FINAL

CLASS ENVIRONMENTAL ASSESSMENT FOR WATER AND WASTEWATER SERVICING IN THE COMMUNITY OF NOBLETON

WATER SYSTEM CAPACITY OPTIMIZATION STUDY

Study 1A

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

4 JUNE 2019

BLACK & VEATCH

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1 Introduction

1.1 PURPOSE OF STUDY

The purpose of the Water System Capacity and Optimization Study is to:

- Provide a description of the existing water system.
- Analyze historical water use to confirm the baseline water demand.
- Identify the design criteria for the water system analysis.
- Identify any "infra-stretching" opportunities that will allow the system to increase its maximum capacity in consideration of the "Existing Water System Hydraulic Analysis" results.
- Conduct a brief feasibility analysis of each identified opportunity.

1.2 BACKGROUND

Nobleton is a community in King Township, located in York Region. Currently, Nobleton is serviced by standalone water and wastewater systems to meet the needs of the current population. The York Region Water and Wastewater Master Plan (2016) indicated that both the water and wastewater systems would not have sufficient capacity to meet requirements to support growth to the 2041 Master Plan horizon. Therefore, the Master Plan recommended undertaking the current project, a Schedule C Class Environmental Assessment (EA), to identify preferred servicing solutions to accommodate growth.

1.3 PREVIOUS STUDIES AND PLANNING DOCUMENTS

1.3.1 Regional Official Plan

York Region continues to experience rapid population and employment growth. In accordance with the York Region Official Plan 2010 (OP) significant population growth is expected within the next 25 years, to the planning horizon of 2031. With a population of 1,156,000 residents as of mid-2015, it is anticipated that the Region will reach a population of 1.5 million people by 2031.

The York Region Official Plan has forecasted a population growth within the Township of King from 20,300 people in 2006 to 34,900 people in 2031. This represents an increase of 14,600 people. Employment is expected to increase from 7,100 in 2006 to 11,900 in 2031, for an increase of 4,800. The York Region Official Plan does not specify population distribution within King Township.

1.3.2 King Township Rural Official Plan

The current King Township Official Plan was approved in 1970 and is colloquially known as the "Parent Official Plan". This document establishes land use, transportation, and development policies for King Township.

In the 1990s, community plans were prepared for each of the villages in King Township (Nobleton, Schomberg, and King City) as well as for the hamlets. Specifically, the Nobleton Community Plan was added to the King Township Official Plan through Official Plan Amendment (OPA) 57, adopted by Council in the 1997, with latest Office Consolidation in 2005.

Presently, King Township is working toward preparing an update to the Official Plan, published in draft form in November 2017 and expected to be finalized in 2018/2019.

The King Township Official Plan shows population growth forecasts for the Village of Nobleton to increase from 5,600 in 2016 to 7,000 in 2031. However, the Official Plan notes that:

> *"[the current population forecast] reflects limitations posed by the municipal sanitary sewer services".*
>
> *"The potential exists for additional development and population growth to occur on lands that are within the Village of Nobleton settlement area boundary".*
>
> *"The total population of the Village of Nobleton could reach between 9,600 and 10,900 persons based on the amount of land designated for residential development / redevelopment. This additional development and population growth will require an amendment to this plan and can be considered when the Township completes its next municipal comprehensive review to the planning horizon of 2041."*

1.3.3 Water and Wastewater Master Plan

In November 2016, the Regional Municipality of York updated its water and wastewater Master Plan with the purpose of determining the water and wastewater infrastructure requirements needed to support provincially mandated growth forecasts and proposed community expansion of about 9,500 people by 2041, and to develop a long term strategy to ensure that York Region continues to serve its residents in an environmentally and economically sustainable manner (York Region, 2016).

The updated Master Plan explains how the Region will meet the goal of sustainable growth through adopting a new "One Water" approach, which aims to realize the value of water whether in a lake, river, aquifer or municipal system. The updated Master Plan will also integrate water and wastewater initiatives with the Region's Official Plan, Transportation Master Plan and other strategies to ensure the needs to service growth are met cost effectively.

For the community of Nobleton, in order to develop a cost effective, resilient water and wastewater infrastructure plan to service future growth to 2041 and beyond, the Master Plan has subjected the community to a Class Environmental Assessment (the "Class EA") study. The Class EA will allow for the evaluation of environmental effects of alternatives to a project, alternative methods of carrying out a project and to select a preferred solution necessary to provide municipal services required to meet projected population growth in the community of Nobleton.

The Master Plan recommended conducting a Schedule C Class EA project to provide alternatives to increase the water supply capacity to support proposed community expansion to about 9,500 people by 2041 through either addition of new wells and/or revision of existing MOECC PTTW. Similarly, a Schedule C Class EA project was recommended for wastewater servicing as well.

The Class EA project aims to enable the future development of the Greenfield lands currently designated by the approved Nobleton Community Plan, and fulfill the Township of King's infill opportunities and intensification target to 2041 in residents in an environmentally and economically sustainable manner.

2 Existing Water System

2.1 EXISTING WATER SYSTEM DESCRIPTION

The Regional Municipality of York (also referred to as York Region or the Region in this report) is responsible for the water production, treatment, storage and transmission to its local area municipalities, including the Community of Nobleton in the Township of King. The Nobleton water supply system consists of three groundwater wells and two elevated storage tanks that provide service to the Nobleton Pressure District. There is also a booster station that services a higher elevation area in the northwest portion of the distribution system. The wells operate based on level at either of the elevated tanks. The booster station operates independently from the rest of the water system controls.

2.1.1 Supply

Table 1 provides a brief summary of the Nobleton wells. The current combined permitted daily withdrawal (Permit To Take Water) is 4,460,000 Litres (4.460 ML/D). This is equivalent to the sum of Nobleton Well #2 and Nobleton Well #3 or #5. In other words, the current limit ensures that one of the large wells (#3 or #5) is available as redundancy during maximum day demand conditions. If all three wells could operate simultaneously, then the total supply capacity could be 6.956 ML/D. It is noted that Wells #3 and #5 can operate together as long as the daily limit is not exceeded.

Table 1: Nobleton Well Summary

FACILITY	NOBLETON WELL #2	NOBLETON WELL #3	NOBLETON WELL #5	COMBINED LIMIT
Location	22 Faris Avenue	14 Royal Avenue	12800 Highway 27	
Commissioning Year	1960	1960	2015	
PTTW Limit (ML/D)	1.964	2.496	2.496	4.460
Standby Generator	No	Yes	Yes	
Disinfectant	Chlorine Gas	Sodium Hypochlorite	Chlorine Gas	

(MOECC, 2014) (York Region, 2013) (York Region, 2016) (York Region, 2015)

Each of the Nobleton wells are installed within the Scarborough Aquifer. The wells are developed within this stratified aquifer at depths below 83 metres below ground surface.

Based on discussions with the Region's operations staff, it is understood that the wells are currently on an auto-duty-rotate. This is done to ensure that all three of the wells are consistently used at a similar frequency. This is proper operational practice both for maintenance purposes, as well as to ensure that all of the wells are maintained below the annual permitted water takings.

2.1.2 Storage

Nobleton South Elevated Tank has a storage volume of 2,045m^3 and is located at 117 Russell Snider Drive. Nobleton North Elevated Tank, built in 2012, has a storage volume of 1,800m^3 and is located at 13740 Highway 27. The combined storage volume available in Nobleton is 3,845m^3.

2.1.3 Distribution

The Nobleton water distribution network consists of both York Region's infrastructure and the Township of King's infrastructure. The Region only owns a few watermains, which are either inlet/outlets for the elevated storage facilities or are within the three well facilities. The remainder of the distribution network is owned and operated by the Township of King, as shown in Figure 1.

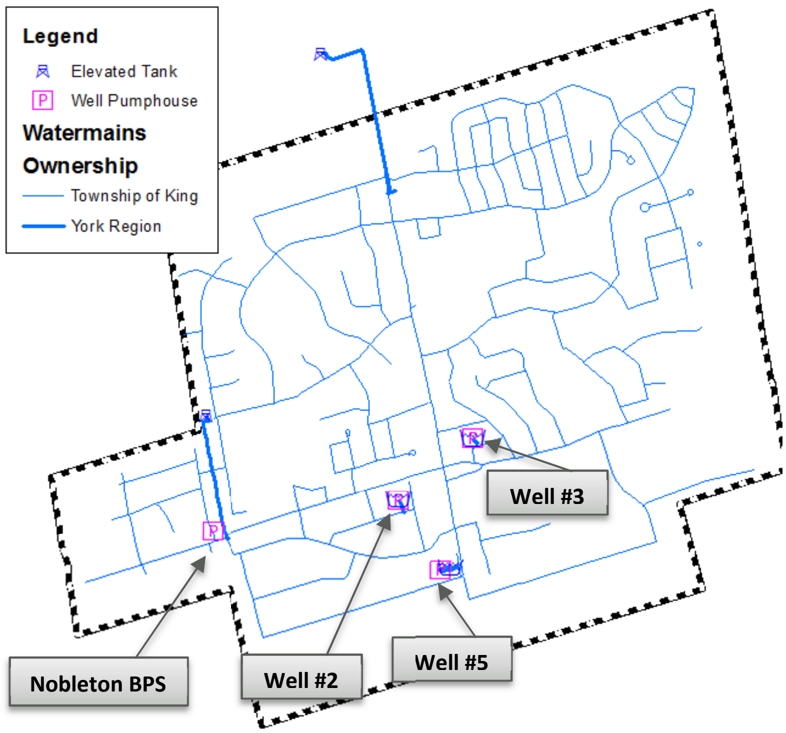

Figure 1: Existing Nobleton Water System

2.2 EXISTING WATER SYSTEM CONDITION

On November 9th, site visits to each of the Nobleton Wells were conducted with York Region Operations staff. Based on the available condition assessment reports, operator feedback and the site visits, the following can be summarized about the condition of each well facility:

Nobleton Well #2:

- Nobleton Well #2 is in generally good condition. The most recent Condition Assessment Report (Yaku / Associated Engineering / Pro F&E, 2014) documents three grouped capital projects over the next 25 years; including: Site Works, Yard Piping and Storage & Distribution in 2023; Upgrade Controls, Health & Safety, Rehabilitate Building Elements and Electrical in 2026; and Upgrade Well Pump, Piping & Valving, Chemical Systems and Casing & Screen Performance in 2038.

- York Region operations did not note any issues with the use of Nobleton Well #2. However, it is noted that Nobleton Well #2 is the only Nobleton well without a generator for standby power. Well #2 was constructed in 1961 and no rehabilitation has been required to date.

Nobleton Well #3:

- Nobleton Well #3 is in generally good condition. The most recent Condition Assessment Report (Yaku / Associated Engineering / Pro F&E, 2014) documents three grouped capital projects over the next 25 years; including: Site Works, Yard Piping, Storage & Distribution and Plumbing Upgrades in 2015; Upgrade Controls, Health & Safety, Rehabilitate Building Elements and Electrical in 2026; and Upgrade Well Pump, Piping & Valving and Chlorination System in 2039.

- York Region operations did not note any issues with the use of Nobleton Well #3, except that they have a preference to switch the sodium hypochlorite to chlorine gas.

- Rehabilitation of Well #3 was recommended in 2009 and was successfully completed in 2010 to return the well capacity to in excess of 22.5 L/s.

Nobleton Well #5:

- Well #5 was commissioned in 2015 and is in generally excellent condition.

- York Region operations did not note any issues with the use of Nobleton Well #5.

2.3 EXISTING POPULATION AND WATER DEMANDS

2.3.1 Population

Population and employment estimates for 2016 were provided by the Regional Municipality of York and are summarized below:

Table 2: Existing Population and Employment Estimates

YEAR	# OF UNITS	POPULATION	EMPLOYMENT
2016	1,610	5,520	772

2.3.2 Historical Production Data Review

As part of the historical demand review, hourly production records (SCADA) were obtained from 2012 to 2018. This was then used to analyze the average day demands (ADD), maximum day demands (MDD) and typical diurnal pattern in the Nobleton system over the past seven years, by conducting a flow balance.

2.3.2.1 Historical Average and Maximum Day Demands

The historical average day and maximum day demands are summarized in Table 3 and Figure 2.

Table 3: Recent Historical Nobleton Production Data (2012-2018) Summary

YEAR	2012	2013	2014	2015	2016	2017	2018
Average Day Demand (L/s)	13.9	14.9	14.9	16.1	21.1	20.4	23.1
Maximum Day Demand (L/s)	33.1	30.0	29.1	33.6	44.0	37.4	45.5
Maximum Day Demand Date	11-Jul	20-Aug	25-Jul	29-Jul	23-Jun	26-Sep	4-Jul
ADD : MDD Peaking Factor	2.39	2.01	1.96	2.09	2.09	1.83	1.97

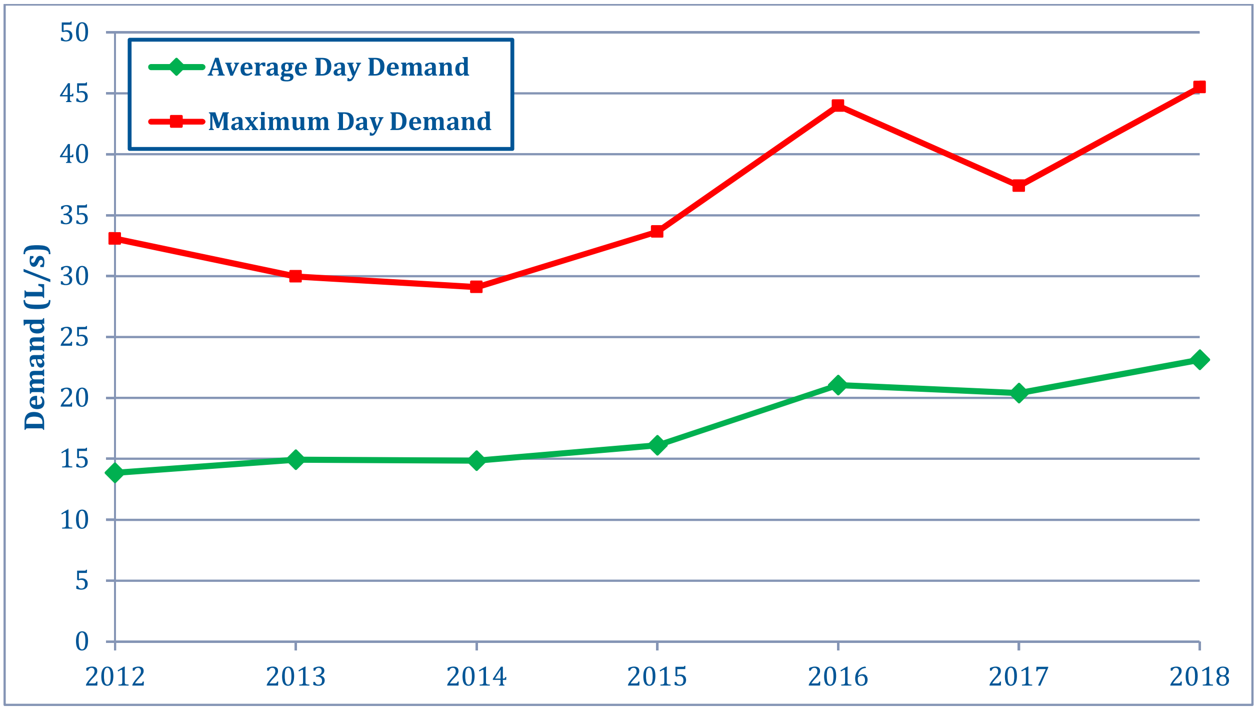

Figure 2: Historical Nobleton Average and Maximum Day Demands

Based on the information provided in Table 3 and Figure 2, the following observations can be made:

- Average day demands have been rising gradually over the past 7 years with a particularly large jump in 2016.
- The average day demands showed a distinct increase after 2015 due to population growth in newly developed areas. The average day demand is slightly higher in 2018 than 2016 due to continued population growth between 2016 and 2018. It should be noted that even though there is no confirmed population number for 2018, the new billing accounts show that there is a population increase. Therefore, the average day demands are generally equivalent on a per capita demand basis in 2016 and 2018.
- Maximum day demands are significantly more variable since they are much more impacted by the year-to-year variation in weather patterns (rainfall and temperature). See Section 2.3.2.2 for more on the influence of weather on demands.
- The highest historical maximum day demand, which occurred in 2018, was 45.5 L/s. The 2016 maximum day demand was similarly high at 44.0 L/s. Although an exact population in 2018 is unknown, it is noted that on a per capita basis the maximum day demand would be higher in 2016 than 2018.

2.3.2.2 Weather Influence on Average Day Demand

To get a better understanding of the increase in demands from 2015 to 2018, the daily data for each year is examined with certain weather parameters overlaid (cooling degree days and rain precipitation). This is not discussed for 2012 to 2014, since it does not provide any additional value. Cooling degree days is a measure of how much (in degrees) and for how long (in days) the outside air temperature was above a certain level. (Weather data was obtained from Environment Canada)

Figure 3, Figure 4, Figure 5 and Figure 6 display the 2015, 2016, 2017 and 2018 daily demand trends, respectively.

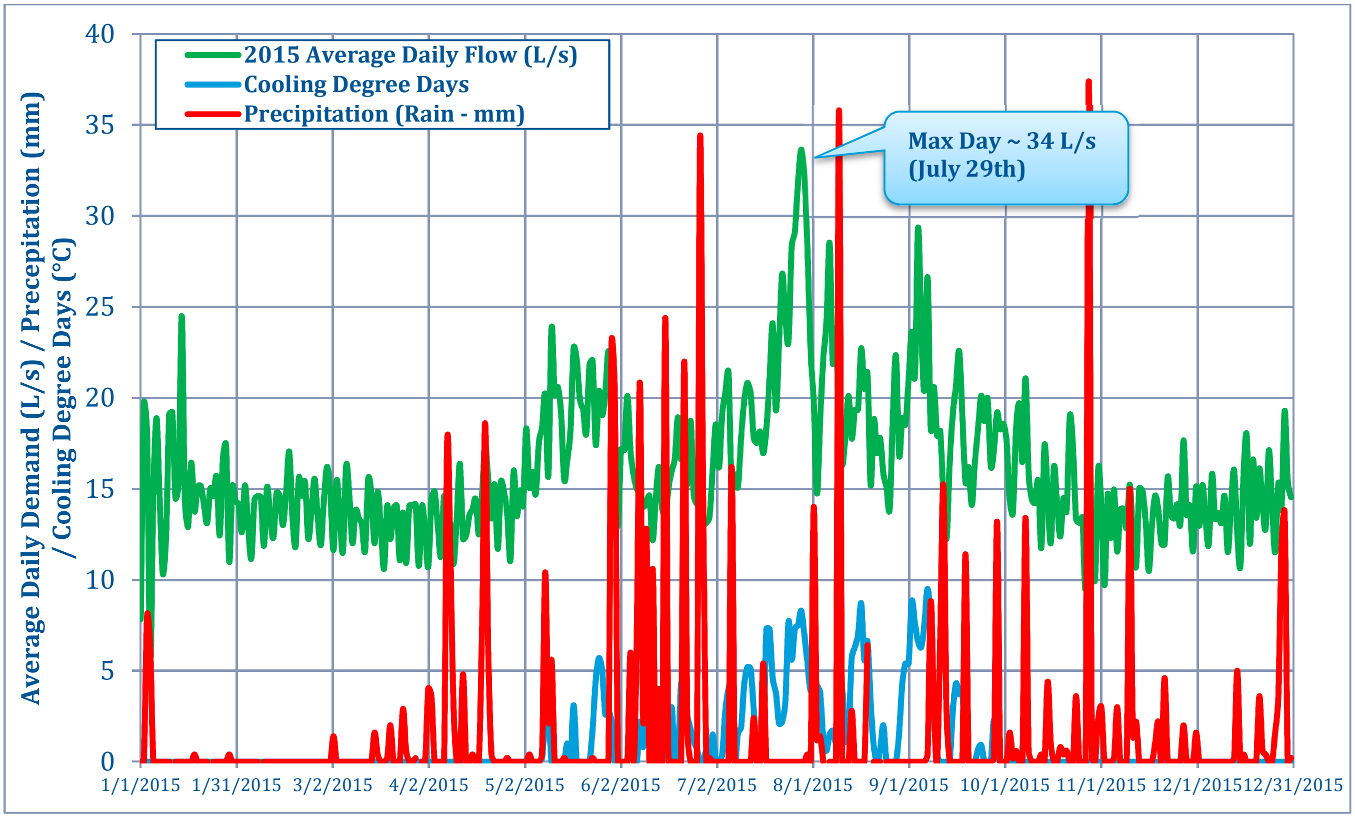

Figure 3: 2015 Daily Demand and Weather Trends

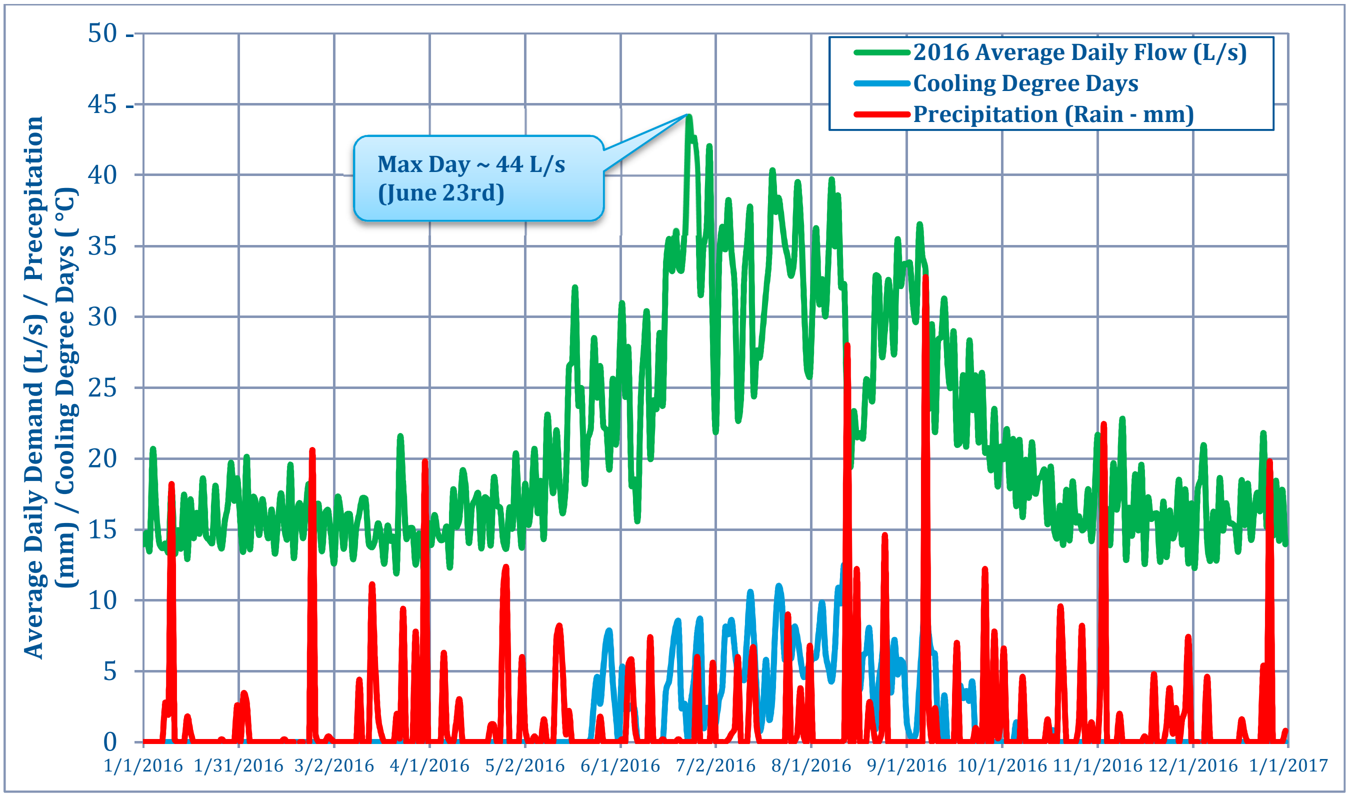

Figure 4: 2016 Daily Demand and Weather Trends

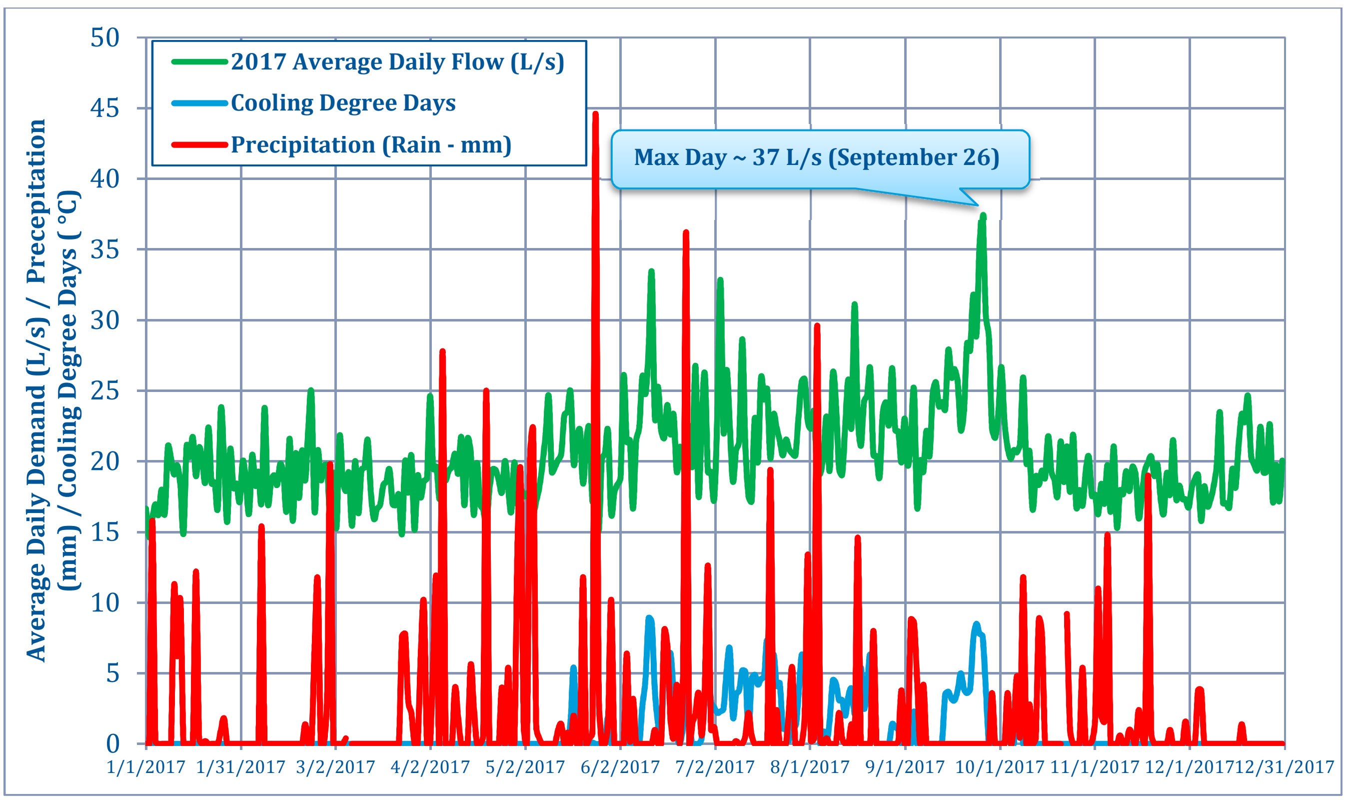

Figure 5: 2017 Daily Demand and Weather Trends

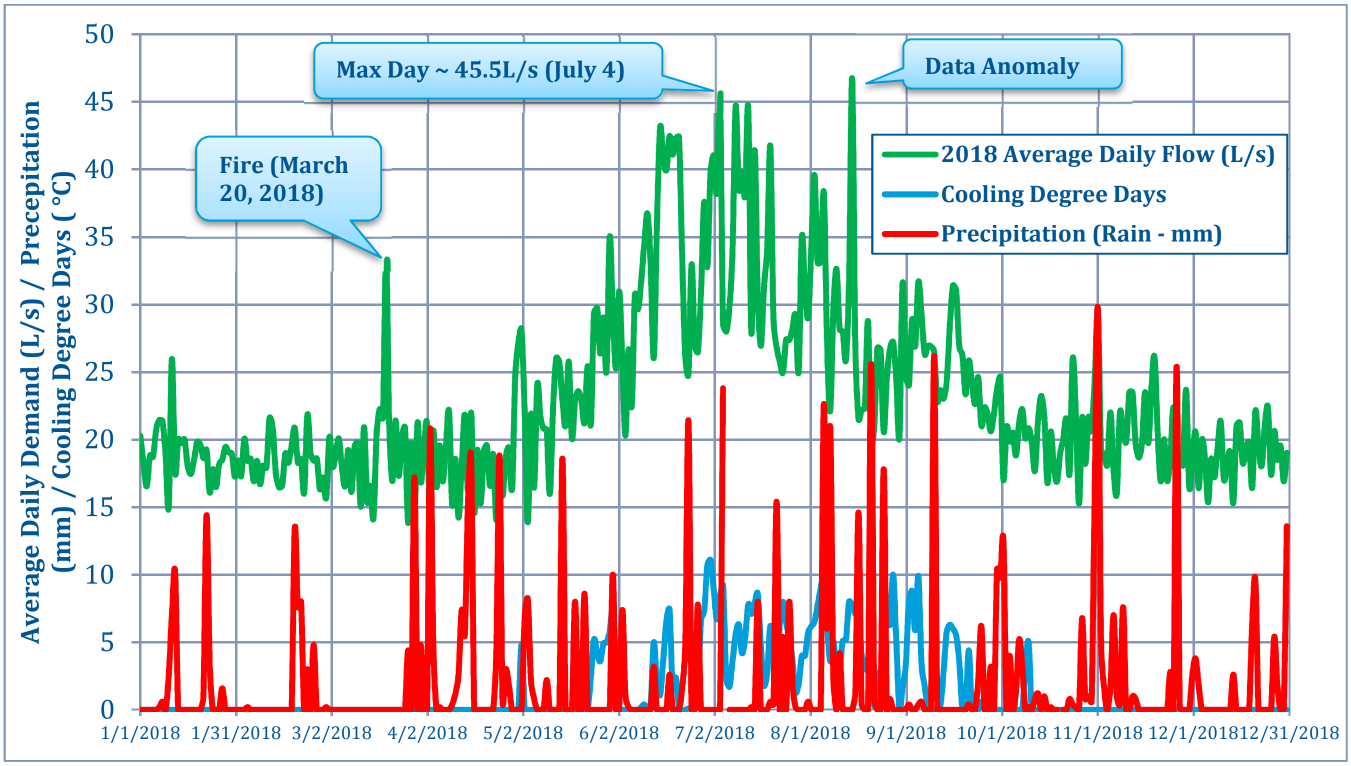

Figure 6: 2018 Daily Demand and Weather Trends

Tables 4, 5, 6 and 7 summarize key observations stemming from the above figures.

Table 4 Summary of 2015 Demand and Weather Trends

DATE RANGE	TEMPERATURE TREND	RAINFALL TREND	AVERAGE DEMAND TREND
January 1 to April 30, 2015	Generally cool or cold	Low rainfall, except for two large events in April and one moderate event in January	Generally stable, with an average of 14.0 L/s over this period
May 1 to September 30, 2015	Variable, with warmer days	Variable, many rainfall events	ADD lower during rain events and higher during dry spells Overall average of 18.9 L/s during this period
October 1 to December 31, 2015	Generally cool or cold	Numerous rainfall events	Generally stable, with an average of 14.3 L/s over this period
2015 Overall	--	--	Overall ADD of 16.1 L/s

Table 5 Summary of 2016 Demand and Weather Trends

DATE RANGE	TEMPERATURE TREND	RAINFALL TREND	AVERAGE DEMAND TREND
January 1 to April 30, 2016	Generally cool or cold	Three large and four moderate rainfall events over this period	Generally stable, with an average of 15.5 L/s over this period
May 1 to September 30, 2016	Variable, with warmer days	Generally low rainfall with several small events over this period	ADD trended up as temperatures increased and trended down as temperatures decreased Overall average of 28.1 L/s during this period
October 1 to December 31, 2016	Generally cool or cold	Low rainfall, except for two large events	Generally stable, with an average of 16.7 L/s over this period
2016 Overall	--	--	Overall ADD of 21.1 L/s

Table 6 Summary of 2017 Demand and Weather Trends

DATE RANGE	TEMPERATURE TREND	RAINFALL TREND	AVERAGE DEMAND TREND
January 1 to April 30, 2017	Generally cool or cold	Frequent rainfall events over this period	Generally stable, with an average of 18.7 L/s over this period
May 1 to September 30, 2017	Variable, with some warmer days. Generally cool summer.	Frequent rainfall with large rainfall events each month.	ADD maintained reasonably stable due to frequent rainfall and cool temperature. Hottest dry spell occurs in September, causing maximum day. Overall average of 22.5 L/s during this period
October 1 to December 31, 2017	Generally cool or cold	Low rainfall	Generally stable, with an average of 19.2 L/s over this period
2017 Overall	--	--	Overall ADD of 20.4 L/s

Table 7 Summary of 2018 Demand and Weather Trends

DATE RANGE	TEMPERATURE TREND	RAINFALL TREND	AVERAGE DEMAND TREND
January 1 to April 30, 2018	Generally cool or cold	A few moderate rainfall events over this period	Generally stable, with an average of 18.6 L/s over this period
May 1 to September 30, 2018	Variable, with sustained warmth from end of June to August	Occasional moderate rainfall events over this period. (Not as dry as 2016, but still hot and dry year)	ADD trended up during hot and dry spell from mid-May to end of June. Occasional storm dropped water use before rebounding. Overall average of 28.6 L/s during this period
October 1 to December 31, 2018	Generally cool or cold	Numerous rainfall events including two large events	Generally stable, with an average of 19.9 L/s over this period
2018 Overall	--	--	Overall ADD of 23.1 L/s

Based on the information provided in Figures 3 to 6 and Tables 4 to 7, the following observations can be made:

- During the low demand months (January to April and October to December) when weather related impacts to water demand are small, the demands experienced are gradually increasing from 2015 to 2018. For example, from January to April, the average demand in 2015 was approximately 14.0 L/s, which then increased to 15.5 L/s in 2016, 18.7 L/s in 2017 and 18.6 L/s in 2018. This is understood to be caused by the increased population associated with the developments in the southwest and northeast parts of Nobleton over the past few years.
- The data shows a correlation between ADD and weather/rainfall, so it follows that the reason for the large jump in average day demand from 2015 to 2016 (as per Table 3) is due to a combination of two factors:
    1. Increased population in Nobleton from new developments causes an increase in the base demand throughout the year. However, as seen from the January to March data, this increase is reasonably small (14.1 L/s to 15.4 L/s). New growth in Nobleton is noticeable when looking at the billing data records (in Section 0) which shows an increase in the number of residential billing records from 1467 to 1633.
    2. The hot and dry summer that occurred from May through to September 2016 caused a much higher "average summer demand" to occur as compared with 2015. The higher temperatures and lack of precipitation caused water users to significantly increase their water consumption, particularly for irrigation purposes.
- Proof of the hot and dry summer in 2016 is also clear. During May, June and July 2016 there are no significant rain events (>10mm). Comparatively in 2015, there are numerous high

precipitation events in June. This combined with an increased number of cooling degree days in 2016 causes the increased water use.

- It is also interesting to see the impact of a storm event on the water consumption. In the middle of August 2016, the first significant storm event of the summer occurred. This immediately caused a drop in the average daily demand from >30 L/s to ~20 L/s.

- 2017 was a comparably cool and wet summer compared to 2016. This causes the average summer demand and the maximum day demand to be significantly lower. It also causes the 2017 average day demand to be lower than 2016, despite the increased water demands during the cooler months.

- 2018 was another hot and generally dry year. Therefore, similar to 2016, an increased maximum day demand and average day demand occurred. The annual average day demand is higher than any other prior year, however, on a per capita basis it is generally equivalent to 2016.

The 2016 maximum daily demand is a good benchmark for the Nobleton system demands because it is based on a hot and dry summer. The Nobleton Class EA should start with the assumption that this type of weather occurrence can happen again, therefore the water consumption rates should be based on this as a starting point. Therefore, the analysis of the existing system was based on an average day demand of 21 L/s and a maximum day demand of 44 L/s.

2.3.2.3 Diurnal Patterns

Diurnal patterns are critical for hydraulic modelling because it allows the model to simulate the actual variation in demands over the course of a day. Figure 7 displays the average diurnal patterns experienced in the Nobleton system over the past four years. Similarly, Figure 8 displays the average diurnal patterns experienced in Nobleton during the hot summer months in 2016 and 2018.

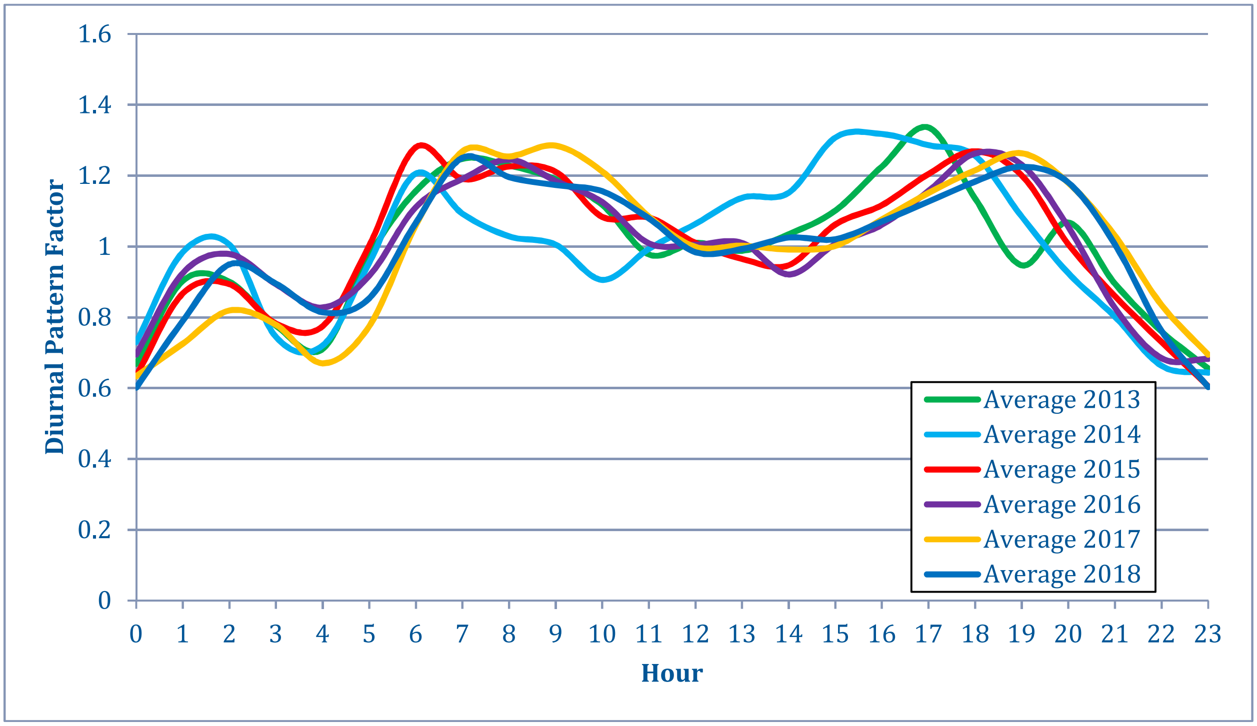

Figure 7: Average Diurnal Pattern by Year - Nobleton

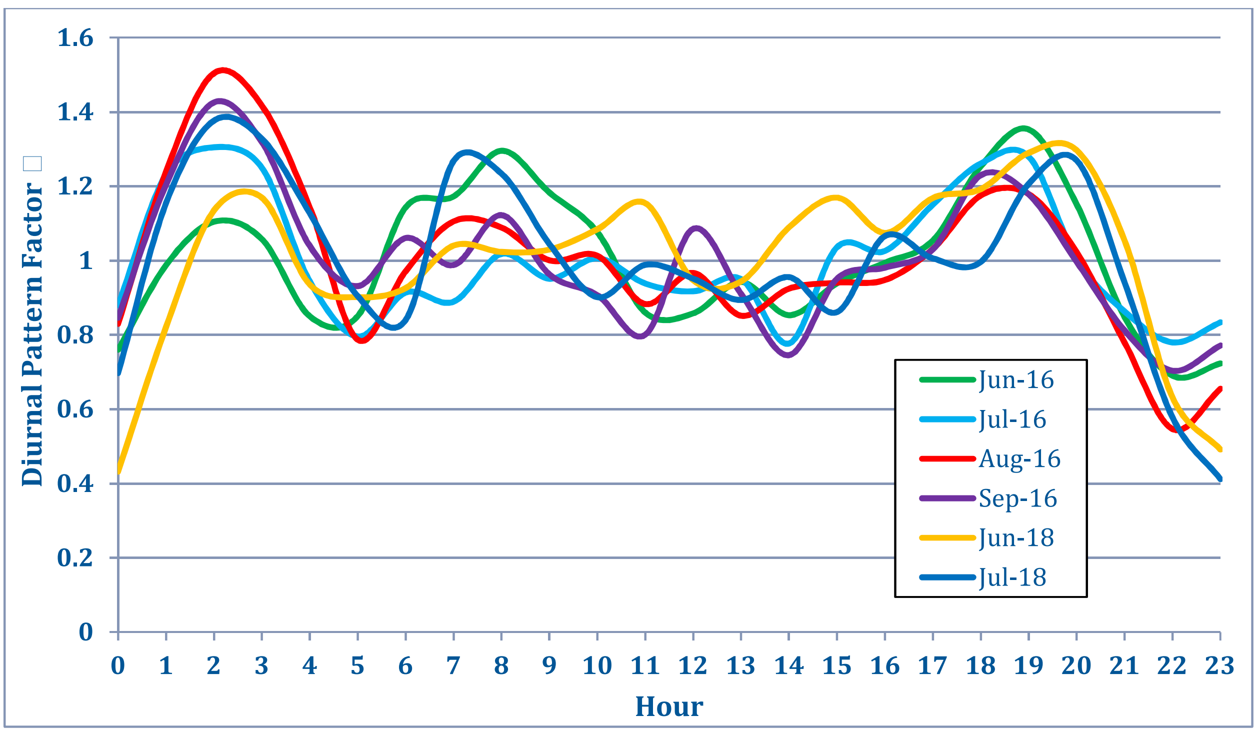

Figure 8: Average Diurnal Pattern by Summer Month in 2016/2018 - Nobleton

Based on Figure 7 and Figure 8, the following observations can be made:

- Generally, the average diurnal pattern over the course of a year (Figure 7) displays a standard two peak pattern, where there is a morning peak (6-9am) and an evening peak (5-8pm). However, there is also a small peak shown in the middle of the night that appears to be due to grass watering in the summer (sprinkler systems).

- The average pattern is largely consistent from year to year. Therefore, for the average day modelling, the 2016 pattern was selected. However, the average diurnal pattern is not a suitable pattern to use when simulating the maximum day demand scenario in the model. Therefore, the diurnal patterns were also analyzed for the summer of 2016.

- The summer months of 2016 and 2018 displayed a noticeably different water demand trend than during the rest of the year. The largest peak demand seems to occur during the overnight hours (1am to 4am). This is most prevalent in August and September 2016, as well as, July 2018. This is a trend seen in many small communities that have large lot sizes because of the increased use of water for irrigation purposes. During the summer months, the overall pattern is still generally a two peak pattern, but with an overnight (grass watering) peak and a smaller evening peak.

- For the maximum day hydraulic modelling, it is most appropriate to select the August 2016 pattern since it displays the largest peak during the day of about 1.5 times the daily average. By selecting this pattern, the model will be able to more accurately simulate real time demands in the system.

2.3.3 Historical Billing Data Review

Monthly billing totals were provided for each customer in Nobleton for 2015, 2016 and 2017. 2018 data was not available at the time of preparation of this report. This was analyzed in two ways:

1. The spatial allocation of demands uses this data since it is possible to geographically locate each address. Details of this allocation are provided in Study 2A: Existing Water System Hydraulic Analysis.
2. The total billed water consumption was compared to the production data to get a better understanding of the ratio of non-revenue water to total water production in the Nobleton water system.

Table 8 summarizes the billing data averages (in L/s) for each month in 2015, 2016 and 2017.

Table 8: Billing Data (2015-2017) Summary in L/s

YEAR	TYPE	# OF RECORDS	AVERAGE FLOW (L/S) BILLED												
			Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
2015	Res	1489	10.7	10.8	10.8	12.1	12.2	12.2	13.3	13.5	13.4	10.6	10.5	10.5	11.7
	ICI	46	0.9	0.9	0.9	0.6	0.6	0.6	0.5	0.5	0.5	0.5	0.5	0.5	0.6
	Total	**1535**	**11.6**	**11.7**	**11.7**	**12.7**	**12.7**	**12.8**	**13.8**	**14.0**	**13.9**	**11.1**	**11.0**	**11.0**	**12.3**
2016	Res	1674	11.4	11.4	11.4	13.4	13.4	13.7	21.0	20.3	20.5	11.2	11.0	10.9	14.1
	ICI	48	0.5	0.5	0.5	0.8	0.8	0.8	0.6	0.6	0.6	0.5	0.5	0.5	0.6
	Total	**1722**	**11.8**	**11.9**	**11.9**	**14.1**	**14.2**	**14.5**	**21.6**	**20.9**	**21.1**	**11.6**	**11.4**	**11.4**	**14.7**
2017	Res	1674	12.8	12.8	12.8	12.7	12.7	12.8	14.5	14.6	14.6	11.4	11.4	11.4	12.8
	ICI	47	0.5	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6
	Total	**1722**	**13.3**	**13.3**	**13.3**	**13.3**	**13.3**	**13.4**	**15.2**	**15.3**	**15.3**	**12.1**	**12.1**	**12.1**	**13.5**

Based on the information provided in Table 8 and other data from the production data review, the following observations can be made:

- Comparing 2015 billed flow (12.3 L/s) with 2015 production records flow (16.1 L/s), it is seen that approximately 76% of total produced water was billed. Therefore, in 2015, 24% of the total volume of water produced is considered non-revenue water.
- Comparing 2016 billed flow (14.7 L/s) with 2016 production records flow (21.1 L/s), it is seen that approximately 70% of total produced water was billed. Therefore, in 2016, 30% of the total volume of water produced is considered non-revenue water.
- Comparing 2017 billed flow (13.5 L/s) with 2017 production records flow (20.4 L/s), it is seen that approximately 66% of total produced water was billed. Therefore, in 2016, 34% of the total volume of water produced is considered non-revenue water.

- These approximations match reasonably well with the historical non-revenue water values that are listed in the Long Term Water Conservation Strategy (LTWCS) Annual Reports and are summarized below:

Table 9 Historical Non-Revenue Water Values from Long Term Water Conservation Strategy Annual Reports

LOCATION	PERCENTAGE OF NON REVENUE WATER (%)			
	2010	2012	2014	2015
Township of King	18.0	25.0	28.2	26.6
York Region	14.0	14.0	13.2	15.0

(York Region, 2013) (York Region, 2014) (York Region, 2016) (York Region, 2017)

- Based on the historical non-revenue water estimates for the Township of King and the estimated values for Nobleton based on billing and production records, the modelling assumes a 26.5% non-revenue water component of total system demand.

- Furthermore, it is important to note that, in each year, the largest water user in Nobleton accounted for approximately 1% of the total billed water. Therefore, there are not any particularly large water users in the Town of Nobleton that would require special consideration when modelling.

2.3.4 Existing Unit Consumption Rates

Based on the existing 2016 residential and employment population (see Section 2.3.1), as well as, the actual billed demands (see Section 2.3.3), it is possible to calculate the 2016 unit consumption rates for residential and employment population separately.

Table 10 2016 Nobleton Unit Consumption Rates

LOCATION	2016 ADD (L/S)	2016 POPULATION	UNIT CONSUMPTION RATE (EXCLUDING NON-REVENUE WATER) IN LPCD	UNIT CONSUMPTION RATE (INCLUDING NON-REVENUE WATER) IN LPCD
Residential	14.1	5,530	220.3	316.4
Employment	0.57	772	63.8	91.6
Non-Revenue Water	6.7	n/a	n/a	n/a

It is noted that residential consumption rates in Nobleton are noticeably higher than those used in the 2016 Master Plan. Conversely, the employment rates in Nobleton are significantly lower.

Unit consumption rates in 2015 and 2017 are noticeably lower due to the previously discussed weather trends. Therefore, 2016 is used as a baseline for unit consumption rates. It is noted that the 2018 results appear to show similar unit consumption rates as 2016 since both the average day demand and population increased. Overall, this confirms that the 2016 results are not an isolated outlier and this data should be used as the baseline for unit consumption rates.

3 Water System Design Criteria

The following table summarizes the design criteria that are to be used throughout the analysis. These design criteria serve as a guideline to identify when hydraulic performance is acceptable and when certain upgrades may be necessary to address any deficiencies.

Table 11: Design Criteria Summary

PARAMETER	CRITERIA	LEVEL OF SERVICE	COMMENTS
Well Capacity	Well Supply Capacity	Combined Well Supply > Maximum Day Demand	- The combined capacity of the three wells should exceed the maximum day demand to ensure summer demands can be met.
	Well Firm Capacity	Combined Well Supply with Largest Well Out of Service > Average Day Demand	- The combined capacity of the three wells with the largest single well out of service should still exceed the average day demand to ensure that normal conditions can be met even during emergency conditions when a well is taken down for maintenance, etc..
Storage Volume	Equalization Storage	Maximum Day Demand x Equalization Rate	- York Region design requirement for Equalization Rate is 25%. This is consistent with MOECC guidelines. - The suitability of this can be evaluated by analyzing the actual historical diurnal patterns from recent high demand periods. - Consideration could be made to reducing this percentage, based on historical data, or by removing the non-revenue water component of the MDD.
	Fire Storage	10,000 L/min for 2 hours	- York Region standards dictate that a small pressure district has fire flow storage equal to a flow of 10,000 L/min for 2 hours. - MOECC guideline is similar, but is based on the size of the population serviced. - For population of 6,001-10,000, the recommendation is 159L/s (9540L/min) for 2 hrs. - For population of 10,001-13,000, the recommendation is 189L/s (11340L/min) for 3 hrs.
	Emergency Storage	25% of Fire+ Equalization	- York Region uses 25% of the total fire and equalization storage. This is consistent with MOECC guidelines.
	Total Storage	Equalization + Fire + Emergency	- York Region total storage design requirement is the sum of the three storage components (equalization, fire and emergency). This is consistent with MOECC guidelines.
Pressure	Minimum Pressure - Normal Conditions	>40psi	- As per York Region Design Guidelines. Also consistent with MOECC guideline.
	Minimum Pressure - Fire Flow Conditions	>20psi (distribution) >25psi (transmission)	- As per York Region Design Guidelines. Also consistent with MOECC guideline. (generally 25psi for the transmission system minimum)
	Maximum Pressure	<100psi	- As per York Region Design Guidelines. Also consistent with MOECC guideline.
Fire Flow	System Demand	Maximum Day Demand	- Fire flow availability is to be analyzed during the maximum day demand
	Minimum Flow	5,000 L/min	- For residential customers;

PARAMETER	CRITERIA	LEVEL OF SERVICE	COMMENTS
			- Only Regional system is to be checked with Regional fire flow standards.
	Maximum Flow	10,000 L/min	- For ICI customers; - - Only Regional system is to be checked with Regional fire flow standards. -
Pipe Capacity	Maximum Velocity	<2.0m/s during normal conditions	- This parameter is used to identify pipes that may be contributing to pressure and/or flow deficiencies. - - Considered secondary criteria. Does not automatically trigger an improvement. -

(MOECC, 2008) (York Region, 2017) -

4 System Capacity Optimization Summary

The following section summarizes the existing system capacity and any optimization opportunities that exist regarding supply, storage and distribution. Detailed calculations and information about the hydraulic modelling is included in Study 2A: Existing System Hydraulic Analysis.

4.1 SUPPLY

4.1.1 Supply Capacity

The three existing Nobleton wells currently have a combined daily taking limit of 4.46 ML/D (51.62 L/s). This is equivalent to the sum of Nobleton Well #2 and Nobleton Well #3 or #5. In other words, the current limit ensures that one of the large wells (#3 or #5) is available as a standby pump whilst the other two wells act as duty supply during maximum day demand conditions. Each of the three wells are used throughout the year, as operations rotate the duty pumps on a weekly basis. No changes to the operational practices in Nobleton are recommended at this time.

4.1.2 Optimization Opportunities

Once the residential and employment unit consumption rates are established, the number of people that can be serviced with the existing firm capacity (largest well acting as standby) can be established. Furthermore, the approximate year in which the required supply will exceed the firm capacity can also be established.

Once the existing PTTW limit is reached, the only remaining optimization opportunity (excluding any option that increases the PTTW) is to use some of the surplus storage capacity that isn't required for equalization, emergency and fire storage. This requires an analysis of not only the maximum day demand, but also the maximum week demand because if demand exceeds the supply on the maximum demand day, then it is also quite likely to exceed the supply limit on the subsequent days (where demand is often equally as high).

4.1.3 Feasibility Analysis

Assuming 26.5% non-revenue water and the current residential and employment unit consumption rates (220 Lpcd and 64 Lpcd, respectively), there is potential for a residential population of approximately 6,800 and an employment population of 950 before the current PTTW/supply limit (51.62 L/s) is exceeded.

If the system demand exceeds the PTTW limit, the spare storage capacity (discussed in Section 4.2) can potentially be utilized to provide a small amount of additional supply to residents. This option is not normally considered because with distribution storage, the additional demand that can be supplied by surplus storage is generally very small. Any small surpluses in storage are generally best to be used to increase operational flexibility and to provide additional buffer for any extreme weather events (droughts, etc.). Based on the analysis shown in Section 3.3 of "Study 2A: Existing System Hydraulic Analysis", using the surplus storage could theoretically allow the maximum daily demand to reach ~56 L/s before additional well capacity is needed. However, this is based on various assumptions about the frequency of consecutive maximum demand days that are not easily predicted. Therefore, it is recommended that the Region increases the existing PTTW and supply capacity before the demand exceeds the current PTTW (51.62 L/s). Using surplus storage to meet supply deficits is a high-risk option that is not recommended.

4.2 STORAGE

4.2.1 Storage Surplus or Deficit

The two existing Nobleton storage facilities have a combined storage capacity of 3.845 ML. This is sufficient storage volume until the maximum day demand increases above 86.85 L/s. Detailed calculations to support this can be found in Section 3.2.2 of Appendix 1 (Existing System Hydraulic Analysis).

4.2.2 Optimization Opportunities

No optimization is needed to increase storage capacity since there is already sufficient storage capacity up to a maximum day demand of 86.85 L/s.

4.3 DISTRIBUTION NETWORK

4.3.1 System Bottlenecks and Limitations

Based on the hydraulic analysis of the system, there are no system bottlenecks or limitations that are preventing the Region's well supply and storage volume to be distributed to the Township of King owned infrastructure in Nobleton. At minimum, the existing distribution network is capable of servicing the combined capacity of the three wells PTTWs (80.51 L/s). Detailed analysis to support this can be found in Section 3.2.3 of Appendix 1 (Existing System Hydraulic Analysis).

5 Conclusions

The following summarizes the results of the existing system analysis and system capacity optimization:

- The highest average day demands occurred in 2016 (21.1 L/s) and 2018 (23.1 L/s). The average day demand is slightly higher in 2018 due to the population growth between 2016 and 2018. However, the average day demands are generally equivalent on a per capita demand basis in 2016 and 2018.
- The highest historical maximum day demand, which occurred in 2018, was 45.5 L/s. The 2016 maximum day demand was similarly high at 44.0 L/s. Although an exact population in 2018 is unknown, it is noted that the maximum day demand on a per capita basis would be higher in 2016 than 2018.
- The 2016 maximum daily demand is a good benchmark for the existing Nobleton system demands because it is based on a hot and dry summer. The Nobleton Class EA should start with the assumption that this type of weather occurrence can happen again, therefore the water consumption rates should be based on this as a starting point.
- Based on the historical non-revenue water estimates for the Township of King and the calculated values for Nobleton based on billing and production records, the modelling assumes a 26.5% non-revenue water component of total system demand.
- Based on the results of the existing system hydraulic analysis, there are no hydraulic limitations (bottlenecks) in the existing pipelines.
- The first limitation that will arise in the Nobleton system is the combined daily taking limit (PTTW) from the three Nobleton wells. The current combined daily taking limit of the Nobleton wells (51.62 L/s).
- The current PTTW will need to increase once Nobleton's maximum day demands exceed 51.62 L/s.
- If an increase in the PTTW is obtained, the Nobleton system could be able to increase its maximum day demand capacity to the sum of the individual daily taking limits for the three Nobleton wells (80.51 L/s). Since it is desired that the Region's system maintains the ability to provide firm capacity (one well available as standby), this would also require the addition of a new well of at least 2.496 ML/D capacity.

WELL	PERMITTED CAPACITY (ML/D)	PERMITTED CAPACITY (L/S)
Nobleton PW #2	1.964	22.7
Nobleton PW #3	2.496	28.9
Nobleton PW #5	2.496	28.9
Current Combined Daily Taking Limit (with Largest Well Out of Service)	4.460	51.6

- A hydrogeological study is required to confirm that the three existing Nobleton wells are capable of simultaneously operating at their permitted capacity without a negative impact on the groundwater supply.
- Any flow requirements beyond 80.51 L/s will require further increases to:
  - the Permit To Take Water; and
  - An increase in supply capacity from existing wells or new well(s)
- The existing storage capacity of the Nobleton system is sufficient to meet maximum day demands up to 86.85 L/s. Any flow requirements beyond 86.85 L/s will require either:
  - Additional storage capacity; or
  - Modifications to the calculations for equalization/fire/emergency storage.
- When the maximum day demand is less than ~56 L/s, it is possible that the surplus storage capacity can be used to offset slight deficiencies in the existing PTTW (51.62 L/s). However, this would be stretching the system to its absolute limit and is generally not recommended due to the unknowns regarding the frequency of consecutive maximum demand days that are not easily predicted.

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FINAL DRAFT

CLASS ENVIRONMENTAL ASSESSMENT FOR WATER AND WASTEWATER SERVICING IN THE COMMUNITY OF NOBLETON

WASTEWATER SYSTEM CAPACITY OPTIMIZATION STUDY

Study 1B

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

17 JANUARY 2019

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List of Abbreviations

ADF	Average Day Flow (Annual)
ADWF	Average Dry Weather Flow
BI	Base Infiltration
BOD	Biodegradable Oxygen Demand
ECA	Environmental Compliance Approval
EA	Environmental Assessment
F/M_v	Food to Micro-organism Ratio
g/c/d	grams per capita per day
HRT	Hydraulic Retention Time
I/I	Inflow and Infiltration
kg/d	kilograms per day
km	kilometer
L/s	Liters per second
L/c/d	Liters per capita per day
$L/s/m^2$	Liters per second per square meters
m	meters
m^3	cubic meters
m^3/day	cubic meters per day
m^3/hr	cubic meters per hour
MECP	Ministry of Environment, Conservation and Parks
mg/L	milligrams per liter
ml	milliliter
MLD	million liters per day
MLSS	Mixed Liquor Suspended Solids
MLVSS	Mixed Liquor Volatile Suspended Solids
MMF	Maximum Month Flow
MOE	Ministry of Environment
MOP	Manual of Practice
OLR	Organic Loading Rate
PDF	Peak Day Flow
PF	Peak Factor
PHF	Peak Hourly Flow
PIF	Peak Instantaneous Flow
pp	persons
PS	Pumping Station
PTTW	Permit To Take Water
PVC	Polyvinyl Chloride Pipe
SCADA	Supervisory Control and Data Acquisition
SRT	Solids Retention Time
SWD	Side Water Depth
TAN	Total Ammonia Nitrogen

TDH	Total Dynamic Head
TKN	Total Kjeldahl Nitrogen
TP	Total Phosphorus
TSS	Total Suspended Solids
TWAS	Thickened Waste Activated Sludge
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge
WPCP	Water Pollution Control Plan
WRRF	Water Resource Recovery Facility

1 Introduction

1.1 BACKGROUND

Nobleton is a community in King Township in the Regional Municipality of York (Region). Currently, Nobleton is serviced by stand-alone water and wastewater systems. The Regional Water and Wastewater Master Plan (2016) indicated that both the water and wastewater systems would not have sufficient capacity to support growth to the 2041 Master Plan horizon. Therefore, the Master Plan recommended undertaking the current project, a Schedule C Class Environmental Assessment (EA), to identify preferred servicing solutions to accommodate growth (York Region, 2016).

1.2 PREVIOUS STUDIES AND PLANNING DOCUMENTS

The following documents were reviewed during the development of report.

1.2.1 Regional Official Plan

The Region continues to experience rapid population and employment growth. In accordance with the York Region Official Plan 2010, significant population growth is expected within the next 25 years, to the planning horizon of 2031 (York Region). With a population of 1,156,000 residents as of mid-2015, it is anticipated that the Region will reach a population of 1.5 million people by 2031.

The York Region Official Plan has forecasted a population growth within King Township from 20,300 people in 2006 to 34,900 people in 2031. This represents an increase of 14,600 people. Employment is expected to increase from 7,100 in 2006 to 11,900 in 2031, for an increase of 4,800. The York Region Official Plan does not specify population distribution within King Township. The population for the community of Nobleton, which is part of King Township, is discussed in this report.

1.2.1.1 King Township Rural Official Plan

The current King Township Official Plan was approved in 1970 and is known as the "Parent Official Plan" (Township of King, 1970). This document establishes land use, transportation, and development policies for King Township.

In the 1990s, community plans were prepared for each of the villages in King Township (Nobleton, Schomberg, and King City) as well as for the hamlets. The Nobleton Community Plan was added to the King Township Official Plan through Official Plan Amendment 57, adopted by the Regional Council in 1997; the latest Office Consolidation was in 2005.

1.2.2 Water and Wastewater Master Plan

The Region updated the Regional Water and Wastewater Master Plan in November 2016. The objectives of this update are to:

- Determine the water and wastewater infrastructure requirements needed to support provincially mandated growth forecasts and proposed community expansion; and
- Develop a long-term strategy to ensure that the Region continues to serve its residents in an environmentally and economically sustainable manner (York Region, 2016).

The updated Master Plan explains how the Region will meet the sustainable growth goal by adopting a new "One Water" approach, which aims to realize the value of water whether in a lake, river, aquifer, or municipal system. The updated Master Plan integrates water and wastewater initiatives with the Region's Official Plan, Transportation Master Plan, and other strategies to ensure the needs to service growth are met cost effectively.

The Master Plan recommended conducting a Schedule C Class EA project to provide alternatives to increase the water supply capacity to support proposed community expansion to about 9,500 people by 2041 through either addition of new wells and/or revision of existing Ministry of the Environment, Conservation and Parks (MECP) Permit to Take Water (PTTW). Similarly, a Schedule C Class EA project was also recommended for wastewater servicing.

This Class EA project aims to enable future development of the greenfield lands currently designated by the approved Nobleton Community Plan and fulfill King Township's infill opportunities and intensification targets to the buildout residential population in an environmentally and economically sustainable manner.

Inflow and infiltration (I/I) reduction is an integral part of the Master Plan's Preferred Servicing Alternative. The aim of this reduction initiative is to reduce the loading on the wastewater conveyance system. In March 2016, the Region updated the Inflow and Infiltration Reduction Strategy, incorporating a One Water approach. The 2016 Inflow and Infiltration Reduction Strategy update sets the direction. For new developments, the strategy aims to prevent deficiencies in new sewers before the municipality can anticipate significant cost savings later. The 2016 Inflow and Infiltration Reduction Strategy update recommended raising construction and inspection standards, as well as engaging the Province to support implementation of new development design standards and construction practices.

1.2.3 Proposed New Development in the Community of Nobleton

In February 2018, the Region provided a proposed population increase in the community of Nobleton to 10,800 people. This proposed growth will be used as the basis for this Schedule C Class EA to assess alternative water and wastewater servicing solutions and select preferred alternatives to accommodate population growth to 10,800 people in the community of Nobleton. This will also enable future development of greenfield lands currently designated by the approved Nobleton Community Plan and fulfill King Township's infill opportunities and intensification targets to 2041.

1.3 PURPOSE OF THIS DOCUMENT

A System Capacity Optimization Study of the Nobleton Water Resource Recovery Facility (WRRF) is to be completed as part of the Class EA for water and wastewater servicing in the community of Nobleton. The purpose of this report is to satisfy this requirement. The purpose of the System Capacity Optimization Study is to evaluate the flows and loads that can be accommodated by the existing system with little or no investment.

2 Existing Wastewater System

2.1 WASTEWATER COLLECTION SYSTEM

The Nobleton wastewater collection system consists of a gravity sewage system which includes two pumping stations. Bluff Trail PS in the northeast of the catchment and Janet Avenue Pumping Station (PS) toward the south of the catchment. The Janet Avenue PS pumps all of the flows from the catchment to the Nobleton WRRF.

The current network does not cover all of the community of Nobleton, and some areas are still on septic tanks. There is currently an ongoing plan that will connect the remaining properties to the sewer system by 2021. This plan will include the installation of additional sanitary sewers that will drain to the Janet Avenue PS.

The PS at Janet Avenue was constructed in 2012 to convey the flows from the community of Nobleton to the WRRF. As-built drawings shows a wet well/dry well arrangement of the PS. From these drawings, it was inferred that initially the flows enter an inlet chamber before draining through one of three orifices into the wet well. The volume of the wet well is approximately 20 m^3. Above the wet well, there is a larger area that the flow can fill during wet weather. In the dry well, three dry pit submersible non-clog pumps operate on a two-duty and one-standby regime. In addition, there is an emergency overflow at the PS that prevents flooding if there are issues with the pumps. A section of the Janet Avenue PS is shown in Figure 2-1.

From the telemetry data and the flow survey calibration, the existing pump capacity was estimated to be 53 L/s at 54 TDH for each pump. The pumps currently operate as two-duty and one-standby arrangement which gives a firm capacity of 106 L/s. The maximum pump rate which has occurred was in 2017 with a rate of 97 L/s.

The existing forcemain delivers the flow from the Janet Avenue PS to the Nobleton WRRF. The existing forcemain is a 300mm polyvinyl chloride (PVC) pipe. It rises from the PS at Janet Avenue to a peak level of 284.02 mAD along King Road before dropping down to a level of 242.25 mAD at the Nobleton WRRF.

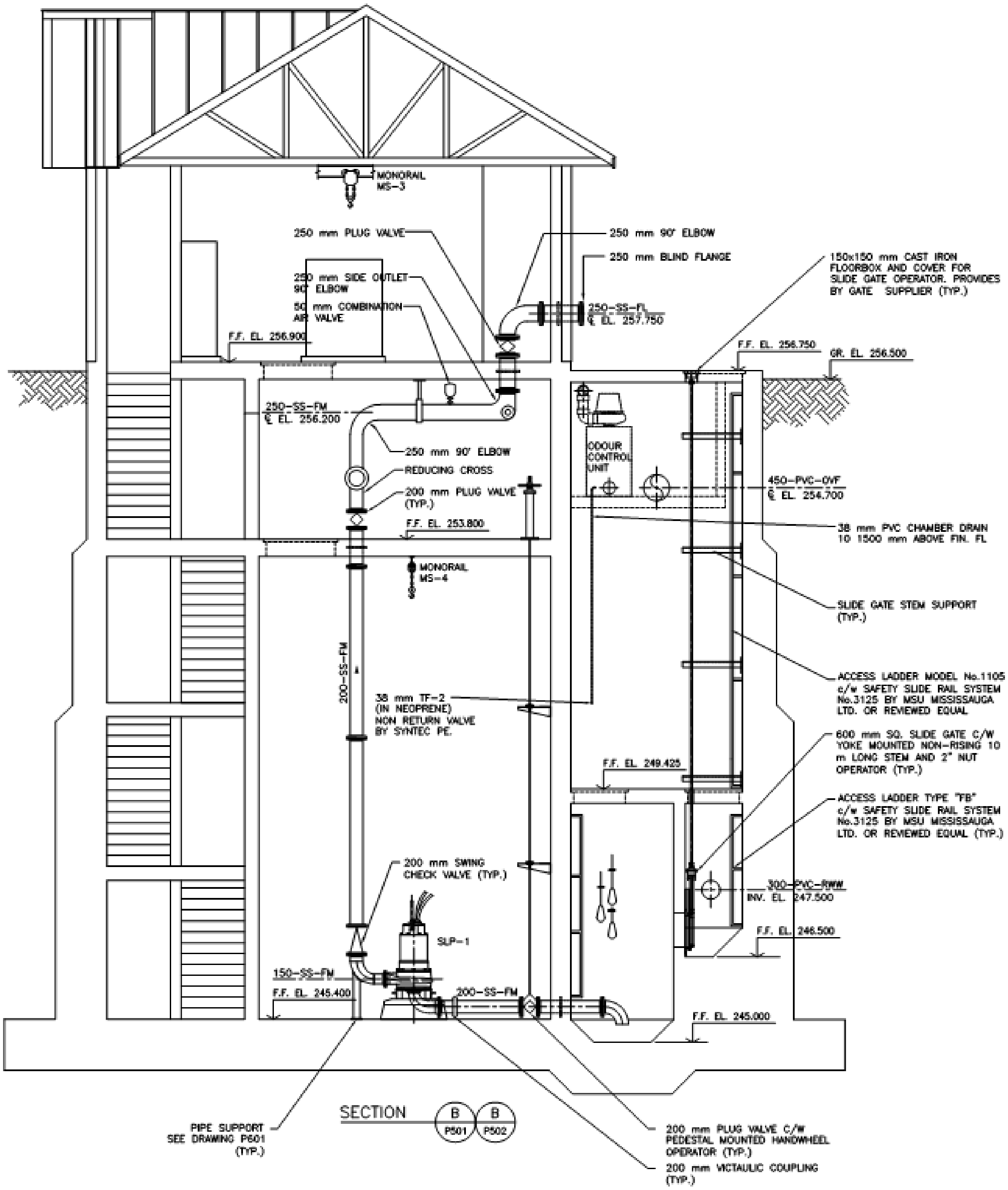

Figure 2-1: Section of Wet Well at Janet Avenue PS (As-Built Drawings, 2012)

2.2 WASTEWATER TREATMENT PLANT

The Nobleton WRRF is an extended aeration plant with tertiary filtration. The rated capacity is 2,925 m^3/day with a peak design flow of 9,177 m^3/day. The plant was originally designed to service 6,500 people and approval was granted to increase to 6,590 people. The treatment facility consists of the following unit processes prior to discharge to the Humber River via a constructed wetland:

- Inlet Works: Screening and Grit Removal System;
- Secondary Treatment: Extended Aeration Activated Sludge Process with Nitrification;
- Tertiary Filtration and UV Disinfection: Deep Bed Granular Filters, Continuous Backwash System equipped with Filter Reject Tanks;
- Chemical Feed System: Alum and Sodium Hydroxide; and
- Sludge Handling System with a gravity thickener and a thickened sludge storage tank.

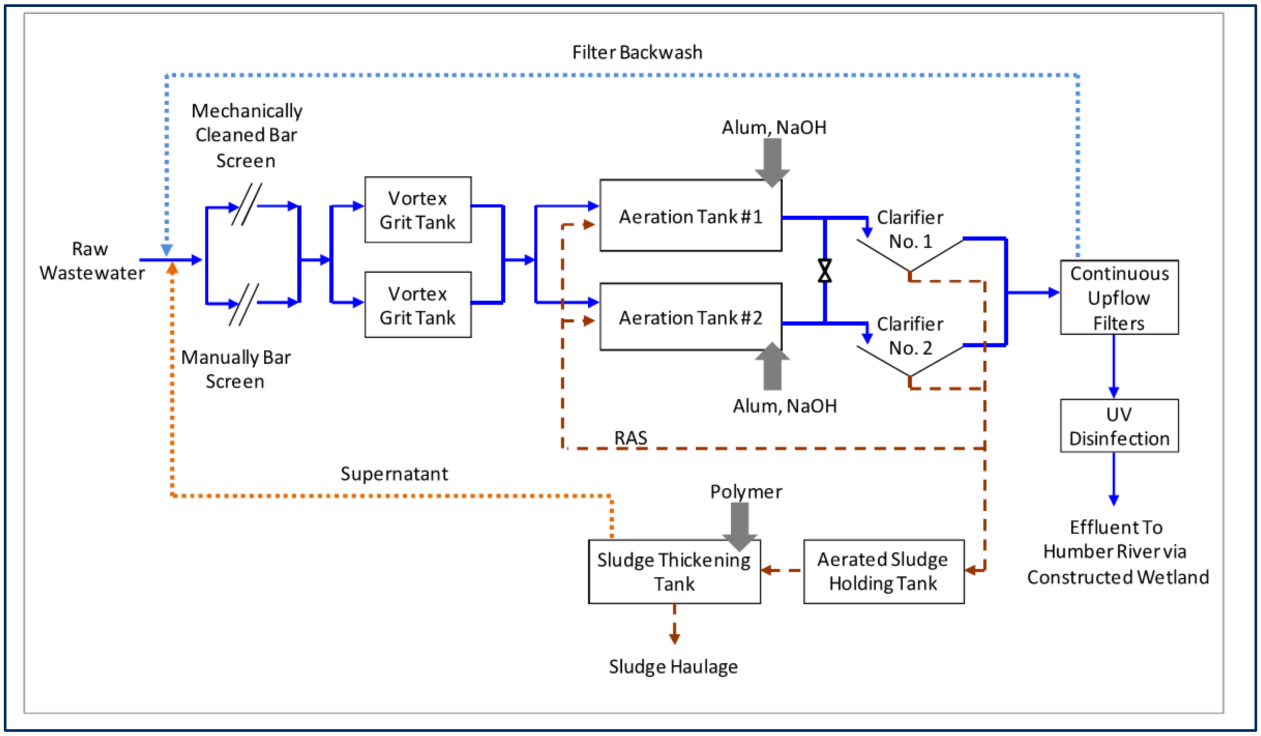

Figure 2-2: Nobleton WRRF Process Schematic (Source: Nobleton WRRF Operation Manual)

Figure 2-2 presents a process flow schematic of the Nobleton WRRF. Preliminary treatment consists of one mechanically cleaned bar screen and one manual bar screen, and two vortex grit removal systems. Secondary treatment consists of two rectangular aeration basins equipped with fine bubble diffusers. Three blowers (two duty, one standby) are used to supply process air. The mixed liquor from the aeration tanks flows into two circular secondary clarifiers for sedimentation. The clarifier effluent flows by gravity into four Parkson continuously backwashed, upflow, deep bed granular media filters (DynaSand). Tertiary effluent is disinfected year-round using UV disinfection. UV disinfection consists of two banks of Trojan low pressure, high intensity UV lamps. The treated effluent from the facility is discharged by gravity via 1.5 km of 450 mm concrete pipe to the Humber River via a constructed wetland.

Waste activated sludge (WAS) from the secondary clarifiers is gravity thickened in a sludge thickening tank, and then conveyed to an aerated sludge holding tank for storage. Supernatant from the sludge thickening tank is returned to the headworks upstream of the screens. The thickened sludge is hauled to the Duffin Creek WPCP for disposal.

Phosphorous is precipitated by alum addition at upstream of the secondary clarifier inlet and the tertiary filter influent channel. Sodium hydroxide can be added to the aeration tanks to provide supplemental alkalinity. Currently, sodium hydroxide is not added as there is sufficient alkalinity in the raw sewage to sustain nitrification.

Table 2-1 presents a summary of major process equipment and reactors for each unit process.

Table 2-1. Unit Process Summary

ITEM	UNIT	VALUE (1)	COMMENTS
Screening System			
Number of Screens	#	2	One mechanical duty unit One manual standby unit
Type	mm	12 50	Mechanical Manual
Peak Flow Capacity (Duty)	MLD	9.177	Mechanical Only
Grit Removal System			
Number of Grit Tanks	#	2	
Type	--	--	Vortex, with mechanical mixers
Dimension	m	2.0	Diameter
Capacity	MLD	9.177	PHF Rate For each Grit Tank
Screening Screw Conveyor			
Number of Conveyors	#	1	
Dimensions	mm	292 x 6,180	Diameter x Length
Inlet Capacity	m^3/h	1.5	
Discharge Capacity	m^3/h	1.5	
Aeration Tanks			
Number of Tanks	#	2	Only one tank is currently in operation
Dimension (each)	m	18 x 13.5 x 6.3	Width x Length x Height (SWD)
Volume (each)	m^3	1,536	
Volume (total)	m^3	3,072	

ITEM	UNIT	VALUE (1)	COMMENTS
Air Blowers			
Number of Blowers	#	3	Two duty/one standby, 22 kW each
Capacity	L/sec	213	Each (Rated at 70 kPa)
Diffusers			
Type	-	-	Fine bubble membrane diffusers
Total Number of Diffusers	-	1,452	726 each tank
Design Clean Water Transfer Efficiency	%	37.3	
Secondary Clarifiers			
Number of Systems	#	2	
Dimensions	m	15.15 x 4.85	Diameter x Depth (SWD)
Surface Area (Total)	m^2	360	Two units
Tertiary Filter			
Type	-	-	Parkson DynaSand® deep bed granular filters
Number of Filter Cells	#	4	Two modules per filter cell
Filtration Area (Total)	m^2	37.2	
Filtration Depth	m	2.4	
Media Grain Size	mm	1.4	
Uniformity Coefficient	-	1.6	
Filter Reject Pumping			
Number of Pumps	-	2	One duty, one stand-by
Type	-	Submersible	
Capacity	L/s	7.8	Each, at 10 TDH
Filter Drain Pumping			
Number of Pumps	-	2	One duty, one stand-by
Type	-	Submersible	
Capacity	L/s	5	Each, at 14.4 TDH
Chemical Feed: Phosphorus Removal			
Chemical	-	Alum (48%)	
Storage Capacity	m^3	20	
UV Disinfection			
Peak Flow Capacity	MLD	9.177	

ITEM	UNIT	VALUE (1)	COMMENTS
Number of Banks	#	2	Low-pressure, low intensity system
Number of Modules	#	12	
Number of Lamps	#	72	
Channel	mm	458 x 8,000	Width x Length
Total Channel Depth	mm	1,450	
Design UV Transmission	%	65	Minimum
Design Influent TSS	mg/L	30	30-day average
Sludge Thickening Tank			
Tank Dimensions	m	4.1 x 4.2 x 6.35	Length x Width x SWD
Total Tank Volume	m^3	109	
Emergency Sludge Loading Pump Capacity	L/s	25	At 12-meter TDH
Aerated Sludge Holding Tank			
Tank Dimensions	m	6.52 x 4.2 x 4.75	Length x Width x SWD
Total Tank Volume	m^3	130	
Diffuser Type	-	-	Course bubble diffuser
Sludge Loading Pump Capacity	L/s	-	At 12-meter TDH

Notes:
(1) Based on Certification of Approval Number No. 1506-9P4GR8

2.3 TREATMENT OBJECTIVES

According to the Ontario Ministry of the Environment, Conservation and Parks (MECP) facility classification under O.Reg 129/04, Licensing of Sewage Works Operators (made under the Ontario Water Resources Act, 1990), the Nobleton WRRF is classified as a Class III wastewater treatment facility. It is operated under Amended Environmental Compliance Approval (ECA) No. 8678-B38R26 issued September 20, 2018. The plant is required to meet monthly concentration limits for carbonaceous BOD_5 ($_CBOD_5$), TSS, TP, and total ammonia nitrogen (TAN), and monthly average loading limits for these parameters. Table 2-2 presents the existing ECA effluent objectives and limits for the Nobleton WRRF.

Table 2-2: Nobleton WRRF ECA Effluent Objectives and Limits (Certification of Approval Number No. 8678-B38R26)

PARAMETER	EFFLUENT OBJECTIVES (mg/L)	EFFLUENT LIMITS	
		MONTHLY AVERAGE CONCENTRATION (mg/L)	ANNUAL TOTAL EFFLUENT LOADING (kg/yr)
$CBOD_5$	5.0	10.0	-
TSS	7.0	10.0	-
TP	0.1	0.15	160
TAN	0.5 (May 1 – Oct 31) 2.0 (Nov 1 – Arp 30)	1.0 (May 1 – Oct 31) 3.0 (Nov 1 – Apr 30)	-
E. coli (1)	100 CFU/100 mL	200 CFU/100 mL	-
pH	6.5 – 8.5 inclusive	6.0 – 9.5 inclusive	-

Notes:
1. *Based on monthly geometric mean density.*

3 Design Basis for System Assessment

3.1 DESIGN BASIS USED IN 2007 DESIGN

The Nobleton WRRF was originally designed in 2007 to provide for a service population of 6,500 people. The overall design flows used in the 2007design are as follow (TSH Design Report, 2007):

- Average per capita sewage flow — 450 L/c/day
- Average daily design flow — 2,925 m^3/day (2.925 MLD)
- Peaking factor (Harmon Formula) — 3.14 (for 6,500 people)
- Peak design flow — 9,177 m^3/day (9.117 MLD)

The Nobleton WRRF was built to serve an un-serviced population and therefore, was designed based on assumed generation rates and population. As such, the typical raw sewage quality for residential areas were used as a design basis in the 2007 TSH Design Report and are summarized in Table 3-1.

Table 3-1: Raw Sewage Quality 2007 Design Basis (TSH Design Report, 2007)

PARAMETER	TSH DESIGN REPORT, 2007			VALUE USED IN DESIGN
	Loading Rate (g/c/d)	Average Day Loading(1) (kg/d)	Unit Loading (mg/L)	Unit Loading (mg/L)
BOD_5	75	488	167	200
TSS	90	585	200	250
TKN	15	98	33	40
TP	4	26	9	10

Note:
(1) *Average day loading values were calculated using a design population of 6,500.*

The Nobleton WRRF has been in operation for the past six (6) years (since 2012). With actual/historical flows and raw sewage quality data available, the following sections will establish design flows, raw sewage characteristics, and generation rates to be used in this report.

3.2 POPULATION IN SERVICE

The wastewater servicing population for the community of Nobleton is required to determine wastewater flow and mass generation rates to analyze the system capacity and performance of the existing wastewater system. The total population serviced by the Nobleton WRRF is determined with the following considerations and is summarized in Table 3-2.

- Prior to the construction of the wastewater conveyance system to the Nobleton WRRF, residential households used septic tanks. Over the years, the Region has awarded multiple contracts to transition the individual septic systems to lateral connections into the mainline sanitary servicing operation.

- In September 2018, Black & Veatch provided an estimate of the total amount of properties connected to the mainline sanitary servicing operation from December 2011 to March 2018, refer to Black & Veatch's technical memorandum to York Region, *Confirmation of Historical Wastewater Servicing Population*. The number of connections were used to estimate the total population served by the Nobleton WRRF.
- The population per household was estimated to be 3.1 persons/unit which is consistent with previous population studies (Hatch Mott MacDonald Nobleton WPCP Technical Memorandum Service Population Review and Capacity Assessment, 2015) and with 2016 Census Data (occupied and unoccupied units).

Table 3-2. 2014-2017 Nobleton WRRF Existing Servicing Population (Black & Veatch, 2008)

YEAR	TOTAL HOUSEHOLDS CONNECTED TO SANITARY SEWER	TOTAL POPULATION SERVICED BY NOBLETON WRRF
2014	943	2,923
2015	1,006	3,119
2016	1,175	3,643
2017	1,255	3,891

3.3 HISTORICAL WASTEWATER FLOWS AND GENERATION RATES

3.3.1 Available Flow Meters

Raw wastewater is conveyed through a 4.04 km forcemain (300 mm diameter) from the Janet Avenue PS to the Nobleton WRRF. Flows entering the Nobleton WRRF are measured by two (2) magnetic flow meters:

- Sewage Pump Station Discharge Flow Meter (RSP_FIT1)
- Plant Influent Flow Meter (RSHW_FIT1), installed on the forcemain prior to the Inlet Channel

Average day flow (ADF) and peak instantaneous flow (PIF) from 2014 to 2017 were assessed for both flow meters for comparison and are summarized in Table 3-3. The data suggest that the total average peaking factor from both flow meters have been consistent, with negligible differences.

Table 3-3: Average Flows and Peak Instantaneous Flows Comparison for the Flow Meters, MLD

	PUMP STATION FLOW METER			PLANT INFLUENT FLOW METER		
	AVERAGE DAY FLOW (ADF)	PEAK INSTANTANEOUS FLOW (PIF)	PEAK FACTOR	AVERAGE DAY FLOW (ADF)	PEAK INSTANTANEOUS FLOW (PIF)	PEAK FACTOR
2014	0.86	5.26	6.11	0.88	5.26	5.97
2015	0.98	7.33	7.50	0.99	7.32	7.39
2016	1.10	6.60	6.00	1.14	6.60	5.78
2017	1.38	8.34	6.04	1.45	8.83	6.09
Average	-	-	6.40	-	-	6.30

3.3.2 Historical Wastewater Flows and Generation Rates

The average day flow (ADF) and average dry weather flow (ADWF) for 2014 to 2017 are summarized in Table 3-4. The ADWF was calculated in accordance with the MOCP Design Guidelines (MOE, 2008). The guideline defines the dry weather period to be 5 dry days within an 8-day period (or more) without rain. The weather data were collected from online resources, and the above-mentioned criteria were applied to identify the ADWF from 2014 to 2017, as presented in Table 3-4. Using the population provided in Table 3-2, the average wastewater generation rates from 2014 to 2017 are summarized in Table 3-4.

Table 3-4: Summary of Historical Wastewater Generation Rates

YEAR	POPULATION IN SERVICE	AVERAGE DRY WEATHER FLOW (ADWF) BASED ON MOE DESIGN GUIDELINES		ANNUAL AVERAGE DAY FLOW (ADF)	
		Flow	Generation Rate	Flow	Generation Rate
2014	2,923	0.90 MLD	308 L/c/d	0.88 MLD	300 L/c/d
2015	3,119	0.95 MLD	306 L/c/d	0.99 MLD	318 L/c/d
2016	3,643	1.21 MLD	331 L/c/d	1.14 MLD	313 L/c/d
2017	3,891	1.41 MLD	364 L/c/d	1.45 MLD	374 L/c/d
	Average:		327 L/c/d		326 L/c/d

The data suggest that the annual average wastewater generation rates have been consistently below the design flow of 450 L/c/d and only averaged up to 326 L/c/d from 2014 to 2017. The highest annual average flow recorded is approximately 374 L/c/d in 2017, where higher flows were recorded due to high number of wet weather events in the summer of 2017.

3.3.3 Historical Water Demand

In order to determine the average residential wastewater generation rate, a review of the historical water demand was conducted. The Water System Capacity Optimization Study (Report 1A, 2018) identified that the total population serviced by the water system in 2016 was 5,520 and the average residential water demand is 14.1 L/sec (excluding employment and non- revenue water). This results in an average residential water demand of 220 L/sec.

As part of the Water and Wastewater Master Plan Update (York Region, 2009), 2005 wastewater flow data collected from the Region's treatment plants were compared to 2005 water billing data. It was estimated that on average, monthly water consumption accounted for 92 percent of the wastewater generated, but monthly values ranged from 74 percent in May (during spring when high infiltration is expected) to 137 percent in August during summer when outdoor water use is high). On this basis, it was assumed that 100 percent of the average water demand rate would be utilized to represent the average residential wastewater generation rate for this project.

3.3.4 Impact of Extraneous Flow

According to the numerous flow monitoring and investigations undertaken by the Region, high levels of groundwater infiltration and rainfall derived inflow and infiltration (RDII) have been reported in the system (Civica, Municipal Water Resources, 2016). Peak flow into the Nobleton WRRF has been associated with various wet weather events and I/I. On the basis of annual average day flow of 327 L/c/d and average residential wastewater generation rate (average water demand) of 220 L/c/d, the estimated ongoing extraneous flows (such as infiltration and inflow) under dry weather condition is approximately 107/c/d, as presented in Table 3-5.

Table 3-5: Summary of Estimated Ongoing Extraneous Flow During Dry Weather Conditions

ADWF	AVERAGE RESIDENTIAL WASTEWATER GENERATION RATE	ESTIMATED AVERAGE EXTRANEOUS FLOW UNDER DRY WEATHER
327 L/c/d	220 L/c/d	107 L/c/d *(33% of ADWF)*

3.4 FUTURE WASTEWATER FLOW PROJECTION

Based on the above assessment, the ADWF is approximately 327 L/c/d, with extraneous flow under dry weather condition estimated to be thirty-three percent of the total ADWF. In order to consider the impact of I/I on an annual basis, the 2017 data was used to determine the annual average wastewater generation into the Nobleton WRRF.

In 2017, higher flows were recorded due to high number of wet weather events experienced in the summer. Based on the 2017 data as per Table 3-4, it is recommended that 370 L/c/d be used as the basis to project future annual average day flow for a total service population of 10,800 people.

Under the assumption of 220 L/c/d of residential wastewater generation rate, the projected wastewater generation rate of 370 L/c/d will account for approximately 150 L/c/d of extraneous flow (approximately 40 percent). For the purpose of this capacity assessment, a value of 370 L/c/d is recommended for both the existing population and future growth. Using this value, the future average wastewater flow for a total population of 10,800 is 3,996 m^3/day (Table 3-6).

Table 3-6: Average Wastewater Flow Projection for 10,800 Population

NOBLETON DESIGN BASIS AVERAGE WASTEWATER GENERATION RATE	AVERAGE FLOW FOR 10,800 PEOPLE
370 L/c/d	3,996 m^3/d

3.5 HISTORICAL WASTEWATER FLOW PEAKING FACTORS

The historical flows into the Nobleton WRRF from January 2014 to December 2017 are used to determine the following flow variations which are summarized in Table 3-7.

Table 3-7: Summary of Historical Raw Sewage Flows and Peaking Factors into the Nobleton WRRF

YEAR	ADF	MMF(1) (PEAKING FACTOR)	PDF (PEAKING FACTOR)	PIF (PEAKING FACTOR)	PHF(2) (PEAKING FACTOR)
2014	0.88 MLD	1.20 MLD (1.4)	1.95 MLD (2.2)	5.26 MLD (6.0)	4.10 MLD (4.7)
2015	0.99 MLD	1.30 MLD (1.3)	1.78 MLD (1.8)	7.32 MLD (7.4)	4.10 MLD (4.1)
2016	1.14 MLD	1.77 MLD (1.6)	2.55 MLD (2.2)	6.60 ML D (5.8)	4.77 MLD (4.2)
2017	1.45 MLD	1.99 MLD (1.4)	3.89 MLD (2.7)	8.83 MLD (6.1)	8.60 MLD (5.9)
Average Peaking Factor		**1.4**	**2.2**	**6.3**	**4.7**

Notes;
Sources: SCADA Data: RSHW_FIT1
(1) Maximum Monthly Flow was determined using a 30-day moving average.
(2) Peak Hourly Flow based off the hourly average of the Peak Instantaneous Flow (5-min Flow), using a moving average of 12

3.5.1 Impacts of Return Flows

The internal recycle of supernatant and filter backwash have been assessed to determine their impact on the Nobleton WRRF unit processes. The additional flows due to internal recycling are summarized in Table 3-8.

Table 3-8: Impacts of Internal Recycle of Supernatant and Filter Backwash to the Influent Flows

PARAMETER	UNITES	HISTORICAL VALUES (2014 2017)
Supernatant Flows		
WAS Production	m^3/day	20
Average Weekly Hauled Sludge Volume	m^3	45
Average Daily Hauled Sludge Volume	m^3	6.5
Percentage of flow increase due to Supernatant Recycling	%	< 0.2 (ADF)
Filter Backwash		
Percentage of ADF increase Due to filter backwash wastewater	%	5-10
Percentage of PHF increase Due to filter backwash wastewater	%	1
Percentage of PIF increase Due to filter backwash wastewater	%	0.8

Based on the above analysis, the internal recycle of supernatant and filter backwash adds approximately 5-10 percent to the ADF into the Nobleton WRRF. Further breakdown of the impact of the filter backwash to the PFH and PIF into the Nobleton WRRF shows additional flow of approximately 1 percent, which is negligible. Therefore, the internal recycling of supernatant and filter backwash wastewater were not considered for peak capacity assessment.

3.6 INFLUENT MASS GENERATION RATES

3.6.1 Nobleton Historical Wastewater Influent Loads

The historical raw wastewater data from January 2014 to December 2017 were analyzed to determine the influent loads for BOD_5, TSS, TKN, and TP. Using the historical influent loads along with the service population presented in Table 3-2, the historical influent unit load factors into the Nobleton WRRF were calculated from 2014 to 2017. These values are summarized in Table 3-9, Table 3-10 and Table 3-11.

Table 3-9: Historical Influent Concentrations based on Nobleton's Historical Operational Data

PARAMETER	2014(1)	2015	2016	2017
BOD_5 (mg/L)	135	151	129	149
TSS (mg/L)	151	148	95	150
TKN (mg/L)	29	34	33	35
TP (mg/L)	3.7	4.2	4.0	4.5

Note:
(1) Samples from January 2014 to July 2014 were taken in the Inlet Works area. From July 2014, composite sampler was moved at the Pumping Station.
Sources: Data were based from the York Region/Durham Lab Data (Outside Lab Data)

Table 3-10: Historical Influent Loads Based on Nobleton's Historical Operational Data

PARAMETER	2014	2015	2016	2017
BOD_5 (kg/d)	113	144	138	201
TSS (kg/d)	128	146	104	206
TKN (kg/d)	24	33	36	48
TP (kg/d)	3.1	4.1	4.4	6.1

Table 3-11: Historical Influent Unit Load Rate in Nobleton WRRF

PARAMETER	2014	2015	2016	2017	AVERAGE
BOD_5 (g/c/d)	39	50	38	52	**45**
TSS (g/c/d)	44	47	29	53	**43**
TKN (g/c/d)	8	10	10	12	**10**
TP (g/c/d)	1.10	1.30	1.20	1.60	**1.3**

3.6.2 Impacts of Return Flows into Influent Loads

The internal recycle of supernatant and filter backwash have been assessed to determine their impact on influent load for TSS. In order to provide the most recent analysis, 2017 data has been used for this evaluation. The additional TSS loading due to internal recycling are summarized in Table 3-12.

Table 3-12: Impacts of Internal Recycle of Supernatant and Filter Backwash to the Unit Loads

PARAMETER	UNITS	HISTORICAL VALUES (2017)
Supernatant Flows		
WAS Production	m^3/day	20
WAS Concentration (1)	mg/L	8000
WAS Load	kg/day	160
Solids Capture Rate	%	90
Additional TS loading due to Supernatant Flows	kg/d	16
Influent TSS Load	kg/d	206
Percentage of TSS loading increase due to Supernatant Recycling	%	7%
Filter Backwash		
Secondary TSS Effluent (2)	mg/L	7.0
Final TSS Effluent (3)	mg/L	4.0
Additional TSS from Filter Backwash	mg/L	3.0
Filter Backwash Flow Rate (4)	MLD	0.73
Additional TSS loading due to Filter Backwash	kg/day	2.19
Historical TSS Load	kg/d	206
Percentage of TSS loading increase due to Filter Backwash	%	1%

Note:
(1) RAS /WAS concentration is only measured for 2014 and 2015. The 2015 value was used for 2017;
(2) Secondary TSS effluent is only measured for 2014 to 2015. The average value between 2014 and 2015 was used for the secondary TSS effluent
(3) An average effluent concentration of 6.68 mg/L was recorded in 2017, approximately 50% higher than concentration recorded in 2014 to 2017. Therefore, average between TSS effluent concentration between 2014 and 2017 was used instead.
(4) Based on typical values, assuming 5% of 2017 influent flow

Based on the above analysis, the internal recycle of supernatant and filter backwash adds approximately 5-10 percent to the TSS loading into the Nobleton WRRF.

3.6.3 Wastewater Influent Loads Recommendations

In order to establish the unit load factors for the future service population of the Nobleton WRRF, the following approach was used:

- Historical data were used to calculate the unit load factors for the existing service population of 3,891 people
- Typical literature values used to calculate the unit load factors for the future growth beyond 3,891 people up to 10,800 people. The typical literature values are summarized in Table 3-13.

The sum of the current and future unit load factors will be used to determine the overall load into the Nobleton WRRF. Summary of the influent unit load factors for BOD, TSS, TKN and TP to be used as the design basis for the Nobleton Optimization Study are summarized in Table 3-13.

Table 3-13: Nobleton Basis of Design Influent Unit Load Factors

PARAMETER	EXISTING SERVICE POPULATION BASED ON HISTORICAL DATA	PROPOSED GROWTH BASED ON GUIDELINES (MOE AND METCALF AND EDDY)
Population	**3,891**	**6,909 (10,800 – 3,891)**
BOD_5 (g/c/d)	45	75 (1)
TSS (g/c/d)	43	90 (1)
TKN (g/c/d)	10	13.3 (2)
TP (g/c/d)	1.3	4 (3)

Notes:
(1) BOD_5 and TSS values were based on the 2008 MOE Design Guidelines Sewage Works
(2) TKN and TP values were based on Metcalf and Eddy, 2003
(3) Value used in the original design criteria for the Nobleton WRR (TSH Design Report, 2007)

3.6.4 Load Peaking Factors

Maximum month influent loads are needed for various aspects of plant process assessment. The maximum month mass load peaking factors for BOD_5, TSS, TKN, and TP are shown in Table 3-14. The detailed data assessment is included in Appendix A. These unitless factors were based on historical operational data from the Nobleton WRRF.

Table 3-14: Design Influent Load Monthly Peaking Factors

PARAMETER	NOBLETON HISTORICAL PEAKING FACTOR				NOBLETON DESIGN BASIS
	2014	2015	2016	2017	
BOD_5	1.4	1.2	1.4	1.4	**1.4**
TSS	1.4	1.2	1.4	1.3	**1.3**
TKN	1.1	1.1	1.1	1.2	**1.1**
TP	1.1	1.2	1.3	1.2	**1.2**

3.6.5 Temperature

Temperature is an important factor affecting biomass activity, which is important in maintaining efficient biological wastewater treatment. The wastewater temperature was reviewed from January 2014 to December 2017 and suggests that the wastewater temperature at the Nobleton WRRF have been consistent over the past four (4) years, with an annual average temperature ranging from 15 degrees Celcius to 17 degrees Celcius. Using the influent temperature data, the minimum and maximum month, annual average and minimum and maximum daily was determined to develop a design basis to be used for alternative option development at a later stage of the EA. The proposed design influent temperature is presented in Table 3-15.

Table 3-15: Nobleton Design Influent Temperature

PARAMETER	NOBLETON HISTORICAL DATA				DESIGN TEMPERATURE
	2014	2015	2016	2017	
Minimum Temperature, ºC	6	8	11	8	**8**
Minimum Month Temperature, ºC	8	10	13	15	**12**
Annual Average Temperature, ºC	15	15	17	16	**16**
Maximum Month Temperature, ºC	19	20	20	20	**20**
Maximum Day Temperature, ºC	20	21	21	21	**21**

3.7 SUMMARY OF DESIGN BASIS

3.7.1 Design Flow

The Nobleton WRRF is anticipated to support a population growth of up to 10,800 people in 2041. Based on the assessment of historical data, Table 3-16 presents the annual average day flow and peaking factors that will be used as design criteria to evaluate the existing system capacity and identify optimization opportunities for the Nobleton WRRF.

Table 3-16: Nobleton WRRF Design Flow for a Future Service Population of 10,800 people

DESIGN FLOW CRITERIA	BASELINE (2017)	FUTURE
Residential Population	3,891	10,800
Wastewater Generation Rate	370 L/c/d	370 L/c/d
Average Day Flow Capacity	2,438 m^3/day	3,966 m^3/day
Peaking Factors		
Maximum Month Flow (MMF)	1.4	1.4
Peak Day Flow (PDF)	2.2	2.2
Peak Hour Flow (PHF)	4.7	4.7
Peak Instantaneous Flow (PIF)	6.3	6.3

3.7.2 Unit Load Factors

The unit load factors for the overall future service population of 10,800 people will be based on the sum of the current and future unit load factors presented in Table 3-17.

Table 3-17: Nobleton WRRF Design Unit Load Factors for Future Service Population of 10,800 people

PARAMETER	BASELINE (3,891 ppl)		GROWTH (6,909 ppl)		MAXIMUM MONTH PEAK FACTORS
	Loading Rate (g/c/d)	Average Day Loading (kg/d)	Loading Rate (g/c/d)	Average Day Loading (kg/d)	
BOD	45	175	75	518	1.4
TSS	43	167	90	622	1.3
TKN	10	39	13.3	92	1.1
TP	1.3	5	4	28	1.2

4 Existing Conveyance System Assessment

4.1 SEWER NETWORK

4.1.1 Historical Flow

Supervisory control and data acquisition (SCADA) data are available for the Janet Avenue PS to evaluate the flows pumped to the Nobleton WRRF. Table 4-1 shows the variation in the total volumes that were pumped to the Nobleton WRRF for the period from 2014 to 2017. They indicated that average daily flows have increased every year, which is due to the increase of service population. The peak daily volume varies for each year, which is related to the size of the largest rainfall event that occurred that year, as shown on Figure 4-1.

Table 4-1 Historical Daily Pumped Volumes to Nobleton WRRF

YEAR	2014	2015	2016	2017
Average Daily Pumped Volume (m^3)	864	978	1,100	1,380
Maximum Daily Pumped Volume (m^3)	1,950	1,780	2,550	3,890
Maximum Pumped Volume Day	21/02/2014	28/06/2015	01/04/2016	05/05/2017
Minimum Daily Pumped Volume (m^3)	620	732	806	1,086
Minimum Pumped Volume Day	16/02/2014	06/03/2015	06/10/2016	01/01/2017

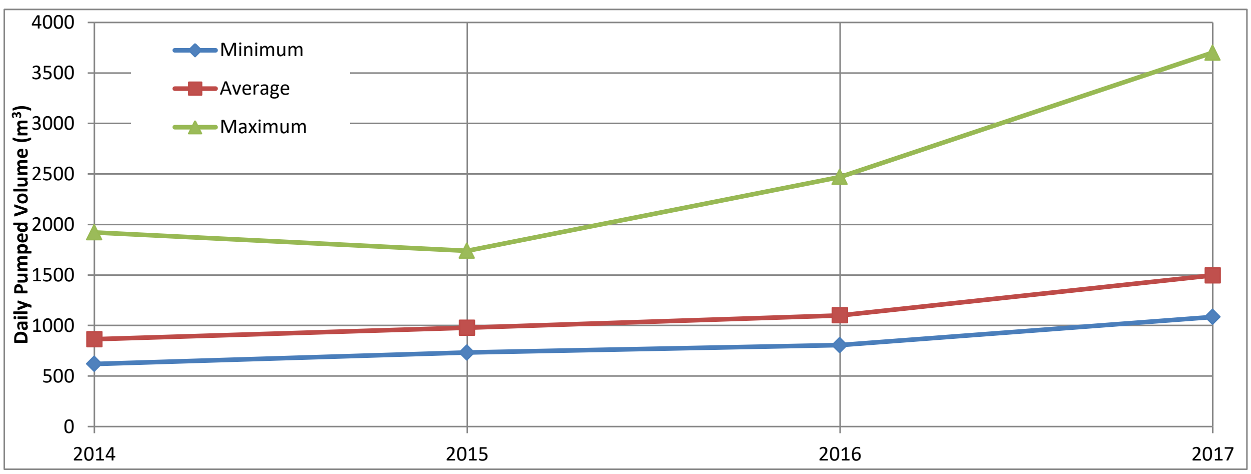

Figure 4-1: Historical Nobleton Minimum, Average, and Maximum Pumped Volumes

4.1.2 Collection System Modeling

The hydraulic model was calibrated against a series of flow monitors located around the catchment. The model was built in InfoWorks ICM version 6.5. These monitors have been in place since 2014. The model was calibrated against historical data for 2016 using the estimated population of 3,643. To achieve a calibrated model, the flows generated by the population are equal to 834 m^3 a day. To match the recorded flows at the various flow monitors, it has been necessary to add infiltration. In total, an additional 4.5 L/s (386 m^3/d) has been added across the catchment. This gives a total volume of 1,220 m^3/d arriving at the pumping station which is equal to 335 L/cap/d. This modeled value is approximately 3 percent higher than the calculated average dry weather flow of 327 L/c/d (see Table 3-4). This is an acceptable difference between modeling results and historical data.

During storm conditions, the amount of additional flow (e.g., from precipitation) which quickly enters the system is relatively small. However, there appears to be a contribution from slow infiltration into the sewer after the storm events, including:

- The one area that drains down Highway 27 has a large contribution. This area includes a small commercial area, and the modeling has shown that there is a requirement for an average of 15 m^2 of connected roof area per property to be added to match the observed flows seen from this area.
- The model has also shown that there is another slow response element to the flows, which has been included as rainfall-related groundwater infiltration. The largest contribution to the flows comes from the area along King Road.

Based on the hydraulic model of the sewer system, it is determined that most of the existing system has sufficient capacity to drain the current flows to the Janet Avenue PS. The analysis shows that there are some locations within the catchment where surcharging is predicted to occur within the network because of the insufficient capacity of the pipes, but no flooding is predicted. The main location where the surcharging occurs is around the Janet Avenue PS.

4.2 JANET AVENUE PUMPING STATION

At the Janet Avenue PS, telemetry data are available on the depth within the wet well and the flows in the rising main. The pumps are equipped with variable frequency drives (VFDs); and are operated between 20 L/s and 40 L/s during dry weather. This is likely because the pumps do not operate below a 40 percent turndown capacity. This leads to a large variability in the flows arriving at the Nobleton WRRF.

The hydraulic modeling results cannot match with the observed liquid depths in the wet well and the frequency of pump operation. From a review of the data, there appears to be some restriction on the inlet to the pumping station. Therefore, a restriction has been included in the model to reasonably match to the data; but there is still a discrepancy which cannot be rectified without additional information on the operation of the pumps. The inclusion of the restriction in the model causes the pipes upstream to surcharge during large storms. During dry weather flows, there is no effect on the levels upstream.

With two pumps operating at its full capacity, the total pumping rate will be 106 L/s or 9,158 m^3/d. At an observed peaking factor of 6.3, the Janet Avenue PS has an equivalent ADF capacity of 1,454 m^3/d and an equivalent serviceable population of 3,929 persons.

5 Nobleton Water Resource Recovery Facility

5.1 HEADWORKS BUILDING

The quantity of screenings and grit generated at the Nobleton WRRF is not measured, therefore, the performance of the existing screen and grit removal system could not be assessed as part of this study.

The mechanically cleaned bar screen is rated at a peak instantaneous flow capacity of 9,177 m^3/d. Based on the peak factor of 6.3 for the peak instantaneous flow and the revised wastewater generation rate of 370 L/c/d, the existing screening facility has an equivalent ADF capacity of 1,457 m^3/d and an equivalent serviceable population of approximately 3,937people.

The grit removal system consists of two (2) vortex grit chambers (1 duty, 1 standby) each with a rated capacity of 9,177 m^3/day. Based on the revised wastewater generation rate of 370 L/c/d and a historical average peaking factor of peak instantaneous flow of 6.3, the existing grit removal facility has an equivalent average day capacity of 1,457 m^3/d and a serviceable population of approximately 3,937 people.

5.2 AERATION TANKS

To date, only one of the two aeration tanks has been in service. To analyze the historical performance and operation of the aeration tanks within the Nobleton WRRF, various biological treatment operating parameters have been assessed from January 2014 to December 2017 and are summarized in Table 5-1. Historical values were compared with the typical operating parameters for a nitrifying extended activated sludge process based on the Design Guidelines for Sewage Works (MOE, 2008) and typical literature values.

Table 5-1: Biological Treatment System Historical Operating Parameters from 2014 to 2017

PARAMETER	UNIT	HISTORICAL OPERATION				TYPICAL VALUES	
		2014	2015	2016	2017	MOE (2008)	METCALF & EDDY (2003)
MLSS	mg/L	2,686	3,437	2,953	2,792	3,000 – 5,000	2,000 – 5,000
MLVSS	mg/L	1,792	2,322	2,173	2,066	-	-
MLVSS/MLSS	-	0.67	0.68	0.74	0.74	-	-
WAS (1)	kg/d	141	133	160	153	-	-
BOD Loading (Historical)	kg/d	113	144	138	201	-	-
VSS Yield	kg VSS/ kg BOD_5 removed	0.71	0.62	0.70	0.64	-	-

PARAMETER	UNIT	HISTORICAL OPERATION				TYPICAL VALUES	
		2014	2015	2016	2017	MOE (2008)	METCALF & EDDY (2003)
RAS Flow	MLD	0.96	1.16	1.30	1.38	-	-
Return Rate (% of Average Day Flow)	%	118	120	117	100	50 - 200	50 - 150
F/M_v	g BOD_5 / g MLVSS/d	0.04	0.05	0.04	0.06	0.05 – 0.15	0.04 – 0.10
Organic Loading Rate (OLR)	kg BOD_5/ m^3/d	0.064	0.093	0.102	0.190	0.17 – 0.24	0.1 – 0.3
Solids Retention Time (SRT)	days	29	40	28	28	>15	20 – 40
Hydraulic Retention Time (HRT)	hours	42	37	32	25	>15	20 - 30

Notes:
(1) WAS Mass calculated using the average RAS concentration of 8,000 mg/L due to limited WAS concentration data

A review of the biological treatment system historical operating parameters of Nobleton WRRF suggests that:

- The aeration tanks have been operating with a large range of Mixed Liquor Suspended Solid (MLSS) concentrations ranging from approximately 1,700 mg/L to 4,900 mg/L. A notable increase of MLSS concentration have been recorded in 2015, which can be associated with the high operating SRT (40 days) in 2015.
- Due to the lower strength wastewater being treated at the Nobleton WRRF, the historic OLR values to the aeration tanks were slightly below the typical MOE's Design Guideline values for EA facilities.
- The average Return Activated Sludge (RAS) flow rates have been gradually increasing over the past four (4) years. The recycle rate, as a percentage of the Average Day Flow (ADF), ranged from 118 percent to 120 percent, which is within the typical operational range.
- The food to microorganism (F/M_v) ratio ranged from 0.04 to 0.06 g BOD_5/g of MLVSS/d, which is at the lower end of the typical MECP Design Guidelines (2008) range of values for an EA process. Despite low F/M_v conditions in the aeration tank at the Nobleton WRRF, the historical sludge volume index (SVI) was below 100 mL/g, which is indicative of a good settling sludge.
- The operating SRT ranged from 28 to 40 days, which is significantly higher than the typical value of 15 days for an extended aeration system.
- The VSS yield ratio is 0.66 on average due to long operating SRTs.

The following assumptions were used to assess the aeration tank capacity:

- SRT of 15 days, as recommended by the MECP Design Guideline, will be used to assess the process capacity of the existing aeration tank capacity;
- The operating MLSS concentrations should be approximately 3,500 mg/L under the average day loading condition and below 5,000 mg/L under the maximum month loading conditions;
- A VSS yield of 0.8 kg VSS / kg BOD_5 removed for a 15-day SRT; and
- A measured historical average MLVSS/MLSS ratio of 0.70.

Based on the above assumptions, the process capacity of the existing aeration tanks is assessed and summarized in Table 5-2.

Table 5-2. Aeration Tank Capacity Assessment

LIMITING PARAMETER	ESTIMATED ADF CAPACITY
HRT > 15 hrs	4,915 m^3/d
OLR = 0.24 $kg/m^3/d$	4,042 m^3/d
SRT = 15 d with MLSS of 3,500-5,000 mg/L (1)	3,670 m^3/d
Estimated ADF Capacity	3,670 m^3/d
Estimated equivalent serviceable population (2)	9,919 people

Notes:
(1) Based on MLVSS/MLSS of 0.70 and a VSS yield of 0.80 kg VSS/kg BOD5
(2) Based on 45 g BOD/c/d for the current service population of 3,891 and 75 g BOD/c/d for future growth.
Detailed calculation is included in Appendix B.

Based on the above assessment, the existing two aeration tanks have an equivalent ADF capacity of 3,670 m^3/d and serviceable population of 9,919 people.

5.3 PROCESS AIR BLOWERS

There are three fixed speed blowers (2 duty 1 standby), each rated at 766 m^3/hr. Currently, one blower is typically in operation. There is no automated DO control available to automatically adjust the DO concentrations in the aeration basins. DO concentration was reviewed from January 2014 to December 2017 and is summarized in Table 5-3. The average DO concentration has been found to be in between 4.07 mg/L to 5.45 mg/L, which is significantly higher than the typical DO residual of 2.0 mg/L.

Table 5-3: Summary of DO Concentrations

PARAMETER	UNIT	AVERAGES			
		2014	2015	2016	2017
DO	mg/L	5.45	4.32	4.45	4.07

The following assumptions are used to assess the existing aeration system capacities:

- The aeration system capacity assessment is based on supplying oxygen for the removal of BOD and TKN under average daily loading for BOD and peak day loading for TKN.

- A DO residual of 2.0 mg/L should be maintained under the average and peak day loading conditions.
- DO residual can be below 2.0 mg/L under peak hour process demand conditions due to diurnal flow variations.
- Average summer temperature of approximately 20 degrees Celcius was used to assess blower capacity for conservative measures as summer operations requires higher air flows

Based on the above assumptions, the existing blower process capacity is calculated in Table 5-4.

Table 5-4: Air Requirements for the Existing Aeration Tank System

ITEM	UNIT	VALUE	COMMENTS
Blower Capacity			
Two duty blowers	m3/hr	1,532	Two units at their rated capacity (213 L/sec each)
SOTE	%	37	
Total SOR	kg/d	3,794	BOD Average Day Loading = 476.90 TKN Peak Day Loading = 166.37
AOR/SOR	-	0.37	Based on the assumed conditions: α = 0.5, β = 0.95, θ = 1.024 DO = 2 mg/L, Wastewater temperature = 20 °C Depth of Diffusers = 6.0 m, and Plant Elevation = 180 m
Total AOR	kg/d	1,404	
Process Capacity			
TKN Oxygen Demand	-	4.6	MECP Guideline
BOD Oxygen Demand	-	1.5	MECP Guideline
Maximum Service Population	pp	7,915	
Equivalent average day flow capacity	m^3/d	2,929	370 L/cap/d
Total AOR Needed	kg/d	1,404	

Based on the results shown in Table 5-4, the existing blowers have an equivalent ADF capacity of 2,929 m^3/d and serviceable population of 7,915 people.

5.4 SECONDARY CLARIFIERS

There are two 15.15 m diameter circular secondary clarifiers at the Nobleton WRRF. Currently, only one clarifier is in operation. The measured effluent TSS concentrations, based on the provided plant data from January 2014 to June 2017, were plotted in Figure 5-1. Based on the information provided, the current secondary clarifier can achieve effluent TSS concentration below 10 mg/L over 75 percent of the time. The secondary clarifier effluent TSS has been constantly below 20 mg/L, within the typical clarifier performance.

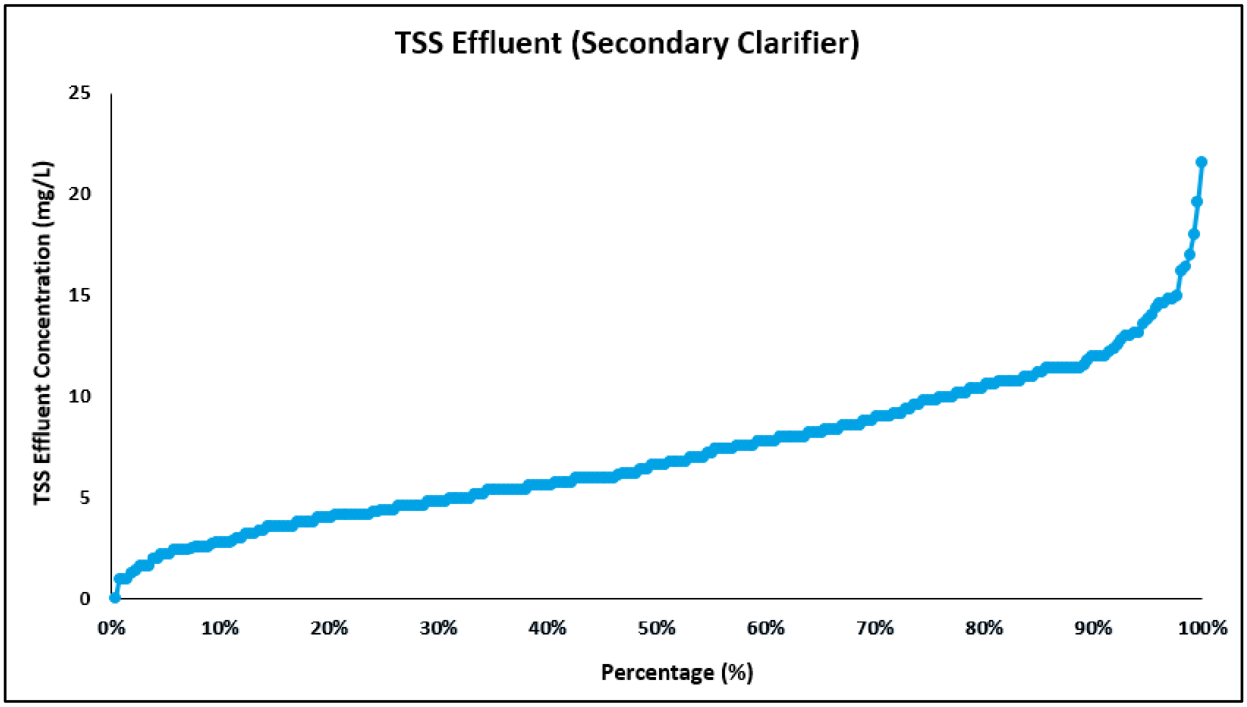

Figure 5-1: Secondary Effluent TSS Concentrations, 2014-2017

Table 5-5 provides a summary of the secondary clarifier operating parameters compared to the typical literature values. Based on the results presented in Table 5-5, historically the peak day SLR have been well below the MECP's Guideline values. For the peak hourly SOR, in 2017 where a much higher flow was experienced in comparison to previous years, the SOR was found to be 48 $m^3/m^2/d$. However, the performance of the secondary clarifier was still acceptable under the peak SOR of 48 $m^3/m^2/d$. Note that this only happened during a rare event in 2017.

Table 5-5: Nobleton Secondary Clarifier Historical Operation

PARAMETER	2014	2015	2016	2017	MECP GUIDELINES	METCALF & EDDY (MCGRAW HILL, 2013)
PHF (MLD)	4.10	4.10	4.77	8.60	-	-
PDF(MLD)	1.95	1.78	2.55	3.89	-	-
Peak SOR ($m^3/m^2/d$)	23	23	27	48	<37	40 to 64
Peak SLR ($kg/m^2/d$)	44	56	63	82	<170	100 to 240

Notes:
(1) Based on a surface area of 180 m^2 with one clarifier in operation
(2) SOR based on Peak Hourly Flow rate
(3) SLR based on Peak Day Flow

The capacity assessment of the existing secondary clarifiers is presented in Table 5-6.

Table 5-6: Existing Secondary Clarifiers Capacity Assessment

LIMITING PARAMETER	PDF CAPACITY	PHF CAPACITY
Peak hourly SOR < 37 $m^3/m^2/d$		13,333 m^3/d
Peak daily SLR < 170 $kg/m^2/d$	8,423[(1)] m^3/d	

Note:
[(1)] Based on MLSS of 5,000 mg/L under maximum month loading and RAS: ADF of 1.0

Based on the preliminary assessment shown in Table 5-5, the existing secondary clarifiers have a PHF capacity of 13,333 m^3/d and PDF capacity of 8,423 m^3/d. The PDF capacity is estimated based on a MLSS concentration of 5,000 mg/L under maximum month loading condition and a RAS:ADF ratio of 1.0.

With the current peaking factors of 2.2 and 4.7 for the PDF and PHF, respectively, the existing secondary clarifiers have an equivalent ADF capacity of 2,837 m^3/d and an equivalent serviceable population of 7,667 people.

5.5 PHOSPHOROUS REMOVAL

5.5.1 Chemical Addition

Alum is currently added for phosphorous removal upstream of the secondary clarifiers and upstream of the tertiary filters. The alum addition is flow paced. Historically, from 2014 to 2017, the average effluent TP concentration is below 0.1 mg/L (monthly average objective), indicating good phosphorous removal through chemical precipitation and tertiary filtration. Table 5-7 summarizes the historical alum dosages for phosphorous removal. It should be noted that there are no separate data for alum dosages upstream of the secondary clarifiers and upstream of the tertiary filters. The values presented in Table 5-7 are based on average daily dosages.

Hatch Mott MacDonald calculated the theoretical alum dosing rate of 163 mg alum /L to achieve an average effluent TP concentration of 0.1 mg/L (2015) based on MOP No. 37 (WEF, 2013). Black & Veatch collected and analyzed published literature data for chemical phosphorous removal with metal salts (iron and aluminum based) in municipal WWTPs. The molar ratio between metal and aluminum needs to be above 5 in order to achieve an effluent TP concentration of 0.1 mg/L or below.

Based on literature review and Black & Veatch's experience, the current average alum dosing rate is sufficient to achieve the monthly average objective of 0.1 mg TP/L in effluent; and the current dosing rate could be potentially optimized to reduce consumption. However, the chemical dosing locations and their impact on chemical reaction cannot be assessed based on chemical dosing rates. They will be discussed in detail in Section 5.5.2.

Table 5-7. Historical Alum Dosage

PARAMETER	2014	2015	2016	2017
Alum solution dose, mg alum /L	246	210	219	277
Aluminum dose, mg Al/L	13.8	12.3	10.9	12.1
Influent TP, mg/L	3.7	4.2	4.0	4.5
Molar Ratio of Al : TP	7.0	6.7	5.8	6.6

5.5.2 Tertiary Filtration

The efficacy of filtration is dependent on the degree of chemical flocculation achieved upstream of the filters. Tertiary filtration is provided by four upflow Parkson deep bed granular filters. Historically, from 2014 to 2017, two filters have been continuously in operation. Effluent samples are collected once per week and analyzed by an external accredited laboratory. This section of the report is based on the external laboratory testing results.

Historical final monthly average TSS effluent concentrations measured by the outside laboratory, from January 2014 to December 2017, are plotted on Figure 5-2. The graph indicates higher monthly average TSS concentrations in 2017 as compared with 2014 to 2016. In 2017, the final effluent TSS often exceeded the monthly treatment objective with the exception of August, September, and December. In addition, the monthly TSS limit was also exceeded in February 2017.

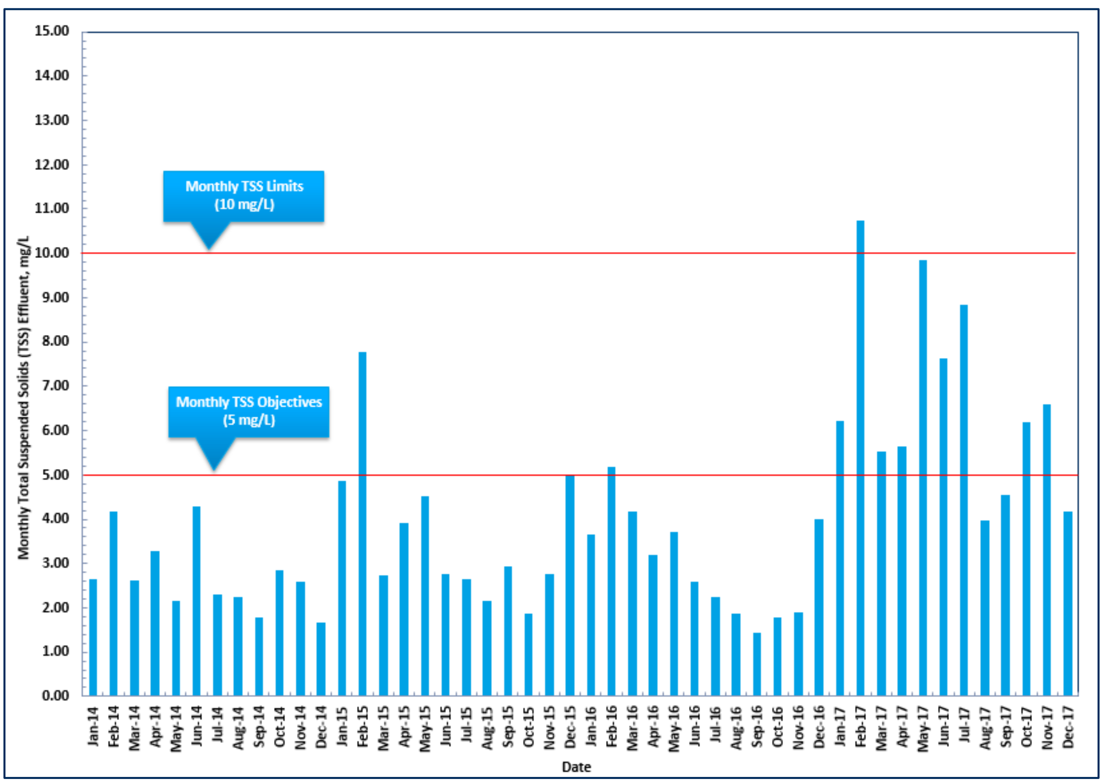

Figure 5-2: Final TSS Effluent Concentration

Historical final monthly average TP effluent concentrations from January 2014 to December 2017 have been plotted on Figure 5-3. Based on the concentrations measured by the outside laboratory, the monthly TP objectives of 0.10 mg/L was exceeded twice during 2015, in the months of February and May. The highest exceedances recorded was in 2017 where the monthly TP objective was exceeded six times. In addition, in 2017, the monthly TP limit was also exceeded twice in February and May. High TP in tertiary effluents experienced in 2017 is in line with the observations made for the 2017 effluent TSS concentrations.

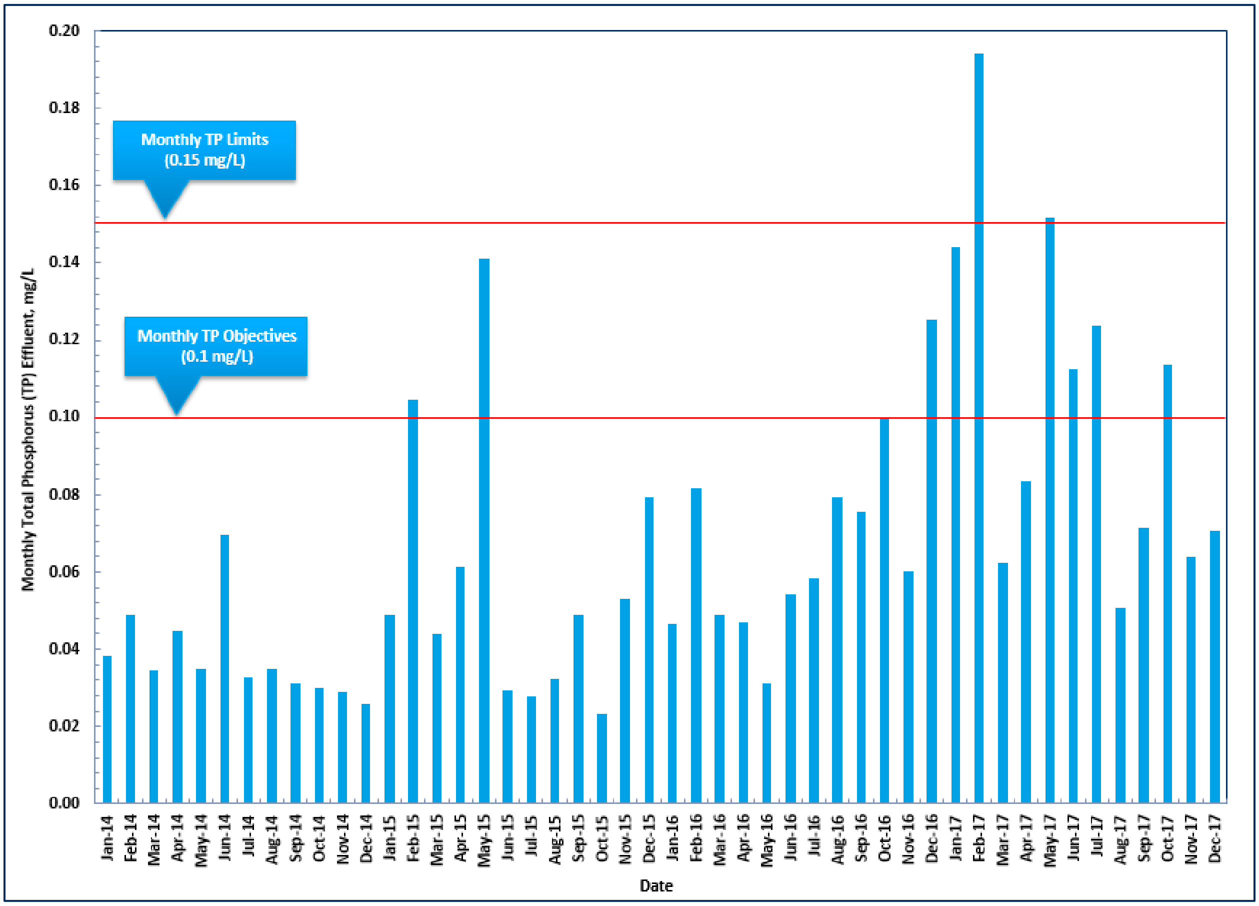

Figure 5-3: Final TP Effluent Concentrations

The following areas were investigated to identify potential causes for the exceedances of monthly TP objectives:

- **Tertiary Effluent Phosphorous**: the soluble and total tertiary effluent phosphorous concentrations were reviewed, as shown in Figure 5-4. It shows that both soluble and total phosphorous in the tertiary filter effluent increased since October 2015. This suggests that secondary effluent conditioning with alum might not be sufficient to convert soluble phosphorous to the particulate form.

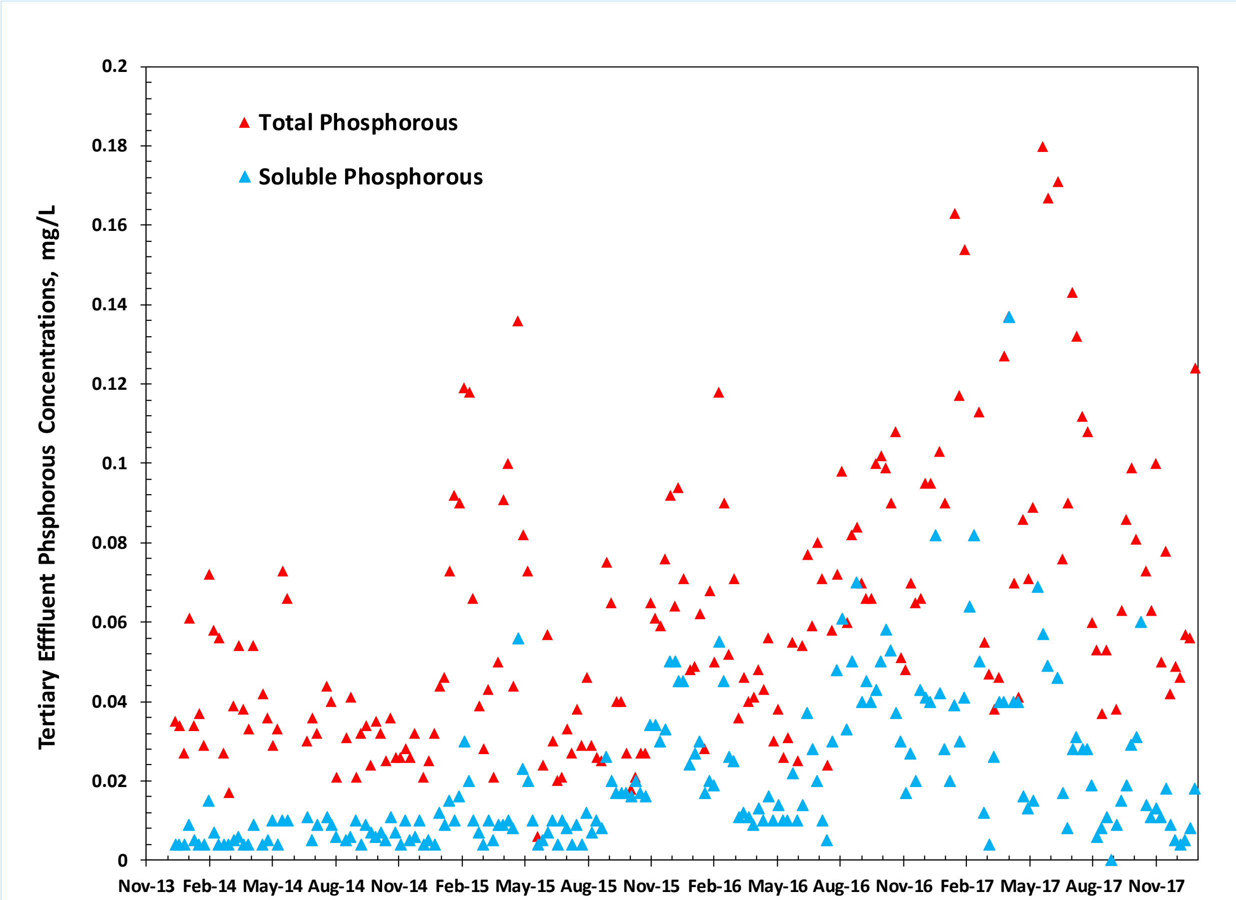

Figure 5-4. Tertiary Effluent Phosphorous Concentrations

- **Peak Hourly Hydraulic Loading Rate**: For a deep bed filter, MECP guideline (2008) suggests that the peak hourly hydraulic loading rate (including recycled flows) should not exceed 3.3 L/(m^2.s). Only two filters were in operation during the period of 2014 – 2017, with a total surface area of 18.6 m^2. For a peak hourly hydraulic loading rate of 3.3 L/(m^2.s), the peak hourly hydraulic flowrate into the filters (with two filters in operation) would be approximately 5.3 MLD. Based on historical data, hourly flow rate into the filters greater than 5.3 MLD occurred once in May 2017, twice in June 2017, and three times in July 2017. These high hourly flowrates corresponded with the exceedance of effluent TP objective from the filters. Because effluent samples were taken once per week, one sample with high effluent TP could cause exceedance for the monthly average concentrations.

- **Peak Solids Loading Rate**: The MCEP Guideline (2008) suggests that the peak solids loading rate should not exceed 83 mg/(m^2.s) for deep bed sand filters. With two sand filters in operation (18.6 m^2 in surface area), the peak solids loading onto the tertiary filters should be less than 133 kg/d. This value is significantly higher than the secondary clarifier effluent TSS loading.

- **Chemical Addition**: There is no separate chemical dosing values into the filters; Figure 5-5 shows the overall alum dosing rate from 2014 to 2017. It shows that chemical dosing rates (either in molar ratio or dosing concentrations) have been consistent from 2014 to 2017. Because the filter effluent in 2017 had increased soluble phosphorous, the chemical dosing rates upstream of the filters should be monitored to confirm if sufficient chemicals were added upstream of the filters.

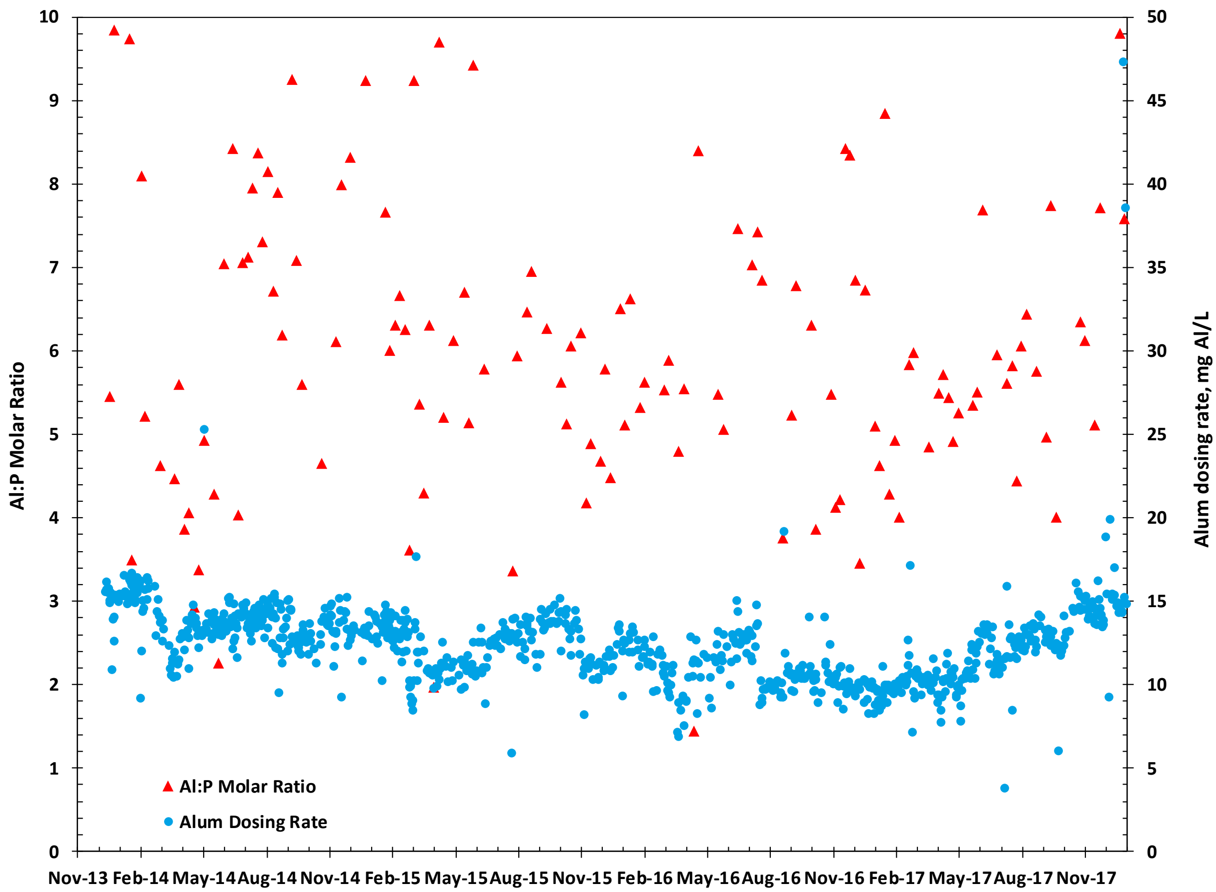

Figure 5-5. Chemical Dosing for Phosphorous Removal

- **Secondary Effluent TSS**: The secondary effluent TSS concentrations were reviewed and plotted on Figure 5-6. It shows that higher secondary effluent TSS concentrations resulted in higher tertiary effluent TSS concentrations. This observation is consistent with the typical filtration performance. Figure 5-6 also shows that secondary effluent TSS concentrations increased from 2014 to 2017. The MLSS concentrations were stable between 2014 and 2017; and the sludge volume index (SVI) has been consistently below 100. Therefore, the increased secondary effluent TSS could be potentially caused by the raw sewage flow increase, particularly in 2017. Based on Table 5-5, the secondary clarifier experienced high peak flows in 2017. This could cause high TSS concentrations in the secondary clarifier effluent

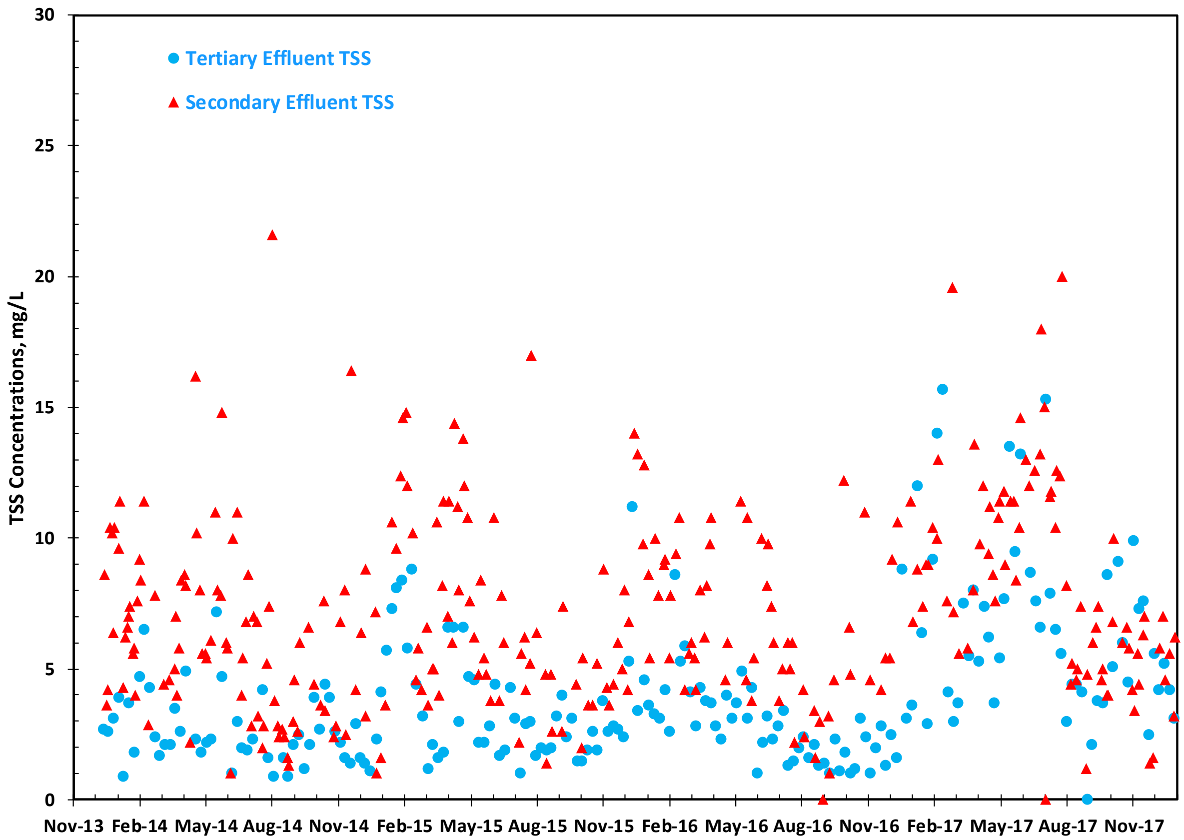

Figure 5-6. Secondary and Tertiary Effluent TSS Concentrations

- **Secondary Effluent Phosphorous**: Secondary effluent phosphorous concentrations were reviewed and plotted on Figure 5-7. It shows that high phosphorous concentrations (soluble and total) corresponded with the secondary effluent TSS concentrations (Figure 5-6). It also shows that secondary effluent has increased soluble phosphorous in 2017, although alum dosing in 2017 was comparable with 2014-2016. This implies potential issues with chemical precipitation for phosphorous in the secondary treatment system, including: i) insufficient mixing between soluble phosphorous and alum in secondary treatment system; ii) insufficient chemical reaction time. Note these two potential causes require field tests for confirmation.

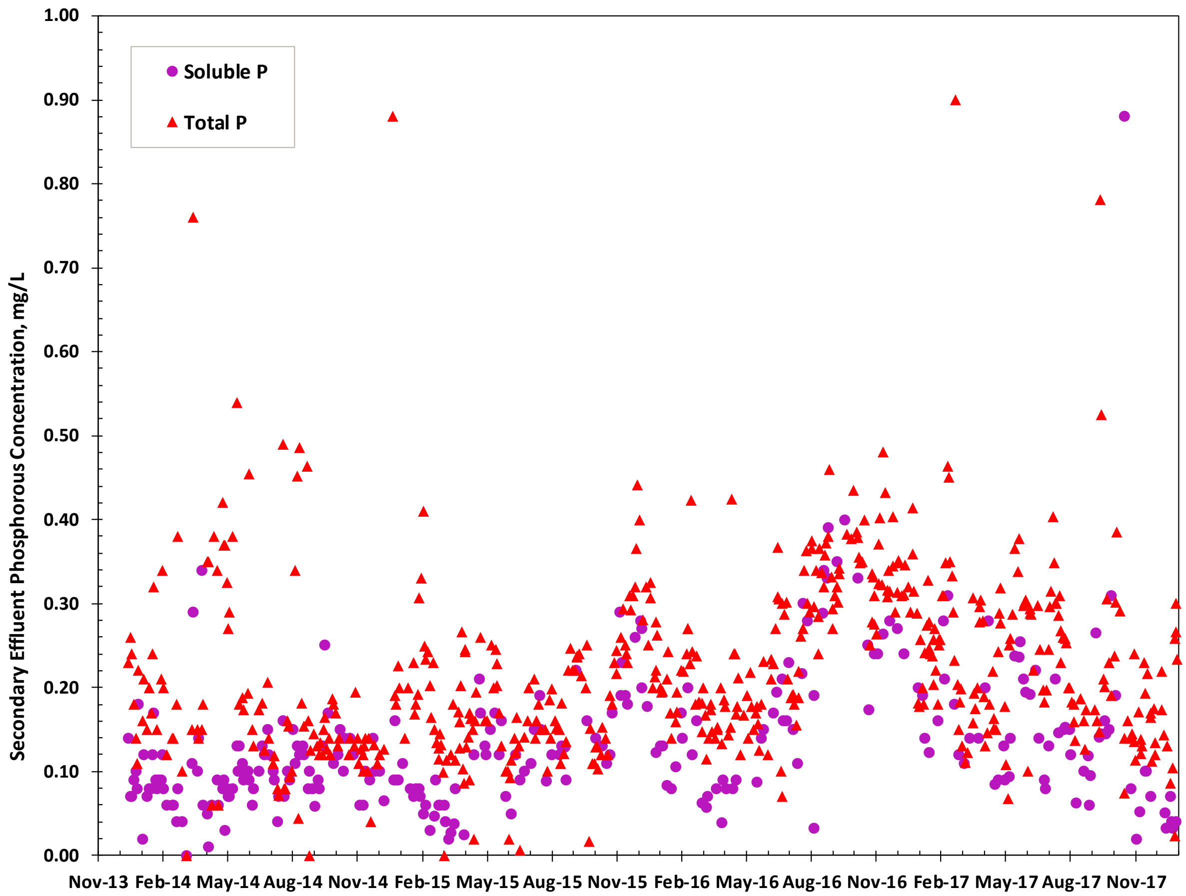

Figure 5-7. Secondary Effluent Phosphorous Concentrations

Based on the above assessment, the final effluent TP concentrations greater than the compliance and objective limits could be caused by:

- Insufficient chemical precipitation in the secondary treatment system, which caused high soluble phosphorous into the tertiary filters;
- High secondary effluent TSS concentrations which caused elevated effluent solids and total phosphorous into the tertiary filtration facility; and
- Potential insufficient chemical addition or reaction upstream of the filters to convert soluble phosphorous into the particulate form.

The capacity of a tertiary filter depends on both its hydraulic capacity and performance capacity, as summarized below:

- Hydraulic Capacity:
  - The hydraulic capacity of a deep bed granular filtration unit is determined using peak hour hydraulic loading rate of 3.3 L/s/m^2. Using this design criteria, it is estimated that the existing four filters have a PHF capacity of 10,490 m^3/d with all four units online. This PHF capacity

does not provide redundancy. If one unit can be out of service as standby, the PHF capacity of the existing filters would be 7,278 m^3/d.

- Because the 2007 design concept did not include a standby unit for the filters, it is assumed that all four units would be in service to provide treatment capacity. Therefore, the equivalent ADF capacity of the existing filters is 2,232 m^3/d; and the equivalent serviceable population would be 6,032 people.

- Performance Capacity:
  - During the development of this report, the assimilative capacity study for the Humber River is yet to be completed. For capacity assessment, the receiving water (Humber River) is considered as a Policy 2 receiving water body in this report. Therefore, the future TP loading into the Humber River could be the same as the current ECA requirement of 160 kg-P/year.
  - Two Region's WRRFs that use tertiary filtration to achieve low TP are summarized in Table 5-8. These facilities have comparable rated capacity to the Nobleton WRRF; and their effluent monthly TP limit is 0.1 mg/L. Based on the historical operating data, these two facilities met their compliance limit of 0.1 mg/L.
  - It is assumed that the existing deep bed filters can meet the limit of 0.1 mg/L (monthly average).

Table 5-8. Region's WRRF with Tertiary Filtration to Achieve Low TP

FACILITY	CAPACITY	TP REMOVAL PROCESS	EFFLUENT TP PERMIT	AVERAGE EFFLUENT TP CONCENTRATION	RANGE OF MONTHLY AVERAGE TP CONCENTRATION
Schomberg WRRF	2.1 MLD	Chemical addition, filtration	0.10 mg/L	0.04 mg/L	0.02 to 0.10 mg/L
Mount Albert WRRF	2.04 MLD	Chemical addition, filtration	0.10 mg/L	0.06 mg/L	0.02 to 0.10 mg/L

- Based on the above assessment, the capacity of the existing tertiary filters is governed by the peak hourly hydraulic loading onto the filters. The existing filters have an equivalent ADF of 2,232 m^3/d and an equivalent serviceable population of 6,032 people.

5.6 UV DISINFECTION

The historical data over the past 3 years indicates that the Nobleton WRRF plant has consistently met both the monthly effluent limit and effluent objective of 200 counts/100 mL and 100 counts/100 mL, respectively. It was identified that the maximum flow that can be conveyed through the UV system and meet the disinfection requirements is 9,842 m^3/d (Hatch Mott MacDonald, 2015).

For the purposes of the capacity assessment, the capacity of the UV disinfection system under the peak hourly flow is 9,842 m^3/d at a design UV transmittance (UVT) of 65 percent. Using the peak hourly flow peaking factor of 4.7, the average day capacity is approximately 2,094 m^3/d or an equivalent serviceable population of 5,660 people.

5.7 SLUDGE MANAGEMENT

Table 5-9 presents a summary of the historical average sludge production at the Nobleton WRRF based on 2014 to 2017 data.

Table 5-9. Sludge Production (2014-2017)

PARAMETER	UNIT	2014	2015	2016	2017
Waste Activated Sludge					
WAS Production	m^3/d	15	16	20	20
Total TS (1)	mg/L	6,000	8,000	8,000	8,000
Daily Solids Production	kg/d	90	128	160	160
Solids Production/Wastewater	g TS/m^3	105	130	140	110
Hauled Thickened Sludge					
TWAS Production	m^3/d	6.0	6.6	6.5	8.7
Total TS% (2)	%	3.0	3.0	2.4	2.0
Daily Solids Production	kg/d	120	132	130	174
Solids Production/Wastewater	kg TS/m^3	140	133	114	120

Note:
(1) RAS /WAS concentration is only measured for 2014 and 2015. The 2015 value was used for 2016 and 2017;

Based on the historical data, the sludge production at the Nobleton WRRF is between 100 and 140 g TS/m^3 wastewater treated. This is comparable with the MECP Guideline value of 120 g TS/m^3 for an extended aeration plant with chemical addition for phosphorous removal.

The sludge thickening tank has a surface area of 17.22 m^2. The operating solids loading rates between 2014 and 2017 were between 5.2 $kg/m^2/d$ and 9.3 $kg/m^2/d$. This operating range is well within the MECP Design Guideline of 12 to 36 kg/m2/d (2008). Using 36 $kg/m^2/d$ for the maximum month operating loading conditions, the maximum month WAS mass production would be approximately 620 kg/d. With a maximum month peaking factor of 1.4 and an operating SRT of 15 days, the estimated ADF capacity is 2,873 m^3/d and can provide services to 6,722 persons.

The sludge storage tank has a total volume of 130 m^3, which can provide over 10 days of thickened WAS storage for the current operation. Currently, the thickened WAS is hauled offsite by approximately one truck per week.

To assess the capacity of the existing sludge handling facility, it is assumed that a minimum of 3-day storage (considering long-weekends operation and weather-related events) should be provided for the thickened WAS in the sludge holding tank. With the proposed 10,800 people, the projected total TWAS will be approximately 44 m^3/d under the maximum month loading condition. The existing sludge holding tank can provide 3-day storage. It is expected that one truck of haulage per day will be required for the proposed population of 10,800 persons.

5.8 EFFLUENT CHAMBER AND OUTFALL

The final effluent from the Nobleton WRRF is conveyed into an effluent storage tank located in the lower level of the process building. The effluent overflows a weir into the effluent storage tank into a final effluent chamber where the effluent is discharged to the Humber River via 1.5 kilometers of 450 mm sewer concrete pipe along 11th Concession and through a constructed wetland. A desk top hydraulic assessment was conducted to evaluate the peak hydraulic capacity of the effluent discharge chamber and the existing outfall. Based on the Hydraulic Grade Line (HGL) in the as-built drawings (2012), the liquid elevation in the effluent chamber is 242.94 m. The outfall hydraulic capacity was estimated to maintain a liquid level of 242.94 m in the effluent chamber as shown in the Hydraulic Grade Line (HGL) of the as-build drawings. The key findings are summarized below:

- The outfall is mostly hydraulically steep, runs with a free surface, and is generally supercritical or close to supercritical.
  The water level in the effluent chamber is therefore largely independent of the outfall hydraulics, and depends mainly on the inlet arrangement at the upstream of the effluent chamber.
- At a liquid elevation of 242.94 m as shown in the Hydraulic Grade Line (HGL) of the as-built drawings, the maximum hydraulic capacity into the outfall is approximately 9,200 m^3/d.

The maximum liquid level can be increased in the effluent chamber to gain additional hydraulic capacity from the effluent chamber to the outfall, as summarized in Table 5-10.

Table 5-10. Effluent Chamber and Outfall Peak Hydraulic Capacity

LIQUID LEVEL IN EFFLUENT CHAMBER	PEAK OUTFALL CAPACITY	EQUIVALENT ADF
242.94 m (1)	9,200 m^3/d	1,460 m^3/d
243.38 m (2)	10,500 m^3/d	1,667 m^3/d
243.60 m (1)	12,000 m^3/d	1,905 m^3/d
243.98 m (submerging effluent weir) (1)	15,000 m^3/d	2,381 m^3/d

Notes:
(1) Black & Veatch's calculation
(2) Hatch Mott Macdonald, 2015, Nobleton WPCP Service Population Review and Capacity Assessment

For the purpose this capacity assessment, the peak flow capacity of the effluent chamber and outfall is approximately 10,500 m^3/d, which provides an equivalent ADF capacity of 1,667 m^3/d and an equivalent serviceable population of 4,505 people. This would operate the effluent chamber at a maximum water level of 243.38 m downstream of the weir under the peak flow of 10,500 m^3/d, resulting in 0.22 m below the weir.

5.9 HISTORICAL PROCESS REVIEW AND CAPACITY ASSESSMENT SUMMARY

5.9.1 Summary of Process and Other Related Performance Issues

Based on the information available, the following have been identified as having a potential impact on plant operation:

- The Nobleton WRRF experiences high PHF and PIF, with an average peaking factor of 4.3 and 6.3, respectively. These peaking factors are significantly higher than peaking factor of 3.14 used in

2007 design. As a result, the capacities of some process units are less than the currently rated capacity of 2,925 m^3/d, including:

- Screening facility
- Grit removal facility
- Secondary clarifiers
- Tertiary filtration
- UV disinfection
- Sludge thickener
- Sludge storage tank
- Effluent chamber and outfall

- The existing aeration system does not have an automatic control for DO adjustment. As a result, the average DO residual is significantly higher than 2.0 mg/L.
- The existing blowers have the ADF capacity of 2,929 m^3/d, which is just within the rated capacity of the existing Nobleton WRRF.
- The existing filters would need to meet a more stringent effluent TP limit if the Nobleton WRRF receives flows beyond the current ECA rated ADF capacity. Stress testing is recommended for confirmation.
- The hydraulic and process capacity of the UV disinfection system is based on a UVT value of 65 percent. The current filter effluent UVT is unknown and the actual capacity of the UV disinfection system is recommended to be confirmed through stress testing.
- The desk top hydraulic assessment of the outfall confirms that the outfall is mostly hydraulically steep, runs with a free surface, and is generally supercritical or close to supercritical. The hydraulic bottle neck for effluent discharge is within the effluent chamber and its inlet arrangement.

5.9.2 Capacity Assessment Summary

The summary of the results of the capacity assessment for unit process for the Nobleton WRRF is summarized in Table 5-11.

Table 5-11: Summary of the Capacity Assessment for Nobleton WRRF

TREATMENT UNIT	EXISTING SYSTEM CAPACITY ASSESSMENT		
	AVERAGE DAY FLOW	PEAK DAY FLOW	PEAK FLOW
Screens			9,177 m^3/d
Grit Removal			9,177 m^3/d
Aeration Tanks	3,670 m^3/d		
Secondary Clarifiers		8,423 m^3/d	13,333 m^3/d
Aeration System	2,929 m^3/d		

TREATMENT UNIT	EXISTING SYSTEM CAPACITY ASSESSMENT		
	AVERAGE DAY FLOW	PEAK DAY FLOW	PEAK FLOW
Tertiary Filtration			10,490 m^3/d
UV Disinfection			9,842 m^3/d
Gravity Thickener	2,873 m^3/d		
Sludge Storage Tank	3,996 m^3/d		
Effluent Chamber and Outfall			10,500 m^3/d

Figure 5-8 summarizes the capacity of the key unit processes on the equivalent ADF and serviceable population basis. The equivalent ADF capacity was based on the peaking factors identified with the historical data.

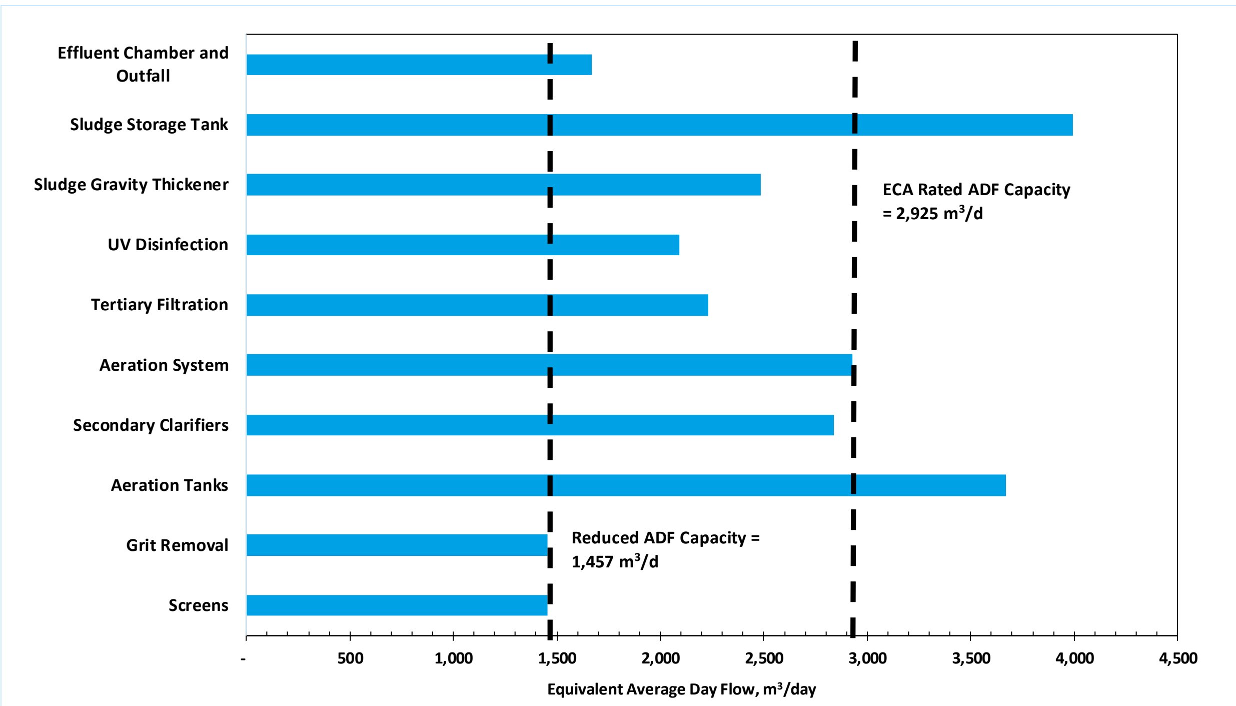

Figure 5-8: Unit Process Equivalent ADF Capacities and Serviceable Population

Based on the above summary, the following conclusions can be drawn:

- The Nobleton WRRF has an ADF capacity of approximately 1,457 m^3/d limited based on the screening capacity and grit removal tanks.
- The secondary treatment system has an ADF capacity of approximately 2,837 m^3/d limited by the secondary clarifiers.

- The tertiary treatment facility has an ADF capacity of approximately 2,232 m^3/d limited by the peak hydraulic loading rate. This ADF is based on all four units in service. If a standby unit is required, the ADF capacity of the tertiary treatment will be reduced.
- The UV system has an ADF capacity of approximately 2,094 m^3/d limited by hydraulic capacity of the UV system.
- The effluent chamber and outfall has an ADF capacity of approximately 1,667 m^3/d limited by the hydraulic bottleneck of the arrangement upstream of the effluent chamber. The outfall is mostly hydraulically steep, runs with a free surface, and is generally supercritical or close to supercritical.

6 Optimization Opportunities for EA Considerations

Based on the findings summarized in Section 5, this section of the report proposes a list of optimization opportunities to either improve facility performance or increase capacity without major capital investment. Note that the optimization opportunities presented in this section should not be reviewed as the recommendations for implementation. Instead, these opportunities will be reviewed in detail during Phase 3 of this EA project in conjunction with the development of alternative treatment concepts in Phase 3.

6.1 WET WEATHER I/I REDUCTION IN COLLECTION SYSTEM

6.1.1 Capacity and Performance Limiting Factor

Based on the hydraulic modeling for the existing wastewater collection system, the rainfall derived I/I is a significant contributor to the peak flows within the Nobleton sewershed. The following two opportunities should be considered for wet weather I/I reduction in the collection system:

- **Inspection of Areas with High Infiltration**. Within the catchment, constant (base) infiltration has been identified as 4.5 L/s, which is over 30 percent of the dry weather flow. This level of infiltration is equal to 106 L/c/d. This is distributed across the catchment; the worst areas are upstream of Bluff Trail PS and the area that drains along King Road. Removing this base infiltration from the catchment would be difficult due to the issues of identifying the exact locations of its source. It is likely that the cause of the infiltration is due to a large number of sewers which are below the water table. Figure 6-1 shows the variation across the catchment in terms of the amount of base infiltration applied within the model. The analysis Civica completed in 2016 identified that more than 30 percent of the pipes within the area that drains to Bluff Trail are below the water table. However, the analysis that was carried out highlighted that in the areas where over 50 percent of the pipes are below the water table, no base infiltration was identified from the modeling. This would suggest that the pipes in this area are in good condition, which is what would be expected as the system is of new construction and has not been in service for that long. Across the majority of the rest of the catchment, a proportion of the pipes are below the water table where the base infiltration could be getting into the sewer system. In those other areas where infiltration is identified, additional work will need to be undertaken to try and identify the possible reasons behind the large amount of infiltration in the area (Civica, 2016).
- **Roof Connection Survey**. Although the system is supposed to be sanitary-only, the modeling identified that there is a fast storm response and slow response from rainfall-related infiltration entering the system. The fast storm response could be from roof connections or potentially from the location of gullies close to properties. To understand the fast response and to determine the exact source and amount a contributing area, a survey would need to be undertaken to determine if there are properties within the catchment that have had their roofs connected to the sanitary system.

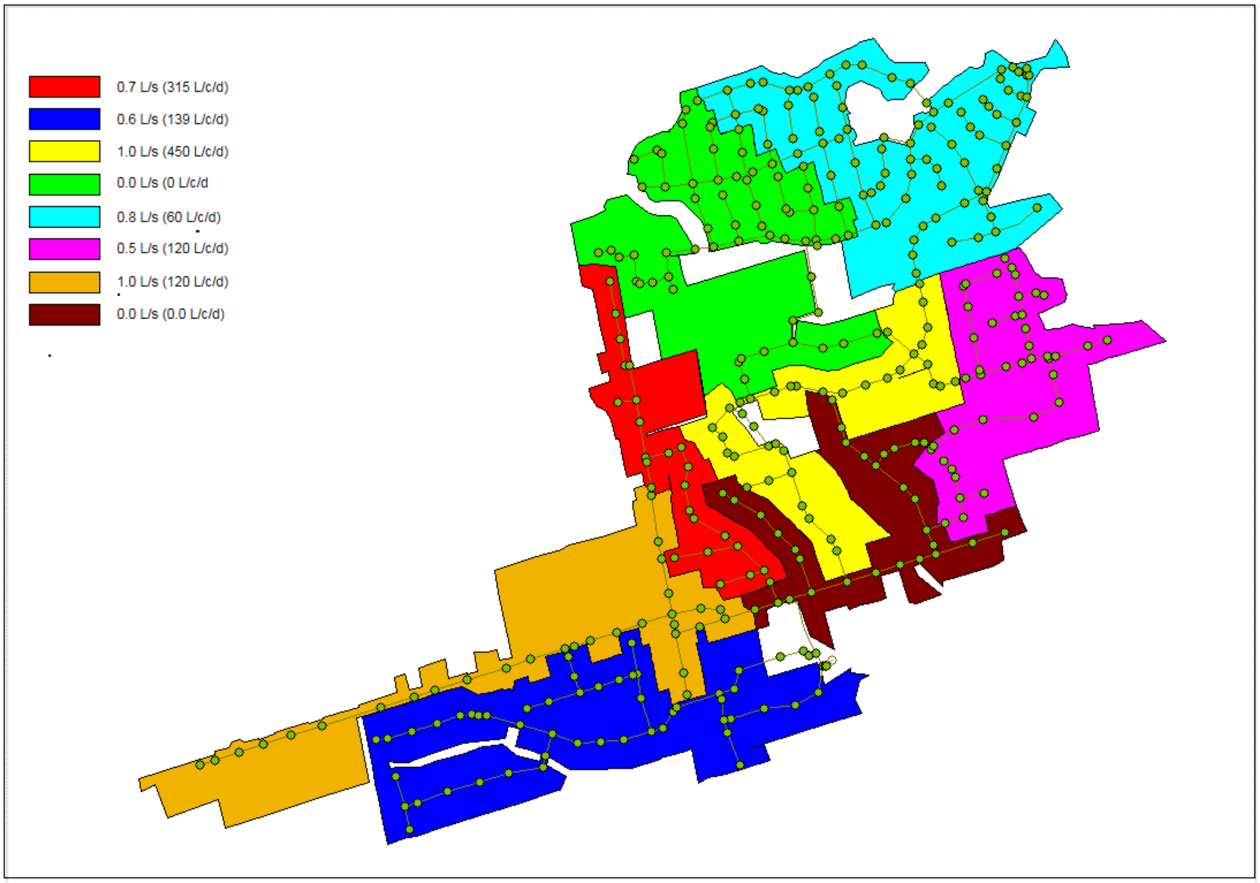

Figure 6-1: Amounts of Base Infiltration Applied Across the Catchment

6.1.2 Optimization Options

The recommended optimization strategy for the existing conveyance system includes:

- Conduct a review of Region's CCTV survey program to understand if areas of poor pipe work can be identified at this stage. Also potentially it will rule out of areas to carry out CCTV where information was already collected. It is unlikely that this will show any issues due to the age of the system.
- Investigate the locations where properties may have their roofs connected to the sewer. If these can be disconnected, the peak flows into the system can be reduced.; and
- To remove some of the surcharging within the system, the operation of Janet Avenue PS needs to be reviewed to understand if the capacity can be improved.

6.2 PEAK INSTANTANEOUS FLOW INTO NOBLETON WRRF REDUCTION

6.2.1 Capacity and Performance Limiting Factor

As identified in Section 5, high peak flows into the Nobleton WRRF limit the capacity of different treatment processes. Based on the historical data, the peaking factors for the PHF and PDF are 4.7 and 6.3 respectively. These peaking factors are significantly higher than the design peak factor of 3.14.

The Janet Ave PS has a wet well volume of approximately 20 m^3, which does not provide peak shaving. A review of the historical wastewater flow variations into Nobleton WRRF was conducted. The highest peak hourly flow events are identified for Year 2014 – Year 2017, as summarized in Table 6-1. The data suggest that duration of the peak hour flows into the Nobleton WRRF from 2014 to 2017 is approximately 1 to 3 hours. Frequency of these high flows were estimated to be approximately 8 to 10 times a year.

Table 6-1: Peak Hourly Flows into the Nobleton WRRF

YEAR	DATE	DURATION	PEAK HOURLY FLOW	PEAK FACTOR
2014	February 20th	1 hour	4.10 MLD	4.7
2015	May 10th	1 hour	4.10 MLD	4.1
2016	March 31st	3 hours	4.70 MLD	4.2
2017	June 23rd	3 hours	8.60 MLD	5.9

Notes:
Source: 2014 – 2017 SCADA data, 5-min interval flows

6.2.2 Optimization Opportunity

Because the frequency and duration of the peak flow events are low, one potential optimization opportunity is to construct a flow equalization facility to reduce the peaking factor for peak flows down to 3.14, similar to the existing value in the ECA.

Based on all the high flow events from 2014-2017, the wet weather event occurred on June 23, 2017 had the highest magnitude of peak flows. This rainfall event had a peaking factor of 5.9 for the peak hourly flow and a duration of 3 hours. The characteristics of this event could be used for sizing the EQ tank. However, the sizing of the EQ tank should be determined in conjunction with the future growth should the Nobleton WRRF be used to serve the growth areas. Therefore, the detailed discussion on EQ tank sizing should be conducted in Phase 3 of the project if this optimization opportunity is selected.

6.3 JANET AVENUE RAW SEWAGE PUMPING STATION

6.3.1 Capacity and Performance Limiting Factors

Based on the review of the 5-min pumping rate out of the Janet Ave PS, the flow is conveyed intermittently to the Nobleton WRRP. The flow to the Nobleton WRRF is generally more constant during peak flow. During low flow conditions, the duty pump cycles on and off. When the pump is in operation, the flow rate is typically approximately 20 L/S or 1,730 m^3/d. As a result, the Nobleton WRRF also receives flow intermittently.

6.3.2 Optimization Opportunities

With Nobleton WRRF currently receiving an ADF approximately 40 percent of the rated capacity, it is recommended that one of the pumps be replaced with a smaller size unit so that more constant but lower flowrates can be pumped into the Nobleton WRRF. During high flow events, the second pump can be activated to maintain the liquid level in the wet well to avoid flooding.

6.4 HEADWORKS

6.4.1 Capacity and Performance Limiting Factors

The following capacity and performance limiting factors are identified:

- There are two screens, one mechanically cleaned and one manually cleaned. Each screen is rated at a peak flow rate of 9,177 m^3/d;
- There are two grit tanks (one duty one standby), with each unit rated for a peak flow of 9,177 m^3/d.

6.4.2 Optimization Opportunities

To increase the capacity of the headworks, the following optimization opportunities can be considered:

- Replace the manually cleaned screen with a mechanically cleaned screen. This would increase the process capacity through the screen facility;
- Operate the two grit tanks as lead / lag units. During high flow events, both grit tanks can be in operation to gain capacity.

After the implementation of the above opportunities, the capacity of the headworks will be increased significantly.

6.5 AERATION SYSTEM

6.5.1 Capacity and Performance Limiting Factor

It was identified that the DO concentrations in the aeration tanks are much higher than the typical value of 2.0 mg/L. One of the causes is that there is no automated DO control and the blowers are fixed speed. This results in high energy cost for aeration.

6.5.2 Optimization Opportunities

Provide an automatic DO control approach to match the oxygen demand in the aeration tank to achieve an average DO residual of 2.0 mg/L.

A process evaluation was done to include an anoxic selector at the beginning of the aeration tanks (the first 10-25% volume) to reduce process oxygen demand, recover alkalinity, and to improve sludge settleability. The following features could be considered:

- For the anoxic zone, membrane diffusers with mechanical mixers could be used.
- During winter operation, the anoxic zone could be operated as an oxic zone to provide the required aerobic sludge retention time for the extended aeration system, if needed.
- During summer operation, the anoxic zone could be operated with air off to provide denitrification to achieve approximately 8 percent reduction in oxygen demand. Because denitrification in the aeration tank can reduce denitrification potential in the secondary clarifiers, the sludge settleabilty could be improved. This could be beneficial to the Nobleton WRRF during

high flow events. In addition, denitrification can also recover alkalinity by approximately 134 kg/d as $CaCO_3$. The detailed calculation is included in Appendix A.

6.6 PHOSPHOROUS REMOVAL SYSTEM

6.6.1 Capacity and Performance Limiting Factors

As identified previously, as flows increase beyond the current ECA rated ADF capacity, the required level of treatment by the tertiary filters will become increasingly more demanding based on the effluent TP loading of 160 kg/year.

6.6.2 Optimization Opportunities

The following tests are recommended to develop optimization opportunities for the existing phosphorous removal system:

- Optimization of alum dosing location: based on the review of the soluble and total phosphorous in the secondary effluent and tertiary effluent flows, the chemical dosing location should be optimized to improve reaction between alum and soluble phosphorous in the secondary treatment system as well as upstream of the tertiary filters. In addition, adding another dosing location (upstream of aeration tanks) could also be considered.
- Jar testing be conducted to determine the optimal coagulant (alum) or coagulation combination (e.g., alum with polymer), chemical dosages and dosage points. Based on the results of jar testing, the chemical dosing system for phosphorous removal at the Nobleton WRRF could be optimized to improve performance or decrease chemical costs.
- Stress testing should be conducted to verify the potential peak treatment capacity of the existing tertiary filters and to assess their hydraulic capacity. Based on the MECP Design Guideline, the deep bed filters are designed at 3.3 L/ (m^2.s) under peak hourly flow rate. However, this flow rate does not correspond to the effluent TP concentrations. The filter loading rate is based partly on the effluent target. Table 4-13 presents the annual instances of peak flows to the facility and the corresponding effluent phosphorus concentration on or around the day of peak flows. The data suggests that even at high loading rates, the filters were able to maintain an effluent TP concentration below 0.1 mg/L. This suggests that the existing filters have potential to be operated at higher HLRs than the value recommended by the MECP. The outcome of the stress testing could be used to optimize the performance and capacity of the existing filters.

Table 6-2: Effluent Phosphorus Corresponding to Peak Flow Events to the Facility

DATE	PEAK FLOW (MLD)	HLR (lps/m^2)	EFFLUENT PHOSPHORUS, mg/L
2014	1.95	2.24	0.09
2015	1.78	2.05	0.02
2016	2.55	2.93	0.05
2017	3.89	4.45	0.03

6.7 UV DISINFECTION

6.7.1 Capacity and Performance Limiting Factors

The UV system was designed with the assumption of a UVT of 65 percent. According to the supplier, the maximum flow that can be conveyed through the UV system and meet the disinfection requirements is 9,842 m^3/d (Hatch Mott MacDonald, 2015). A hydraulic review also identified that the peak flow of 10,500 m^3/d can be conveyed through the UV cannel with a freeboard of 0.53 m (Hatch Mott MacDonald, 2015). There is a potential to reassess the capacity of the existing UV system while meeting the effluent disinfection target.

6.7.2 Opportunities

The following test should be conducted to confirm the possibility of increase capacity of the UV system:

- Conduct stress testing on the UV disinfection system to verify peak and ADF capacity of the existing UV disinfection system. During the stress testing, the UV channel hydraulics can be assessed to verify the hydraulic limitations, identify the source(s) of the bottleneck, and to develop solutions to mitigate the hydraulic constraints.

6.8 SAMPLING AND MONITORING

6.8.1 Capacity and Performance Limiting Factors

The following deficiencies are identified:

- The current monitoring program at the Nobleton WRRF does not allow for the evaluation of impact of return flows on the liquid treatment train.
- The characteristics of the WAS, RAS, and TWAS are not tested.

6.8.2 Optimization Opportunities

The following are recommended to improve the monitoring program:

- The location of the raw sewage sampler should be relocated at the headworks building.
- The supernatant and filter backwash wastewater should be relocated upstream of raw wastewater samples.
- Take weekly samples on RAS/WAS and TWAS to confirm their characteristics, including TS and VSS. The results will help to confirm the solid production at the Nobleton WRRF.

7 Summary and Recommendations

The key findings of this optimization report include:

- The existing Nobleton WRRF has an ADF capacity of approximately 1,457 m^3/d limited based on the screening capacity and grit removal tanks. This capacity is an equivalent serviceable population of 3,938 persons.
- The existing Nobleton WRRF experiences high PHF and PIF, with an average peaking factor of 4.3 and 6.3, respectively. These peaking factors are significantly higher than peaking factor of 3.14 used in 2007 design. As a result, the capacities of some process units are less than the currently rated capacity of 2,935 m^3/d.
- The Nobleton WRRF could be optimized to gain additional capacity with the following opportunities:
  - Construct a flow EQ facility to reduce peak hourly and instantaneous flows. This could increase the equivalent serviceable population.
  - Replace one of the existing blowers to increase the firm capacity of the aeration system.
  - With the combination of the above two measures, the existing Nobleton WRRF could be re-rated with a higher ADF capacity.
  - Note that the above optimization opportunities are developed for consideration in Phase 3 of the project. They are not intended to be used as the baseline capacity for the Nobleton WRRF.

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Appendix A

Calculations

Nobleton WRRF Optimization Calculation Sheet (1 of 3)

Section 1: Design Basis Summary

Historical Data	Value					Comments
	Unit	2014	2015	2016	2017	
Annual Average Flow	MLD	0.88	0.98	1.14	1.45	
Maximum Month Flow (MMF)	MLD	1.20	1.30	1.77	1.99	
Maximum Month Peaking Factor	-	1.37	1.33	1.55	1.37	
Maximum Week Flow (MWF)	MLD	1.39	1.39	2.10	2.61	
Maximum Week Peaking Factor	-	1.59	1.42	1.84	1.80	
Peak Day Flow (PDF)	MLD	1.95	1.78	2.55	3.89	
Peak Day Peaking Factor	-	2.22	1.81	2.24	2.68	
Peak Hourly Flow (PHF)	MLD	4.70	4.10	5.77	8.60	
Peak Hourly Flow Peaking Factor	-	5.37	4.18	5.06	5.93	
Peak Instantaneous Flow (PIF)	MLD	5.26	7.32	6.60	8.83	
Peak Instantaneous Flow Peaking Factor	-	6.01	7.47	5.79	6.09	
Average Dry Weather Flow	MLD	1.64	0.98	1.06	1.44	
Average BOD Loading	kg/d	113	155	138	148	
Average TSS Loading	kg/d	128	146	104	157	
Average TKN Loading	kg/d	24	33	36	38	
Average TP Loading	kg/d	3.1	4.1	4.4	4.9	
Service Population	ppl	2,923	3,119	3,643	3,891	
Wastewater Generation Rate	L/cap/d	300	314	313	373	
Wastewater Loading Rate						
BOD	g/cap/d	38.65	49.68	37.87	38.01	Typical Value = 75 in a range of 50-120
TSS	g/cap/d	43.70	46.93	28.52	40.22	Typical value = 90 in a range of 60-150
TSS/BOD	-	1.13	0.94	0.75	1.06	Typical value: 1.1-1.2
TKN	g/cap/d	8.33	10.46	9.89	9.69	Typical value 9-21.7
TKN/BOD	-	0.22	0.21	0.26	0.26	Typical Value: 0.16
TP	g/cap/d	1.06	1.30	1.20	1.25	Typical value: 2.7 - 4.5
TP/BOD	-	0.03	0.03	0.03	0.03	Typical value: 0.04
Findings:						
1	The average wastewater generation rate (L/c/d) appears to be much lower than the design value of 450 L/c/d					
2	The average BOD / TSS/ TP loading (g/cap/d) appear to be lower than typical value					
3	TKN loading (g/cap/d) appears to be within the expected range					

Recommendations for Design Basis	Unit	Existing Population	Future Growth
Wastewater Generation Rate			
	L/cap/d	370	370
Wastewater Loading Rate			
BOD	g/cap/d	45	75
TSS	g/cap/d	43	90
TKN	g/cap/d	10	13.3
TP	g/cap/d	1.3	2.3
Future Service Population	ppl	3891	6909
Wastewater Flow Peaking Factors			
MMF		1.4	1.4
PDF	-	2.2	2.2
PHF	-	4.7	4.7
PIF	-	6.3	6.3
Max Month Mass Loading Peaking Factors			
BOD5	-	1.4	1.4
TSS	-	1.3	1.3
TKN	-	1.1	1.1
TP	-	1.2	1.2

Nobleton WRRF Optimization Calculation Sheet (2 of 3)

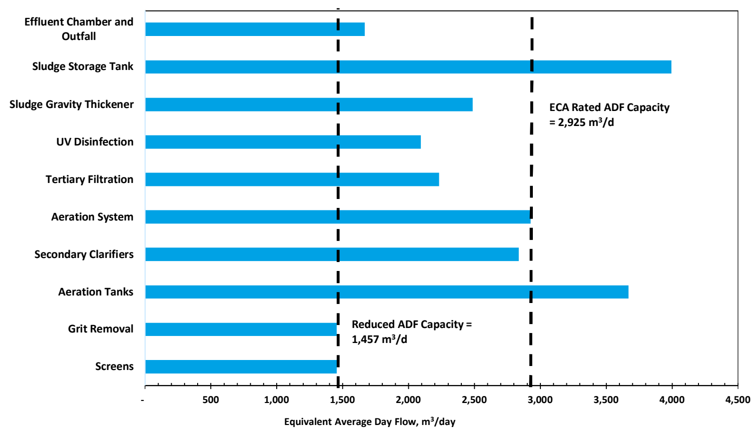
Section 2: Nobleton WRRF Capacity Assessment

Process Area	Unit	Value			Comments
		ADF Capacity	PHF or PDF capacity	PIF Capacity	
Headworks					
Screen	m3/d			9,177	rated capacity of each screen unit
Grit Tanks	m3/d			9,177	one duty one standby, each rated at 9,177 m3/d
Aeration Tanks					
Number of Aeration Tanks	ea	2	2	2	currently only one in operation
Volume of Aeration Tank	m3	3,072	3,072	3,072	
Historical Operation					
MLSS					
2014	mg/L	2,686			
2015	mg/L	3,437			
2016	mg/L	2,953			
2017	mg/L	2,792			
MLVSS					
2014	mg/L	1,792			
2015	mg/L	2,322			
2016	mg/L	2,173			
2017	mg/L	2,066			
MLVSS/MLSS					
2014	-	0.67			
2015	-	0.68			
2016	-	0.74			
2017	-	0.74			
WAS Flow					
2014	m3/d	17.68			
2015	m3/d	16.64			
2016	m3/d	20.00			
2017	m3/d	19.21			
WAS Concentration	mg/L	8,000			Based on limited data
WAS Mass					
2014	kg/d	141			
2015	kg/d	133			
2016	kg/d	160			
2017	kg/d	153			
SRT					
2014	d	29			MLSS * One Aeration Tank Volume ÷ WAS MASS
2015	d	40			
2016	d	28			
2017	d	28			
Aeration Tank Capacity Assessment					
Design SRT	days	15			typical value
Design MLSS	mg/L	3500			Typical Value
MLVSS/MLSS	-	0.70			Average of historical value
Calculated VSS Inventory in Aeration Tanks	kg	7,526	-		MLSS x Two Aeration Tank Volume x MLVSS/MLSS
Daily VSS Generation	kg/d	502			VSS / SRT
VSS Yield	kg VSS/kg BOD Removed	0.8			Typical Value
BOD Loading into Aeration Tanks	kg/d	627.20			Daily VSS Generation ÷ VSS Yield
Equivalent Population Capacity	**ppl**	**9,919**			***BOD loading from existing population + future growth***
Equivalent Average Flow	m3/d	3,670			
HRT	Hrs	20			all within typical value
Aeration Blowers					
Number of Units	-	3	3	3	2 duty 1 standby
Capacity, each	m3/hr	767	767	767	each
Historical DO					
2014	mg/L	5.45			High DO residuals - opportunity for optimization
2015	mg/L	4.32			
2016	mg/L	4.45			
2017	mg/L	4.07			
Blower Capacity Assessment					
Service Population	**ppl**	**7,915**			try different population number to match the blower firm capacity
TKN Loading, average	kg/d	92.43			
TKN Loading, peak day loading	kg/d	166.37			using average PF from 2014-2017 (PF = 1.8)
BOD Loading, average	kg/d	476.90			
AOR for max month loading					
BOD Oxygen Demand	kg/d	715			1.5 kg O / kg BOD
Nitrification Oxygen Demand	kg/d	689			Assuming 90% TKN to be nitrified at 4.6 kg O / kg TKN
Total AOR	kg/d	1,404			
AOR/SOR	-	0.37			Calculated
SOR for max. month loading	kg/d	3,795			
SOTE	%	37			
Standard Air Demand for Max. Month Loading	m3/hr	1,534			
Firm capacity of two blowers	m3/hr	1,534			
If denitrification is provided:					
Degree of denitrification	%	25%			Assumption
NOX-N denitrified	kg/d	37			assuming 90% TKN nitrified
Oxygen credit	kg/d	(107)			2.86 kg Ox per kg NOX denitrified
Total AOR	kg/d	1,297			8% reduction in AOR demand
Alkalinity Recovery	kg/d	134			3.57 kg Alkalinity recovery per kg NOX-N denitrified
Secondary Clarifiers					
Number of Clarifiers	ea	2	2	2	
Surface area of each clarifier	m2	180	180	180	
Peak Surface Overflow Rate = 37 m3/m2/d	m3/d			13,333	
Peak Solids Loading Rate = 170 kg/m2/d	m3/d		8,423		calculated under peak day flow at 100 RAS, 5,000 mg/L MLSS
Tertiary Filters					
Number of filters	ea	4	4	4	each filter has two cells
surface area of each filter	m2	9.3	9.3	9.3	
Peak HLR = 3.3 L/s/m2 or 282 m3/m2/d	m3/d			10,490	all filters in service
UV Disinfection					
Number of channel	ea	1	1	1	
Number of banks	ea	2	2	2	
Peak capacity	m3/d		9,842		Vender info
Sludge Management - Gravity Thickener					
Gravity Thickner surface area	m2	17.22			4.1 m (L) x 4.2 m (W) x 6.35 m (SWD)
Maximum solids loading rate	kg/m2/d	36			MECP Design Guideline
Maximum WAS loading rate onto gravity thickener	kg/d	620			maximum month loading condition
Service Population	ppl	6,722			
Equivalent ADF	m3/d	2,487			
Sludge Management - Sludge Storage					
Total population as per growth	ppl	10,800			total service population
BOD5 loading, average	kg/d	693			BOD from existing population + from future growth
VSS Yield	kg/d	555			
VSS/TSS	-	0.70			
TS production, average daily	kg/d	792			
WAS Concentration, average	mg/L	6,000			assumed
WAS Volume	m3/d	132			
TWAS TS%	%	2.5			
TWAS Volume - average loading	m3/d	32			
TWAS Volume - maximum month loading	m3/d	44			existing storage tank can provide 3 day storage volume

Nobleton WRRF Optimization Calculation Sheet (3 of 3)

Section 3: Nobleton WRRF Capacity Assessment Summary

Process Area	ADF	PDF			Equivalent Population
Screens			9,177	1,457	3,937
Grit Removal			9,177	1,457	3,937
Aeration Tanks	3,670			3,670	9,919
Secondary Clarifiers		8,423	13,333	2,836.80	7,667
Aeration System	2,929			2,929	7,915
Tertiary Filtration			10,490	2,232	6,032
UV Disinfection			9,842	2,094	5,660
Sludge Gravity Thickener	2,487			2,487	6,722
Sludge Storage Tank	3,996			3,996	10,800
Effluent Chamber and Outfall			10,500	1,667	4,505



FINAL

CLASS ENVIRONMENTAL ASSESSMENT FOR WATER AND WASTEWATER SERVICING IN THE COMMUNITY OF NOBLETON

EXISTING WATER SYSTEM HYDRAULIC ANALYSIS

Study 2A

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

4 JUNE 2019

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Distribution List

BV FILE NO.	RevNo	Issued to	Date	Reason for Issue
	0	York Region – Afshin Naseri	11 Dec 2017	Draft for YR Review
	1	York Region – Afshin Naseri	18 Dec 2018	Revised Final
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Revision Log

BV FILE NO.	RevNo	Prepared By	Reviewed By	Authorized By	Issue/Revision Description
	0	SJ	OK	SL	Initial Draft
	1	SJ	OK	JB	Revised Final
	2	SJ	OK	JB	Final with complete 2018 data

Black & Veatch Signatures □

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Project Advisor - Water -

1 Introduction

1.1 PURPOSE OF STUDY

The purpose of the Existing Water System Hydraulic Analysis is to:

- Confirm the existing capacity of the water system (well supply, storage, and distribution);
- Identify any hydraulic limitations (bottlenecks, etc.);
- Assess the maximum flow that the existing system can support before any major upgrades are required;

This report will be a supporting document for the Water System Capacity & Optimization Study (Study 1A).

The next stage of the Hydraulic Analysis for this project will focus on the Future System Hydraulic Analysis and will be documented in the Water Needs Assessment and Justification Study.

1.2 BACKGROUND

Nobleton is a community in King Township, located in York Region. Currently, Nobleton is serviced by standalone water and wastewater systems to meet the needs of the current population. The York Region Water and Wastewater Master Plan (2016) indicated that both the water and wastewater systems would not have sufficient capacity to meet requirements to support growth to the 2041 Master Plan horizon. Therefore, the Master Plan recommended undertaking the current project, a Schedule C Class Environmental Assessment (EA), to identify preferred servicing solutions to accommodate growth.

1.3 EXISTING WATER SYSTEM INFRASTRUCTURE

The Regional Municipality of York (also referred to as the Region and York Region) is responsible for the water production, treatment, storage and transmission to its local area municipalities, including the Community of Nobleton in the Township of King. The Nobleton water supply system consists of three groundwater wells and two elevated storage tanks that provide service to the Nobleton Pressure District. There is also a booster pumping station (BPS) that services a higher elevation area in the northwest portion of the distribution system. The wells operate based on level at either of the elevated tanks. The booster pumping station operates independently from the rest of the water system controls.

2 Model Review and Update

2.1 EXISTING HYDRAULIC MODEL

2.1.1 General Model Information

The existing hydraulic model of the Nobleton water distribution system was provided by the Regional Municipality of York via the Township of King (the Township) and R.J. Burnside & Associates Limited. The model was provided in the InfoWater software format.

The model provided was only set-up for steady state modelling runs. Therefore, in order to make the model suitable for the purposes of the Nobleton Environmental Assessment, extended period simulation scenarios were created for average day demands and maximum day demands. This required the addition of various modelling aspects, including:

- Pump/Well controls that turn on/off based on storage level
- Diurnal (daily demand) patterns that are assigned to each demand node
- Two separate demand allocations that are specific to average day demands and maximum day demands, respectively.

2.1.2 Facility Data

Additionally, the model provided did not have the wells simulated. Initially in the provided model, the system HGL (hydraulic grade line) was set based on the storage levels and all flow was "back-fed" from the storage facility to the nodal demands during the simulation. This is not suitable for the purpose of this study. The actual well flows must be simulated to confirm that the wells and distribution network are able to transfer water to the storage tanks. Therefore, the model was updated to include the following detailed well/pump information;

- Pump Curve (or Design Point, when curve is unavailable)
- Pump Elevation
- Ground water level (based on drawdown level and top of casing information); Simulated as a fixed head reservoir

In the provided model, storage facilities (elevated tanks) were simulated as cylindrical storage tanks. Although, the low water level, top water level and storage volume were generally correct, it is more appropriate to use tank volume – height curves for the elevated tanks since the storage facilities are not perfectly cylindrical. Therefore, the volume-height curves were added to the model based on provided as-built information.

2.1.3 Model Demands

The assumptions used to develop the system demands in the model provided by the Township were not provided. Therefore, as part of the hydraulic analysis, a detailed demand analysis was completed using historical billing data (2015 to 2017) and historical SCADA data (hourly from

2012-2018) to more accurately determine the water consumption patterns in the Town of Nobleton.

2.1.4 Model Network

Figure 1 displays the existing network in the provided InfoWater model:

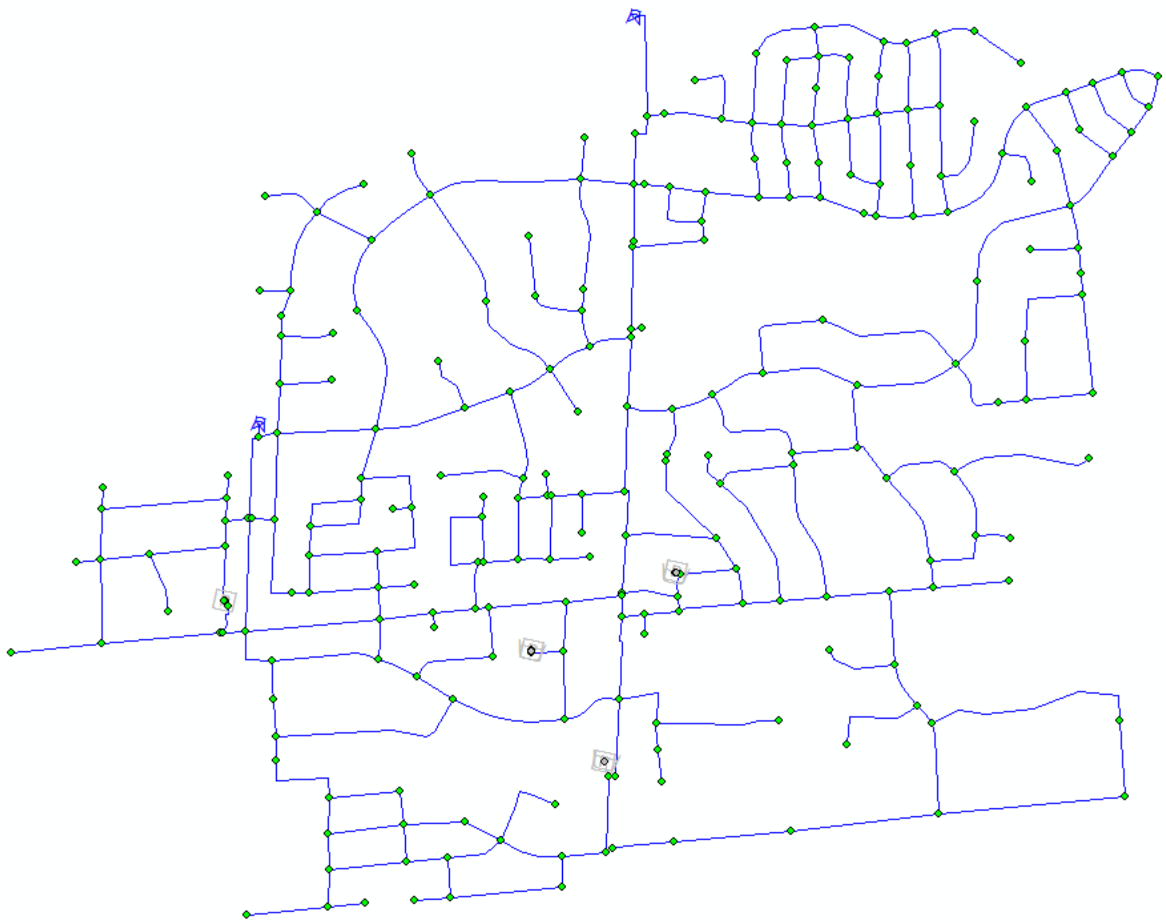
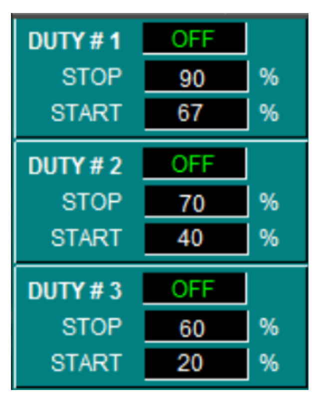


Figure 1: Provided Model – Pipeline Network

Generally, the provided model had an accurate representation of the existing watermain network. The provided model included both existing watermains and future proposed watermains, which were identified with an installation year of 2031. A few minor updates were required to ensure that the existing watermain network matches the latest GIS, as provided by the Region.



Additionally, pump controls were added based on the tank level at Nobleton South Elevated Tank (ET). The appropriate duty #1, duty #2 and duty#3 controls were added based on the SCADA screenshot provided in the Well #3 control narrative.

2.2 MODEL UPDATES

The following sections summarize the updates that were made to ensure that the hydraulic model is up-to-date and suitable for the analysis.

2.2.1 General Updates

Two new scenarios were created in the model for the existing system hydraulic analysis.

1. EXISTING_ADD: Existing (2017) Average Day Demand Scenario
2. EXISTING_MDD: Existing (2017) Maximum Day Demand Scenario

A screenshot of the InfoWater Scenario Explorer can be seen below:

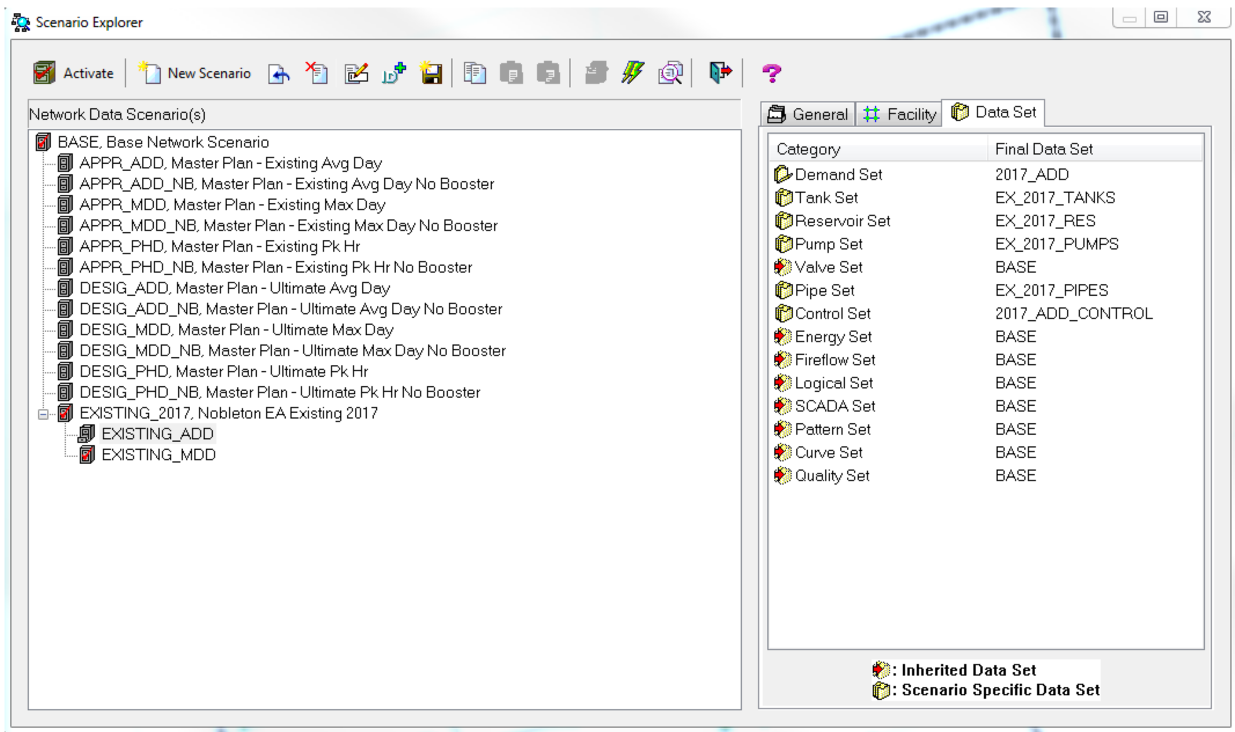


Figure 2: Updated Model – Scenario Explorer

As shown in the above graphic, new data sets have been created for demands, tanks, reservoirs, pumps, pipes and controls in order to model the existing scenarios. It is noted that the demands are based on the maximum day demands that occurred in 2016, whereas the physical infrastructure is based on the latest known (2017) infrastructure.

The only datasets that are different in the average and maximum day demand scenarios are the demand set and the control set. Currently, the model actually uses the same controls in both scenarios, but providing separate control sets provides the user flexibility when optimizing the system for two different demand conditions.

2.2.2 Facility Updates

2.2.2.1 Well Facilities

Each well facility was revised in the model so that it more accurately simulates the actual well operations. In order to accomplish this, each well is simulated in the model using a fixed head reservoir and a pump, where the reservoir simulates the ground water level at the well.

Ground water level at the supply source was updated based on the available drawdown level information provided by the Region. For example, for Nobleton Well 2, the ground water was modelled with an elevation set at 240m based on the most recent step test conducted. Figure 3 displays the step test that was conducted on October 19, 2012 for Nobleton Production Well #2. (York Region, Environmental Services, 2012).

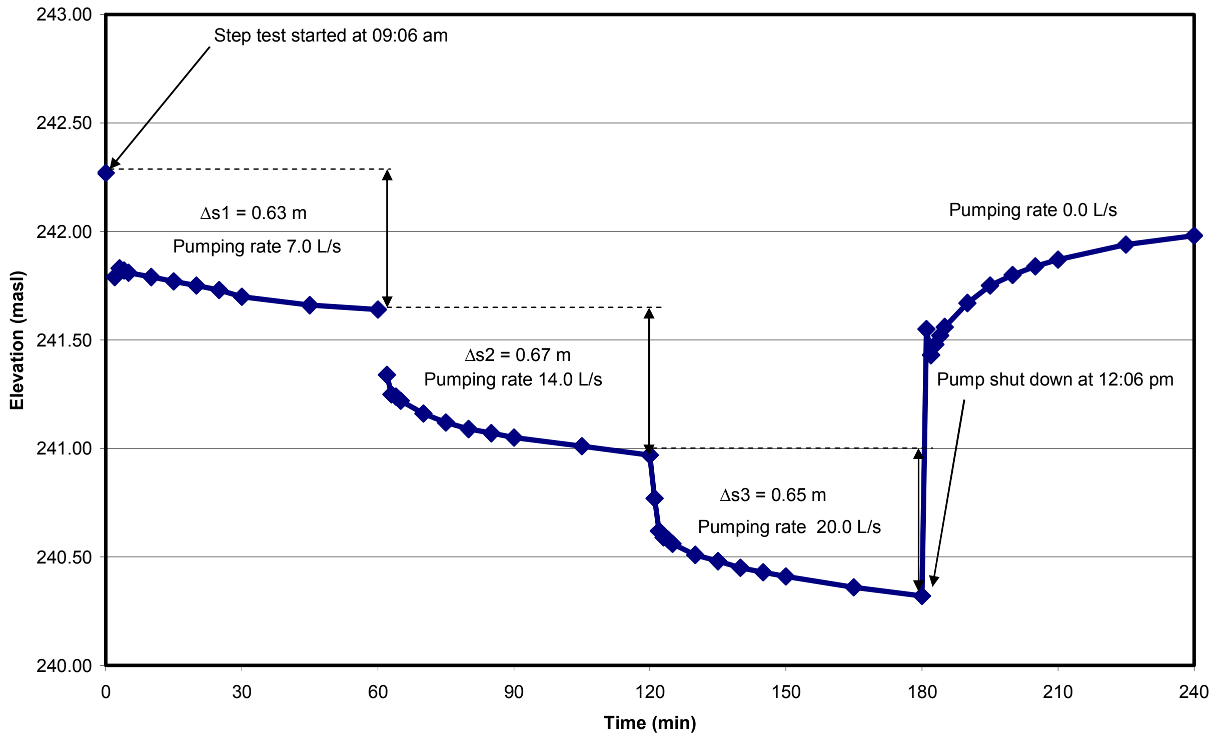


Figure 3: Step Test Results at Nobleton Well #2

The pumps in the model were simulated based on the best available information. The pump curve is available for Well #5 and it was included in the model whereas for Wells #2 and #3, pump curves are not available; therefore, only the design point for each pump was added to the model instead.

2.2.2.2 Storage Facilities

Storage facilities (Nobleton North ET and Nobleton South ET) were both updated to include volume-height curves based on the available as-builts provided. Figure 4 shows the new curve that is included in the model for Nobleton North ET, where 0m in the x-axis is equivalent to the ground elevation of the tank (not the bottom of bowl minimum elevation):

	X	Y
1	0.000	0.000
2	35.216	14.434
3	35.520	37.942
4	35.825	68.801
5	36.130	102.951
6	36.435	140.706
7	36.740	182.244
8	37.044	227.741
9	37.349	277.371
10	37.654	331.319
11	37.959	389.759
12	38.264	452.868
13	38.568	520.827
14	38.873	593.810
15	39.178	670.266
16	39.483	746.813
17	39.788	823.361
18	40.092	899.908
19	40.397	976.455
20	40.702	1053.002
21	41.007	1129.549
22	41.312	1206.096
23	41.616	1282.643
24	41.921	1359.190
25	42.226	1435.737
26	42.531	1512.284
27	42.836	1588.831
28	43.140	1665.378
29	43.445	1741.925
30	43.750	1818.436
31	44.080	1890.000

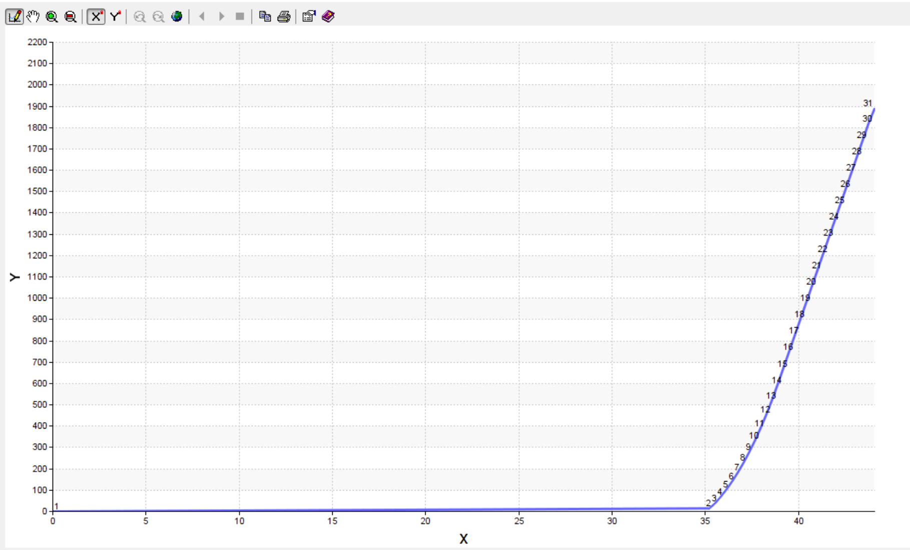


Figure 4: Sample Volume-Height Curve for Nobleton North Elevated Tank

2.2.3 Demand Allocation Updates

A detailed analysis of the historical water demands (average day and maximum day), as well as, an analysis of the diurnal patterns is provided in Study 1A: Water System Capacity and Optimization Study. The following section is focused primarily on the demand allocation process and how the demands are entered into the hydraulic model.

The first step of completing the demand allocation was linking the geocoded address shapefile with the historical Nobleton billing data (2015 to 2017). The following graphic visually demonstrates the success rate for matching addresses to billing data in 2015 and 2016:

Figure 5: Address and Billing Data 2015 & 2016

The following takeaways can be gathered from Figure 5:

- Generally, most of the existing addresses in Nobleton have a successful match to the billing data.
- There are two key growth areas, where new homes/customers recently started being billed for water. This is clear because they were billed in 2016, but not in 2015. These are shown by the green dots at the southwest and north parts of the Nobleton system.
- Various blue dots are scattered outside the Nobleton service boundary because these locations are not serviced by the municipal water system.
- There are a large number of blue dots that are located at the southwest part of the system. These are understood to be part of a new development that were not part of the billed system in 2016, but are either already in the 2017 network or will be added in the near future.

When allocating the billed demand to the model, the following process was used in InfoWater:

- Apply demand to nearest pipe and then nearest node using InfoWater Allocator tool;

- Demands were separated for residential and ICI (Industrial/Commercial/Institutional);
    - Residential demand is included in the Demand 1 column in InfoWater
    - ICI demand is included in the Demand 2 column in InfoWater
    - Non-revenue water is included in the Demand 3 column in InfoWater
- Non-revenue water was added evenly to all demand nodes. The quantity of non-revenue water was based on the difference between billed data and water production records (SCADA). These results are shown in Study 1A: Water System Capacity and Optimization Study.
- Demand allocation for the average day demand scenario was based on the March 2016 billing data and the demand allocation for the maximum day demand scenario was based on the September 2016 billing data. These months were chosen because they demonstrated a good match to the average and maximum day demands, respectively. There were also no obvious billing data anomalies in these months.

2.2.4 Network Updates

Generally, the watermain network in the provided model matched the GIS well. There were, however, some watermains listed as “proposed” in the provided model that seemed to exist by September 2016 based on the billing data. Therefore, some assumptions were made in order to activate the most up-to-date watermains. The existing watermain network in the updated model is shown in Figure 6, where blue pipes are active and grey pipes are inactive (future) watermains that were already digitized in the provided model.

Figure 6: Updated Model – Pipeline Network

3 Existing Water System Capacity Review and Optimization

The Existing Water System (Stage 1) Hydraulic Analysis evaluated what flow the existing Nobleton infrastructure is capable of servicing.

3.1 EXISTING SYSTEM CAPACITY REVIEW

The following section presents the results of the following two modeling scenarios:

- Existing Average Day Demand
  - Average Day Demand was established based on the historical SCADA data from the past seven years.
  - 2016 and 2018 have the highest average day demands recorded in Nobleton. Due to population growth from 2016 to 2018, it is understood that the average demand is generally equivalent on a per capita basis.
  - It is useful to note that population growth in the Southwest and Northeast parts of Nobleton has increased the average demand during the winter months (January to April). During this time period, average demand has increased from 15.5L/s in 2016 to 18.7 L/s in 2017 and 18.6 L/s in 2018.
- Existing Maximum Day Demand
  - Maximum Day Demand was established based on the historical SCADA data from the past five years.
  - 2016 and 2018 had similarly high maximum day demands in Nobleton. Due to the growth in population from 2016 to 2018, the maximum day demand in 2018 is marginally higher, but is lower than 2016 on a per capita basis. Therefore, the 2016 data was used for the existing system analysis.

Table 1: Historical Water Demands in Nobleton based on SCADA (Water Production Records)

YEAR	2012	2013	2014	2015	2016	2017	2018
Average Day Demand (L/s)	13.9	14.9	14.9	16.1	**21.1**	20.4	**23.1**
Maximum Day Demand (L/s)	33.1	30.0	29.1	33.6	**44.0**	37.4	**45.5**

3.1.1 Well Capacity

The existing three wells in Nobleton (Well #2, Well #3 and Well #5) each have their own Permit To Take Water limit, as well as having a combined daily taking limit. The existing permitted capacities for the Nobleton wells are summarized in the table below. (MOECC, 2014)

Table 2: Existing Wells - Permitted Daily Withdrawals

WELL	PERMITTED CAPACITY (ML/D)	PERMITTED CAPACITY (L/S)
Nobleton PW #2	1.964	22.73
Nobleton PW #3	2.496	28.89
Nobleton PW #5	2.496	28.89
Current Combined Daily Taking Limit (with Largest Well Out of Service)	4.460	51.62

From Table 2, the following can be seen:

- The current combined daily taking limit of the Nobleton wells (51.62L/s) is greater than the historical maximum day demand that occurred in 2016 (44L/s).
- If the permitted daily taking was increased to equal the total of all three wells operating simultaneously, then the maximum capacity of the Nobleton system would be 6.956ML/D (80.51L/s).

In the hydraulic modeling, both an average day demand scenario and a maximum day demand scenario were simulated. Table 3 summarizes the well flows that were simulated in the hydraulic model:

Table 3: Hydraulic Model – Well Flows

WELL	AVERAGE DAY DEMAND - MODELLED FLOW* (L/S)	MAXIMUM DAY DEMAND - MODELLED FLOW* (L/S)
Nobleton PW #2	Not used	Average 16 L/s Maximum 22 L/s
Nobleton PW #3	Average 22 L/s Maximum 28 L/s	Average 28 L/s Maximum 28 L/s
Nobleton PW #5	Not used	Not used
*Note that Wells #2 and #3 are operating based on the current duty controls that were set, as described in Section 2.1.4, where Well #3 is the Duty #1 pump, Well #2 is the Duty #2 pump and Well #5 is the Duty #3 pump.		

3.1.2 Storage Capacity

As described in Section 16 of the York Region Design Guidelines (Water Systems), storage capacity requirements in the Region are based on the following criteria: (York Region, 2017)

Total Storage = Equalization Storage + Fire Storage + Emergency Storage, where

- Equalization (Balancing) Storage is the storage required to meet the diurnal variation of the maximum day condition, equal to 25% of Maximum Day demand. It is noted that this assumption of 25% is a "rule-of-thumb" guideline that could potentially be reduced based on an analysis of actual diurnal patterns.
- Fire Storage is the volume required for firefighting as defined by the Fire Underwriter's Survey Guidelines.
  - Historically, the Region has used a fire storage guideline of 10,000 L/min (166.7L/s) for a duration of 2 hours for smaller pressure districts with smaller commercial, medium and high density residential developments. The Town of Nobleton would fit into this category. This criteria was used during the most recent Master Plan.
  - This guideline also relates well with the MOECC guideline of 159L/s for 3 hours duration for communities with a population between 6,001 and 10,000.
- Emergency Storage is the additional volume for emergency events (e.g. prolonged power loss, watermain breaks, unusual fire demands, higher than usual demands, etc.), equal to 25% of (Equalization + Fire Storage).

Table 4: Existing Storage Requirements vs. Available Capacity

STORAGE COMPONENT	VOLUME (M^3)	NOTES
Equalization Storage Required	950	25% of maximum day demand
Fire Storage Required	1,200	10,000 L/min for two hours
Emergency Storage Required	538	25% of (Equalization +Fire Storage)
Total Storage Required	**2,688**	Equalization + Fire + Emergency
Current Available Storage	**3,845**	

Table 4 shows the required equalization, fire and emergency storage based on the existing 2016 maximum day demand. As seen in the above table, there is currently more than enough storage capacity (3,845m^3) to cover the storage requirements of the existing Nobleton system (2,688m^3). This means that there is currently an excess storage volume of 1,157m^3, which is available to support growth.

3.1.3 Distribution System

Generally speaking, based on the hydraulic analysis of the existing average day and maximum day demand scenarios, there are no significant bottlenecks, pressure issues or fire flow availability issues in the system.

In the hydraulic model, the existing Nobleton Booster Pumping Station provides flows ranging between 0.7 L/s and 3.0 L/s in the average day demand and maximum day demand scenarios, respectively.

Figure 7 displays the minimum pressures in the hydraulic model during the maximum day demand scenario.

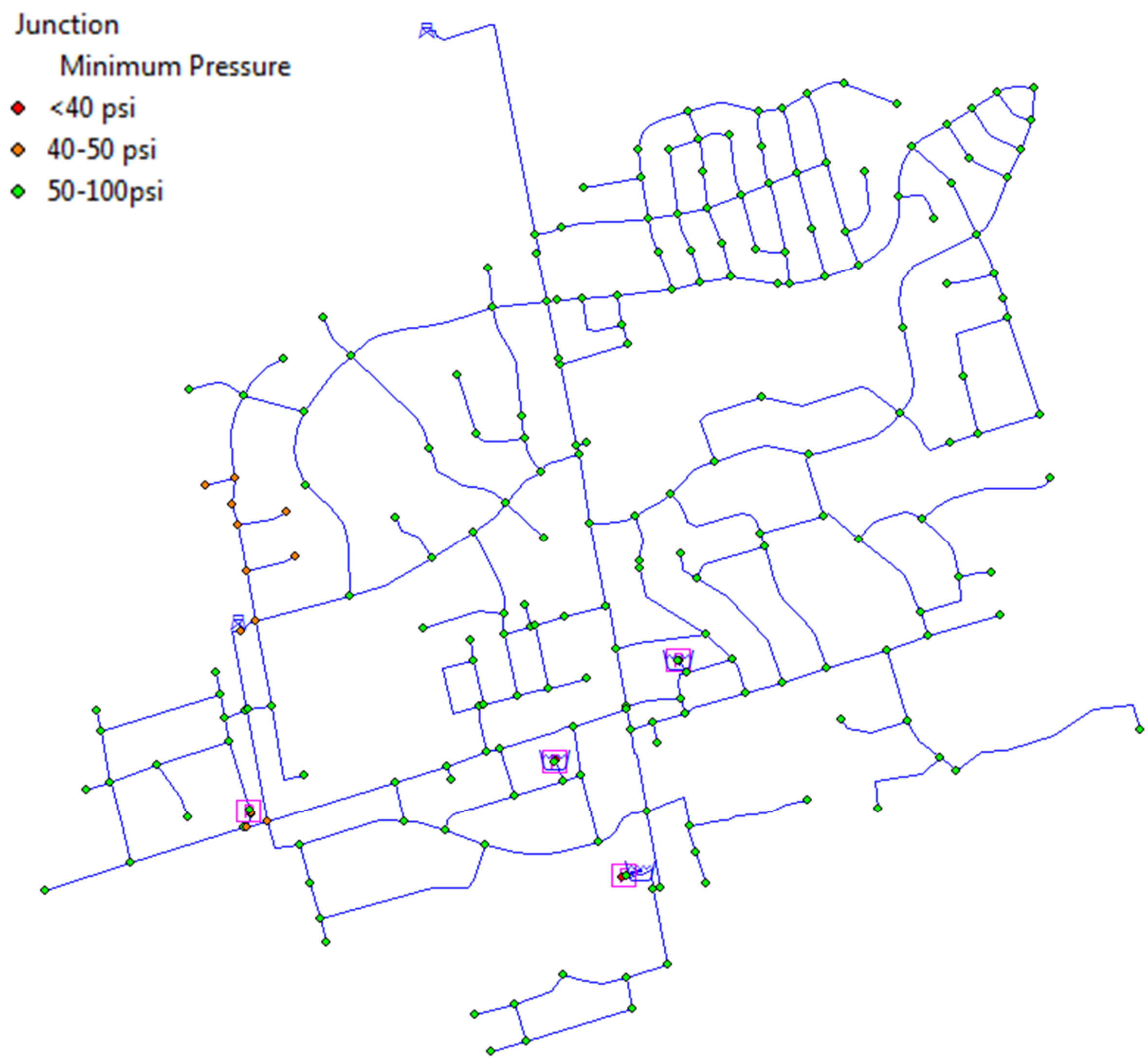


Figure 7: Maximum Day Demand Scenario – Minimum Pressure

From Figure 7, it can be seen that the area just north of the Nobleton South ET does experience marginally acceptable (46- 50psi) pressures due to being at a higher elevation than most of the system. During this time step, the nearby tank drops down to a level of ~5.6m (HGL ~319.6m).

Figure 8 displays the maximum velocities in the hydraulic model during the maximum day demand scenario.

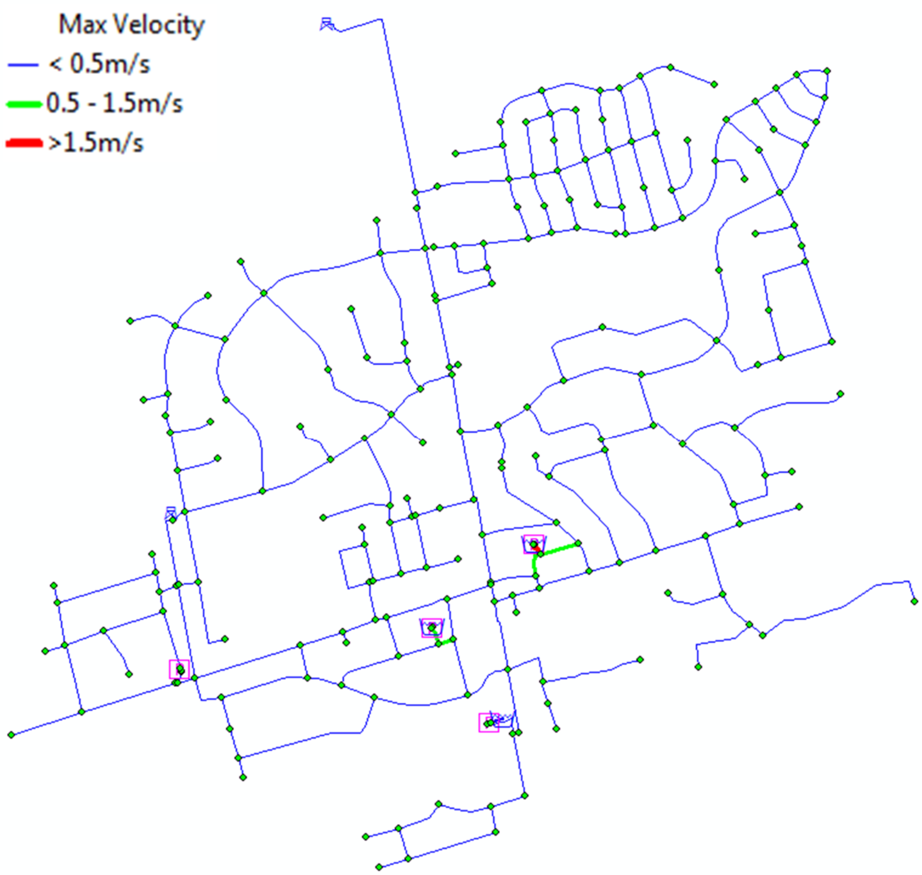


Figure 8: Maximum Day Demand Scenario – Maximum Velocity

From Figure 8, it can be seen that there are no bottlenecks in the existing system because the velocities are low (<0.5m/s) throughout the system. The discharge piping of the two wells that were used during the simulation do experience slightly higher velocities, but still within acceptable ranges.

Furthermore, storage facilities are able to balance over the simulation period, which demonstrates that the well and transmission capacity are sufficient.

Figure 9 displays the fire flow availability at each junction in the hydraulic model during the maximum day demand scenario.

Figure 9: Maximum Day Demand Scenario – Fire Flow Availability

From Figure 9, the following observations can be made:

- At the discharges from all Regional facilities, there is >167L/s of fireflow available. This is critical since it demonstrates that adequate supply can be maintained at all of the connections from the Regional system to the Township of King's system. Therefore, the Regional network is deemed adequate.
- The remaining areas of lower available fire flow (<100L/s) are generally confined to two locations/situations:
  - Small diameter dead end watermains
    - This is a localized issue that should be analyzed in detail and addressed as part of the Township of King Master Plan or a similar study.
  - The pressure boosted area west of Russell Snider Drive and the Nobleton BPS. This area is currently set up such that all flow travels through the 150mm suction/discharge line from the Nobleton BPS, therefore the available fire flow is

low. However, there are two closed boundary valves that could be opened in the event of a fire to increase fire flow availability.

In order to address this fire flow deficiency, the two closed valves (on 150mm on Sunset Drive and the 150mm on King Road west of the BPS) could be converted to check valves. This would ensure the valves only allow flow transfer to the boosted area when its pressure is lower than the rest of the Nobleton system (such as during a fire event). The results of this are shown below:

Figure 10: Maximum Day Demand Scenario – Fire Flow Availability (With Check Valves)

It is noticeable that the areas that previously had low fire flow are now able to supply at least 50L/s, except at certain small diameter dead-end watermains that have significant local headloss.

In summary, the existing Nobleton system is capable of servicing the current system demands without any significant issues or bottlenecks. There is sufficient storage, well capacity and transmission main capacity to satisfy the existing Nobleton maximum day demand (44 L/s). In conjunction with the Township of King, the Region should consider adding check valves at the closed boundary valves for the boosted area in Nobleton to help increase fire flow availability.

3.2 EXISTING SYSTEM OPTIMIZATION OPPORTUNITIES

The next part of the Existing System Analysis is intended to determine whether the existing infrastructure is able to supply a total maximum day demand in Nobleton of 6.956 ML/D. This demand is equivalent to the combined permitted withdrawal limits for all three of the existing Nobleton wells.

The following section will present the results of the modeling scenario for Maximum Day Demand of 6.956 ML/D (80.5 L/s) with existing (2017) infrastructure

3.2.1 Well Capacity

As previously shown in Table 2, the combined theoretical capacity of the three existing Nobleton wells is 6.956 ML/D. This would require an increase in the combined Permit To Take Water allowance from 4.46 ML/D to 6.956 ML/D.

Furthermore, it is important to check that the three wells can simultaneously operate at their permitted flows for an extended period of time. This is confirmed hydraulically in the model, but needs to also be confirmed with the hydro-geological study as well.

In the hydraulic modeling, the maximum day demand scenario with demand of 6.956 ML/D was simulated. Table 5 summarizes the well flows that were simulated in the hydraulic model.

Table 5: Hydraulic Model – Well Flows for 6.956 ML/D Demand Scenario

WELL	AVERAGE MODELLED FLOW (L/S)	MAXIMUM MODELLED FLOW (L/S)	PERMITTED CAPACITY (L/S)
Nobleton PW #2	20.4	22.8	22.73
Nobleton PW #3	28.5	28.8	28.89
Nobleton PW #5	27.3	39.7	28.89

Table 5 shows that the average modelled flow is maintained below the permitted daily taking limit for each well. Based on the simulated flows, the storage facilities are able to maintain storage level throughout the simulation.

3.2.2 Storage Capacity

Storage capacity calculations remain the same in this scenario, except the calculations are based on the assumed Maximum Day Demand of 6.956 ML/D. Table 6 summarizes the storage capacity requirements vs. storage capacity with the simulated 6.956 ML/D demand.

Table 6: Storage Requirements for 6.956 ML/D Maximum Day Demands vs. Available Capacity

STORAGE COMPONENT	VOLUME (M^3)	NOTES
Equalization Storage Required	1,739	25% of maximum day demand
Fire Storage Required	1,200	10,000 L/min for two hours
Emergency Storage Required	735	25% of (Equalization +Fire Storage)
Total Storage Required	**3,674**	Equalization + Fire + Emergency
Current Available Storage	**3,845**	

- As seen in Table 6, there is sufficient existing storage capacity for the Nobleton system, even if maximum day demand in Nobleton increased to 6.956 ML/D (80.5 L/s) . The remaining surplus storage capacity would be 171m^3.
- The maximum day demand that would be possible whilst remaining within the existing storage capacity is 7.5ML/D (86.85 L/s).

3.2.3 Distribution System

The last step is to confirm that the watermain network is capable of distributing the increased flows from the wells to the rest of the system. Based on the hydraulic analysis of this increased well flow scenario, there are still no significant bottlenecks, pressure issues or fire flow availability issues in the system.

Figure 11 displays the minimum pressures in the hydraulic model during the increased well flow (6.956 ML/D) scenario.

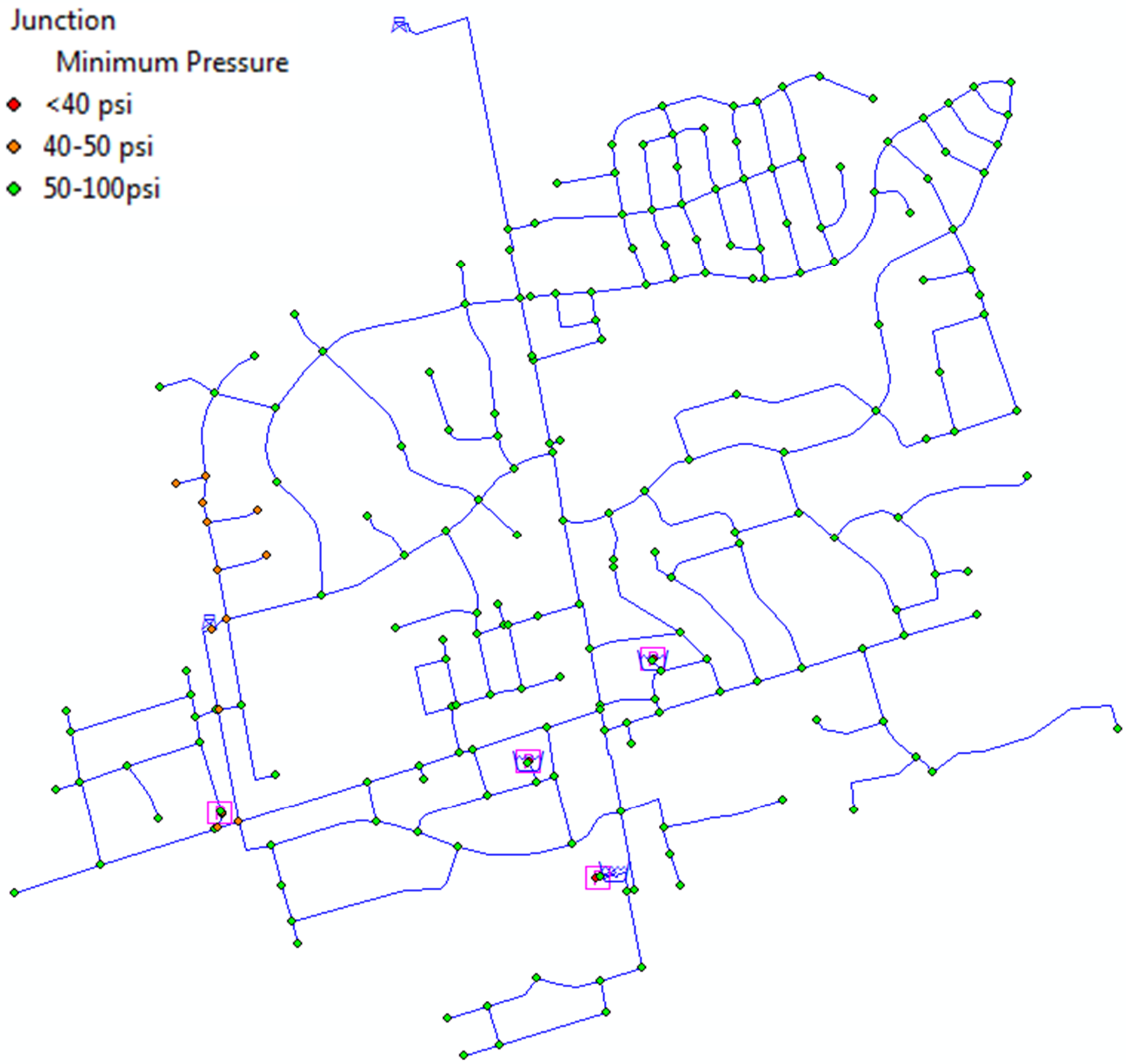


Figure 11: Increased Well Flow (6.956ML/D) Scenario – Minimum Pressure

Figure 11 shows that the area just north of the Nobleton South ET does experience marginally acceptable (40- 50psi) pressures due to being at a higher elevation than most of the system. This was previously the case; therefore there is no significant impact of the increased flow on system pressures.

Figure 12 displays the maximum velocities in the hydraulic model during the increased well flow (6.956 ML/D) scenario.

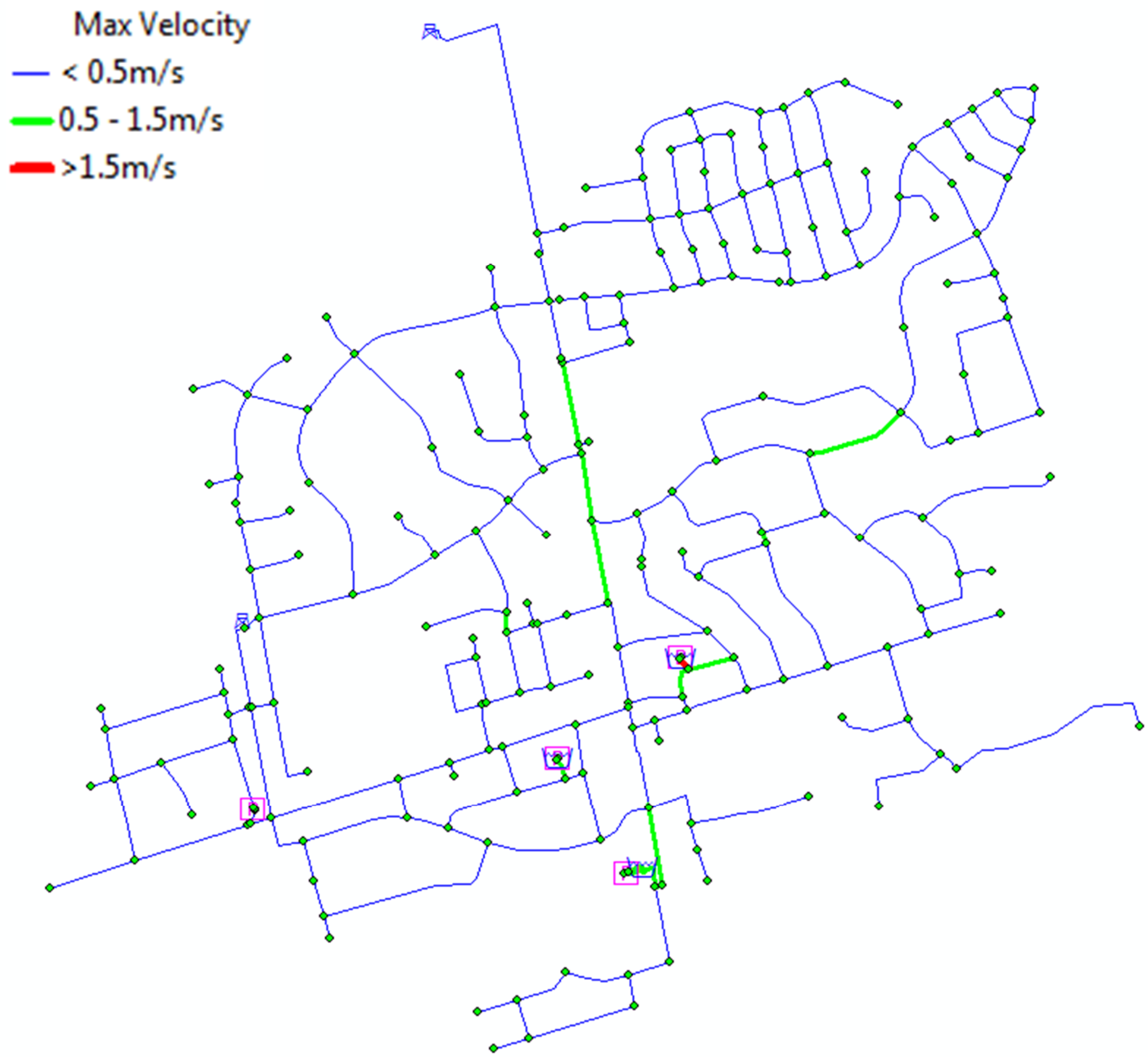


Figure 12: Increased Well Flow (6.956ML/D) Scenario – Maximum Velocity

Figure 12 shows that even with the increased flows, there are no significant bottlenecks in the existing system because the velocities are low (<0.5m/s) throughout the system. A few watermains do experience slightly higher velocities (>0.5m/s), but are still within acceptable ranges. This is expected because most of the local Nobleton system is sized for fire flow requirements.

3.3 USING SURPLUS STORAGE CAPACITY AS SUPPLY

An additional optimization opportunity involves using the surplus storage capacity that exists in the existing Nobleton system to offset minor deficiencies in the existing PTTW when system demands exceed 51.62 L/s. In order to determine how far above 51.62 L/s the Nobleton maximum day demand could go before additional infrastructure is necessary; the maximum week demands in 2016 were analyzed. By uniformly increasing the 2016 demands (44 L/s MDD) to mimic a future maximum week demand condition where maximum day demand slightly exceeds 51.62 L/s on multiple occasions, the additional storage volume required on each day can be calculated.

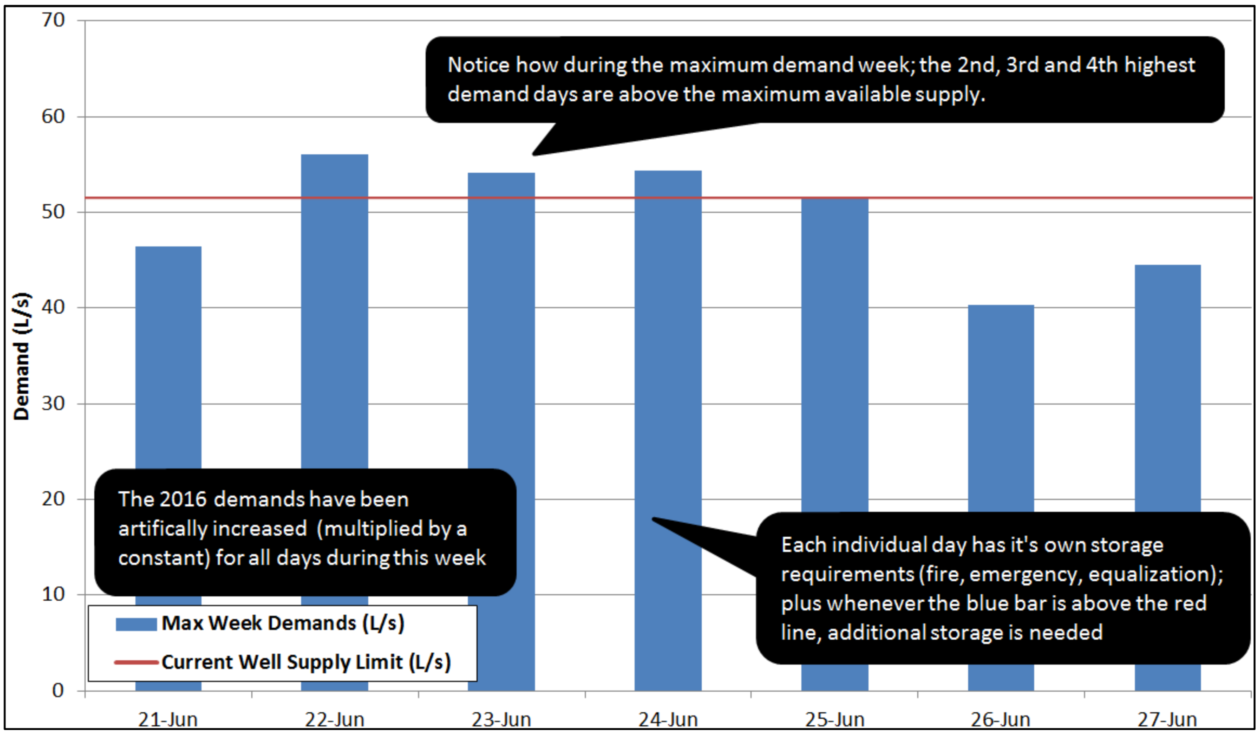


Figure 13: Theoretical Demand during Maximum Week

Figure 13 shows daily demands over a high demand week. These demands were uniformly increased from actual 2016 data to mimic a potential future maximum demand week. It is noticeable that three consecutive days are greater than the current PTTW (51.62 L/s). During each of the days when demand exceeds well capacity, additional storage volume would be required to make up the difference. Figure 14 shows the additional storage volume required to supply the additional demands over the course of each day. Furthermore, in Figure 15, it can be seen that each individual day still has their daily storage requirements based on the maximum day demand (consisting of equalization, fire and emergency volume). Figure 15 then shows how it is necessary to add the storage volumes (as calculated in Figure 14) to the required daily storage volume (as calculated in Figure 15). The sum of these volumes together needs to remain within the total available storage volume in Nobleton (3.845 ML). Based on this, it can be seen that the approximate maximum day demand that can be met after incorporating the surplus storage volume is approximately 56 L/s. However, using this volume as additional supply would be stretching the system to its absolute limit and is not recommended due to the unknowns regarding the frequency of consecutive maximum demand days, which cannot easily be predicted.

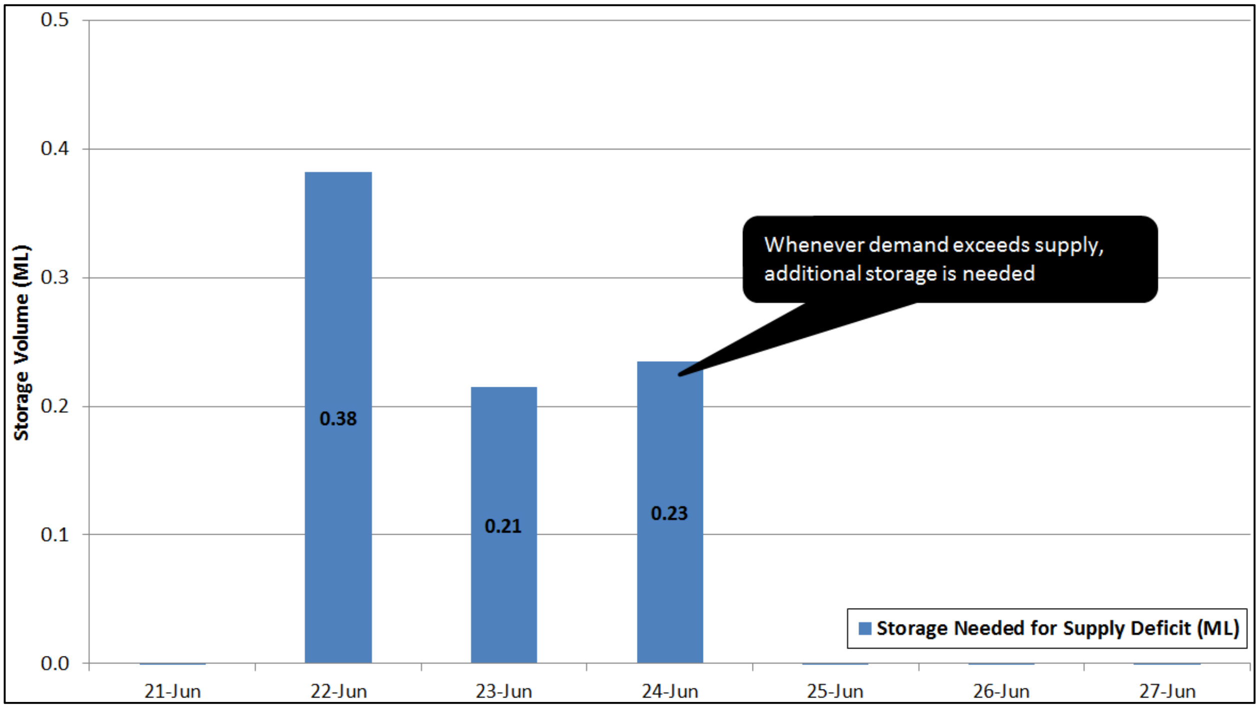


Figure 14: Storage Volumes Required to Compensate for Daily Supply Deficits

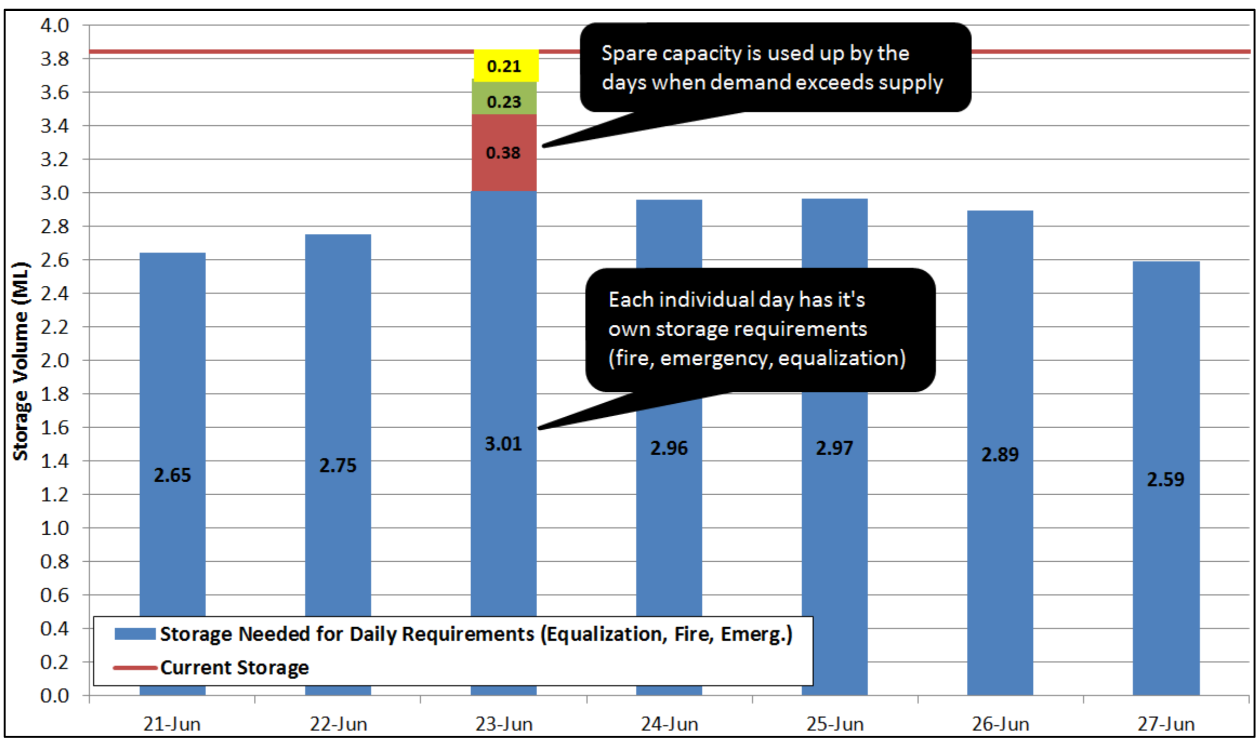


Figure 15: Daily Storage Volumes Required for Each Day (Equalization, Fire and Emergency)

4 Summary and Conclusions

The following conclusions can be made based on the results of the existing system analysis:

- There are no hydraulic issues or bottlenecks in the existing system.
- The first limitation that will arise in the Nobleton system is the combined daily taking limit (PTTW) from the three Nobleton wells.
    - The current combined daily taking limit of the Nobleton wells (51.62 L/s).
    - Maximum daily demand in 2016 was 44 L/s and was 45.5L/s in 2018.
- If an increase in the PTTW is obtained, the Nobleton system could be able to increase its maximum day demand capacity to the sum of the individual daily taking limits for the three Nobleton wells (80.51 L/s). Since it is desired that the Region's system maintains the ability to provide firm capacity (one well available as standby), this would also require the addition of a new well of at least 2.496ML/D capacity.

WELL	PERMITTED CAPACITY (ML/D)	PERMITTED CAPACITY (L/S)
Nobleton PW #2	1.964	22.731
Nobleton PW #3	2.496	28.889
Nobleton PW #5	2.496	28.889
Current Combined Daily Taking Limit (Largest Well Out)	**4.460**	**51.620**

- A hydrogeological study is required to confirm that the three existing Nobleton wells are capable of simultaneously operating at their permitted capacity without a negative impact on the groundwater supply.
- Any flow requirements beyond 80.51 L/s will require further increases to:
    - the Permit To Take Water; and
    - An increase in supply capacity from existing wells or new well(s)
- The existing storage capacity of the Nobleton system is sufficient to meet maximum day demands up to 86.85 L/s. Any flow requirements beyond 86.85 L/s will require either:
    - Additional storage capacity; or
    - Modifications to the calculations for equalization/fire/emergency storage.
- When the maximum day demand is less than ~56L/s, it is possible that the surplus storage capacity can be used to offset slight deficiencies in the existing PTTW (51.62L/s). However, this would be stretching the system to its absolute limit and is not recommended due to the unknowns regarding the frequency of consecutive maximum demand days, which are not easily predicted. With increasing drought frequency and severity, it is recommended that the surplus storage volume remains for emergencies, rather than using it to compensate for supply deficits.

5 Bibliography

MOECC. (2014, October 14). Amended Permit to Take Water Number 0550-9PPRJ9.

York Region. (2017, March 9). Design Guidelines: Section 16 - Water Systems. Newmarket, Ontario, Canada.

York Region, Environmental Services. (2012, December). Nobleton PW2 Step Test.

APPENDIX A: MODEL DEMANDS

	Existing ADD Demands							Existing MDD Demands					
	Residential Demands	Diurnal Pattern	ICI Demands	Diurnal Pattern	NRW Demands	Diurnal Pattern		Residential Demands	Diurnal Pattern	ICI Demands	Diurnal Pattern	NRW Demands	Diurnal Pattern
Model ID	Demand 1 (lps)	Pattern 1 (Char)	Demand 2 (lps)	Pattern 2 (Char)	Demand 3 (lps)	Pattern 3 (Char)	Model ID	Demand 1 (lps)	Pattern 1 (Char)	Demand 2 (lps)	Pattern 2 (Char)	Demand 3 (lps)	Pattern 3 (Char)
375-A	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	375-A	0	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
375-B	0.033	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	375-B	0.064	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-1	0.049	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-1	0.077	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-101	0.09	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-101	0.118	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-103	0.144	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-103	0.515	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-105	0.048	ADD_PATTERN_HIST	0.001	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-105	0.086	MDD_PATTERN_HIST	0.004	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-107	0.044	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-107	0.152	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-109	0.04	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-109	0.124	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-11	0.116	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-11	0.219	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-111	0.127	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-111	0.497	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-114	0.005	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-114	0.047	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-115	0.013	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-115	0.069	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-117	0.004	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-117	0.005	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-118	0	ADD_PATTERN_HIST	0.02	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-118	0	MDD_PATTERN_HIST	0.039	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
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J-179	0	ADD_PATTERN_HIST	0.167	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-179	0	MDD_PATTERN_HIST	0.313	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
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J-193	0.024	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-193	0.029	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-195	0.058	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-195	0.103	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-197	0.127	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-197	0.215	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-199	0.245	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-199	0.417	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-201	0.06	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-201	0.157	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-203	0.309	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-203	1.042	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-205	0.137	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-205	0.228	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-207	0.119	ADD_PATTERN_HIST	0.067	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-207	0.183	MDD_PATTERN_HIST	0.181	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-209	0.061	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-209	0.103	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-21	0.005	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-21	0.016	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-211	0.107	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-211	0.208	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-213	0.023	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-213	0.031	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-215	0.134	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-215	0.235	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-217	0.038	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-217	0.043	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-219	0.123	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-219	0.187	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-221	0.071	ADD_PATTERN_HIST	0.062	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-221	0.099	MDD_PATTERN_HIST	0.083	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-223	0.084	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-223	0.133	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-225	0.024	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-225	0.19	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST

	Existing ADD Demands							Existing MDD Demands					
	Residential Demands	Diurnal Pattern	ICI Demands	Diurnal Pattern	NRW Demands	Diurnal Pattern		Residential Demands	Diurnal Pattern	ICI Demands	Diurnal Pattern	NRW Demands	Diurnal Pattern
Model ID	Demand 1 (lps)	Pattern 1 (Char)	Demand 2 (lps)	Pattern 2 (Char)	Demand 3 (lps)	Pattern 3 (Char)	Model ID	Demand 1 (lps)	Pattern 1 (Char)	Demand 2 (lps)	Pattern 2 (Char)	Demand 3 (lps)	Pattern 3 (Char)
J-227	0.041	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-227	0.059	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-229	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-229	0	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-23	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-23	0	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-231	0.008	ADD_PATTERN_HIST	0.051	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-231	0.011	MDD_PATTERN_HIST	0.028	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-233	0.009	ADD_PATTERN_HIST	0.001	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-233	0.014	MDD_PATTERN_HIST	0.008	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-235	0.051	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-235	0.038	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-239	0.084	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-239	0.09	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-25	0.036	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-25	0.074	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-27	0.027	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-27	0.105	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-29	0.115	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-29	0.134	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-3	0.184	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-3	0.287	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-300	0.073	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-300	0.129	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-301	0.029	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-301	0.029	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-302	0.013	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-302	0.026	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-31	0.192	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-31	0.605	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-33	0.146	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-33	0.222	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-35	0.024	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-35	0.111	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-37	0.076	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-37	0.267	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-39	0.058	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-39	0.117	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-400	0	ADD_PATTERN_HIST	0.014	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-400	0	MDD_PATTERN_HIST	0.013	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-41	0.262	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-41	0.475	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-43	0.116	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-43	0.379	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-45	0.031	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-45	0.093	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-47	0.042	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-47	0.088	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-5	0.012	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-5	0.005	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-51	0.124	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-51	0.222	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-53	0.069	ADD_PATTERN_HIST	0.021	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-53	0.063	MDD_PATTERN_HIST	0.026	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-55	0.036	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-55	0.044	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-57	0.042	ADD_PATTERN_HIST	0.001	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-57	0.053	MDD_PATTERN_HIST	0.003	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-59	0.007	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-59	0.01	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-61	0.007	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-61	0.022	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-63	0.091	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-63	0.013	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-65	0.009	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-65	0.017	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-67	0.015	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-67	0.021	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-69	0.099	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-69	0.146	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-7	0.068	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-7	0.159	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-700	0.06	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-700	0.11	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-701	0.059	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-701	0.157	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-702	0.042	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-702	0.184	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-703	0.045	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-703	0.212	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-704	0.078	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-704	0.081	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-705	0.222	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-705	0.643	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-706	0.038	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-706	0.209	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-707	0.239	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-707	0.302	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-708	0.129	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-708	0.318	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-709	0.008	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-709	0.023	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-71	0.118	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-71	0.27	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-710	0	ADD_PATTERN_HIST	0.004	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-710	0	MDD_PATTERN_HIST	0.002	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-711	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-711	0	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-740	0.011	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-740	0.017	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-742	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-742	0	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-75	0.017	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-75	0.018	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-750	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-750	0	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-751	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-751	0	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-755	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-755	0.012	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-756	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-756	0.025	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-764	0.003	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-764	0.018	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-766	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-766	0.002	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-767	0.045	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-767	0.092	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-768	0.011	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-768	0.04	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-774	0.021	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-774	0.057	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-775	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-775	0	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-778	0.084	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-778	0.467	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-779	0.36	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-779	1.088	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-780	0.322	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-780	0.975	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-781	0.042	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-781	0.086	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-782	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-782	0	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-783	0.046	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-783	0.067	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-79	0.078	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-79	0.159	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-81	0.066	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-81	0.181	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-83	0.044	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-83	0.159	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-85	0.079	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST	J-85	0.094	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST

Existing ADD Demands						
	Residential Demands	Diurnal Pattern	ICI Demands	Diurnal Pattern	NRW Demands	Diurnal Pattern
Model ID	Demand 1 (lps)	Pattern 1 (Char)	Demand 2 (lps)	Pattern 2 (Char)	Demand 3 (lps)	Pattern 3 (Char)
J-87	0.031	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
J-89	0.032	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
J-9	0.008	ADD_PATTERN_HIST	0.011	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
J-91	0.146	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
J-93	0.209	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
J-95	0.099	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
J-97	0.035	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
J-99	0.079	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
J-RISINGST	0.089	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
N-J-1	0.008	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-1	0	ADD_PATTERN_HIST	0.011	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-10	0.169	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-11	0.188	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-12	0.13	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-13	0.263	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-14	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-15	0.027	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-16	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-17	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-18	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-19	0.042	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-2	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-20	0.134	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-21	0.158	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-22	0.169	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-23	0.117	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-24	0.123	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-25	0.112	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-26	0.1	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-27	0.006	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-28	0.133	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-29	0.08	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-3	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-36	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-39	0.096	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-4	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-40	0.161	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-41	0.171	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-42	0.122	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-43	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-44	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-45	0	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-46	0.055	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-47	0.122	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
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SD-J-52	0.074	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
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SD-J-54	0.133	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
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SD-J-56	0.134	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
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SD-J-58	0.152	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-59	0.193	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-6	0.03	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-62	0.028	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-63	0.003	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
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SD-J-65	0.124	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-68	0.271	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-7	0.013	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-72	0.09	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-73	0.007	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-74	0.103	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-76	0.125	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-8	0.073	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
SD-J-9	0.172	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST
STUPP	0.166	ADD_PATTERN_HIST	0	ADD_PATTERN_HIST	0.026	ADD_PATTERN_HIST

Existing MDD Demands						
	Residential Demands	Diurnal Pattern	ICI Demands	Diurnal Pattern	NRW Demands	Diurnal Pattern
Model ID	Demand 1 (lps)	Pattern 1 (Char)	Demand 2 (lps)	Pattern 2 (Char)	Demand 3 (lps)	Pattern 3 (Char)
J-87	0.124	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-89	0.042	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-9	0.016	MDD_PATTERN_HIST	0.019	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-91	0.405	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-93	0.474	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-95	0.286	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-97	0.109	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-99	0.143	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
J-RISINGSTA	0.083	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
N-J-1	0.032	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-1	0	MDD_PATTERN_HIST	0.012	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-10	0.401	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-11	0.488	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-12	0.261	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-13	0.455	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-14	0	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-15	0.029	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-16	0.115	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-17	0.042	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-18	0.092	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-19	0.062	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-2	0	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-20	0.288	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-21	0.312	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-22	0.365	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-23	0.209	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-24	0.281	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
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SD-J-28	0.334	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-29	0.215	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-3	0.001	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-36	0.081	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-39	0.218	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-4	0.005	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-40	0.28	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-41	0.38	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-42	0.239	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-43	0.001	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-44	0.015	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-45	0.129	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-46	0.178	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-47	0.397	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-48	0.306	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-49	0.246	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-5	0.001	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-50	0.364	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-51	0.393	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-52	0.406	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-53	0.342	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-54	0.317	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-55	0.046	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-56	0.293	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-57	0.186	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-58	0.356	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-59	0.369	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-6	0.115	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-62	0.062	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-63	0.055	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-64	0.07	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-65	0.314	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-68	1.091	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-7	0.327	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-72	0.345	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-73	0.009	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-74	0.289	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-76	0.236	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-8	0.213	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
SD-J-9	0.334	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST
STUPP	0.812	MDD_PATTERN_HIST	0	MDD_PATTERN_HIST	0.035	MDD_PATTERN_HIST

FINAL

CLASS ENVIRONMENTAL ASSESSMENT FOR WATER AND WASTEWATER SERVICING IN THE COMMUNITY OF NOBLETON

FUTURE WATER SYSTEM (PHASE 2) HYDRAULIC ANALYSIS

Study 2A

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

4 JUNE 2019

BLACK & VEATCH

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1 Introduction

1.1 PURPOSE OF STUDY

The purpose of the Phase 2 (Future Water System) Hydraulic Analysis is to:

- Evaluate the ability of the Nobleton water system to meet the projected future water - demands (supply wells, storage, and distribution); -
- Identify any hydraulic limitations (bottlenecks, etc.);

This report will be a supporting document for the Water Needs Assessment and Justification Study.

1.2 BACKGROUND

Nobleton is a community in King Township, located in York Region. Currently, Nobleton is serviced by standalone water and wastewater systems to meet the needs of the current population. The York Region Water and Wastewater Master Plan (2016) indicated that both the water and wastewater systems would not have sufficient capacity to meet requirements to support growth to the 2041 Master Plan horizon. Therefore, the Master Plan recommended undertaking the current project, a Schedule C Class Environmental Assessment (EA), to identify preferred servicing solutions to accommodate growth.

1.3 EXISTING WATER SYSTEM INFRASTRUCTURE

The Regional Municipality of York (also referred to as the Region and York Region) is responsible for the water production, treatment, storage and transmission to its local area municipalities, including the Community of Nobleton in the Township of King. The Nobleton water supply system consists of three groundwater wells and two elevated storage tanks that provide service to the Nobleton Pressure District. There is also a booster pumping station (BPS) that services a higher elevation area in the northwest portion of the distribution system. The wells operate based on level at either of the elevated tanks. The booster pumping station operates independently from the rest of the water system controls.

2 Buildout Projection

2.1 HISTORICAL DEMANDS

The following table presents the historical average and maximum day demands in the Nobleton Water System:

Table 1: Historical Water Demands in Nobleton based on SCADA (Water Production Records)

YEAR	2012	2013	2014	2015	2016	2017	2018
Average Day Demand (L/s)	13.9	14.9	14.9	16.1	**21.1**	20.4	**23.1**
Maximum Day Demand (L/s)	33.1	30.0	29.1	33.6	**44.0**	37.4	**45.5**

2.2 BUILDOUT DEMAND PROJECTION

Nobleton Water System design criteria was evaluated based on historical data in Study 1A: Water System Capacity Optimization Study. Subsequent to Workshop #2 and further discussions with York Region, the following design criteria were established:

Table 2: Water Demand Design Criteria

DESIGN CRITERIA	2016	FUTURE
Residential Population	5,520	10,800
Employment Population	772	1,800
Residential Per Capita Demand (L/cap/d)	220	220
Employment Per Capita Demand (L/cap/d)	64	182*
Non-Revenue Water %	26.5%	26.5%
ADD:MDD Peaking Factor	2.1	2.1
* Since the current Nobleton employment per capita demand is significantly lower than the remainder of York Region, it is recommended that for future employment projections the higher per capita demand rate of 182 L/cap/d be used. The type of future employment in Nobleton is currently unknown, so this will allow for slightly larger consuming employment users than those that currently exist. The selected 182 L/cap/d is based on the York Region Master Plan 2016 Employment per capita rate.		

With the above criteria established, the average and maximum day demands can be calculated and are presented in Table 3:

Table 3: Projected Future Water Demands

CATEGORY	FUTURE DEMAND (L/S)
Average Day Demand	42.6
Maximum Day Demand	89.5

The demands shown in Table 3 are established as the design basis for alternative solutions that do not include any water conservation. However, understanding that water conservation improvements could be considered as alternatives (or as a component of an alternative), the above demands may be lower in other alternative solutions.

2.3 EXISTING SYSTEM CAPACITY SUMMARY

Based on the well capacity and storage capacity in the Nobleton Water System (presented in detail in Study 1A: Water System Capacity Optimization Study), the following summarizes the current water system capacity limitations in Nobleton:

Table 4: Existing Water System Capacity Summary

CATEGORY	CAPACITY LIMIT
Nobleton Well #2	22.7 L/s
Nobleton Well #3	28.9 L/s
Nobleton Well #5	28.9 L/s
Existing Permit to Take Water Limit (Two Nobleton Wells)	51.6 L/s
Three Existing Nobleton Wells (Total Capacity, not Firm Capacity)	80.5 L/s
Nobleton North ET (m^3)	1,800 m^3
Nobleton South ET (m^3)	2,045 m^3
Total Storage Capacity (m^3)	3,845 m^3
Storage Capacity Equivalent Demand Limit	86.85 L/s

Furthermore, according to York Region's desktop assessment of the potential maximum sustainable capacity of the existing Nobleton Production Wells, it is expected that Nobleton Well 2 could have a potential capacity up to 67 L/s. with various facility upgrades (pump, treatment, etc.). Additionally, it is believed that the Nobleton Well #5 site also has potential for additional capacity. The current limiting factor at Nobleton Well #5 is the screen transmitting capacity which may not allow for any additional sustainable production. Therefore, an added well at the same site may be more feasible.

3 Model Update

3.1 BASELINE DEMAND ALLOCATION

As part of the Phase 1 hydraulic analysis, geocoded address data and historical Nobleton billing data (2015 & 2016) was used to allocate the baseline existing demands to the model. When allocating the billed demand to the model, the following process was used in InfoWater:

- Apply demand to nearest pipe and then nearest node using InfoWater Allocator tool;
- Demands were separated for residential and ICI (Industrial/Commercial/Institutional);
    - Residential demand is included in the Demand 1 column in InfoWater

  - ICI demand is included in the Demand 2 column in InfoWater
  - Non-revenue water is included in the Demand 3 column in InfoWater

- Non-revenue water was added evenly to all demand nodes. The quantity of non-revenue water was based on the difference between billed data and water production records (SCADA).

- Demand allocation for the average day demand scenario was based on the March 2016 billing data and the demand allocation for the maximum day demand scenario was based on the September 2016 billing data. These months were chosen because they demonstrated a good match to the average and maximum day demands, respectively. There were also no obvious billing data anomalies in these months.

3.2 ULTIMATE DEMAND ALLOCATION

For the future demand scenarios, the population growth (and the respective increases in demand) should be allocated to the model based on the best available planning data. However, since this project is focused on the Regional infrastructure, and not the local watermains, the exact location of the population growth within Nobleton is not critical for the following reasons:

- The Regional infrastructure in Nobleton consists primarily of the three wells, the two storage facilities and the Nobleton Booster Pumping Station (NBPS). The wells and the storage facilities are sized based on the entire Nobleton system demand, therefore, the location of population growth does not meaningfully impact their sizing.
- The NBPS is the one item that could theoretically be impacted depending on where the Nobleton growth occurs. This booster station facility serves a higher elevation part of the Nobleton service area. If areas within this high elevation area are intended to significantly increase in population (and demand), then this would impact the NBPS and would likely require an expansion of the NBPS.

Figure 1 displays a map of the existing billing data in Nobleton along with the future development parcels. Each future development parcel includes an approximation of the # of units expected there based on the Region's Planning Department's Population Projection.

These figures are intended to show that although the exact distribution of population growth is still uncertain, the parcels of land designated for future growth are well established. Furthermore, it is critical to note that these development areas do not overlap with the high elevation areas (>285m) at the west side of Nobleton that would require them to be part of the Nobleton Booster Pumping Station Zone. Any major changes to the future population distribution that moves growth into these high elevation areas would be critical since it may also lead to a need for an expansion of the Nobleton Booster PS. However, current distribution of future growth does not show any need for expansion of the Nobleton BPS.

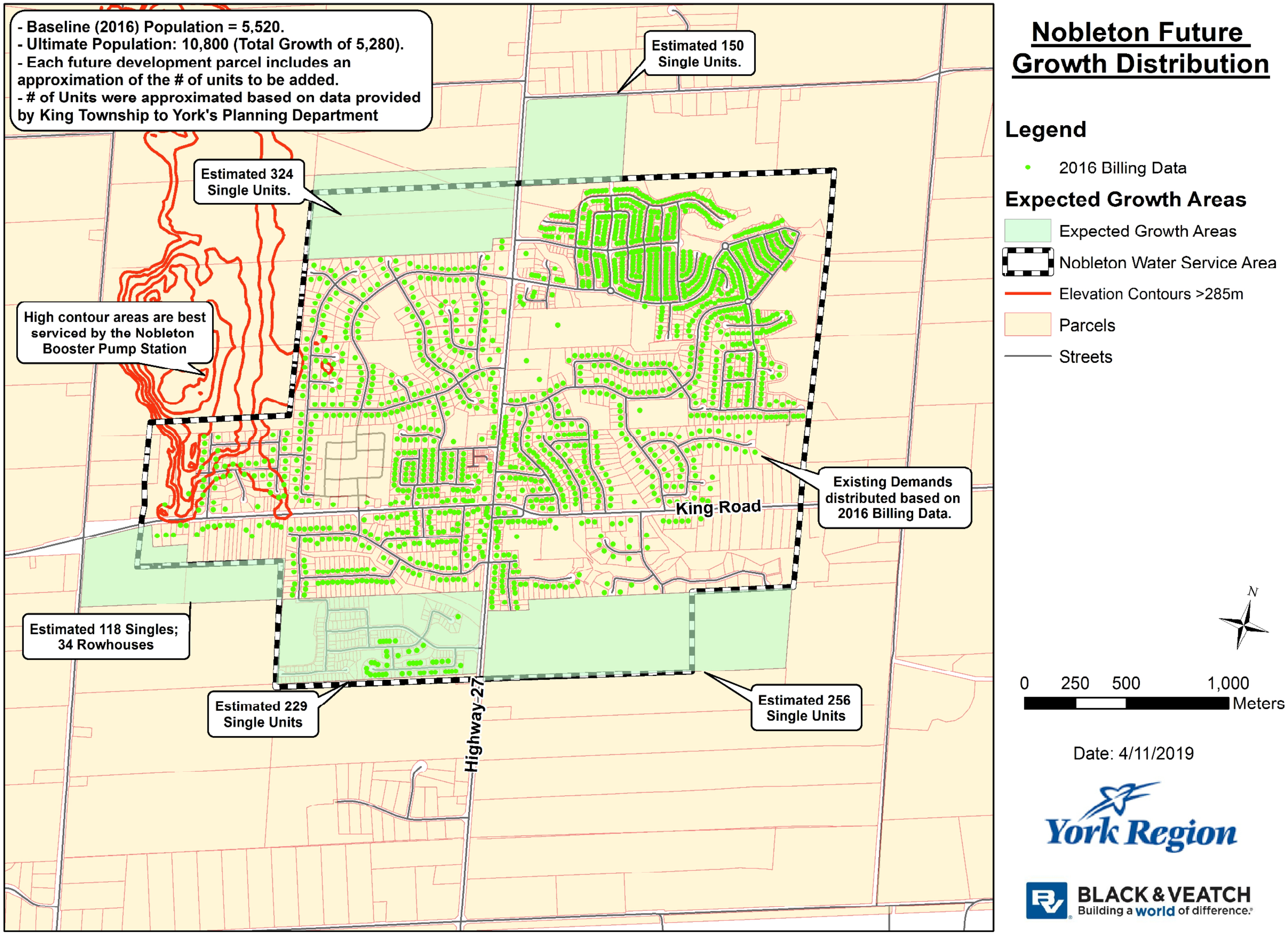

Figure 1 Areas of Expected Growth in Nobleton

4 Ultimate Water System Scenario Results

4.1 SUPPLY

Based on the existing well capacities and the projected maximum day demand of 89.5L/s, additional supply capacity is required for the Nobleton Water System. Furthermore, for the purposes of having increased system redundancy, it is critical to the Region that the well supply system be designed such that the largest well can be taken out of service during maximum day demands and still have sufficient supply capacity. So, in a scenario where water consumption operates under a "business-as-usual" approach (no further conservation), the combined existing and future Nobleton wells will require a firm capacity of at least 89.5 L/s. As part of the EA, alternative solutions could also include water conservation measures that reduce the water design criteria (per capita consumption rate, non-revenue water %, peaking factor, etc.). Various water supply alternatives will be detailed and evaluated in a later phase of the Class EA. The long list of alternative solutions will be investigated and the EA Study investigation will include various options such as:

- Do-Nothing/Limit Growth
- Implement Water Conservation Measures
- Increase Supply from Existing Well Sites
- Increase Supply from New Well Sites
- Increase Supply by connecting to Lake Based System

4.2 STORAGE

As detailed in Study 1A: Water System Capacity Optimization Study, the existing storage capacity of the Nobleton system is enough to meet the fire, emergency and equalization storage requirements that correspond to an MDD in Nobleton of up to 86.85 L/s. Since the projected maximum day demand is slightly higher than this (89.5L/s), a marginal amount of additional storage would ultimately be required. However, it is unlikely that a new storage facility would be added to make up such a small deficit. Therefore, water conservation measures (to reduce the maximum day demand to below 86.85L/s) would be considered. Alternatively, additional supply capacity could be used to offset any minor storage deficits by pumping some of the equalization storage.

4.3 DISTRIBUTION / TRANSMISSION

Based on the analysis of the ultimate maximum day demand scenario, no bottlenecks, pressure issues or fire flow availability issues are caused by limitations of the Regional infrastructure.

The only Regional watermains that may need to be added are related to the ultimate location of a new Nobleton well and the potential expansion of the Nobleton Well #2. When evaluating alternate well locations, the required connecting watermain will need to be established and documented. Additionally, if the capacity of Nobleton Well #2 is increased, then the discharge piping and connection to local piping will need to be reviewed and confirmed for appropriateness.

Based on the projected distribution of growth, which does not show growth in the high elevation areas, there is no need for an expansion of the Nobleton Booster PS.