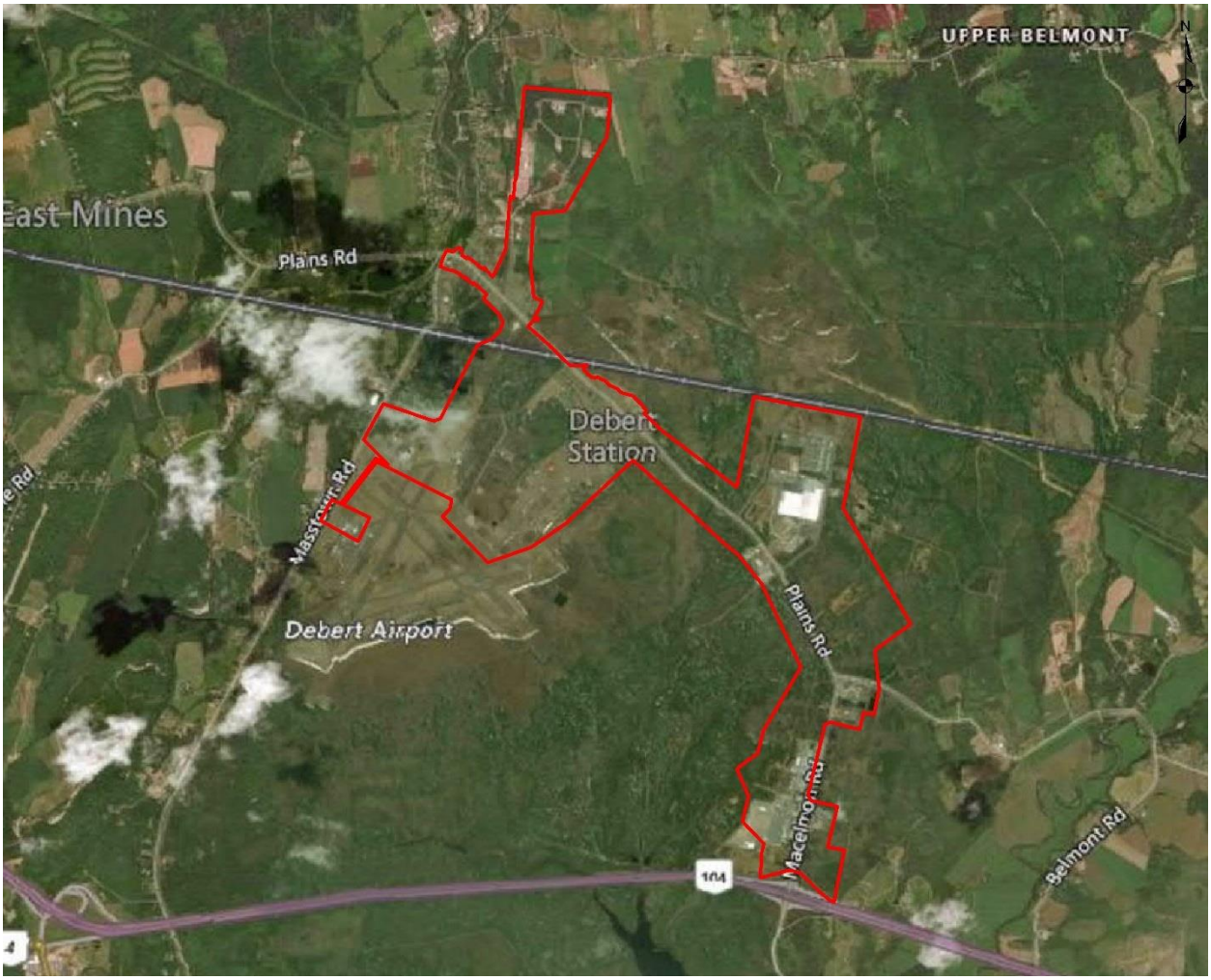


Figure 1-1: Debert Water System Extents

The purpose of this project is to evaluate the functionality of the existing water system in Debert, as well as to evaluate the existing system's ability to accommodate new developments, including identifying capacity limitations within the system. The RFP highlighted two future developments within the community of Debert: a 528-unit residential development on Hudson Street, and the new Mi'kmawey Debert Cultural Center (MDCC) on Plains Road. Locations of the identified future developments can be seen in Figure 1-2. As part of the capacity assessment, DesignPoint and Dillon have identified upgrade options for the system to facilitate growth within the serviced area of Debert.

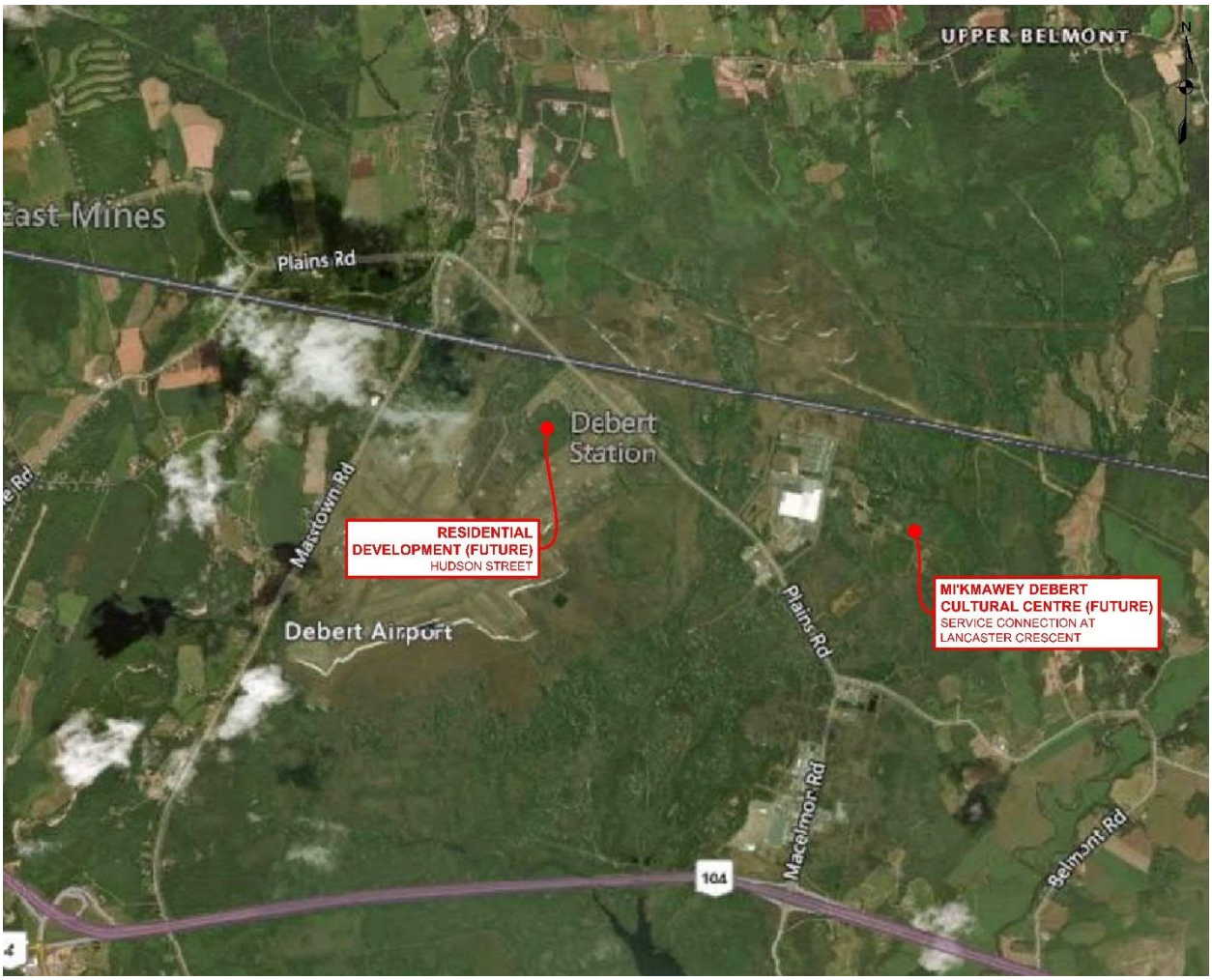


Figure 1-2: Proposed Future Development Locations in Debert

2.0 BACKGROUND

The community of Debert has strong historical ties with the Canadian Armed Forces, with notable military presence in Debert dating back to the beginning of the Second World War. The oldest of the municipal water servicing in the community was established in the early 1940's to service the Camp Debert army facility on Ventura Drive.

Canadian Forces Station Debert was closed in 1996, effectively ceasing military activity in the community. Following this, military lands and facilities were decommissioned and transferred to government or development authorities, primarily for the purpose of commercial and industrial development. These decommissioned lands are a significant piece of what is known today as the Debert Business Park. A map showing the extents of the Debert Business Park can be seen in Figure 2-1.

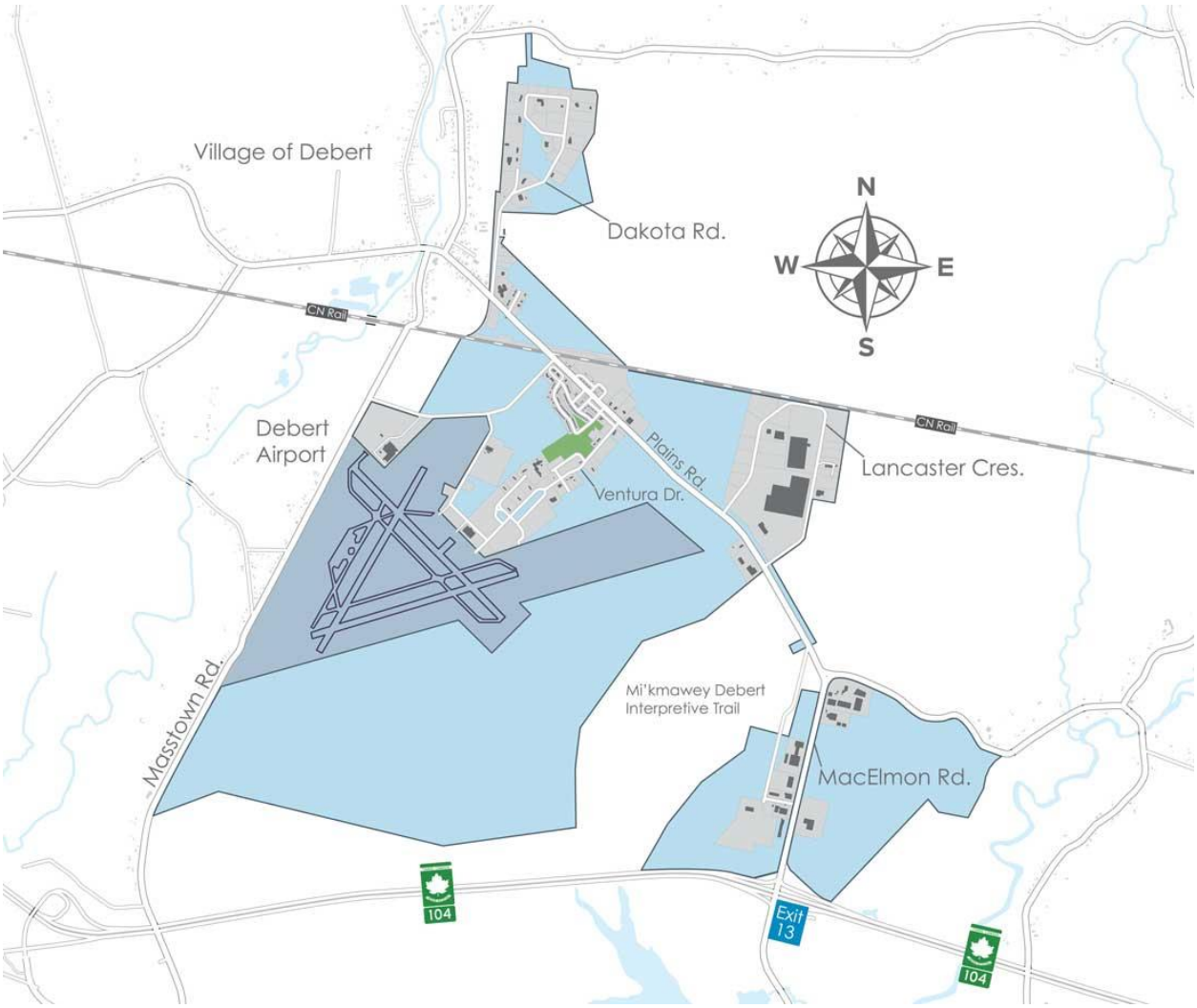


Figure 2-1: Debert Business Park Map (Courtesy of Debert Business Park, https://debertbusinesspark.ca/our-businesses/)

Today, land use in Debert consists of low and high-density residential areas, as well as commercial, institutional, and industrial uses. The community of Debert has an approximate population of 1,471 as of 2006, and it is estimated that approximately 549 people are serviced by the Debert water system. There is a strong industry presence in Debert, with future industrial and commercial growth facilitated through the Debert Business Park organization. Notable areas of the business park include the MacElmon-Lysander-Canso area, Lancaster Crescent, the Ventura-Tracker-Vickers area (lands previously owned by the Department of National Defense), the Debert Airport, and Dakota Road.

As the term "industrial land use" can refer to a wide range of activities and processes, industrial operations are often classified as either light industrial or heavy industrial based on the level of resource consumption (i.e. water, electricity) during industrial processes. It is understood that industrial applications in Debert range from light industry (i.e. warehouses) to heavy industry (i.e. waste management services) with respect to water consumption.

2.1 Existing System

Based on GIS data provided by the Municipality, the Debert water distribution network consists of approximately 27.8 kilometers of active water pipe and 113 fire hydrants. At the time of this report, municipal water in Debert is sourced from two (2) groundwater wells. Both wells have disinfection measures in place, where the disinfection of raw groundwater is achieved via injection of sodium hypochlorite. At key points in the system for treatment and distribution such as control buildings and water storage sites, flow data and water quality parameters are monitored and managed using supervisory control and data acquisition (SCADA) software. Municipal water is currently stored at two (2) locations in Debert: a water tower on Hudson Street, and a low-elevation storage reservoir on Plains Road. The locations of existing key water infrastructure can be seen in Figure 2-2. A schematic of the existing water system is included in Appendix B.

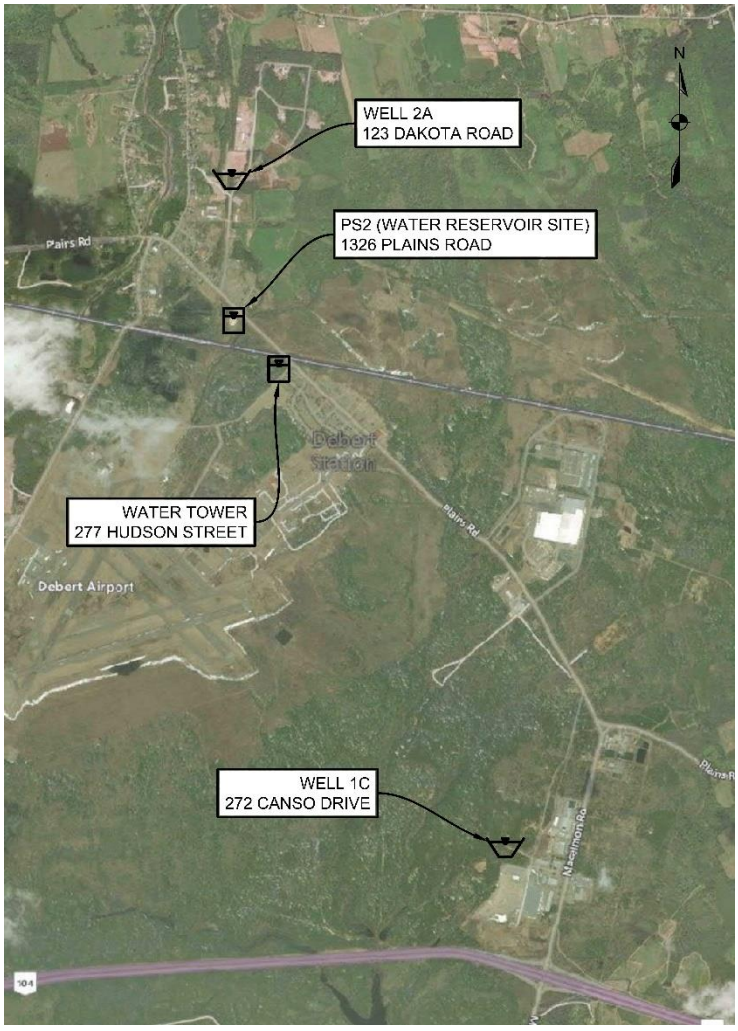


Figure 2-2: Storage Facility and Well Locations in Debert

One of the active wells that supplies the community is "Well 2A" located at 123 Dakota Road, shown below in Figure 2-3.

Chlorine-dosed water leaves Well 2A through a dedicated 100 mm water line along Dakota Road that is directly connected to the water storage reservoir on Plains Road. It is understood that Well 2A only functions to fill the water storage reservoir on Plains Road and does not provide a direct supply to the distribution system.

The Plains Road water reservoir, referred to as "PS2", mainly provides backup fire protection for the system via an existing diesel fire pump and does not typically supply water for domestic use. However, it is understood that the storage reservoir is periodically drained into the distribution system to provide the necessary turnover to prevent the stored water from becoming stagnant, as per the Debert Water Utility Operations Manual (last updated May 2022). PS2 is shown below in Figure 2-4. PS2 was constructed in 1979 and is approximately 3.7 meters (m) tall and 39.9 meters in diameter. The PS2 pump house is outfitted with a jockey pump that recirculates water in the reservoir when the reservoir is offline. Based on conversations with municipal staff, the existing fire pump at PS2 is manually operated and does not turn on automatically during fire flow conditions. Additionally, municipal staff have indicated that activation of the fire pump can result in high pressures throughout the distribution system.



Figure 2-3: Existing Well 2A, 123 Dakota Road



Figure 2-4: Existing PS2, 1326 Plains Road

The second active well supplying the municipal water system in Debert is "Well 1C" located at 272 Canso Crescent. Well 1C is shown below in Figure 2-5. Well 1C was constructed in 1995 and operates as the main domestic supply for the municipal water system. Well 1C connects directly to the municipal distribution system through an existing 350 mm water main near the intersection of Lysander Avenue and Canso Crescent. Prior to connection to the distribution system, chlorine-dosed water moves through a serpentine 200 mm water line with a total length of approximately 410 m.

It is understood that there is a submersible pump at Well 1C which operates based on the water level in the existing water tower at 277 Hudson Street. The water tower was constructed in 2018 and is approximately 27.5

meters tall and 12.7 meters in diameter. Based on design drawings for the water tower provided by the Municipality and prepared by EXP (Released for Construction, July 2017), it is understood that there is a single 200 mm line that both fills and drains the water tower. Currently, there are no on-site water disinfection measures implemented at the water tower, however the water tower is equipped with the capability for on-site chlorine dosing if necessary. The Hudson Street water tower is shown in Figure 2-6.



Figure 2-5: Existing Well 1C, 356 Lysander Avenue



Figure 2-6: Existing Water Tower, 277 Hudson Street

As previously noted, treated groundwater from Well 1C flows into an existing 350 mm water main that runs along Canso Crescent and eventually connects into an existing 400 mm water main near the intersection of Plains Road and MacElmon Road. This 400 mm water main extends approximately 3.6 km northwesterly along Plains Road from MacElmon Road to Dakota Road, linking the northernmost and southernmost extents of the Debert municipal water system. The backup fire supply PS2 connects directly into the 400 mm water main at 1326 Plains Road. It is noted that part of the 400 mm water main along Plains Road is downsized to 300 mm for a length of approximately 480 m between the intersections of Lancaster Crescent.

The existing 400 mm water main provides water service to multiple branches of the network, including the Lancaster Crescent area. This area receives service via existing 300 mm water mains. There is also an existing 300 mm water main along Hudson Street extending southwest to Tracker Road, which connects into the Debert Airport and Ventura business park branches of the system and supplies water to 70 Hudson Street. All other serviced areas, including Dakota Road, the Canso-Lysander-MacElmon area, the Ventura business park, and residential areas, are serviced via 150 mm, 200 mm, and 250 mm water mains. Existing water main sizes and approximate locations are shown in the attached water schematics in Appendix B.

The Debert water system is comprised of a single pressure zone. During domestic demands, water is mainly supplied to the system from the Hudson Street water tower, from which potable water flows by gravity into

the distribution system. When the water tower is supplying the system, the water level in the tower influences the hydraulic grade line (HGL) of the system. Based on provided design drawings for the water tower (EXP, July 2017) and hydrant flow test results, it is indicated that the approximate hydraulic grade line of the Debert water system is 84 m (276 ft). The HGL will fluctuate slightly depending on the water level in the tower.

It is noted that certain circumstances may result in a change in the HGL, and subsequently pressures in the system. The activation of pumps would impact the HGL of the system, depending on the head delivered by the pump. This could occur when the Hudson Street water tower is filling and the submersible pump at Well 1C is supplying water to the system. This could also occur when PS2 is active and supplying the system, either through activation of the fire pump during a fire or activation of the jockey pump to circulate stored water into the distribution system.

3.0 WATER DISTRIBUTION SYSTEM

3.1 Model Schematic

A hydraulic water model was developed for the Debert water distribution system using Haestad Methods' WaterCAD/WaterGEMS. The hydraulic model was constructed by placing junctions at key locations throughout the study area (i.e. intersections, high/low points) complete with elevations based on the existing grade. Junctions were connected with pipes, and each pipe was assigned a diameter, length, and roughness coefficient. A steady-state hydraulic water model was created for this analysis. A schematic of the Debert water system can be seen in Figure 3-1.

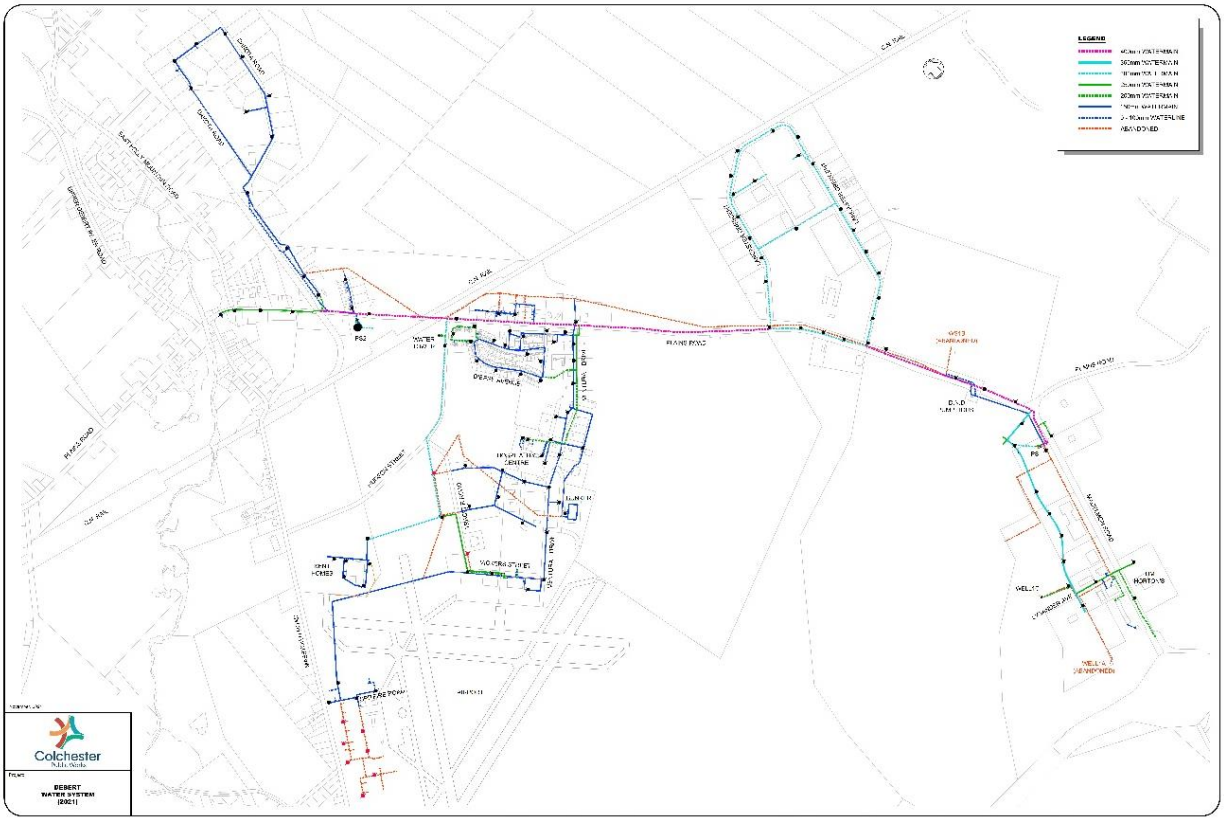


Figure 3-1: Debert Water System Schematic (Courtesy of Colchester Water, https://www.colchester.ca/water)

It is understood that during domestic demands, the Hudson Street water tower typically serves as the main source of potable water for the community with the submersible pump at Well 1C operating based on the water level in the tower. Due to the pump at Well 1C only turning on periodically, the steady-state hydraulic model was developed with the Hudson Street water tower acting as the singular source of water supply for domestic demand scenarios (average daily demand, maximum daily demand, peak hourly demand).

In the hydraulic model, different water levels in the tower were used depending on the domestic demand scenario to represent changing system conditions. During the average daily demand scenario, the water tower was modelled with a water level of 84.0 m to represent typical conditions and the approximate hydraulic grade

line of the system. During the peak hourly demand scenario, the water tower was modelled with a water level of 81.3 m in elevation, as per the low domestic water level identified in the tower design drawings (EXP, July 2017). Using the low water level provides model results which represent the lowest expected pressures.

As previously detailed, the diesel fire pump at PS2 is manually operated on an as-needed basis. To reflect the actual fire flow response from the Debert water system, the low-level storage reservoir PS2 was also incorporated into the model as a storage tank. When PS2 is engaged during fire flow scenarios, stored water from PS2 is pumped into the existing 400 mm water main on Plains Road via the existing diesel fire pump. This pump was added to the hydraulic model and defined based on the pump curve for the existing diesel fire pump at PS2. The water level in PS2 was assumed to be 49.8 m, based on an expected drawdown of approximately 2.0 m after providing a maximum fire flow of 13,620 L/min for the required duration of three (3) hours. For all non-fire flow scenarios, Well 2A and PS2 are inactive as they do not typically provide domestic supply to the network.

The existing water system has more than one potential fire flow supply. Fire flow can be supplied to the system from the Hudson Street water tower via gravity flow, from PS2 via the existing fire pump, and directly from Well 1C. Because of this, various fire flow scenarios were assessed in the model with differing fire flow supply configurations. Available fire flow for both existing and future development scenarios was assessed with fire flow supply from PS2 only, as well as with fire flow supplied from both the water tower and PS2 simultaneously.

3.1.1 System Flow Testing

In order to more accurately represent the existing system in the hydraulic model, four (4) hydrant flow tests were completed at various locations within the existing water network, which were used to calibrate the hydraulic model. Hydrant flow test locations were selected to obtain data for different areas throughout the existing system, and can be seen in Figure 3-2.

To represent the losses in the system observed during the hydrant flow tests, the Hazen Williams roughness coefficient for the existing pipes was altered. The four hydrant flow tests cover the following areas: the Canso-MacElmon-Lysander business park, the Lancaster Crescent business park, the Ventura-Vickers-Tracker business park, and the Dakota Road-Plains Road area. Pipes were grouped together when modifying roughness coefficients based on general areas and proximity to test locations. The Hazen Williams roughness coefficients used ranged between 90-120 for existing pipes. The results of the hydrant flow testing are enclosed in Appendix A.

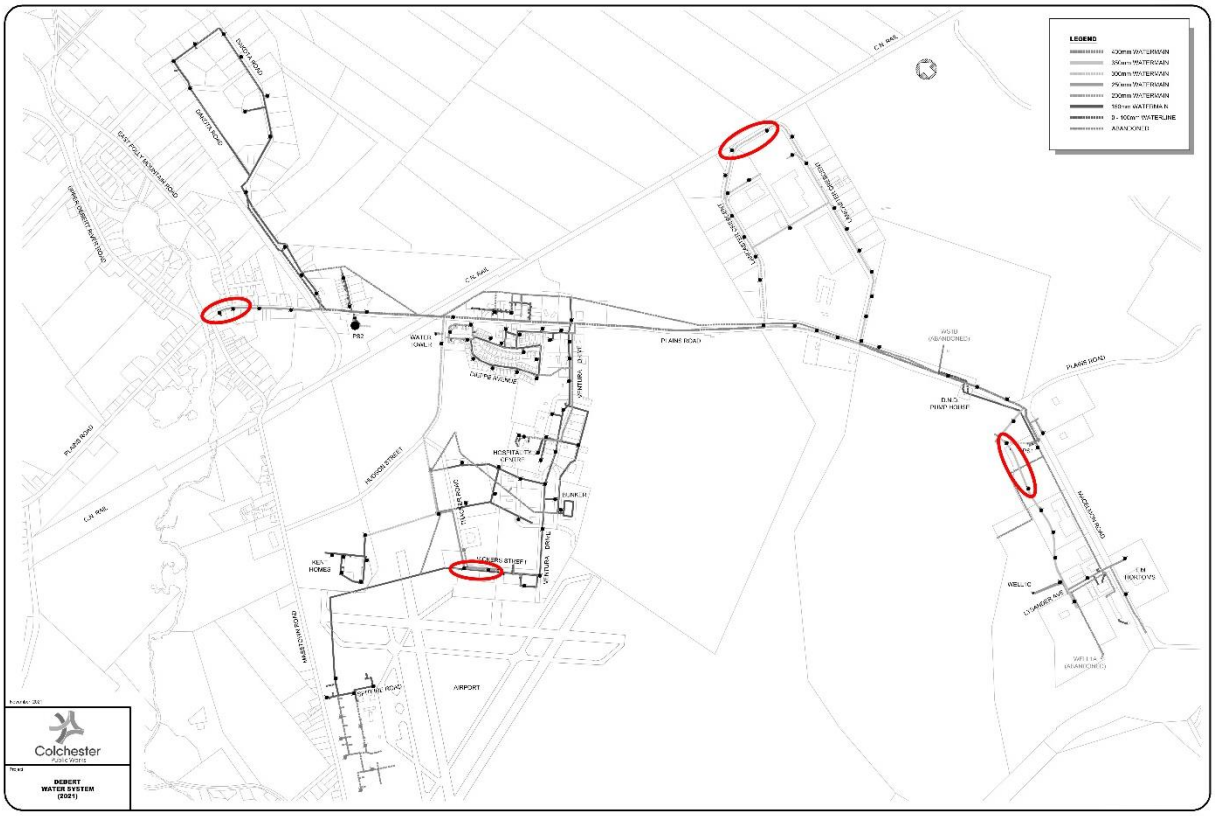


Figure 3-2: Hydrant Flow Test Locations (Shown on Debert Water Map, Courtesy of https://www.colchester.ca/water)

The static pressures measured at each test site range from approximately 50-74 psi, however static pressures outside of this range are expected at low and high points in the system.

3.2 Domestic Analysis

3.2.1 Existing Conditions

To model existing water usage, domestic demands for existing development were incorporated into the model using meter read data provided by the Municipality. Meter readings provide valuable insight into actual water usage for customers in Debert, as opposed to relying on theoretical demand estimates. Meter data also provides the opportunity to identify "unaccounted-for" water that has been supplied to the system but not reflected in customer billings. Water not accounted for in customer billings is typically lost due to leaks in the system or unmetered municipal water use.

The Municipality provided quarterly water usage information for each customer that is serviced by the Debert water system between 2021 to 2023, which also included a summary of usage totals over the period of a year. The usage totals for April 2022-March 2023 (inclusive) were compiled and analyzed to determine an average daily demand for each data point. Usage totals over this period were used to represent current metered customer usage for the Debert water system, as it is the most recent year period of meter data provided by the Municipality. Each data point corresponded to an account number and civic address, and when cross-referenced with GIS data provided by the Municipality, a northing and easting coordinate was associated with

each customer. Assigning coordinates to each customer allowed the demands to be automatically spatially located in the hydraulic model.

The Municipality also provided additional data to be used in the analyses, including the following:

- Flow data for the period of one calendar year (2022) for Well 1C, Well 2A, and the Hudson Street water tower.
- Estimated yearly water losses from bleeders at three (3) locations.

To determine a representative volume of unaccounted-for water usage, total water usage within the system based on flowmeter data was compared to the sum of customer billing data and estimated water loss at the bleeders. A sample period of January 2022-June 2022 (inclusive) was selected as a sample set for the calculations to maintain consistency with the Debert water billing schedule. Meter data provided by the Municipality indicated that meters were read on March 30th and June 28th in 2022, establishing billed usage for the periods of January 5-March 30, 2022 and March 31-June 28, 2022, respectively. Though meter data was also provided for the period of June to September in 2022, the calculations were limited to the previously described billing periods due to incomplete flowmeter data for the period of September to December 2022. Flowmeter data during the sample period was analyzed and results showed a total water usage volume of 75,830 cubic meters over approximately 167 days (time steps with missing data have been excluded). For the same time period, a total billed volume of 24,474 cubic meters and an estimated water loss via bleeders of 7,870 cubic meters was calculated, meaning 43,486 cubic meters (approximately 57%) of water have not been accounted for during this sample period.

Based on the above analysis, using raw meter data in the hydraulic model would underestimate actual total usage in the system. This would impact the accuracy of the hydraulic model and model results, as lower demands mean less flow through pipes and result in lower pipe velocities and higher HGLs (and subsequently system pressures) due to decreased losses in the system. To more accurately reflect actual water usage in the hydraulic model, the average daily demands based on meter data were scaled up by a factor of 2.46. This assumes an even distribution of unaccounted-for water. The estimated water losses via bleeders were also incorporated into the hydraulic model at the three (3) locations specified by the Municipality.

Upon additional review of the flowmeter data, a period of January 2022-August 2022 (inclusive) was selected as a sample set to determine actual peaking factors that represent maximum daily demand and peak hourly demand exhibited by the system during this period. This time period differs than that used when calculating unaccounted-for water as this analysis is not limited to water billing periods. Because of this, an additional two months was added to the time period to include as much data as possible while still excluding incomplete data during the period of September to December of 2022. Flow data from January 2022-August 2022 (inclusive) was analyzed to determine a total volume used during this period, and subsequently the maximum daily demand and peak hourly demand exhibited by this system. Time steps with missing data were excluded from the analysis, which resulted in a total of 233 days of valid data. Representative peaking factors of 1.47 for maximum daily demand and 3.06 for peak hourly demand were determined and applied to the scaled meter data to represent the different demand scenarios.

The compiled data was then inserted into the hydraulic model as customer meter nodes. Base demands for customer meter nodes were then allocated to the nearest junctions within the water model, with minor corrections being made where necessary with consideration to lateral connection locations.

3.2.2 Future Development

Domestic demands for the future residential development on Hudson Street were estimated as per the ACWWA Water Supply Guidelines (2022). The design population used in this analysis was determined based on the typical industry standard of 3.35 people per low-density residential unit and 2.25 people per high-density residential unit. The design population used for the proposed housing development on Hudson Street is 1,364, based on the provided concept plan prepared by Fathom Studio (January 2022) and assumes a total of 160 low-density residential units (i.e. single unit and two-family dwellings, and townhomes) and 368 high-density residential units (i.e. multi-unit buildings). Peaking factors for residential domestic demands were determined as per the Water Supply Guidelines, based on an approximate total serviced population between 1,001 and 2,000 (including the existing serviced population, estimated at 594 people). Resulting peaking factors were 2.50 and 3.75 for maximum daily demands and peak hourly demands, respectively. Junction locations and pipe information are included in the schematics in Appendix B.

The other proposed future development in Debert is the Mi'kmawey Debert Cultural Center (MDCC), where services have been proposed to connect to the existing infrastructure on Lancaster Crescent. Crandall Engineering (now Englobe) completed a Design Narrative for the development in January 2021, which included preliminary domestic water requirements. The average daily flow (factored based on an assumed ten hours of operation) provided in the Design Narrative was used as the basis for the theoretical demand estimates used in this analysis and hydraulic model.

The ACWWA Water Supply Guidelines state that the provided peaking factors that are based on total serviced population are acceptable to use for both residential and commercial uses. However, the Halifax Water Design Specifications (2023) identify peaking factors specifically for commercial development. As it is understood that commercial developments such as the MDCC are expected to exhibit different water usage patterns compared to residential developments, peaking factors for commercial land use as per Halifax Water (2023) were used in the analysis. Peaking factors of 1.1 and 1.2 for maximum daily demand and peak hourly demand, respectively, were applied to the theoretical average daily demand for the future development.

Domestic demands for existing development were added to each junction based on meter data, while anticipated future demands were added as per the demands listed in Table 3-1 below:

Table 3-1: Water Requirements for Future Developments

Criteria	Average Daily Demand	Maximum Daily Demand	Peak Hourly Demand
Low-Density Residential (Single Family)	1,172.5 L/unit/day (310 USG/unit/day)	2,931.3 L/unit/day (775 USG/unit/day)	4,396.8 L/unit/day (1162 USG/unit/day)
High-Density Residential (Multi-Unit)	787.5 L/unit/day (208 USG/unit/day)	1,968.8 L/unit/day (520 USG/unit/day)	2,954.4 L/unit/day (781 USG/unit/day)
Debert Mi'kmawey Museum	76.6 m^3/day (20,236 USG/day)	84.2 m^3/day (22,243 USG/day)	91.2 m^3/day (24,096 USG/day)
Totals[1]	554.0 m^3/day (146,351 USG/day)	1,277.7 m^3/day (337,612 USG/day)	1,881.6 m^3/day (49,7067 USG/day)
[1]Theoretical water demand totals for the Hudson Street residential development are based on 160 low-density and 368 high-density residential units, as per Fathom Studio concept plan (January 2022).			

As per the ACWWA Water Supply Guidelines, normal operating pressures should range between 50 to 70 psi (350 to 480 kPa). Pressures ranging between 40 to 100 psi (275 to 700 kPa) are less favorable but are still considered acceptable.

3.3 Fire Flow Analysis

A scenario was run in the model which applied a fire flow demand to each junction during the maximum daily demand scenario. The fire flow analysis applies the fire demand to a junction, checks all the remaining junctions for available pressure, and then repeats this process for each junction until all the junctions have been analyzed.

As per the ACWWA Water Supply Guidelines (2022), fire flow requirements are to be determined in accordance with the latest edition of Water Supply for Public Fire Protection from the Fire Underwriters Survey (FUS) and the latest edition of M31 Distribution System Requirements for Fire Protection from the American Water Works Association (AWWA). To determine fire flow requirements for existing development with a reasonable level of accuracy in accordance with the FUS, aspects of the detailed building design including the type of building construction and total floor area are required. When analyzing an existing system, generic building conditions are assumed which result in conservative fire flow values.

The Halifax Water Design Specifications (2023) provide another reference for fire flow requirements. Halifax Water estimated fire flow requirements are compiled from the Insurance Advisory Organization (now FUS) and are based on land use, ranging from 3,300 L/min (870 USGPM) for single and two-family residences to 13,620 L/min (3,600 USGPM) for multi-unit, industrial, commercial, and institutional (MICI) uses. The published values by Halifax Water are generic values which allow the FUS requirements to be broadly applied based on conservative assumptions. Assessing the existing system based on these values is an effective manner of applying FUS requirements.

For this analysis, minimum fire flow requirements as per the Halifax Water Design Specifications were referenced and are listed in Table 3-2. Based on the scale of the analysis and hydraulic model, it was considered

acceptable to generally prescribe fire flow requirements based on land use as per Halifax Water (2023), as it would be impractical to assess all buildings on an individual basis as per the FUS.

Table 3-2: Fire Flow Recommendations

Development Type	Fire Flow	Duration
Single/Double Family Homes	3,300 L/min (870 USGPM)	1.5 hours
Townhouse	4,542 L/min (1,200 USGPM)	1.75 hours
Multi-Unit, Industrial, Commercial, Institutional Use	13,620 L/min (3,600 USGPM)	3 hours

The ACWWA Water Supply Guidelines state that during fire flow, residual pressures at the flow point should be maintained at 22 psi (150 kPa), and system pressures should not drop below 20 psi (140 kPa). The maximum allowable pipe velocity in the system during fire flow scenarios is 3.0 m/s. These parameters were used for the purpose of this analysis.

3.4 Water Storage Analysis

A high-level assessment of existing and future storage requirements in comparison to available existing storage was completed for the Debert water system. As previously discussed, the existing water system has two water storage facilities. As per the design drawings for the Hudson Street water tower (EXP, July 2017), the tower has an approximate total storage volume of 3.48 megaliters (ML). The low-level storage reservoir PS2, which provides backup fire supply to the system, has an approximate total storage volume of 4.58 ML based on the tank dimensions provided in the Debert Water Utility Water Operations Manual (May 2022). Excluding certain circumstances (i.e. when water stored in PS2 is pumped into the distribution system), the water storage facilities have separate water supplies.

As per the ACWWA Water Supply Guidelines (2022), potable water storage requirements consist of three (3) main components: emergency storage, fire flow storage, and peak balancing storage. Fire flow storage requirements should be based on local specifications and guidelines if applicable, or the latest edition of Water Supply for Public Fire Protection from the FUS. This storage requirement assumes that the maximum supply available from the water treatment facility is equivalent to the maximum daily demand of the system. It is noted that if the water treatment facility has the capability to supply more than the maximum daily demand, fire flow storage requirements may be reduced.

Peak balancing storage is necessary to compensate for the difference between peak demands and the available supply, as water treatment facilities generally do not have the capacity to independently meet peak demands. Peak balancing storage requirements vary between water supply systems and are dependent on usage patterns and the system diurnal curve. The storage requirement for peak balancing is typically estimated at 25% of the total maximum daily demand.

Storage facilities may also have dead storage, which represents the volume below the water level where gravity flow is no longer hydraulically viable.

Water storage requirements were assessed for both existing and future development scenarios and are detailed in Section 3.5.4.

3.5 Results

3.5.1 Average Daily Demand Scenario

Existing Conditions

The model was run applying average daily demands based on existing water usage and the calculated scaling factor for unaccounted-for water as previously detailed. The results of the analysis, including the junction input data, are shown in Appendix C. Domestic pressures during the average daily demand scenario for existing development range between 26-96 psi. Figure 3-3 as follows shows color-coded results of service pressures at each junction during the average daily demand existing development scenario. See attached water schematics in Appendix B for detailed information on junction and pipe labels.

Model results show several junctions exhibiting pressures outside of the preferred range of 50-70 psi, and in some cases below the typical minimum allowable pressure of 40 psi. The resulting junction pressures cover a wide range, which is to be expected as the system consists of a single pressure zone. Due to the varying topography, junctions in low-lying areas experience high pressures and junctions at high elevations experience low pressures. To maintain adequate static pressures, one pressure zone will typically only cover an elevation range of approximately 30 m (the equivalent of 42 psi). The lands serviced by the Debert water system have a maximum elevation difference of 50 m (16 m to 66 m), which helps explain the low pressure issues.

Domestic pressures are low in the Dakota Road area, ranging from 26-47 psi where existing grade elevations range from

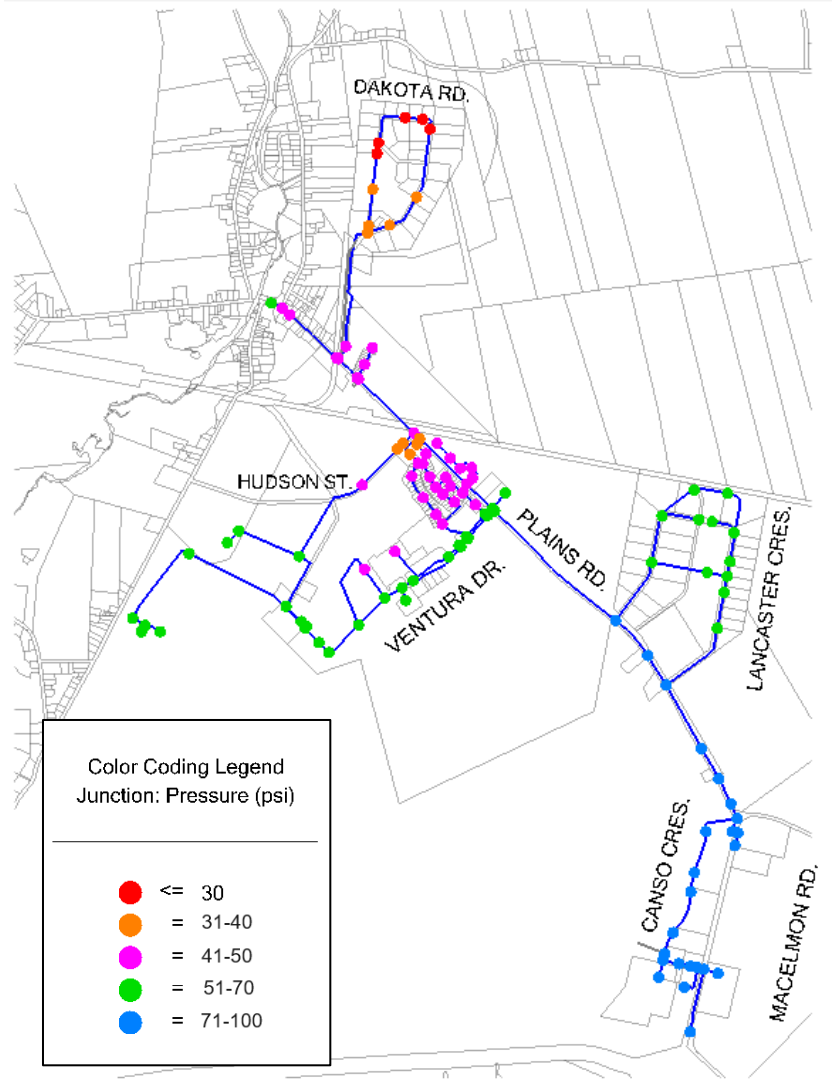


Figure 3-3: Service Pressure Results, Average Daily Demand

approximately 51-66 m. Service pressures are just below the acceptable threshold on Hudson Street adjacent the water tower, where the lowest service pressures are approximately 39 psi at an existing grade elevation of approximately 57 m. Conversely, pressures are higher and range from 72-96 psi in the MacElmon-Canso-Lysander area at the southern extents of the network, where existing grade elevations range from approximately 16-33 m.

Apart from Dakota Road and Hudson Street adjacent the water tower, model results show service pressures for all other areas in Debert during average daily demands to be within the acceptable range of 40-100 psi (ACWWA Water Supply Guidelines, 2022).

Future Development

For this scenario, the model was run applying average daily demands based on existing water usage and including estimated average daily demands for future developments. The results of the analysis, including the junction input data, are shown in Appendix C. Domestic pressures during the average daily demand scenario for existing and future development range between 26-96 psi.

The steady-state hydraulic analysis results show future development having a negligible impact on pressures in the distribution system during average daily demands. For this reason, Figure 3-3 can also be referenced for color-coded results of service pressures at each junction during the average daily demand future development scenario. As the hydraulic model was run as a steady-state analysis only, fluctuations in water levels in the tower and the subsequent operation of Well 1C did not factor into this analysis.

During average daily demands, domestic pressures on Hudson Street adjacent the proposed residential development and on Lancaster Crescent at the proposed MDCC service connection point are approximately 46 psi and 68 psi, respectively. It is reiterated that these results are based on an assumed water level of 84.0 m in the Hudson Street water tower.

3.5.2 Peak Hourly Demand Scenario

Existing Conditions

The model was run applying peak hourly demands based on existing water usage and calculated scaling and peaking factors as previously detailed. The results of the analysis, including the junction input data, are shown in Appendix C. Domestic pressures during the peak hourly demand scenario for existing development range between 22-92 psi. Figure 3-4 as follows shows color-coded results of service pressures at each junction during the peak hourly demand existing development scenario. See attached water schematics in Appendix B for detailed information on junction and pipe labels.

Similar to the average daily demand scenario, model results show several junctions exhibiting pressures outside of the preferred range of 50-70 psi, and in some cases below the typical minimum allowable pressure of 40 psi. The resulting junction pressures cover a wide range due to the varying topography in the serviced area. Domestic pressures are low in the Dakota Road area, ranging from 22-43 psi where existing grade elevations range from approximately 51-66 m. Due to increased flows and an assumed low water level in the water tower, the northwest area of the Dieppe-Passchendaele subdivision, Vimy Road, and the area adjacent the Hudson Street-Plains Road intersection experience service pressures below the acceptable minimum. Pressures in these areas range from 35-39 psi, where existing grade elevations range from approximately 53-57 m.

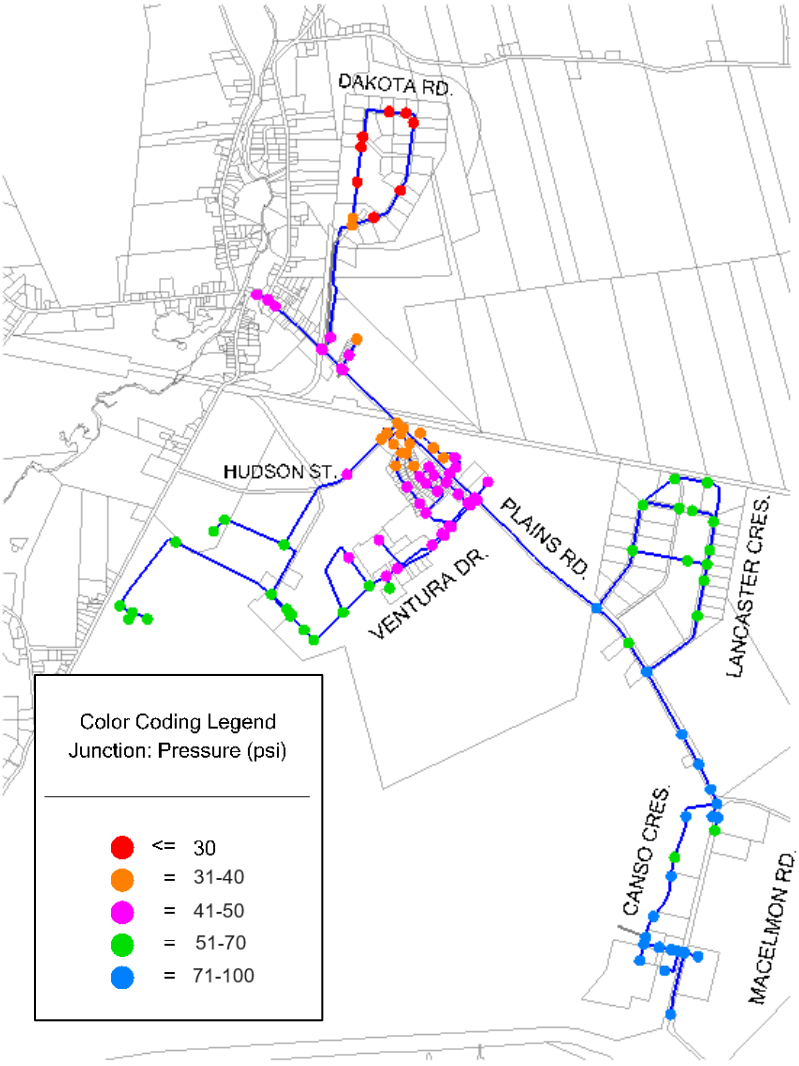


Figure 3-4: Service Pressure Results, Peak Hourly Demand – Existing Development

Pressures are higher and range from 68-92 psi in the MacElmon-Canso-Lysander area at the southern extents of the network, where existing grade elevations range from approximately 16-33 m.

Future Development

For this scenario, the model was run applying peak hourly demands based on existing water usage and including estimated peak hourly demand for future developments. The results of the analysis, including the junction input data, are shown in Appendix C. Domestic pressures during the peak hourly demand scenario for existing and future development range between 21-91 psi. Figure 3-5 as follows shows color-coded results of service pressures at each junction during the peak hourly demand future development scenario. Additionally, junctions in the water schematic C-WA01 in Appendix B have been color-coded based on the resulting pressures for this scenario.

The steady-state hydraulic analysis results show future development having only a minimal impact on pressures in the distribution system during peak hourly demands, with junction pressures approximately 1 psi lower than the results for existing demands only. This minor reduction in pressures is to be expected for a gravity-fed distribution system such as this. As the hydraulic model was run as a steady-state analysis only, fluctuations in water levels in the tower and the subsequent operation of Well 1C did not factor into this analysis.

Although the areas of existing development with lower than acceptable pressures are expected to experience a reduction in pressure as a result of the proposed development, this is primarily due to these areas being too high in elevation relative to the hydraulic grade line of the system. The proposed developments do not have a major impact on these areas.

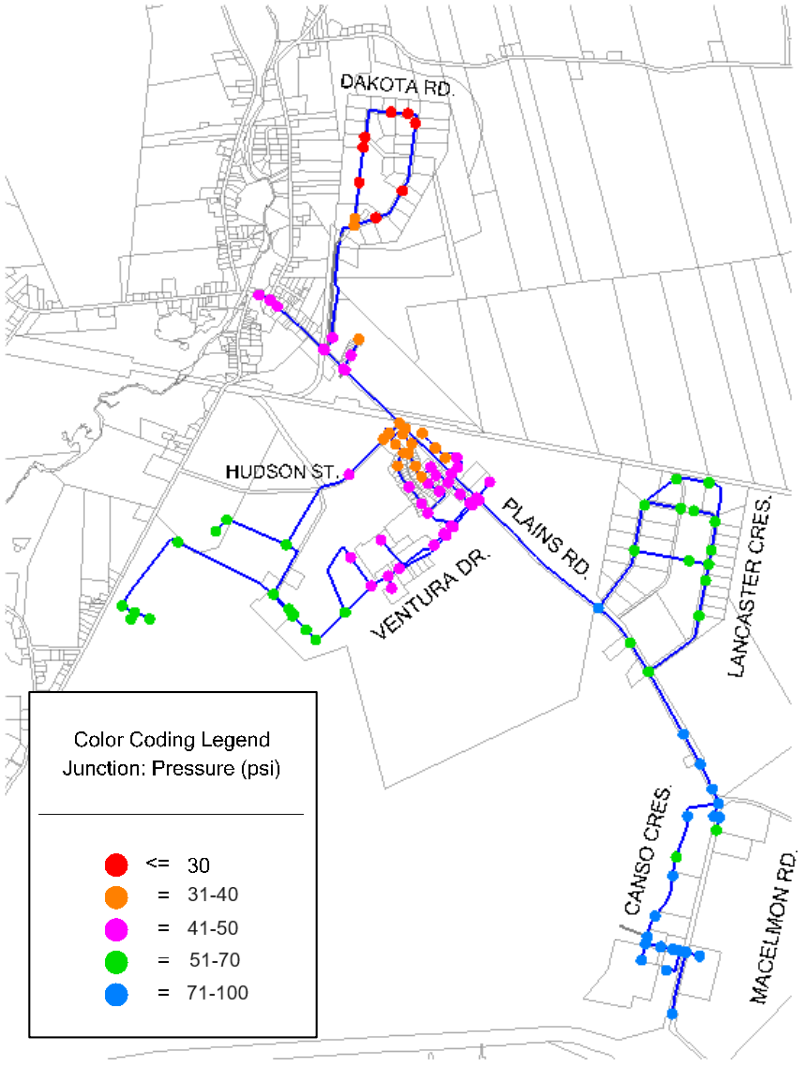


Figure 3-5: Service Pressure Results, Peak Hourly Demand – Future Development

Based on the steady-state analysis of the existing system, the existing linear system can adequately supply domestic demands for the future developments indicated in the RFP (Hudson Street residential development, MDCC serviced from Lancaster Crescent). During peak hourly demands, domestic pressures on Hudson Street adjacent the proposed residential development and on Lancaster Crescent at the proposed MDCC service connection point are approximately 42 psi and 64 psi, respectively. It is reiterated that these results are based on an assumed water level of 81.3 m in the Hudson Street water tower.

Based on a HGL of 81.3 m in the existing system, future development should be limited to finished grade elevations less than or equal to 52 m to keep service pressures above the acceptable minimum of 40 psi. This maximum finished grade elevation will need to be reviewed on a case-by-case basis, as the additional friction losses resulting from the increased water demands would lower the HGL of the system as well. Figure 3-6 as follows shows the areas in Debert with ground elevations of approximately 52 m or higher (based on vertical datum CGVD2013).

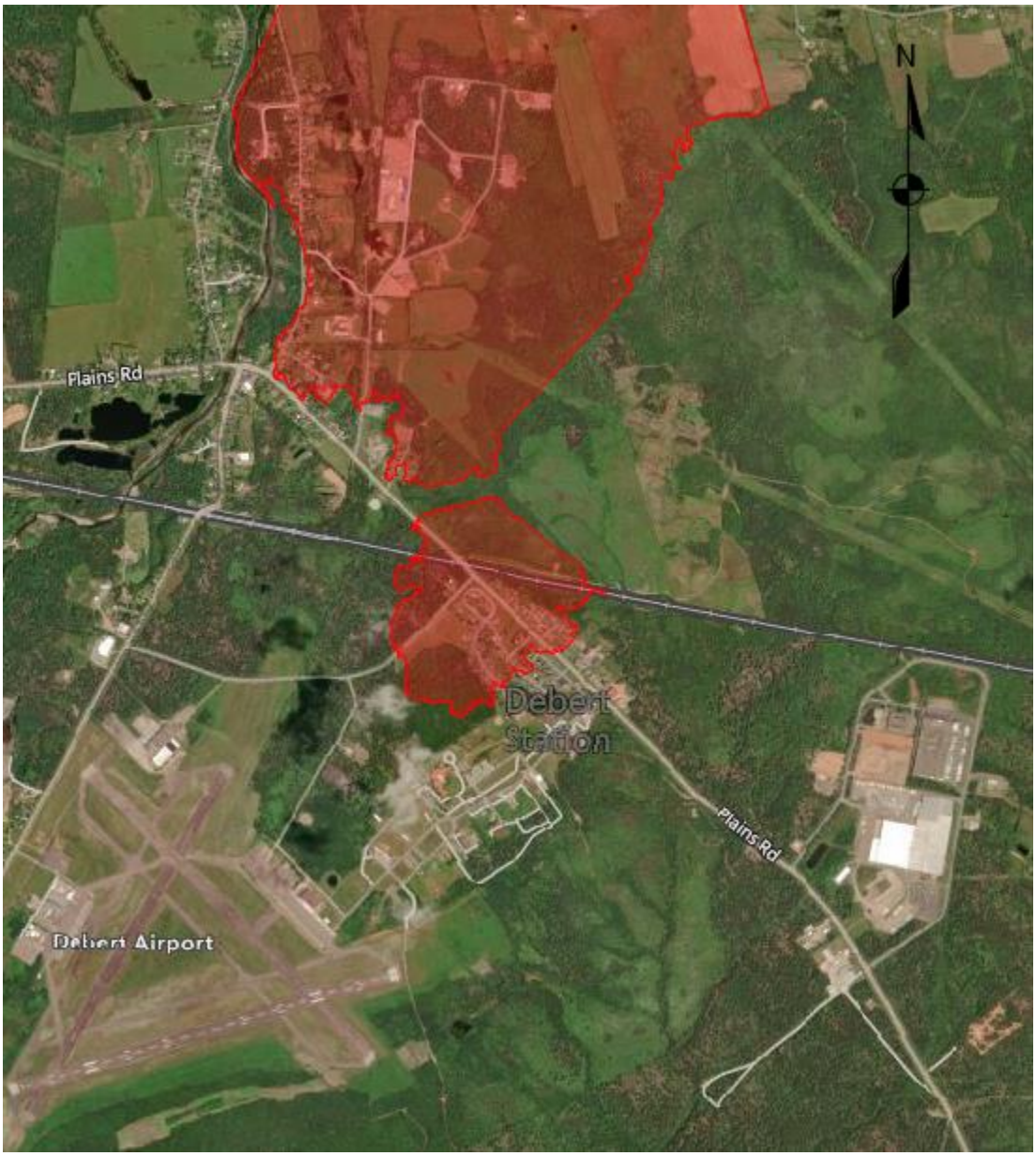


Figure 3-6: Approximate Extents of Areas with Ground Elevations Greater than or Equal to 52 m (Vertical Reference CGVD2013)

3.5.3 Maximum Daily Demand + Fire Flow Scenario

Existing Conditions

As shown in the results in Appendix D, available fire flow was calculated for each junction during existing maximum daily demand conditions while maintaining minimum 22 psi residual pressure, minimum 20 psi system pressures, as well as a maximum velocity of 3.0 m/s in the pipes. The results were then compared to the typical fire flow recommendations listed in Table 3-2 based on the existing development types adjacent to each junction. Different scenarios for fire flow supply were assessed as part of this analysis.

With PS2 alone providing fire flow to the system, model results show available fire flows for existing development ranging from 1,976 to 12,289 L/min. Model results show no junctions meeting or exceeding the 13,620 L/min typical recommendation for MICI land uses in this scenario. Ranges of available fire flow for certain areas have been listed under "Scenario A" in Table 3-3 as follows, which reflects available fire flows with no pipe velocities exceeding 3.0 m/s and fire flow supply from PS2 only.

Model results show that in some instances, the limiting factor for available fire flow is the maximum velocity in the 300 mm water main connecting PS2 to the distribution system. As this is a short length of pipe (approximately 30 m from the control building to the existing 400 mm water main) that is isolated and typically only active during fire flow conditions, it may be acceptable to the Municipality for this water line to exhibit velocities higher than 3.0 m/s. To assess this scenario, available fire flows were also checked with no velocity constraints on the 300 mm pipe between PS2 and the distribution system. This is referred to as "Scenario B", and the ranges of available fire flow for certain areas under this scenario are summarized in Table 3-3 as follows. This scenario showed available fire flows for existing development ranging from 1,976 to 13,510 L/min.

The model was also assessed with both PS2 and the Hudson Street water tower providing fire flow to the system, as including an additional source of fire flow supply compared to Scenarios A and B is expected to impact the results. For this scenario, it was assumed that the water level in the tower corresponded to the high fire level (HFL) of 81.3 m as per the design drawings for the tower (EXP, July 2017). This is referred to as "Scenario C", and a summary of available fire flows throughout the system are summarized in Table 3-3 as follows. Figure 3-7 shows color-coded results of available fire flow at each junction for Scenario C. Additionally, the results shown in Appendix D for the existing development scenario are based on results for Scenario C in the hydraulic model. It is noted that this scenario includes no velocity constraints on the 300 mm pipe leaving PS2, or the 200 mm pipe that fills and drains the water

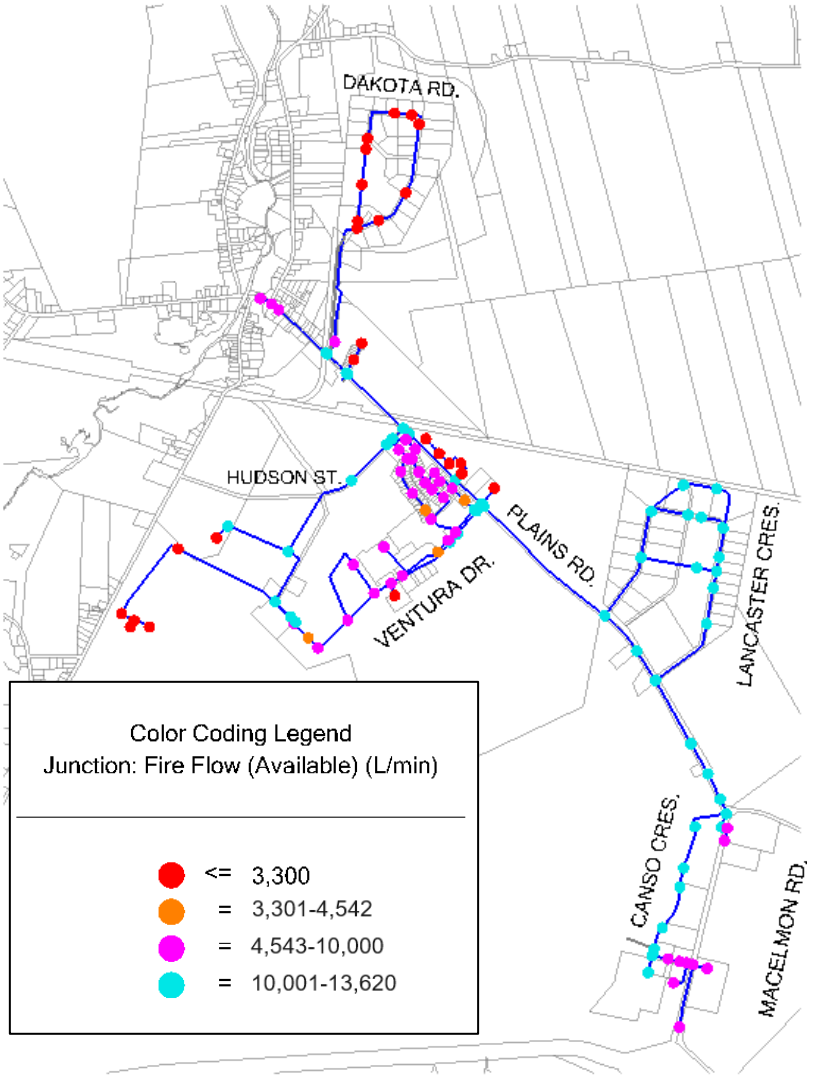


Figure 3-7: Available Fire Flow Results – Scenario C, Existing Development

tower. This scenario showed available fire flows for existing development ranging from 1,976 to 13,620 L/min. See attached water schematics in Appendix B for detailed information on junction and pipe labels.

Based on discussions with the Municipality, it is understood that the Hudson Street water tower is outfitted with an altitude valve which would prevent filling of the water tower when the system HGL is high, which may occur as a result of activation of the fire pump at PS2. In modelling Scenario C, a check valve was included on the pipe leaving the Hudson Street water tower to prevent filling of the tower when PS2 and the fire pump is active.

A summary of fire flow results for existing development conditions can be seen in Table 3-3 as follows, along with the recommended fire flow for the area based on existing and projected land use.

Table 3-3: Existing Development Fire Flow Results

Existing Development	Available Fire Flow (Scenario A[1]) (L/min)	Available Fire Flow (Scenario B[2]) (L/min)	Available Fire Flow (Scenario C[3]) (L/min)	Recommended Fire Flow
Dresden-Vimy Area	3,150	3,150	3,150	3,300 L/min
Paleo Subdivision	3,174	3,174	3,174	3,300 L/min
Dieppe-Passchendaele Subdivision	4,161 – 7,769	4,161 – 7,769	4,161 – 7,769	4,542 L/min
Dakota Road	2,180 – 5,590	2,180 – 5,590	2,180 – 5,590	13,620 L/min
Plains Road	5,572 – 12,289	5,572 – 13,510	5,572 – 13,620	13,620 L/min
Hudson Street (incl. 70 Hudson)	3,180 – 12,289	3,180 – 13,195	3,180 – 13,620	13,620 L/min
Ventura-Tracker-Vickers Area	3,181 – 12,102	3,181 – 12,102	3,181 – 12,102	13,620 L/min
Debert Airport	1,976 – 2,664	1,976 – 2,664	1,976 – 2,664	13,620 L/min
Lancaster Crescent	11,578 – 12,289	11,578 – 12,820	11,580 – 12,820	13,620 L/min
MacElmon-Canso-Lysander Area	5,654 – 12,289	5,654 – 12,480	5,654 – 12,480	13,620 L/min

[1]Scenario A: Supply from PS2, no velocity exceedances ($V_{max} \leq 3.0$ m/s leaving PS2)
[2]Scenario B: Supply from PS2, pipe leaving PS2 excluded from velocity limits ($V_{max} > 3.0$ m/s leaving PS2)
[3]Scenario C: Supply from PS2 and water tower, pipes leaving PS2 and water tower excluded from velocity limits ($V_{max} > 3.0$ m/s leaving PS2 and water tower)

In general, the main distribution factors that are limiting available fire flow include low system pressures, high velocities in small-diameter water mains, and friction losses during high flows. Small-diameter water mains can convey less flow before reaching the maximum pipe velocity of 3.0 m/s in comparison to larger-diameter water mains. Model results show that available fire flow for areas serviced via 250 mm and smaller water mains are likely limited by maximum pipe velocities. Areas where available fire flow is generally limited by velocity constraints include Plains Road from Dakota Road to E Folly Mountain Road, the Ventura-Tracker-Vickers area, the Dieppe-Passchendaele subdivision, the Vimy-Dresden area, the Paleo subdivision, and the area of MacElmon-Canso-Lysander that is serviced via small-diameter water main.

Available fire flow is also limited in select areas due to low residual pressures. Residual pressures decrease during fire flows due to high flow conditions, as the system HGL decreases due to increased friction losses in the pipes. This issue is exacerbated in areas with small-diameter pipes that have low C values, as per the model calibration based on hydrant flow test results. Generally, this issue is observed during fire flow conditions in the Dakota Road area, in the area of the Debert Airport and 70 Hudson Street, and at the high-elevation locations on Lancaster Crescent.

It was observed that for most areas serviced via large-diameter pipe, specifically 300 mm and larger, available fire flows are limited by low system pressures. This is expected, as larger-diameter water mains are generally not as susceptible as small-diameter water mains to high velocities and significant friction losses during fire flow. It was observed that these areas were limited by low system pressures at localized high points in the system, specifically in the Dakota Road area, the Dieppe-Passchendaele subdivision, the Lancaster Crescent area, and the MacElmon-Canso-Lysander area.

It is also noted that the variance in pump head supplied from the fire pump over the wide range of available fire flows can create issues in the system. Review of the existing fire pump curve and model results indicate that when demands exceed approximately 12,000 L/min, the fire pump does not provide sufficient head to achieve a HGL consistent with the typical HGL of the system during domestic demands. This is notable, as there are junctions in the Dakota Road area that experience service pressures at or close to 22 psi, which is also the minimum allowable residual pressure during fire flow. These junctions are very sensitive to changes in the system HGL, as a minor drop in the HGL resulting from low pump head will result in lower system pressures in these areas and subsequently lower available fire flows in these areas.

Conversely, low flows through the existing fire pump deliver high pump head to the system, which may increase the system HGL to unacceptable levels. It was previously noted that the typical HGL of the system is 84 m, which resulted in maximum service pressures of approximately 96 psi in the MacElmon-Canso-Lysander area. This maximum pressure is just under the acceptable maximum of 100 psi, therefore a minor increase in the HGL due to low flows through the fire pump would increase pressures to unacceptable levels in low-elevation areas. It was noted that at lower flows through the fire pump, for example 3,300 L/min, the fire pump output approximately 77 m of pump head. This resulted in a system HGL of approximately 126 m and pressures of approximately 156 psi at the low point in the system on MacElmon Road. This pressure exceeds the acceptable maximum as per the Water Supply Guidelines by 56 psi.

Model results for Scenario C showed that with the addition of the water tower as a fire flow supply, available fire flow either remains the same or increases, and in some instances the maximum fire flow recommendation of 13,620 L/min for MICI sites is achieved.

Future Development

The same calculations were also completed for fire flow scenarios A, B, and C, with the inclusion of the estimated maximum daily demand for the proposed developments. The results were then compared to the typical fire flow recommendations listed in Table 3-2, and detailed results including junction inputs can be seen in Appendix D. Junctions in the water schematic C-WA02 in Appendix B have been color-coded based on the resulting available fire flow for this scenario. It is noted that the results in Appendix D and water schematic results in Appendix B are based on fire flow Scenario C, which assumes that both the Hudson Street water tower and PS2 are supplying fire flow to the system, and the outlet pipes for the storage structures are excluded from the maximum velocity limitation of 3.0 m/s.

With PS2 alone providing fire flow to the system, model results show available fire flows for existing and future development ranging from 1,961 to 11,402 L/min. These available fire flows do not meet or exceed the 13,620 L/min typical recommendation for MICI land uses. Factors limiting available fire flow are the same as described for the existing development only scenario. Similarly to the existing development scenario, model results were assessed both with and without a maximum velocity constraint on the existing 300 mm pipe connecting PS2 to the distribution system (Scenario B). Model results were also assessed with both PS2 and the Hudson Street water tower providing fire flow to the system (Scenario C). With both PS2 and the Hudson Street water tower providing fire flow to the system, model results show available fire flows for existing and future development ranging from 1,961 to 13,620 L/min. Figure 3-8 shows color-coded results of available fire flow at each junction for Scenario C.

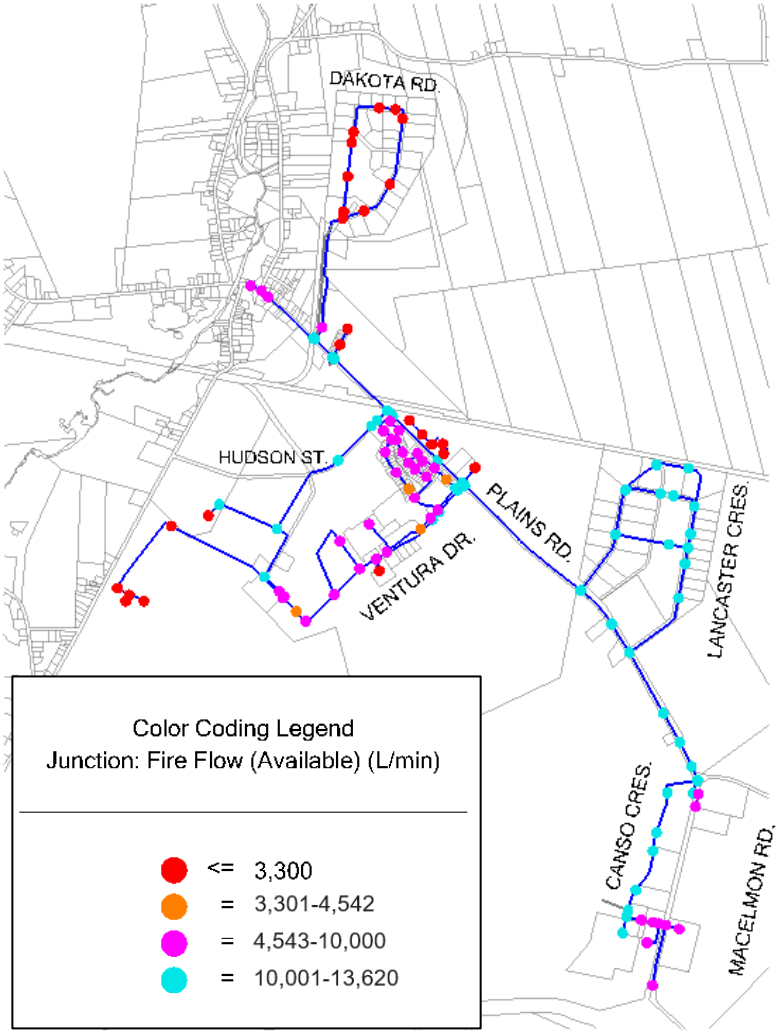


Figure 3-8: Available Fire Flow Results – Scenario C, Future Development

A summary of fire flow results for future development conditions can be seen in Table 3-4 as follows, along with the recommended fire flow for the area based on existing and projected land use:

Table 3-4: Future Development Fire Flow Results

Existing Development	Available Fire Flow (Scenario A[1]) (L/min)	Available Fire Flow (Scenario B[2]) (L/min)	Available Fire Flow (Scenario C[3]) (L/min)	Recommended Fire Flow
Dresden-Vimy Area	3,150	3,150	3,150	3,300 L/min
Paleo Subdivision	3,174	3,174	3,174	3,300 L/min
Dieppe-Passchendaele Subdivision	4,161 – 7,768	4,161 – 7,768	4,161 – 7,768	4,542 L/min
Dakota Road	2,169 – 5,590	2,169 – 5,590	2,169 – 5,590	13,620 L/min
Plains Road	5,572 – 11,402	5,572 – 12,622	5,572 – 13,620	13,620 L/min
Hudson Street (incl. 70 Hudson)	3,180 – 11,402	3,180 – 12,333	3,180 – 13,620	13,620 L/min
Proposed Hudson Street Development	11,402	11,887	12,932	13,620 L/min
Ventura-Tracker-Vickers Area	3,181 – 11,402	3,181 – 12,092	3,181 – 12,093	13,620 L/min
Debert Airport	1,961 – 2,639	1,961 – 2,639	1,961 – 2,639	13,620 L/min
Lancaster Crescent	11,023 – 11,402	11,023 – 12,133	11,023 – 12,614	13,620 L/min
MacElmon-Canso-Lysander Area	5,654 – 11,402	5,654 – 11,897	5,654 – 11,897	13,620 L/min
Debert Mi'kmawey Museum	11,402	12,133	12,614	13,620 L/min

[1]Scenario A: No Velocity Exceedances ($V_{max} \leq 3.0$ m/s leaving PS2)
[2]Scenario B: Pipe leaving PS2 excluded from velocity limits ($V_{max} > 3.0$ m/s leaving PS2)
[3]Scenario C: Pipes leaving PS2 and water tower excluded from velocity limits ($V_{max} > 3.0$ m/s leaving PS2 and water tower)

If velocity exceedances in the existing 300 mm pipe from PS2 are not acceptable, model results show that the available fire flow at the future Hudson Street residential junction (J-12) and the future MDCC site (J-102) is 11,402 L/min, with no system upgrades. If velocities in this pipe that exceed 3.0 m/s are acceptable, the available fire flows at J-12 and J-102 are increased to 11,887 L/min and 12,133 L/min respectively, with a maximum velocity of 3.2 m/s in the 300 mm line from PS2. Lastly, if fire flow is supplied from both PS2 and the water tower, the available fire flows at J-12 and J-102 are increased to 12,932 L/min and 12,614 L/min respectively, with a maximum velocity of 3.1 m/s in the 300 mm line from PS2. For junctions with an available fire flow of 13,620 L/min, a maximum velocity of 3.3 m/s was observed in the in the 300 mm line from PS2.

When comparing available fire flows in the future development scenario to the existing development scenario, significant impacts to available fire flow were observed almost exclusively at junctions serviced from existing 300 mm, 350 mm, and 400 mm water mains. As previously discussed, model results show that available fire flow for areas serviced by water mains that are 300 mm and larger is mostly limited by system and/or residual pressures in areas where service pressures are already low due to higher ground elevations. Additional flow from the future developments decreases the system HGL due to exacerbated pump performance issues (higher total flows are required, which results in less pump head) and increased friction losses in the system. For this

reason, areas serviced via the existing 300 mm, 350 mm, and 400 mm water mains see a more significant reduction in available fire flow in the future development scenario.

In areas serviced by pipes with diameters 250 mm and smaller (Dakota Road, Ventura-Tracker-Vickers area, part of the MacElmon-Canso-Lysander area), the main fire flow limitation observed is maximum pipe velocities in the smaller-diameter pipes. This factor is limiting available fire flow in both existing and future development scenarios, with the additional demand from the proposed developments having only a small impact on the results due to increased friction losses and a lowered HGL.

For all scenarios included in this analysis, available fire flows for the proposed developments do not meet the typical fire flow recommendation of 13,620 L/min for these types of developments. However from experience, these fire flows are typically sufficient for these types of developments. As such, the existing system is likely capable of providing adequate fire protection to the proposed developments. This must be confirmed during detailed design of the buildings within the developments. It is noted that certain design and construction strategies can reduce fire flow requirements, such as chosen building materials and incorporation of an automatic sprinkler system.

3.5.4 Water Storage

Peak Balancing Storage

The ACWWA Water Supply Guidelines (2022) state that peak balancing requirements for a water supply system can be estimated as 25% of the maximum daily demand of the system. Based on this requirement, peak balancing storage requirements were calculated for both the existing and future development scenarios.

In calculating peak balancing storage requirements, consideration was also given to the constant water loss through bleeders at three (3) different locations. To provide a conservative storage requirement estimate for the future development scenario, peak balancing storage was calculated for the maximum daily demand of the system assuming that water loss through the bleeders remains consistent as future developments are built. However, it is acknowledged that as development and subsequently water demands increase, bleeders with the purpose of improving water quality may no longer be required. For this reason, peak balancing storage requirements excluding constant water loss through bleeders have also been included. Table 3-5 below summarizes peak balancing storage requirements for various scenarios.

As previously discussed, the Hudson Street water tower has a total storage volume of 3.48 ML. Based on the design drawings for the tower (EXP, July 2017), there is approximately 0.32 ML of storage volume between the low water level (LWL) and high water level (HWL).

Table 3-5: Peak Balancing Storage Requirements

	Peak Balancing Storage Requirement		Available Domestic Storage
Development Scenario	Including Water Loss from Bleeders	Not Including Water Loss from Bleeders	Hudson Street Water Tower
Existing Development Only	156.4 m^3 (0.16 ML)	145.2 m^3 (0.15 ML)	316.7 m^3 (0.32 ML)
Existing and Future Development	475.8 m^3 (0.48 ML)	464.6 m^3 (0.46 ML)	316.7 m^3 (0.32 ML)

The results of this assessment show that based on the water levels in the design drawings for the water tower (EXP, July 2017), the existing water tower meets peak balancing storage requirements as per the Water Supply Guidelines (2022) for existing development, but is inadequate for the future development scenario. Based on an available peak balancing storage volume of 0.32 ML and a storage requirement of 0.16 ML for existing water usage (including water loss from bleeders), there is approximately 0.16 ML of additional peak balancing storage volume in the water tower. This peak balancing volume corresponds with an additional 0.64 ML, or 640 m^3/day, of maximum daily demand, maintaining the assumption that peak balancing storage requirements are 25% of the maximum daily demand. Assuming the construction of the MDCC, this leaves approximately 556 m^3/day remaining capacity to be applied to the Hudson Street residential development. Based on a theoretical water demand of 875 L/cap/day (350 L/cap/day with a peaking factor of 2.5 applied for maximum daily demand as per Water Supply Guidelines), the additional maximum daily demand corresponds with an additional population of 636. In the context of the Hudson Street residential development and based on the assumption of 3.35 people per low density unit and 2.25 people per high density unit, this is equivalent to:

- 160 low-density units (out of 160 proposed) and 44 high-density units (out of 368 proposed), or
- 282 high-density units (out of 368 proposed).

As previously stated, the total design population of the Hudson Street residential development is 1,364 people.

To provide service to the future developments identified in the RFP, additional peak balancing storage is required. This assessment indicates that as per ACWWA Water Supply Guidelines, an additional 0.16 ML of peak balancing storage is required to service the future developments covered in this assessment.

Fire Flow Storage

Fire flow storage requirements were based on the fire flow recommendation of 13,620 L/min for MICI properties as per Halifax Water Design Specifications (2023). The Halifax Water specifications also state that this fire flow has a required duration of three (3) hours, which is consistent with the FUS (2020). This represents a required fire flow volume of 2.45 ML. In the hydraulic model, a maximum required fire flow of 13,620 L/min was used for both existing and future development scenarios.

Fire flow storage was assessed for both PS2 and the Hudson Street water tower. As previously stated, PS2 has an approximate total storage volume of 4.58 ML, which exceeds the required fire flow volume by 2.13 ML. The design drawings for the water tower (EXP, July 2017) identified a low fire level (LFL) of 74.0 m and a high fire level (HFL) of 81.9 m. There is approximately 1.00 ML of available fire flow storage between the LFL and HFL.

The results of this assessment show that PS2 alone has adequate storage available to meet fire flow storage requirements as per this analysis, for both existing development and future development scenarios. Results also showed that based on the water levels in the design drawings for the water tower (EXP, July 2017), existing fire flow storage in the Hudson Street water tower alone is inadequate to provide 13,620 L/min of fire flow to the system for a three-hour duration. If the Municipality intends to decommission PS2 in the future or create redundancy for water storage facilities, an additional 1.45 ML of fire flow storage will be required to provide 13,620 L/min of flow for a three-hour period.

3.6 Recommendations

3.6.1 Upgrades for Proposed Developments

The results of the high-level water storage assessment showed that as per the ACWWA Water Supply Guidelines, there is inadequate peak balancing storage available to accommodate the future developments identified in the RFP. To facilitate future growth within the community, it is recommended to add 0.19 ML of peak balancing storage to the system. Two options have been explored to meet this requirement.

There is existing available storage in PS2 that has the potential to be utilized as peak balancing storage. This option would require implementation of an on-site booster pump to supply domestic flow to the system at an appropriate HGL for the system (i.e. 84.0 m). The recommended pump configuration/ sizing and incidentals for this option are subject to detailed design, as this option is dependant on the operational preferences and long-term plans for the Municipality. To adequately assess different operational configurations and controls for this option at the preliminary design stage, it is recommended to upgrade the hydraulic model to an Extended Period Simulation (EPS).

Peak balancing storage can also be added to the system through the construction of an additional water tower at the Hudson Street water tower site. A second tower was identified as a possible option on the design drawings for the water tower (EXP, July 2017). Along with providing the minimum peak balancing storage volume, a second water tower would also add redundancy to the system. This would allow for maintenance to either tower to occur without major system impacts.

As previously noted, model results show that the existing linear system cannot adequately supply the typical fire flow recommendation of 13,620 L/min for the proposed developments indicated in the RFP. However it is reiterated that the available fire flows are likely adequate, depending on the specific needs of the developments. If the maximum required fire flow for the proposed developments is less than the available fire flow as per this assessment, the linear system can adequately service the proposed future developments identified in the RFP. If it is determined that the proposed developments require more fire flow than what the existing system can provide, upgrades to the existing system are required.

Notwithstanding maximum velocities in the pipe leaving PS2, available fire flow to the Hudson Street residential development is primarily limited by pipe velocities in the existing 300 mm water main on Hudson Street. These pipes are identified as P-2, P-25, and P-26 in the water schematics attached in Appendix B. If PS2 is the sole supply of fire flow to the system and maximum velocity constraints are not taken into consideration, a low residual pressure will also limit available fire flow. To achieve a fire flow of 13,620 L/min for the Hudson Street development, it is recommended to upsize existing pipes to increase available fire flow and alleviate velocity exceedances. To achieve an available fire flow of at least 13,620 L/min, model results show that it is also necessary to employ both PS2 and the Hudson Street water tower as fire flow supplies.

Model results show that the velocity exceedances can be alleviated by upsizing the water main from the intersection of Plains Road and Hudson Street to the connection point of the proposed development. A connection point has been assumed at junction J-12 in the model, representing an approximate length of 420 m of pipe to be upsized. As the connection point is assumed, this length is subject to change. If this pipe length is upsized to 400 mm, model results show acceptable maximum pipe velocities under 3.0 m/s during this fire

flow scenario. See Figure 3-9 as follows for a schematic representation of this pipe upsizing option, referred to as Pipe Upsizing Option A.

Redundancy in water systems through additional connections such as this can also improve system supply and reliability, including during fire flow conditions. An alternative to upsizing the length of pipe from 300 mm to 400 mm as previously stated would be to upsize the existing pipes that provide supply from the southwest direction at Tracker Road. It has been identified that the existing Ventura Drive system is inadequately sized to provide the typical MICI fire flow to this area of the Debert Business Park (Ventura-Tracker-Vickers area). It is also understood that the existing 150 mm pipes in this area could be up to 80 years old. If the 300 mm water main on Hudson Street is extended to form a loop along Tracker Road, Vickers Street, and Ventura Drive extending to the Ventura Drive-Plains Road intersection, model results show that a typical fire flow of 13,620 L/min can be supplied to the proposed Hudson Street residential development. This project would involve the replacement of approximately 2,030 m of existing water main in total, likely necessitating a longer term and multi-phase approach compared to Option A. These results are assuming that velocities in excess of 3.0 m/s out of PS2 in the 300 mm water line are acceptable. Though a larger undertaking than the previous upsizing recommendation, this project would provide an opportunity to replace the existing aging 150 mm water main, provide redundancy to the system, and increase available fire flow in this area of the Debert Business Park for both existing and future development. See Figure 3-10 below for a schematic representation of this pipe upsizing option, referred to as Pipe Upsizing Option B.

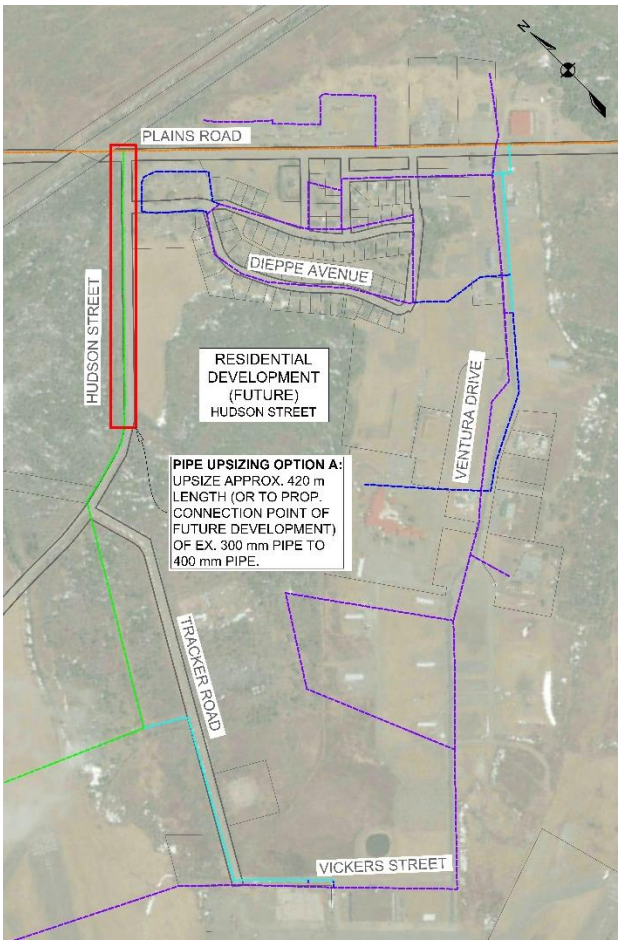


Figure 3-9: Pipe Upsizing Option A

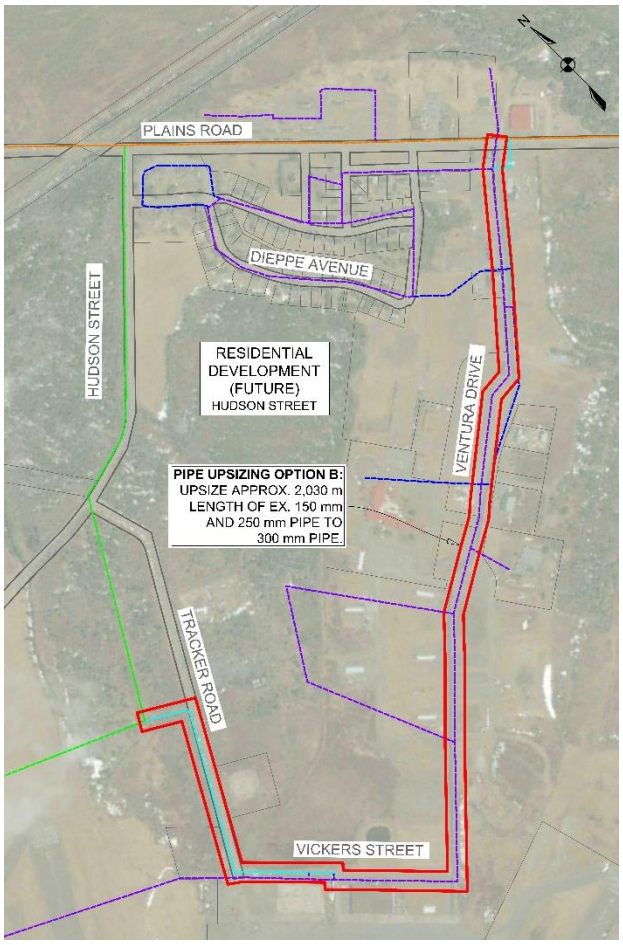


Figure 3-10: Pipe Upsizing Option B

Due to residual pressure limitations, a maximum of 12,783 L/min of fire flow can be achieved at the MDCC connection location. An increase in the HGL at the connection location is required to increase residual pressures to acceptable levels to provide 13,620 L/min of fire flow while meeting all fire flow requirements as per the ACWWA Water Supply Guidelines (2022). This proposed development is sufficiently far from both potential sources of fire flow included in the hydraulic model (PS2 and water tower) therefore it was considered not feasible to upsize pipes to increase available fire flow. Another potential solution to increase the HGL would be replacement of the existing fire pump at PS2 with a pump that can supply more head at the required flow. This is considered to be an excessive measure to marginally increase available fire flow to the proposed development. As previously discussed, experience shows that available fire flows of 12,783 L/min are usually sufficient for this type of development. If this available fire flow is not sufficient, design and construction strategies can be likely utilized to limit the required fire flow to be at or below 12,783 L/min.

It is acknowledged that the proposed development is in closer proximity to Well 1C, which is expected to provide flow from the southern direction during high flow conditions where the water tower is being drained. At the time of this report, this supply has not been included as there is insufficient information known regarding the submersible well pump. If pump information becomes available, it is recommended to update the hydraulic model to include Well 1C as a water source.

Opinions of Probable Cost

Opinions of Probable Cost (OPC) were determined for each upgrade option. In the absence of a defined scope of work or preliminary design for the upgrade options, the OPCs are presented as a Class D cost estimate with a standard contingency of 45% applied. The OPCs are in 2024 Canadian dollars and do not include taxes.

The Opinions of Probable Cost presented in Table 3-6 are presented as estimates only and are based on experience and best judgement. The OPCs are not guaranteed and may vary, as actual project costs are subject to change as a result of factors that are unknown at the time of this report, including market conditions, industry workload, and approval processes.

Table 3-6: Opinion of Probable Cost for Proposed Distribution Upgrades

Proposed Upgrade		Description		Cost
1	Additional Storage	A	Utilize existing PS2 storage structure for additional domestic water storage.	**$0.76M**
		B	Construction of an additional water tower at the Hudson Street water tower site.	**$3.00M**
2	Pipe Upsizing	A	Replace approx. 2,030 m of existing 150 mm pipe with 300 mm pipe.	**$4.06M** (at $2,000/m)
		B	Replace approx. 420 m of existing 300 mm pipe with 400 mm pipe.	**$0.91M** (at $2,175/m)
3	Additional Supply Well (Fairchild Place)	Construction of an approx. 1,600 m direct transmission main between the Fairchild Place well and the Hudson Street water tower. Assumed transmission line size of 300 mm.		**$3.20M** (at $2,000/m)

It is noted that for Proposed Upgrade 1A, if it is preferred that Well 2A remains as the water source for PS2, additional upgrades relating to water treatment will be required. For more details and OPCs for these upgrades, see Section 5.2.

The OPC for Proposed Upgrade 1B in Table 3-6 is based on tower dimensions required to meet peak balancing flow requirements. If the Municipality intends to also add sufficient fire flow storage to meet the ACWWA requirements without any flow contribution from PS2, the additional water tower would require more storage than what is necessary to increase peak balancing storage only. It is projected that increasing the water tower size as required to accommodate additional fire flow storage would increase the OPC for the additional water tower by $1.00M, increasing the total additional storage cost to $4.00M.

An OPC has been provided in Table 3-6 related to a transmission main for the Fairchild Place well (Proposed Upgrade 3), which is a potential new supply well that is covered in more detail in Section 5.2. To facilitate a direct connection between the well and the Hudson Street water tower, a transmission main that is at minimum approximately 1.6 km long would be required, assuming transmission line runs through public rights-of-way only and easements are not utilized.

3.6.2 General Recommendations

Proposed System Changes

The model results show maximum domestic pressures in the existing system to be slightly exceeding 90 psi, but less than 100 psi, which is considered acceptable as per the ACWWA Water Supply Guidelines. Pressures are highest in the network at the southernmost extents of the serviced area, in the MacElmon-Canso-Lysander area. If in the future, the water network in this area is extended and finished grade elevations are lower than approximately 16.5 meters, pressure reducing valves should be incorporated for water services to maintain water pressures lower than 90 psi.

The model results also show domestic pressures in the Dakota Road area to be less than the typical acceptable minimum of 40 psi. As the Dakota Road loop is an isolated branch of the network, this provides an ideal opportunity to establish a new boosted pressure zone for this area. It is recommended to incorporate a new booster station on the Dakota Road water line to boost domestic pressures to the ideal range of 50-70 psi. Incorporation of a boosted zone would also be beneficial for future development. It was previously detailed that for larger-diameter water mains, in this case 300 mm and larger, available fire flows are generally limited by low system pressures. Establishing a boosted zone would improve service to existing customers in the Dakota Road area and alleviate one of the primary issues limiting available fire flow within the system.

As previously noted, available fire flow in the MacElmon-Lysander-Canso area, the Ventura-Tracker-Vickers area, the Dakota Road area, and at the Debert Airport is limited by velocity in the pipes reaching the maximum of 3.0 m/s. As existing water mains are replaced in these areas over time, it is recommended to upsize water mains in these areas to increase available fire flow. Experience shows that typically for dead-end water mains, a minimum pipe diameter of 350 mm or 400 mm is required to deliver the full recommended MICI fire flow and meet fire flow requirements. Smaller pipe diameters are typically sufficient to supply high fire flow to areas with looped systems where water can be supplied from multiple directions. Recommended pipe diameters for replacement of specific existing pipe sections in Debert other than what has been noted in the previous section are subject to detailed design and fire flow analysis.

As previously described, it was observed that the Hudson Street water tower alone does not have sufficient storage volume available to meet fire flow storage requirements, based on the design drawings for the tower (EXP, July 2017). Depending on long-term plans for the municipal water system, it may be beneficial to add 1.45 ML of fire flow storage to the system, potentially through the construction of an additional water tower at the Hudson Street water tower site which would add both peak balancing storage and fire flow storage to the system. Having a sufficient fire flow storage volume outside of the PS2 reservoir would add redundancy to the system in case of storage facility issues, as well as provide the option to disconnect or decommission PS2 in the future. If PS2 is to be decommissioned in the future, the water towers supplying fire flow to the system will need to be equipped with a fire pump.

Additional Analysis

Based on the assessment of unaccounted-for water usage, it was determined that only approximately 43% of water use was accounted for in billing data. It is highly recommended to further investigate this issue, as there could be significant leakage in the system. It is recommended to begin with investigating any abnormalities in billing data, such as missing readings or misreads due to malfunctioning equipment. Upon completing this, a robust leak detection program should be conducted.

It is also recommended to upgrade the hydraulic model to run an Extended Period Simulation (EPS). Running the model as an EPS would demonstrate how the system responds to varying flow conditions over time, with Well 1C operating and the water tower filling and draining based on actual setpoints in the system. An EPS analysis also allows the water age/ quality to be modelled. This type of analysis can provide useful information, such as areas which may require regular flushing to maintain acceptable water quality. It is noted that if the model is to be upgraded to run an EPS, the submersible pump at Well 1C will need to be identified.

4.0 WATER TREATMENT

4.1 Background Review and Operation Conditions

Dillon Consulting Limited (Dillon) technical staff attended a meeting and site visit with the Municipality for a capacity study on September 7, 2023, along with DesignPoint, to review the priorities of the community for developing upgrades to the existing Debert Water Utility system. During the site visit a visual inspection of accessible infrastructure was completed of the raw water wells (WS1C and WS2A), pumping stations (PS1 and PS2), control buildings for both pumping stations and two water reservoirs (Hudson Street water storage tank and Plains Road water storage tank) along with discussions on operation and maintenance concerns to obtain a better understanding of infrastructure issues and conditions. Following the site visit Dillon completed a review of relevant background information to gain a better understanding of the infrastructure. Information reviewed included:

- Background Reports and Studies:
  - Debert Water Utility System Assessment Report 2023;
  - Debert Air Industrial Park Wellfield and Water Distribution System 2005;
  - Debert Air Industrial Park CFS Camp Debert Infrastructure Assessment 1998;
  - Debert Water Utility Operations Manual 2020;
  - Approval for Operation- Water Treatment Facility 2022; and
  - Approval for Operation- Groundwater Withdrawal 2015.
- Data:
  - Municipal Drinking Water Supplies Annual Reports 2020-2022; and
  - Daily water flows and Regular water sampling 2020-2022.

4.2 Current Operating Conditions

The Debert Water Treatment Plant operates under Operating Approval to Operate No. 2013-088390-02 expiring December 31, 2029, and Withdrawal Approval No. 2015-092356 expiring May 21, 2025. The system operates two (2) Non-GUDI wells and two (2) reservoirs. Disinfection is provided by Sodium Hypochlorite.

4.2.1 Water Withdraw

Table 4-1 below summarizes the Withdrawal Approval limits compared to the water demands from the Town based on the 2019-2022 data provided by the Municipality. Only 2019 and 2020 data were available for WS2A as it was not operational in 2022.

Table 4-1: Conformance to Withdrawal Approval

Approval Requirements	Approval Limit (m^3)	WS1C (m^3)	WS2A (m^3)	Total (m^3)
Annual Maximum	547,500	172,540	282.6	172,822.6
30 day Maximum	45,000	22,200	165	22,370.1
Average (over 30 days)	1,500	740	5.5	745.5
Maximum (Averaged over 3 days)	Total: 2,585.816 WS1C: 1,603.860 (245 igpm) WS2A: 981.95 (150 igpm)	1,010.8	38.5	1,049.3

Based on the Government of Nova Scotia well log data base, the yield for WS1C is 1135 L/min (250 igpm) which is just above the max withdrawal approval rate of 245 igpm. No specific yield was provided for WS2A however, an average pumping rate of 981.82 m^3/day is congruent with the three-day maximum Approval pumping rate. Specific pumping test for maximum safe yields for the wells were not provided. As such it is not clear if there is possible additional capacity within the aquafer to withdraw additional water from the existing wells. As a result, a confirmatory pumping test or proof of pumping test results would be required to determine the maximum yield of the wells to increase the pumping rate of the wells.

Figure 4-1 below shows the withdrawal rate and 30-day average over time for WS1C used for operational demand. Overall, there is no significant trending increase in water demand from 2020 to 2022.

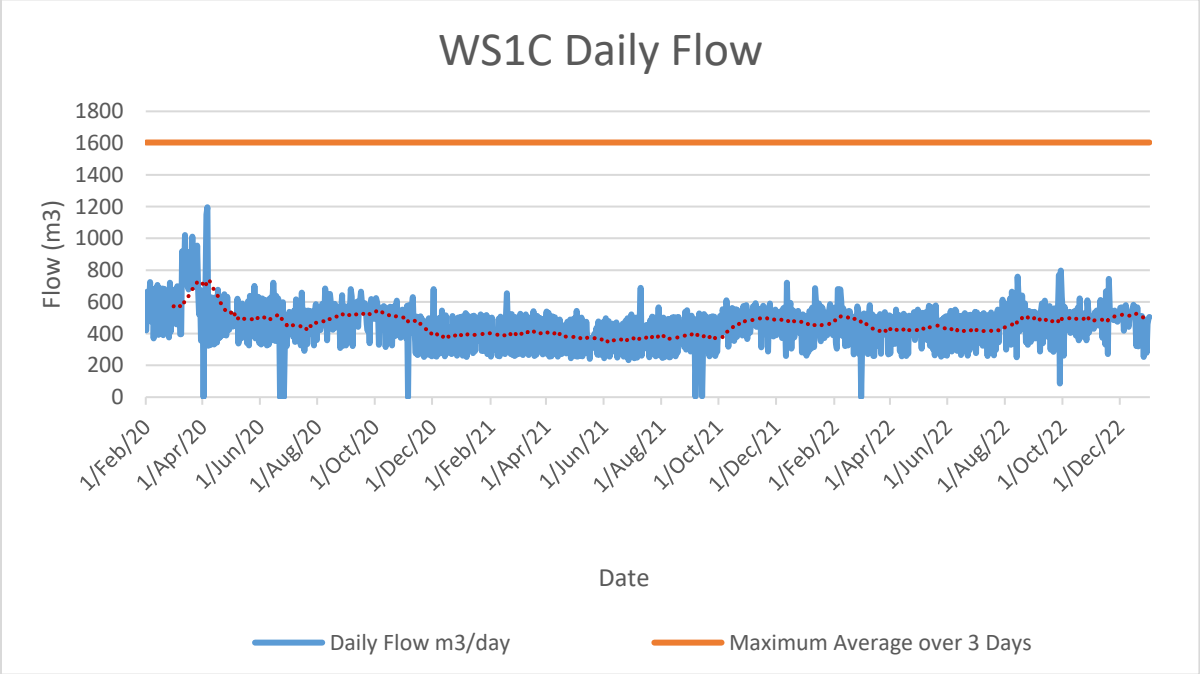


Figure 4-1: WS1C 2020-2022 Daily Flow

From the graph above, the January 2020 data has been excluded from the assessment due to errors identified in the provided information. There were other instances where the flow chart indicated a value of zero which can be attributed to either communication failures, or a programming malfunction in the Programmable Logic Controller (PLC) at WS1C as a result these technical issues may have impacted the accuracy of the data during that period.

WS2A is not operated regularly with a maximum of 32 day per year operational periods in a given year. In each instance, the withdrawal rate was below 677.4 m^3 per month. Due to the frequency of operation, no flow analysis was completed.

4.2.2 Water Quality

Per the approval to operate the following conditions are required by the system:

- Maximum Turbidity of 1 NTU 95% of the time from the wells;
- Maximum Turbidity of 5 NTU in the distribution system; and
- Chlorine residual throughout the system to be 0.20 mg/L free chlorine and not to exceed 4.0 mg/L free chlorine.

Based on the information provided for 2020- 2022 the wells meet the 1 NTU 95% of the time for requirement for Turbidity. Where high values of turbidity values were noted, there was also a reduction of CT residual in the system however only one (1) instance of low chlorine residual was noted in the distribution system in February 2020. Variation in turbidity and chlorine has been due to specific changes in the system (i.e., pump replacement, alarm failure, etc.) as described by the Municipality in the annual reports and has been controlled or mitigated by operational adjustments.

Chlorine residual data from 2020 to 2022 for WS1C, WS2A, Hudson Street Water Tower, and PS 2 show levels remaining consistent below the maximum limit of 4.0 mg/L for all location these are monitored at 5-minute intervals. This consistent pattern indicates that the 0.20 to 4.0 mg/L limit was consistently met. Instances where the chlorine residuals fell below the required value, the well shut down. Most instances where low residual was noted there were explanations for the low chlorine residual due to instances of communication losses and failures between the wells and pumping stations, resulting in instances of zero chlorine residual data. Data was provided for PS2 Reservoir for only two (2) years (2020 and 2021) as the well pump needed to be replaced.

4.3 Future Flowrate Demands

DesignPoint provided theoretical flows for the proposed future developments which include the Mi'kmawey Debert Cultural Centre (MDCC) and Hudson Street Residential area. A summary of these demands is provided in Table 4-2.

Table 4-2: Future Development Additional Flow Demand

	Additional Development Demand
Average Daily Demand	554 m^3/day
Maximum Daily Demand	1,277.8 m^3/day
Peak Hourly Demand	78.5 m^3/hour

Table 4-3 below incorporates the future development demands provided by DesignPoint and 2022 flowrate data provided by the Municipality in comparison to the Approval to withdraw and withdraw limits for each well.

Table 4-3: Approval to Withdraw Limit Compared to the Future Required Flow

	Approval Limit	WSC1	WS2A	Future Required Flow
Average Day Demand	1,500 m^3/day	1603.8 m^3/day	981.9 m^3/day	1,065 m^3/day
Maximum Day Demand	2,585.8 m^3/day			2,075.8 m^3/day

The new average daily flow rate falls within an acceptable rate of the approval, however, the new maximum daily demand of 2075.8 m^3/day exceeds the WSC1 approval limit for withdrawal. This is a noted concern as the current operational practices of the utility operate WSC1 exclusively for domestic water supply while WS2A is used to satisfy fire flow requirements.

Based on the current operational practices of the facility WS2A functions exclusively as a fire protection supply within the system which WS1C operates to meet the current demand of the system. To maintain the same operational approach an additional water supply will be required, assuming that WSCA control building and storage will still be dedicated for fire flow. If it is preferred that WSC2 be used for distribution, confirmation that sufficient fire storage and flow will be required.

4.4 Equipment Evaluation

From the background information provided by the Municipality and site observations, Dillon has outlined the capacity of the existing system. A general process flow diagram is provided in Appendix E: Water Treatment System Process Flow Diagrams to reference the treatment sequence.

There are three main sections of the water system used for supply and treatment:

1. Well (WS1C) and Control Building;
2. Hudson Street Water Tower and Pumping Station 1 (PS1); and
3. Well (WS2A), Control Building and Plains Rd PS2/Reservoir.

The Debert Water Utility includes two supply wells, water from WSC1 is disinfected and then sent to distribution. The control to operate WS1C is based on the Hudson Street Water Tower level. Water feeds this tower from the distribution system. WS2A, PS2 and reservoir operate as a distinct system from the main operations with WS1C and the Hudson Street Water Tower. The system is used for fire flow operations, WS2A provides water to the tank when it is emptied. The tank is then solely used for fire flow events through the use of a fire pump.

Based on the background documentation the water treatment system has been rated 1135 Lpm in normal operation and additional capacity available for fire flow requirements. The sections below outline the current capacity of the existing equipment. For the Debert water treatment system, the equipment evaluation has been divided into three sections noted above.

The condition assessment results will be incorporated into the capacity assessment for those cases where the condition of a piece of equipment has reduced its effective capacity. For components that are found to not have sufficient capacity to meet the projected future demands, Dillon will explore options to determine what operational change can occur, what equipment or unit processes can be upgraded, or if outright replacement of the facility is required.

4.4.1 Well (1C) and Control Building

Water is conveyed from WS1C through a simplex submersible raw water pump to the control building where duplex variable speed chemical dosing pumps inject sodium hypochlorite stored in a 150-litre drum tank. The chlorine metering pumps operate in duty/standby mode as the feed rate is paced to match the flow rate from WS1C measured by flowmeter, however, there is no auto switchover should one pump fail. Operators are notified by the SCADA System via a low residual alarm that shuts down the well pump. The chlorinated water flows into a series of serpentine pipes to provide contact time prior to distribution. An Olympian back-up generator is present at WS1C for emergency backup power to maintain the water supply.

Table 4-4 outlines the existing capacities of the Well (1C) and Control Building equipment and the future recommended capacities based on the future demand requirement conditions noted in Section 4.2.

Table 4-4: Well (WS1C) and Control Building Capacity Summary

Equipment	Quantity	Configuration	Details	Current Max Capacity	Future Required Capacity
Well (WS1C)	1	Duty	Five- stage Pleuger Flowserve submersible pump (PN081-05-06)	1135 Lpm @ TDH of 112.8m	1440 Lpm
Duplex Dosing pumps	2	Duty/Standby	Pulsatron model LMB3TA-VHC9, Premia 75 Series Micro	1.9 L/hr and 1.89L/hr.	1.34 L/hr.
Sodium Hypochlorite tank	1	Duty	HDPE	150 L (8.3 days @ CT calculated conditions)	150 L for (4.5 days @ CT calculated conditions)
Chlorine Contact Piping	1	Duty	D: 0.2 m L: 382 m Baff:1 Assumed 12 mg*min/L required	1.12 mg/L CL_2 residual required @ 1115 LPM (245 igmp)	1.6 mg/L CL_2 residual required @ 1440 LPM (316 igmp)

The existing disinfection system is appropriately sized for the maximum output capabilities of the WS1C well pump and can meet CT residual requirements with 1.12 mg/L sodium hypochlorite based on the worst-case operating temperatures. As well the consumption rate of sodium hypochlorite allows for reasonable chemical restocking by the operations staff approximately every eight (8) days under these conditions.

The calculated future required flow rate for optimal contact time is established at 1440 Lpm (2075.8 m^3/day) is anticipated to result in a new residual concentration of 1.6 mg/L calculated for the worst-case conditions. Comparatively, the current flow rate requires 1.12 mg/L chlorine residual at 1135 Lpm under the same operating conditions.

Calculated chemical consumption assuming a 10% active concentration for the sodium hypochlorite solution, the dosing flow rate will increase from 0.72 L/hr. to 1.28 L/hr. The current duplex dosing equipment is sufficient to meet the new increase in dosing rate. However, there will be less operation flexibility and increased changes in sodium hypochlorite day tanks by operations staff under this future operating condition.

4.4.2 Hudson Street Water Tower and Pumping Station 1 (PS1)

The water from the distribution system enters the control building of the Hudson Street water storage tank. It first passes through a dual-direction flowmeter before reaching the water quality monitors. which are responsible for monitoring chlorine residual and temperature. Subsequently, the water is then stored and recirculated in the storage tank through a recirculation pump. The storage tank is also equipped with a separate de-chlorination chamber that drains to the Hudson Street ditch.

No water treatment is provided at this location, however, monitoring does occur for tank water level, flow, temperature, and chlorine residual. There is no method of re-chlorination at the tanks though recirculation is provided.

Table 4-5 outlines the existing capacities of the Hudson Street Water Tower equipment and the future recommended capacities based on the future demand requirement conditions noted in Section 4.2.

Table 4-5: Hudson Street Water Tower

Equipment	Quantity	Configuration	Details	Max Capacity
Water Tower	1	Duty	12.7 m Diameter and 27.5 m tall Operating points HL: 24.90m Pump off: 24.7 m Pump on: 23.00 m LL: 22.0 m	367.36 m^3 operating volume
Recirculation pump	1	Duty	Grundfos Magna 3 100/120 F450.	512 L/min @12.2 m

The Hudson Water Tower has limited operating volume to meet the hydraulic requirements of the system and it is not part of the treatment requirements. Water is fed from the distribution system to the Hudson St. Water Tower where CT residual has already been achieved. No concerns were noted regarding treatment for the stagnation or chlorine residual in the tower.

4.4.3 Well (2A), Control Building and Plains Rd PS2/Reservoir

Water is conveyed from WS2A which is located at 123 Dakota Road through a simplex submersible raw water pump to the control building where a single variable speed dosing pump injects the water with sodium hypochlorite stored in a 150-litre drum tank. The chlorinated water flows through the supply pipe that also provides contact time before entering into the control building at PS2 and the Plains Rd Reservoir. The chlorine residual is monitored as it enters the storage tank to confirm contact time and as it is recirculated. Should the CT be insufficient an alarm notification in sent to remove water from the tank and allow the well pump to turn on and provide additional chlorinated water.

The PS and the Plains Road reservoir have two jockey/recirculation pumps, one of the jockey pumps is not operational, a fire pump is serviced by a separate outlet from the reservoir. This fire pump serves as the main fire suppression system for the Debert area.

This fire pump is manually initiated when required. No further water is sent to distribution regularly according to current operational practices. A stand-by 30 kW Onan diesel generator with a Ford 4.15 L four-cylinder diesel motor rated 60 Hp is present on-site to operate the PS in the case of power failure.

Table 4-6 outlines the existing capacities of the Well (2A), Control Building, and Plains Rd PS2/Reservoir equipment and the future recommended capacities based on recommendations in Section 4.4.2 No changes are recommended for WS2A as it operates independently for emergency situations. This assumes that the new development does not increase the fire requirement in the Town. Confirmation of fire flow and storage requirements should be conducted when plans for development are completed including type and density of

construction. As such no required capacities have been provided however current operating levels are noted in relation to the size of the existing equipment. This assumes that the fire storage and flow are appropriately designed for the system.

Table 4-6: Well (2A), Control Building and Plains Rd PS2/Reservoir

Equipment	Quantity	Configuration	Details	Max Capacity	Required for Operation
Well 2A Pump	1	Duty	Webtrol, WS150075L	Noted as (150 GPM is assumed igpm) to be confirmed	150 GPM Assumed to be Sufficient
Sodium Hypochlorite tank	1	Duty	Drum, level remotely monitored	Current 150 L for 12.4 Days	150 L for 12.4 Days
Chemical dosing pump	1	Duty	Wallace and Tiernan Permia 75 Model # P75MPH2XAVHC9AXE	Current Max Capacity 1.89 L/hr.	Future Required Capacity 0.50 L/hr.
Chlorine Contact Piping	1	Duty	D:0.1 m L: 850m Baff:1 Assumed 12 mg*min/L required	1.23 mg/L required @ 682.5LPM (150 IGMP)	Assumed to be Sufficient
Water Storage Tank		Duty	3.6m H and 39.9m Diameter Volume 4,500 m^3	Operating volume 3,050m^3 @ Observed operating point 2.44 m	Assumed to be Sufficient
Fire Pump	1	Duty	Fairbanks Morse 5824	11,365 LPM (2500 GPM assumed is igpm) @100 PSI	Assumed to be Sufficient
Jockey Pump	2	Duty/not operational	1st Booster Pump Model: Fairbanks Morse 4-inch centrifugal pump 2nd Booster Pump Model: Aurora Centrifugal Pump.	1st Booster Pump: 1,135 Lpm, related head to be confirmed 2nd Booster Pump: Not operational	Assumed to be Sufficient

The existing disinfection system is appropriately sized for the maximum output capabilities of the WS 2C well pump and the ability to meet CT residual requirements with 1.23 mg/L sodium hypochlorite based on the worst-case operating temperatures. As well the consumption rate of sodium hypochlorite allows for reasonable chemical restocking by the operations staff approximately every 12 days under these conditions.

There was noted stagnation in the water tank, primarily because it is only used occasionally. Operation staff noted that when the chlorine residual in the tank is low, they empty a portion of the tank and re-fill it with

chlorinated water from the well. This process is completed manually and there is no method in place to re-chlorinate the tank without removing stagnant water. In addition, as chlorine levels decrease in the tank, there is a noticeable increase in turbidity. This may not solely be attributed to the extended storage period, as turbidity typically does not develop over time without the presence of sediment in the tank. It might be indicative of the need for tank cleaning rather consequence of long-term storage.

Future Population Projection

The latest population data available for Debert dates to 2006, indicating a population of 1,471 people. However, this information is outdated, and no recent data was found. To estimate the current population, we relied on the maximum demand from 2022 averaged over three days.

Considering the assessed capacity of the Debert water system, including both the 2022 maximum daily demand we have found there is a current excess capacity of nearly 593 m3/day. Using this information, we assessed the maximum number of units that could be accommodated before the need for a new well arises.

Table 4-7: Potential Additional Development Before Additional Well Required

Criteria	Max Daily Flow Demand	Remaining Current Capacity	Option 1	Option 2	Option 3
Debert Mi'kmawey Museum	84.2 m^3/day	593 m^3/day	1	1	1
Low Density Residential	2.93 m^3/unit day		160	-	86
High Density Residential	1.96 m^3/unit day		20	258	129

Analysis of the maximum daily flow demands for low-density and high-density residential units, based on DesignPoint Table 3-1, reveals significant insights. For low-density units, with a higher daily flow demand per unit, the estimated capacity allows for approximately 160 units plus an additional 20 high density units before a new well is needed. On the other hand if only high-density units, with a lower daily flow demand per unit are constructed, the water supply could sustain 258 units before requiring a new well.

5.0 CONCLUSIONS

5.1 Water Distribution

For domestic demand scenarios, average daily demands with an average water level in the water tower, and peak hourly demands with a low level in the water tower were assessed in the hydraulic model. Domestic pressures during the average daily demand scenario for existing development range between 26-96 psi, while domestic pressures during the peak hourly demand scenario for existing development range between 22-92 psi.

The resulting junction pressures cover a wide range due to the varying topography in the serviced area. Domestic pressures are low in the Dakota Road area, ranging from 22-43 psi during peak hourly demands (26-47 psi during the average daily demand scenario). Pressures are higher and range from 68-92 psi during peak hourly demands (72-96 psi during the average daily demand scenario) in the MacElmon-Canso-Lysander area at the southern extents of the network.

Model results show available fire flows for existing development ranging from 1,976 to 13,620 L/min across various scenarios, depending on if one or all fire flow storages are engaged and if maximum velocity requirements are applied to the outlet pipes for the storage structures. Distribution factors that are likely to be limiting available fire flow include low system pressures in certain areas, maximum velocity exceedances in small water mains, and high friction losses.

Recommendations for Existing System

Regarding water system performance for existing development only, the following recommendations are provided:

1. Establish a new boosted pressure zone for the Dakota Road area through construction of a booster station on Dakota Road, to boost domestic pressures to the ideal range of 50-70 psi.
2. Several areas were identified that are serviced by pipes that are not large enough to adequately supply the typical MICI fire flow and maximum daily demands and meet fire flow requirements. As water mains in these areas are replaced over time, it is recommended to upsize water mains in these areas to increase available fire flow.
3. Investigate and identify sources of unaccounted-for water in the system, both by investigating abnormalities in billing data and conducting a robust leak detection program.
4. Upgrade the hydraulic model to an Extended Period Simulation (EPS) to observe how the system responds to varying flow conditions over time. An EPS model also allows water age/quality to be modelled.

5.1.1 Future Development

Based on the steady-state analysis of the existing linear system, the system can adequately supply domestic demands to the future developments indicated in the RFP (Hudson Street residential development, MDCC serviced from Lancaster Crescent). However, the water storage assessment indicated that the existing water tower does not have sufficient capacity to meet peak balancing storage requirements for the future development scenario.

Model results show that the available fire flow at the future Hudson Street residential junction (J-12) and the future MDCC site (J-102) range between 11,402-12,932 L/min and 11,402-12,614 L/min, respectively. For all scenarios included in this analysis, available fire flows at the future development locations do not meet the typical fire flow recommendation of 13,620 L/min for these types of developments. However, experience has shown that the available fire flows at the proposed development locations are typically adequate for these types of developments.

Model results show that current storage volumes in the water tower do not have the capacity to meet peak balancing storage requirements for the future development scenario. Model results also showed that to deliver a fire flow of 13,620 L/min to the Hudson Street residential development, pipe upsizing is required in addition to fire flow supply from both the water tower and PS2.

Recommendations for Future Development

The following recommendations are provided for the distribution system:

1. Construct additional water storage that is sufficient to meet peak balancing requirements for the future development scenario. Two options have been considered in this report. See the following:
    a. Utilize available storage in PS2 as peak balancing storage. This option would require implementation of an on-site booster pump to supply domestic flow to the system at an appropriate HGL for the system (i.e. 84.0 m). Further detail of a booster pump for PS2 is subject to detailed design. This option will require additional water treatment upgrades if Well 2A is maintained as the supply for PS2. This is the preferred option as it would provide increased service with existing infrastructure. Should PS2 be decommissioned in the future, Option 1b could be implemented.
    b. Construct a second water tower at the Hudson Street water tower site. Depending on future plans for the Debert water system, additional storage can also be sized to create adequate fire flow storage between the two (2) water towers, adding redundancy to the system and providing the option to disconnect or decommission PS2 in the future. If PS2 is to be decommissioned in the future, the water towers supplying fire flow to the system will need to be equipped with a fire pump.
2. Upsizing existing water mains to supply the typical required fire flow to the Hudson Street residential development. Two options have been considered in this report covering a small to larger scope of work. See the following:
    a. Upsize approximately 2,030 m of 150 mm water main on Tracker Road, Vickers Street and Ventura Drive to 300 mm. This would provide the typical MICI fire flow to the proposed development and would replace aging infrastructure, provide redundancy to the system, and increase overall available fire flow to the area. While this option requires a larger capital investment, it would result in better overall system performance by improving undersized infrastructure along the route and providing redundancy.
    b. Upsize approximately 420 m of 300 mm water main on Hudson Street to 400 mm diameter pipe (provide a consistent 400 mm line from PS2 at Plains Road to the connection point of the proposed development).

3. If in the future the water network in the MacElmon-Canso-Lysander area is extended and finished grade elevations are lower than approximately 16.5 meters, pressure reducing valves should be incorporated for water services to maintain maximum water pressures of 90 psi.

Based on the model results and water system configuration, significant measures would be required to achieve 13,620 L/min of fire flow at the MDCC, including fire pump replacement. Experience shows that the available fire flow at the MDCC site is usually sufficient for this type of development. It is also noted that design and construction strategies can be employed to reduce fire flow requirements at the detailed design stage.

5.2 Water Treatment

Overall, the Debert Water treatment system operates within the current operating requirements but requires additional water capacity to meet the predicted future needs of the community (inclusive of future development additional growth). Should the Municipality wish to maintain the current operation of one well used for production and the other for fire flow, the following items below are recommended.

The costing provided is an opinion of probable cost. This estimate is subject to change based on several factors beyond Dillon's control, including market fluctuations, unforeseen project complexities, and variations in material or labor costs. These values should be considered budgetary as a detailed design or investigation is required to determine specific upgrade requirements.

Well (1C) and Control Building

It is recommended that the production volume of the wells in Debert be increased to meet the future flow requirements of the design based on the demands indicated in future developments. Due to the size of the increase and the relative capacities of other wells currently installed, it is recommended that this be accomplished through the addition of a well feeding the control building at WSC1. A pumping test could be completed to determine if it is possible to increase the overall production of the existing well however this would require the well to be taken offline for a three-day period likely resulting in significant water issues in the community. A second well used for production will provide increased security should the WSC1 pump need to be replaced or upgraded in the future. An assessment for the new well will be required to confirm the flows and treatment required. Assuming the well has similar characteristics to the existing wells (Non- GUDI, GCDWQ compliant) limited additional work would be needed to connect the well to the distribution system.

Should the combined flows of the new or upgraded WSC1 be greater than the expected future flow rate, confirmation of chlorine contact time will be required. Based on future flows from the proposed development sufficient chlorine contact time can be achieved.

A new well and connection of the existing line leaving WS1C is estimated to cost $300,000 to $500,000 however this is highly dependent on the geology in the area, the number of wells that need to be drilled and understanding of the hydrogeology in the region. There is also variability in the length and location of the transmission line required depending on the selected location.

Hudson Street Water Tower and Pumping Station 1 (PS1)

Depending on the system operations with the future developments in the community, should an impact be identified in CT residual levels within the Hudson Street Water Tower or difficulty maintaining CT residual

within the system two changes could be considered. Firstly, the operating pressure for the recirculation pump exceeds the recommended levels based on its capacity. The pump is currently in place to confirm chlorine residual in the tank and maintain water flow through the tank. Should operating conditions change and the CT within the tank becomes difficult to maintain this recirculation could be upgraded to increase mixing in the tank. Secondly, a chlorine dosing system could be added to provide a method of re-chlorination. No issues with Chlorine residual levels were noted for the current water flows in the system.

A new pump is estimated to cost $20,000 to $25,000 and a new chlorine dosing system is $10,000 to $20,000. Neither of these upgrades are considered critical for the operation of the system at this time.

Well (2A), Control Building and Plains Rd PS2/Reservoir

To enhance the overall efficiency and water quality within the Well (2A), Control Building, and Plains Rd PS2/Reservoir system, routine tank cleaning procedures could be implemented to mitigate the risk of stagnation and sediment accumulation which will impact water quality. This could also be completed by having a regular small volume of water sent to distribution at the same rate as WS2A production thus maintaining the fire storage and circulation of water within the tank regularly.

A review of the water quality for Well WS2A indicated that it could be a feasible water source, however treatment would be required to remove iron and manganese concentrations in excess of the aesthetic objectives of the Health Canada Guidelines for Canadian Drinking Water Quality. Should this avenue be chosen, the estimated cost based on recent projects of similar scale would be in the range of 1.5 to 2.5 million dollars. Factors which can impact the cost of this project would be if existing buildings can be re-purposed to house treatment equipment, the scale and complexity of the selected treatment process and other design factors. Suitable treatment technologies could include greensand filtration, traditional oxidation and filtration or biological filtration. We would recommend the Municipality complete a pre-design study to select an appropriate treatment technology, and pre-select an equipment package. The engineering effort estimate to complete such a pre-design would be in the range of $75-$100,000.

Proposed Fairchild Place Well

The Municipality has noted that a new supply well in the area of Fairchild Place has been drilled and developed. While water quality data for this well was not provided, assuming the well has similar quality to WS1C, chlorination and appropriate chlorine contact time would be required prior to connection to the distribution system. The new well would have to be equipped with a power supply, pitless adapter, pump and piping, as well as level monitoring equipment. A small wellhead building and RTU panel could be constructed next to the wellhead to house required electrical and controls equipment, and relay data back to the Municipality's SCADA system, as well as house a chlorine dosing system along with the associated process piping. Should further water quality testing find a need for additional treatment, the building would need to be sized appropriately to house additional filters or other treatment units.

To ensure effective disinfection, a CT pipe should be constructed with sufficient volume to ensure 4-log inactivation of viruses. Alternatively, the well could be piped to the Hudson Street Reservoir directly, such that the CT in the transmission main and reservoir combined achieve adequate disinfection.

We estimate that the costs associated with designing and installing the required wellhead equipment would be approximately $50,000, while the building along with the required power supply, and process equipment would be in the range of $250,000 to $500,000 depending on the final sizing and construction materials selected. These costs would increase if treatment beyond chlorination is required.

APPENDIX A: HYDRANT FLOW TEST RESULTS

WATER FLOW TEST SUMMARY

Type of Area:	Residential / Commercial	Test No:	Test #1 - Debert
Location:	1320 Masstown Rd & 1206 Plains Rd	Test By:	Tom Cameron
Municipality:	Colchester	Date:	October 25, 2023

- **SYSTEM DATA**

Size of Main: | Dead End: | Two Ways: | Loop:

Source Reliable: | If not, explain:

Comments: Results are in US GPM and PSI. Hyd coefficient is 0.09

- **TEST DATA**

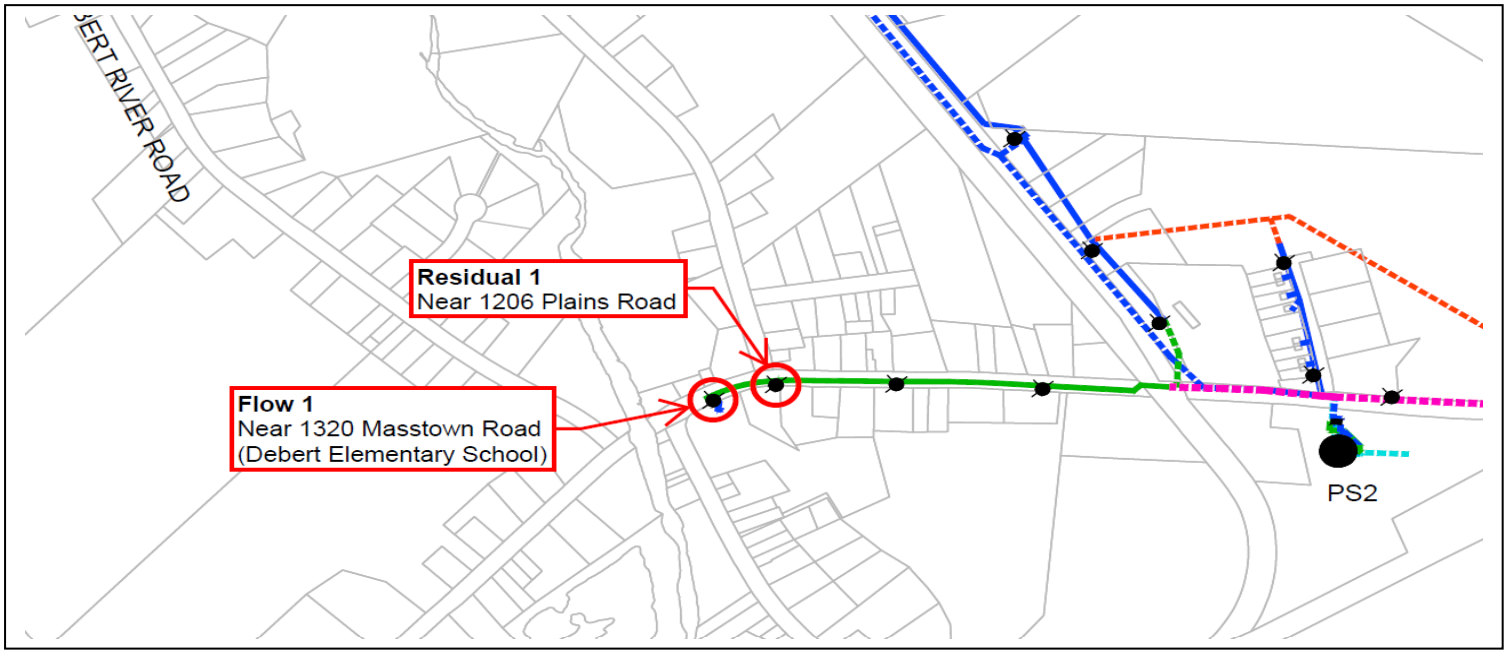
Location of Test Hydrants: Residual: See sketch below for location

Flow: See sketch below for location

Normal Pressure: 50 Time: Between 1:30-4:00 A.M. [ ] P.M. [X]

Test No.	No. of Outlets	Orifice Size (IN)	Pitotless Nozzle Reading (PSI)	Equivalent Flow (US GPM)	Total Flow (US GPM)	Residual Pressure (PSI)	Comments
1	1	2.5”	15	604	604	42	Had to Use
2	2	2.5”	12 / 13	540 / 562	1102	36	Hose Monsters
3							

Hydrant flow information from the files of the Aqua Data Atlantic, regardless of their original source, are maintained for internal use only. Although such information is often shared with others, people or firms who make use of this data do so at their own risk.



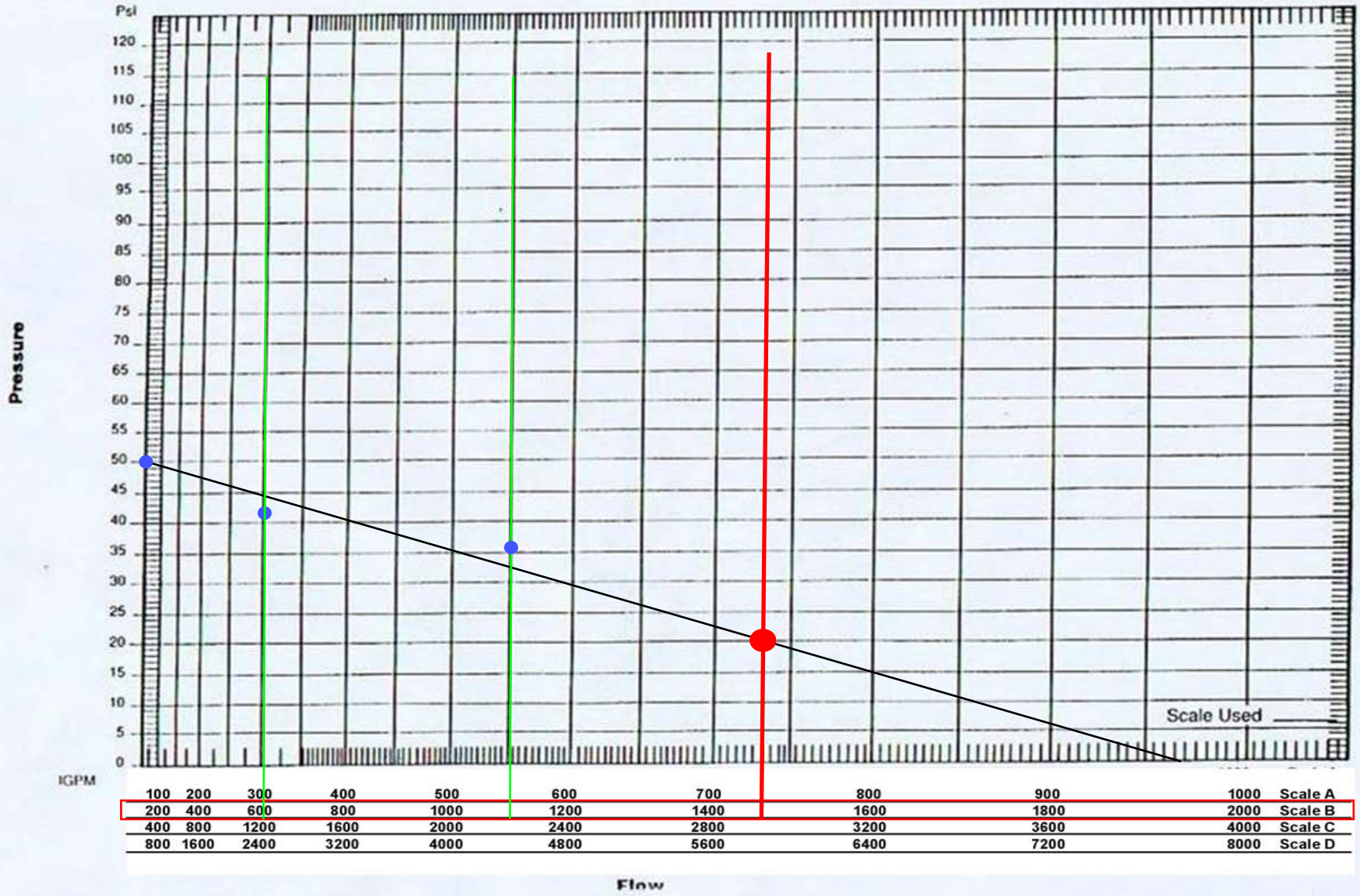
AQUA DATA Atlantic

Water Flow Test Summary

Location: 1320 Masstown Rd & 1206 Plains Rd

Municipality: Debert, NS

Date: Oct 25, 2023



WATER FLOW TEST SUMMARY

Type of Area:	Residential / Commercial	Test No:	Test #2 - Debert
Location:	151 Tracker Rd & Vickers St	Test By:	Tom Cameron
Municipality:	Colchester	Date:	October 25, 2023

- **SYSTEM DATA**

Size of Main: | Dead End: | Two Ways: | Loop:

Source Reliable: | If not, explain:

Comments: Results are in US GPM and PSI. Hyd coefficient is 0.09

- **TEST DATA**

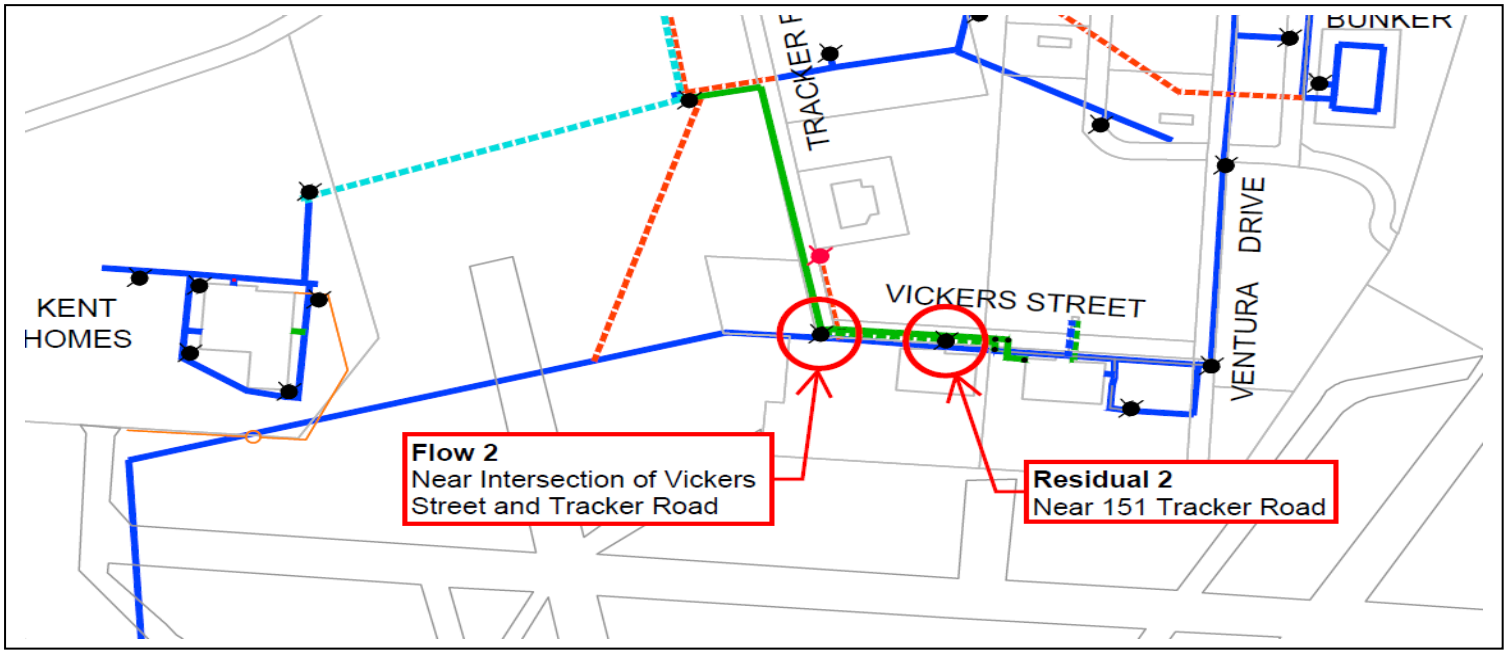
Location of Test Hydrants: Residual: See sketch below for location

Flow: See sketch below for location

Normal Pressure: 62 Time: Between 1:30-4:00 A.M. [ ] P.M. [X]

Test No.	No. of Outlets	Orifice Size (IN)	Pitotless Nozzle Reading (PSI)	Equivalent Flow (US GPM)	Total Flow (US GPM)	Residual Pressure (PSI)	Comments
1	1	2.5”	23	802	604	52	
2	2	2.5”	16 / 17	669 / 689	1358	44	
3							

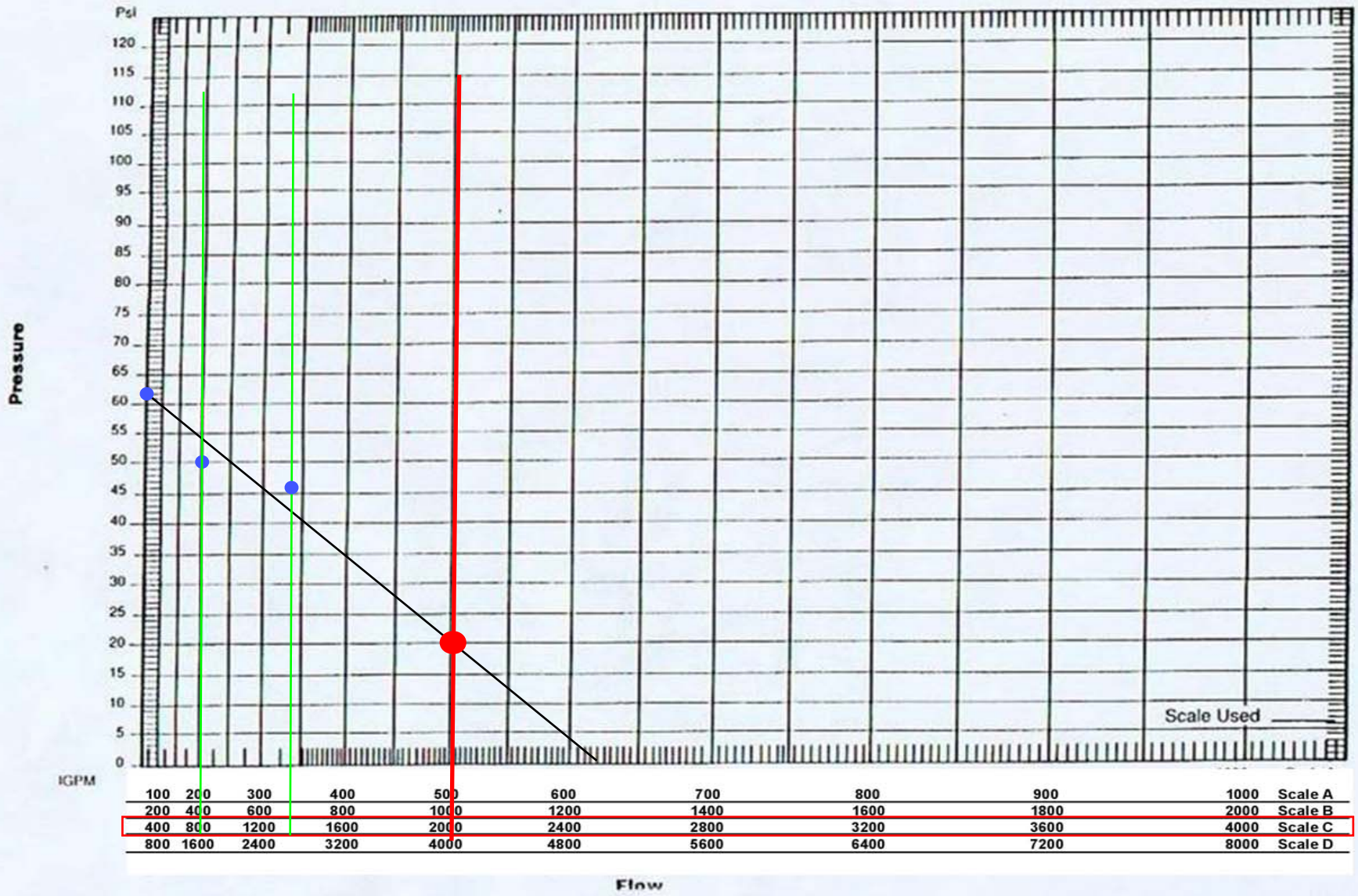
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AQUA DATA Atlantic

Water Flow Test Summary

Location: 151 Tracker Rd & Vickers St Municipality: Debert, NS Date: Oct 25, 2023



WATER FLOW TEST SUMMARY

Type of Area:	Residential / Commercial	Test No:	Test #3 - Debert
Location:	153 & 246 Lancaster Crescent	Test By:	Tom Cameron
Municipality:	Colchester	Date:	October 25, 2023

- **SYSTEM DATA**

Size of Main: | Dead End: | Two Ways: | Loop:

Source Reliable: | If not, explain:

Comments: Results are in US GPM and PSI. Hyd coefficient is 0.09

- **TEST DATA**

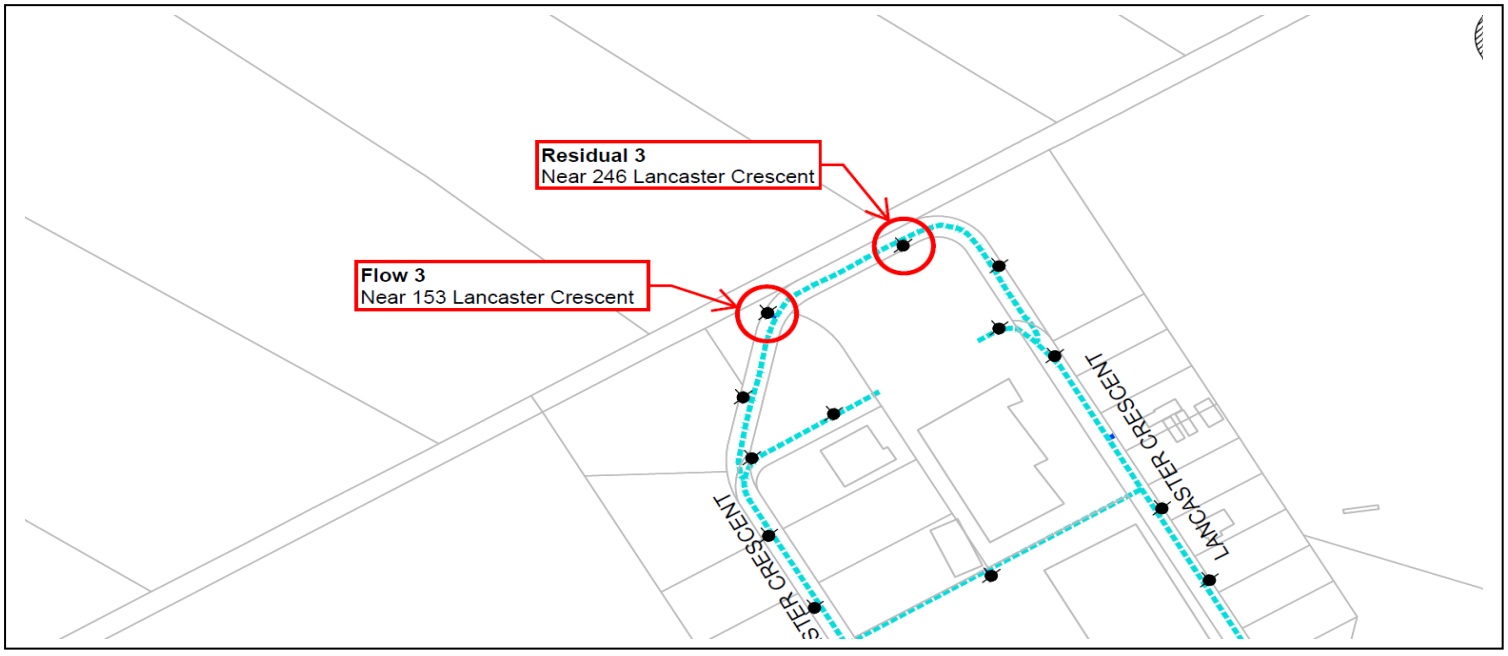
Location of Test Hydrants: Residual: See sketch below for location

Flow: See sketch below for location

Normal Pressure: 60 Time: Between 1:30-4:00 A.M. [ ] P.M. [X]

Test No.	No. of Outlets	Orifice Size (IN)	Pitotless Nozzle Reading (PSI)	Equivalent Flow (US GPM)	Total Flow (US GPM)	Residual Pressure (PSI)	Comments
1	1	2.5”	22	784	784	50	
2	2	2.5”	16 / 17	669 / 689	1358	46	
3							

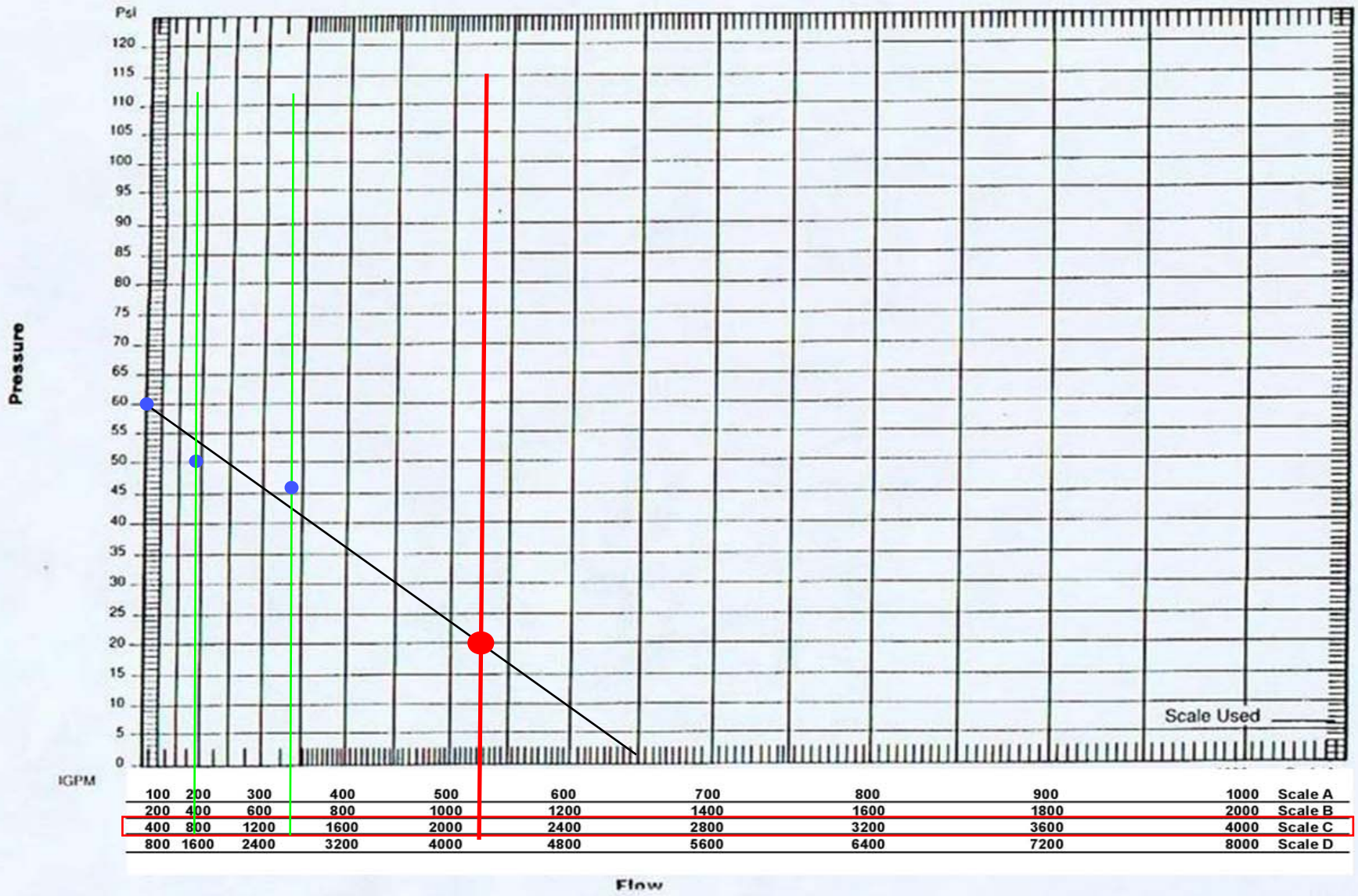
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AQUA DATA Atlantic

Water Flow Test Summary

Location: 153 & 246 Lancaster Crescent Municipality: Debert, NS Date: Oct 25, 2023



WATER FLOW TEST SUMMARY

Type of Area:	Residential / Commercial	Test No:	Test #4 - Debert
Location:	643 MacElmon Road	Test By:	Tom Cameron
Municipality:	Colchester	Date:	October 25, 2023

- **SYSTEM DATA**

Size of Main: | Dead End: | Two Ways: | Loop:

Source Reliable: | If not, explain:

Comments: Results are in US GPM and PSI. Hyd coefficient is 0.09

- **TEST DATA**

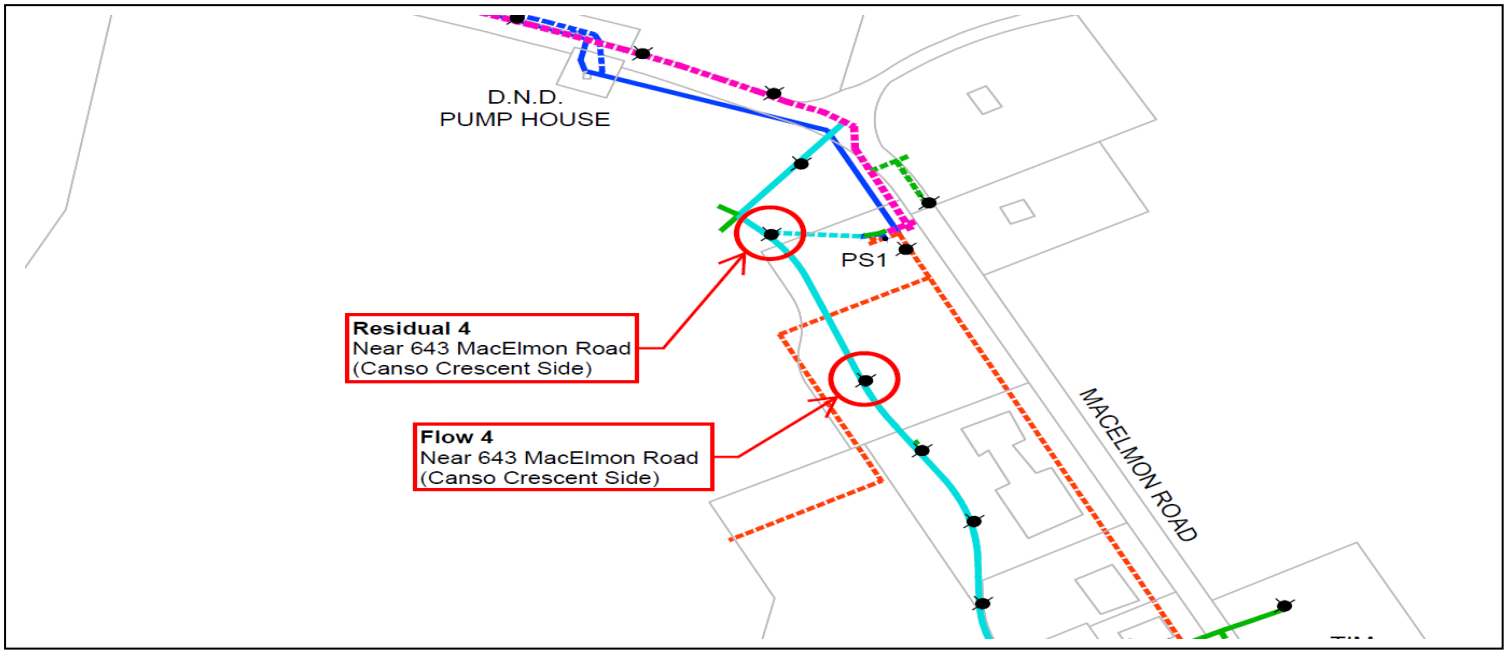
Location of Test Hydrants: Residual: See sketch below for location

Flow: See sketch below for location

Normal Pressure: 74 Time: Between 1:30-4:00 A.M. [ ] P.M. [X]

Test No.	No. of Outlets	Orifice Size (IN)	Pitotless Nozzle Reading (PSI)	Equivalent Flow (US GPM)	Total Flow (US GPM)	Residual Pressure (PSI)	Comments
1	1	2.5”	26	853	784	66	
2	2	2.5”	20 / 22	748 / 784	1532	58	
3							

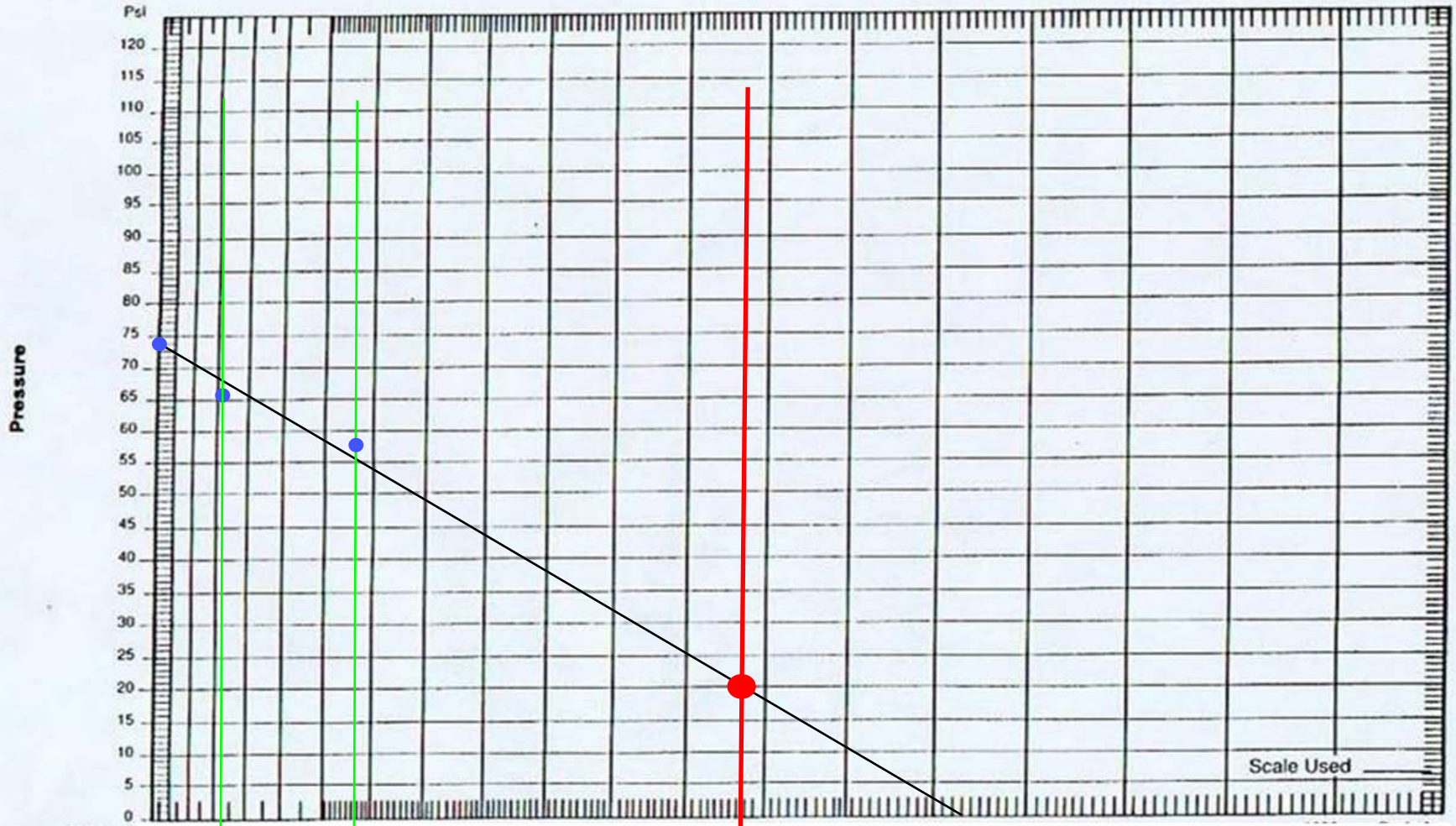
Hydrant flow information from the files of the Aqua Data Atlantic, regardless of their original source, are maintained for internal use only. Although such information is often shared with others, people or firms who make use of this data do so at their own risk.



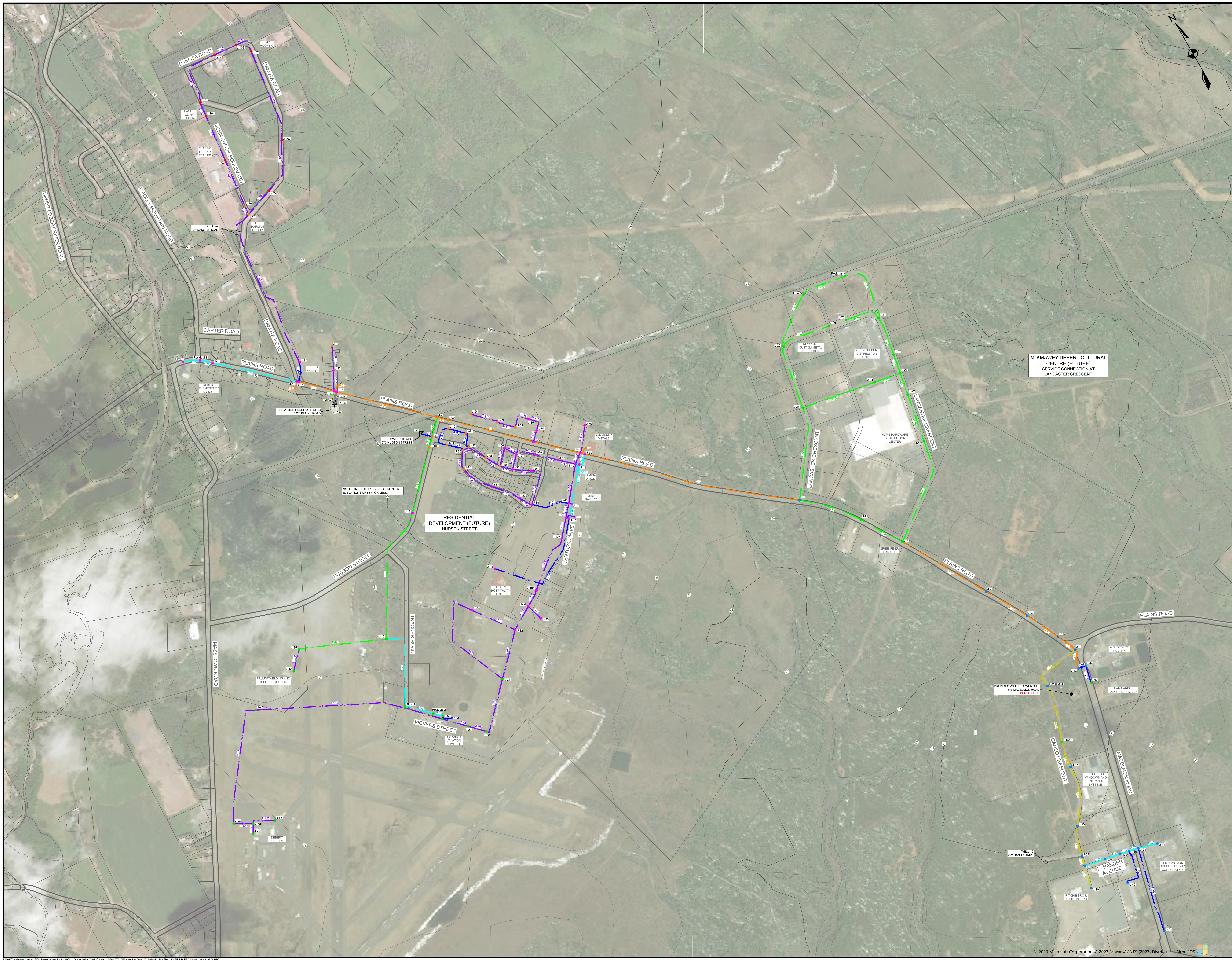
AQUA DATA Atlantic

Water Flow Test Summary

Location: 643 MacElmon Road Municipality: Debert, NS Date: Oct 25, 2023



APPENDIX B: WATER SYSTEM SCHEMATIC



LEGEND

EXISTING 100 mm PIPE	WA-E
EXISTING 150 mm PIPE	WA-E
EXISTING 200 mm PIPE	WA-E
EXISTING 250 mm PIPE	WA-E
EXISTING 300 mm PIPE	WA-E
EXISTING 350 mm PIPE	WA-E
EXISTING 400 mm PIPE	WA-E
EXISTING PIPE (INACTIVE)	WA-E

JUNCTION	J-000	TANK	T-0
WELL	R-0	PUMP	PMP-0

NOTES:

1. RESULTING PRESSURES ARE BASED ON THE PEAK HOURLY DEMAND SCENARIO INCLUDING FUTURE DEVELOPMENT, AND ASSUME A LOW DOMESTIC WATER ELEVATION OF 81.3 m IN THE WATER TOWER (T-1).
2. EXISTING PIPE INFORMATION SHOWN IS BASED ON BEST AVAILABLE INFORMATION, INCLUDING GIS DATA PROVIDED BY THE MUNICIPALITY OF THE COUNTY OF COLCHESTER.
3. ASSUMED UNIT COUNT FOR FUTURE HUDSON STREET RESIDENTIAL DEVELOPMENT IS AS FOLLOWS: 160 LOW-DENSITY RESIDENTIAL UNITS AND 368 HIGH-DENSITY RESIDENTIAL UNITS, BASED ON 2022 CONCEPT PLAN (FATHOM).
4. ASSUMED WATER DEMAND FOR FUTURE MI'KMAWEY DEBERT CULTURAL CENTRE CALCULATED AS PER ACWWA WATER SUPPLY GUIDELINES (2023) AND DESIGN NARRATIVE COMPLETED BY CRANDALL ENGINEERING (NOW ENGLOBE), DATED JANUARY 2021.
5. FUTURE DEVELOPMENT SHOULD BE LIMITED TO FINISHED GRADE ELEVATIONS OF 53 m OR LESS TO KEEP PRESSURES ABOVE THE ACCEPTABLE MINIMUM.
6. ELEVATIONS INCLUDING CONTOURS ARE BASED ON VERTICAL DATUM CGVD2013.

Color Coding Legend
Junction: Pressure (psi)

- <= 30
- = 31-40
- = 41-50
- = 51-70
- = 70-100

METRES 150 0 150 300
1:5000

ISSUE	DATE	DESCRIPTION
2	MAR. 25, 2024	ISSUED WITH FINAL REPORT
1	JAN. 22, 2024	ISSUED WITH DRAFT REPORT

CONSULTANT

DESIGNPOINT
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902.832.5597 designpoint.ca

REGISTERED PROFESSIONAL ENGINEER — DATE MAR. 25, 2024 — A. J. SKETCHLEY — 11560 — PROVINCE OF NOVA SCOTIA

REGISTERED PROFESSIONAL ENGINEER — DATE MAR. 25, 2024 — L. A. KING — 20200507 — PROVINCE OF NOVA SCOTIA

CLIENT

MUNICIPALITY OF THE COUNTY OF COLCHESTER

PROJECT DESCRIPTION

DEBERT WATER SYSTEM CAPACITY ASSESSMENT

DEBERT, NOVA SCOTIA

SHEET DESCRIPTION

WATER SCHEMATIC DOMESTIC DEMAND RESULTS

Drawn	Engineer	Project No.	Drawing No.
H. SAMPSON	L. KING	23-396	C-WA01
Scale	Filename		1 OF 2
1:5000	23-396_WA_DEB.dwg		



LEGEND

EXISTING 100 mm PIPE	100.0 100.0 100.0
EXISTING 150 mm PIPE	150.0 150.0 150.0
EXISTING 200 mm PIPE	200.0 200.0 200.0
EXISTING 250 mm PIPE	250.0 250.0 250.0
EXISTING 300 mm PIPE	300.0 300.0 300.0
EXISTING 350 mm PIPE	350.0 350.0 350.0
EXISTING 400 mm PIPE	400.0 400.0 400.0
EXISTING PIPE (INACTIVE)	000.0 000.0 000.0

JUNCTION	J-000	TANK	T-0
WELL	R-0	PUMP	PMP-0

NOTES:
1. RESULTING AVAILABLE FIRE FLOWS BASED ON MAXIMUM DAILY DEMAND + FIRE FLOW SCENARIO INCLUDING FUTURE DEVELOPMENT. FIRE FLOW SUPPLY FROM HUDSON STREET WATER TOWER AND PS2, AND NO VELOCITY LIMITATIONS ON STORAGE OUTLET PIPES. WATER ELEVATION OF 49.8 m ASSUMED IN PS2, AND 81.3 m ASSUMED IN THE HUDSON STREET WATER TOWER (T-1).
2. EXISTING PIPE INFORMATION SHOWN IS BASED ON BEST AVAILABLE INFORMATION, INCLUDING GIS DATA PROVIDED BY THE MUNICIPALITY OF THE COUNTY OF COLCHESTER.
3. ASSUMED UNIT COUNT FOR FUTURE HUDSON STREET RESIDENTIAL DEVELOPMENT AS FOLLOWS: 160 LOW-DENSITY RESIDENTIAL UNITS AND 366 HIGH-DENSITY RESIDENTIAL UNITS, BASED ON 2022 CONCEPT PLAN (FATHOM).
4. ASSUMED WATER DEMAND FOR FUTURE MI'KMAWEY DEBERT CULTURAL CENTRE CALCULATED AS PER ACWWA WATER SUPPLY GUIDELINES (2023) AND DESIGN NARRATIVE COMPLETED BY CRANDALL ENGINEERING (NOW ENGLOBE), DATED JANUARY 2021.
6. ELEVATIONS INCLUDING CONTOURS ARE BASED ON VERTICAL DATUM CGVD2013.

Color Coding Legend
Junction: Available Fire Flow (L/min)

- <= 3,300
- = 3,301-4,542
- = 4,543-10,000
- = 10,001-13,620

150 0 METRES 150 300

1:5000

ISSUE	DATE	DESCRIPTION
2	MAR. 25, 2024	ISSUED WITH FINAL REPORT
1	JAN. 22, 2024	ISSUED WITH DRAFT REPORT

CONSULTANT

DESIGNPOINT
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902.832.5597 designpoint.ca

REGISTERED PROFESSIONAL ENGINEER PROVINCE OF NOVA SCOTIA — DATED MAR. 25, 2024 — A. J. SMITH — 11560

REGISTERED PROFESSIONAL ENGINEER PROVINCE OF NOVA SCOTIA — DATED MAR. 25, 2024 — L. A. KING — 2020050?

CLIENT

MUNICIPALITY OF THE COUNTY OF COLCHESTER

PROJECT DESCRIPTION

DEBERT WATER SYSTEM CAPACITY ASSESSMENT

DEBERT, NOVA SCOTIA

SHEET DESCRIPTION

WATER SCHEMATIC
AVAILABLE FIRE FLOW RESULTS

Drawn: H. SAMPSON	Engineer: L. KING	Project No.: 23-396	Drawing No.: C-WA02
Scale: 1:5000	Filename: 23-396_WA_DEB.dwg		2 of 2

T:\2023\23-396 Municipality of Colchester - Capacity Studies\21 - Drawings\Eng Design\Sheets\23-396_WA_DEB.dwg, Plot Date: 2024-Mar-25, Plot Size: ISO FULL BLEED A0 (841.00 X 1189.00 MM)

APPENDIX C: WATER MODEL RESULTS (DOMESTIC)

Water Model Results

Average Daily Demand Scenario

Existing Conditions - Average Daily Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
Flow 1	48.0	231	84.0	51
Flow 2	40.1	0	84.0	62
Flow 3	44.3	0	84.0	56
Flow 4	33.2	0	84.0	72
J-1	56.6	0	84.0	39
J-2	55.6	0	84.0	40
J-3	51.8	0	84.0	46
J-4	48.3	4	84.0	51
J-5	30.9	0	84.0	75
J-6	31.8	88	84.0	74
J-7	27.6	0	84.0	80
J-8	29.7	0	84.0	77
J-9	22.6	0	84.0	87
J-10	21.1	0	84.0	89
J-11	17.3	426	84.0	95
J-12	51.3	0	84.0	46
J-13	41.1	0	84.0	61
J-14	49.9	0	84.0	48
J-15	50.5	0	84.0	48
J-16	50.6	0	84.0	48
J-17	51.1	1694	84.0	47
J-18	57.0	5	84.0	38
J-19	63.4	2	84.0	29
J-20	21.9	276	84.0	88
J-21	39.6	0	84.0	63
J-22	42.6	0	84.0	59
J-23	39.2	0	84.0	64
J-24	38.9	0	84.0	64
J-25	42.0	0	84.0	60
J-26	40.0	0	84.0	62
J-27	39.6	0	84.0	63
J-28	35.8	0	84.0	68
J-29	40.7	0	84.0	61
J-30	42.4	0	84.0	59
J-31	45.2	0	84.0	55
J-32	45.7	57	84.0	54
J-33	50.2	95	84.0	48
J-34	46.3	0	84.0	54
J-35	45.9	0	84.0	54
J-36	54.7	122	84.0	42
J-37	54.6	353	84.0	42
J-38	52.1	274	84.0	45

Water Model Results

Average Daily Demand Scenario

Existing Conditions - Average Daily Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
J-39	49.3	49	84.0	49
J-40	48.1	14	84.0	51
J-41	55.3	239	84.0	41
J-42	19.0	32	84.0	92
J-43	30.6	0	84.0	76
J-44	31.5	3359	84.0	75
J-45	49.6	294	84.0	49
J-46	46.3	0	84.0	53
J-47	46.0	293	84.0	54
J-48	46.3	0	84.0	53
J-49	47.8	36	84.0	51
J-50	48.0	19	84.0	51
J-51	40.7	3	84.0	62
J-52	23.1	0	84.0	87
J-53	19.2	0	84.0	92
J-54	16.5	0	84.0	96
J-55	52.7	97	84.0	44
J-56	52.4	21	84.0	45
J-57	51.6	232	84.0	46
J-58	49.7	0	84.0	49
J-59	48.2	0	84.0	51
J-60	32.3	34	84.0	73
J-61	52.1	199	84.0	45
J-62	53.6	163	84.0	43
J-63	54.6	201	84.0	42
J-64	55.1	52	84.0	41
J-65	35.6	84	84.0	69
J-66	21.4	374	84.0	89
J-67	30.4	1272	84.0	76
J-68	22.7	148	84.0	87
J-69	42.9	48	84.0	58
J-70	41.0	227	84.0	61
J-71	33.6	3	84.0	72
J-72	63.0	1258	84.0	30
J-73	63.7	3	84.0	29
J-74	61.2	2	84.0	32
J-75	61.4	1	84.0	32
J-76	57.4	45	84.0	38
J-77	39.2	5	84.0	64
J-78	35.7	568	84.0	69
J-79	35.2	2	84.0	69
J-80	40.6	17	84.0	62

Water Model Results

Average Daily Demand Scenario

Existing Conditions - Average Daily Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
J-81	51.7	3	84.0	46
J-82	49.6	0	84.0	49
J-83	51.8	291	84.0	46
J-84	53.5	55	84.0	43
J-85	52.6	622	84.0	45
J-86	50.6	257	84.0	47
J-87	52.4	387	84.0	45
J-88	53.8	500	84.0	43
J-89	50.5	150	84.0	48
J-90	50.3	337	84.0	48
J-91	55.8	401	84.0	40
J-92	38.5	1848	84.0	65
J-93	25.1	0	84.0	84
J-94	47.4	0	84.0	52
J-95	53.5	223	84.0	43
J-96	27.5	0	84.0	80
J-97	31.2	0	84.0	75
J-98	39.8	0	84.0	63
J-99	55.9	0	84.0	40
J-100	56.8	0	84.0	39
J-101	56.1	169	84.0	40
J-102	36.1	0	84.0	68
J-103	63.9	0	84.0	29
J-104	65.8	0	84.0	26
J-105	46.5	4	84.0	53
J-106	47.0	9	84.0	53
J-107	60.2	14	84.0	34
Residual 1	51.8	0	84.0	46
Residual 2	40.4	0	84.0	62
Residual 3	43.0	0	84.0	58
Residual 4	31.2	0	84.0	75

Water Model Results

Average Daily Demand Scenario

Including Future Development - Average Daily Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
Flow 1	48.0	231	84.0	51
Flow 2	40.1	0	83.9	62
Flow 3	44.3	0	83.9	56
Flow 4	33.2	0	83.9	72
J-1	56.6	0	84.0	39
J-2	55.6	0	84.0	40
J-3	51.8	0	83.9	46
J-4	48.3	4	83.9	51
J-5	30.9	0	83.9	75
J-6	31.8	88	83.9	74
J-7	27.6	0	83.9	80
J-8	29.7	0	83.9	77
J-9	22.6	0	83.9	87
J-10	21.1	0	83.9	89
J-11	17.3	426	83.9	95
J-12	51.3	19892	83.9	46
J-13	41.1	0	83.9	61
J-14	49.9	0	84.0	48
J-15	50.5	0	84.0	48
J-16	50.6	0	84.0	47
J-17	51.1	1694	84.0	47
J-18	57.0	5	83.9	38
J-19	63.4	2	83.9	29
J-20	21.9	276	83.9	88
J-21	39.6	0	83.9	63
J-22	42.6	0	83.9	59
J-23	39.2	0	83.9	64
J-24	38.9	0	83.9	64
J-25	42.0	0	83.9	60
J-26	40.0	0	83.9	62
J-27	39.6	0	83.9	63
J-28	35.8	0	83.9	68
J-29	40.7	0	83.9	61
J-30	42.4	0	83.9	59
J-31	45.2	0	83.9	55
J-32	45.7	57	83.9	54
J-33	50.2	95	83.9	48
J-34	46.3	0	83.9	54
J-35	45.9	0	83.9	54
J-36	54.7	122	83.9	42
J-37	54.6	353	83.9	42
J-38	52.1	274	83.9	45

Water Model Results

Average Daily Demand Scenario

Including Future Development - Average Daily Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
J-39	49.3	49	83.9	49
J-40	48.1	14	83.9	51
J-41	55.3	239	83.9	41
J-42	19.0	32	83.9	92
J-43	30.6	0	83.9	76
J-44	31.5	3359	83.9	75
J-45	49.6	294	83.9	49
J-46	46.3	0	83.9	53
J-47	46.0	293	83.9	54
J-48	46.3	0	83.9	53
J-49	47.8	36	83.9	51
J-50	48.0	19	83.9	51
J-51	40.7	3	83.9	61
J-52	23.1	0	83.9	86
J-53	19.2	0	83.9	92
J-54	16.5	0	83.9	96
J-55	52.7	97	83.9	44
J-56	52.4	21	83.9	45
J-57	51.6	232	83.9	46
J-58	49.7	0	83.9	49
J-59	48.2	0	83.9	51
J-60	32.3	34	83.9	73
J-61	52.1	199	83.9	45
J-62	53.6	163	83.9	43
J-63	54.6	201	83.9	42
J-64	55.1	52	83.9	41
J-65	35.6	84	83.9	69
J-66	21.4	374	83.9	89
J-67	30.4	1272	83.9	76
J-68	22.7	148	83.9	87
J-69	42.9	48	83.9	58
J-70	41.0	227	83.9	61
J-71	33.6	3	83.9	71
J-72	63.0	1258	83.9	30
J-73	63.7	3	83.9	29
J-74	61.2	2	83.9	32
J-75	61.4	1	83.9	32
J-76	57.4	45	83.9	38
J-77	39.2	5	83.9	64
J-78	35.7	568	83.9	68
J-79	35.2	2	83.9	69
J-80	40.6	17	83.9	62

Including Future Development - Average Daily Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
J-81	51.7	3	84.0	46
J-82	49.6	0	84.0	49
J-83	51.8	291	84.0	46
J-84	53.5	55	84.0	43
J-85	52.6	622	83.9	44
J-86	50.6	257	83.9	47
J-87	52.4	387	83.9	45
J-88	53.8	500	83.9	43
J-89	50.5	150	83.9	47
J-90	50.3	337	83.9	48
J-91	55.8	401	83.9	40
J-92	38.5	1848	83.9	65
J-93	25.1	0	83.9	84
J-94	47.4	0	83.9	52
J-95	53.5	223	83.9	43
J-96	27.5	0	83.9	80
J-97	31.2	0	83.9	75
J-98	39.8	0	83.9	63
J-99	55.9	0	84.0	40
J-100	56.8	0	84.0	39
J-101	56.1	169	83.9	40
J-102	36.1	3192	83.9	68
J-103	63.9	0	83.9	29
J-104	65.8	0	83.9	26
J-105	46.5	4	83.9	53
J-106	47.0	9	83.9	52
J-107	60.2	14	83.9	34
Residual 1	51.8	0	84.0	46
Residual 2	40.4	0	83.9	62
Residual 3	43.0	0	83.9	58
Residual 4	31.2	0	83.9	75

Existing Conditions - Peak Hourly Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
Flow 1	48.0	706	81.2	47
Flow 2	40.1	0	81.2	58
Flow 3	44.3	0	81.2	52
Flow 4	33.2	0	81.1	68
J-1	56.6	0	81.2	35
J-2	55.6	0	81.2	36
J-3	51.8	0	81.2	42
J-4	48.3	11	81.2	47
J-5	30.9	0	81.2	71
J-6	31.8	271	81.2	70
J-7	27.6	0	81.1	76
J-8	29.7	0	81.1	73
J-9	22.6	0	81.1	83
J-10	21.1	0	81.1	85
J-11	17.3	1304	81.1	91
J-12	51.3	0	81.2	43
J-13	41.1	0	81.2	57
J-14	49.9	0	81.2	45
J-15	50.5	0	81.2	44
J-16	50.6	0	81.2	44
J-17	51.1	5185	81.2	43
J-18	57.0	16	81.2	34
J-19	63.4	6	81.2	25
J-20	21.9	845	81.1	84
J-21	39.6	0	81.2	59
J-22	42.6	0	81.2	55
J-23	39.2	0	81.2	60
J-24	38.9	0	81.2	60
J-25	42.0	0	81.2	56
J-26	40.0	0	81.2	59
J-27	39.6	0	81.2	59
J-28	35.8	0	81.2	65
J-29	40.7	0	81.2	58
J-30	42.4	0	81.2	55
J-31	45.2	0	81.2	51
J-32	45.7	57	81.2	50
J-33	50.2	292	81.2	44
J-34	46.3	0	81.2	50
J-35	45.9	0	81.2	50
J-36	54.7	374	81.1	38
J-37	54.6	1080	81.1	38
J-38	52.1	840	81.2	41

Existing Conditions - Peak Hourly Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
J-39	49.3	151	81.2	45
J-40	48.1	42	81.2	47
J-41	55.3	731	81.1	37
J-42	19.0	97	81.1	88
J-43	30.6	0	81.1	72
J-44	31.5	10281	81.1	71
J-45	49.6	900	81.2	45
J-46	46.3	0	81.2	50
J-47	46.0	896	81.2	50
J-48	46.3	1	81.2	50
J-49	47.8	109	81.2	47
J-50	48.0	57	81.2	47
J-51	40.7	9	81.2	58
J-52	23.1	0	81.1	82
J-53	19.2	0	81.1	88
J-54	16.5	0	81.1	92
J-55	52.7	297	81.2	40
J-56	52.4	64	81.2	41
J-57	51.6	712	81.2	42
J-58	49.7	1	81.2	45
J-59	48.2	0	81.2	47
J-60	32.3	105	81.1	69
J-61	52.1	610	81.2	41
J-62	53.6	498	81.2	39
J-63	54.6	615	81.2	38
J-64	55.1	160	81.2	37
J-65	35.6	257	81.2	65
J-66	21.4	1145	81.1	85
J-67	30.4	3894	81.1	72
J-68	22.7	452	81.1	83
J-69	42.9	147	81.2	54
J-70	41.0	695	81.2	57
J-71	33.6	10	81.2	68
J-72	63.0	1276	81.2	26
J-73	63.7	9	81.2	25
J-74	61.2	5	81.2	29
J-75	61.4	3	81.2	28
J-76	57.4	137	81.2	34
J-77	39.2	16	81.2	60
J-78	35.7	568	81.2	65
J-79	35.2	7	81.2	65
J-80	40.6	53	81.2	58

Existing Conditions - Peak Hourly Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
J-81	51.7	9	81.2	42
J-82	49.6	0	81.2	45
J-83	51.8	890	81.2	42
J-84	53.5	169	81.2	39
J-85	52.6	1904	81.2	41
J-86	50.6	787	81.2	43
J-87	52.4	1186	81.2	41
J-88	53.8	1530	81.1	39
J-89	50.5	461	81.2	44
J-90	50.3	1032	81.2	44
J-91	55.8	1227	81.1	36
J-92	38.5	5656	81.2	61
J-93	25.1	0	81.1	80
J-94	47.4	0	81.2	48
J-95	53.5	683	81.2	39
J-96	27.5	0	81.1	76
J-97	31.2	0	81.1	71
J-98	39.8	0	81.2	59
J-99	55.9	0	81.2	36
J-100	56.8	0	81.2	35
J-101	56.1	516	81.1	36
J-102	36.1	0	81.2	64
J-103	63.9	0	81.2	25
J-104	65.8	0	81.2	22
J-105	46.5	11	81.2	49
J-106	47.0	28	81.2	49
J-107	60.2	42	81.2	30
Residual 1	51.8	0	81.2	42
Residual 2	40.4	0	81.2	58
Residual 3	43.0	0	81.2	54
Residual 4	31.2	0	81.1	71

Including Future Development - Peak Hourly Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
Flow 1	48.0	706	80.9	47
Flow 2	40.1	0	80.7	58
Flow 3	44.3	0	80.8	52
Flow 4	33.2	0	80.8	68
J-1	56.6	0	80.9	35
J-2	55.6	0	80.9	36
J-3	51.8	0	80.9	41
J-4	48.3	11	80.8	46
J-5	30.9	0	80.8	71
J-6	31.8	271	80.8	70
J-7	27.6	0	80.8	76
J-8	29.7	0	80.8	73
J-9	22.6	0	80.8	83
J-10	21.1	0	80.8	85
J-11	17.3	1304	80.8	90
J-12	51.3	74590	80.7	42
J-13	41.1	0	80.7	56
J-14	49.9	0	80.9	44
J-15	50.5	0	80.9	43
J-16	50.6	0	80.9	43
J-17	51.1	5185	80.9	42
J-18	57.0	16	80.9	34
J-19	63.4	6	80.9	25
J-20	21.9	845	80.8	84
J-21	39.6	0	80.8	59
J-22	42.6	0	80.8	54
J-23	39.2	0	80.8	59
J-24	38.9	0	80.8	60
J-25	42.0	0	80.7	55
J-26	40.0	0	80.7	58
J-27	39.6	0	80.7	58
J-28	35.8	0	80.7	64
J-29	40.7	0	80.7	57
J-30	42.4	0	80.8	55
J-31	45.2	0	80.8	50
J-32	45.7	57	80.8	50
J-33	50.2	292	80.8	43
J-34	46.3	0	80.8	49
J-35	45.9	0	80.8	50
J-36	54.7	374	80.8	37
J-37	54.6	1080	80.8	37
J-38	52.1	840	80.8	41

Including Future Development - Peak Hourly Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
J-39	49.3	151	80.8	45
J-40	48.1	42	80.8	47
J-41	55.3	731	80.8	36
J-42	19.0	97	80.8	88
J-43	30.6	0	80.8	71
J-44	31.5	10281	80.8	70
J-45	49.6	900	80.8	44
J-46	46.3	0	80.8	49
J-47	46.0	896	80.8	50
J-48	46.3	1	80.8	49
J-49	47.8	109	80.8	47
J-50	48.0	57	80.8	47
J-51	40.7	9	80.7	57
J-52	23.1	0	80.8	82
J-53	19.2	0	80.8	87
J-54	16.5	0	80.8	91
J-55	52.7	297	80.8	40
J-56	52.4	64	80.8	40
J-57	51.6	712	80.8	41
J-58	49.7	1	80.8	44
J-59	48.2	0	80.8	46
J-60	32.3	105	80.8	69
J-61	52.1	610	80.9	41
J-62	53.6	498	80.8	39
J-63	54.6	615	80.8	37
J-64	55.1	160	80.8	37
J-65	35.6	257	80.7	64
J-66	21.4	1145	80.8	84
J-67	30.4	3894	80.8	72
J-68	22.7	452	80.8	83
J-69	42.9	147	80.8	54
J-70	41.0	695	80.8	57
J-71	33.6	10	80.8	67
J-72	63.0	1276	80.9	25
J-73	63.7	9	80.9	24
J-74	61.2	5	80.9	28
J-75	61.4	3	80.9	28
J-76	57.4	137	80.9	33
J-77	39.2	16	80.8	59
J-78	35.7	568	80.7	64
J-79	35.2	7	80.7	65
J-80	40.6	53	80.7	57

Including Future Development - Peak Hourly Demand

Junction	Elevation (m)	Demand (L/h)	Hydraulic Grade (m)	Pressure (psi)
J-81	51.7	9	80.9	41
J-82	49.6	0	80.9	44
J-83	51.8	890	80.9	41
J-84	53.5	169	80.9	39
J-85	52.6	1904	80.8	40
J-86	50.6	787	80.8	43
J-87	52.4	1186	80.8	40
J-88	53.8	1530	80.8	38
J-89	50.5	461	80.8	43
J-90	50.3	1032	80.8	43
J-91	55.8	1227	80.8	35
J-92	38.5	5656	80.8	60
J-93	25.1	0	80.8	79
J-94	47.4	0	80.8	48
J-95	53.5	683	80.8	39
J-96	27.5	0	80.8	76
J-97	31.2	0	80.8	71
J-98	39.8	0	80.8	58
J-99	55.9	0	80.9	36
J-100	56.8	0	80.9	34
J-101	56.1	516	80.8	35
J-102	36.1	3830	80.8	64
J-103	63.9	0	80.9	24
J-104	65.8	0	80.9	21
J-105	46.5	11	80.8	49
J-106	47.0	28	80.8	48
J-107	60.2	42	80.9	29
Residual 1	51.8	0	80.9	41
Residual 2	40.4	0	80.7	57
Residual 3	43.0	0	80.8	54
Residual 4	31.2	0	80.8	70

APPENDIX D: WATER MODEL RESULTS (MAXIMUM DAILY DEMAND + FIRE FLOW)

Fire Flow Results

Maximum Daily Demand Scenario

Existing Conditions - Maximum Daily Demand + Fire Flow

Junction	Fire Flow (Available) (L/min)	Pressure (Calculated Residual) (psi)	Velocity of Maximum Pipe (m/s)	Pipe w/ Maximum Velocity	Pressure (Calculated Zone Lower Limit) (psi)	Junction w/ Minimum Pressure (Zone)
Flow 1	8830	61	3.0	P-33	60	Residual 1
Flow 2	10524	29	3.0	P-28	29	J-51
Flow 3	12001	22	1.8	P-52	27	Residual 3
Flow 4	12160	22	2.1	P-15	26	J-67
J-1	13620	35	2.9	P-2	33	J-104
J-2	13620	36	1.8	P-29	29	J-104
J-3	13620	35	1.8	P-4	29	J-101
J-4	13620	37	1.8	P-4	26	J-101
J-5	13620	45	1.8	P-7	25	J-101
J-6	13620	28	2.1	P-8	20	Flow 3
J-7	13396	28	2.0	P-8	20	Flow 4
J-8	12701	25	1.9	P-8	20	Flow 4
J-9	12255	25	2.1	P-16	20	Flow 4
J-10	8812	79	3.0	P-20	68	J-104
J-11	8812	76	3.0	P-20	68	J-104
J-12	13620	23	3.0	P-25	32	J-100
J-13	12203	22	2.5	P-25	21	J-25
J-14	13620	45	0.1	P-2	22	J-104
J-15	13320	42	1.8	P-30	20	J-104
J-16	13320	42	1.8	P-30	20	J-104
J-17	5590	95	3.0	P-36	74	J-104
J-18	2525	33	2.4	P-37	20	J-104
J-19	2194	22	2.1	P-37	23	J-103
J-20	12255	24	2.1	P-16	20	Flow 4
J-21	12444	22	1.9	P-47	20	Residual 3
J-22	12259	22	1.9	P-51	22	Flow 3
J-23	12673	27	2.0	P-52	20	Flow 3
J-24	12645	25	1.8	P-52	20	Flow 3
J-25	11152	22	2.6	P-60	24	J-80
J-26	6884	80	3.0	P-63	78	J-104
J-27	2664	22	2.5	P-64	27	J-28
J-28	2113	22	2.0	P-65	22	J-78
J-29	4452	89	3.0	P-66	85	J-104
J-30	4806	73	3.0	P-67	77	J-33
J-31	5946	25	3.0	P-68	27	J-33
J-32	5968	23	2.9	P-103	20	J-33
J-33	4819	22	2.8	P-70	58	J-32
J-34	5625	35	3.0	P-103	36	J-35
J-35	3181	76	3.0	P-73	86	J-104
J-36	5486	45	3.0	P-76	44	J-91
J-37	5398	47	3.0	P-76	46	J-101
J-38	7769	33	3.0	P-87	38	J-55

Fire Flow Results
Maximum Daily Demand Scenario

Existing Conditions - Maximum Daily Demand + Fire Flow

Junction	Fire Flow (Available) (L/min)	Pressure (Calculated Residual) (psi)	Velocity of Maximum Pipe (m/s)	Pipe w/ Maximum Velocity	Pressure (Calculated Zone Lower Limit) (psi)	Junction w/ Minimum Pressure (Zone)
J-39	7267	72	3.0	P-97	65	J-101
J-40	12102	44	3.0	P-106	33	J-101
J-41	5425	43	3.0	P-76	43	J-101
J-42	5654	110	3.0	P-92	82	J-104
J-43	12700	23	1.9	P-8	20	J-60
J-44	5572	115	3.0	P-94	82	J-104
J-45	5648	46	3.0	P-95	82	J-104
J-46	11660	39	3.0	P-96	32	J-101
J-47	12057	37	3.0	P-106	25	J-101
J-48	5351	92	3.0	P-104	83	J-104
J-49	11750	50	3.0	P-106	38	J-101
J-50	13620	37	1.8	P-4	25	J-101
J-51	10343	22	2.9	P-28	23	J-26
J-52	12255	25	2.1	P-16	20	Flow 4
J-53	8812	79	3.0	P-20	68	J-104
J-54	5655	97	3.0	P-109	82	J-104
J-55	7047	41	3.0	P-77	44	J-95
J-56	5744	59	3.0	P-111	67	J-55
J-57	7185	44	3.0	P-87	46	J-56
J-58	4507	91	3.0	P-87	84	J-104
J-59	3181	90	3.0	P-112	86	J-104
J-60	5572	108	3.0	P-94	82	J-104
J-61	3150	96	3.0	P-114	86	J-104
J-62	3150	65	3.0	P-114	63	J-64
J-63	3150	51	3.0	P-114	50	J-64
J-64	3150	30	3.0	P-114	51	J-63
J-65	1976	22	1.9	P-65	29	J-78
J-66	8812	81	3.0	P-20	68	J-104
J-67	12247	22	2.1	P-16	20	Flow 4
J-68	8812	83	3.0	P-20	68	J-104
J-69	11580	22	2.7	P-50	34	Flow 3
J-70	11869	22	2.8	P-49	31	Flow 3
J-71	13620	30	2.3	P-8	22	Flow 3
J-72	2205	22	2.1	P-37	23	J-103
J-73	2226	22	2.1	P-37	21	J-104
J-74	2329	25	2.2	P-37	20	J-104
J-75	2461	22	2.3	P-37	21	J-104
J-76	2482	31	2.4	P-37	20	J-104
J-77	12573	24	2.2	P-47	20	Flow 3
J-78	2048	22	1.9	P-65	22	J-65
J-79	2022	22	1.9	P-65	25	J-78
J-80	3180	101	3.0	P-130	86	J-104

Fire Flow Results

Maximum Daily Demand Scenario

Existing Conditions - Maximum Daily Demand + Fire Flow

Junction	Fire Flow (Available) (L/min)	Pressure (Calculated Residual) (psi)	Velocity of Maximum Pipe (m/s)	Pipe w/ Maximum Velocity	Pressure (Calculated Zone Lower Limit) (psi)	Junction w/ Minimum Pressure (Zone)
J-81	8830	63	3.0	P-33	63	Residual 1
J-82	13620	45	1.8	P-30	22	J-104
J-83	3174	94	3.0	P-131	86	J-104
J-84	3174	78	3.0	P-131	86	J-104
J-85	3150	82	3.0	P-114	79	J-64
J-86	4161	86	3.0	P-80	85	J-104
J-87	5138	58	3.0	P-80	66	J-88
J-88	6240	27	3.0	P-76	38	J-91
J-89	5663	70	3.0	P-87	77	J-101
J-90	6174	58	3.0	P-78	68	J-101
J-91	5452	42	3.0	P-76	43	J-101
J-92	12705	23	1.7	P-29	20	Flow 3
J-93	12255	24	2.1	P-16	20	Flow 4
J-94	4724	102	3.0	P-99	84	J-104
J-95	4781	66	3.0	P-76	67	J-101
J-96	12767	28	2.0	P-8	20	Flow 4
J-97	12920	23	2.0	P-8	20	Flow 4
J-98	12656	22	1.9	P-52	20	Flow 3
J-99	13620	35	1.8	P-3	29	J-104
J-100	13262	32	3.0	P-25	33	J-104
J-101	5430	41	3.0	P-76	44	J-91
J-102	12820	24	1.8	P-29	20	Flow 3
J-103	2180	22	2.1	P-37	25	J-19
J-104	2188	22	2.1	P-37	26	J-73
J-105	5264	49	3.0	P-103	55	J-34
J-106	4133	92	3.0	P-103	85	J-104
J-107	2359	22	2.3	P-37	22	J-103
Residual 1	8830	60	3.0	P-33	63	J-81
Residual 2	10480	22	3.0	P-28	22	J-51
Residual 3	12087	22	1.8	P-52	23	Flow 3
Residual 4	12480	23	2.2	P-14	20	Flow 4

Fire Flow Results

Maximum Daily Demand Scenario

Including Future Development - Maximum Daily Demand + Fire Flow

Junction	Fire Flow (Available) (L/min)	Pressure (Calculated Residual) (psi)	Velocity of Maximum Pipe (m/s)	Pipe w/ Maximum Velocity	Pressure (Calculated Zone Lower Limit) (psi)	Junction w/ Minimum Pressure (Zone)
Flow 1	8830	54	3.0	P-33	54	Residual 1
Flow 2	10320	22	2.9	P-28	22	J-51
Flow 3	11390	22	1.8	P-52	26	Residual 3
Flow 4	11600	22	2.0	P-15	26	J-67
J-1	13620	35	2.9	P-2	33	J-104
J-2	13620	36	1.8	P-29	29	J-104
J-3	13620	35	1.8	P-4	28	J-101
J-4	13620	36	1.8	P-4	25	J-101
J-5	13620	44	1.8	P-7	25	J-101
J-6	13536	28	2.1	P-8	20	Flow 3
J-7	13323	28	2.0	P-8	20	Flow 4
J-8	12340	25	1.9	P-8	20	Flow 4
J-9	11694	26	2.0	P-16	20	Flow 4
J-10	8812	72	3.0	P-20	62	J-104
J-11	8812	69	3.0	P-20	62	J-104
J-12	12932	22	3.0	P-25	32	J-100
J-13	11494	22	2.6	P-2	21	J-25
J-14	13620	45	0.2	P-25	22	J-104
J-15	12455	42	1.7	P-30	20	J-104
J-16	12455	42	1.7	P-30	20	J-104
J-17	5590	93	3.0	P-36	72	J-104
J-18	2510	33	2.4	P-37	20	J-104
J-19	2183	22	2.1	P-37	23	J-103
J-20	11694	25	2.0	P-16	20	Flow 4
J-21	11799	23	1.8	P-47	20	Flow 3
J-22	11635	22	1.8	P-51	21	Flow 3
J-23	12085	27	1.9	P-52	20	Flow 3
J-24	11991	25	1.8	P-29	20	Flow 3
J-25	10552	22	2.5	P-60	24	J-80
J-26	6918	74	3.0	P-63	74	J-104
J-27	2637	22	2.5	P-64	28	J-28
J-28	2096	22	2.0	P-65	22	J-78
J-29	4476	86	3.0	P-66	83	J-104
J-30	4827	69	3.0	P-67	74	J-33
J-31	5901	22	3.0	P-68	24	J-33
J-32	5830	23	2.8	P-103	20	J-33
J-33	4732	22	2.8	P-70	57	J-32
J-34	5609	32	3.0	P-103	32	J-35
J-35	3181	75	3.0	P-73	85	J-104
J-36	5486	42	3.0	P-76	40	J-91
J-37	5398	44	3.0	P-76	43	J-101

Fire Flow Results

Maximum Daily Demand Scenario

Including Future Development - Maximum Daily Demand + Fire Flow

Junction	Fire Flow (Available) (L/min)	Pressure (Calculated Residual) (psi)	Velocity of Maximum Pipe (m/s)	Pipe w/ Maximum Velocity	Pressure (Calculated Zone Lower Limit) (psi)	Junction w/ Minimum Pressure (Zone)
J-38	7768	27	3.0	P-87	32	J-55
J-39	7267	67	3.0	P-97	60	J-101
J-40	12093	33	3.0	P-106	23	J-101
J-41	5425	40	3.0	P-76	40	J-101
J-42	5654	106	3.0	P-92	79	J-104
J-43	12343	23	1.9	P-8	20	Flow 4
J-44	5572	112	3.0	P-94	79	J-104
J-45	5648	43	3.0	P-95	79	J-104
J-46	11652	28	3.0	P-96	21	J-101
J-47	11660	32	2.9	P-106	20	J-101
J-48	5353	89	3.0	P-104	80	J-104
J-49	11741	39	3.0	P-106	27	J-101
J-50	13620	37	1.8	P-4	25	J-101
J-51	9848	22	2.8	P-28	23	J-26
J-52	11694	26	2.0	P-16	20	Flow 4
J-53	8812	72	3.0	P-20	62	J-104
J-54	5655	94	3.0	P-109	79	J-104
J-55	7047	37	3.0	P-77	40	J-95
J-56	5744	55	3.0	P-111	64	J-55
J-57	7184	39	3.0	P-87	41	J-56
J-58	4507	89	3.0	P-87	83	J-104
J-59	3181	89	3.0	P-112	85	J-104
J-60	5572	104	3.0	P-94	79	J-104
J-61	3150	95	3.0	P-114	85	J-104
J-62	3150	64	3.0	P-114	62	J-64
J-63	3150	49	3.0	P-114	49	J-64
J-64	3150	29	3.0	P-114	49	J-63
J-65	1961	22	1.9	P-65	29	J-78
J-66	8812	74	3.0	P-20	62	J-104
J-67	11694	22	2.0	P-16	20	Flow 4
J-68	8812	76	3.0	P-20	62	J-104
J-69	11023	22	2.6	P-50	33	Flow 3
J-70	11293	22	2.7	P-49	30	Flow 3
J-71	13620	29	2.3	P-8	22	Flow 3
J-72	2194	22	2.1	P-37	22	J-103
J-73	2215	22	2.1	P-37	21	J-104
J-74	2316	25	2.2	P-37	20	J-104
J-75	2448	22	2.3	P-37	21	J-104
J-76	2467	31	2.4	P-37	20	J-104
J-77	11912	24	2.0	P-47	20	Flow 3
J-78	2033	22	1.9	P-65	22	J-65

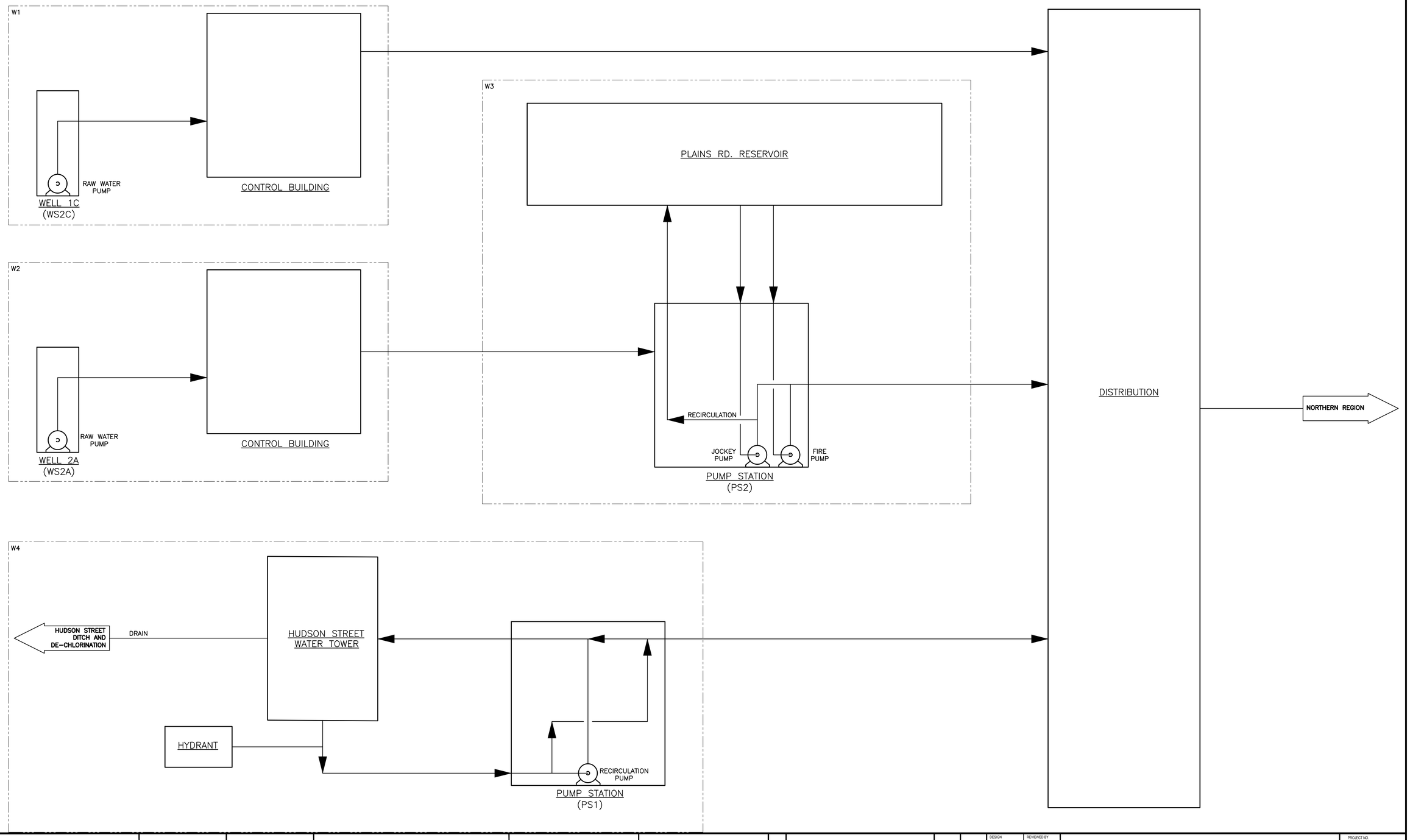
Fire Flow Results

Maximum Daily Demand Scenario

Including Future Development - Maximum Daily Demand + Fire Flow

Junction	Fire Flow (Available) (L/min)	Pressure (Calculated Residual) (psi)	Velocity of Maximum Pipe (m/s)	Pipe w/ Maximum Velocity	Pressure (Calculated Zone Lower Limit) (psi)	Junction w/ Minimum Pressure (Zone)
J-79	2010	22	1.9	P-65	24	J-78
J-80	3180	98	3.0	P-130	85	J-104
J-81	8830	57	3.0	P-33	57	Residual 1
J-82	13620	45	1.8	P-30	22	J-104
J-83	3174	93	3.0	P-131	85	J-104
J-84	3174	77	3.0	P-131	85	J-104
J-85	3150	81	3.0	P-114	77	J-64
J-86	4161	84	3.0	P-80	83	J-101
J-87	5138	55	3.0	P-80	63	J-88
J-88	6240	23	3.0	P-76	34	J-91
J-89	5663	66	3.0	P-87	74	J-101
J-90	6174	54	3.0	P-78	64	J-101
J-91	5452	38	3.0	P-76	40	J-101
J-92	12195	24	1.8	P-29	20	Flow 3
J-93	11694	25	2.0	P-16	20	Flow 4
J-94	4711	100	3.0	P-99	82	J-104
J-95	4781	64	3.0	P-76	64	J-101
J-96	12527	28	1.9	P-8	20	Flow 4
J-97	12862	23	2.0	P-8	20	Flow 4
J-98	12026	23	1.8	P-52	20	Flow 3
J-99	13620	35	1.8	P-3	29	J-104
J-100	12433	32	3.0	P-25	33	J-104
J-101	5430	38	3.0	P-76	41	J-91
J-102	12614	24	1.8	P-29	20	Flow 3
J-103	2168	22	2.1	P-37	25	J-19
J-104	2177	22	2.1	P-37	26	J-73
J-105	5250	46	3.0	P-103	52	J-34
J-106	4122	90	3.0	P-103	83	J-104
J-107	2347	22	2.2	P-37	22	J-103
Residual 1	8830	54	3.0	P-33	57	J-81
Residual 2	9972	22	2.8	P-28	22	J-51
Residual 3	11477	22	1.8	P-52	23	Flow 3
Residual 4	11897	23	2.1	P-14	20	Flow 4

APPENDIX E: WATER TREATMENT SYSTEM PROCESS FLOW DIAGRAMS



LONDON

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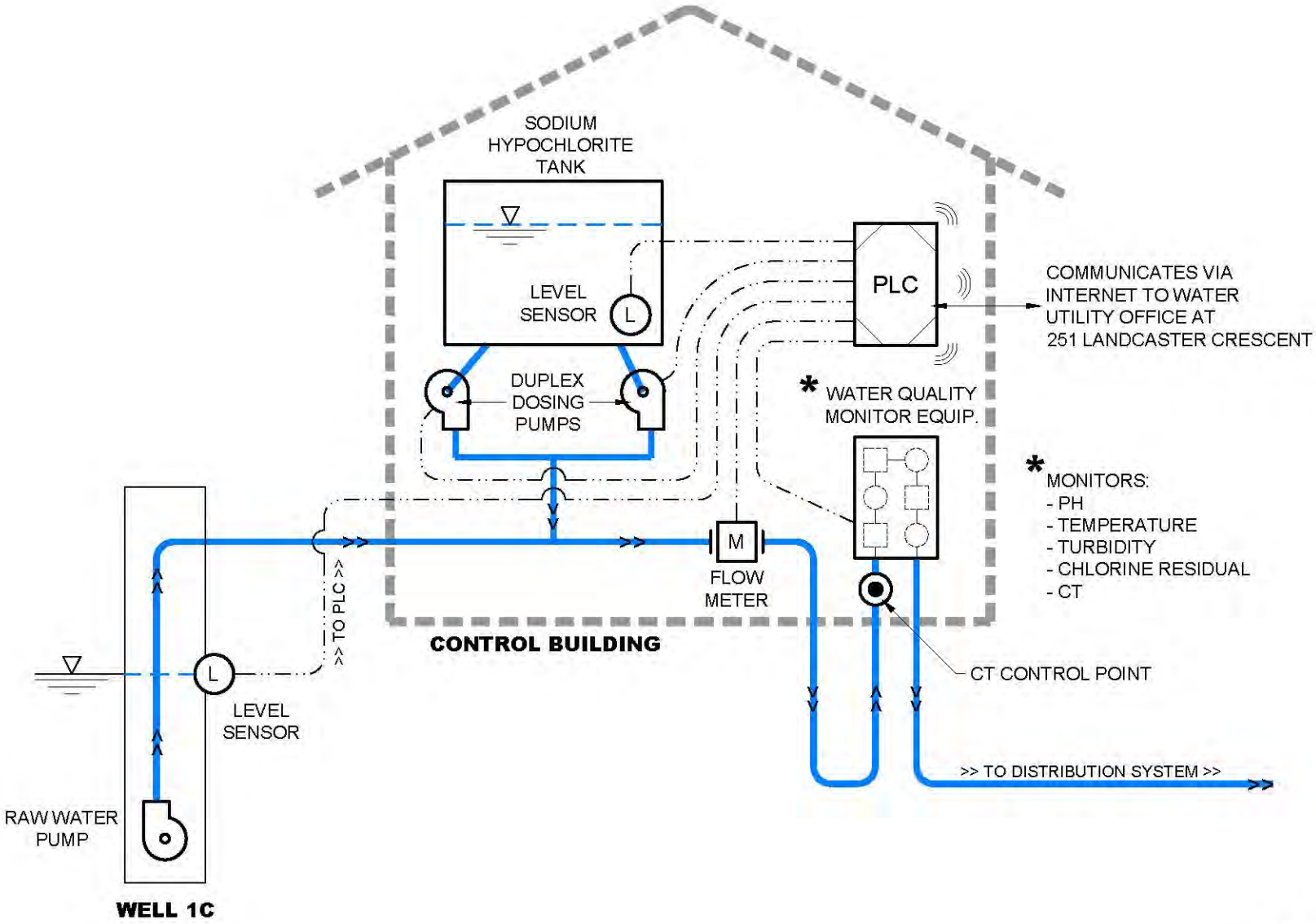
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DEBERT WATER CAPACITY ASSESMENT

WATER TREATMENT SYSTEM PROCESS FLOW DIAGRAM

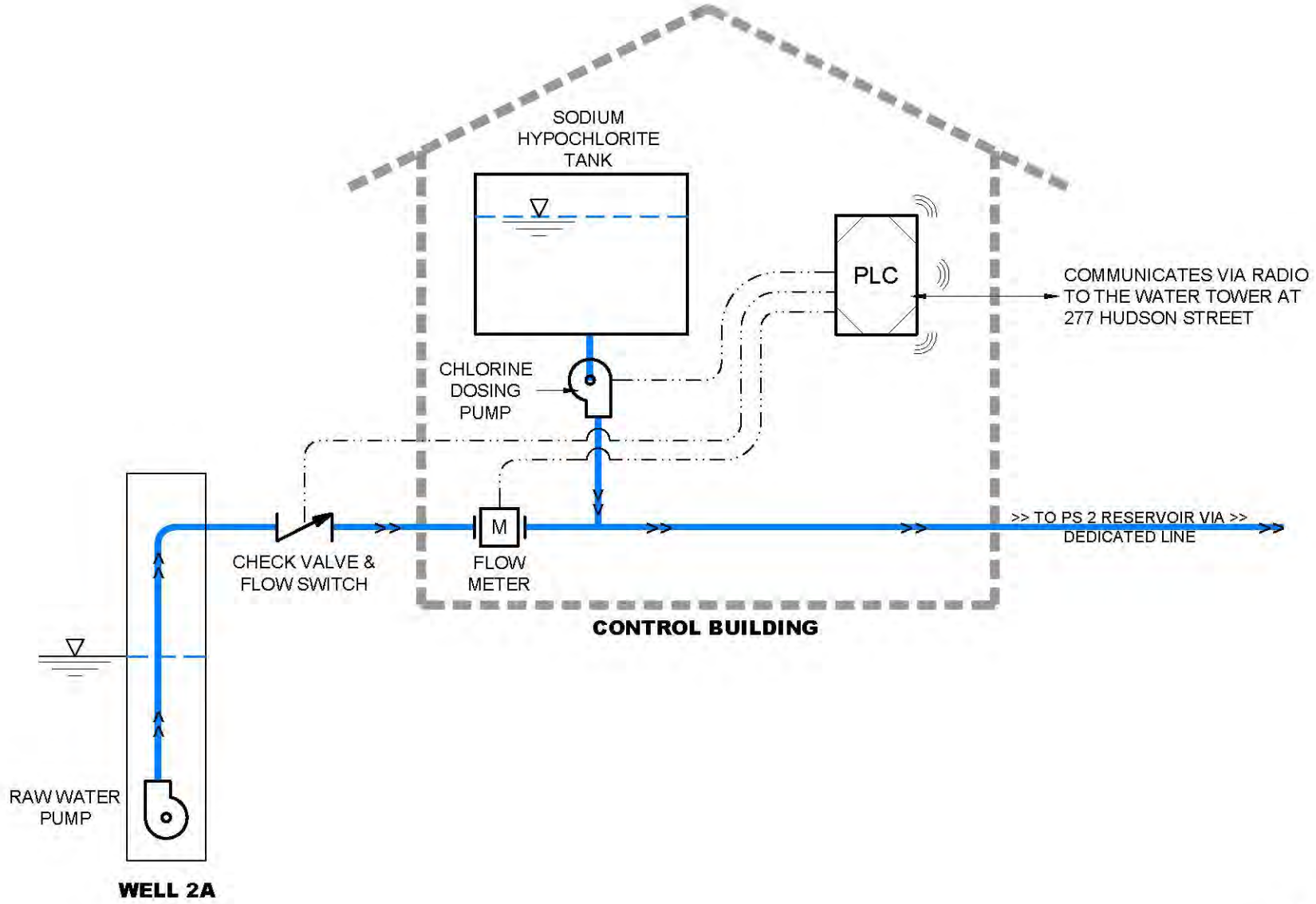
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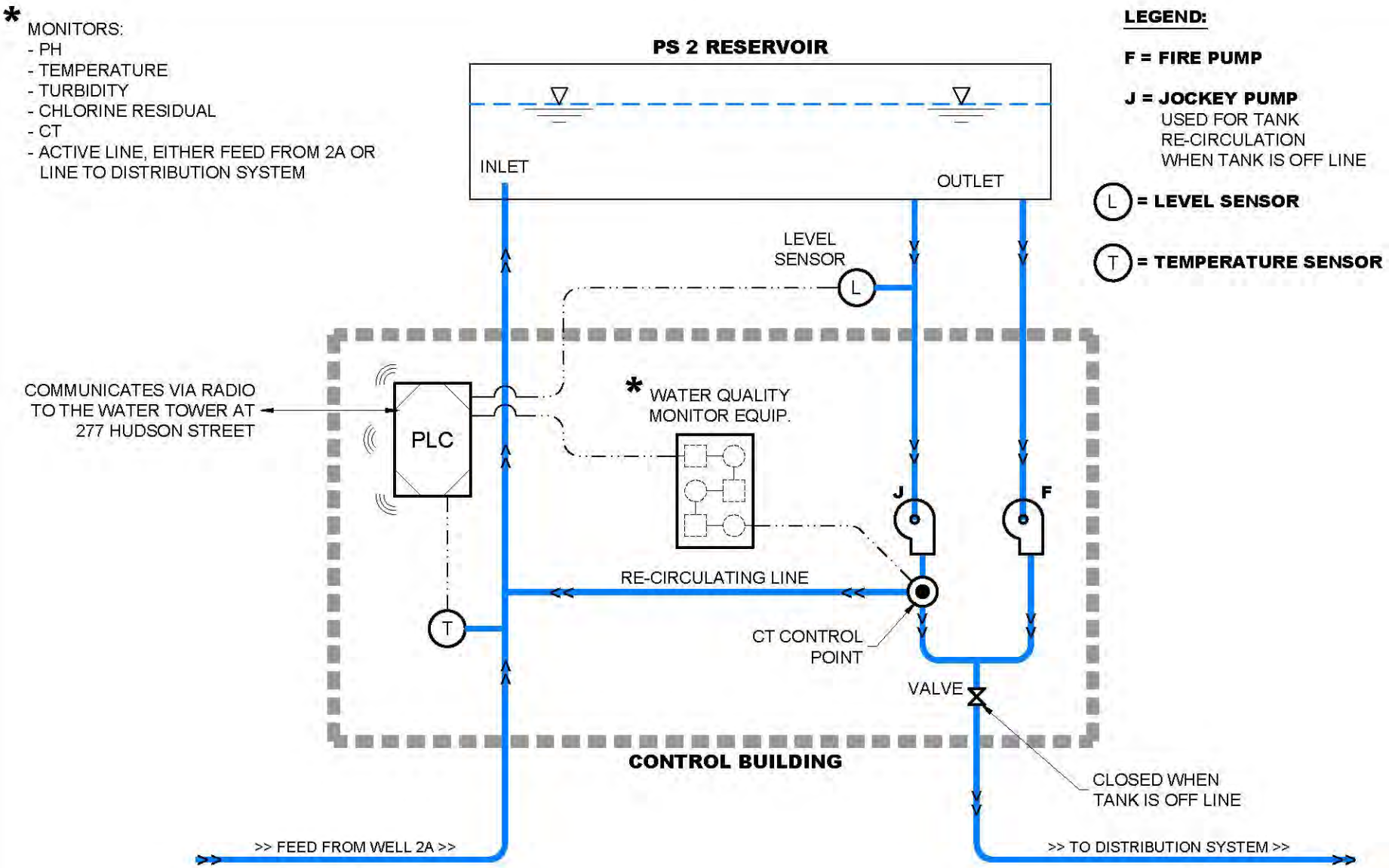
December 17, 2019

Colchester Public Works	Project: **DEBERT WATER UTILITY**	Drawing Title: **WELL 1C PROCESS SCHEMATIC**	Fig. No.: **W1**



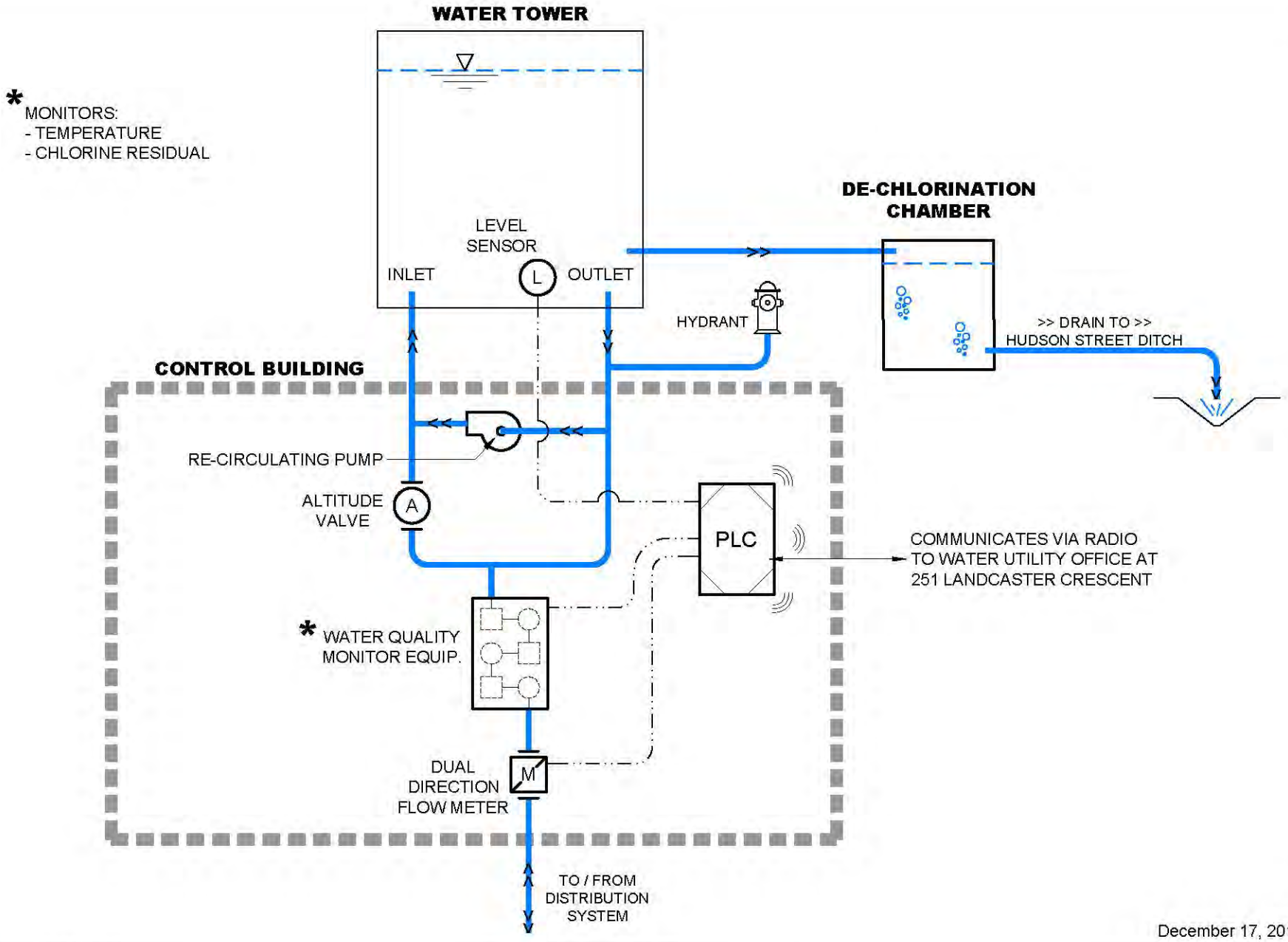
December 17, 2019

Colchester Public Works	Project: **DEBERT WATER UTILITY**	Drawing Title: **WELL 2A PROCESS SCHEMATIC**	Fig. No.: **W2**



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Colchester Public Works	Project: **DEBERT WATER UTILITY**	Drawing Title: **PS 2 PROCESS SCHEMATIC**	Fig. No.: **W3**



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Colchester Public Works	Project: **DEBERT WATER UTILITY**	Drawing Title: **HUDSON STREET WATER TOWER PROCESS SCHEMATIC**	Fig. No.: **W4**