DRAFT FINAL

PHASE 1: IDENTIFY THE PROBLEM OR OPPORTUNITY

Technical Memo #1

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

7 AUGUST 2019

BLACK & VEATCH

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1 Introduction

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.

1.1 REPORT PURPOSE

The purpose of this report is to document Phase 1 of the Environmental Assessment. This report:

1) Documents the purpose and underlying rationale for the project (Section 1)
2) Documents background information relevant to the project (Sections 2, 3, 4 and 5)
3) Confirms the schedule status of the environmental assessment (Section 6)
4) Provides a formal description of the problem (Section 7)

2 Project Background

2.1 STUDY AREA

The study area is the area within which activities associated with the study will occur and where potential environmental effects will be studied. As alternative solutions are further developed in later phases of the Environmental Assessment, the study area may be revised or expanded. The service area boundary is the current Urban Area Village of Nobleton Boundary, as defined in the Township of King's Draft Official Plan. As per Section 1.1.3 of the Provincial Policy Statement (2014), and Section 2.2.8 of the Growth Plan for the Greater Golden Horseshoe (2017), it is expected that future growth will occur within this boundary, and that the area within the boundary has, or will have in future, municipal water and wastewater servicing. The study and service areas can be found in Figure 2-1.

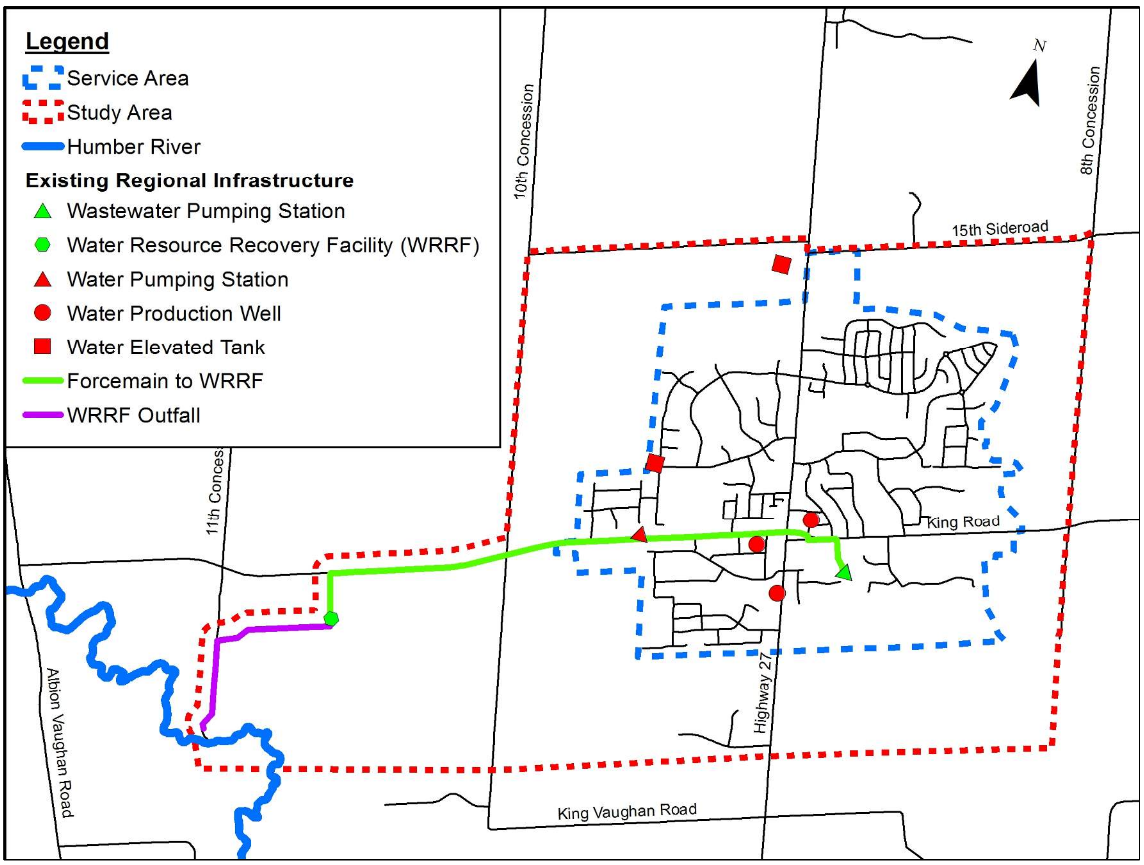

Figure 2-1: Study Area and Service Area

2.2 GROWTH IN THE NOBLETON COMMUNITY

The location and extent of population growth in the Nobleton Community is dependent on King Township's Official Plan. The current Nobleton Community Plan (an amendment to the King Township Official Plan) outlines Nobleton's urban boundary (where growth is to occur), as well as the allowable density of new development. King Township is currently undertaking an Official Plan Review, and in December 2017, and March 2019, released a draft new Official Plan for public review and comment. The final updated King Township Official Plan is expected to be released by the end of 2019.

On May 30, 2016, as part of King Township's Official Plan Review, King Council approved the recommended policy directions in a report entitled "Understanding Greenfield Density and Intensification in King Township" (Meridian Planning, 2016). This provided the framework for a potential population increase in Nobleton. Considering the land currently approved for development, as well as the allowable densities outlined in the current Nobleton Community Plan and the new Draft Official Plan, a future population of 10,800 has been estimated within the Nobleton urban boundary. A population of 10,800 will therefore be used in this study as a basis for future water and wastewater servicing requirements.

It is noted that the forecasted population identified in the King Township Official Plan (Draft, March 2019) is identified as 7000 persons. However, the plan notes that this forecast is limited to the population that can be served by the existing sanitary system. The Draft Official Plan recognizes that if all land designated for residential development and intensification were developed, the total population could reach between 9,600 and 10,900. The population to be used in this study (10,800) is considered a conservative estimate within the range outlined in the Draft Official Plan.

3 Description of Current Water Servicing and Future System Needs

3.1 CURRENT WATER SERVICING

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.

3.1.1 Water Supply

Table 3-1 provides a brief summary of the Nobleton wells. The current combined permitted daily withdrawal (Permit To Take Water) is 4.46 ML/day (51.6 L/s). This is equivalent to the sum of the capacity of Nobleton Well #2 and either 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 80.5 L/s. It is noted that Wells #3 and #5 can operate together as long as the daily limit is not exceeded.

Table 3-1: 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	
Year in Service	1960	1960	2015	
PTTW Limit (L/s)	22.7	28.9	28.9	51.6
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 is installed within the Scarborough Aquifer. The wells are developed within this stratified aquifer at depths below 83 metres below ground surface.

3.1.2 Water Storage

The Nobleton South Elevated Tank, built in 1986, has a storage volume of 2,045m^3 and is located at 117 Russell Snider Drive. The 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.

3.1.3 Water 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 total of less than 5km of 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 3-1. The Nobleton Booster Pumping Station services a higher elevation area in the northwest portion of the distribution system.

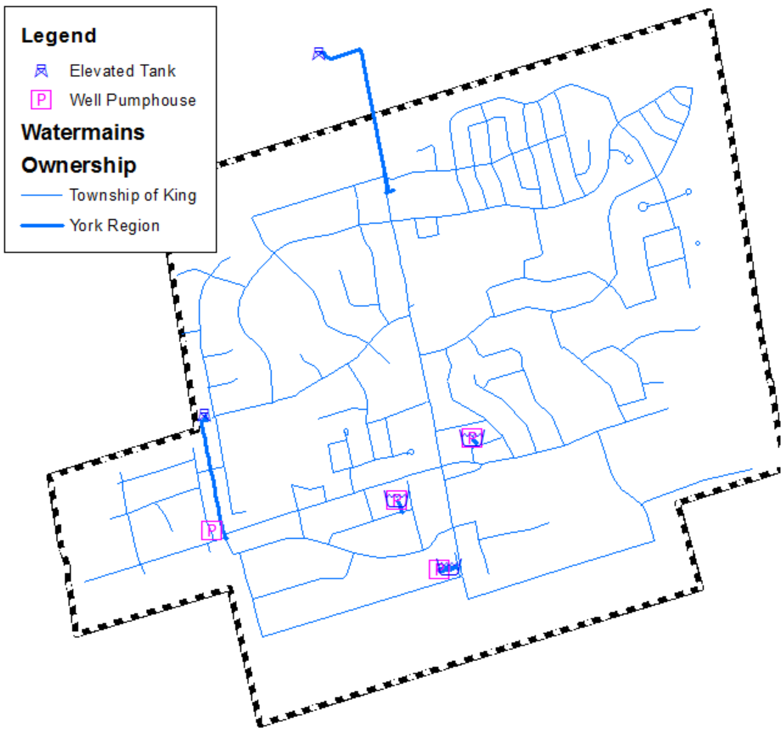

Figure 3-1: Existing Nobleton Water System

3.1.4 Existing Water System Asset Condition

On November 9, 2017, 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 staff 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.

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 staff did not note any issues with the use of Nobleton Well #3, except that they have a preference to switch treatment from sodium hypochlorite to chlorine gas.

Nobleton Well #5:

- Well #5 was commissioned in 2015 and is in generally excellent condition.
- York Region operations staff did not note any issues with the use of Nobleton Well #5.

3.1.5 Existing Water System Demands

An analysis of existing water demands on the Nobleton Water System was conducted for years 2012 through 2018. The following key points were noted:

- The system has an average day demand of 21 L/s and a maximum day demand of 44 L/s.
- Existing unit consumption rates (excluding non-revenue water) within Nobleton are as follows:
  - Residential – 220 L/cap/d
  - Employment – 64 L/cap/d
- 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, a 26.5% non-revenue water component of total system demand is assumed.

A detailed analysis of existing demands on the Nobleton water system is included in Study 1A: Water System Capacity Optimization Report (2018) found in Appendix A.

3.1.6 Existing Water System Capacity

3.1.6.1 Water Supply

The three existing Nobleton wells currently have a combined daily permitted withdrawal (Permit To Take Water) limit of 4.46 ML/D (51.6 L/s). The current limit ensures that one of the large wells

(#3 or #5) is available as a standby while the other two wells act as duty supply during maximum day demand conditions. If all three wells could operate simultaneously, then the total supply capacity could be 6.96 ML/D (80.5 L/s).

Based on the well capacity and storage capacity in the Nobleton Water System (presented in detail in Study 1A: Water System Capacity Optimization Study (2018), see Appendix A), the following summarizes the current water system capacity limitations in Nobleton:

Table 3-2: Existing Water System Capacity Summary

CATEGORY	CAPACITY LIMIT (L/S)
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 (Firm Capacity: Well #2 as well as either Well #3 or Well #5)	51.6 L/s
Three Existing Nobleton Wells (Total Capacity, not Firm Capacity)	80.5 L/s

3.1.6.2 Water Storage

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 C (Study 2A: Water System Hydraulic Analysis).

3.1.6.3 Distribution Network

Based on a hydraulic analysis of the system, there are no system bottlenecks or limitations that would prevent the Region's well supply and storage volume from being distributed to the Township of King owned infrastructure in Nobleton. At minimum, the existing distribution network is capable of servicing the equivalent of the combined capacity of the three wells PTTWs (80.5 L/s). Detailed analysis to support this can be found in Section 3.2.3 of Appendix C (Study 2A: Water System Hydraulic Analysis).

The Nobleton Booster Pumping Station is capable of servicing the current demands in the high elevation area without any significant issues or bottlenecks. With the BPS operating, pressures are generally maintained between 80psi and 100psi. Alternatively, if the BPS is shut down and the two boundary valves opened, normal pressures would be reduced to between 40psi and 60psi. This would still be above minimum guidelines, demonstrating that the currently boosted area could be supplied without the BPS. Fire flow availability would increase with the two open boundary valves compared to the active BPS. In conjunction with the Township of King, the Region should consider adding check valves at the closed boundary valves in Nobleton to help increase fire flow availability. Continuing to use the Nobleton BPS would maintain the upper range of pressures (80-100psi) during normal conditions, but the Region could consider not using the BPS if they are satisfied with maintaining a minimum pressure of 40psi.

3.2 FUTURE WATER SYSTEM NEEDS

3.2.1 Water Demand Projections

Based on a review of historical data from 2012 to 2018 and subsequent discussions with York Region staff, the following Nobleton Water System design criteria were established (Table 3-3). Details of the historical review are provided in Study 1A: Water System Capacity Optimization Study (Appendix A).

Table 3-3: 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.

Using the above criteria, the average and maximum day demands can be calculated and are presented in Table 3-4.

Table 3-4: Projected Future Water Demands

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

The demands shown in Table 3-4 are established as the design basis for alternative solutions and do not account for any water conservation. However, it is noted that water conservation improvements could be considered as alternatives (or as a component of an alternative), and, if selected, future demands would decrease.

3.2.2 Water Supply Needs

York Region's Design Guidelines require that, "[t]he total developed groundwater source capacity shall equal or exceed the design maximum day demand and equal or exceed the design average day demand with the largest producing well out of service." However, the in order to improve system resiliency, it is desired to have a firm capacity (largest well out of service) that meets the MDD of 89.5 L/s. It is noted that in order for the water supply firm capacity to exceed the MDD of 89.5 L/s, the total well yield requirements will need to exceed 89.5L/s.

Based on the existing well capacities (total of 80.5 L/s; firm of 51.6 L/s) and the projected maximum day demand of 89.5 L/s, additional water supply capacity is required for the Nobleton Water System so that the firm capacity exceeds the MDD. An increase in supply to meet future demands of 89.5 L/s could be achieved in a number of different ways, including increasing the capacity of the existing wells, adding new production wells, connecting to another water supply source or a combination of alternatives. The current system is able to provide the future average day demands. However, as part of the EA, it is understood that certain alternatives could also include water conservation measures that reduce the water design criteria (per capita consumption rate, non-revenue water %, peaking factor, etc.). Various alternatives that balance increased supply and reduced water demands will be considered as part of the Class EA.

3.2.3 Storage Needs

The existing storage capacity of the Nobleton system is sufficient to meet the fire, emergency and equalization storage requirements that correspond to a MDD in Nobleton of up to 86.85 L/s. Since the projected maximum day demand is higher (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) will be considered. Alternatively, additional well supply capacity could be used to offset any minor storage deficits by pumping some of the equalization storage.

3.2.4 Distribution / Transmission Needs

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.

The only Regional watermains that would be affected by an increase in system flows are related to the ultimate location of new water supply infrastructure. When evaluating alternate supply locations, the requirements for connected watermain(s) will be established and documented.

4 Description of Current Wastewater Servicing and Future System Needs

4.1 CURRENT WASTEWATER SERVICING

York Region is responsible for the wastewater collection and treatment from its local area municipalities, including the Community of Nobleton in the Township of King.

4.1.1 Wastewater Collection System

The Nobleton wastewater collection system consists of over 50km of gravity sewer. All of the gravity sewers in the collection system are owned by the Township of King, except for a short section of pipe, less than 50m, upstream of the Janet Avenue Pumping Station, which is owned by York Region.

There are two pumping stations within the collection system: Bluff Trail Pumping Station, owned by the Township of King; and Janet Avenue Pumping Station, owned by York Region. The Janet Avenue Pumping Station pumps all the flows from the collection system to the Nobleton Water Resource Recovery Facility via a 300mm diameter forcemain.

It is noted that the current wastewater collection system does not cover the entire community of Nobleton; some areas are still on septic tanks. There is an ongoing Township of King project to connect the remaining properties within Nobleton to the sewer system by 2021.

4.1.2 Wastewater Treatment

The Nobleton Water Resource Recovery Facility is an extended aeration plant with tertiary filtration. Its rated capacity is 2,925 m3/day with a peak design flow of 9,177 m3/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;
- Post-Secondary Treatment: 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.

4.1.3 Wastewater Flows and Generation Rates

An analysis of historical wastewater flows was conducted for years 2014 through 2017, and is summarized in Table 4-1.

Table 4-1: 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	0.83 MLD	284 L/c/d	0.88 MLD	300 L/c/d
2015	3,119	0.95 MLD	304 L/c/d	0.99 MLD	318 L/c/d
2016	3,643	1.03 MLD	283 L/c/d	1.14 MLD	313 L/c/d
2017	3,891	1.32 MLD	340 L/c/d	1.45 MLD	374 L/c/d
	Average:		303 L/c/d		326 L/c/d

It was found that, over the four years analyzed, the annual average day flow within the Nobleton wastewater system was 326 L/cap/d, and the annual average dry weather flow was 303 L/cap/d. The highest annual average flow is approximately 374 L/c/d in 2017, where higher flows were recorded due to a relatively high number of wet weather events in the summer of 2017.

In 2017, higher than average flows were recorded due to large number of wet weather events experienced in the summer. As it is likely that similarly wet years might occur in the future, 370 L/c/d was used as the basis to evaluate current system capacity, as well as project future annual average day flows for a total service population of 10,800 people.

On the basis of an average flow of 370 L/c/d and average residential wastewater generation rate (average water demand) of 220 L/c/d (assuming a 100% return ratio from the water system to the wastewater system), the estimated ongoing extraneous flows (groundwater infiltration) is approximately 150 L/c/d, approximately 40% of average daily flow.

This aligns with the numerous flow monitoring and associated investigations undertaken by the Region, where high levels of groundwater infiltration (GWI) 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.

Average peaking factors from 2014 through 2017 were calculated and are summarized in Table 4-2.

Table 4-2: Summary of Historical Raw Sewage Flows and Peaking Factors into the Nobleton WRRF

YEAR	ANNUAL AVERAGE DAILY FLOW	MAXIMUM MONTHLY FLOW(1) (PEAKING FACTOR)	PEAK DAILY FLOW (PEAKING FACTOR)	PEAK INSTANTANEOUS FLOW (PEAKING FACTOR)	PEAK HOURLY FLOW(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 is calculated using an hourly moving average with the 5-minute instantaneous flow data

The existing Nobleton WRRF experiences high peak hourly and peak instantaneous flows, with average peaking factors of 4.7 and 6.3, respectively. These peaking factors are significantly higher than the peaking factor of 3.14 used in the 2007 design.

4.1.4 Existing Wastewater System Capacity

4.1.4.1 Wastewater Collection System

Based on the hydraulic model analysis of the sewer system, it was 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 during large rainfall events but no flooding is predicted. The main location where the surcharging is predicted to occur is around the Janet Avenue PS.

At an observed peaking factor of 6.3 for the peak instantaneous flow, the Janet Avenue PS would be unable to meet the future peak instantaneous flows. This is based on the assumption that peak instantaneous flows would last until the wet well operating level reaches the high operating level. A detailed assessment is included in Study 1B: Wastewater System Capacity Optimization Study Report (Appendix B).

4.1.4.2 Wastewater Treatment

4.1.4.2.1 Wastewater Influent Loads

Current historical average day influent loads and maximum month peaking factors at the Nobleton Water Resource Recovery Facility were calculated based on average values from 2014 through 2017 and are presented in Table 4-3.

Table 4-3: Summary of Historical Influent Loadings at the Nobleton WRRF

LOADING PARAMETER	LOADING RATE (G/C/D)	AVERAGE DAY LOADING (KG/D)	MAXIMUM MONTH PEAK FACTORS
BOD	45	175	1.4
TSS	43	167	1.3
TKN	10	39	1.1
TP	1.3	5	1.2

4.1.4.2.2 Hydraulic Capacity

While the hydraulic capacity of some unit processes is assessed based on average wastewater flow, the capacity of other processes is limited by the peak daily, hourly, or instantaneous flow. As the actual average peaking factors at the WRRF are higher than the peaking factor assumed in the 2007 design of the facility (see Section 4.1.3), some existing unit processes have a capacity less than the ECA rated capacity. The summary of the results of the capacity assessment for unit process for the Nobleton WRRF is provided in Table 4-4.

Table 4-4: Summary of the Capacity Assessment for Nobleton WRRF

TREATMENT UNIT	EXISTING SYSTEM CAPACITY ASSESSMENT (M3/D)		
	AVERAGE DAY FLOW	PEAK DAY FLOW	PEAK FLOW
Screens			9,177 (instantaneous)
Grit Removal			9,177 (instantaneous)
Aeration Tanks	3,670		
Secondary Clarifiers		8,423	13,333 (hourly)
Aeration System	2,929		
Tertiary Filtration			10,490 (hourly)
UV Disinfection			9,842 (hourly)
Gravity Thickener	2,873		
Sludge Storage Tank	3,996		
Effluent Chamber and Outfall			10,500 (instantaneous)

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

- The existing Nobleton WRRF experiences high Peak Hourly Flow and Peak Instantaneous Flow, with average peaking factors of 4.3 and 6.3, respectively. These peaking factors are significantly higher than the peaking factor of 3.14 used in the 2007 design. As a result, the capacities of some process units are less than the currently rated capacity of 2,935 m^3/d.

4.2 FUTURE WASTEWATER SYSTEM NEEDS

4.2.1 Wastewater Demand Projections

Based on a review of historical data and subsequent discussions with York Region staff, the following Nobleton Wastewater System design criteria were established. Details of the historical review are provided in Study 1B: Wastewater System Capacity Optimization Study (Appendix B).

Table 4-5: Wastewater Design Flow Criteria

DESIGN FLOW CRITERIA	FUTURE
Residential Population	10,800
Wastewater Generation Rate	370 L/c/d
Average Day Flow Capacity Requirement	3,996 m^3/day
Peaking Factors	
Maximum Month Flow (MMF)	1.4
Peak Day Flow (PDF)	2.2
Peak Hour Flow (PHF)	4.7
Peak Instantaneous Flow (PIF)	6.3

4.2.2 Wastewater Collection System Needs

Based on the hydraulic model analysis of the sewer system, it is concluded that the existing trunk sewer has sufficient capacity to drain the future projected flows to the Janet Avenue PS.

At an observed peaking factor of 6.3 for the peak instantaneous flow, the Janet Avenue PS has an equivalent ADF capacity of 1,430 m^3/d and an equivalent serviceable population of 3,865 persons. This is based on the assumption that the peak instantaneous flow would last until the wet well operating level reaches the high operating level. The detailed assessment is included in Study 1B: Wastewater System Capacity Optimization Study Report (Appendix B). To meet future flows, an additional equivalent ADF of 2566 m^3/day would be required at the PS.

It is noted that the existing forcemain from the Janet Avenue PS also has insufficient capacity to accommodate the future peak flows from the collection system.

4.2.3 Wastewater Treatment Needs

4.2.3.1 Wastewater Influent Loads

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. Further information can be found in Study 1B: Wastewater System Capacity Optimization Study Report (Appendix B).

The sum of the current and future loadings have been used to determine the overall future load into the Nobleton WRRF. These loadings are summarized in Table 4-6.

Table 4-6: Summary of Expected Future Influent Loadings at the Nobleton WRRF

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

4.2.3.2 Hydraulic Capacity

The Nobleton WRRF is currently limited by the capacity of its screens and grit removal tanks. The future treatment capacity required to meet the needs of a population of 10,800 is 3,996 m^3/day. The peak instantaneous flow associated with this future ADF is 25,175 m^3/day (at a peaking factor of 6.3). The plant currently cannot treat these future flows and requires additional capacity.

This additional capacity might be achieved in several ways. For example, future flows might be reduced through the reduction of inflow and infiltration into the sewer collection system, by reducing peak flows to the plant through the addition of an EQ tank, by adding additional capacity to the existing plant's unit processes, by building a new wastewater treatment facility, or by diverting flows to another treatment facility outside of Nobleton.

5 Relevant Legislation, Plans and Policies

5.1 REGIONAL OFFICIAL PLAN (2016 OFFICE CONSOLIDATION)

York Region continues to experience rapid population and employment growth. In accordance with the York Region Official Plan (OP) significant population growth is expected within 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 purpose of the Region's Official Plan is to, "guide economic, environmental and community building decisions to manage growth". One of the Region's major goals is, "To provide the services required to support the Region's residents and businesses to 2031 and beyond, in a sustainable manner". Based on this goal, the Region's objective for water and wastewater servicing is, "To deliver safe, clean drinking water and provide long term water and wastewater services to York Region's communities, that are safe, well-managed, and sustainable". To meet this objective, the following Policies are outlined in the Region's Official Plan:

"7.3.12 To supply the Urban Area and Towns and Villages with water from the Great Lakes or from Lake Simcoe, subject to the restrictions of the Greenbelt Plan, Lake Simcoe Protection Plan, or other Provincial plans and statutes. A limited amount of groundwater resources will be used and managed in a way that sustains healthy flow into creeks, streams and rivers.

7.3.15 That development within and expansions to the urban uses within Towns and Villages […] will occur on the basis of full municipal water and wastewater treatment services where such facilities currently exist. For existing or previously approved development in Towns and Villages, water and wastewater treatment services will be continued where feasible and in keeping with the provisions of local official plans and this Plan.

7.3.16 That within the Oak Ridges Moraine, Greenbelt, and Lake Simcoe watershed, all improvements or new water and wastewater infrastructure systems shall conform with the Oak Ridges Moraine Conservation Plan, the Greenbelt Plan or the Lake Simcoe Protection Plan.

7.3.17 That the construction or expansion of partial services is prohibited in the Oak Ridges Moraine unless it has been deemed necessary to address a serious health or environmental concern identified by the Medical Officer of Health or other designated authority.

7.3.18 To provide reliable water and wastewater services to residents and businesses to ensure continuing community well-being and the economic vitality of the Region.

7.3.25 To ensure that wastewater effluent is managed to minimize impacts on the quality of the receiving water.

7.3.27 To incorporate energy-recovery systems into water and wastewater facilities where possible in order to reduce the health and environmental impacts of greenhouse gas and other emissions on air quality.

7.3.30 That the planning and design of water and wastewater infrastructure will consider potential impacts from climate change.

7.3.31 To ensure secure and resilient Regional water and wastewater systems to maintain continual service.

7.3.32 That water and wastewater services will be planned, constructed and operated in a manner that protects, enhances, and provides net benefit to the Region's natural and cultural heritage.

7.3.34 That the water and wastewater systems be sized to consider the potential for expansion of the service area, intensification and increased allocation where permitted by York Region Master Plans and Provincial Plans."

The Region has also developed the following policy related to water and wastewater servicing:

"5.6.21 That within the Greenbelt Plan Area, the following policies apply to Towns and Villages:

a. that where Towns or Villages do not currently have Lake Ontario or Lake Simcoe based water and wastewater services, extensions to or expansions of existing lake-based services is prohibited, unless the servicing is required to address failed individual on-site sewage or water services or to ensure protection of public health as determined by the Medical Officer of Health. The capacity of water and wastewater services in this case will be limited to the servicing requirements for the existing settlement plus capacity for potential development within the approved settlement boundary as it existing on the date the Greenbelt plan came into effect"

In addition to the policies outlined above, 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. Additional development and population growth will require an amendment to the Official Plan and can be considered when the Township completes its next municipal comprehensive review to the planning horizon of 2041.

<u>Relevance to EA:</u>

The Official Plan is relevant to the Class EA study as it outlines the policies that guide the economic, environmental and community building decisions to manage growth. It emphasizes the need to develop water and wastewater services that support the economic growth of the Region while protecting the Region's natural and cultural heritage.

5.2 YORK REGION CORPORATE STRATEGIC PLAN (2015)

The 2015-2019 York Region Strategic Plan is a roadmap that guides toward the vision of the future. It serves as a plan to get the Region from where they are to where they want to be in 2051 and focuses on Economic Vitality, Healthy Communities, Sustainable Environment and Good Government.

The key Regional Performance Measures listed in the Strategic Plan that relate to the Nobleton Water and Wastewater Servicing Class EA are the following:

- Maintain percentage of treated water returned to environment within regulated standards;
- Reduce quantity of inflow and infiltration in Regional and local wastewater systems;
- Decrease average residential water demand.

Relevance to EA:

The Region's Corporate Strategic Plan is relevant to the Class EA because it emphasizes key performance measures for water and wastewater systems, including an emphasis on reducing inflow and infiltration and reducing residential water demands.

5.3 KING TOWNSHIP DRAFT OFFICIAL PLAN (2017)

The purpose of the King Township Official Plan is to provide direction and a policy framework for managing growth, land use and infrastructure decisions. 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). 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 2019.

The Draft Official Plan notes the following regarding the Nobleton community:

- The population forecast for Nobleton reflects limitations posed by the municipal sanitary sewer services that can accommodate a total population in Nobleton of 6,750 to 7,000 to 2031. [Section 2.3.2.4, Draft King Township Official Plan]
- Notwithstanding the above, 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. [Section 2.3.2.5, Draft King Township Official Plan]
- Additional development and population growth will require an amendment to the Official Plan and can be considered when the Township completes its next municipal comprehensive review to the planning horizon of 2041. In addition to an amendment to the Official Plan, the additional development described above will also require a servicing solution to the satisfaction of the Township of King and Region of York.

Relevance to EA:

The King Township Official Plan is relevant to the Class EA because it specifies the limitations and framework for Nobleton's population growth.

5.4 WATER AND WASTEWATER MASTER PLAN (2016)

This document reports on the update of the Water and Wastewater Master Plan for The Regional Municipality of York. The updated Master Plan will guide investments in water and wastewater systems to support the Region's projected growth to 2041. The Master Plan had the following major objectives that relate to the Class EA:

- Develop a cost-effective, resilient water and wastewater infrastructure plan to service future growth to 2041 and beyond
- Develop an integrated, long-term strategy to provide sustainable water and wastewater services

The Master Plan also noted the following regarding stand-alone communities:

- Communities currently serviced by stand-alone water and/or wastewater systems will continue to be serviced by stand-alone systems. These include Keswick and Sutton (Town of Georgina), Mount Albert (Town of East Gwillimbury), Ballantrae (Town of Whitchurch-Stouffville), Ansnorveldt, Nobleton and Schomberg (Township of King). Kleinburg Water Resource Recovery Facility will continue to service new developments up to its permitted capacity, after which all new developments will be serviced by the York Durham Sewage System.

This Class EA is intended to address the following projects included in the Water and Wastewater Master Plan:

- W28 – Nobleton Water System Expansion
- WW21 – Nobleton Water Resource Recovery Facility Expansion

Further to the Master Plan, York also developed the "One Water Action Plan" which includes the following action areas:

1. Implement the Long-Term Water Conservation Strategy and Water Reuse
2. Implement Inflow and Infiltration Reduction
3. Enhance Integration of Asset Renewal with Growth Projects
4. Develop Climate Change Adaptation and Mitigation Strategies
5. Continue Energy Optimization and Renewable Energy Initiatives; and
6. Ensure Financial Sustainability.

<u>Relevance to EA:</u>

The Region's Water and Wastewater Master Plan is relevant to the Class EA because it serves as the guiding document on water and wastewater system investments to 2041. It specifically mentions the desire to continue servicing stand-alone systems as stand-alone systems.

5.5 PROVINCIAL POLICY STATEMENT (2014)

The Provincial Policy Statement provides policy direction on matters of provincial interest related to land use planning and development. As a key part of Ontario's policy-led planning system, the Provincial Policy Statement sets the policy foundation for regulating the development and use of land. It also supports the provincial goal to enhance the quality of life for all Ontarians.

The following key policies from the 2014 Provincial Policy Statement are summarized below:

- 1.6.6.1 - Planning for sewage and water services shall:
  - direct and accommodate expected growth or development in a manner that promotes the efficient use and optimization of existing:
    1. municipal sewage services and municipal water services; and
    2. private communal sewage services and private communal water services, where municipal sewage services and municipal water services are not available;
  - ensure that these systems are provided in a manner that:
    1. can be sustained by the water resources upon which such services rely;
    2. is feasible, financially viable and complies with all regulatory requirements; and
    3. protects human health and the natural environment;
  - promote water conservation and water use efficiency.

Relevance to EA:

The Provincial Policy Statement is relevant to the Class EA because it again emphasizes the need to develop water and wastewater services to meet the expected growth, while sustaining our water resources and protecting the natural and cultural environment.

5.6 GREENBELT PLAN (2017)

The Greenbelt Plan, together with the Oak Ridges Moraine Conservation Plan (2017) and the Growth Plan for the Greater Golden Horseshoe (2017) establishes a land use planning framework for the area, and identifies areas where urbanization should not occur.

The Community of Nobleton is denoted as a Town/Village in the Protected Countryside of the Greenbelt Area. It is surrounded on all sides by Protected Countryside Areas, therefore any proposed infrastructure must satisfy the policies set forth in the Greenbelt Plan (particularly Section 4.2). The following policies therefore apply:

4.2.1 General Infrastructure Policies

For lands falling within the Protected Countryside, the following policies shall apply:

2.The location and construction of infrastructure and expansions, extensions, operations and maintenance of infrastructure in the Protected Countryside are subject to the following:

a) Planning, design and construction practices shall minimize, wherever possible, the amount of the Greenbelt, and particularly the Natural

Heritage System and Water Resource System, traversed and/or occupied by such infrastructure;

b) Planning, design and construction practices shall minimize, wherever possible, the negative impacts on and disturbance of the existing landscape, including, but not limited to, impacts caused by light intrusion, noise and road salt;

c) Where practicable, existing capacity and co-ordination with different infrastructure services shall be optimized so that the rural and existing character of the Protected Countryside and the overall hierarchy of areas where growth will be accommodated in the GGH established by the Greenbelt Plan and the Growth Plan are supported and reinforced;

4.2.2 Sewage and Water Infrastructure Policies

In addition to the policies of section 4.2.1, for sewage and water infrastructure in the Protected Countryside the following policies shall apply:

1. Planning, design and construction of sewage and water infrastructure shall be carried out in accordance with the policies in subsection 3.2.6 of the Growth Plan

2. The extension of municipal or private communal sewage or water services outside of a settlement area boundary shall only be permitted in the case of health issues or to service existing uses and the expansion thereof adjacent to the settlement area. Notwithstanding the above, where municipal water services exist outside of settlement areas, existing uses within the service area boundary as defined by the environmental assessment may be connected to such a service.

<u>Relevance to EA:</u>

The Greenbelt Plan is relevant to the EA as it constrains the extension of municipal sewage or water services outside of the Nobleton community, and only permits such extensions in the case of health issues, or to service existing uses and their expansion adjacent to the community.

5.7 GROWTH PLAN FOR THE GREATER GOLDEN HORSESHOE (2017)

The Growth Plan for the Greater Golden Horseshoe works in conjunction with the Greenbelt Plan, the Oak Ridges Moraine Conservation Plan, and the Niagara Escarpment Plan to provide a framework for growth, as well as key growth management goals, for the Greater Golden Horseshoe area. The Greater Golden Horseshoe area includes all of York Region, including the community of Nobleton.

The following policies in the Growth Plan apply to this study:

3.2.6.2. Municipal water and wastewater systems and private communal water and wastewater systems will be planned, designed, constructed or expanded in accordance with the following:

a. opportunities for optimization and improved efficiency within existing systems will be prioritized and supported by strategies for energy and water conservation and water demand management;

b. the system will serve growth in a manner that supports achievement of the minimum intensification and density targets in this Plan;

3.2.6.3. For settlement areas that are serviced by rivers, inland lakes, or groundwater, municipalities will not be permitted to extend water or wastewater services from a Great Lakes source unless:

a. the extension is required for reasons of public health and safety, in which case, the capacity of the water or wastewater services provided in these circumstances will be limited to that required to service the affected settlement area, including capacity for planned development within the approved settlement area boundary;

b. in the case of an upper- or single-tier municipality with an urban growth centre outside of the Greenbelt Area:
   i. the need for the extension has been demonstrated;
   ii. the increased servicing capacity will only be allocated to settlement areas with urban growth centres; and
   iii. the municipality has completed the applicable environmental assessment process in accordance with the Ontario Environmental Assessment Act;

c. the extension had all necessary approvals as of July 1, 2017 and is only to service growth within the settlement area boundary delineated in the official plan that is approved and in effect as of that date.

Relevance to EA:

The Growth Plan for the Greater Golden Horseshoe is relevant to this EA as it limits the extension of water or wastewater services from a Great Lakes Source if the settlement area is serviced by groundwater. In the case of the community of Nobleton, the Growth Plan states that an extension would not be permitted unless required for reasons of public health and safety.

5.8 OAK RIDGES MORAINE CONSERVATION PLAN (2017)

The purpose of the Oak Ridges Moraine Conservation Plan is to provide land use and resource management planning direction to provincial ministers, ministries, and agencies, municipalities, landowners and other stakeholders on how to protect the Moraine's ecological and hydrological features and functions.

The north portion of the community of Nobleton is designated as a settlement area under the Oak Ridges Moraine Conservation Plan, and the area north of Nobleton is considered a "Natural Core Area" and "Natural Linkage Area" under the Oak Ridges Moraine Conservation Plan.

The Oak Ridges Moraine Conservation Plan states that new infrastructure corridors or facilities will only be allowed in the Natural Core Areas and Natural Linkage Areas if they are shown to be necessary and there is no reasonable alternative. These new infrastructure corridors would also have to meet stringent review and approval standards. The policies outlining this are as follows:

11 (3) The following uses are permitted with respect to land in Natural Core Areas, subject to Parts III and IV:

4. Infrastructure uses.

12 (3) The following uses are permitted with respect to land in Natural Linkage Areas, subject to Parts III and IV:

4. Infrastructure uses.

41 (2) An application for the development of infrastructure in or on land in a Natural Linkage Area shall not be approved unless,

(a) the need for the project has been demonstrated and there is no reasonable alternative;

41 (5) Infrastructure may be permitted to cross a key natural heritage feature or a hydrologic feature if the applicant demonstrates that,

(a) the need for the project has been demonstrated and there is no reasonable alternative

Relevance to EA:

The Oak Ridges Moraine Conservation Plan is relevant to the Class EA because the Oak Ridges Moraine is located at the northeast portion of Nobleton. Within these areas, certain restrictions exist both in terms of land use and infrastructure which need to be considered.

5.9 HUMBER RIVER WATERSHED PLAN (2008)

The Humber River Watershed Plan – Pathways to a Healthy Humber (2008), was prepared by the Toronto and Region Conservation Authority (TRCA), in partnership with municipal, provincial and federal government representatives and other stakeholders including the Humber Watershed Alliance. The Watershed Plan provides guidance to local, regional and provincial governments and the TRCA as they update their policies and programs for environmental protection, conservation, and restoration within the contexts of land and water use, and the planning of future development. It also provides direction to local non-governmental organizations and private landowners with regard to best management practices and opportunities for environmental stewardship. The Watershed Plan is based on a strong understanding of current conditions developed through analysis of environmental monitoring information, combined with leading edge approaches to predicting potential future conditions that involved modelling and expert input.

Relevance to EA:

The Humber River Watershed Plan is relevant to the Class EA because the current water reclamation facility discharges to the Humber River. Therefore, any changes in discharge quantity or quality needs to be analyzed and discussed in collaboration with the TRCA.

5.10 GREAT LAKES – ST. LAWRENCE RIVER BASIN SUSTAINABLE WATER RESOURCES AGREEMENT (INTRA-BASIN TRANSFER OF WATER) (2007)

The Ontario Water Resources Act, 1990 as amended by the Safeguarding and Sustaining Ontario's Water Act, 2007, bans transfers of water from one Great Lakes watershed to another except under strictly regulated conditions. This is a challenge for the Region, because it straddles the Lake Huron (Simcoe) and Lake Ontario watersheds. The Region has received permission to transfer no more than 105 million litres a day of water and must meet ongoing conditions for this transfer.

Currently, all water originating in Nobleton is maintained within the Lake Ontario (Humber River) Watershed, therefore it does not impact the intra-basin transfer limit.

<u>Relevance to EA:</u>

The Intra-Basin Transfer Agreement is relevant to the Class EA because it emphasizes the need to maintain a balance between the Lake Ontario and Lake Huron watersheds. Currently, all water originating in Nobleton is maintained within the Lake Ontario watershed. When developing other servicing alternatives for the community of Nobleton, this agreement must be considered.

5.11 CLEAN WATER ACT (2006)

The Clean Water Act (2006) protects Ontario's drinking water resources by delineating vulnerable areas around surface water intakes and wellheads. These vulnerable areas are known as Wellhead Protection Areas (WHPAs) and Intake Protection Zones (IPZs). In the case of the community of Nobleton, these areas are detailed in the Approved Source Protection Plan prepared by the CTC Source Protection Region.

<u>Relevance to EA:</u>

As this project includes the development of wastewater servicing alternatives for the Community of Nobleton, and the operation or maintenance of a system that collects, stores, transmits, treats or disposes of sewage is an identified threat to drinking water sources, WHPAs within the study area have been identified (see Figure 5-1), and will be considered within the Municipal Class EA. There are no IPZs within the study area. It is also noted that the identification of any new water source as part of the preferred solution may require delineation of new WHPAs or IPZs.

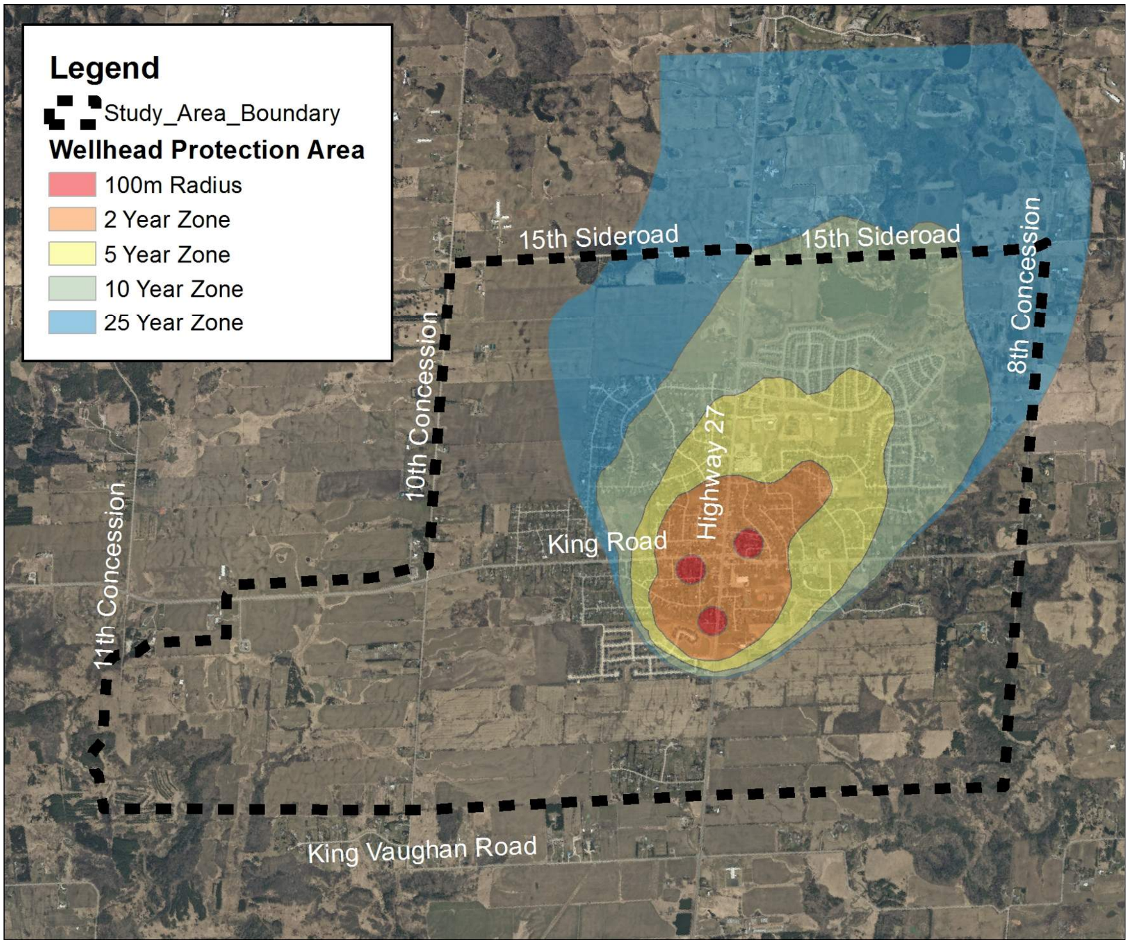

Figure 5-1: Wellhead Protection Areas within the Study Area

6 The Municipal Class EA Process

Under the Ontario Environmental Assessment Act, complex projects that have the potential to cause adverse environmental impacts, minimal or significant, with major public interest, must prepare a Municipal Class Environmental Assessment (MCEA) to be approved by the Ministry of the Environment, Conservation and Parks (MECP). The Ontario Municipal Engineers Association (MEA) provides a framework under which Municipal Class Environmental Assessments (MCEAs) are undertaken.

An MCEA is a streamlined planning process to produce an environmental assessment where the applicable projects are of routine nature with predictable and manageable environmental effects and is one that includes municipal road, water, and sewer projects. Projects can vary in their environmental impacts and are categorized in schedules, as shown below in Table 6-1.

Table 6-1: Description of the Class of Undertakings

SCHEDULE	DESCRIPTION
Schedule A	■ Pre-approved projects as the environmental impacts are minimal (e.g. normal or emergency operational and maintenance activities).
Schedule A+	■ Pre-approved projects that must advise public prior to implementation.
Schedule B	■ Potential for adverse environmental impacts. ■ Proponent is required to proceed with a screening process involving mandatory consultation with those affected (e.g. public, review agencies). ■ Projects generally include minor expansions and improvements to existing facilities.
Schedule C	■ Potential for significant adverse environmental impacts ■ Proponent is required to proceed with a full MCEA planning and documentation process as outlined in the MCEA. The Environmental Study Report (ESR) must be prepared and filed for review by the public and review agencies. ■ Projects generally include major expansions to existing facilities or the construction of new facilities

6.1 CONFIRMATION OF PROJECT SCHEDULE

As a significant increase in capacity is required for both the water and wastewater systems to meet expected future growth, a major expansion to existing facilities, or the construction of new facilities may be required. As per Table 6-1 above, the requirement for a Schedule C Class EA is confirmed.

6.2 CANADIAN ENVIRONMENTAL ASSESSMENT ACT

The need for a study to be conducted under the Canadian Environmental Assessment Act (CEAA) can be triggered by municipal-level projects if the following requirements are met:

- Provision of federal funding
- Requirement for federal land
- Requirement for federal approval (e.g. Fisheries Act, Species at Risk Act, or any other applicable federal acts)

Currently, no federal funding of the project is anticipated; therefore, a CEAA would not be required for this reason.

The infrastructure in the existing Nobleton water and wastewater systems are either located on property owned by York Region, or are located within easements. While the location(s) of any future water or wastewater infrastructure resulting from this project have not been identified at this stage in the Environmental Assessment, it is not anticipated that federal land will be required and that a CEAA will be triggered for this reason.

While there is a reasonable likelihood the project will impact the Humber River, and the Fisheries Act may be a regulatory trigger for a CEAA, the Toronto and Region Conservation Authority has been designated by the Department of Fisheries and Ocean as the first point of contact for the project, and will provide guidance if authorization and/or assessment under the CEAA is required.

Another possible regulatory trigger under the CEAA is the federal Species at Risk Act. While it is not currently anticipated that the project will have impacts on any species at risk, further assessment of potential impacts will be undertaken at later phases in the Municipal Class Environmental Assessment and the need for a CEAA may be triggered at that time.

7 Problem / Opportunity Statement

As part of Phase 1 of the Municipal Class Environmental Assessment process, a problem or opportunity statement must be identified. In general, projects are undertaken to address identified problems or deficiencies, or because of an opportunity that had been previously defined. This statement must encompass the entire project, and in this case, is therefore common to both water and wastewater infrastructure.

The problem/opportunity statement for this MCEA is as follows:

"Identify innovative, safe and reliable water and wastewater servicing solutions for the community of Nobleton in King Township, to support approved population growth to 10,800 persons, while optimizing the use of existing systems. The preferred solution must be socially, environmentally and financially sustainable."

A more concise form of the problem/opportunity statement was developed for public consultation purposes, and is as follows:

"Develop long-term water and wastewater servicing solutions to support current and future residents in the Nobleton community"

8 References

Black & Veatch. (2018). *Nobleton System Capacity Optimization Study (Water).*

Black & Veatch. (2018). *Water Needs Assessment & Justification Study.*

Black & Veatch. (2019). *Nobleton System Capacity Optimization Study (Wastewater).*

Black & Veatch. (2019). *Wastewater Needs Assessment & Justification Study.*

Civica, Municipal Water Resources. (2016). *Sanitary Sewer Investigation within the Village of Nobleton.* Township of King: The Regional Municipality of York.

Meridian Planning. (2016). *Understanding Greenfield and Intensification Densities in King Township.*

Meridian Planning. (2017). *King Township Official Plan - Draft.*

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

Province of Ontario. (2006). *Clean Water Act.*

Province of Ontario. (2007). *Safeguarding and Sustaining Ontario's Water Act.*

Province of Ontario. (2014). *Provincial Policy Statement.*

Province of Ontario. (2017). *Greenbelt Plan.*

Province of Ontario. (2017). *Growth Plan for the Greater Golden Horseshoe.*

Province of Ontario. (2017). *Oak Ridges Moraine Conservation Plan.*

The Regional Municipality of York. (2016). *Official Plan - 2016 Office Consolidation.*

The Regional Municipality of York. (2016). *Water and Wastewater Master Plan.*

Toronto and Region Conservation Authority. (2008). *Humber River Watershed Plan.*

Yaku / Associated Engineering / Pro F&E. (2014, May). Nobleton Well #2 Condition Assessment Report. - Ontario, Canada.

Yaku / Associated Engineering / Pro F&E. (2014, May). Nobleton Well #3 Condition Assessment Report. Ontario, Canada.

York Region. (2013). Nobleton Well #2 Operation Manual. Ontario, Canada.

York Region. (2015). *Corporate Strategic Plan - 2015 to 2019.*

York Region. (2015). Nobleton Well #5 Operation Manual. Ontario, Canada.

York Region. (2016). Nobleton Well #3 Operation Manual. Ontario, Canada.

PHASE 2: IDENTIFY ALTERNATIVE SOLUTIONS

Technical Memo #2

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

5 MARCH 2021

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ADD	Average Day Demand
ADF	Average Day Flow (Annual)
DWWP	Drinking Water Works Permit
EA	Environmental Assessment
I/I	Inflow and Infiltration
km	Kilometer
L/s	Liters per second
MECP	Ministry of Environment, Conservation and Parks
m^3/day	cubic meters per day
MDD	Max Day Demand
ML	Million Litres
MLD	million liters per day
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
RDII	Rainfall Derived Infiltration and Inflow
TM	Technical Memorandum
WWF	Wet Weather Flow
WRRF	Water Resource Recovery Facility

1 Introduction

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 the water and wastewater systems would have insufficient capacity to meet the requirements to support growth to the 2041 Master Plan horizon. Therefore, the Master Plan recommended undertaking a Schedule C Class Environmental Assessment (EA), to identify servicing solutions to accommodate growth.

The purpose of Technical Memorandum 2 (TM2) is to identify alternative water and wastewater servicing solutions, and to provide a recommended solution for servicing the community of Nobleton.

1.1 SUMMARY OF WORK PREVIOUSLY COMPLETED

Black & Veatch submitted Technical Memorandum 1 (TM1): *Phase 1: Identify the Problem or Opportunity,* dated June 4, 2019. TM1 identified an opportunity to develop long-term water and wastewater servicing solutions to support the current and forecasted population growth in the community of Nobleton to 10,800 persons. Various water and wastewater studies were conducted in order to provide the supporting evidence for TM1. The previous studies completed for the water and wastewater systems are summarized in Figures 1-1 and 1-2, respectively.

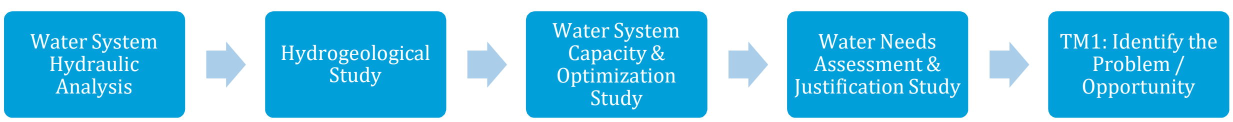

Figure 1-1: Summary of Previous Water System Studies

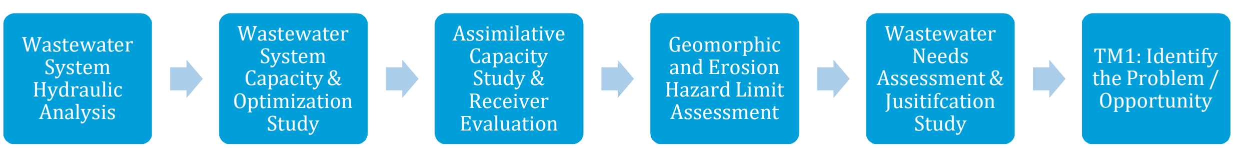

Figure 1-2: Summary of Previous Wastewater System Studies

1.1.1 Water System Future Capacity Needs Summary

Black & Veatch conducted a detailed water system capacity assessment in Study 1A: Water System Capacity Optimization Study. Table 1-1 summarizes the existing water system capacity and the forecasted future water system demands.

Table 1-1: Summary of Existing Limits and Future Demand for the Nobleton Water System

EXISTING WATER SYSTEM	CURRENT CAPACITY / FUTURE DEMAND
Well #2 Capacity	22.7 L/s
Well #3 Capacity	28.9 L/s
Well #5 Capacity	28.9 L/s
Well Supply Firm Capacity (Permit to Take Water: Largest Unit Out of Service)	51.6 L/s
Water Storage Capacity (Existing storage volume is converted to the equivalent Maximum Day Demand that it can currently service)	87.40 L/s
Forecasted Future Average Day Demand (ADD)	42.6 L/s
Forecasted Future Maximum Day Demand (MDD)	89.5 L/s

Table 1-1 demonstrates that the combined capacity of the three existing Nobleton Wells (#2, #3 and #5) would be 80.5L/s. However, the current Permit to Take Water for the Nobleton Wells not only limits the individual wells to stay within their individual capacities, but it also limits the combined capacity of the three wells. This combined Permit to Take Water capacity is equivalent to the firm capacity of the Nobleton wells. Firm capacity is the sum of the well capacities, except with the largest unit out of service. In this case, that would mean that Well #3 or Well #5 is assumed to be out of service (or on standby), so the current combined daily limit is only 51.6 L/s.

The Permit to Take Water (PTTW) limit and the firm capacity of the existing Nobleton wells is well below the forecasted maximum day demand (MDD) of 89.5 L/s. Therefore, additional water supply is required to meet the forecasted growth. To address this need, various water supply alternatives will be developed and evaluated in this technical memorandum.

In terms of storage capacity, the existing Nobleton system has storage volume capable of providing storage requirements (fire, equalization and emergency storage) up to the equivalent of a maximum day demand of 87.40 L/s. Since the projected MDD is 89.5 L/s, this means that there would ultimately be a marginal storage deficit if no action was taken. In terms of a storage volume, this is equivalent to a storage need of 3.916ML compared to an existing capacity of 3.860 ML (marginal deficit of 0.06ML). To address this need, various storage alternatives will be developed and evaluated in this technical memorandum.

1.1.2 Wastewater System Future Capacity Needs Summary

Black & Veatch conducted a detailed wastewater system capacity assessment in Study 1B: Wastewater System Capacity Optimization Study. Based on the assessment, the existing Nobleton wastewater collection system and Water Resource Recovery Facility (WRRF) experience high peak flows (wet weather flow). The existing Janet Avenue Pumping Station and Nobleton WRRF currently do not have the capacity to meet future average day flow (ADF) or peak instantaneous flow (PIF) requirements (Table 1-2).

As a result, there is a need to provide additional wastewater service capacity for the Janet Avenue PS and the Nobleton WRRF to support future ADF and PIF requirements of 3,996 m^3/day and 25,174 m^3/day, respectively.

Table 1-2: Summary of Existing Capacity of the Nobleton Wastewater System

CATEGORY	JANET AVENUE PUMPING STATION	NOBLETON WATER RESOURCE RECOVERY FACILITY (WRRF)
2007 Design	9,177 m^3/d (PIF)[(2)]	2,925 m^3/d (ADF)[(1)]
Future Flow Requirements	25,174 m^3/d (PIF) [(2)]	3,996 m^3/d (ADF) [(1)]

Notes:
(1) ADF represents annual average day flow. *ADF* = Average Day Flow
(2) PIF represents Peak Instantaneous Flow
m^3/d = Cubic Meters per Day

1.2 OTHER WATER SYSTEM BACKGROUND

The Region of York's Water Resources Group conducted a desktop groundwater supply options study in order to assess the ability of the existing groundwater resources to help meet future water demands (York Region, *Characterization and Comprehensive Review of Groundwater Supply Resources of the Regional Municipality of York*, 2019).

The following summarizes the report as it relates to the existing Nobleton well facilities:

- A desktop assessment of the potential for increased capacity at the existing municipal wells was conducted based on three main considerations:
  - Estimation of the maximum theoretical yield for the existing production wells
  - Analysis of the available drawdown in the wells
  - Background review to identify potential limiting factors to a well capacity increase
- The estimate of the maximum theoretical capacity for each well was obtained by calculating the well screen transmitting capacity using available well screen data. The well screen transmitting capacity was estimated as the design yield that the well screen is able to convey at an assumed entrance velocity of 0.03 m/s (0.1 ft/s).
  - Although a conservative approach was used to provide a screen transmitting capacity estimate that most likely represents a sustainable rate for the well, it is still

essential that the rates be verified through field investigation, together with consideration of potential limiting factors.

  - The estimated screen transmitting capacity of Nobleton Well #2, which has a current maximum permitted rate of 22.7 L/s, is 67L/s.
  - The estimated screen transmitting capacity of Nobleton Well #3 could not be estimated due to lack of data. Field investigation would be required to confirm screen capacity. However, due to known site constraints at Well Site #3, expansion at Well Site #3 was not considered further.
  - The estimated screen transmitting capacity of Nobleton Well #5, which has a current maximum permitted rate of 28.9 L/s, is 28L/s.
  - Based on screen transmitting capacity, the only Nobleton Well with potential to increase supply is Nobleton Well #2.
- Since Nobleton Well #2 was identified as having the potential for increased capacity based on the screen, an analysis of the available drawdown was also undertaken to determine whether there is likely to be sufficient drawdown available in each well to allow for increased pumping rates. A conservative estimate of the available drawdown for each of the identified production wells was then calculated as the difference between the static water level and the lowest safe level, with the subtraction of the allowances for seasonal groundwater level fluctuations in the aquifer and potential interference impacts from other production wells.
  - Nobleton Well #2 was identified as having sufficient drawdown available to meet the screen transmitting capacity (67L/s)
- No further limiting factors (eg. Historical well performance testing results; rehabilitation record; aquifer health) were identified that would prevent Nobleton Well #2 from potentially increasing water takings.

2 Screening and Evaluation Methodology

The Nobleton Water and Wastewater Schedule C Class Environmental Assessment developed, refined and evaluated various potential servicing strategies (for both the water and wastewater systems) to address the problem statement using a two-stage process.

A two-stage process was selected for the evaluation of alternatives because it provides a clear and simple way to identify which alternatives are technically feasible, whilst meeting the current regulations. Subsequently, with a shortlist of feasible alternatives, a detailed comparison can be conducted, using evaluation criteria that are based on the Municipal Engineers Association Class Environmental Assessment process requirements.

The decision-making process is based on a two-stage methodology (Figure 2-1):

- ***Stage 1:*** *Screening of Long List of Alternatives* – Only reasonable and feasible alternatives are to be considered as part of the Municipal Class EA process. This stage will determine the feasibility of an alternative by comparing it with a set of "pass/fail" screening criteria. The screening criteria will be used to screen out solutions from the long list of alternatives to create a short list of alternatives for further consideration in Stage 2.
- ***Stage 2:*** *Evaluation of Short List of Alternatives* – The short list of alternatives from Stage 1 are subject to detailed evaluation and will be assessed against the evaluation criteria. The evaluation criteria reflect various factors that have been established to be of most importance to the project. For evaluation, each evaluation criterion will be assigned a performance rating which will be used to comparatively evaluate the short list of alternative solutions. Alternatives will be rated based on how well it performs in addressing the specified criterion. Overall performance of each alternative will be determined based on the combination of individual criterion performance rating. The evaluation uses the "Traffic Light Assessment" method, where each alternative is scored as green, yellow or red for each criterion. This method was selected since it is highly intuitive to the general public, whilst also providing sufficient detail to differentiate between the various alternatives.

Figure 2-1: Screening and Evaluation Methodology

2.1 SCREENING CRITERIA

The screening criteria for this Class EA is composed of two categories, Technical and Jurisdictional/Regulatory and are summarized in Table 2-1.

Table 2-1: Screening Criteria for Nobleton's Water and Wastewater Alternative Servicing Solutions

PASS/FAIL SCREENING CRITERIA
Technical • The alternative will be able to support the forecasted growth and provide capacity for the community of Nobleton.
Jurisdictional/Regulatory • The alternative will be able to comply with all existing and proposed regulations and land use policies, including: o Provincial Policy Statement o Green Belt Plan o Oak Ridges Moraine Conservation Plan o Watershed Management Plan o Great Lakes – St. Lawrence River Basin Sustainable Water Resources Agreement o Municipal and Community Plans for York Region o York Region Master Plan, Standards & Design Guidelines

2.2 EVALUATION METHODOLOGY

The resulting short-listed solutions from the screening process are then subject to a detailed evaluation. Evaluation criteria have been developed and categorized to assess short-term (construction and commissioning) and long-term (permanent) impacts of the proposed alternative water and wastewater servicing solutions. The list of detailed evaluation criteria and performance ratings are provided in Table 2-2.

Table 2-2: Description of Evaluation Criteria for Short List Alternatives

CRITERIA		DESCRIPTION/CONSIDERATIONS	PERFORMANCE RATING
TECHNICAL			
A.	Constructability	• What are the major construction challenges and risks (e.g. crossing environmentally sensitive areas, noise, odour, dust, public safety, traffic, etc.) associated with the alternative? To what extent does it impact the community? • How much volume and complexity of construction will be associated with the alternative?	🟢 Low Impact *(Low Construction Impact/Complexity)* 🟡 Moderate Impact *(Moderate Construction Impact/Complexity)* 🔴 High Impact *(Higher Construction Impact/Complexity)*
B.	Redundancy of Supply/Service	• Will the alternative be able to provide improvements in redundancy of supply or service?	🟢 High Redundancy 🟡 Moderate Redundancy 🔴 Low Redundancy
C.	Resilience to Climate Change	• Is the alternative resilient against changing climate conditions, such as: o Changes to water supply quantity and quality (e.g. due to drought) o Increase of intensity and frequency of wet weather flow events	🟢 High Resilience 🟡 Moderate Resilience 🔴 Low Resilience
D.	Operations & Maintenance (O&M) Requirements	• What will be the level of additional and new O&M resources (e.g. human resources) required for the alternative? • What will be the level of complexity and maintainability of new and optimized assets?	🟢 Low Complexity/ O&M Requirements 🟡 Moderate Complexity/ O&M Requirements 🔴 High Complexity/ O&M Requirements
E.	Adaptability to Existing Infrastructure	• What will be the level of modification required to the existing infrastructure to adapt to the alternative? What is the relative ease of connection to the existing alternative?	🟢 High Adaptability 🟡 Moderate Adaptability 🔴 Low Adaptability
F.	Maximizing Use of Existing Infrastructure)	• Will the alternative be able to maximize the capacity of the existing infrastructure to reduce new asset needs?	🟢 High Degree (*Efficient use of Existing Infrastructure)* 🟡 Moderate Degree (*Partial use of Existing Infrastructure)* 🔴 Low Degree (*Inefficient use of Existing Infrastructure)*
NATURAL ENVIRONMENT			
G.	Aquatic Vegetation and Wildlife	o Will the alternative have significant impacts during construction and/or from ongoing operations on: o Streams and rivers o Local aquatic species and habitats o Environmentally sensitive areas, aquatic species at risk or locally significant aquatic species	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
H.	Terrestrial Vegetation and Wildlife	o Will the alternative have significant impacts during construction and/or from ongoing operations on: o Trees and vegetation o Local terrestrial species and habitats o Environmentally sensitive areas, species at risk and locally significant species	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
I.	Groundwater Resources	o Will the alternative have significant impacts during construction and/or from ongoing operations on aquifers and groundwater resources such as: groundwater quantity, groundwater recharge quality and flow regime and groundwater discharge to streams and wetlands?	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
J.	Surface Water Resources	o Will the alternative have significant impacts during construction and/or from ongoing operations on adjacent surface water resources (e.g. Humber River) and related biological communities?	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
K.	Greenhouse Gas Emissions	o What will be the level of impact of greenhouse gas emissions associated with the alternative? *(Greenhouse gas emission will be evaluation based on the alternative's energy intensity requirements.)*	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
SOCIO-ECONOMIC ENVIRONMENT			
L.	Short-term Community Impacts (Impacts to Community during Construction)	o Will the alternative have significant short-term impacts to the community during construction, including: o Noise, dust and odour o Local traffic	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact

CRITERIA		DESCRIPTION/CONSIDERATIONS	PERFORMANCE RATING
M.	Long-term Community Impact	o Will the alternative have significant long-term impacts on the community, including: o Impact of Operating Facility o Visual Impact o Public Acceptance/Resistance (Any potential resistance to the proposed servicing solution? [e.g. Resistance to Growth/Resistance to Well Supply])	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
N.	Archaeological Sites	o Will the alternative have significant impacts during construction and/or from ongoing operations on registered/known archaeological features?	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
O.	Cultural/Heritage Features	o Will the alternative have significant impacts during construction and/or from ongoing operations on known cultural landscapes and built heritage features?	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
FINANCIAL			
P.	Capital Cost	o What will be the relative capital cost for the alternative?	🟢 Low Cost Alternative 🟡 Moderate Cost Alternative 🔴 High Cost Alternative
Q.	Lifecycle Cost	o What will be the relative lifecycle cost for the alternative?	🟢 Low Cost Alternative 🟡 Moderate Cost Alternative 🔴 High Cost Alternative
R.	Land Acquisition Cost	o What will be the relative land acquisition cost for the alternative?	🟢 Low Cost Alternative 🟡 Moderate Cost Alternative 🔴 High Cost Alternative
JURISDICTIONAL/REGULATORY			
S.	Land Requirements	o What will be the relative area of non-regional land or easement required to construct the alternative?	🟢 Low Requirement 🟡 Moderate Requirement 🔴 High Requirement
T.	Ability to Accommodate Potential Future Regulatory Changes	o Will the alternative have the ability to adapt to potential future changes in drinking water quality and final effluent requirements?	🟢 High Adaptability 🟡 Moderate Adaptability 🔴 Low Adaptability
U.	Permits and Approval	o What will be the level of permits and approvals required to construct the alternative?	🟢 Low Requirement 🟡 Moderate Requirement 🔴 High Requirement

3 Water System

The water system alternatives evaluation is split up into two main categories:

1) Alternative Solutions to Address the Storage Deficit

2) Alternative Solutions to Address the Supply Deficit

The first consideration will be the storage alternatives since the storage deficit is marginal and could be addressed by a wide variety of solutions including: 1) adding new storage; 2) increased well supply or; 3) reducing the storage needs (by reducing demand or equalization storage needs).

3.1 WATER SYSTEM STORAGE

3.1.1 Long List of Alternative Water Storage Solutions

In terms of storage capacity, the existing Nobleton system has storage volume capable of providing storage requirements (fire, equalization and emergency storage) up to the equivalent of a maximum day demand of 87.40 L/s. Since the projected MDD is 89.5 L/s, this means that there would ultimately be a marginal storage deficit, if no action was taken. To address this need, various storage alternatives are developed and evaluated in this technical memorandum.

To address the identified need, six (6) alternative storage solutions were developed for this project and are listed below:

1. **Do Nothing.** Permit the growth, but do not increase the storage capacity of the existing water supply system. This concept is typically included in the Class EA process for comparative purposes. It is a hypothetical concept which permits the forecasted growth without providing any solution to address the deficit;

2. **Limit Growth.** Limit the growth up to the existing capacity of the current water supply system. This concept is typically included in the Class EA process for comparative purposes;

3. **Water Conservation.** This concept considers methods to reduce the projected maximum day water demand from 89.5 L/s to below 87.40L/s so that additional storage is not necessary. This could involve implementing practices for efficient water use to reduce water usage per person and/or to reduce the maximum day peaking factor by reducing summer demands in particular;

4. **Modification of Existing Design Guidelines.** This concept considers modifying the current York Region Design Guideline for storage sizing. Currently, the equalization component of storage volume is calculated as 25% of maximum day demand, which is a general rule of thumb that is considered suitable for most systems based on a typical diurnal pattern. A detailed review of the actual diurnal pattern in Nobleton could suggest that this percentage be reduced, thereby eliminating the need for additional storage;

5. **New Storage Facility.** The existing Nobleton storage facilities were built in 1985 and 2012 respectively, so, both storage facilities are considered to have a life expectancy beyond 2040. Therefore, a new storage facility would not be considered a timely replacement since the replaced storage would have ordinarily had a moderate amount of service life

remaining. However, since the storage deficit is so small, a third elevated tank (while maintaining the existing two tanks) is unreasonable since the third tank would either too small to be effective operationally or be oversized for the system needs. So, this alternative considers a new storage facility that would act as an upsized replacement of the older Nobleton South Elevated Tank (2.041ML). Once a new tank is built and commissioned, the existing Nobleton South Elevated Tank would be able to be removed from service. This concept considers the addition of a new storage facility (with volume of at least 2.055ML) to meet the storage deficit of the Nobleton water system at the projected future demand;

6. **Supplement Increased Supply to Offset Storage Deficit.** This concept considers increasing the combined PTTW and supply capacity in Nobleton to exceed the forecasted maximum day demand (>89.5L/s). By exceeding the maximum day demand (even slightly), it allows for the wells to operate at a higher rate during the hours when demand exceeds the average maximum day demand. This reduces the amount of equalization storage required because some of the equalization is pumped (rather than being stored);

3.1.2 Screening of Long List of Alternative Water Storage Solutions

The long list of alternative water storage solutions is screened according to the screening criteria presented in Section 2.1. Each alternative's ability to meet the criteria is noted by the following symbols, "✓" for Pass and "✗" for Fail. The screening results are presented in Table 3-1.

The screening process eliminated the following four out of the six proposed water storage solutions:

- The first two alternatives, "Do Nothing" and "Limit Growth", are eliminated due to their inability to provide additional capacity for the forecasted growth.
- The third alternative, "Water Conservation" is eliminated due to limitations and uncertainty on the effectiveness of further water conservation measures in the Nobleton community. The Region of York does not expect further reductions to per capita water consumption in Nobleton. Recent development in Nobleton would already have included a degree of water conservation (low flow water fixtures, etc.), but there has been no clear sign of per capita consumption being reduced yet.
- The fourth alternative, "Modification of Existing Design Guidelines" is eliminated since modifying the existing design guideline would not meet the jurisdictional/regulatory criteria. It is currently deemed that there is insufficient evidence to definitively prove that the equalization storage needs in Nobleton are less than the standard (25% of maximum day demand), therefore, it is not advisable to change the design criteria.

The following two storage alternatives, which are deemed feasible to support forecasted growth in the community of Nobleton, are carried forward for detailed evaluation:

- Alternative 5: "New Storage Facility"
- Alternative 6: "Supplement Increased Supply to Offset Storage Deficit"

Table 3-1: Screening of the Long List of Alternative Water Storage Solutions

LONG LIST OF ALTERNATIVE WATER STORAGE SOLUTIONS	SCREENING CRITERIA		NOTES
	TECHNICAL	JURISDICTIONAL/ REGULATORY	
1. Do Nothing	×	✓	○ **This alternative is unable** to provide additional storage capacity for the forecasted growth, so it does not meet the technical or jurisdictional/regulatory requirements. However, it is not screened out in order to provide a baseline for comparison of the alternatives.
2. Limit Growth	×	✓	○ **Eliminated** due to its inability to meet the forecasted growth.
3. Water Conservation	×	✓	○ **Eliminated** due to limitations and uncertainty on the effectiveness of further water conservation measures in the Nobleton community. ○ The Region of York does not expect further reductions to per capita water consumption in Nobleton. Recent development in Nobleton would already have included a degree of water conservation (low flow water fixtures, etc.), but there has been no clear sign of per capita consumption being reduced yet. ○ Despite this alternative not being carried forward, the Region of York is still continuing to emphasize the benefits of water conservation to the public. Water conservation will be carried forward as a separate ongoing program in York Region.
4. Modification of Existing Design Guidelines	○ ✓	○ ×	○ Eliminated since a modification to the existing design guideline does not meet the jurisdictional/regulatory criteria. It is currently deemed that there is insufficient evidence to definitively prove that the equalization storage needs in Nobleton are less than the standard (25% of maximum day demand).
5. New Storage Facility	✓	✓	○ **Proceed to Detailed Evaluation.** A new storage facility would be able to support forecasted growth in the community of Nobleton while meeting the jurisdictional and regulatory requirements.
6. Supplement Increased Supply to Offset Storage Deficit	✓	✓	○ **Proceed to Detailed Evaluation.** Increasing the combined PTTW and supply capacity in Nobleton to exceed the maximum day demand (>89.5L/s) would allow for the forecasted growth since the equalization storage need could be reduced; thereby eliminating the need for additional storage.

3.1.3 Short List of Alternative Water Storage Solutions

In addition to the "Do Nothing" alternative, two alternative water storage solutions are carried forward for detailed evaluation. A description of each alternative is provided in the subsequent sections.

Table 3-2: Short List of Alternative Water Storage Solutions for Detailed Evaluation

SHORT LISTED ALTERNATIVE WATER STORAGE SOLUTIONS
A. Add New Storage Facility
B. Supplement Increased Supply to Offset Storage Deficit

3.1.3.1 Storage Alternative A: Add New Storage Facility

The Nobleton water system currently has two storage facilities which have a combined useable storage volume of 3.860 ML. Based on the projected maximum day demand of 89.5L/s, the storage requirement for the Nobleton water system is 3.916 ML (marginal deficit of 0.06 ML).

The existing Nobleton storage facilities were built in 1985 and 2012 respectively, so, both storage facilities are considered to have a life expectancy beyond 2040. Therefore, a new storage facility would not be considered a timely replacement since the replaced storage would have ordinarily had a moderate amount of service life remaining. Furthermore, since the storage deficit is so small, a third elevated tank (while maintaining the existing two tanks) is unreasonable since the third tank would either too small to be effective operationally or be oversized for the system needs.

So, a new storage facility would most reasonably be built as a larger replacement of the older Nobleton South Elevated Tank (2.045ML) since it is the older facility (built 1985). Once a new tank is built and commissioned, the existing Nobleton South Elevated Tank would be able to be removed from service. This concept considers the addition of a new storage facility (with volume of at least 2.055ML) to meet the storage deficit of the Nobleton water system at the projected future demand.

3.1.3.2 Storage Alternative B: Supplement Increased Supply to Offset Storage Deficit

This concept considers increasing the PTTW and supply capacity in Nobleton over and above the maximum day demand (>89.5L/s). By having supply capacity exceed the maximum day demand, it allows for the wells to operate at a higher rate during the hours when demand exceeds the average maximum day demand. This reduces the amount of equalization storage required because some of the equalization storage is pumped. Ordinarily this may not be feasible, but since the storage deficit is small, the additional supply capacity would also be small and would have minimal impact on the water takings from the Nobleton area.

Supply capacity would be increased in a manner that aligns with the water supply solutions recommended in Section 3.2. Short-listed water supply solutions include increasing existing well capacities, introducing a new production well and adding a connection to lake-based supply (see Section 3.2). Assuming that the combined well firm capacity (and PTTW) exceeds the MDD by 2L/s and is able to operate at this higher rate during 12 hours of the day when demands exceed the average MDD, then the equalization storage that is "offset" or no longer required in the system would be equal to 0.0864ML (slightly more than the 0.06ML deficit). Therefore, if a well-based solution is recommended, the two expanded/new production wells along with their associated treatment facility would each need to be increased by an additional 2L/s (above the capacity increases required to meet maximum day demands) to cover the storage deficit. If lake-based supply is recommended, then an additional 2L/s must be available for transfer to Nobleton.

The recommended water supply solution (see Section 3.2.5) is to Increase Capacity of Existing Well(s) in Combination with New Production Well(s). So, a combined well firm capacity and PTTW of 91.5L/s is considered in this scenario, which would mean that while keeping Wells #3 and #5 at 28.9 L/s each, Well #2 and a new well would each need a supply capacity of at least 33.7L/s.

3.1.4 Evaluation of Short List of Alternative Water Storage Solutions

A detailed evaluation of the short-listed alternative water storage solutions is carried out in accordance with the evaluation methodology described in Section 2.2 and is presented in Table 3-3.

3.1.5 Selection of Recommended Water Storage Solution

The detailed evaluation of the short-listed alternative water storage solutions favored **Alternative B: "Supplement Increased Supply to Offset Storage Deficit"** to be the recommended servicing solution due to the following considerations:

- **Technical –** Alternative B scored higher under the technical category primarily due to its ability to maximize the use of existing infrastructure, while avoiding unnecessary new assets. This also results in less volume and complexity of construction compared to Alternative A, thus minimizing potential impacts/disturbance to the community during construction. The Do Nothing Alternative cannot meet forecasted growth.
- **Environmental** – All alternatives are expected to have low or no significant impact to vegetation and wildlife, and surface water resources, groundwater resources and greenhouse gas emissions. Alternative B would require minimally greater use of groundwater resources than Alternative A which involves a small overall increase to the well supply. Neither has significant impact on existing resources. The impacts of increasing the peak well supply during maximum day demand conditions at two well facilities will be evaluated as part of the ongoing groundwater exploration study for Alternative B. However, Alternative B is not expected to have significant impact on groundwater resources.
- **Socio-Economic** – Under the socio-economic category, Alternative B scored higher than Alternative A primarily due to the added short-term community impacts that would be caused by the construction of an Elevated Tank near a residential area. Like most construction, short-term impacts/nuisance to the community are expected due to increased traffic, noise and dust. Comparatively, Alternative B would only slightly increase the capacity of facilities that are already being upgraded or installed as part of the Water Supply solution (see Section 3.2). As no additional impact is expected from the slight increase, Alternative B is identified as having low short-term community impacts. The Do Nothing Alternative also has low socio-economic impact, apart from the inability to meet forecasted growth that would help the local economy grow.
- **Financial** – Between Alternatives A and B, Alternative B is anticipated to be the lower cost alternative in terms of capital cost, land acquisition and overall life cycle cost. Alternative A requires a high amount of upfront capital costs since it involves a new storage tank and does not maximize the investments already made in the system. Comparatively, Alternative B would have a lower capital and life-cycle cost because the costs would only be those associated with the

slightly higher supply target at these facilities, which would be any incremental costs for larger components. The Do Nothing alternative would have no associated costs.

- **Jurisdictional** – Alternative B has the lowest overall impact in terms of Jurisdictional/Regulatory requirements. Alternative A would require some land acquisition and a DWWP amendment as well as construction permitting. The Do Nothing Alternative would require no additional permits or approvals but would have no ability to adapt to potential future changes in drinking water quality requirements.

Overall, Alternative B (supplementing supply to offset storage) scored well in all five categories of the detailed evaluation criteria and generally outscored Alternative A. Therefore, Alternative B was found to be the recommended storage solution to address the storage deficit in Nobleton.

Table 3-3: Short Listed Alternative Water Storage Solutions - Detailed Evaluation

EVALUATION CRITERIA	DO NOTHING	ALTERNATIVE A: ADD NEW STORAGE FACILITY	ALTERNATIVE B: SUPPLEMENT INCREASED SUPPLY TO OFFSET STORAGE DEFICIT
CONCEPTS	Included in the Class EA process for comparative purposes. Hypothetical concept which permits the forecasted growth without providing any solution to address the servicing needs.	New storage facility (2.055ML) to be built as a replacement of the Nobleton South Elevated Tank (2.045ML). Once the new facility is completed, the Nobleton South Elevated Tank would be decommissioned. Currently, assumed that the storage facility would be in close proximity to the existing Nobleton South ET site.	Supplement supply capacity to offset the storage deficit. If wells are recommended, based on the Well Supply Evaluation (Section 3.2), then the two expanded/new production wells along with their associated treatment facility would need to be increased by an additional 2L/s each to cover the storage deficit. If lake-based supply is recommended, then an additional 2L/s must be available for transfer to Nobleton.
TECHNICAL			
A. CONSTRUCTABLITY • What are the major construction challenges and risks (e.g. crossing environmentally sensitive areas, noise, odour, dust, public safety, traffic, etc.) associated with the alternative? • To what extent does it impact the community? • How much volume and complexity of construction will be associated with the alternative?	**LOW IMPACT** ➢ No construction to be conducted as part of "Do-Nothing"	**MODERATE IMPACT** ➢ Moderate impact expected in the residential neighborhood adjacent to the existing Nobleton South Elevated Tank during construction. ➢ No major constructability challenges are expected for the construction of the new storage facility.	**LOW IMPACT** ➢ Since this alternative only considers a small increase in capacity to work that would already be required as part of the Well Supply Evaluation (Section 3.2), no major constructability challenges or additional impact are expected due to the increased capacity.
B. REDUNDANCY OF SUPPLY/SERVICE • Will the alternative be able to provide improvements in redundancy of supply or service?	**LOW REDUNDANCY** ➢ Without any system upgrades, the forecasted growth cannot be met. Therefore, there is also insufficient redundancy.	**HIGH REDUNDANCY** ➢ Two storage facilities will still exist which provides flexibility to have one storage facility out of service without significant impact to service.	**HIGH REDUNDANCY** ➢ Two storage facilities will still exist which provides flexibility to have one storage facility out of service without significant impact to service. ➢ Marginally greater risk than Alternative A, since pumped equalization could be unavailable during system-wide blackouts, however this risk would be mitigated by standby power at well facilities.
C. RESILIENCE TO CLIMATE CHANGE • Will the alternative have the resilience against changing climate conditions, such as changes to water supply quantity and quality (e.g. high water demands, drought)	**LOW RESILIENCE** ➢ Without any system upgrades, the forecasted growth cannot be met. Therefore, there is also no resilience to increasing demands due to climate change	**MODERATE RESILIENCE** ➢ New storage facility is generally resistant to changing climate. ➢ Similarly impacted by changing water demands / drought / increasing temperatures as Alternative B.	**MODERATE RESILIENCE** ➢ Marginally increased supply is generally resistant to changing climate. ➢ Similarly impacted by changing water demands / drought / increasing temperatures as Alternative A.
D. O & M REQUIREMENTS • What will be the level of additional and new O&M resources (e.g. human resources) required for the alternative? • What will be the level of complexity and maintainability of new and optimized assets?	**LOW COMPLEXITY** ➢ No upgrades, so there are no additional facilities to operate and maintain.	**LOW COMPLEXITY** ➢ Low additional resource requirements to maintain and operate the new storage facility since it is considered a replacement of an existing storage facility. ➢ No impact to system complexity.	**LOW COMPLEXITY** ➢ Low additional resource requirements because this alternative only considers a small increase in supply capacity to facilities that are already being considered as part of the Well Supply Evaluation. ➢ No impact to system complexity.

E. ADAPTABILITY TO EXISTING INFRASTRUCTURE • What will be the level of modification required to the existing infrastructure to adapt to the alternative? What is the relative ease of connection to the existing alternative?	**HIGH ADAPTABILITY** ➢ No planned upgrades, so there is no new infrastructure that needs to connect to the existing system.	**HIGH ADAPTABILITY** ➢ Minor modifications would be required near the existing Nobleton South ET to ensure a smooth transition to the new Elevated Tank during the respective commissioning and decommissioning phases for the tanks. ➢ No significant challenges.	**HIGH ADAPTABILITY** ➢ Negligible difference is expected to occur at the supply facilities from the required additional 2L/s supply capacity. Similar modifications required to existing infrastructure. ➢ No significant challenges.
F. MAXIMIZING USE OF EXISTING INFRASTRUCTURE • Will the alternative be able to maximize the capacity of the existing infrastructure to reduce new assets needs?	**LOW DEGREE** ➢ Without any system upgrades, there is no ability to maximize the capacity of existing infrastructure.	**LOW DEGREE** ➢ Replacing an existing storage facility with a larger facility, even though the existing storage is not at the end of its useful life does not fully maximize the existing infrastructure.	**HIGH DEGREE** ➢ Supplementing the supply capacity of existing and/or planned facilities, in order to avoid the need for a new storage facility, maximizes the existing infrastructure and helps to avoid unnecessary new assets.
OVERALL TECHNICAL RATING Based on all above technical criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**HIGH IMPACT** ➢ Without any system upgrades, the forecasted growth cannot be met.	**MODERATE IMPACT** ➢ Moderate impacts due to constructability. Moderate resilience to climate change. Low impacts associated with high redundancy, low complexity of O&M and ability to adapt to existing infrastructure. ➢ Does not fully maximize use of existing infrastructure.	**LOW IMPACT** ➢ Moderate resilience to climate change. Low impacts associated with constructability, high redundancy, low complexity of O&M and ability to adapt to existing infrastructure. Maximizes use of existing infrastructure.
OVERALL TECHNICAL SUMMARY	Alternative B scored highest under the technical category primarily due to its ability to maximize the use of existing infrastructure, while avoiding unnecessary new assets. This also results in less volume and complexity of construction compared to Alternative A, thus minimizing potential impacts/disturbance to the community during construction. Alternatives A and B provide similar levels of redundancy and resiliency. The Do Nothing option has low impacts associated with construction, O&M complexity and adaption to existing infrastructure, but cannot meet forecasted growth.		
ENVIRONMENTAL			
G. AQUATIC VEGETATION AND WILDLIFE ➢ Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Streams and river ○ Local aquatic species and habitat ○ Environmentally sensitive areas, aquatic species at risk and locally significant aquatic species	**LOW IMPACT** ➢ Without any system upgrades, there is no impact to aquatic vegetation /wildlife.	**LOW IMPACT** ➢ No significant risk to aquatic vegetation and wildlife are expected ➢ Minimal impact expected from replacement of elevated tank near Nobleton South Elevated Tank site. Potential short-term impact during construction. Non-damaging construction techniques and erosion controls will be employed to minimize construction impact.	**LOW IMPACT** ➢ No significant risk to aquatic vegetation and wildlife are expected ➢ Minimal impact expected from work associated with the 2L/s increase in supply capacity for the supply that is already being considered in the Well Supply Evaluation (Section 3.2).
H. TERRESTRIAL VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Trees and vegetation ○ Local terrestrial species and habitats ○ Environmentally sensitive areas, species at risk and locally significant species	**LOW IMPACT** ➢ Without any system upgrades, there is no impact to terrestrial vegetation/wildlife.	**LOW IMPACT** ➢ Minimal impact is expected from replacement of elevated tank near Nobleton South Elevated Tank site	**LOW IMPACT** ➢ Minimal impact is expected from work associated with the 2L/s increase in supply capacity for the supply that is already being considered in the Well Supply Evaluation (Section 3.2).

I. GROUNDWATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on aquifers and groundwater resources such as: groundwater quantity, groundwater recharge quality and flow regime and groundwater discharge to streams and wetlands?	**LOW IMPACT** ➢ Without any system upgrades, there is no impact to groundwater resources.	**LOW IMPACT** ➢ Storage alternative has negligible impact on aquifer and groundwater resources	**MODERATE IMPACT** ➢ Impacts of increasing the peak well supply during maximum day demand conditions at two well facilities will be evaluated as part of the ongoing groundwater exploration study. Not expected to have significant impact, however, Alternative A would have negligible impact. ➢ The 6" pump testing at Sites F & H and the pump testing at existing Well #2 indicate that there is sufficient available drawdown at each of the wells to sustain a rate of 34L/s.
J. SURFACE WATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on adjacent surface water resources (e.g. Humber River) and related biological communities?	**LOW IMPACT** ➢ Without any system upgrades, there is no impact to surface water resources. ➢	**LOW IMPACT** ➢ No significant risk to surface water resources	**LOW IMPACT** ➢ No significant risk to surface water resources
K. GREENHOUSE GAS EMISSIONS • What will be the level of greenhouse gas emissions associated with the alternative? *(Greenhouse gas emission will be evaluation based on the alternative's energy intensity requirements.)*	**LOW IMPACT** ➢ Without any system upgrades, there is no added impact greenhouse gas emissions.	**LOW IMPACT** ➢ Available storage ensures that the peak hourly energy requirements are reduced. However, the same total amount of water would be supplied each day, so there is negligible difference between the two alternatives.	**LOW IMPACT** ➢ Available storage ensures that the peak hourly energy requirements are reduced. However, the same total amount of water would be supplied each day, so there is negligible difference between the two alternatives.
OVERALL ENVIRONMENTAL RATING Based on all above environmental criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**LOW IMPACT** ➢ Without any system upgrades, there are no environmental impacts.	**LOW IMPACT** ➢ No significant risks to aquatic vegetation and wildlife and surface water resources. ➢ Minimal impacts to terrestrial vegetation and wildlife expected. ➢ Negligible impact to groundwater resources and greenhouse gas emissions.	**LOW IMPACT** ➢ No significant risks to aquatic vegetation and wildlife and surface water resources. ➢ Minimal impacts to terrestrial vegetation and wildlife expected. ➢ No significant impact expected to groundwater resources, and negligible impact to greenhouse gas emissions.
OVERALL ENVIRONMENTAL SUMMARY	All alternatives are expected to have low or no significant impact to vegetation and wildlife, and surface water resources, groundwater resources and greenhouse gas emissions. Alternative B would require minimally greater use of groundwater resources than Alternative A which involves a small overall increase to the well supply. Neither has significant impact on existing resources. The impacts of increasing the peak well supply during maximum day demand conditions at two well facilities will be evaluated as part of the ongoing groundwater exploration study for Alternative B. However, Alternative B is not expected to have significant impact on groundwater resources.		
SOCIO-ECONOMIC			
L. SHORT-TERM COMMUNITY IMPACTS • Will the alternative have significant short-term impacts to the community during construction, including: o Noise, dust and odour o Local traffic	**LOW IMPACT** ➢ Without any system upgrades, there is no additional construction that would lead to community impacts.	**MODERATE IMPACT** ➢ Nobleton South Elevated Tank is within a residential neighborhood, so, a tank replacement would lead to moderate noise, dust and construction traffic on a short-term basis, although this can be mitigated to some extent.	**LOW IMPACT** ➢ Short-term impact/nuisance to the community are expected during construction/expansion of well facilities, including noise, dust and impact to the local traffic. ➢ However, this alternative is only focused on slightly increasing the capacity of facilities that are already being considered in the Well Supply Evaluation (Section 3.2). No additional impact is expected, so this is identified as low impact.

M. LONG-TERM COMMUNITY IMPACT • Will the alternative have significant long-term impact to the community, including: o Benefit to Community o Impacts from Facility Operations o Visual Impact o Public Acceptance/Resistance	**MODERATE IMPACT** ➢ Without any system upgrades, it is not possible to meet the forecasted growth. This would impact the community since the growth helps the local economy grow.	**LOW IMPACT** ➢ Long-term, replacing the storage facility is no different than the current arrangement in terms of facility operations, visual impact. Low impact is therefore expected long-term	**LOW IMPACT** ➢ Minimal visual and operating impacts are expected, especially if new well site is at same location as existing Well Site #5. ➢ Regardless, this alternative is only focused on slightly increasing the capacity of facilities that are already being considered in the Well Supply Evaluation (Section 3.2). No additional impact is expected, so this is identified as low impact.
N. ARCHAEOLOGICAL SITES • Will the alternative have significant impacts during construction and/or from ongoing operations on registered/known archaeological features?	**LOW IMPACT** ➢ Without any system upgrades, there is no additional construction that would lead to archaeological impact.	**LOW IMPACT** ➢ New location of storage facility likely to be in close proximity to existing Nobleton South ET. ➢ Stage 1 archeological assessment has not identified any significant risk of archaeological potential at either site. A Stage 2 assessment is required to further validate certain parts of the Well #2 Site.	**LOW IMPACT** ➢ Stage 1 archeological assessment has not identified any significant risk of archaeological potential at any of the potentially expanded well facilities. A Stage 2 assessment is required to further validate certain parts of the Well #2 Site and the Potential Well Site F. ➢ Since this alternative is only focused on slightly increasing the capacity of facilities that are already being considered in the Well Supply Evaluation (Section 3.2). No additional impact is expected, so this is identified as low impact.
O. CULTURAL/HERITAGE FEATURES • Will the alternative have significant impacts during construction and/or from ongoing operations on known cultural landscapes and built heritage features?	**LOW IMPACT** ➢ Without any system upgrades, there is no additional construction that would lead to a cultural/heritage impact.	**LOW IMPACT** ➢ Known heritage properties in Nobleton are not located close to the potential site locations. ➢ Currently, nothing suggests that the replacement of the tank at the existing Nobleton South ET site would impact cultural/heritage features.	**LOW IMPACT** ➢ Known heritage properties in Nobleton are not located close to the considered well site locations. ➢ Since this alternative is only focused on slightly increasing the capacity of facilities that are already being considered in the Well Supply Evaluation (Section 3.2). No additional impact is expected, so this is identified as low impact.
OVERALL SOCIO-ECONOMIC RATING Based on all above socio-economic criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**LOW IMPACT** ➢ Without any system upgrades, no socio-economic impacts apart from inability to meet forecasted growth.	**LOW IMPACT** ➢ Low to moderate short- and long-term impacts to community. ➢ Low impacts to archeological and cultural/heritage sites/features.	**LOW IMPACT** ➢ Low short- and long-term impacts to community. ➢ Low impacts to archeological and cultural/heritage sites/features.
OVERALL SOCIO-ECONOMIC SUMMARY	Under the socio-economic category, Alternative B scored higher than Alternative A primarily due to the added short-term community impacts that would be caused by the construction of an Elevated Tank near a residential area. Like most construction, short-term impacts/nuisance to the community are expected due to increased traffic, noise and dust. Comparatively, Alternative B would only slightly increase the capacity of facilities that are already being upgraded or installed as part of the Water Supply solution (see Section 3.2). As no additional impact is expected from the slight increase, Alternative B is identified as having low short-term community impacts. The Do Nothing Alternative also has low socio-economic impact, apart from the inability to meet forecasted growth that would help the local economy grow.		
FINANCIAL			
P. LAND ACQUISITION COST • What will be the relative land acquisition cost for the alternative?	**LOW IMPACT** ➢ Without any system upgrades, there is no land acquisition needed.	**MODERATE REQUIREMENT** ➢ New location of storage facility likely to be in close proximity to existing Nobleton South ET, but not likely to fit on the existing site without purchasing some adjacent land to the west.	**LOW REQUIREMENT** ➢ Since this alternative is only focused on slightly increasing the capacity of facilities that are already being considered in the Well Supply Evaluation (Section 3.2). No additional land acquisition is expected, to be caused by the 2L/s surplus required in this alternative.

Q. CAPITAL COST • What will be the relative capital cost for the alternative?	**LOW IMPACT** ➢ Without any system upgrades, there is no upfront capital cost.	**HIGH COST ALTERNATIVE** ➢ High amount of upfront capital costs for this alternative since it involves a new storage tank and does not maximize the investments already made in the existing tank.	**MODERATE COST ALTERNATIVE** ➢ Comparatively lower amount of capital cost since the costs would only be costs associated with the slightly higher flow requirement at these facilities, which would be any incremental costs for larger components.
R. LIFECYCLE COST • What will be the relative lifecycle cost for the alternative?	**LOW COST ALTERNATIVE** ➢ With no system upgrades there is no associated lifecycle cost. O&M costs limited to existing costs.	**HIGH COST ALTERNATIVE** ➢ Operating costs no different than the baseline scenario since there are no extra pumping costs or O&M costs. ➢ Main factor in rating is the capital cost.	**MODERATE COST ALTERNATIVE** ➢ Although, there could be marginally higher energy costs than Alternative A, these are not significant since the system would still supply the same total flow each year. It would simply supply slightly more during peak hours which is likely to increase energy costs slightly. ➢ O&M costs would be similar to the baseline facilities (without the 2L/s) increase in capacity. ➢ Significantly lower capital costs are the main factor in the rating being lower than Alternative A.
OVERALL FINANCIAL RATING Based on all above financial criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**LOW IMPACT** ➢ Without any system upgrades, no associated costs.	**HIGH IMPACT** ➢ Moderate land acquisition costs. ➢ High capital costs and high lifecycle costs associated with alternative.	**MODERATE IMPACT** ➢ No land acquisition cost, apart from costs already considered as a part of Well Supply Evaluation (Section 3.2). ➢ Moderate capital costs and lifecycle costs associated with alternative.
OVERALL FINANCIAL SUMMARY	Between Alternatives A and B, Alternative B is anticipated to be the lower cost alternative in terms of capital cost, land acquisition and overall life cycle cost. Alternative A requires a high amount of upfront capital costs since it involves a new storage tank and does not maximize the investments already made in the system. Comparatively, Alternative B would have a lower capital and life-cycle cost because the costs would only be those associated with the slightly higher supply target at these facilities, which would be any incremental costs for larger components. The Do Nothing alternative would have no associated costs.		
JURISDICTIONAL/REGULATORY			
S. LAND REQUIREMENTS • What will be the level of area of non-regional land or easement required to construct the alternative?	**LOW REQUIREMENT** ➢ Without any system upgrades, there is no land acquisition needed.	**MODERATE REQUIREMENT** ➢ New location of storage facility likely to be in close proximity to existing Nobleton South ET, but not likely to fit on the existing site without purchasing some adjacent land to the west.	**LOW REQUIREMENT** ➢ Since this alternative is only focused on slightly increasing the capacity of facilities that are already being considered in the Well Supply Evaluation (Section 3.2). No additional land acquisition is expected, to be caused by the 2L/s surplus required in this alternative.
T. ABILITY TO ACCOMMODATE POTENTIAL FUTURE REGULATORY CHANGES • Will the alternative have the ability to adapt to potential future changes in drinking water quality requirements?	**LOW ADAPTABILITY** ➢ Without any system upgrades, does not have the ability to adapt to potential future changes.	**HIGH ADAPTABILITY** ➢ No impact anticipated in drinking water quality requirements that would be affected by new storage facility.	**HIGH ADAPTABILITY** ➢ Meets current water quality regulations. Potential changes to water treatment requirements not expected to have significant impact. ➢ Has the ability to adapt to future changes in drinking water quality requirements.
U. PERMITS AND APPROVALS • What will be the level of permits and approvals required to construct the alternative?	**LOW REQUIREMENT** ➢ Without any system upgrades, there are no additional permits/ approvals required.	**MODERATE REQUIREMENT** ➢ Will require a Drinking Water Works Permit (DWWP) Amendment to have a new storage facility to replace the existing one. ➢ Site plan and local permits as required for the design and construction of the new facility.	**LOW REQUIREMENT** ➢ If conducted simultaneous to the other upgrades to the supply facilities (as considered in the Well Supply Evaluation Section 3.2), then no additional permits (DWWP, PTTW updates, local permits for construction) are required for this alternative. All permits are associated with the supply evaluation.

OVERALL JURISDICTIONAL/ REGULATORY RATING Based on all above jurisdictional/ regulatory criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**MODERATE IMPACT** ➢ Without any system upgrades, there is no need for land acquisition or additional permits/approvals. ➢ Has no ability to adapt to potential future changes in drinking water quality requirements.	**MODERATE IMPACT** ➢ Requires new land acquisition and some additional permits/approvals. ➢ Is able to adapt to potential future changes in drinking water quality requirements.	**LOW IMPACT** ➢ Requires no new land acquisition, or additional permits/approvals. ➢ Is able to adapt to potential future changes in drinking water quality requirements.
OVERALL JURISDICTIONAL/ REGULATORY SUMMARY	Alternative B has the lowest overall impact in terms of Jurisdictional/Regulatory requirements. Alternative A would require some land acquisition and a DWWP amendment as well as construction permitting. The Do Nothing Alternative would require no additional permits or approvals but would have no ability to adapt to potential future changes in drinking water quality requirements.		

3.2 WATER SYSTEM SUPPLY

3.2.1 Long List of Alternative Water Supply Solutions

To support forecasted growth of 10,800 persons and meet the projected maximum demand of 89.5 L/s, additional water supply is required. To address the identified need, eight (8) alternative servicing solutions were developed for this project and are listed below:

1. **Do Nothing.** Permit the growth, but do not increase the capacity of the existing water supply system;
2. **Limit Growth.** Limit the growth up to the existing capacity of the current water supply system;
3. **Water Conservation.** Implement practices for efficient water use to reduce water usage per person;
4. **Increase Capacity of Existing Well(s)**. Increase water production and treatment capacity from existing well sites through facility upgrades and increases to PTTWs;
5. **Increase Capacity of Existing Well(s) in Combination with a New Production Well.** Maximize production and treatment capacity of existing well sites and establish a new well site and its associated water treatment facility;
    a) **Increase Existing Well #2 & Add New Well at Exploration Site H:** (Site H is located at the same site as the Existing Nobleton Well #5; further details on the Well Exploration Sites can be found in the Nobleton Groundwater Drilling Site Selection Report)
    b) **Increase Existing Well #2 & Add New Well at Exploration Site F:** (Site F is located along lands adjacent to Highway 27 approximately 950m south of King Road; further details on the Well Exploration Sites can be found in the Nobleton Groundwater Drilling Site Selection Report)
6. **Increase Capacity Only with New Production Wells.** Establish new well sites to increase total supply and treatment capacity;
7. **Blended System with Addition of Lake Based Connection to Existing Wells**. New transmission main (and booster pump station) to connect to existing nearby lake-based water system (Kleinburg or King City); and
8. **New Water Supply Source from Humber River.** Construct a new water treatment plant, pump station and watermain to use the Main Branch of the Humber River as a new water supply source.

3.2.2 Screening of Long List of Alternative Water Supply Solutions

The long list of alternative water servicing solutions is screened according to the screening criteria presented in Section 2.1. Each alternative's ability to meet the criteria is noted by the following symbols, "✓" for Pass and "✗" for Fail. The screening results are presented in Table 3-4.

The screening process eliminated the following five out of the eight proposed water servicing solutions:

- The first two alternatives, "Do Nothing" and "Limit Growth", are eliminated due to their inability to provide additional capacity for the forecasted growth.
- The third and fourth alternative, "Water Conservation" and "Increase Capacity of Existing Well(s)", are eliminated since they cannot account for all the growth in water supply needs. However, it is recommended that "Water Conservation" be accounted for in the overall servicing strategy with its added benefits in potentially reducing the size of any future infrastructure requirements.
- The seventh alternative, "Blended System with Addition of Lake Based Connection to Existing Wells" does not meet the regulatory requirements associated with the Greenbelt Plan. However, this alternative would become feasible if increasing well capacity in Nobleton is deemed not feasible. Therefore, the evaluation of this alternative will conditionally proceed to detailed evaluation. Ongoing groundwater exploration study is being undertaken in order to confirm whether future well supply could meet the quantity and quality required to service the community of Nobleton.
- The eighth and final alternative, "New Supply Source from the Main Branch of the Humber River", is also eliminated due to the Humber River's limited capacity as a new source of water supply.

The following three alternative solutions, which are deemed feasible to support forecasted growth in the community of Nobleton are carried forward for detailed evaluation:

- Alternative 5: "Increase Capacity of Existing Well(s) in Combination with New Production Well(s)";
- Alternative 6: "Increase Capacity Only with New Production Well(s)"; and
- Alternative 7: "Blended System with Addition of Lake Based Connection to Existing Wells".

Table 3-4: Screening of the Long List of Alternative Water Supply Solutions

LONG LIST OF ALTERNATIVE WATER SERVICING SOLUTIONS	SCREENING CRITERIA		NOTES
	TECHNICAL	JURISDICTIONAL/	
1. Do Nothing	×	✓	o **This alternative is unable** to provide additional capacity for the forecasted growth. However, it is not screened out in order to provide a baseline for comparison of the alternatives.
2. Limit Growth	×	✓	o **Eliminated** due to its inability to meet the forecasted growth.
3. Water Conservation	×	✓	o **Eliminated** as a stand-alone alternative because water conservation alone is unable to account for all the growth in water supply needs, resulting in an inability to meet the forecasted growth. o However, it is recommended that this alternative be accounted for in the overall servicing strategy since it can help partially reduce the projected average and maximum day demands, thereby reducing future capacity need requirements.
4. Increase Capacity of Existing Well(s)	×	✓	o **Eliminated** as a stand-alone alternative as it cannot support the forecasted growth. Out of three existing wells, only one of the wells (Well #2) is considered to have the potential to increase capacity (as discussed in Section 1.2) o By only expanding the capacity of Well #2, the three existing wells would only be able to increase firm capacity up to a maximum of approximately 57.8 L/s, which is significantly lower than the required capacity (Black & Veatch, *Study 1A: Water System Capacity Optimization Study,* 2019).
5. Increase Capacity of Existing Well(s) in Combination with New Production Well.	✓	✓	o **Proceed to Detailed Evaluation.** Able to support forecasted growth in the community of Nobleton while meeting the jurisdictional and regulatory requirements.
6. Increase Capacity Only with New Production Wells	✓	✓	o **Proceed to Detailed Evaluation.** Able to support forecasted growth in the community of Nobleton while meeting the jurisdictional and regulatory requirements.
7. Blended System with Addition of Lake Based Connection to Existing Wells	✓	✓ (*)	o ***Conditionally* Proceed to Detailed Evaluation.** The Greenbelt Plan restricts the extension of lake-based water servicing, unless well supply is proven to be insufficient to service the forecasted community growth, due to either quality reasons (water quality unable to meet required standards) or quantity (insufficient well capacity available from aquifer). o This alternative would become feasible if increasing well capacity in Nobleton is deemed not feasible. Concurrently, an ongoing groundwater exploration study is being undertaken to determine whether future well supply is able to meet the quantity and quality required to service the community of Nobleton. Therefore, the evaluation of this alternative will be carried forward and used in the event that the wells are not sufficient.
8. New Water Supply Source from the Main Branch of the Humber River	×	✓	o **Eliminated** due to the Humber River's limited capacity as a new source of water supply. Based on the Assimilative Capacity Study report (Hutchinson, *Humber River Assimilative Capacity Study*, 2019), 7Q20 flow, which represents the minimum 7-day average flow of Humber River in a recurrence period of 20 years, was reported to be 510 L/s. o The average maximum daily demand of 89.5 L/s would be approximately 17.5 percent of the 7Q20 flow. This is a large amount of water to take from a river, therefore, it is concluded that the Humber River does not have enough capacity to meet the demands of Nobleton.

3.2.3 Short List of Alternative Water Supply Solutions

Three alternative water supply solutions are carried forward for detailed evaluation. A description of each alternative is provided in the subsequent sections, with a graphical comparison of the three alternatives presented in Figure 3-1.

Table 3-5: Short List of Alternative Water Supply Solutions for Detailed Evaluation

SHORT LISTED ALTERNATIVE WATER SUPPLY SOLUTIONS
A. Increase Capacity of Existing Well(s) in Combination with New Production Well(s)
B. Increase Capacity Only with New Production Well(s)
C. Blended System with Addition of Lake Based Connection to Existing Wells

3.2.3.1 - Alternative A: Increase Capacity of Existing Well(s) in Combination with New Production Well(s)

The Nobleton water supply system currently consists of three groundwater wells with a combined firm capacity of 51.6 L/s. As previously summarized in Section 1.2, it is understood that Well #2 has the potential for increased capacity up to 67 L/s (current capacity limit of 22.7 L/s).

Alternative A would involve a capacity increase to the existing Well #2 and its associated treatment facility. Based on the information from the Operation Manual, it was expected that, while maintaining sequestration for iron and manganese treatment, the capacity of Well #2 could be increased up to at least 32 L/s without any major upgrades to the existing treatment facility.

Results of a short-term pumping test conducted at Nobleton Production Well 2 (Nobleton PW2) on March 27, 2020 indicated that there is sufficient drawdown to sustain a rate of 34 L/s for at least 60 minutes. It was recommended that a longer pumping test (48 hours to 72 hours in duration) be conducted on Nobleton PW2 to confirm the well's and aquifer's abilities to sustain the target rate over the long term and establish the corresponding zone of influence (refer to Technical Memorandum: Nobleton PW2 Pumping Test Conducted on March 27, 2020 (Regine Cheung, April 30, 2020)).

At Nobleton Production Well #2, the capacity of the sodium silicate tank and chlorine contact tank was confirmed to ensure that they could operate at a flow of at least 34L/s (without requiring major work/expansions at the well facility). With the existing treatment processes, the increased flow rates required would lead to an increase in the chemical feed rates required in order to meet the target dosages reflected in the original design and current operations practice. Initial review of the existing treatment process equipment indicates that the in-place treatment process has the ability to treat the additional capacity with moderate increases to the amount of chemical feed. Assessment of existing Well #2 facilities indicated that additional facilities or treatment process capacity is not needed, therefore no change to the current site footprint are expected.

In addition to an expansion at Well #2, one new production well with its associated treatment facility would be required. This treatment facility is assumed to continue with the treatment processes used at the existing Nobleton wells (sequestration). Currently, it is assumed that the new well will have a PTTW and capacity of 32 L/s and the expanded Well #2 will increase its PTTW and rated capacity to 32 L/s (34L/s with storage deficit offset). Combined, the overall well production capacities would meet the projected maximum day demand of 89.5L/s, as presented in Table 3-6, plus the surplus supply capacity that would be required to offset the minor storage deficit.

Two alternative well sites are currently being evaluated to determine which is recommended. Site H is located at the existing site of Nobleton Well #5. Site F is located along lands adjacent to Highway 27 approximately 950m south of King Road. Further details on the Well Exploration Sites can be found in the Nobleton Groundwater Drilling Site Selection Report.

Table 3-6: Water Alternative A Conceptual Breakdown of Current and Future Well Capacity

CATEGORY	CAPACITY LIMIT	CONCEPTUAL FUTURE CAPACITY
Well #2 Capacity	22.7 L/s	**~ 32 L/s (expansion)**
Well #3 Capacity	28.9 L/s	28.9 L/s
Well #5 Capacity	28.9 L/s	28.9 L/s
New Production Well	**-**	**~ 32 L/s (new)**
Well Supply Firm Capacity (Largest Well out of Service)	51.6 L/s	89.8 L/s
Total Capacity	80.5 L/s	121.8 L/s

Alternative A involves increasing the capacity of existing Nobleton Well #2 and adding a single new production well. Since two potential new well sites are being explored in detail, this alternative can be further broken down into two sub-alternatives:

- Alternative A1: Increase Capacity of Existing Well #2 in Combination with New Production Well @ Site F (where Site F is located at a greenfield site)
- Alternative A2: Increase Capacity of Existing Well #2 in Combination with New Production Well @ Site H (where Site H is located at the existing Well #5 site)

3.2.3.2 Alternative B: Increase Capacity Only with New Production Well(s)

Similar to Alternative A, this proposed alternative would also rely on groundwater sources to provide additional water supply to meet the projected increases in water demand. Under this proposed alternative, additional groundwater production will be achieved solely through construction of new production well(s).

A single new production well (along with the existing facilities) would be insufficient to meet the 89.5 L/s demand, while maintaining the largest well out of service. So, Alternative B would require two new production wells along with their associated treatment facilities to meet projected demand. The conceptual breakdown of current and future well capacity, which would meet the projected maximum day demand of 89.5L/s, is presented in Table 3-7.

Table 3-7: Water Alternative B Conceptual Breakdown of Current and Future Well Capacity

CATEGORY	CAPACITY LIMIT	CONCEPTUAL FUTURE CAPACITY
Well #2 Capacity	22.7 L/s	22.7 L/s
Well #3 Capacity	28.9 L/s	28.9 L/s
Well #5 Capacity	28.9 L/s	28.9 L/s
New Production Well	-	**19.0 L/s (new)**
New Production Well #2	-	**19.0 L/s (new)**
Well Supply Firm Capacity (Largest Well out of Service)	51.6 L/s	89.6 L/s
Total Capacity	80.5 L/s	118.5 L/s

3.2.3.3 - Alternative C: Blended System with Addition of Lake Based Connection to Existing Wells

York Region currently utilizes the Lake Based Water System to bring drinking water from Lake Ontario to service various communities. For this alternative, a new transmission main connection to the existing lake-based water system would be constructed and sized adequately to meet projected demands. This new transmission main would become the main source of water supply within the community, and the existing wells would be used as backup/emergency supply.

There are multiple different possible connections for the lake-based water system. The two closest connections are via King City and via Kleinburg, but other considerations such as a connection from Bolton in Peel and a direct connection further south in the Region of York could also be considered. In order to provide sufficient information to conduct the alternative evaluation, a brief comparison of the two closest connection options are presented in Table 3-8.

Table 3-8: Lake-Based Connections Comparison (via King City and via Kleinburg)

CONNECTION VIA KING CITY	CONNECTION VIA KLEINBURG

• Approximate Length: 9-10 km
• Pump Requirement: No additional pumping required (King City TWL = 344.50m; Nobleton TWL: 323.25m)
• Assumed Route (along King Road)
  o Rural route
  o 9 stream crossings
  o Within Greenbelt Zone; runs adjacent to “significant forest” lands
  o Runs adjacent to wetland
  o Major highway crossing (Highway 400)
• King City Booster Pump Station may require upgrades depending on the planned growth in King City
	• Approximate Length: 5 km
	• Pump Requirement: Booster pump station required (Kleinburg TWL = 271m; Nobleton TWL: 323.25m)
	• Assumed Route (along Highway 27)
	  o Rural route
	  o 5 stream crossings
	  o Within Greenbelt Zone; runs adjacent to “significant forest” lands
	  o No wetlands along route; only few artificial water bodies
	• Kleinburg Booster Pump Station may require upgrades depending on the planned growth in Kleinburg

Based on the information provided above, a connection via Kleinburg is expected to have the least impact due to its reduced environmental impact and shorter distance to Nobleton. Therefore, for the detailed evaluation, the lake-based system will be based on the Kleinburg connection. However, if the lake-based system alternative is found to be the recommended solution, a more detailed comparison and analysis between the various alignments will be completed in Phase 3 of this Class EA. Hydraulic modelling of the various alignments would be conducted during Phase 3 to identify any bottlenecks that may exist within the existing Kleinburg or King City systems.

It is still important to note however, that since this alternative does not meet the regulatory requirements associated with the Greenbelt Plan, it should only ultimately be used if additional well supplies are proven to be insufficient, in either quality and quantity, to service the community of Nobleton. This can be seen from the following excerpt from the May 2017 Update of the "Growth Plan for the Greater Golden Horseshoe":

Section 3.2.6.3

For settlement areas that are serviced by rivers, inland lakes, or groundwater, municipalities will not be permitted to extend water or wastewater services from a Great Lakes source unless:

a. *the extension is required for reasons of public health and safety, in which case, the capacity of the water or wastewater services provided in these circumstances will be limited to that required to service the affected settlement area, including capacity for planned development within the approved settlement area boundary;*
b. *in the case of an upper- or single-tier municipality with an urban growth centre outside of the Greenbelt Area:*
    i. *the need for the extension has been demonstrated;*
    ii. *the increased servicing capacity will only be allocated to settlement areas with urban growth centres; and*
    iii. *the municipality has completed the applicable environmental assessment process in accordance with the Ontario Environmental Assessment Act;*
c. *the extension had all necessary approvals as of July 1, 2017 and is only to service growth within the settlement area boundary delineated in the official plan that is approved and in effect as of that date.*

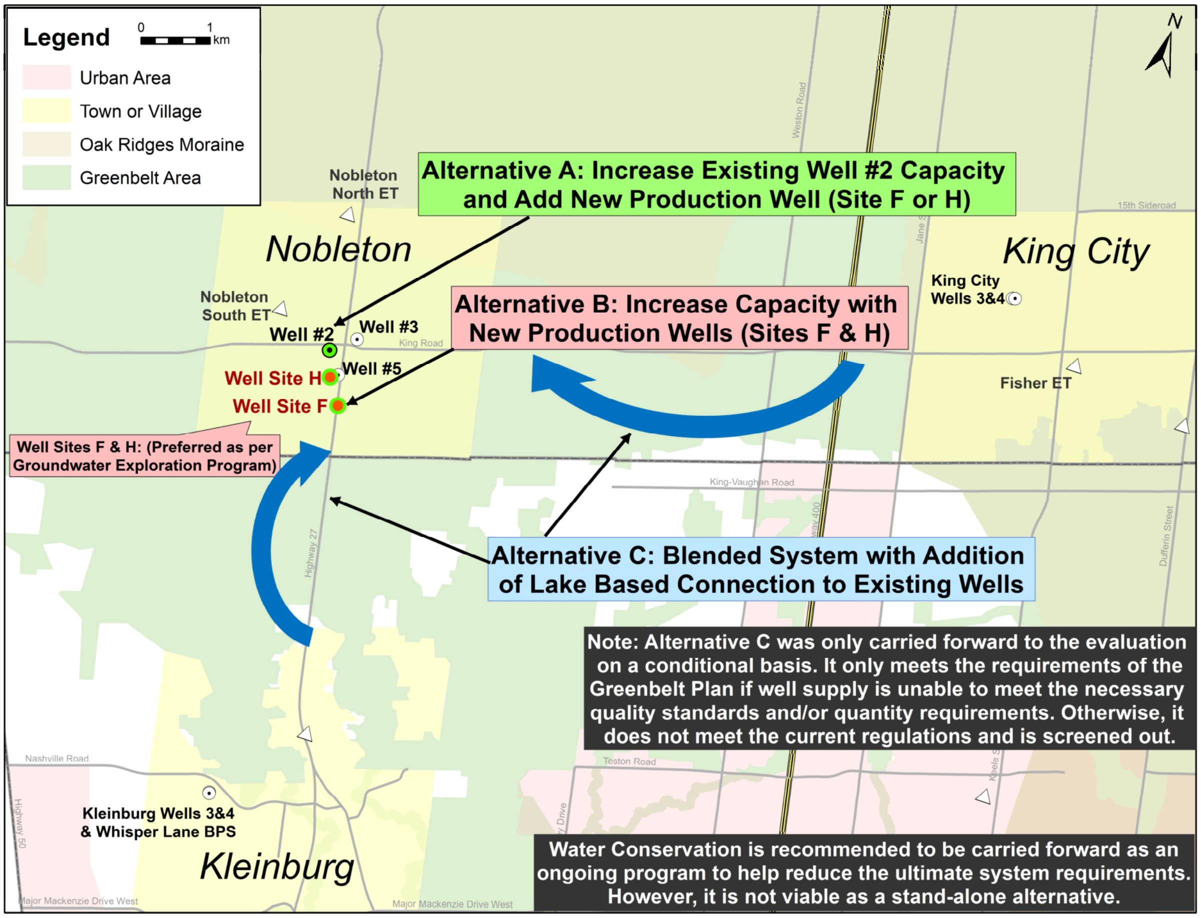

Figure 3-1: Nobleton System with Water System Alternatives

3.2.4 Evaluation of Short List of Alternative Water Supply Solutions

A detailed evaluation of the short-listed alternative water servicing solutions is carried out in accordance with the evaluation methodology described in Section 2.2 and is presented in Table 3-9.

3.2.5 Selection of Recommended Water Supply Solution

The detailed evaluation of the short-listed alternative water servicing solutions favoured **Alternative A2: "Increase Capacity of Existing Well #2 in Combination with New Production Well @ Site H"** to be the recommended servicing solution due to the following considerations:

- **Technical –** Alternative A1 and A2 scored similarly high due to their aim to maximize the capacity of existing Well #2. Although, they do not provide the same degree of redundancy as Alternative C, a blended (lake & well supply system), the proposed wells in Alternatives A1 and A2 would still be able to reliably meet the maximum day demands with one well out of service. Both Alternatives A1 and A2 maximize the use of existing infrastructure at Well Site # 2, while Alternative A2 also maximizes the use of the existing Well Site #5. Both alternatives have low levels of O&M complexity associated. Alternative A1 allows more space for maintenance work and Alternative A2 allows for greater convenience of daily operation, with two wells at one site. Alternative A2 is considered better than Alternative A1 in terms of constructability and adaptability to existing infrastructure, as connecting to the existing distribution network at Site F would impact traffic along Highway 27 and require stream crossing. Alternative A2 would result in the lowest volume and complexity of construction compared to other alternatives, thus minimizing potential disturbance to the community during construction. Alternative A2 ranked highest overall. The Do Nothing Alternative would not be able to meet forecasted growth.
- **Environmental** – There are no significant risks expected to aquatic and terrestrial vegetation and wildlife under Alternative A1, A2 or B. Some impact is expected to groundwater resources in comparison to having a lake-based system, however, groundwater production is within acceptable limits to ensure no significant risk to existing resources. Alternative C is expected to have significant impact on aquatic and terrestrial vegetation and wildlife, as well as greenhouse gas emissions. Without any system upgrades there would be no environmental impacts associated with the Do Nothing Alternative.
- **Socio-Economic** – Under the socio-economic criteria, Alternative A2 scores better than the other alternatives. Like most construction, short-term impacts/nuisance to the community are expected due to increased traffic, noise and dust to adjacent areas. For Alternatives A1, A2 and B, Site F and Site H are both near residential areas. For A1 and B, Site F is adjacent to Highway 27, leading to some significant short-term traffic impacts along Highway 27. New well sites can be designed to mitigate long-term impacts to the community (e.g. visual and operating impacts), but Alternative A2 has the advantage of being confined to an existing well site. Based on the Stage 1 Archaeological Assessment, risk is low at each site, but Site F would require a Stage 2 AA, which is not required at Site H, impacting A1 and B. Without any system upgrades associated, the Do Nothing Alternative has low socio-economic impacts, apart from its inability to meet planned growth.
- **Financial** – Alternatives A1 and A2 were found to be similarly low-cost alternatives in terms of the overall lifecycle cost, despite higher initial capital and land acquisition costs at Site F, and slightly lower O&M costs at Site H. Alternative B is moderate in cost and Alternative C is the highest cost overall. Alternative A2 is ranked the highest. Without any system upgrades, the Do Nothing Alternative has no associated costs.
- **Jurisdictional** – All alternatives have the ability to accommodate potential future changes in drinking water quality requirements, except the Do Nothing Alternative. However, for permits

and approval, due to the new transmission watermain crossing the Greenbelt Plan's "Protected Countryside", it would be far more challenging to acquire approval for construction of Alternative C than Alternatives A1, A2 or B. Alternatives A1 and B would require land acquisition which would not be required for Alternative A2. So, Alternative A2 is ranked the highest.

Overall, Alternative A2 scored well in all five categories of the detailed evaluation criteria. It slightly outscored Alternative A1 and noticeably outscored Alternatives B and C. Therefore, Alternative A was found to be the recommended servicing solution to address the identified need to increase the water supply and support the forecasted growth in the community of Nobleton.

Table 3-9: Short Listed Alternative Water Supply Solutions - Detailed Evaluation

EVALUATION CRITERIA	DO NOTHING	ALTERNATIVE A1: INCREASE CAPACITY OF EXISTING WELL #2 IN COMBINATION WITH NEW PRODUCTION WELL @ SITE F	ALTERNATIVE A2: INCREASE CAPACITY OF EXISTING WELL #2 IN COMBINATION WITH NEW PRODUCTION WELL @ SITE H	ALTERNATIVE B: INCREASE CAPACITY ONLY WITH NEW PRODUCTION WELLS	ALTERNATIVE C: BLENDED SYSTEM WITH ADDITION OF LAKE BASED CONNECTION TO EXISTING WELLS
CONCEPTS	➢ Included for comparative purposes. ➢ Hypothetical concept which permits the planned growth without providing any solution.	➢ Minor system upgrades to existing Well #2 (from 22.7 L/s to ~ 32 L/s) and its treatment facility. ➢ Plus one new production well (~ 32 L/s) along with its associated treatment facility @ Site F	➢ Minor system upgrades to existing Well #2 (from 22.7 L/s to ~ 32 L/s) and its treatment facility. ➢ Plus one new production well (~ 32 L/s) along with associated treatment facility upgrades @ Site H (Well Site #5)	➢ Two new production wells (each well ~19 L/s) along with their associated treatment facility ➢ One new production well (~ 32 L/s) along with its associated treatment facility @ Site F ➢ One new production well (~ 32 L/s) along with associated treatment facility upgrades @ Site H (Well Site #5)	➢ Three existing wells would be maintained as backup/standby supply to the water system. (they would be capable of providing ultimate average day demands) ➢ Lake based supply would become the primary supply. Currently, assumed connection via Kleinburg which requires ~5km of transmission main to connect to Nobleton and a new booster pump station (BPS)
TECHNICAL					
A. CONSTRUCTABLITY	LOW IMPACT	MODERATE IMPACT	LOW IMPACT	MODERATE IMPACT	HIGH IMPACT
• What are the major construction challenges and risks (e.g. crossing environmentally sensitive areas, noise, odour, dust, public safety, traffic, etc.) associated with the alternative? • To what extent does it impact the community? • How much volume and complexity of construction will be associated with the alternative?	➢ No construction to be conducted as part of "Do-Nothing"	➢ Minor impact expected in the residential neighborhood adjacent to Well #2 during upgrades, but no major construction challenges expected. ➢ No major constructability challenges are expected for the construction of the new well. Site F is a greenfield site which would have minimal construction challenges. ➢ There would be some traffic impacts associated with connecting to the existing distribution network along Highway 27, a heavily used throughway. ➢ Connection to distribution network requires stream crossing. ➢ Longer construction schedule than A2.	➢ Minor impact expected in the residential neighborhood adjacent to Well #2 during upgrades, but no major construction challenges expected. ➢ Maintaining operation of Well # 5 during construction of A2 at existing site would require more constructability review and staging, than construction of A1 at greenfield Site F. ➢ However, no significant volume or complexity of construction is expected. ➢ Connection to the existing distribution network could be made at Site H, resulting in fewer challenges than connection to A1. ➢ Shorter construction schedule than A1.	➢ No major constructability challenges are expected but moderate impact to the community due to construction of two new wells is expected. ➢ Site F is a greenfield site which would have minimal construction challenges. However, as described under Alternative A1, connection to existing distribution network would impact traffic along Highway 27 and require stream crossing. ➢ Construction of a new well at Site H would require constructability review and staging to maintain operation of Well #5. However, no significant volume or complexity of construction are expected and connection to the existing distribution network could be made on site. ➢ Relatively high volume of construction required for two new production wells, as compared to one new production well each, for Alternatives A1 and A2.	➢ Approximately 5 stream crossings are expected for the transmission main, and dewatering may be required which could pose moderate construction challenges. ➢ Low utility conflicts are expected for transmission main installation due to rural location. Most construction work is to be within right-of-way however, transmission main will cross through Green Belt zones. Moderate impacts on local traffic due to road construction (right-of-way) are expected. ➢ Relatively higher volume of construction required due to construction of the 5km transmission main and a booster pump station.

B. REDUNDANCY OF SUPPLY/SERVICE	LOW REDUNDANCY	MODERATE REDUNDANCY	MODERATE REDUNDANCY	MODERATE REDUNDANCY	HIGH REDUNDANCY
• Will the alternative be able to provide improvements in redundancy of supply or service?	➢ Without any system upgrades, the planned growth cannot be met. Therefore, there is also insufficient redundancy.	➢ Largest well can be taken out of service while still being able to supply the maximum demand. ➢ Minor redundancy concern that the wells are all located within the same groundwater source. However, likelihood of aquifer issue is low. ➢ Relative to Alternative A2, somewhat greater available well supply (pumping rate). Allows for better redundancy if other wells taken out of service.	➢ Largest well can be taken out of service while maintaining supply. ➢ Minor redundancy concern that the wells are all located within the same groundwater source. However, likelihood of aquifer issue is low. ➢ Slightly less redundancy than Alternative A1 since Site H would share a facility with existing Well #5. If a local surface level spill occurs both wells could be affected, though the risk is minor.	➢ Largest well can be taken out of service while still being able to supply the maximum demand. ➢ Minor redundancy concern that the wells are all located within the same groundwater source. However, likelihood of aquifer having quantity or quality issue is low. ➢ Minor risk if local surface level spill occurs at Site H, could affect new well and existing Well #5 as both wells would share a facility.	➢ Improvement in redundancy due to the addition of lake-based supply via transmission main along with the existing well supply. ➢ Increased reliability from any supply issues due to having two different supply sources: lake based (surface water) and groundwater.
C. RESILIENCE TO CLIMATE CHANGE	LOW RESILIENCE	MODERATE RESILIENCE	MODERATE RESILIENCE	MODERATE RESILIENCE	MODERATE RESILIENCE
• Will the alternative have the resilience against changing climate conditions, such as changes to water supply quantity and quality (e.g. high water demands, drought)	➢ Without any system upgrades, the planned growth cannot be met. Therefore, there is also no resilience to increasing demands due to climate change	➢ Deep groundwater well supply is generally resistant to changing climate (some impacts on quantity from droughts). ➢ Quality of groundwater is more resilient to climate change than lake-based supplies due to the potential algae blooms in lakes. ➢ Less flexibility to high demands	➢ Deep groundwater well supply is generally resistant to changing climate (some impacts on quantity from droughts). ➢ Quality of groundwater is more resilient to climate change than lake-based supplies due to the potential algae blooms in lakes. ➢ Less flexibility to high demands	➢ Deep groundwater well supply is generally resistant to changing climate (some impacts on quantity from droughts). ➢ Quality of groundwater is more resilient to climate change than lake-based supplies due to the potential algae blooms in lakes. ➢ Less flexibility to high demands	➢ Lake based system would have more flexibility to increase supply within shorter notice in comparison to groundwater supply ➢ Deep groundwater well supply is generally resistant to changing climate (some impacts on quantity from droughts). ➢ Quality of groundwater is more resilient to climate change than lake-based supplies due to the potential algae blooms in lakes. ➢ This alternative has flexibility since it could use either source if/when future challenges arise.

D. O & M REQUIREMENTS • What will be the level of additional and new O&M resources (e.g. human resources) required for the alternative? • What will be the level of complexity and maintainability of new and optimized assets?	**LOW COMPLEXITY**	**LOW COMPLEXITY**	**LOW COMPLEXITY**	**MODERATE COMPLEXITY**	**HIGH COMPLEXITY**
	➢ No upgrades, so there are no additional facilities to operate and maintain.	➢ Low additional resource requirements to maintain and operate one new production well. ➢ No major changes in O&M requirements are expected at existing wells. Well #2 would have additional treatment O&M requirements to replenish chlorine & sodium silicate. ➢ No major impact to system complexity. ➢ More space at this site for significant maintenance work than Alternative A2.	➢ Low additional resource requirements to maintain and operate one new production well at same site as existing Well #5. ➢ No major changes in O&M requirements are expected at existing wells. Well #2 would have additional treatment O&M requirements to replenish chlorine & sodium silicate. ➢ No major impact to system complexity. ➢ Convenient for daily tasks to have two wells at same site. However, less space at this site for significant maintenance work.	➢ Moderate additional resource requirements to maintain and operate two new production wells. ➢ No major changes in O&M requirements are expected at existing wells. ➢ No major impact to system complexity. ➢ Convenient for daily tasks to have two wells at Site H. Space constraints for significant maintenance work due to operating two wells from one facility.	➢ Potential O&M increases because the high-water age of supply from the lake-based system would likely require increased flushing (lower-tier). ➢ Low additional resource requirements to maintain and operate new booster pump station. ➢ Existing wells are still to be maintained as backup/emergency supply (some blending of sources will occur when the wells operate with the lake-based supply, which may potentially cause water quality issues).
E. ADAPTABILITY TO EXISTING INFRASTRUCTURE • What will be the level of modification required to the existing infrastructure to adapt to the alternative? What is the relative ease of connection to the existing alternative?	**HIGH**	**MODERATE ADAPTABILITY**	**HIGH ADAPTABILITY**	**MODERATE ADAPTABILITY**	**LOW ADAPTABILITY**
	➢ No planned upgrades, so there is no new infrastructure that needs to connect to the existing system.	➢ Minor modifications required at existing Well #2 and its associated treatment facility to increase capacity. ➢ Connecting piping from new production well to existing distribution piping would requires stream crossing and traffic impacts to Highway 27. ➢ No impact to other infrastructure. ➢ New connection to sanitary sewer required, or storage facility for disposal of sanitary and treatment process waste.	➢ Minor modifications required at existing Well #2 and its associated treatment facility to increase capacity. ➢ Connecting piping from new production well to existing facility expected to be straightforward. ➢ Initial assessment of Well #5 Site indicates that it can allow for the expansion of the existing treatment facility to accommodate both the new and existing wells. However, some existing infrastructure may need to be relocated slightly and construction staging would need to minimize disruption to Well #5 operation.	➢ No new changes required to existing infrastructure. ➢ At Site H connecting piping from new production well to existing facility expected to be straightforward. ➢ At Site F connecting piping from new production well to existing distribution piping would require stream crossing and traffic impacts to Highway 27.	➢ Modification is expected to the existing infrastructure. There is a need to convert chlorine disinfection at Nobleton wells to chloramine disinfection to be consistent with the lake-based water supply (or vice-versa). ➢ Potential challenges in Kleinburg system if upgrades are needed at Kleinburg BPS. ➢ Lake-based supply systems have reduced alkalinity which could impact wastewater treatment process requirements.

F. MAXIMIZING USE OF EXISTING INFRASTRUCTURE • Will the alternative be able to maximize the capacity of the existing infrastructure to reduce new assets needs?	**LOW DEGREE** ➢ Without any system upgrades, there is no ability to maximize the capacity of existing infrastructure.	**HIGH DEGREE** ➢ Continues to use all existing wells and aims to maximize capacity of existing Well #2.	**HIGH DEGREE** ➢ Continues to use all existing wells and aims to maximize capacity of existing Well #2. ➢ Uses existing Well # 5 treatment facility however, will require duplication of some pumping and/or treatment piping and equipment.	**MODERATE DEGREE** ➢ Continues to use existing wells to their current limits, however, it does not maximize potential takings from existing wells (Well #2).	**LOW DEGREE** ➢ New connection would become primary source of water supply. Existing wells would only be used for emergency or backup supply.
OVERALL TECHNICAL RATING • Based on all above technical criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**HIGH IMPACT** ➢ Without any system upgrades, the planned growth cannot be met.	**MODERATE IMPACT** ➢ Moderate impacts due to constructability and ability to adapt to existing infrastructure. Low complexity of O&M. Maximizes use of existing infrastructure. ➢ All groundwater alternatives provide moderate redundancy and resiliency.	**LOW IMPACT** ➢ Low impacts associated with constructability, low complexity of O&M and ability to adapt to existing infrastructure. Maximizes use of existing infrastructure. ➢ All groundwater alternatives provide moderate redundancy and resiliency.	**MODERATE IMPACT** ➢ Moderate impacts due to constructability, O&M complexity and ability to adapt to existing infrastructure. Moderately maximizes use of existing infrastructure. ➢ All groundwater alternatives provide moderate redundancy and resiliency.	**HIGH IMPACT** ➢ High impacts due to constructability, high complexity of O&M, low adaptability to existing infrastructure and low degree of maximizing use of existing infrastructure.
OVERALL TECHNICAL SUMMARY	➢ Alternative A1 and A2 scored similarly high due to their aim to maximize the capacity of existing Well #2. Although, they do not provide the same degree of redundancy as a blended (lake & well) supply system, the proposed wells in Alternatives A1 and A2 would still be able to reliably meet the maximum day demands with one well out of service. Both A1 and A2 maximize the use of existing infrastructure at Well Site # 2, while A2 also maximizes use of existing Well Site #5. Both alternatives have low levels of O&M complexity associated, with A1 allowing more space for maintenance work and A2 allowing for greater convenience of daily operation, with two wells at one site. Alternative A2 is considered better than Alternative A1 in terms of constructability and adaptability to existing infrastructure, as connecting to the existing distribution network at Site F would impact traffic along Highway 27 and require stream crossing. Alternative A2 would result in the lowest volume and complexity of construction compared to other alternatives, thus minimizing potential disturbance to the community during construction. Alternative A2 is ranked highest.				

ENVIRONMENTAL					
G. AQUATIC VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: streams and river; local aquatic species and habitat; environmentally sensitive areas, aquatic species at risk and locally significant aquatic species.	**LOW IMPACT** ➢ Without any system upgrades, there is no impact to aquatic vegetation /wildlife.	**LOW IMPACT** ➢ No significant risk to aquatic vegetation and wildlife are expected ➢ Minimal impact expected from expansion of existing well. ➢ Potential short-term impact during construction of new well due to erosion and sediment washout. Non-damaging construction techniques and erosion controls will be employed to minimize impact.	**LOW IMPACT** ➢ Slightly higher risk to aquatic vegetation and wildlife is expected than Alternative A1 since Site H is adjacent to a watercourse. This watercourse is linked to redside dace and therefore has stringent discharge requirements. ➢ Minimal impact expected from expansion of existing well. ➢ Potential short-term impact during construction of new well due to erosion and sediment washout. Non-damaging construction techniques and erosion controls will be employed to minimize construction impact.	**LOW IMPACT** ➢ Slightly higher risk to aquatic vegetation and wildlife is expected than Alternative A1 since Site H is adjacent to a watercourse. This watercourse is linked to redside dace and therefore has stringent discharge requirements. ➢ Potential short-term impact during construction of two new wells due to erosion and sediment washout. Non-damaging construction techniques and erosion controls will be employed to minimize construction impact.	**MODERATE IMPACT** ➢ Moderate to significant impact with approximately 5 stream crossings are expected. Although, non-damaging construction techniques would be employed, the risk remains intact. ➢ Small risk of impact resulting from future watermain break resulting in the discharge of chlorinated water to the streams. ➢ Potential short-term impact during construction of new pump station due to erosion and sediment washout. Non-damaging construction techniques and erosion controls will be employed to minimize construction impact.
H. TERRESTRIAL VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: trees and vegetation; local terrestrial species and habitats; environmentally sensitive areas, etc.	**LOW IMPACT** ➢ Without any system upgrades, there is no impact to terrestrial vegetation/wildlife.	**MODERATE IMPACT** ➢ Minimal impact is expected from upgrades at existing well ➢ New well site currently being considered does avoid environmentally sensitive areas, wetlands, water bodies, etc. ➢ Limited impact expected, but some impact from construction likely to remain.	**MODERATE IMPACT** ➢ Minimal impact is expected from upgrades at existing well ➢ New well site currently being considered does avoid environmentally sensitive areas, wetlands, water bodies, etc. ➢ Limited impact expected, but some impact from construction likely to remain.	**MODERATE IMPACT** ➢ New well sites currently being considered do both avoid environmentally sensitive areas, wetlands, water bodies, etc. ➢ Limited impact expected, but some impact from construction likely to remain.	**HIGH IMPACT** ➢ Potential impact due to construction in right-of-way through the Green Belt zone are expected ➢ Depending on the location of new BPS, there is potential risk associated with construction of the new pump station on a greenfield site. Phase 3 site selection would generally consider this impact.

I. GROUNDWATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on aquifers and groundwater resources? (discharge to water bodies, quantity, recharge quality)	**LOW IMPACT**	**MODERATE IMPACT**	**MODERATE IMPACT**	**MODERATE IMPACT**	**LOW IMPACT**
	➢ Without any system upgrades, there is no impact to groundwater resources.	➢ High transmissivity of aquifer indicates groundwater supply potential. No significant risk to groundwater resources is expected; groundwater production is to be within acceptable limits. ➢ Based on aquifer testing, new well at Site F is expected to achieve the target pumping rate of 35+ L/s. ➢ A1 would result in less groundwater interference effects to the existing municipal well network than A2. ➢ A1 is expected to have minor interference effects (<1 m drawdown) with three (3) private wells screened in the Scarborough Aquifer located on Hilda Road/ Diana Drive. This interference is not expected to adversely affect groundwater quality or quantity in the existing private wells.	➢ High transmissivity of aquifer indicates groundwater supply potential. No significant risk to groundwater resources is expected; groundwater production is to be within acceptable limits. ➢ Based on aquifer testing, new well at Site H is expected to achieve the target pumping rate of 35+ L/s. ➢ A new pumping well at A2 will have moderate interference effects with the existing municipal well network, particularly Well #5. However, detailed hydraulic testing demonstrated that these effects would not adversely affect yields from A2 or the existing municipal well network. ➢ No private wells are expected to be affected under A2.	➢ High transmissivity of aquifer indicates groundwater supply potential. No significant risk to groundwater resources is expected; groundwater production is to be within acceptable limits. ➢ Based on aquifer testing, wells at Site F and H are both expected to achieve the target pumping rate of 35+ L/s. ➢ A new well at Site F is expected to have minor interference effects (<1 m drawdown) with three (3) private wells screened in the Scarborough Aquifer located on Hilda Road/ Diana Drive. This interference is not expected to adversely affect groundwater quality or quantity in the existing private wells. ➢ A new pumping well at Site H will have moderate interference effects with the existing municipal well network, particularly Well #5. However, detailed hydraulic testing demonstrated that these effects would not adversely affect yields from the new well or the existing municipal well network.	➢ No significant risk to groundwater resources is expected.
J. SURFACE WATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on adjacent surface water resources? (Humber)	**LOW IMPACT**	**LOW IMPACT**	**LOW IMPACT**	**LOW IMPACT**	**LOW IMPACT**
	➢ Without any system upgrades, there is no impact to surface water resources.	➢ No significant risk to surface water resources	➢ No significant risk to surface water resources	➢ No significant risk to surface water resources	➢ No significant risk to surface water resources

K. GREENHOUSE GAS EMISSIONS • What will be the level of greenhouse gas emissions associated with the alternative? *(Greenhouse gas emission evaluation is estimated based on energy intensity)*	**LOW IMPACT** ➢ Without any system upgrades, there is no added impact greenhouse gas emissions.	**MODERATE IMPACT** ➢ Energy required from wells are generally low. Existing Nobleton wells have an approximate energy intensity of 900 kWh/ML	**MODERATE IMPACT** ➢ Energy required from wells are generally low. Existing Nobleton wells have an approximate energy intensity of 900 kWh/ML	**MODERATE IMPACT** ➢ Energy required from wells are generally low. Existing Nobleton wells have an approximate energy intensity of 900 kWh/ML	**HIGH IMPACT** ➢ Energy required to pump from Lake Ontario to Nobleton is significantly higher than groundwater wells. Lake Ontario energy intensity is greater than 1500 kWh/ML
OVERALL ENVIRONMENTAL RATING • Based on all above environmental criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**LOW IMPACT** ➢ Without any system upgrades, there are no environmental impacts.	**MODERATE IMPACT** ➢ No significant risks to aquatic vegetation and wildlife and surface water resources. ➢ Minimal impacts to terrestrial vegetation and wildlife expected. ➢ Moderate impacts to groundwater resources, and greenhouse gas emissions.	**MODERATE IMPACT** ➢ No significant risks to aquatic vegetation and wildlife and surface water resources. ➢ Minimal impacts to terrestrial vegetation and wildlife expected. ➢ Moderate impacts to groundwater resources, and greenhouse gas emissions.	**MODERATE IMPACT** ➢ No significant risks to aquatic vegetation and wildlife and surface water resources. ➢ Minimal impacts to terrestrial vegetation and wildlife expected. ➢ Moderate impacts to groundwater resources, and greenhouse gas emissions.	**HIGH IMPACT** ➢ Moderate to significant impacts expected to aquatic and terrestrial vegetation and wildlife. ➢ High impacts to greenhouse gas emissions.
OVERALL ENVIRONMENTAL SUMMARY	➢ There are no significant risks expected to aquatic and terrestrial vegetation and wildlife under Alternative A1, A2 or B. Some impact is expected to groundwater resources in comparison to having a lake-based system, however, groundwater production is within acceptable limits to ensure no significant risk to existing resources. Alternative C is expected to have significant impact on aquatic and terrestrial vegetation and wildlife, as well as greenhouse gas emissions. Without any system upgrades there would be no environmental impacts associated with the Do Nothing Alternative.				

SOCIO-ECONOMIC					
L. SHORT-TERM COMMUNITY IMPACTS • Will the alternative have significant short-term impacts to the community during construction, including: noise, dust and odour; or local traffic.	**LOW IMPACT**	**MODERATE IMPACT**	**LOW IMPACT**	**MODERATE IMPACT**	**HIGH IMPACT**
	➢ Without any system upgrades, there is no additional construction that would lead to community impacts.	➢ Well #2 is within the residential neighborhood, although upgrades are expected to be minor, noise, dust and construction traffic will cause some short-term impacts to the neighborhood although this can be mitigated to some extent. ➢ Short-term impact/nuisance to the community are expected during construction of the new well, including noise, dust and impact to the local traffic. Connecting to the existing distribution network at Highway 27 would impact traffic along highway. Mitigation measures will be employed during design and construction to minimize impact. ➢ Well Site F is adjacent to Highway 27 and within 300m radius of residential properties, so some short-term impact will exist.	➢ Well #2 is within the residential neighborhood, although upgrades are expected to be minor, noise, dust and construction traffic will cause some short-term impacts to the neighborhood although this can be mitigated to some extent. ➢ Short-term impact/nuisance to the community are expected during construction of the new well, including noise, dust and impact to the local traffic. Mitigation measures will be employed during design and construction to minimize impact. Construction confined to existing sites. ➢ Well Site H is adjacent to some residential properties, increasing effects of short-term impacts such as noise and dust on local community.	➢ Short-term impact/nuisance to the community are expected during construction of the new well at Site H, including noise, dust and impact to the local traffic. Mitigation measures will be employed during design and construction to minimize impact. Construction confined to existing site. ➢ Well Site H is adjacent to some residential properties, increasing effects of short-term impacts such as noise and dust on local community. ➢ Connecting to the existing distribution network at Site F would impact traffic along Highway 17. Mitigation measures will be employed during design and construction to minimize impact. ➢ Well Site F is adjacent to Highway 27 and within 300m radius of residential properties, so some short-term impact will exist.	➢ Construction of new transmission main would impact local traffic, routes will be assessed to minimize impact. ➢ Likely that a 5km stretch of Highway 27 would cause greater short-term impact than well alternatives. ➢ Short-term impact/nuisance to the community are expected during construction of pump station, including: noise, dust and impact to the local traffic. Mitigation measures will be employed during design and construction to minimize impact.
M. LONG-TERM COMMUNITY IMPACT • Will the alternative have significant long-term impact to the community, including: Benefit to Community; Impacts from Facility Operations; Visual Impact; Public Acceptance/ Resistance.	**MODERATE IMPACT**	**MODERATE IMPACT**	**MODERATE IMPACT**	**MODERATE IMPACT**	**HIGH IMPACT**
	➢ Without any system upgrades, it is not possible to meet the planned growth. This would impact the community since the growth helps the local economy grow.	➢ One new facility to accommodate treatment would be constructed. New well site can be designed to mitigate long-term impact to community. ➢ Minimal visual and operating impacts are expected. ➢ Potential ongoing aesthetic complaints from residents regarding groundwater quality due to high iron and manganese. ➢ Potential impacts to community from new wellhead protection area (e.g. restrictions on herbicide and pesticide use on nearby agricultural land). Mitigation measures could be applied to reduce impacts on community.	➢ Expanded existing facility to accommodate treatment would be constructed. Upgraded well site can be designed to mitigate long-term impact to community. ➢ Minimal visual and operating impacts are expected. ➢ Potential ongoing aesthetic complaints from residents regarding groundwater quality due to high iron and manganese.	➢ One new facility to accommodate treatment would be constructed and a second existing facility would be expanded. New and upgraded well sites can be designed to mitigate long-term impact to community. ➢ Minimal visual and operating impacts are expected. ➢ Potential ongoing aesthetic complaints from residents regarding groundwater quality due to high iron and manganese. ➢ Potential impacts to community from new wellhead protection area (e.g. restrictions on herbicide and pesticide use on nearby agricultural land). Mitigation measures could be applied to reduce impacts on community.	➢ No major long-term impact is expected after construction of transmission main. For the booster pump station, a small size pump station will provide more flexibility to search for a suitable site (e.g. with minimal likelihood of impact to community). ➢ Minimal visual impact is expected. ➢ The switch to lake supply could reduce water quality complaints. However, public resistance may be expected due to potential resistance to lake-based supply in case it encourages further growth/sprawl. ➢ Does not follow the Growth Plan for the Greater Golden Horseshoe, so public resistance is expected.

N. ARCHAEOLOGICAL SITES • Will the alternative have significant impacts during construction and/or from ongoing operations on registered/ known archaeological features?	**LOW IMPACT** ➢ Without any system upgrades, there is no additional construction that would lead to archaeological impact.	**LOW IMPACT** ➢ New location would be on a greenfield site (farmland). ➢ Stage 1 archeological assessment has not identified any major risk of archeological potential at Site F. ➢ Stage 2 archeological assessment would be required (pedestrian survey); however no major risk of archeological potential is expected based on Stage 1 AA findings.	**LOW IMPACT** ➢ New location would be at the existing Nobleton Well #5 property. ➢ Stage 1 archeological assessment has not identified any risk of archeological potential at Site H, since the entire parcel was previously assessed in 2007.	**LOW IMPACT** ➢ Proposed locations require confirmation that no archaeological impacts exist. ➢ Stage 2 archeological assessment would be required (pedestrian survey); however no major risk of archeological potential is expected based on Stage 1 AA findings.	**MODERATE IMPACT** ➢ New transmission main to be within right-of-way, therefore minimal risk of impact expected. ➢ Sites for the new pump station could potentially be on a greenfield site. ➢ Larger area for Stage 1 archeological assessment would be required if this alternative was to proceed further.
O. CULTURAL / HERITAGE FEATURES • Will the alternative have significant impacts during construction and/or from ongoing operations on known cultural landscapes and built heritage?	**LOW IMPACT** ➢ Without any system upgrades, there is no additional construction that would lead to a cultural/heritage impact.	**LOW IMPACT** ➢ The well locations considered are not located near any of the heritage properties in Nobleton.	**LOW IMPACT** ➢ The well locations considered are not located near any of the heritage properties in Nobleton.	**LOW IMPACT** ➢ The well locations considered are not located near any of the heritage properties in Nobleton.	**LOW IMPACT** ➢ New transmission main to be within right-of-way, therefore, minimal risk of impact expected. ➢ There are no heritage properties along the considered route from Kleinburg to Nobleton.
OVERALL SOCIO-ECONOMIC RATING • Based on all above socio-economic criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**LOW IMPACT** ➢ Without any system upgrades, no socio-economic impacts apart from inability to meet planned growth.	**MODERATE IMPACT** ➢ Moderate short- and long-term impacts to community. ➢ Low impacts to archeological and cultural/heritage sites/features.	**LOW IMPACT** ➢ Low to moderate short- and long-term impacts to community. ➢ Low impacts to archeological and cultural/heritage sites/features.	**MODERATE IMPACT** ➢ Moderate short- and long-term impacts to community. ➢ Low impacts to archeological and cultural/heritage sites/features.	**HIGH IMPACT** ➢ High short- and long-term impacts to community. ➢ Low to moderate impacts to archeological and cultural/heritage sites/features.
OVERALL SOCIO-ECONOMIC SUMMARY	➢ Under the socio-economic criteria, Alternative A2 scores better than the other alternatives. Like most construction, short-term impacts/nuisance to the community are expected due to increased traffic, noise and dust to adjacent areas. For Alternatives A1, A2 and B, Site F and Site H are both near residential areas. For A1 and B, Site F is adjacent to Highway 27, leading to some significant short-term traffic impacts along Highway 27. New well sites can be designed to mitigate long-term impacts to the community (e.g. visual and operating impacts), but Alternative A2 has the advantage of being confined to an existing well site. Based on the Stage 1 Archaeological Assessment, risk is low at each site, but Site F would require a Stage 2 AA, which is not required at Site H, impacting A1 and B. Without any system upgrades associated, the Do Nothing Alternative has low socio-economic impacts, apart from its inability to meet planned growth, which would impact the local economy.				

FINANCIAL					
P. LAND ACQUISITION COST • What will be the relative land acquisition cost for the alternative?	**LOW IMPACT** ➢ Without any system upgrades, there is no land acquisition needed.	**MODERATE COST ALTERNATIVE** ➢ One new site for the new production well at Site F will need to be purchased. ➢ Upgrades for existing Well #2 expected to be within existing footprint.	**LOW COST ALTERNATIVE** ➢ All upgrades/expansion expected to be within the existing parcels owned at Well Site #2 and Well Site #5, so no land acquisition is required.	**MODERATE COST ALTERNATIVE** ➢ One new site for the new production well at Site F will need to be purchased.	**MODERATE COST ALTERNATIVE** ➢ New transmission main to be within the right-of-way, no additional cost for land acquisition expected. ➢ New land would be required for a PS from Kleinburg to Nobleton, however, a relatively smaller area/land would be required in comparison to the production wells & treatment.
Q. CAPITAL COST • What will be the relative capital cost for the alternative?	**LOW IMPACT** ➢ Without any system upgrades, there is no upfront capital cost.	**MODERATE COST ALTERNATIVE** ➢ Comparatively moderate amount of construction needed. ➢ At Site F, new well and treatment facility will be required along with costs of connecting watermain from Site F to the existing Nobleton system along Highway 27.	**LOW COST ALTERNATIVE** ➢ Least amount of construction needed. ➢ Site H will require a new well and contact chamber (dedicated to the new well). ➢ Site H is located at the existing Well Site #5 and would use the upgraded existing treatment facility, avoiding the cost of a new facility.	**MODERATE COST ALTERNATIVE** ➢ Comparatively moderate amount of construction required with two new well facilities. ➢ Connecting two new wells to the existing distribution network would be more costly than connecting one new well at Site F or H, alone (Alternatives A1 and A2, respectively).	**HIGH COST ALTERNATIVE** ➢ Comparatively high amount of construction required with approximately 5km of piping and a new pump station. ➢ Would also require modifications to treatment at existing wells (chlorine to chloramine disinfection).
R. LIFECYCLE COST • What will be the relative lifecycle cost for the alternative?	**LOW COST ALTERNATIVE** ➢ With no system upgrades there is no associated lifecycle cost. O&M costs limited to existing costs.	**LOW COST ALTERNATIVE** ➢ One additional production well & treatment facility to maintain and operate. ➢ Higher initial capital and land acquisition costs, but the overall lifecycle is only slightly higher when compared to A2.	**LOW COST ALTERNATIVE** ➢ One additional production well & upgraded treatment facility to maintain and operate. ➢ Slightly lower O&M with Site H facilities included on existing site. ➢ Over lifecycle, slightly lower lifecycle costs when compared to A1.	**MODERATE COST ALTERNATIVE** ➢ Two new production wells & treatment facilities to maintain and operate.	**HIGH COST ALTERNATIVE** ➢ High water age from lake-based system is likely to require closer management of flushing. ➢ Need to maintain existing wells, despite infrequent use. ➢ Additional O&M cost from Peel/Toronto

OVERALL FINANCIAL RATING • Based on all above financial criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**LOW IMPACT** ➢ Without any system upgrades, no associated costs.	**MODERATE IMPACT** ➢ Moderate land acquisition and capital costs associated with alternative. ➢ Similar overall lifecycle cost when compared to A2.	**LOW IMPACT** ➢ No land acquisition cost and lowest capital cost associated with alternative. ➢ Lowest overall lifecycle cost.	**MODERATE IMPACT** ➢ Moderate land acquisition and capital costs. ➢ Higher lifecycle costs associated with operating two new wells, as compared to A1 and A2.	**HIGH IMPACT** ➢ Moderate land acquisition costs. ➢ High capital costs and high lifecycle costs associated with alternative.
OVERALL FINANCIAL SUMMARY	➢ Alternatives A1 and A2 were found to be similarly low-cost alternatives in terms of the overall lifecycle cost, despite higher initial capital and land acquisition costs at Site F, and slightly lower O&M costs at Site H. Alternative B is moderate in cost and Alternative C is the highest cost overall. Alternative A2 is ranked the highest. Without any system upgrades, the Do Nothing Alternative has no associated costs.				
JURISDICTIONAL / REGULATORY					
S. LAND REQUIREMENTS • What will be the level of area of non-regional land or easement required to construct the alternative?	**LOW REQUIREMENT** ➢ Without any system upgrades, there is no land acquisition needed.	**MODERATE REQUIREMENT** ➢ A new production well will require new land acquisition at Site F.	**LOW REQUIREMENT** ➢ With upgrades within the existing parcels owned at Well Site #2 and Well Site #5, then no land acquisition is required.	**MODERATE REQUIREMENT** ➢ A new production well will require new land acquisition at Site F.	**MODERATE REQUIREMENT** ➢ Pump station would require new land acquisition, but a relatively smaller footprint required in comparison to the new production wells.
T. ABILITY TO ACCOMMODATE POTENTIAL FUTURE REGULATORY CHANGES • Will the alternative have the ability to adapt to potential future changes in drinking water quality requirements?	**LOW ADAPTABILITY** ➢ Without any system upgrades, does not have the ability to adapt to potential future changes.	**HIGH ADAPTABILITY** ➢ Meets current water quality regulations. Potential changes to water treatment requirements not expected to have significant impact. ➢ Has the ability to adapt to future changes in drinking water quality requirements.	**HIGH ADAPTABILITY** ➢ Meets current water quality regulations. Potential changes to water treatment requirements not expected to have significant impact. ➢ Has the ability to adapt to future changes in drinking water quality requirements.	**HIGH ADAPTABILITY** ➢ Meets current water quality regulations. Potential changes to water treatment requirements not expected to have significant impact. ➢ Has the ability to adapt to future changes in drinking water quality requirements.	**HIGH ADAPTABILITY** ➢ Lake-based treatment process tends to be highly adaptable to changing regulatory requirements.
U. PERMITS AND APPROVALS • What will be the level of permits and approvals required to construct the alternative?	**LOW REQUIREMENT** ➢ Without any system upgrades, there are no additional permits/ approvals required.	**MODERATE REQUIREMENT** ➢ Will require a new PTTW from the MECP for increased water takings. PTTW also required during construction (dewatering). ➢ Site plan and local permits as required for the design and construction of the new production well and its associated treatment facility. ➢ Permit required for stream crossing.	**MODERATE REQUIREMENT** ➢ Will require a new PTTW from the MECP for increased water takings. PTTW also required during construction (dewatering). ➢ Site plan and local permits as required for the design and construction of new infrastructure on the existing site. ➢ Due to Site H's proximity to the adjacent watercourse with redside dace, there are additional permits (and restrictions) regarding discharge that would need to be adhered to.	**MODERATE REQUIREMENT** ➢ Will require two new PTTW from the MECP for increased water takings. PTTW also required during construction (dewatering). ➢ Site plan and local permits as required for the design and construction of the new production wells and the associated treatment facilities. ➢ Due to Site H's proximity to the adjacent watercourse with redside dace, there are additional permits (and restrictions) regarding discharge that would need to be adhered to. ➢ Permit required for stream crossing.	**HIGH REQUIREMENT** ➢ New transmission main would cross the Greenbelt Plan's "Protected Country Side" and would be challenging to acquire approvals due to Greenbelt protection. ➢ Permits are required for the design and construction of the new watermain. ➢ Permit requirements for dewatering (stream crossings). ➢ Would require a modification of the Water Purchasing Agreements to bring Lake-Based water to Nobleton

OVERALL JURISDICTIONAL/ REGULATORY RATING • Based on all above jurisdictional/ regulatory criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**MODERATE IMPACT** ➢ Without any system upgrades, there is no need for land acquisition or additional permits/approvals. ➢ Has no ability to adapt to potential future changes in drinking water quality requirements.	**MODERATE IMPACT** ➢ Requires new land acquisition and some additional permits/approvals. ➢ Is able to adapt to potential future changes in drinking water quality requirements	**LOW IMPACT** ➢ Requires no new land acquisition, however does require some additional permits/approvals. ➢ Is able to adapt to potential future changes in drinking water quality requirements	**MODERATE IMPACT** ➢ Requires new land acquisition and some additional permits/approvals. ➢ Is able to adapt to potential future changes in drinking water quality requirements	**HIGH IMPACT** ➢ Requires some new land acquisition and potentially challenging permits/approvals. ➢ Is able to adapt to potential future changes in drinking water quality requirements
OVERALL JURISDICTIONAL/ REGULATORY SUMMARY	➢ All alternatives have the ability to accommodate potential future changes in drinking water quality requirements, except the Do Nothing Alternative. However, for permits and approval, due to the new transmission watermain crossing the Greenbelt Plan's "Protected Countryside", it would be far more challenging to acquire approval for construction of Alternative C than Alternatives A1, A2 or B. Alternatives A1 and B would require land acquisition which would not be required for Alternative A2. So, Alternative A2 is ranked the highest.				

4 Wastewater System Alternative Solutions

4.1 LONG LIST OF ALTERNATIVE SOLUTIONS

To support forecasted growth of 10,800 persons and meet the meet the future average day flow (ADF) of 3,996 m^3/d and peak flow of 25,174 m^3/d, eight (8) alternative servicing solutions were developed for this project and are listed below:

1. **Do Nothing.** Permit the growth, but do not increase the capacity of the existing wastewater system.
2. **Limit Growth.** Limit the growth up to the existing capacity of the current wastewater system.
3. **Water Conservation and Inflow and Infiltration (I&I) Reduction.** Implement practices for efficient water use and reduction of inflow and infiltration (I&I) into the sewage collection system to reduce future flows.
4. **Expand and Upgrade the Existing Janet Avenue Pumping Station, Forcemain and Nobleton WRRF and Outfall.** Increase the capacity of the existing wastewater facilities, forcemain and outfall to meet the future flow requirements.
5. **Construct a New Pumping Station, Forcemain and New WRRF and Outfall.** Maintain existing treatment and conveyance ADF capacity of 2,925 m^3/d and peak design flow of 9,177 m^3/d. Construct a new pump station, new forcemain and a new treatment facility, including a new outfall, to meet future flow requirements.
6. **Convey Additional Flows to Neighbouring WRRFs.** Maintain existing treatment and conveyance ADF capacity of 2,925 m^3/d and peak design flow of 9,177 m^3/d. Construct new pipelines or pump station to convey future excess flows to neighboring WRRFs. Currently, the Kleinburg WRRF does not have capacity available to allocate to the Community of Nobleton. However, it is understood that, in the future, the Community of Kleinburg would be ultimately serviced by the West Vaughan Sewage System (WVSS) (Lake Ontario-based treatment at Duffin Creek WPCP or G.E. Booth WPCP; not decided at the time of preparing this document) and the Kleinburg WRRF would be decommissioned (HMM, 2013). This is a long-term plan that would be implemented after the Kleinburg WRRF reaches its capacity.
7. **Convey All Flows to Lake-based Treatment Systems.** Decommission or repurpose the existing Nobleton WRRF and convey all current and future flows to either the York-Durham Sewage System (YDSS) or West Vaughan Sewage System (WVSS).
8. **Maintain Existing Treatment Facilities and Convey Additional Flows to Lake-based Treatment Facilities.** Maintain existing treatment and conveyance ADF capacity of 2,925 m^3/d and peak design flow of 9,177 m^3/d. Construct new pipelines and/or pump station to convey future excess flows to either the YDSS or to the WVSS.

4.2 SCREENING OF LONG LIST OF ALTERNATIVE SOLUTIONS

The long list of alternative wastewater servicing solutions is screened according to the “Pass/Fail” criteria presented in Section 2.1. The “pass/fail” criteria and each alternative’s ability to meet each criterion noted by the following symbols, “✓” for Pass and “✗” for Fail. See Table 4-1.

The screening process eliminated the following six out of eight proposed wastewater servicing solutions.

- The first two alternatives, “Do Nothing” and “Limit Growth”, do not provide additional capacity for forecasted growth.
- The third alternative, “Water Conservation and I&I Reduction” by itself is eliminated as a stand-alone alternative, as it alone is unable to support the forecasted growth.
- The sixth, seventh and eight alternatives, “Convey Additional Flows to Neighbouring WRRFs,” “Convey All Flows to Lake-based Treatment Facilities,” and “Maintain Existing Treatment Facilities and Convey Additional Flows to Lake-based Treatment Facilities” are eliminated as they do not meet jurisdictional/regulatory requirements for forecasted growth.

The two following alternative solutions, which are deemed feasible to support forecasted growth in the community of Nobleton, are carried forward for detailed evaluation:

- Alternative 4: “Expand and Upgrade the Existing Janet Avenue Pumping Station and Nobleton WRRF”
- Alternative 5: “Construct a New Water Resource Recovery Facility (WRRF)”

Table 4-1: Screening of Long-List Wastewater Alternative Servicing Solutions

LONG LIST OF ALTERNATIVE WASTEWATER SERVICING SOLUTIONS	SCREENING CRITERIA		NOTES
	TECHNICAL	JURISDICTIONAL/ REGULATORY	
1. Do Nothing	✗	✗	○ **Eliminated** due to its inability to provide additional capacity for the forecasted growth. However, this alternative will be retained in the detailed evaluation in order to provide a baseline for comparison.
2. Limit Growth	✗	✗	○ **Eliminated** due to its inability to meet the forecasted growth.
3. Water Conservation and Inflow and Infiltration (I&I) Reduction.	✗	✓	○ **Eliminated** as an alternative because I&I reduction alone is unable to account for all the increase in wastewater flows, resulting in inability to meet forecasted growth. ○ However, it is recommended that this alternative be accounted for in the overall servicing strategy as it can help reduce peak wastewater flows.
4. Expand and Upgrade the Existing Janet Avenue Pumping Station, Forcemain and Nobleton WRRF and Outfall	✓	✓	○ **Proceed to Detailed Evaluation.** Able to support forecasted growth in the community of Nobleton while meeting the jurisdictional and regulatory requirements.
5. Construct a New Pumping Station, Forcemain and New Water Resource Recovery Facility (WRRF) and Outfall	✓	✓	○ **Proceed to Detailed Evaluation.** Able to support forecasted growth in the community of Nobleton while meeting the jurisdictional and regulatory requirements.
6. Convey Additional Flows to Neighbouring WRRFs	✓	✗	○ **Eliminated.** It may be technically feasible to convey flows south to the Kleinburg WRRF; however, the Kleinburg WRRF currently does not have capacity allocated to accept any flows from Nobleton. This may change in the long-term future depending on the outcome of the WVSS project where flows generated in the community of Kleinburg will ultimately be conveyed to the WVSS after the Kleinburg WRRF capacity is reached (HMM, 2013). At that time, it may be technically possible to repurpose the Kleinburg WRRF to treat flows from Nobleton. ○ This Alternative is not in accordance with requirements set forth in the Greenbelt Plan (2017). ○ This Alternative is also inconsistent with the York Region Water and Wastewater Master Plan (2016).
7. Convey All Flows to Lake-based Treatment Facilities	✓	✗	○ **Eliminated.** Although it is technically feasible to construct conveyance facilities, this alternative contradicts the requirements of the Greenbelt Plan (2017). ○ This Alternative is inconsistent with the York Region Water and Wastewater Master Plan (2016).
8. Maintain Existing Treatment Facilities and Convey Additional Flows to Lake-based Treatment Facilities	✓	✗	○ **Eliminated.** Although it is technically feasible to construct conveyance facilities, this alternative contradicts the requirements of the Greenbelt Plan (2017). ○ This Alternative is inconsistent with the York Region Water and Wastewater Master Plan (2016).

4.3 SHORT LIST OF ALTERNATIVE SOLUTIONS

Two alternative wastewater servicing solutions are carried forward for detailed evaluation. Description of each alternative is provided in the subsequent sections.

Table 4-2: Short List of Alternative Wastewater Servicing Solutions

SHORT LISTED ALTERNATIVE WASTEWATER SERVICING SOLUTIONS
A. Expand and Upgrade the Existing Janet Avenue Pumping Station, Forcemain and Nobleton WRRF and Outfall
B. Construct a New Pumping Station, Forcemain and WRRF and Outfall

4.3.1 - Alternative A: Expand and Upgrade the Existing Janet Avenue Pumping Station, Forcemain and Nobleton WRRF and Outfall

The existing wastewater collection and treatment system would be upgraded and expanded as follows (Figure 4-1):

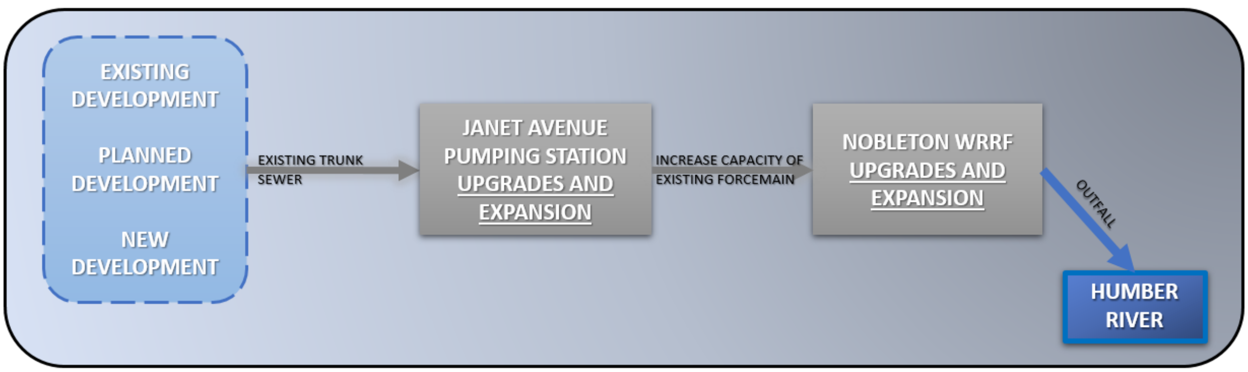

Figure 4-1: Alternative A: Expand and Upgrade the Existing Janet Avenue Pumping Station, Forcemain and Nobleton WRRF and Outfall

- Collection System – The existing trunk sewer has sufficient capacity to accommodate the future peak flows. Therefore, no expansion would be required.
- Janet Avenue Pumping Station – The existing Janet Avenue Pumping Station has a peak capacity of 9,177 m^3/d as identified in Study 1B. In order to accommodate the future peak flows, the Janet Avenue Pump Station would need to be expanded by either replacing the existing pumps with larger units or provision of additional pumps.
- Forcemain – The existing forcemain from the Janet Avenue Pumping Station to the Nobleton WRRF would need to be expanded to accommodate the future peak flows through either replacement with a larger pipe or addition of a second forcemain.
- Nobleton WRRF – The existing Nobleton WRRF would need to be expanded and upgraded to meet the future ADF, peak flows and effluent quality requirements identified in Study 1B. The expansion approach could include construction of additional treatment trains (from inlet works to disinfection) or intensification of the existing treatment trains or the combination of both. The detailed expansion and upgrade approach would be discussed in detail in Phase 3 should Alternative A be selected as the recommended alternative.

- Effluent Discharge and Outfall – The bottleneck for existing effluent discharge is within the effluent chamber and its inlet arrangement rather than with the outfall itself. Future peak flows greater than the existing rated peak flow of 9,177 m^3/d would need to be discharged into the existing outfall at MH 113 to prevent flooding in the existing facility.

- ***Wet Weather Flow (WWF) Management Strategy*** – Study 1B found that the existing infrastructure experiences high peak instantaneous flows, representing an average peaking factor of 6.3. This peaking factor was used to calculate future peak instantaneous flow requirements of 25,175 m^3/d. A WWF management strategy for reduction of high peak flows (WWF) into the wastewater system could reduce infrastructure costs for upgrades and expansion at the Janet Avenue Pumping Station and Nobleton WRRF. The following WWF management strategies could be considered:
  - *Flow Equalization* – High peak flows during wet weather events can be reduced by controlling the flow rates through the wastewater system. The approach for flow equalization could be:
    - Offline Equalization Storage Facility at the Collection System or Forcemain
    - Online Equalization Storage Facility at the Janet Avenue Pumping Station and/or Nobleton WRRF
    - Effluent Pump Station at the Nobleton WRRF

    The design concept for flow equalization to reduce peak flows during wet weather events will be developed during Phase 3, should Alternative A be selected as the recommended solution.
  - *Rainfall Derived Infiltration and Inflow (RDII) Reduction* – The Region has identified a high level of groundwater infiltration and RDII into the sewage collection system. Over the years, the Region has taken action to address sources of RDII and reduce peak flows into the wastewater system. The planned and the new development areas can be constructed with more stringent construction requirements and practices to reduce RDII.

4.3.2 Alternative B: Construct a New Pumping Station, Forcemain and WRRF and Outfall

Alternative B (Figure 4-2) would maintain the existing treatment and conveyance ADF capacity of 2,925 m^3/d and peak design flow of 9,177 m^3/d. New infrastructure, including a new pump station and a new WRRF, would be constructed to meet future flow requirements. The wastewater collection and treatment system for Alternative B is depicted in Figure 4-2:

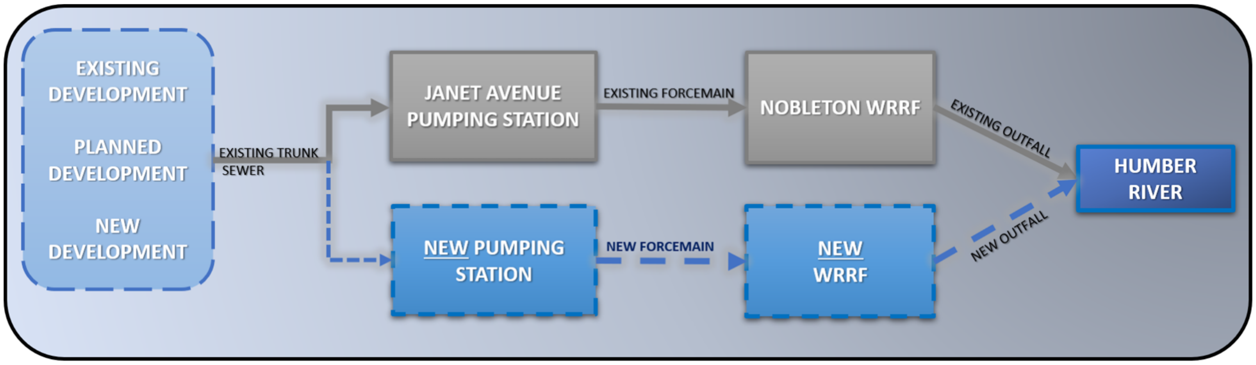

Figure 4-2: Alternative B: Construct a New Pumping Station, Forcemain and WRRF and Outfall

- ***Existing Wastewater System Facilities* -** Under this alternative, there are no system upgrades required at the existing facilities. The Janet Avenue Pumping Station and Nobleton WRRF would be maintained and continue to meet their current capacities.
- ***New Wastewater System Facilities* -** A new pumping station and a new WRRF would be constructed to meet future flow requirements. A WWF strategy could also be implemented for the new wastewater system to minimize the impact of high flows on infrastructure requirements.
  - Collection System - For the new development areas, a new collection system would be constructed and connected to the existing trunk sewer. From the existing trunk sewer, a new connection would be constructed to feed flows into the new Pumping Station.
  - New Pumping Station - A new pumping station constructed to accommodate the future peak flows.
  - Forcemain - A new forcemain conveying flow from the new pumping station to the new WRRF would be constructed.
  - New WRRF - The new WRRF would be constructed with to meet future ADF and peak flow requirements. The detailed design concepts for a new WRRF will be developed in Phase 3 should Alternative B be selected as the recommended solution.
  - Effluent Discharge and Outfall - A new outfall conveying effluent flow from the new WRRF into the Humber River would be constructed.
- ***WWF Management Strategy*** - A WWF management strategy for reduction of high peak flows into the wastewater system could reduce infrastructure costs for the new pumping station and WRRF. The following WWF management strategies can be considered:
  - *Flow Equalization* –The approach for flow equalization could be:
    - Offline Equalization Storage Facility Upstream of the New Pumping Station
    - Online Equalization Storage Facility at the new Pumping Station and/or the new WRRF
    - Effluent Pump Station at the new WRRF

    The design concept for flow equalization to reduce peak flows during wet weather events will be developed during Phase 3 should Alternative B be selected as the recommended solution.

- *Rainfall Derived Infiltration and Inflow (RDII) Reduction* – The Region has identified a high level of groundwater infiltration and RDII into the sewage collection system. Over the years, the Region has taken action to address sources of RDII and reduce peak flows into the wastewater system. The planned and the new development areas can be constructed with more stringent construction requirements and practices to reduce RDII.

4.4 EVALUATION OF SHORT LIST OF ALTERNATIVE SOLUTIONS

A detailed evaluation of short-listed alternative wastewater servicing solutions is carried out in accordance with the evaluation methodology described in Section 2.2 and are presented in Table 4-3.

4.5 SELECTION OF RECCOMENDED ALTERNATIVE SOLUTION

The detailed evaluation of the short-listed alternative wastewater servicing solutions favored **Alternative A: "Expand and Upgrade the Existing Janet Avenue Pumping Station, Forcemain and Nobleton WRRF and Outfall"** be the recommended servicing solution due to the following considerations:

- **Technical** – Alternative A ranked highest overall due to its ability to maximize the use of existing infrastructure and limit additional operations and maintenance resource requirements. This would be achieved through expansion and upgrading of existing infrastructure, requiring moderately more modification and optimization of existing facilities without introducing significant additional O&M requirements. In comparison Alternative B would not require changes to existing infrastructure, but would require new infrastructure and facilities, introducing additional construction and O&M requirements, with the need to operate and maintain additional facilities. Alternative B does not maximize the use of existing infrastructure. Alternatives A and B provide comparable resiliency and redundancy, with Alternative B also providing the potential for system redundancy, through interconnection between the separate facilities. The Do Nothing option has low impacts associated with construction, O&M complexity and adaption to existing infrastructure, but cannot meet forecasted growth.
- **Environmental** – Alternative A ranked highest overall as impacts are limited to upgraded and expanded existing sites and infrastructure, mitigating impacts to aquatic/terrestrial vegetation and wildlife, as well as greenhouse gas emissions. Both Alternatives A and B present a minimal potential risk to Humber River, with increase in effluent discharge to the river. However, findings of the assimilative capacity study would be used to determine final effluent quality required to mitigate impact on the river. Alternative A has a lower energy intensity requirement than Alternative B as operating two new facilities, including a new WRRF and pumping station, for a single community, is highly energy intensive. Without any system upgrades there would be no environmental impacts associated with the Do Nothing Alternative.
- **Socio-Economic** – Alternative A ranked highest overall as impacts are limited to upgraded and expanded existing sites and infrastructure. This mitigates short-term construction impacts, such as noise, dust and increased construction traffic, and minimizes potential impacts to archeological sites and cultural/heritage features. No significant long-term community impacts are expected, although there would be some increased sludge truck haulage from the upgraded and expanded WRRF, impacting local traffic. In comparison, Alternative B would have a high short-term and long-term community impact due to the construction of new facilities, increased

sludge truck haulage and potential visual impacts and negative public perception associated with building a second treatment facility. The Do Nothing Alternative also has low socio-economic impact, apart from the inability to meet forecasted growth that would help the local economy grow.

- **Financial** – Alternative A was found to be lowest cost alternative in all three criteria under the Financial category. By maximizing the capacity of the existing infrastructure and with expansion expected to be within the current or close proximity to the footprint of the existing facilities, Alternative A, was found to have lesser capital, lifecycle and land acquisition costs than Alternative B. The Do Nothing alternative would have no associated costs.
- **Jurisdictional** – Alternative A ranked highest as it requires limited land acquisition and fewer permits/approvals, while being able to adapt to potential future changes in final effluent requirements. Alternative B would require significant new land acquisition and additional permits/approvals, while the Do Nothing Alternative has no ability to adapt to potential future changes in drinking final effluent requirements.

Overall, Alternative A ranked the highest in all five main categories of the detailed evaluation criteria in comparison to Alternative B. Therefore, overall, Alternative A was identified be the recommended servicing solution to support the current and forecasted population growth in the community of Nobleton.

Table 4-3: Short Listed Alternative Wastewater Servicing Solutions Detailed Evaluation

EVALUATION CRITERIA	DO NOTHING	ALTERNATIVE A: EXPAND AND UPGRADE THE EXISTING JANET AVENUE PUMPING STATION, FORCEMAIN AND NOBLETON WRRF AND OUTFALL	ALTERNATIVE B: CONSTRUCT A NEW PUMPING STATION, FORCEMAIN AND NEW WRRF AND OUTFALL
TECHNICAL	Included in the Class EA process for comparative purposes. Hypothetical concept which permits the forecasted growth without providing any solution to address the servicing needs.	Increase the capacity of the existing wastewater facilities to meet the future flow requirements of 3,996 m^3/d ADF and 25,175 m^3/d peak flow.	Maintain existing treatment and conveyance ADF capacity of 2,925 m^3/d and peak design flow of 9,177 m^3/d. Construct a new treatment facility, including a new pump station and a new WRRF, to meet future ADF and peak flow requirements.
A. CONSTRUCTABLITY • What will be the major construction challenges and risks (e.g. crossing environmentally sensitive areas, noise, odour, dust, public safety, traffic, etc.) associated with the alternative? • At what extent does it impact the community? • How much volume and complexity of construction will be associated with the alternative	**LOW IMPACT** ➢ No construction to be conducted as part of "Do-Nothing"	**MODERATE IMPACT** ➢ The existing Janet Avenue Pumping Station and the Nobleton WRRF have limited space for the required expansion. ➢ Expansion of the existing pump station, forcemain, and the Nobleton WRRF could impact the local community (disturbance through traffic, dust, and noise). ➢ High volume of construction expected at the existing facility for expansion to meet future flow requirements. Constructability at the existing facilities for expansion would be challenging.	**MODERATE IMPACT** ➢ Construction of the new pump station, forcemain and WRRF could impact the local community (increased disturbance through traffic, dust and noise). ➢ Significant volume of construction is expected during the construction of the new pump station, forcemain and WRRF.
B. REDUNDANCY OF SUPPLY/SERVICE • Will the alternative be able to provide improvements in redundancy of service?	**LOW REDUNDANCY** ➢ Without any system upgrades, the forecasted growth cannot be met. Therefore, there is also insufficient redundancy.	**MODERATE REDUNDANCY** ➢ Existing system would be able to provide reliable wastewater collection and treatment system for future growth. ➢ Moderate redundancy for treatment capacity could be accommodated via expansion.	**HIGH REDUNDANCY** ➢ A new treatment system along with upgrades to the existing facility would be able to provide reliable wastewater collection and treatment system for future growth. ➢ Potential for system redundancy may be achieved through interconnection between separate facilities
C. RESILIENCE TO CLIMATE CHANGE • Will the alternative have the resilience against changing climate conditions, such as changes to wastewater flows (e.g. increase of intensity and frequency of wet weather flows)?	**LOW RESILIENCE** ➢ Without any system upgrades, the forecasted growth cannot be met. Therefore, there is also no resilience to increasing demands due to climate change	**MODERATE RESILIENCE** ➢ The existing system showed high RDII into the sewer system, however, the Region has been taking continuous measures for RDII reduction. The new development area could be constructed with tighter requirement to reduce RDII. ➢ Reduction in I&I would result in reduced peak flows into the existing facilities.	**MODERATE RESILIENCE** ➢ The existing system showed high RDII into the sewer system, however, the Region has been taking continuous measures for RDII reduction. The new development area could be constructed with tighter requirement to reduce RDII. ➢ Reduction in I&I would result in reduced peak flows into the existing facilities.
D. O & M REQUIREMENTS • What will be the level of additional and new O&M resources (e.g. human resources) required for the alternative? • What will be the level of complexity and maintainability of new assists?	**LOW COMPLEXITY** ➢ No upgrades, so there are no additional facilities to operate and maintain.	**LOW COMPLEXITY** ➢ No major changes would be expected in O&M requirements for the existing facility and the new collection system. ➢ New assets (from system upgrade and expansion) would be part of the existing facility which could be maintained holistically. No major complexity for maintenance of the new assets would be expected.	**HIGH COMPLEXITY** ➢ No major changes required in O&M requirements for the existing facilities. ➢ However, there would be new O&M requirements and resources required to maintain the new treatment facilities. ➢ Maintaining two separate treatment facilities would have added complexity in O&M requirements.

E. ADAPTABILITY TO EXISTING INFRASTRUCTURE • What will be the level of modification required to the existing infrastructure to connect to the alternative? What is the relative ease of connection to the existing alternative?	**HIGH ADAPTABILITY** ➢ No planned upgrades, so there is no new infrastructure that needs to connect to the existing system.	**MODERATE ADAPTABILITY** ➢ Modification would be required for the existing pumping station expansion and Nobleton WRRF expansion to meet future flow requirements ➢ Optimization and some modification would be required for the existing pump station and Nobleton WRRF	**HIGH ADAPTABILITY** ➢ No changes required to the existing wastewater system infrastructure, new facilities will be built to service all future growth. ➢ No major challenges are expected for connection from existing trunk sewer to the new facilities
F. MAXIMIZING USE OF EXISTING INFRASTRUCTURE • Will the alternative be able to maximize the capacity of the existing infrastructure?	**LOW DEGREE** ➢ Without any system upgrades, there is no ability to maximize the capacity of existing infrastructure.	**HIGH DEGREE** ➢ Aims to continuously use and optimize all existing facilities such as the existing trunk sewer, pump station and WRRF to service future needs	**LOW DEGREE** ➢ Brand new facility would be constructed for future growth and current needs, does not aim to maximize capacity of existing wastewater infrastructure
OVERALL TECHNICAL RATING Based on all above technical criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**HIGH IMPACT** ➢ Without any system upgrades, the forecasted growth cannot be met.	**MODERATE IMPACT** ➢ Low impacts associated low complexity of O&M. Maximizes use of existing infrastructure. ➢ Moderate impacts due to constructability and ability to adapt to existing infrastructure. Alternative provides moderate redundancy and resiliency.	**HIGH IMPACT** ➢ Low impacts associated with ability to adapt to existing infrastructure. ➢ Moderate impacts due to constructability. Alternative provides moderate resiliency. ➢ High impact associated with O&M complexity. Does not maximize use of existing infrastructure.
OVERALL TECHNICAL SUMMARY	Alternative A ranked highest overall due to its ability to maximize the use of existing infrastructure and limit additional operations and maintenance resource requirements. This would be achieved through expansion and upgrading of existing infrastructure, requiring moderately more modification and optimization of existing facilities without introducing significant additional O&M requirements. In comparison Alternative B would not require changes to existing infrastructure, but would require new infrastructure and facilities, introducing additional construction and O&M requirements, with the need to operate and maintain additional facilities. Alternative B does not maximize the use of existing infrastructure. Alternatives A and B provide comparable resiliency and redundancy, with Alternative B also providing the potential for system redundancy, through interconnection between the separate facilities. The Do Nothing option has low impacts associated with construction, O&M complexity and adaption to existing infrastructure, but cannot meet forecasted growth.		
ENVIRONMENTAL			
G. AQUATIC VEGETATION AND WILDLIFE ➢ Will the alternative have significant short and long-term impacts on: ○ Streams and river ○ Local aquatic species and habitat ○ Environmentally sensitive areas, aquatic species at risk and locally significant aquatic species	**LOW IMPACT** ➢ Without any system upgrades, there is no impact to aquatic vegetation /wildlife.	**MODERATE IMPACT** ➢ No significant risk expected to aquatic vegetation and wildlife during system expansion and upgrades of the Janet Avenue Pumping Station and the Nobleton WRRF, as expansion is expected to be in close proximity or within the existing footprint. ➢ Short term impacts during construction for replacement or twinning of existing forcemain or new connection to existing outfall are expected, but non-damaging construction techniques would be employed to minimize impact. ➢ Proven technology will be used to ensure that effluent quality meet requirements prior to discharge to Humber River to minimize impact.	**HIGH IMPACT** ➢ A new WRRF could have potential impact to the aquatic environment as new outfall would need to be installed. ➢ Proven technology will be used to ensure that effluent quality meet requirements prior to discharge to Humber River to minimize impact ➢ New treatment facility will require a second source of discharge, requiring a new capacity study at the point of discharge.

H. TERRESTRIAL VEGETATION AND WILDLIFE • Will the alternative have significant short and long-term impacts on: o Trees and vegetation o Local terrestrial species and habitats o Environmentally sensitive areas, species at risk and locally significant species	**LOW IMPACT** ➢ Without any system upgrades, there is no impact to terrestrial vegetation/wildlife.	**LOW IMPACT** ➢ Low risk expected to terrestrial vegetation and wildlife. System upgrade and expansion expected to be in close proximity or within the current footprint of the existing facilities. ➢ Short term impacts during construction for replacement or twinning of existing forcemain or new connection to existing outfall (proximity to wetlands) are expected, but non-damaging construction techniques would be employed to minimize impact.	**MODERATE IMPACT** ➢ Depending on the location of new treatment facility, potential risk to vegetation and wildlife with construction of the new Pumping Station and new WRRF on a greenfield site. ➢ Connection from existing trunk sewer to the new Pumping Station will be within right-of-way to reduce impact on terrestrial vegetation and wildlife.
I. GROUNDWATER RESOURCES • Will the alternative have significant short and long-term impacts on aquifers and groundwater resources such as: groundwater quantity, groundwater recharge quality and flow regime and groundwater discharge to streams and wetlands?	**LOW IMPACT** ➢ Without any system upgrades, there is no impact to groundwater resources.	**LOW IMPACT** ➢ Low impact expected to groundwater resources.	**LOW IMPACT** ➢ Low impact expected to groundwater resources.
J. SURFACE WATER RESOURCES • Will the alternative have significant short and long-term impacts on adjacent surface water resources (e.g. Humber River) and related biological communities?	**LOW IMPACT** ➢ Without any system upgrades, there is no impact to surface water resources.	**LOW IMPACT** ➢ Findings of assimilative capacity study would be used to determine final effluent quality to mitigate impact on the Humber River.	**LOW IMPACT** ➢ Findings of assimilative capacity study would be used to determine final effluent quality to mitigate impact on the Humber River.
K. GREENHOUSE GAS EMISSIONS • What will be the level of greenhouse gas emissions associated with the alternative? *(Greenhouse gas emission will be evaluation based on the alternative's energy intensity requirements)*	**LOW IMPACT** ➢ Without any system upgrades, there is no added impact greenhouse gas emissions.	**MODERATE IMPACT** ➢ Some changes expected with energy intensity requirement with the current system but not as significant in comparison to Alternative B. Energy saving technologies will be accounted for system upgrades and expansion.	**HIGH IMPACT** ➢ Energy intensity requirement is significantly higher when operating two new facilities, including a new WRRF and pumping station, for a single community.
OVERALL ENVIRONMENTAL RATING Based on all above environmental criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**LOW IMPACT** ➢ Without any system upgrades, there are no environmental impacts.	**LOW IMPACT** ➢ No significant risks to terrestrial or aquatic vegetation and wildlife. Low to moderate short-term impacts expected during construction but non-damaging construction techniques would be employed to minimize impact. ➢ Low impact expected to groundwater and surface water resources. Findings of assimilative capacity study would be used to mitigate impact to surface water resources. ➢ Moderate impacts on greenhouse gas emissions.	**MODERATE IMPACT** ➢ Potential risks to terrestrial or aquatic vegetation and wildlife. High to moderate short-term impacts expected during construction. ➢ Low impact expected to groundwater and surface water resources. Findings of assimilative capacity study would be used to mitigate impact to surface water resources. ➢ High impacts on greenhouse gas emissions - significantly higher energy intensity when operating two facilities.
OVERALL ENVIRONMENTAL SUMMARY	Alternative A ranked highest overall as impacts are limited to upgraded and expanded existing sites and infrastructure, mitigating impacts to aquatic/terrestrial vegetation and wildlife, as well as greenhouse gas emissions. Both Alternatives A and B present a minimal potential risk to Humber River, with increase in effluent discharge to the river. However, findings of the assimilative capacity study would be used to determine final effluent quality required to mitigate impact on the river. Alternative A has a lower energy intensity requirement than Alternative B as operating two new facilities, including a new WRRF and pumping station, for a single community, is highly energy intensive. Without any system upgrades, there are no environmental impacts. Without any system upgrades there would be no environmental impacts associated with the Do Nothing Alternative.		

SOCIO-ECONOMIC			
L. SHORT-TERM COMMUNITY IMPACTS • Will the alternative have significant short-term impacts to the community during construction, including: o Noise, dust and odour o Local traffic	**LOW IMPACT** ➢ Without any system upgrades, there is no additional construction that would lead to community impacts.	**MODERATE IMPACT** ➢ Existing Janet Avenue Pump station has been blended within a residential neighborhood, noise, dust and increased construction traffic during system upgrades could cause some short-term impacts to the neighborhood although which can be mitigated to some extent. ➢ Twinning or replacement of existing forcemain and connection to existing outfall would impact local traffic.	**HIGH IMPACT** ➢ Higher impact/nuisance during construction of the new WRRF to the community in comparison to Alternative A, including: noise, dust and impact to the local traffic. New facility site will be assessed during the design phase and mitigated as needed to reduce impact to community. ➢ Construction of trunk sewer connection, new forcemain and outfall would impact local traffic.
M. LONG-TERM COMMUNITY IMPACT • Will the alternative have significant long-term impact to the community, including: o Impact of operating facility o Visual impact o Public Acceptance/Resistance	**MODERATE IMPACT** ➢ Without any system upgrades, it is not possible to meet the forecasted growth. This would impact the community since the growth helps the local economy grow.	**MODERATE IMPACT** ➢ Increase in sludge truck haulage from the WRRF will impact local traffic. ➢ All new assets for system upgrade are expected to be within the current footprint or within close proximity to the existing facility.	**HIGH IMPACT** ➢ Increase in sludge truck haulage from the WRRF will impact local traffic. ➢ Potential visual impacts and negative public perception ("Not In My Backyard" – NIMBYism) associated with building a second treatment facility. ➢ New facility site will be assessed during the Phase 3 of this EA and mitigated as needed to reduce long-term impact to community, should this alternative is selected.
N. ARCHAEOLOGICAL SITES • Will the alternative have signification short and long-term impacts on registered/known archaeological features?	**LOW IMPACT** ➢ Without any system upgrades, there is no additional construction that would lead to archaeological impact.	**LOW IMPACT** ➢ All construction activities expected to take place on previously disturbed properties. Archeological potential not expected to be significant. ➢ Stage 1 archeological assessment has not identified any significant risk of archaeological potential at any of the potentially expanded well facilities. A Stage 2 assessment is required to further validate certain parts of the forcemain route along King Road.	**MODERATE IMPACT** ➢ Location of new facilities would be on a greenfield site (previously undisturbed farmland). Stage 1 archeological assessment would be conducted to confirm if there is archeological potential.
O. CULTURAL/HERITAGE FEATURES • Will the alternative have signification short and long-term impact on known cultural landscapes and built heritage features?	**LOW IMPACT** ➢ Without any system upgrades, there is no additional construction that would lead to a cultural/heritage impact.	**LOW IMPACT** ➢ All construction activities expected to take place on previously disturbed properties. ➢ Cultural heritage features would be assessed in Phase 3 of this EA.	**MODERATE IMPACT** ➢ Cultural heritage features would be assessed in Phase 3 of this EA.
OVERALL SOCIO-ECONOMIC RATING Based on all above socio-economic criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**LOW IMPACT** ➢ Without any system upgrades, no socio-economic impacts apart from inability to meet forecasted growth.	**LOW IMPACT** ➢ Moderate short- and long-term impacts to community. ➢ Low impacts to archeological and cultural/heritage sites/features.	**MODERATE IMPACT** ➢ High short- and long-term impacts to community. ➢ Low impacts to archeological and cultural/heritage sites/features.
OVERALL SOCIO-ECONOMIC SUMMARY	Alternative A ranked highest overall as impacts are limited to upgraded and expanded existing sites and infrastructure. This mitigates short-term construction impacts, such as noise, dust and increased construction traffic, and minimizes potential impacts to archeological sites and cultural/heritage features. No significant long-term community impacts are expected, although there would be some increased sludge truck haulage from the upgraded and expanded WRRF, impacting local traffic. In comparison, Alternative B would have a high short-term and long-term community impact due to the construction of new facilities, increased sludge truck haulage and potential visual impacts and negative public perception associated with building a second treatment facility. The		

	Do Nothing Alternative also has low socio-economic impact, apart from the inability to meet forecasted growth that would help the local economy grow.		
FINANCIAL			
P. LAND ACQUISITION COST • What will be relative land acquisition cost for the alternative?	**LOW IMPACT** ➢ Without any system upgrades, there is no land acquisition needed.	**LOW COST ALTERNATIVE** ➢ No land requirement for expansion and upgrade of existing WRRF on existing site. ➢ Minor land requirement may be required during expansion of Janet Avenue Pumping Station. ➢ Twinning or replacement of forcemain expected to be within right-of-way and upgrading of outfall expected to be within existing easement, so no land requirement expected for forcemain or outfall.	**HIGH COST ALTERNATIVE** ➢ Land acquisition would be required for the new WRRF to service new growth area. However, smaller land in comparison is required as new WRRF will only be used to service future growth.
Q. CAPITAL COST • What will be the relative capital cost for the alternative?	**LOW IMPACT** ➢ Without any system upgrades, there is no upfront capital cost.	**MODERATE COST ALTERNATIVE** ➢ Moderate amount of construction required within the existing facilities but considered to be a lower cost alternative in comparison	**HIGH COST ALTERNATIVE** ➢ Construction and commissioning of a new WRRF and pumping station for the newly developed area is expected to cost significantly more.
R. LIFECYLCE COST • What will the relative lifecycle cost for the alternative?	**LOW COST ALTERNATIVE** ➢ With no system upgrades there is no associated lifecycle cost. O&M costs limited to existing costs.	**MODERATE COST ALTERNATIVE** ➢ Lower O&M cost would be expected. ➢ Lower life cycle cost would be is also expected.	**HIGH COST ALTERNATIVE** ➢ Higher O&M cost would be required for two treatment facilities. ➢ Higher life cycle cost would be required for two treatment facilities.
OVERALL FINANCIAL RATING Based on all above financial criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**LOW IMPACT** ➢ Without any system upgrades, no associated costs.	**MODERATE IMPACT** ➢ Moderate land acquisition, capital and lifecycle costs associated with alternative.	**HIGH IMPACT** ➢ High land acquisition, capital and lifecycle costs associated with alternative.
OVERALL FINANCIAL SUMMARY	Alternative A was found to be lowest cost alternative in all three criteria under the Financial category. By maximizing the capacity of the existing infrastructure and with expansion expected to be within the current or close proximity to the footprint of the existing facilities, Alternative A, was found to have lesser capital, lifecycle and land acquisition costs than Alternative B. The Do Nothing alternative would have no associated costs.		
JURISDICTIONAL/REGULATORY			
S. LAND REQUIREMENTS • What will be the level of area of non-regional land or easement required to construct the alternative?	**LOW REQUIREMENT** ➢ Without any system upgrades, there is no land acquisition needed.	**LOW REQUIREMENT** ➢ No land requirement for expansion and upgrade of existing WRRF on existing site. ➢ Minor land requirement may be required during expansion of Janet Avenue Pumping Station. ➢ Twinning or replacement of forcemain expected to be within right-of-way and upgrading of outfall expected to be within existing easement, so no land requirement expected for forcemain or outfall.	**HIGH REQUIREMENT** ➢ New pumping station and WRRF will require land acquisition. ➢ New trunk sewer for the new development area to be within right-of-way, no new land acquisition expected but trunk sewer alignment may need easement.

T. ABILITY TO ACCOMMODATE POTENTIAL FUTURE REGULATORY CHANGES • Will the alternative have the ability to adapt to potential future changes in final effluent requirements?	**LOW ADAPTABILITY** ➢ Without any system upgrades, does not have the ability to adapt to potential future changes.	**HIGH ADAPTABILITY** ➢ Technologies used for upgrade and expansion could be selected to account for more stringent future requirement. ➢ Higher flexibility in choosing new technologies for the expansion to account for potential future changes in final effluent requirements.	**HIGH ADAPTABILITY** ➢ Higher flexibility in choosing new technologies for the new WRRF to account for potential future changes in final effluent requirements.
U. PERMITS AND APPROVALS • What will be the level of permits and approvals required to construct the alternative?	**LOW REQUIREMENT** ➢ Without any system upgrades, there are no additional permits/ approvals required.	**MODERATE REQUIREMENT** ➢ Will require an amended ECA permit. ➢ Site plan and local permits as required for the system upgrade and expansion of the existing system.	**HIGH REQUIREMENT** ➢ Will require a new ECA permit. ➢ Second source of discharge for the new treatment plant will also require approval and permit. ➢ Site plan and local permits as required for the design and construction of the of the new WRRF. ➢ Degree of permits and approval required to construct a new treatment facility is expected to be significantly higher.
OVERALL JURISDICTIONAL/ REGULATORY RATING Based on all above jurisdictional/ regulatory criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	**MODERATE IMPACT** ➢ Without any system upgrades, there is no need for land acquisition or additional permits/approvals. ➢ Has no ability to adapt to potential future changes in drinking final effluent requirements.	**LOW IMPACT** ➢ Not expected to require significant new land acquisition, however does require some additional permits/approvals. ➢ Is able to adapt to potential future changes in final effluent requirements.	**MODERATE IMPACT** ➢ Requires significant new land acquisition and additional permits/approvals. ➢ Is able to adapt to potential future changes in final effluent requirements.
OVERALL JURISDICTIONAL/ REGULATORY SUMMARY	Alternative A ranked highest as it requires limited land acquisition and fewer permits/approvals, while being able to adapt to potential future changes in final effluent requirements. Alternative B would require significant new land acquisition and additional permits/approvals, while the Do Nothing Alternative has no ability to adapt to potential future changes in drinking final effluent requirements.		

5 Summary and Recommendations

The key findings of this report are separated into water and wastewater servicing solutions:

Alternative Water Servicing Solutions

The water system alternatives evaluation is split up into two main categories:

1) Alternative Solutions to Address the Storage Deficit
2) Alternative Solutions to Address the Supply Deficit

Storage

- In terms of storage capacity, the existing Nobleton system has storage volume capable of providing storage requirements (fire, equalization and emergency storage) up to the equivalent of a maximum day demand of 87.40 L/s. Since the projected MDD is 89.5 L/s, this means that there would ultimately be a marginal storage deficit, if no action was taken. To address this need, various storage alternatives are developed and evaluated.
- Out of six alternative water storage solutions, four were screened out during the screening process. The following shortlist of alternative solutions proceeded into detailed evaluation:
  - Alternative A: "New Storage Facility"
  - Alternative B: "Supplement Increased Supply to Offset Storage Deficit"
- The detailed evaluation of the short-listed alternative water storage solutions favored **Alternative B: "Supplement Increased Supply to Offset Storage Deficit"** to be the recommended servicing solution.
- Alternative B considers increasing the combined PTTW and supply capacity in Nobleton to exceed the forecasted maximum day demand by 2 L/s (>91.5L/s). By exceeding the maximum day demand (even slightly), it allows for the wells to operate at a higher rate during the hours when demand exceeds the average maximum day demand. This reduces the amount of equalization storage required because some of the equalization is pumped (rather than being stored).

Supply

- To support forecasted growth of 10,800 persons and meet the projected maximum demand of 89.5 L/s, additional water supply is required. To address the identified need, eight (8) alternative servicing solutions were developed for this project.
- Out of eight alternative water supply solutions five were screened out during the screening process. The following shortlist of alternative solutions proceeded into detailed evaluation:
  - Alternative A: "Increase Capacity of Existing Well(s) in Combination with New Production Well(s)"
  - Alternative B: "Increase Capacity Only with New Production Well(s)"
  - Alternative C: "Blended System with Addition of Lake Based Connection to Existing Wells"

- "Blended System with Addition of Lake Based Connection to Existing Wells" conditionally proceeded to detailed evaluation in case the well supply is proven to be insufficient to service the forecasted community growth, due to either quality reasons (water quality unable to meet required standards) or quantity (insufficient well capacity available from aquifer).
- The detailed evaluation of the three alternative water servicing solutions favored **Alternative A2: "Increase Capacity of Existing Well #2 in Combination with New Production Well @ Site H"** to be the recommended servicing solution due to the following considerations:
  - *Technical* – Alternative A1 and A2 scored similarly high due to their aim to maximize the capacity of existing Well #2. Although, they do not provide the same degree of redundancy as a blended (lake & well) supply system, the proposed wells in Alternatives A1 and A2 would still be able to reliably meet the maximum day demands with one well out of service. Alternative A2 is considered better than Alternative A1 in terms of maximizing use of existing infrastructure, adaptability to existing infrastructure and having minimal additional O&M resource requirements since installing the new well at the same site as existing Well #5 provides some minor advantages.
  - *Environmental* – Similar to Alternative B, there are no significant risks expected to aquatic and terrestrial vegetation and wildlife under Alternative A1 or A2. Some impact is expected to groundwater resources in comparison to having a lake-based system, however, groundwater production is within acceptable limits to ensure no significant risk to existing resources.
  - *Socio-Economic* – Under socio-economic category, Alternative A2 scores marginally better than the other alternatives. Like most construction, short-term impacts/nuisance to the community are expected due to increased traffic, noise and dust to adjacent areas. Site F and Site H are both near residential areas and Site F is adjacent to Highway 27, leading to some short-term impact. New well sites can be designed to mitigate long-term impacts to the community (e.g. visual and operating impacts), but Alternative A2 has the advantage of being confined to existing well sites. Based on the Stage 1 Archaeological Assessment, risk is low at each site, but Site F would require a Stage 2 AA, which is not required at Site H.
  - *Financial* – Alternatives A1 and A2 were found to be similarly low-cost alternatives in terms of the overall lifecycle cost, despite higher initial capital and land acquisition costs at Site F, and slightly lower O&M costs at Site H. Alternative B is moderate in cost and Alternative C is the highest cost overall. Alternative A2 is overall the lowest cost alternative.
  - *Jurisdictional* – All alternatives have the ability to accommodate potential future changes in drinking water quality requirements, except the Do Nothing Alternative. However, for permits and approval, due to the new transmission watermain crossing the Greenbelt Plan's "Protected Countryside", it would be far more challenging to acquire approval for construction of Alternative C than Alternatives A1, A2 or B. Alternatives A1 and B would require land acquisition which would not be required for Alternative A2. So, Alternative A2 is favoured.
- An ongoing groundwater exploration study is being undertaken in order to confirm that future well supply can meet the quantity and quality required to service the community of Nobleton. After analysis of the 6" well testing results at both Site F and Site H, it can be concluded that both Site F and H are expected to achieve the target pumping rate of >34 L/s.

- Per the above Storage section, if Alternative B: “Supplement Increased Supply to Offset Storage Deficit” is the recommended water storage solution, then the combined PTTW and supply capacity in Nobleton must be increased, to exceed the forecasted maximum day demand by 2 L/s (>91.5L/s). This would mean while keeping Wells #3 and #5 at 28.9 L/s each, and under the recommended water supply solution Well #2 and the new production well would each need a supply capacity of at least 33.7L/s. Both Well #2 and the new well would be capable of meeting this small increase in supply capacity.

Alternative Wastewater Servicing Solutions

- Out of eight alternative wastewater servicing solutions six were screened out during the screening process. The following short list of alternative solutions proceeded into detailed evaluation:
  - Alternative A: “Expand and Upgrade the Existing Janet Avenue Pumping Station and Nobleton WRRF”
  - Alternative B: “Construct a New Water Resource Recovery Facility (WRRF)”
- The detailed evaluation of the two alternative wastewater servicing solutions favored **Alternative A: “Expand and Upgrade the Existing Janet Avenue Pumping Station and Nobleton WRRF”** to be the recommended servicing solution under these considerations:
  - *Technical* – Alternative A ranked highest overall due to its ability to maximize the use of existing infrastructure and limit additional operations and maintenance resource requirements.
  - *Environmental* – Alternative A ranked highest overall as impacts are limited to upgraded and expanded existing sites and infrastructure, mitigating impacts to aquatic/terrestrial vegetation and wildlife, as well as greenhouse gas emissions.
  - *Socio-Economic* – Alternative A ranked highest overall as impacts are limited to upgraded and expanded existing sites and infrastructure. This mitigates short-term construction impacts, such as noise, dust and increased construction traffic, and minimizes potential impacts to archeological sites and cultural/heritage features. No significant long-term community impacts are expected, although there would be some increased sludge truck haulage from the upgraded and expanded WRRF, impacting local traffic.
  - *Financial* – Alternative A was found to be the lowest cost alternative in terms of capital, lifecycle and land acquisition costs.
  - *Jurisdictional* – Alternative A ranked highest as it requires limited land acquisition and fewer permits/approvals, while being able to adapt to potential future changes in final effluent requirements.

6 Bibliography -

Black & Veatch. (January 2019). *Class Environmental Assessment for Water and Wastewater Servicing in the Community of Nobleton – Study 1B: Wastewater System Capacity Optimization Study.* Regional Municipality of York

Black & Veatch. (June 2019). *Class Environmental Assessment for Water and Wastewater Servicing in the Community of Nobleton – Study 1A: Water System Capacity Optimization Study.* Regional Municipality of York

Black & Veatch. (June 2019). *Phase 1: Identify the Problem or Opportunity. Technical Memo #1.* Regional Municipality of York

Cheung, R. (April 30, 2020). *Technical Memorandum: Nobleton PW2 Pumping Test Conducted on March 27, 2020.* Water Resources Monitoring, EPP, York Region.

Hutchinson. (May 2019). *Humber River Assimilative Capacity Study 2017-2018 (DRAFT).* Nobleton: Regional Municipality of York

TSH. (February 2007). *Nobleton Sewage Works Design Report (DRAFT).* Nobleton: Regional Municipality of York, Slokker Canada Corporation

York Region. (2016). *The Regional Municipality of York Water and Wastewater Master Plan*

York Region. (2019). *The Regional Municipality of York. Characterization and Comprehensive Review of Groundwater Supply Resources of the Regional Municipality of York (York Region Groundwater Supply Options Study). In Support of the 2021 Water and Wastewater Master Plan Update*

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PHASE 3: ALTERNATIVE DESIGN CONCEPTS

Technical Memo No. 3

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

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Unverified Assumptions (if applicable)

No.	Assumption	Verified By	Date
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ADD	Average Day Demand
ADF	Average Day Flow (Annual)
BAF	Biological Aerated filter
BOD_5	Biochemical Oxygen Demand
CT	Baffling Factor x Contact Time (min) x Concentration (mg/L)
DWWP	Drinking Water Works Permit
EA	Environmental Assessment
ECA	Environmental Compliance Approval
GHG	Greenhouse Gas
GWTS	Groundwater Treatment Strategy
hp	Horsepower
IFAS	Integrated Fixed-Film Activated Sludge
kPa	Kilopascal
kg/d	Kilogram per Day
kg/h	Kilogram per Hour
km	Kilometer
kW	Kilowatt
L/min	Litres per Minute
L/s	Litres per Second
m	Meter
MABR	Membrane Aerated Bioreactor
MBR	Membrane Bioreactor
MECP	Ministry of Environment, Conservation and Parks
m^3/d	Cubic Meters per Day
MDD	Maximum Day Demand
MDWL	Municipal Drinking Water Licence
ML	Million Litres
MLD	Million Litres per Day
mL/min	Millilitres per Minute
MLSS	Mixed Liquor Suspended Solids
mm	Millimeter
MOE	Ministry of Environment
m/s	Meters per Second

O&M	Operations and Maintenance
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
PW2	Production Well No. 2
RAS	Return Activated Sludge
RCC	Reinforced Concrete
RDII	Rainfall Derived Infiltration and Inflow
SPS	Sewage Pumping Station
TDH	Total Dynamic Head
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TP	Total Phosphorous
TSS	Total Suspended Solids
VFD	Variable Frequency Drive
WAS	Waste Activated Sludge
WPCP	Water Pollution Control Plant
WHPA	Wellhead Protection Area
WRRF	Water Resource Recovery Facility
WWF	Wet Weather Flow

1.0 Introduction

Nobleton is a community in King Township in York Region. Currently, Nobleton is serviced by stand-alone water and wastewater systems to meet the needs of the current population. The York Region Water and Wastewater Master Plan (2016) indicated that the water and wastewater systems would require increased capacity to meet the requirements to support growth to the 2041 Master Plan population of 9,500. Therefore, the Master Plan recommended a Schedule C Class Environmental Assessment (EA) to identify servicing solutions to accommodate growth.

Taking into consideration the available land and the allowable population density, the Nobleton Community Plan and the King Township Draft Official Plan estimated a future population of 10,800.within the Nobleton urban boundary. Therefore, to support additional water and wastewater demand, it was determined that the water servicing facilities would need to supply an average day demand of 42.6 litres per second (L/s) and maximum day demand (MDD) of 89.5 L/s; the wastewater facility would need to an support average daily flow (ADF) of 3,996 cubic meters per day (m^3/d) and peak instantaneous flow (PIF) of 25,174 m^3/d.

1.1 Objective of Technical Memorandum

A previous Technical Memorandum 2 (TM2) identified, screened, and evaluated water and wastewater alternative solutions to service the increased population of 10,800. According to the evaluation, the following preferred solutions for water and wastewater servicing were identified and documented:

- Supplement increased water supply to offset storage deficit, and increase capacity of existing Well No. 2 in combination with new production well at Site H; and
- Expand and upgrade the existing Janet Avenue Pumping Station (PS) and Nobleton Water Resource Recovery Facility (WRRF).

The purpose of TM3 is to screen and evaluate alternative design concepts for implementing the preferred water and wastewater servicing solutions identified in Phase 2 of the EA planning process and to recommend the preferred water and wastewater design concepts.

1.2 Summary of Work Previously Completed

1.2.1 Municipal Class Environmental Assessment and Current Status

A flow chart of the EA process is shown on Figure 1-1. The study is currently in Phase 3 of the Class EA process.

Water and wastewater servicing opportunities and problems were identified in Phase 1. Preferred water and wastewater servicing solutions were identified in Phase 2. The current step, Phase 3, is to identify, screen, and evaluate recommended design concepts for the preferred servicing solutions and recommend preferred design concepts.

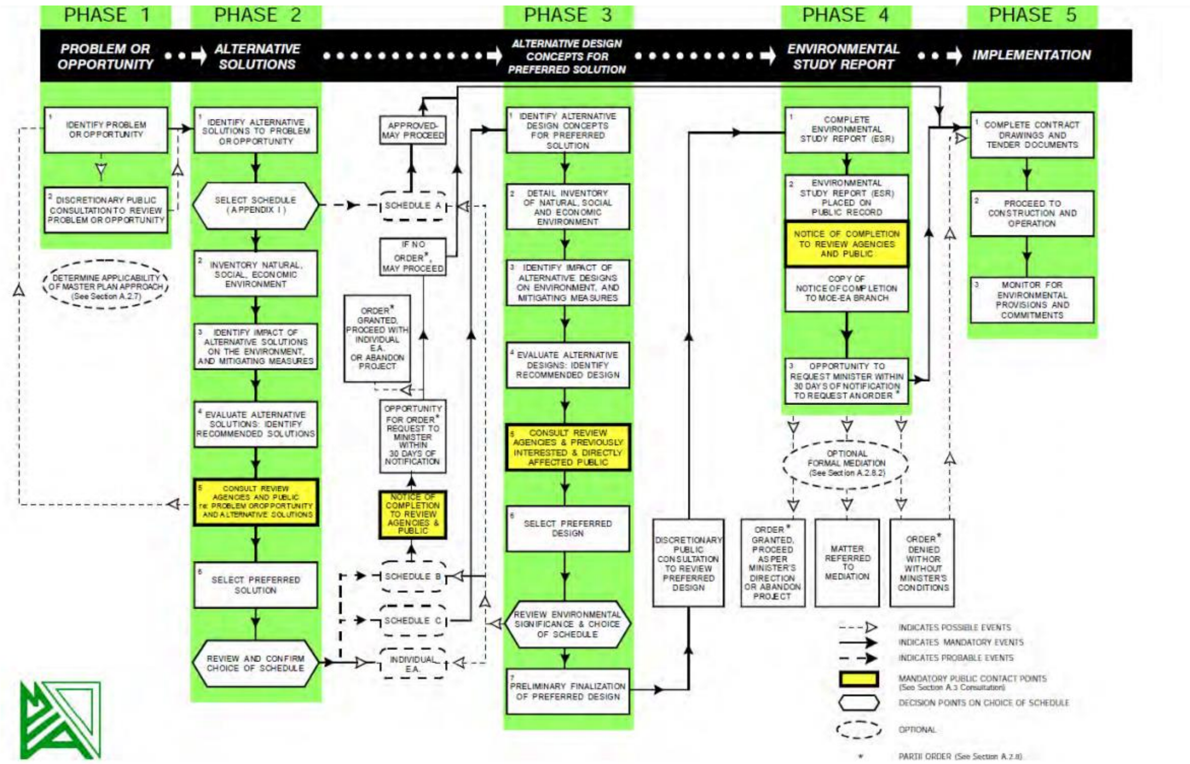

Figure 1-1 Municipal Class Environmental Assessment Process Flow Chart

1.2.2 Phase 1 and Phase 2 Work Completed

Phases 1 and 2 of the current Class EA study are complete. A brief description of the work performed during these phases is provided in the following subsections.

1.2.2.1 Phase 1

Black & Veatch submitted Technical Memorandum 1 (TM1): *Phase 1: Identify the Problem or Opportunity,* dated June 4, 2019. TM1 identified an opportunity to develop long-term water and wastewater servicing solutions to support the current and forecasted population growth in the community of Nobleton to 10,800 persons. Various water and wastewater studies were conducted to provide the supporting evidence for TM1. The previous studies completed for the water and wastewater systems are summarized on Figure 1-2 and Figure 1-3, respectively.

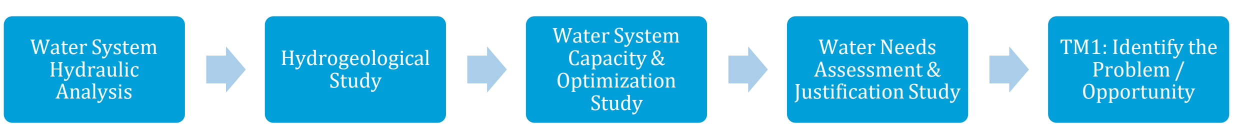

Figure 1-2 Summary of Previous Water System Studies

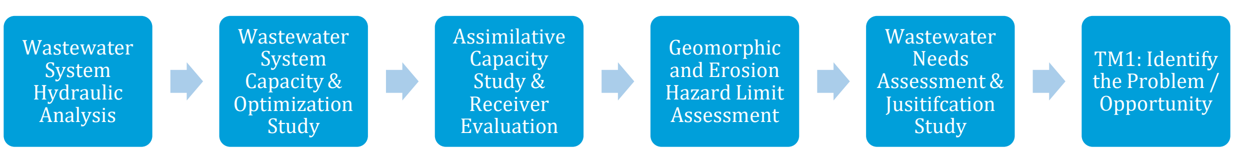

Figure 1-3 Summary of Previous Wastewater System Studies

The problem/opportunity statement developed in Phase 1 is as follows:

"Identify innovative, safe, and reliable water and wastewater servicing solutions for the community of Nobleton in King Township, to support approved population growth to 10,800 persons, while optimizing the use of existing systems. The preferred solution must be socially, environmentally and financially sustainable."

1.2.2.2 Phase 2

Black & Veatch submitted TM2: *Phase 2: Identify Alternative Solutions,* dated March 5, 2021. Water and wastewater servicing solutions were identified and evaluated according to the methodology shown on Figure 1-4.

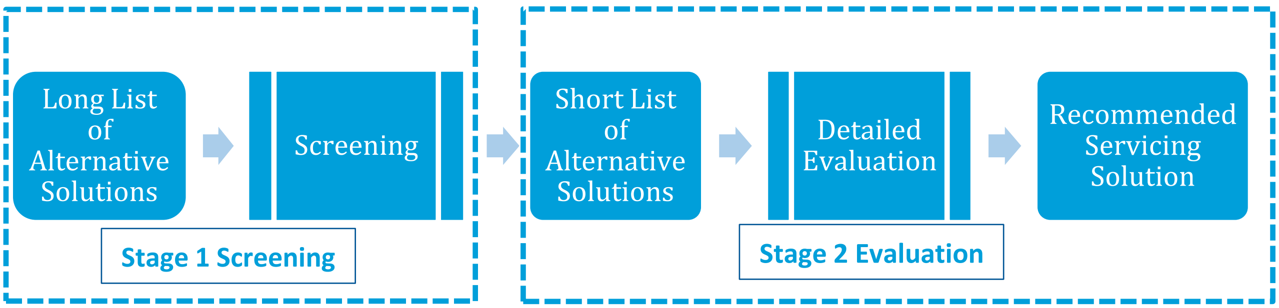

Figure 1-4 Phase 2 Screen and Evaluation Methodology

1.2.3 Water Servicing Solution

1.2.3.1 Water System Future Capacity Needs Summary

Black & Veatch conducted a detailed water system capacity assessment in Study 1A: Water System Capacity Optimization Study. Table 1-1 summarizes the existing water system capacity and the forecasted future water system demands.

Table 1-1 Summary of Existing Limits and Future Demand for the Nobleton Water System

Existing Water System	Current Capacity and Future Demand (litres per second)
Well No. 2 Capacity	22.7
Well No. 3 Capacity	28.9
Well No. 5 Capacity	28.9
Well Supply Firm Capacity (Permit to Take Water: Largest Unit Out of Service)	51.6
Water Storage Capacity (existing storage volume converted to the equivalent MDD that it can currently service)	87.40
Forecasted Future Average Day Demand	42.6
Forecasted Future Maximum Day Demand	89.5

Table 1-1 demonstrates that the combined capacity of the three existing Nobleton wells (No. 2, No. 3, and No. 5) would be 80.5 L/s. However, the current Permit to Take Water (PTTW) for the Nobleton wells not only limits the individual wells to stay within their individual capacities, but it also limits the combined capacity of the three wells. This combined PTTW capacity is equivalent to the firm capacity of the Nobleton wells. Firm capacity is the sum of the well capacities, except with the largest unit out of service. In this case, that would mean that Well No. 3 or Well No. 5 is assumed to be out of service (or on standby), so the current combined daily limit is only 51.6 L/s.

The PTTW limit and the firm capacity of the existing Nobleton wells is well below the forecasted MDD of 89.5 L/s. Therefore, additional water supply is required to meet the forecasted growth. To address this need, various water supply alternatives are developed and evaluated in this TM.

In terms of storage capacity, the existing Nobleton system has storage volume capable of providing storage requirements (fire, equalization, and emergency storage) up to the equivalent of an MDD of 87.40 L/s. The projected MDD is 89.5 L/s, which means there would ultimately be a marginal storage deficit if no action were taken. In terms of storage volume, this is equivalent to a storage need of 3.916 million litres (ML) compared to an existing capacity of 3.860 ML (marginal deficit of 0.06 ML). To address this need, various storage alternatives were developed and are evaluated in this TM.

1.2.3.2 Recommended Water Servicing Solution

The Nobleton water supply system currently consists of three groundwater wells with a combined firm capacity of 51.6 L/s. As previously summarized in TM2, a review of historical well performance, available drawdown, and screen transmitting capacity indicated that Nobleton Production Well No. 2 (PW2) has a theoretical design capacity of 67 L/s. Short-term testing has been completed that assessed the well yield at 32 L/s. From the previous work summarized in TM2, Alternative A was selected for further evaluation.

Alternative A would involve a capacity increase to the existing PW2 and its associated treatment facility. Using the information from the Operation Manual, it was determined that while maintaining sequestration for iron and manganese treatment, the capacity of Well No. 2 could be increased up to at least 32 L/s without any major upgrades to the existing treatment facility.

Results of a short-term pumping test conducted at Nobleton PW2 on March 27, 2020, indicated that there is sufficient drawdown to sustain a rate of 34 L/s for at least 60 minutes. It was recommended that a longer pumping test (48 hours to 72 hours in duration) be conducted on Nobleton PW2 to confirm the well's and aquifer's abilities to sustain the target rate over the long term and establish the corresponding zone of influence (refer to Technical Memorandum: Nobleton PW2 Pumping Test Conducted on March 27, 2020). It was also recommended to assess the impact of well interference caused by the operation of Well No. 3, Well No. 5, and the new well.

At Nobleton PW2, the capacity of the sodium silicate tank and chlorine contact tank was confirmed to ensure that these tanks could operate at a flow of at least 34 L/s (without requiring major work/expansions at the well facility). With the existing treatment processes, the increased flow rates would lead to an increase in the chemical feed rates required to meet the target dosages reflected in the original design and current operations practice. Initial review of the existing treatment process equipment indicates that the in-place treatment process can treat the additional capacity with moderate increases to the amount of chemical feed. Assessment of existing PW2 facilities indicated that additional facilities or treatment process capacity is not needed; therefore, no change to the current site footprint is expected.

In addition to an expansion at PW2, one new production well with its associated treatment facility would be required. This treatment facility is assumed to continue with the treatment processes used at the existing Nobleton wells (sequestration). Currently, it is assumed that the new well would have an instantaneous permitted capacity of 32 L/s, and the expanded PW2 would increase its instantaneous permitted capacity to 32 L/s . Combined, the overall well production capacities would meet the projected MDD of 89.5 L/s, as presented in Table 1-2, plus the surplus supply capacity that would be required to offset the minor storage deficit.

The preferred well site established during Phase 2 is Site H. Site H is located at the existing site of Nobleton Well No. 5. Further details on the well exploration sites can be found in the Nobleton Groundwater Drilling Site Selection Report.

Table 1-2 Water Alternative A Conceptual Breakdown of Current and Future Well Capacity

Category	Capacity Limit	Conceptual Future Capacity
Well No. 2 Capacity	22.7 L/s	**~ 32 L/s (expansion)**
Well No. 3 Capacity	28.9 L/s	28.9 L/s
Well No. 5 Capacity	28.9 L/s	28.9 L/s
New Production Well	**-**	**~ 32 L/s (new)**
Well Supply Firm Capacity (Largest well out of service)	51.6 L/s	89.8 L/s
Total Capacity	80.5 L/s	121.8 L/s

Region of York is considering other upgrades to PW2 and PW5 well sites as part of a Groundwater Treatment Strategy (GWTS), including provision of standby power at PW2 and upgrading to an iron and manganese oxidation/filtration system at both sites. These improvements are provided for reference but are not related to this evaluation.

1.2.4 Wastewater Servicing Solution

1.2.4.1 Wastewater System Future Capacity Needs Summary

Black & Veatch conducted a detailed wastewater system capacity assessment in Study 1B: Wastewater System Capacity Optimization Study. According to this assessment, the existing Nobleton wastewater collection system and WRRF experience relatively high peak flows (wet weather flow). Because of the high flow peaking factors (from 2014 to 2017, average peaking factor for the system was at 6.3), the equivalent ADF capacity is less than the Environmental Compliance Approval (ECA) rated capacity of 2,925 m^3/d.

The Janet Avenue PS has an equivalent ADF capacity of 1,430 m^3/d, an equivalent serviceable population of 3,865 persons, and a peak instantaneous flow capacity of 9,177 m^3/d

The King Street forcemain is a 300 millimeter (mm) diameter polyvinyl chloride (PVC) DR18 and DR26 pipe. Ministry of Environment (MOE) Design Guidelines for Sewage Works (2008), Section 7.9.1, provides guidance on the range of velocities in a sanitary forcemain. This range is from a minimum of 0.6 meters per second (m/s) to achieve scouring, to a maximum of 3.0 m/s. For the King Street forcemain, a velocity of 2.0 m/s is considered adequate to achieve reasonable velocities and headloss. Assuming a velocity of 2.0 m/s, the equivalent capacity of the King Street forcemain is 12,500 m^3/d (145 L/s).

The Nobleton WRRF capacity is limited by screening and grit removal to an equivalent ADF capacity of 1,457 m^3/d and peak hourly flow (PHF) capacity of 9,177 m^3/d. This capacity is equivalent to a serviceable population of 3,938 persons.

The existing effluent outfall is a 450 mm diameter reinforced concrete (RCC) pipe with slope varying from 0.35 percent to 5 percent. The carrying capacity varies from a minimum of 12,528 m^3/d (145 L/s) to 38,707 m^3/d (448 L/s), running 70 percent full.

Therefore, there is a need to provide additional wastewater service capacity at some or all of the existing wastewater infrastructure to support future ADF and PIF requirements of 3,996 m^3/day and 25,174 m^3/day, respectively. Refer to Table 1-3.

Table 1-3 Summary of Existing Capacity of the Nobleton Wastewater System

Category	Janet Avenue Pumping Station	Nobleton Water Resource Recovery Facility
Existing Capacity	9,177 m^3/d (PIF)(2)	1,457 m^3/d (ADF)
Future Flow Requirements	25,174 m^3/d (PIF) (2)	3,996 m^3/d (ADF)

1.2.4.2 Recommended Wastewater Servicing Solution

The detailed evaluation of the short-listed alternative wastewater servicing solutions in TM2 favored Alternative A: “Expand and Upgrade the Existing Janet Avenue Pumping Station, Forcemain and Nobleton WRRF and Outfall” over Alternative B: “Construct a New Pumping Station, Forcemain and WRRF and Outfall.” Alternative A ranked higher in terms of technical, environmental, socioeconomic, financial, and jurisdictional evaluation criteria and was selected as the recommended servicing solution.

The existing wastewater collection and treatment system would be upgraded and expanded as follows (Figure 1-5):

- Collection System – The existing sanitary sewer system has sufficient capacity to accommodate design peak flows as established during TM2 based on hydraulic modelling; therefore, no expansion would be required.
- Janet Avenue Pumping Station – The existing Janet Avenue PS has a peak capacity of 9,177 m^3/d as identified in Study 1B. In order to accommodate the future peak flows, the Janet Avenue PS would need to be expanded by either replacing the existing pumps with larger units or providing additional pumps.
- Forcemain – The existing forcemain from the Janet Avenue PS to the Nobleton WRRF would need to be expanded to accommodate the future peak flows through either replacement with a larger pipe or addition of a second forcemain.
- Nobleton WRRF – The existing Nobleton WRRF would need to be expanded and upgraded to meet the future ADF, peak flows, and effluent quality requirements identified in Study 1B. The expansion approach could include constructing additional treatment trains (from inlet works to disinfection) or intensifying the existing treatment trains or a combination of both.

The detailed expansion and upgrade approach would be discussed in detail in Phase 3 should Alternative A be selected as the recommended alternative.

- Effluent Discharge and Outfall – The bottleneck for existing effluent discharge is within the effluent chamber and its inlet arrangement rather than with the outfall itself. Future peak flows greater than the existing rated peak flow of 9,177 m^3/d would need to be discharged into the existing outfall at MH 113 to prevent flooding in the existing facility.

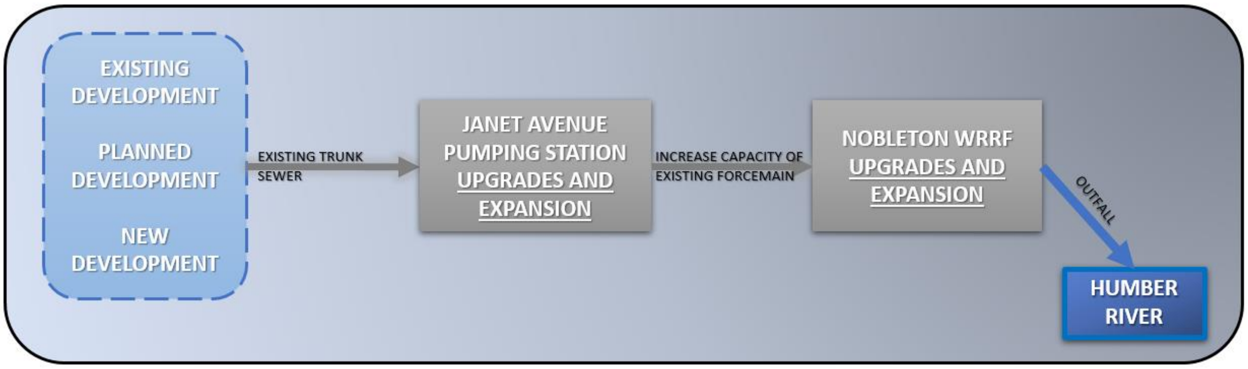

Figure 1-5 Phase 2 Preferred Wastewater Servicing Solution Alternative A: Expand and Upgrade the Existing Janet Avenue Pumping Station and Nobleton WRRF

2.0 Screening and Evaluation Methodology

The Nobleton Water and Wastewater Schedule C Class EA developed, refined, and evaluated various potential servicing strategies (for both the water and wastewater systems) to address the problem statement using a two-stage process.

A two-stage process was selected to evaluate alternatives because it provides a clear and simple way to identify which alternatives are technically feasible and that meet the current regulations. Subsequently, with a short list of feasible alternatives, a detailed comparison can be conducted, using evaluation criteria that are based on the Municipal Engineers Association Class Environmental Assessment process requirements.

The decision-making process is based on a two-stage methodology (Figure 2-1):

- **Stage 1: Screening of Long List of Alternative Design Concepts** – Only reasonable and feasible alternative design concepts are to be considered as part of the Municipal Class EA process. This stage will determine the feasibility of an alternative design concept by comparing it with a set of "pass/fail" screening criteria. The screening criteria will be used to screen out solutions from the long list of alternative design concepts to create a short list of design concepts for further consideration in Stage 2.
- **Stage 2: Evaluation of Short List of Alternative Design Concepts** – The short list of alternative design concepts from Stage 1 are subjected to detailed evaluation and assessed against the evaluation criteria. The evaluation criteria reflect various factors that have been established to be of most importance to the project. For evaluation, each evaluation criterion will be assigned a performance rating which will be used to comparatively evaluate the short list of alternative design concepts. Alternative design concepts will be rated according to how well they perform in addressing the specified criterion. Overall performance of each design concept will be determined based on the combination of individual criterion performance rating. The evaluation uses the "Traffic Light Assessment" method, where each design concept is scored as green, yellow, or red for each criterion. This method was selected since it is highly intuitive to the general public and also provides sufficient detail to differentiate between the various alternatives.

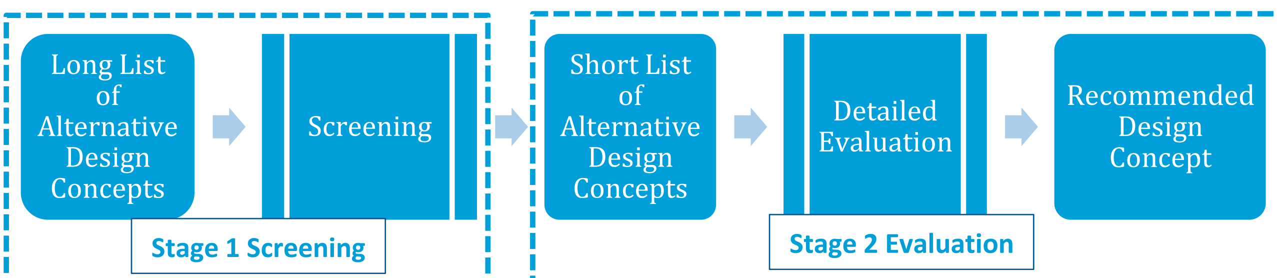

Figure 2-1 Screening and Evaluation Methodology

2.1 Screening Criteria

There are six screening criteria for the design concepts proposed in Phase 3 of this Class EA. The screening criteria are summarized in Table 2-1.

Table 2-1 Screening Criteria for Design Concepts

Criteria	Description
Compatibility with Existing Servicing Infrastructure	The alternative must be able to be integrated with the existing Janet Avenue PS, wastewater collection system, forcemain, Nobleton WRRF, and Wells No. 2 and 5. This would include compatibility in terms of hydraulics, available space, and operations.
Proven Technology	The design concept or technology must be in operation in a full-scale plant in North America (specifically in areas with colder climates). The technology must have been in operation for a minimum of 5 years.
Performance Robustness and Reliability	The design concept or technology must be able to achieve robustness and reliability of performance to meet the project objectives, water quality, effluent requirements, and performance requirements.
Stakeholder Acceptance	Potential impacts from the alternative must be able to be mitigated to an acceptable level to satisfy local and regulatory stakeholders.
Acceptable Construction Impacts	The construction impacts to the natural environment and the adjacent landowners/users must be able to be mitigated to an acceptable level.
Cost	Costs must be acceptable, as evaluated based on high-level assumptions of capital and operating costs of each design concept.

2.2 Evaluation Methodology

The evaluation methodology was developed giving due consideration to York Region's Consultant Manual. These considerations related to the impact of the alternative design concepts on the natural social and economic environment, development of evaluation criteria to carry out comparative evaluation of design concepts, and development of methodology to carry out comparative evaluation.

The list of detailed evaluation criteria and performance ratings are provided in Table 2-2.

Table 2-2 Description of Evaluation Criteria for Short List Design Concepts

Criteria		Description/Considerations	Performance Rating
TECHNICAL			
A.	Constructability	• What are the major construction challenges and risks (crossing environmentally sensitive areas, noise, odour, dust, public safety, traffic, etc.) associated with the alternative? To what extent does it impact the community? • How much volume and complexity of construction will be associated with the alternative?	🟢 Low Impact *(Low Construction Impact/Complexity)* 🟡 Moderate Impact *(Moderate Construction Impact/Complexity)* 🔴 High Impact *(Higher Construction Impact/Complexity)*
B.	Redundancy of Supply/Service	• Will the alternative be able to provide improvements in redundancy of supply or service? • If there is an unexpected event (e.g., power outage, spill, equipment failure) does that impact supply or service?	🟢 High Redundancy 🟡 Moderate Redundancy 🔴 Low Redundancy
C.	Resilience to Climate Change	• Is the alternative resilient against changing climate conditions, such as: ○ Changes to water supply quantity and quality (e.g., due to drought) ○ Increase of intensity and frequency of wet weather flow events	🟢 High Resilience 🟡 Moderate Resilience 🔴 Low Resilience
D.	Operations and Maintenance (O&M) Requirements	• What will be the level of additional and new O&M resources (e.g., human resources) required for the alternative? • What will be the level of complexity and maintainability of new and optimized assets?	🟢 Low Complexity/ O&M Requirements 🟡 Moderate Complexity/ O&M Requirements 🔴 High Complexity/ O&M Requirements
E.	Adaptability to Existing Infrastructure	• What will be the level of modification required to the existing infrastructure to adapt to the alternative? What is the relative ease of connection to the existing infrastructure? • What is the level of interference or effects on other utilities (e.g., are relocations required)? • What is the compatibility of the design concept with the existing infrastructure? This would include compatibility in terms of hydraulics, available space/footprint, and operations.	🟢 High Adaptability 🟡 Moderate Adaptability 🔴 Low Adaptability
F.	Maximizing Use of Existing Infrastructure	• Will the alternative be able to maximize the capacity of the existing infrastructure to reduce new asset needs?	🟢 High Degree (*Efficient use of Existing Infrastructure)* 🟡 Moderate Degree (*Partial use of Existing Infrastructure)* 🔴 Low Degree (*Inefficient use of Existing Infrastructure)*
NATURAL ENVIRONMENT			
G.	Aquatic Vegetation and Wildlife	• Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Streams and rivers. ○ Local aquatic species and habitats. ○ Environmentally sensitive areas, aquatic species at risk, or locally significant aquatic species.	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
H.	Terrestrial Vegetation and Wildlife	• Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Trees and vegetation. ○ Local terrestrial species and habitats. ○ Environmentally sensitive areas, species at risk, and locally significant species.	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
I.	Groundwater Resources	• Will the alternative have significant impacts during construction and/or from ongoing operations on aquifers and groundwater resources such as: groundwater quantity, groundwater recharge quality and flow regime and groundwater discharge to streams and wetlands?	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
J.	Surface Water Resources	• Will the alternative have significant impacts during construction and/or from ongoing operations on adjacent surface water resources (e.g., Humber River) and related biological communities?	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact
K.	Greenhouse Gas Emissions (GHG)	• What will be the level of impact of GHG emissions associated with the alternative? *(*GHG emission will be evaluated according to the alternative's energy intensity requirements.)	🟢 Low Impact 🟡 Moderate Impact 🔴 High Impact

Criteria		Description/Considerations	Performance Rating
SOCIOECONOMIC ENVIRONMENT			
L.	Short-term Community Impacts (impacts to community during construction)	• Will the alternative have significant short-term impacts to the community during construction, including: ○ Noise, dust, and odour. ○ Local traffic.	● Low Impact ● Moderate Impact ● High Impact
M.	Long-Term Community Impact	• Will the alternative have significant long-term impacts on the community, including: ○ Impact of operating facility including air quality, odour, and noise impacts. ○ Visual impact. ○ Public acceptance/resistance (Any potential resistance to the proposed servicing solution? [e.g., resistance to growth/resistance to well supply]).	● Low Impact ● Moderate Impact ● High Impact
N.	Archaeological Sites	• Will the alternative have significant impacts during construction and/or from ongoing operations on registered/known archaeological features?	● Low Impact ● Moderate Impact ● High Impact
O.	Cultural/Heritage Features	• Will the alternative have significant impacts during construction and/or from ongoing operations on known cultural landscapes and built heritage features?	● Low Impact ● Moderate Impact ● High Impact
FINANCIAL			
P.	Land Acquisition Cost	• What will be the relative land acquisition cost for the alternative?	● Low Cost Alternative ● Moderate Cost Alternative ● High Cost Alternative
Q.	Capital Cost	• What will be the relative capital cost for the alternative?	● Low Cost Alternative ● Moderate Cost Alternative ● High Cost Alternative
R.	Life-Cycle Cost	• What will be the relative life-cycle cost for the alternative?	● Low Cost Alternative ● Moderate Cost Alternative ● High Cost Alternative
JURISDICTIONAL/REGULATORY			
S.	Land Requirements	• What will be the relative area of non-regional land or easement required to construct the alternative?	● Low Requirement ● Moderate Requirement ● High Requirement
T.	Ability to Accommodate Potential Future Regulatory Changes	• Will the alternative have the ability to adapt to potential future changes in drinking water quality and final effluent requirements?	● High Adaptability ● Moderate Adaptability ● Low Adaptability
U.	Permits and Approval	• What will be the level of permits and approvals required to construct the alternative?	● Low Requirement ● Moderate Requirement ● High Requirement

3.0 Water System Alternative Design Concepts

3.1 Existing Water Servicing (Installed Infrastructure)

3.1.1 Well Site No. 2

3.1.1.1 Water Conveyance

3.1.1.1.1 Well Pump

The existing well pump is a vertical centrifugal turbine type with a capacity of 22.73 L/s at 83.8 m total dynamic head (TDH). The pump is equipped with a 29.8 kilowatt (kW) (40 horsepower [hp]) motor and a variable frequency drive (VFD).

3.1.1.1.2 Main Line Piping

The main line piping has a nominal diameter of 150 mm.

3.1.1.2 Chlorination System

Chlorine gas is fed for primary and secondary disinfection. The chlorination system consists of a chlorine gas storage and feed system, two V10 chlorinators, each rated at 22 kilograms per day (kg/d), a chlorine contact pipe, and chlorine gas scrubber.

3.1.1.2.1 Chlorine Gas Storage

Chlorine gas is stored in pressurized 68 kilogram (kg) gas cylinders. The existing system has a capacity for a total of 15 cylinders, including 6 full and 7 empty, equating to a total capacity of 408 kg of chlorine gas, plus two additional cylinders on weigh scale with vacuum regulators equipped with auto switchover. The maximum draw rate is 1.75 kilograms per hour (kg/h) to avoid freezing and poor dosing.

3.1.1.2.2 Chlorine Booster Pumps and Eductors

Two chlorine booster pumps (one duty and one standby) are located in the sodium silicate room. Each booster pump provides carrier water to an eductor. Each pump and eductor are rated at 18.32 liters per minute (L/min).

3.1.1.2.3 Chlorine Contact Pipe

The existing below ground chlorine contact pipe chamber is 13.0 meter (m) in length and 1,800 mm in diameter, providing a volume of 33.08 cubic meters (m^3).

3.1.1.3 Iron and Manganese Sequestration System

For sequestration of iron and manganese, a 37.5 percent sodium silicate is fed. The sequestration system consists of a storage tank and dosing pumps.

3.1.1.3.1 Sodium Silicate Storage Tank

Existing sodium silicate storage includes an underground storage tank with a capacity of 2,700 L.

3.1.1.3.2 Sodium Silicate Dosing Pumps

Two sodium silicate dosing pumps (one duty and one standby) are located in the sodium silicate room. These dosing pumps draw sodium silicate from the storage tank and inject it into the main line piping directly downstream of the chlorine injection point. Each pump has a capacity of 7.95 litres per hour.

3.1.1.4 Standby Power Generation

Standby power is required at critical well facilities to maintain water supply during a sustained outage. The site includes a portable generator set connection for powering the facility in the event of a power failure, and Region of York intends to provide a permanent standby power in the future.

3.1.2 Well Site H

3.1.2.1 Water Conveyance

3.1.2.1.1 Well Pump

The existing Well Pump No. 5 is a submersible vertical turbine type with a capacity of 28.9 L/s at 102 m TDH. The pump is equipped with a 44.7 kW (60 hp) motor and a VFD.

3.1.2.1.2 Main Line Piping

Main line piping has a nominal diameter of 150 mm.

3.1.2.2 Chlorination System

Chlorine gas is fed for primary and secondary disinfection. The chlorination system consists of a chlorine gas storage and feed system, two V10 chlorinators at 22.7 kg/d each, a chlorine contact pipe, and chlorine gas scrubber.

3.1.2.2.1 Chlorine Gas Storage

Chlorine gas is stored in pressurized 68 kg gas cylinders. The existing system has a capacity for a total of 24 cylinders including 12 empty and 12 full, equating to a total capacity of 816 kg of chlorine gas.

3.1.2.2.2 Chlorinators

Chlorine gas is fed by two Siemens V10K vacuum chlorinators rated at 22.7 kg/d each. Injectors are 1 inch with a capacity of 10 kg/h.

3.1.2.2.3 Chlorine Booster Pumps and Eductors

The two chlorine booster pumps (one duty and one standby) are located in the process room. Each booster pump provides carrier water to an eductor. Each pump and eductor is rated at 36.67 L/min.

3.1.2.2.4 Chlorine Contact Pipe

The existing chlorine contact system is a combination of a concrete pressure pipe 14.5 meters in length and an 1,829 mm inner diameter, providing a volume of 30.31 m^3, and a PVC DR18 contact pipe 53 meters in length with a 296 mm inner diameter, providing a volume of 38.09 m^3. The total chlorine contact pipe volume is 68.4 m^3.

3.1.2.3 Iron and Manganese Sequestration System

For sequestration of iron and manganese, a 37.5 percent sodium silicate is fed. The sequestration system consists of a storage tank and dosing pumps.

3.1.2.3.1 Sodium Silicate Storage Tank

Existing sodium silicate storage includes an in-ground storage tank with a capacity of 2,600 L.

3.1.2.3.2 Sodium Silicate Dosing Pumps

Two sodium silicate dosing pumps (one duty and one standby) are located in the chemical room. The metering pumps draw sodium silicate from the storage tank and pump it into the water via two injection points. The metering pump capacities are 1098 millilitres per minute (mL/min) for each pump.

3.2 Design Basis

3.2.1 Well Site No. 2

3.2.1.1 Water Conveyance

3.2.1.1.1 Well Pump

The existing well pump will require modification or replacement to increase the capacity of Well Site No. 2 from the existing capacity of 22.73 L/s to the required capacity of 32 L/s.

3.2.1.1.2 Main Line Piping

The existing main line piping is of a sufficient size (150 mm) to accommodate the marginal increase from the design capacity of 22.73 L/s to 32 L/s.

3.2.1.2 Chlorination System

3.2.1.2.1 Chlorine Gas Storage

The existing chlorine gas storage (six full cylinders totaling 408 kg) has sufficient capacity to accommodate the proposed capacity expansion for Well No. 2 to 32 L/s. No expansion of the storage system will be required.

3.2.1.2.2 Chlorine Booster Pumps and Eductors

At the design flow rate of 32 L/s, baffling factor of 0.3, and 10° C water temperature, a free chlorine residual of 0.52 mg/L is required to achieve 2-log virus inactivation. Under the same conditions, but at 5° C water temperature, a free chlorine residual of 0.77 mg/L is required to achieve 2-log virus inactivation. The maximum chlorine feed rate of 1.75 kg/h equates to a dose of 15.2 mg/L at the design flow rate. Thus, the existing chlorine feed system has sufficient capacity to accommodate the proposed capacity increase for Well No. 2 to 32 L/s.

3.2.1.2.3 Chlorine Contact Pipe

The existing chlorine contact pipe is sufficiently sized to accommodate the proposed capacity increase for Well No. 2 to 32 L/s using the existing chlorination system. No additional contact time or contact volume will be required.

3.2.1.3 Iron and Manganese Sequestration System

3.2.1.3.1 Sodium Silicate Storage Tank

At the design flow of 32 L/s, the required feed rate of sodium silicate is 3.93 L/h at the design dose of 20 mg/L. The existing storage tank at this rate provides 25 days of storage. Thus, the existing sodium silicate storage tank has sufficient capacity to accommodate the proposed capacity expansion.

3.2.1.3.2 Sodium Silicate Dosing Pumps

The existing sodium silicate dosing pumps, each rated at 7.95 L/h, have sufficient capacity for the proposed expansion.

3.2.2 Well Site H

3.2.2.1 Water Conveyance

3.2.2.1.1 Well Pump

No changes are required to the existing Well Pump No. 5. A new well pump rated at a capacity of 32 L/s will be installed for Well Site H.

3.2.2.1.2 Main Line Piping

The existing main line piping size of 150 mm is not sufficient to accommodate the combined capacity of both Well Pump H and Well Pump No. 5.

3.2.2.2 Chlorination System

3.2.2.2.1 Chlorine Gas Storage

At a design chlorine dose of 8.5 mg/L, the existing chlorine gas storage system (twelve 68 kg cylinders) will provide 18 days of storage at a flow rate of 60.9 L/s. This meets the minimum storage requirement of 7 days. Thus, no expansion of the storage system will be required.

3.2.2.2.2 Chlorinators

At a design chlorine dose of 8.5 mg/L, the required gas feed rate at a flow rate of 60.9 L/s is 1.86 kg/h. The existing chlorinators have a maximum draw rate of 1.75 kg/h. Thus, additional capacity will be required.

3.2.2.2.3 Chlorine Booster Pumps and Eductors

Based on size of the chlorinators, the booster pumps and eductors appear to have sufficient capacity to accommodate the additional capacity from Well Pump H.

3.2.2.2.4 Chlorine Contact Pipe

Based on a baffle factor of 0.7 in the concrete pressure pipe and a baffle factor of 1.0 in the DR18 PVC pipe and water temperature of 5° C an additional 60 m^3 of volume is required in the chlorine contact pipe to meet CT (baffling factor x contact time [min] x concentration [mg/L]) requirements for 2-log virus inactivation at the minimum residual of 0.2 mg/L at the combined capacity of 60.9 L/s. Alternatively, if additional chlorine contact volume is not provided, a minimum chlorine residual of 0.48 mg/L is required to meet 4.0 mg-min/L CT required for 2-log virus inactivation under the same conditions.

3.2.2.3 Iron and Manganese Sequestration System

3.2.2.3.1 Sodium Silicate Storage Tank

At the design chlorine dose of 18 mg/L, the existing sodium silicate storage tank (2,600 L) will provide 14 days of storage at a flow rate of 60.9 L/s. This meets the minimum storage requirement of 7 days. Thus, no expansion of the storage system will be required.

3.2.2.3.2 Sodium Silicate Dosing Pumps

At a design dose of 18 mg/L and flow rate of 60.9 L/s, the required sodium silicate feed rate is 126 mL/min. The existing two dosing pumps are each rated for 132 mL/min. Thus, no additional capacity in the sodium silicate dosing pumps is required.

3.3 Screening and Evaluation of Design Concepts

3.3.1 Long List of Alternative Water Design Concepts

The long list of screening criteria for the design concepts proposed for Well Site No. 2 and Well Site H provides a broad overview of the viability of each alternative. Criteria include compatibility with existing servicing infrastructure, in terms of hydraulics, available space, operations, and integration into the existing wells, pumping station, and wastewater collection system. The proposed alternative(s) must also be a proven technology already in full-scale operation for a minimum of 5 years in North America. The performance robustness to achieve minimum treatment objectives/water quality of effluent requirements must be met with each alternative. Stakeholders for each alternative, either local or regulatory, must be able to accept the potential impacts of each alternative, including construction impacts to the natural environment and adjacent landowners.

3.3.1.1 Water Storage

Due to large infrastructure costs in providing marginal storage increase, it was established during TM2 that the preferred alternative was to provide additional pumping capacity in lieu of providing additional storage. This section explains the long list of concepts captured in the Well Site No. 2 and Well Site H design concepts.

3.3.2 Screening of Long List of Alternative Water Design Concepts

The long list of alternative water storage solutions was screened according to the screening criteria presented in Section 2.1. Each alternative's ability to meet the criteria is noted by the following symbols, "✓" for Pass and "✗" for Fail. The screening results are presented in Table 3-1 for Well Site No. 2 and Table 3-2 for Well Site H.

The following three alternative solutions, which were deemed feasible to support forecasted growth in the community of Nobleton, were carried forward for detailed evaluation:

- Expanding the existing capacity for Well Site No. 2.
- Expanding the existing treatment train capacity for Well Site No. 5.
- Add second treatment train from Well Site H.

3.3.2.1 Well Site No. 2

3.3.2.1.1 Expanding Existing Facility for Well Site No. 2

The capacity expansion of Well Site No. 2 to 34 L/s can be achieved using existing facility infrastructure and equipment, with the exception of increasing the capacity of the well pump. Thus, this alternative meets all six screening criteria, and no other alternative is required. The existing Well Site No. 2 facility has already met the compatibility, proven technology, and stakeholder criteria. With no construction or additional equipment needed, other than replacement of the well pump, there will be no construction impacts or capital costs to evaluate. After a review of the current operations manual of the facility, it was determined the existing chemical storage, educators, and chemical metering pumps would be able to handle the proposed capacity expansion.

3.3.2.2 Well Site H

3.3.2.2.1 Expanding Existing Treatment Train Capacity at Well Site No. 5

Expanding the existing treatment train capacity at Well Site No. 5 to include water from Well Site H passes all the long list of screening criteria. Major adjustments screened include increasing the capacity of the chlorination system, constructing and testing a new supply well, and implementing a system to deliver water from Well Site H to Well Site No. 5. Since the standing Well Site No. 5 facility will still be used in this alternative, capability, proven technology, and stakeholder acceptability are passable criteria.

3.3.2.2.2 Add Independent Dedicated Treatment Train from Well Site H

Adding an independent dedicated treatment train from Well Site H passes all criteria. The dedicated treatment train would be identical, or very similar to, the facility for Well Site No. 5, so it passes the compatibility, proven technology, and performance robustness. Well Site H and Site No. 5 can be found on the same land, so stakeholder and constructability impacts would be acceptable for this alternative.

3.3.3 Short List of Alternative Water Design Concepts

Taken from the results of the long list, the short list of evaluation criteria compares Well Site No. 2 with Well Site H alternatives in greater detail. Making comparisons helps narrow the selection of alternatives between well sites. The criteria for the short list can be found in Table 2-2. Key concepts include technical, natural environment, socioeconomic, financial, jurisdictional/regulatory, with more specific criteria in each. All criteria were rated between low, moderate, and high for all alternatives with justification.

3.3.4 Evaluation of Alternative Water Design Strategies

A detailed evaluation of the short-listed alternative water storage solutions was carried out in accordance with the evaluation methodology described in Section 2.2 and is presented in Table 3-2.

3.3.4.1 Well Site No. 2

3.3.4.1.1 Use Existing Infrastructure for Expansion for Well No. 2

Because this alternative requires no infrastructure or equipment changes, other than replacement of the well pump, most criteria are rated having low impact, low complexity, and high compatibility. There will be marginal increases in chemical consumption due to the increase in capacity, which would also increase operating costs. In addition, the permit to take water would require modification, and amendments to Drinking Water Works Permit (DWWP) and Municipal Drinking Water Licence (MDWL) would be required.

3.3.4.2 Well Site H

3.3.4.2.1 Expanding Existing Capacity of Well Site No. 5

The capacity expansion of Well Site No. 5 to include water from Well Site H passes all the long list of screening criteria. Major adjustments screened include increasing the capacity of the chlorine feed system and implementing a system to deliver water from Well Site H to Well Site No. 5. Since the standing Well Site No. 5 facility will still be used in this alternative, capability, proven technology, and stakeholder acceptability are passable criteria.

3.3.4.2.2 Add Independent Dedicated Treatment Train from Well Site H

Adding an independent dedicated treatment train from Well Site H causes the most ratings of high impact and high cost, as it would mean building a new facility on top of Well Site H. This results in low adaptability and a low degree of maximizing existing infrastructure. This alternative would also be the most construction-intensive option, with noise, vegetation, wildlife, and effects on local community needing to be considered.

Table 3-1 Screening of the Long List of Alternative Water Design Concepts for Well Site No. 2

Long List of Alternative Water Design Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
Expand existing capacity of Well Site No. 2	✓	✓	✓	✓	✓	✓	• Requires replacement of existing pump • No other structural or equipment changes/additions required

Table 3-2 Screening of the Long List of Alternative Water Design Concepts for Well Site H

Long List of Alternative Water Design Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
Expand existing capacity of Well Site No. 5	✓	✓	✓	✓	✓	✓	• Increase main line piping size • Increase chlorination system capacity • Connect raw water from Well Site H to Well Site No. 5
Add independent dedicated treatment train from Well Site H	✓	✓	✓	✓	✓	✓	• New facility similar to existing Well Site No. 5 • Connect finished water from Well Site H to finished water from Well Site No. 5

After the review of alternative solutions for water storage carried out during Phase 2 of the Class EA, only one alternative was carried forward. Therefore, only one alternative solution has been presented and evaluated (Table 3-3).

Table 3-3 Short Listed Alternative Water Storage Solutions - Detailed Evaluation

Evaluation Criteria	Alternative A: Use Existing Infrastructure for Expansion for Well No. 2
CONCEPTS	
A. CONSTRUCTABLITY • What are the major construction challenges and risks (e.g., crossing environmentally sensitive areas, noise, odour, dust, public safety, traffic, etc.) associated with the alternative? • To what extent does it impact the community? • How much volume and complexity of construction will be associated with the alternative?	**LOW IMPACT** • There will be no constructability challenges, complexity, and risks with this alternative since no new infrastructure would be installed. There will be no new impacts to the community beyond the existing impacts.
B. REDUNDANCY OF SUPPLY/SERVICE • Will the alternative be able to provide improvements in redundancy of supply or service?	**MODERATE REDUNDANCY** A higher capacity will cause a shorter supply of chemicals on hand. Thus, a higher redundancy of chemical delivery service would be required before the chemical reserves are depleted. However, the increase in chemical consumption is marginal.
C. RESILIENCE TO CLIMATE CHANGE • Will the alternative have the resilience against changing climate conditions, such as changes to water supply quantity and quality (e.g., high water demands, drought)?	**HIGH RESILIENCE** With no proposed changes to the existing system, there will be no changes to resilience against changing climate conditions.
D. O&M REQUIREMENTS • What will be the level of additional and new O&M resources (e.g., human resources) required for the alternative? • What will be the level of complexity and maintainability of new and optimized assets?	**LOW COMPLEXITY** There will be a low level of additional O&M resources required beyond the resources already required because no new assets or infrastructure for are needed for this alternative.
E. ADAPTABILITY TO EXISTING INFRASTRUCTURE • What will be the level of modification required to the existing infrastructure to adapt to the alternative? What is the relative ease of connection to the existing alternative?	**HIGH ADAPTABILITY** There will be no modification required to the existing infrastructure.

Evaluation Criteria	Alternative A: Use Existing Infrastructure for Expansion for Well No. 2
F. MAXIMIZING USE OF EXISTING INFRASTRUCTURE • Will the alternative be able to maximize the capacity of the existing infrastructure to reduce new assets needs?	**HIGH DEGREE** This alternative is strictly using the existing infrastructure; no new asset needs.
G. AQUATIC VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Streams and river. ○ Local aquatic species and habitat. ○ Environmentally sensitive areas, aquatic species at risk, and locally significant aquatic species.	**LOW IMPACT** There will be low impact on the aquatic vegetation and wildlife beyond the existing impact of the current system.
H. TERRESTRIAL VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Trees and vegetation. ○ Local terrestrial species and habitats. ○ Environmentally sensitive areas, species at risk, and locally significant species.	**LOW IMPACT** There will be low impact on the terrestrial vegetation and wildlife beyond the existing impact of the current system.
I. GROUNDWATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on aquifers and groundwater resources such as groundwater quantity, groundwater recharge quality and flow regime, and groundwater discharge to streams and wetlands?	**LOW IMPACT** The only impact this alterative would have is a minor increase of groundwater withdrawn from existing operation condition.
J. SURFACE WATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on adjacent surface water resources (e.g., Humber River) and related biological communities?	**LOW IMPACT** As Well No. 2 is a groundwater source, not influenced by surface water, there will be no impact to surface water resources.
K. GREENHOUSE GAS EMISSIONS • What will be the level of GHG emissions associated with the alternative? *(GHG emissions will be evaluation based on the alternative's energy intensity requirements.)*	**LOW IMPACT** There will be low increase of GHG emissions associated with the alternative. Increases could stem from greater frequency of chemical transportation and greater energy demand for the pumps.

Evaluation Criteria	Alternative A: Use Existing Infrastructure for Expansion for Well No. 2
L. SHORT-TERM COMMUNITY IMPACTS • Will the alternative have significant short-term impacts to the community during construction, including: ○ Noise, dust, and odour. ○ Local traffic.	**LOW IMPACT** There will be low level impacts short-term in the community. There would be a marginal increase in the frequency of delivery of chemicals. There would be no noise, dust, or odour impacts.
M. LONG-TERM COMMUNITY IMPACT • Will the alternative have significant long-term impact to the community, including: • Benefit to community. • Impacts from facility operations. • Visual impact. • Public acceptance/resistance.	**LOW IMPACT** This alternative would have minimal long-term impacts to the community since existing infrastructure is largely suitable for the capacity increase.
N. ARCHAEOLOGICAL SITES • Will the alternative have significant impacts during construction and/or from ongoing operations on registered/known archaeological features?	**LOW IMPACT** There would be no archaeological site impacts beyond what already exists because no new site work is needed for this alternative.
O. CULTURAL/HERITAGE FEATURES • Will the alternative have significant impacts during construction and/or from ongoing operations on known cultural landscapes and built heritage features?	**LOW IMPACT** There would be no cultural/heritage feature impacts beyond what already exists with the current system.
P. CAPITAL COST • What will be the relative capital cost for the alternative?	**LOW COST ALTERNATIVE** There would be a low relative capital cost with no new construction required. Cost impacts include replacement of the existing well pump and switchgear, as required. Other cost impacts would stem from a greater frequency of chemical delivery.
Q. 20-YEAR LIFECYCLE COST • What will be the relative 20-year life cycle cost for the alternative?	**LOW COST ALTERNATIVE** This alternative has a relatively low 20 year life-cycle cost.
R. LAND ACQUISITION COST • What will be the relative land acquisition cost for the alternative?	**LOW COST ALTERNATIVE** No new land will be required for this alternative.

Evaluation Criteria	Alternative A: Use Existing Infrastructure for Expansion for Well No. 2
S. LAND REQUIREMENTS • What will be the level of area of non-regional land or easement required to construct the alternative?	**LOW REQUIREMENT** There will be no land required for this alternative.
T. ABILITY TO ACCOMMODATE POTENTIAL FUTURE REGULATORY CHANGES • Will the alternative have the ability to adapt to potential future changes in drinking water quality requirements?	**HIGH ADAPTABILITY** The existing site is adaptable for addition of conventional and advanced treatment technologies to accommodate potential future regulatory changes.
U. PERMITS AND APPROVALS • What will be the level of permits and approvals required to construct the alternative?	**MODERATE REQUIREMENT** Amendable permits include the permit to take water for 32 L/s. Additionally, amendments to the DWWP and MDWL would be required; a review of existing wellhead protection area (WHPA) delineation assumptions will determine whether additional permitting requirements from the Ministry of Environment, Conservation and Parks (MECP) and Source Protection Authority are necessary.

Table 3-4 Short Listed Alternative Water Pumping and Treatment Solutions - Detailed Evaluation

Evaluation Criteria	Alternative 1: Expand Existing Facility	Alternative 2: Add 2nd Treatment Train (Dedicated for Well Site H)
CONCEPTS		
A. CONSTRUCTABLITY • What are the major construction challenges and risks (e.g., crossing environmentally sensitive areas, noise, odour, dust, public safety, traffic, etc.) associated with the alternative? • To what extent does it impact the community? • How much volume and complexity of construction will be associated with the alternative?	**MAJOR IMPACT** There will be major constructability challenges relating to increasing the size of main line piping, valves, and instrumentation, and replacing the chlorinators. Challenges would include removing Well Site No. 5 from service for a significant length of time to perform modifications, which may impact the ability to meet demand.	**MODERATE IMPACT** A second treatment train would include all types constructability challenges when building a new facility—noise, traffic, dust, etc. It may impact the community during the ongoing construction of the facility, but this will be dissipated when construction ends. The complexity of the construction will be low and will allow Well Site No. 5 to remain in service for the majority of construction.
B. REDUNDANCY OF SUPPLY/SERVICE • Will the alternative be able to provide improvements in redundancy of supply or service?	**LESS REDUNDANCY** A higher capacity will cause a shorter supply of chemicals on hand, resulting in less redundancy of chemicals.	**HIGH REDUNDANCY** A second treatment train would increase overall redundancy at the site. The 2nd treatment train would not be impacted from disruptions at Well Site No. 5. Chemical storage would be sized for required redundancy at Well Site H.
C. RESILIENCE TO CLIMATE CHANGE • Will the alternative have the resilience against changing climate conditions, such as changes to water supply quantity and quality (e.g., high water demands, drought)?	**HIGH RESILIENCE** There would be a high resilience against changing climate conditions.	**HIGH RESILIENCE** The second treatment train would be able to have resistance similar to that of the existing treatment facility.
D. O&M REQUIREMENTS • What will be the level of additional and new O&M resources (e.g., human resources) required for the alternative? • What will be the level of complexity and maintainability of new and optimized assets?	**LOW COMPLEXITY** There will be a low level of additional O&M resources required beyond the resources already required due to no additional unique assets or infrastructure for this alternative.	**MODERATE COMPLEXITY** Adding a second treatment train would increase the amount of equipment to be maintained, leading to higher O&M requirements.
E. ADAPTABILITY TO EXISTING INFRASTRUCTURE • What will be the level of modification required to the existing infrastructure to adapt to the alternative? What is the relative ease of connection to the existing alternative?	**MAJOR MODIFICATION** This alternative requires replacement of existing main line piping and chlorinators.	**MINIMAL MODIFICATION** Adding a second treatment train requires minimal modification to the existing Well Site No. 5. The new treatment train would tie in downstream of Well Site No. 5.
F. MAXIMIZING USE OF EXISTING INFRASTRUCTURE • Will the alternative be able to maximize the capacity of the existing infrastructure to reduce new assets needs?	**HIGH DEGREE** This alternative utilizes most existing infrastructure.	**LOW DEGREE** The new treatment train will not use existing infrastructure, other than finished water piping.
G. AQUATIC VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Streams and river. ○ Local aquatic species and habitat. ○ Environmentally sensitive areas, aquatic species at risk, and locally significant aquatic species.	**MODERATE IMPACT** There will be moderate impact on the aquatic vegetation and wildlife during construction due to the need to expose the existing chlorine contact pipe	**MODERATE IMPACT** With ongoing construction and aquifer testing for the new well, personnel, and traffic, the local habitats, animals, and environmentally sensitivity areas may be impacted significantly. Although this depends on the level of local aquatic species and habitat already present at the site.
H. TERRESTRIAL VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Trees and vegetation. ○ Local terrestrial species and habitats. ○ Environmentally sensitive areas, species at risk, and locally significant species.	**MODERATE IMPACT** There will be moderate impact on the terrestrial vegetation and wildlife during construction due to the need to expose the existing chlorine contact pipe.	**MODERATE IMPACT** By building a second building, local vegetation will need to be removed, possibility disrupting any existing habitats and species.

Evaluation Criteria	Alternative 1: Expand Existing Facility	Alternative 2: Add 2nd Treatment Train (Dedicated for Well Site H)
I. GROUNDWATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on aquifers and groundwater resources such as groundwater quantity, groundwater recharge quality and flow regime and groundwater discharge to streams and wetlands?	**LOW IMPACT** There would be a greater withdrawal of groundwater from expansion.	**LOW IMPACT** Neither alternative is expected to impact groundwater resources.
J. SURFACE WATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on adjacent surface water resources (e.g., Humber River) and related biological communities?	**LOW IMPACT** Neither alternative is expected to impact surface water resources.	**LOW IMPACT** Neither alternative is expected to impact surface water resources.
K. GREENHOUSE GAS EMISSIONS • What will be the level of GHG emissions associated with the alternative? *(GHG emissions will be evaluation based on the alternative's energy intensity requirements.)*	**LOW IMPACT** There will be low increase of GHG emissions associated with the alternative. Increases could stem from greater frequency of chemical transportation and greater energy demand for the pumps.	**MODERATE IMPACT** There could be a moderate increase in GHG emissions from all the energy required to operate an additional facility and transportation for supply/servicing.
L. SHORT-TERM COMMUNITY IMPACTS • Will the alternative have significant short-term impacts to the community during construction, including: ○ Noise, dust, and odour. ○ Local traffic.	**MODERATE IMPACT** There would be moderate impact due to construction of expanding the size of the main line piping and replacing the chlorinators, which includes noise, dust, odour, and local traffic. Additionally, increasing the size of the main line piping will require the existing well site to be removed from service, which may impact ability to meet demand.	**MODERATE IMPACT** With construction of a new facility, noise, dust, potential odour, local traffic will be experienced.
M. LONG-TERM COMMUNITY IMPACT • Will the alternative have significant long-term impact to the community, including: ○ Benefit to community. ○ Impacts from facility operations. ○ Visual impact. ○ Public acceptance/resistance.	**LOW IMPACT** Expansion of the existing treatment train will have no long-term impacts on the community.	**MODERATE IMPACT** Addition of a new treatment train will result in additional buildings and equipment, which may be perceived by the community as detrimental.
N. ARCHAEOLOGICAL SITES • Will the alternative have significant impacts during construction and/or from ongoing operations on registered/known archaeological features?	**LOW IMPACT** No archaeological site impacts are assumed beyond those that already exist from the original construction of the treatment facility.	**LOW IMPACT** No archaeological sites impacts are assumed beyond those that already exist from the original construction of the treatment facility.
O. CULTURAL/HERITAGE FEATURES • Will the alternative have significant impacts during construction and/or from ongoing operations on known cultural landscapes and built heritage features?	**LOW IMPACT** No cultural or heritage feature impacts are assumed beyond those that already exist from the original construction of the treatment facility.	**LOW IMPACT** No cultural or heritage feature impacts are assumed beyond those that already exists from the original construction of the treatment facility.
P. CAPITAL COST • What will be the relative capital cost for the alternative?	**LOW COST ALTERNATIVE** The capital cost of increasing the main line piping and replacing the chlorinators is relatively low compared to adding a new treatment train. However, this alternative requires taking the facility out of service for the duration of construction.	**MODERATE COST ALTERNATIVE** The capital cost of adding a new treatment train is moderately higher than expanding the existing treatment train
Q. 20-YEAR LIFECYCLE COST • What will be the relative 20 year life-cycle cost for the alternative?	**LOW COST ALTERNATIVE** The life-cycle cost of increasing the main line piping and replacing the chlorinators is relatively low compared to adding a new treatment train.	**MODERATE COST ALTERNATIVE** The cost of adding a new treatment train is moderately higher than expanding the existing treatment train
R. LAND ACQUISITION COST • What will be the relative land acquisition cost for the alternative?	**LOW COST ALTERNATIVE** No new land would be required for this alternative.	**LOW COST ALTERNATIVE** No new land would be required for this alternative.
S. LAND REQUIREMENTS	**NO LAND REQUIREMENT**	**NO LAND REQUIREMENT**

Evaluation Criteria	Alternative 1: Expand Existing Facility		Alternative 2: Add 2nd Treatment Train (Dedicated for Well Site H)	
• What will be the level of area of non-regional land or easement required to construct the alternative?	No new land would be required for this alternative.		No new land would be required for this alternative.	
T. ABILITY TO ACCOMMODATE POTENTIAL FUTURE REGULATORY CHANGES • Will the alternative have the ability to adapt to potential future changes in drinking water quality requirements?	(green)	**HIGH ADAPTABILITY**	(yellow)	**MODERATE ADAPTABILITY**
	The existing site is adaptable for addition of conventional and advanced treatment technologies to accommodate potential future regulatory changes.		The new treatment train would be adaptable for addition of conventional and advanced treatment technologies to accommodate potential future regulatory changes. However, less space would be available for such technologies.	
U. PERMITS AND APPROVALS • What will be the level of permits and approvals required to construct the alternative?	(yellow)	**MODERATE REQUIREMENT**	(red)	**HIGH REQUIREMENT**
	Fewer permits are anticipated to be required for modifying/expanding the existing train.		More permits are anticipated for a new treatment train.	

4.0 Wastewater System Alternative Design Concepts

4.1 Existing Wastewater Servicing (Installed Infrastructure)

4.1.1 Gravity Collection System

The Nobleton wastewater collection system consists of over 50 kilometers (km) of gravity sewers. All of the gravity sewers in the collection system are owned by the Township of King, except for a short section of pipe, less than 50 m, upstream of the Janet Avenue PS, which is owned by York Region.

The current wastewater collection system does not cover the entire community of Nobleton; some areas are still on septic tanks. There is an ongoing Township of King project to connect the remaining properties within Nobleton to the sewer system by 2021.

Hydraulic modelling carried out during Phase 2 of the Class EA established that the existing sewer infrastructure has sufficient capacity to accommodate the design peak flows. Therefore, no twinning or expansion of the existing sanitary sewer infrastructure would be required to service the design peak flows, which include the areas that are currently on septic tanks.

4.1.2 Janet Avenue Pumping Station

The existing Janet Avenue PS is a dry well/wet well type pumping station with a total of three pumps installed in the dry well. Two pumps provide duty while one pump is a standby pump. Each pump is equipped with a 200 mm suction pipe and a 200 mm discharge pipe. The pump discharge pipes join into a 250 mm header, which is equipped with a flowmeter. The pipe size increases to 300 mm immediately after it exists the station building. The pipe material also changes from stainless steel to PVC.

The Janet Avenue PS has a peak capacity of 9,177 m^3/d (107 L/s), as identified in Study 1B. In order to accommodate the design peak flows of 25,174 m^3/d (292 L/s), the Janet Avenue PS would need to be expanded either by replacing the existing pumps with larger units or providing additional pumps. Depending on whether the capacity will be expanded to 25,174 m^3/d (292 L/s), or to a lesser extent in combination with flow attenuation, a new wet well may or may not be required.

4.1.3 Forcemain

The existing 300 mm DR 18 and DR 26 PVC forcemain runs from the Janet Avenue PS to the Nobleton WRRF. The 4.5 km forcemain is largely aligned along King Street. The 300 mm forcemain is not capable of conveying the design peak instantaneous flow of 25,174 m^3/d (292 L/s). Depending on the preferred design concept for expansion of the Janet Avenue PS, i.e., a full expansion to 25,174 m^3/d (292 L/s) without flow attenuation or a lesser expanded capacity with flow attenuation, the forcemain may or may not be twinned/replaced to accommodate the design peak flows.

TM2 selected the preferred solution that included expansion of the forcemain along with the Janet Avenue PS and the Nobleton WRRF. However, the design concepts developed during TM3 include concepts that avoid twinning of the forcemain to address flow attenuation upstream of the Janet Avenue PS. These concepts are in keeping with the broad approach to the preferred solution identified as part of TM2.

4.1.4 Wet Weather Flow Management

Wet Weather Flow Management Strategy – Study 1B found that the existing infrastructure experiences high peak instantaneous flows, representing an average peaking factor of 6.3. This peaking factor was used to calculate design PIF requirements of 25,174 m^3/d (292 L/s).

A wet weather flow (WWF) management strategy for reduction of high peak flows into the wastewater system could reduce infrastructure costs for upgrades and expansion at the Janet Avenue PS and Nobleton WRRF and eliminate twinning of the King Street forcemain, as well as twinning of the constrained sections of the treated effluent outfall. The following WWF management strategy could be considered:

- **Flow Equalization** – High peak flows during wet weather events could be reduced by controlling the flow rates through the wastewater system. The approach for flow equalization could be to provide an inline, or offline, flow attenuation storage facility at the Janet Avenue PS and/or the Nobleton WRRF and an effluent pump station at the Nobleton WRRF.

4.1.5 Water Resource Recovery Facility

The Nobleton WRRF is an extended aeration activated sludge facility with chemical addition for phosphorous removal and tertiary filtration. Other unit processes include preliminary treatment, effluent disinfection with ultraviolet light, sludge thickening, and sludge storage. The treated effluent is discharged by gravity to the Humber River via a constructed wetland. Residual solids including biological sludge from the extended aeration system and chemical sludge from phosphorus removal are thickened by gravity and stored on-site prior to hauling to Duffin Creek Water Pollution Control Plant (WPCP) for disposal via Aurora Sewae Pumping Station (SPS). The ECA rated capacity of the facility is 2,925 m^3/day with a peak design flow of 9,177 m^3/day.

A summary of the installed infrastructure is shown in Table 4-1. A process flow diagram is shown on Figure 4-1.

Table 4-1 Nobleton WRRF Summary of Installed Infrastructure

Process	Equipment Item	Unit	Value	Comments
Preliminary Treatment – Coarse Screens	**Screening System**			
	Number of Screens	#	2	One mechanical duty unit One manual bar rack standby
	Type	Coarse	--	
	Openings	12	mm	Mechanical screen
		50	mm	Manual bar rack
	ECA Rated Peak Flow Capacity (Duty)	m^3/d	9,177	Mechanical only
	Screening Screw Conveyor			
	Number of Conveyors	#	1	
	Dimensions	mm	292 x 6,180	
	Inlet Capacity	m^3/h	1.5	
	Discharge Capacity	m^3/h	1.5	

Process	Equipment Item	Unit	Value	Comments
Preliminary Treatment – Grit Removal	**Grit Removal System**			
	Number of Grit Tanks	#	2	
	Type	Induced Vortex		
	Dimension	m	2.0	Diameter
	ECA Rated Peak Flow Capacity	m^3/d	9,177	
Secondary Treatment	**Aeration Tanks**			
	Number of Tanks	#	2	One aeration tank in service is adequate for current conditions
	Dimension (each)	m	18 x 13.5 x 6.3	Width x Length x Height (side water depth [SWD])
	Volume (each)	m^3	1,536	
	Volume (total)	m^3	3,072	
	Air Blowers			
	Number of Blowers	#	3	Two duty/one standby, 22 kW each
	Capacity	L/sec	213	Each (rated at 70 kilopascal [kPa])
	Diffuser			
	Type	--	--	Fine bubble membrane diffusers
	Total Number of Diffusers	--	1,452	762 each tank
	Design Clean Water Transfer Efficiency	%	37.3	
	Secondary Clarifiers			
	Number of Clarifiers	#	2	One clarifier in service is adequate for current conditions
	Dimensions	m	15.15 x 4.85	Diameter x Depth (SWD)
	Surface Area (Total)	m^2	360	Two units
	Sludge Return			
	Number of Pumps	#	3	2 duty/1 standby
	Type	Centrifugal		
	Capacity	34	L/s	Each. @ 7.5 m TDH
	Sludge Wasting			
	Number of Pumps	#	2	1 duty/1 standby
	Type	Centrifugal		
	Capacity	9	L/s	Each. @ 5.0 m TDH

Process	Equipment Item	Unit	Value	Comments
Tertiary Treatment	**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 standby
	Type	--	Submersible	
	Capacity	L/s	7.8	Each, at 32.8 m TDH
	Filter Drain Pumping			
	Number of Pumps	--	2	One duty, one standby
	Type	--	Submersible	
	Capacity	L/s	5	Each, at 14.4 TDH
Chemical Treatment	**Alum (phosphorous removal)**			
	Number of Metering Pumps	#	3	2 duty / 1 standby Discharge upstream of clarifiers
	Capacity	L/h	65	Each ump @ 300 kPa
	Capacity	kg alum/d	2,000	
	Number of Metering Pumps	#	2	1 duty / 1 standby. Discharge upstream of filters.
	Capacity	L/h	17.1	Each Pump. @ 300 kPa
	Capacity	kg alum/d	260	
	Number of Storage Tanks	#	1	
	Storage Tank Volume	L	20,000	
	Alkalinity (system not in use)			
	Number of Pumps	#	2	
	Type	Metering		
	Capacity	L/h	4.4	Each pump
	Number of Storage Tanks	#	1	
	Storage Tank Volume	L	10,000	

Process	Equipment Item	Unit	Value	Comments
Effluent Disinfection	**UV Disinfection**			
	Peak Flow Capacity	m^3/day	9,177	
	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 Total Suspended Solids (TSS)	mg/L	30	30 day average
Effluent Chamber and Outfall Sewer	**Outfall Sewer**			
	Diameter	mm	450	Outfall sewer capacity is based on a slope of 0.35% and 70% full pipe.
	Type	Concrete		
	Length	km	1.5	
	Capacity	L/s	145	
Sludge Handling	**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	
	Maximum Solids Loading Rate	kg/m^2/d	36	MECP Design Guidelines
	Maximum Waste Activated Sludge (WAS) Loading Rate	kg/d	620	Maximum month loading condition
	Emergency Sludge Loading Pump Capacity	L/s	25	At 12 m 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	--	--	Coarse bubble diffusers
	Sludge Loading Pump Capacity	L/s	12	At 12 m TDH
	Number of Blowers	#	2	
	Blower Capacity	L/s	93	@ 60 kPa

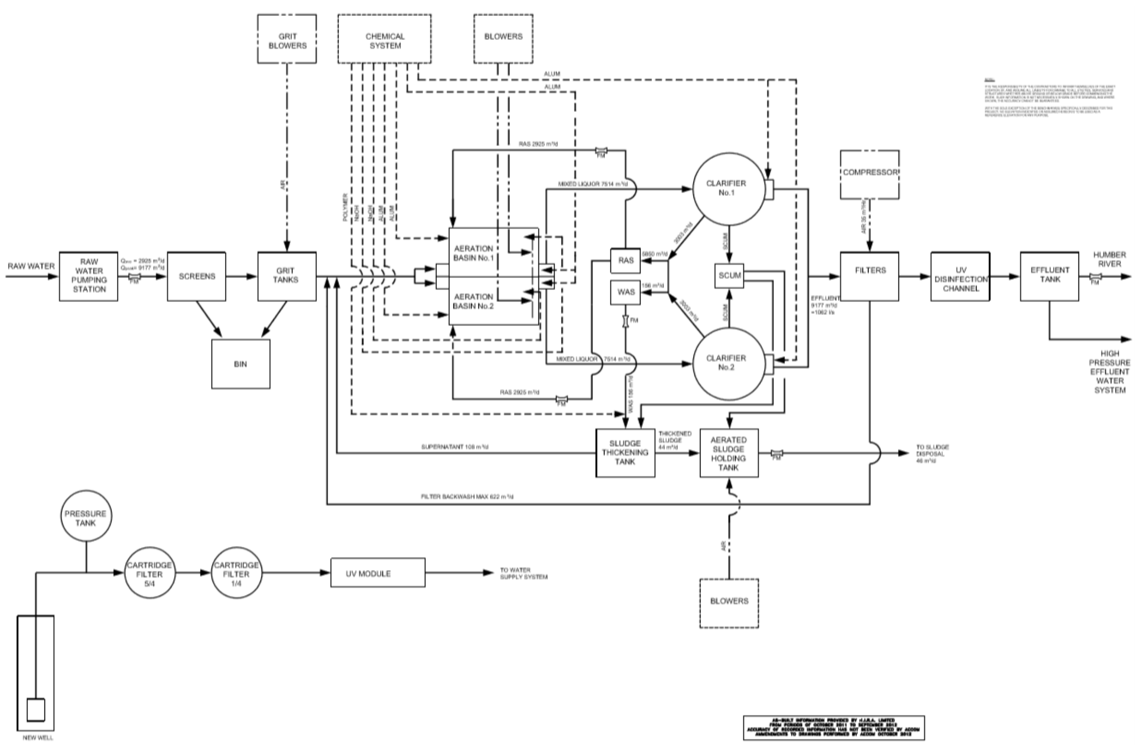

Figure 4-1 Nobleton WRRF Process Flow Diagram

4.1.6 Outfall

A 450 mm diameter RCC conveys treated effluent from the WRRF to the Humber River. The slope of the gravity outfall pipe varies from a minimum of 0.35 percent to 5 percent. Therefore, the limiting capacity of the outfall is 12,500 m^3/d (145 L/s) at 70 percent full.

Depending on the extent of expansion of the Janet Avenue PS, i.e., with or without flow attenuation, twinning of the constricted sections of the effluent outfall (668 m) may or may not be required.

4.2 Design Basis

4.2.1 Collection System

The minimum scour velocity of 0.6 m/s should be achieved at least once during 24 hours according to the MECP Design Guidelines. The dry weather and WWF criteria were documented in the hydraulic modelling and needs assessment studies.

4.2.2 Janet Avenue Pumping Station

The Janet Avenue PS should be able to pump out the PIF received through the inlet pipe.

The maximum design flow for the pumping station would be 25,174 m^3/d (292 L/s). The pumping station may receive lesser flows than that if flow attenuation is provided upstream of the pumping station.

4.2.3 Forcemain

MOE Design Guidelines for Sewage Works (2008), Section 7.9.1, provides guidance on the range of velocities in a sanitary forcemain. This range is from a minimum of 0.6 m/s to achieve scouring, to a maximum of 3.0 m/s. The sanitary forcemain is typically designed for a velocity range of 0.8 m/s to 2.4 m/s. For a 300 mm PVC DR18 pipe with an internal ID of 298 mm, this velocity range equates to a flow range of 4,840 m^3/d (56 L/s) to 14,500 m^3/d (167 L/s).

4.2.4 Wet Weather Flow Management

According to York Region Standards, a 1 in 25 year storm was applied to the hydraulic model to establish the inlet hydrograph at the Janet Avenue PS. The inlet hydrograph generated by the model is included in Appendix B.

The WWF management scenarios calculate equalization storage volume by shaving off the flows above a flow limit established with a view to reduce/eliminate some downstream infrastructure upgrades.

The modelled peak flow obtained from the hydraulic model simulation at the Janet Avenue PS is approximately 246 L/s. However, considering the peak instantaneous to average flow factor observed for the catchment of 6.3 (Table 4-5, TM1), the peak flow at the Janet Ave PS was considered to be 292 L/s. Since the model-produced hydrograph was utilized to calculate the volume of the flow attenuation tank at the Janet Ave PS, the volume obtained was increased by 20 percent to account for the higher PIFs than predicted by the model.

4.2.5 Water Resource Recovery Facility

The design basis for each WRRF process is shown in Table 4-2.

Table 4-2 Nobleton WRRF Treatment Process Design Basis

Treatment Process	Design Basis Criterion[1]	Design Basis	Notes
Equalization	-	-	
Preliminary Treatment – Screening	PIF[2]	25,175 m^3/d	Assumes no flow attenuation. PIF could be reduced depending on the preferred WWF management alternative
Preliminary Treatment – Grit Removal	PHF[2], peak hourly grit loading	18,781 m^3/d (782 m^3/h); 3.1 to 29 L grit/h	Assumes no flow attenuation. PHF[2] could be reduced depending on the preferred WWF management alternative
Primary Treatment	Peak Day Flow (PDF)[2]	8,791 m^3/day	
Secondary Treatment – Extended Aeration (with nitrification)	Average daily biochemical oxygen demand (BOD_5) loading based on design ADF[2]; peak daily total Kjeldahl nitrogen (TKN) loading based on design PDF[2,3]	683 kg BOD_5/d[2]; 144 kg TKN/d	Design minimum wastewater temperature is 8° C. Design maximum wastewater temperature is 20° C
Secondary Treatment – Sedimentation	PHF[2], PDF solids loading	18,781 m^3/d	Assumes no flow attenuation. PHF could be reduced depending on the preferred WWF management alternative

Treatment Process	Design Basis Criterion[1]	Design Basis	Notes
Secondary Treatment (Sludge Return)	50 to 200% of design ADF	1,998 m^3/day (23 L/s) to 7,992 m^3/day (92.5 L/s)	
Secondary Treatment (Sludge Wasting)	0.5 to 25% of design ADF[4]	20 m^3/day (0.23 L/s) to 999 m^3/day (11.6 L/s)	
Chemical Phosphorus Removal	Total phosphorous (TP) load[2], molar ratio of coagulant to TP[8]	Maximum month wastewater TP Load is 40.2 kg/d; the molar ratio of Al:TP averages 6.5	Chemical dosing could be reduced if biological nutrient removal is incorporated into secondary treatment
Disinfection	PHF[2]	18,781 m^3/d	Assumes no flow attenuation. PHF could be reduced depending on the preferred WWF management alternative
Effluent Filtration	PHF[2]	18,781 m^3/d	Assumes no flow attenuation. PHF could be reduced depending on the preferred WWF management alternative
Outfall Sewer	PIF[2]	25,175 m^3/	Assumes no flow attenuation. PIF could be reduced depending on the preferred WWF Management Alternative
Sludge Treatment (digestion and dewatering)	Maximum monthly mass loading[6] and flow rates[7]	479 kg WAS/d; 60 m^3/d	

1. Design Guidelines for Sewage Works (Ontario MECP, 2021) unless otherwise noted.
2. Nobleton EA Phase I: Identify the Problem or Opportunity.
3. Maximum month load.
4. Recommended Standards for Wastewater Facilities (GLUMRB , 2014)
5. General engineering knowledge.
6. Based on Ontario MECP guidelines of 120 g/m^3 wastewater treated for extended aeration activated sludge with phosphorus removal and aerated sludge holding tank.
7. Based on 8,000 mg TSS/L.
8. Based on plant historical operational data.

The current treated effluent objectives and limits are shown in Table 4-3 and are assumed to be subject to minor changes when the ECA is revised for plant expansion.

Table 4-3 Nobleton WRRF Treated Effluent Limits and Objectives

Parameter	Effluent Objectives(Mg/L)	Effluent Limits (Mg/L)	Effluent Limits (Kg/yr)
$cBOD_5$	5.0	10.0	-
TSS	7.0	10.0	-
TP	0.1	0.15	160
Total Ammonia Nitrogen	0.5 (May – Oct) 2 (Nov – Apr)	1.0 (May – Oct) 3.0 (Nov – Apr)	-
E. Coli	100 counts / 100 mL	200 counts / 100 mL	-

4.2.6 Outfall

The gravity outfall twinning, if required, will ideally be installed at slopes similar to the existing slopes. It appears that the sections of the 450 mm outfall installed at 0.35 percent slope will need twinning to accommodate the PIF of 25,174 m^3/d (292 L/s).

4.3 Screening and Evaluation of Design Concepts

4.3.1 Janet Avenue Pumping Station, Flow Attenuation, Forcemain, and Outfall

The existing Janet Avenue PS will be expanded to accommodate the design PIF of 25,174 m^3/d (292 L/s). This could be achieved with or without providing flow attenuation upstream of the pumping station.

4.3.1.1 Long List of Alternative Design Concepts

The long list of alternative design concepts for the Janet Avenue PS and flow attenuation storage is as follows.

Alternative 1: No Flow Attenuation

No flow attenuation will be provided either upstream of the Janet Avenue PS or at the WRRF. This would result in all PIF received at the Janet Avenue PS to be pumped through the forcemain to the WRRF and the outfall. This option would require expansion/twinning of all the downstream infrastructure to accommodate PIF. The key components of this alternative are as follows:

- Expand the Janet Avenue SPS to a firm capacity of 25,174 m^3/d (292 L/s).
- Twin the existing 300 mm sanitary forcemain (4,522 m of 300 mm PVC DR 18 and DR 26 pipe).
- Twin the constricted part of the effluent outfall (668 m of 450 mm RCC Class 100D pipe).

Alternative 2: Flow Attenuation at the WRRF

Flow attenuation will be provided at the WRRF. This wouldresult in all PIF received at the Janet Avenue PS to be pumped through the forcemain, where flow will be equalized in a tank to limit peak flows to the WRRF and the effluent outfall. Flow to the equalization tank at the WRRF would flow into the tank by gravity and would need to be pumped out to the WRRF during low inflow periods. The key components of this alternative are as follows:

- Expand Janet Avenue SPS to a firm capacity of 292 L/s.
- Twin the existing 300 mm sanitary forcemain (4,522 m of 300 mm PVC DR 18 and DR 26 pipe).
- Provide an equalization tank at the WRRF (1,300 m^3 needed to limit peak instantaneous flow to WRRF at 12,500 m^3/d (145 L/s). This equalization tank would be equipped with a PS lift wastewater to the headworks.
- Twin the constricted part of the effluent outfall (668 m of 450 mm RCC Class 100D pipe) (not needed if the PIF is greater than the outfall capacity [145 L/s]).
- For storage volume calculation, please refer to Appendix B.

Alternative 3A: Flow Attenuation at the Janet Avenue Pumping Station with Belowground Storage Tank

Flow attenuation will be provided immediately upstream of the Janet Avenue PS. This would result in limiting PIF to the Janet Avenue PS, forcemain, WRRF, and outfall. The key components of this alternative are the following:

- Provide a belowground flow attenuation tank at the Janet Avenue PS (1,300 m^3) to limit PIF to the WRRF at 12,500 m^3/d (145 L/s).
- Expand the Janet Avenue PS to 145 L/s from 107 L/s.
- For storage volume calculation, please refer to Appendix B.

Alternative 3B: Flow Attenuation at the Janet Avenue Pumping Station with Gravity Pipe

Flow attenuation will be provided immediately upstream of the Janet Avenue PS. This would result in limiting PIF to the Janet Avenue PS, forcemain, WRRF, and outfall. The key components of this alternative are the following:

- Provide an inline or offline gravity pipe to provide flow attenuation storage at the Janet Avenue PS (1,300 m^3) to limit PIF to the WRRF at 12,500 m^3/d (145 L/s).
- Expand the Janet Avenue PS to 145 L/s from 107 L/s.
- For storage volume calculation, please refer to Appendix B.

4.3.1.2 Screening of Janet Avenue Pumping Station, Flow Attenuation, Forcemain, and Effluent Outfall Alternative Design Concepts

The three alternative design concepts developed for the Janet Avenue PS, flow attenuation, forcemain, and effluent outfall developed and presented above were evaluated against the screening criteria presented in Table 4-4. As an outcome of the screening process, Alternative 3 was selected to proceed to the next stage of detailed evaluation. Alternatives 1 and 2 were screened out.

Table 4-4 Screening of Wastewater Pumping, Flow Attenuation, Forcemain, and Effluent Design Concepts

Long List of Alternative Janet Avenue Pumping Station Design Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness and Reliability	Stakeholder Acceptance	Construction Impacts	Cost	
1. Alternative 1: No Flow Attenuation	✓	✓	×	✓	✓	×	• **Eliminated** because of these reasons: • This alternative would result in expansion of all the components starting from the Janet Avenue PS and downstream to the outfall. • Expanded PS, twinned forcemain, WRRF, twinned outfall would remain below utilization most of the time except during large storm events. • Cause of odour concerns for the forcemains if one is taken offline for long periods. • Relatively expensive in comparison withAlternative 3
2. Alternative 2: Flow Attenuation at the WRRF	✓	✓	×	✓	✓	×	• **Eliminated** due to reasons below: • This alternative would result in expansion of all the components starting from the Janet Avenue PS and downstream to the outfall. • Expanded PS, twinned forcemain, WRRF, twinned outfall would remain below utilization most of the time except during large storm events. • Cause of odour concerns for the forcemains if one is taken offline for long periods. • An equalization tank would be provided at the WRRF, which would have an additional PS. • Most expensive alternative in comparison with Alternative 3.

Long List of Alternative Janet Avenue Pumping Station Design Concepts	Screening Criteria: Compatibility	Proven Technology	Performance Robustness and Reliability	Stakeholder Acceptance	Construction Impacts	Cost	Notes
3. Alternative 3A: Flow Attenuation at the Janet Avenue Pumping Station with Belowground Storage Tank	✓	✓	✓	✓	✓	✓	• **Proceed to detailed evaluation** because of these reasons: • This alternative will eliminate twinning of the 300 mm forcemain, minimizing cost and construction impacts. • This alternative will also eliminate twinning of the 450 mm effluent outfall sections. • The alternative would minimize the expansion of the Janet Avenue PS. Major civil and structural works may be avoided. • Providing flow attenuation at the Janet Avenue PS will result in the PS being expanded to a lesser capacity, 12,528 m^3/d (145 L/s) and also eliminate the twinning of the 4.5 km long forcemain and 668 m of constricted sections of the 450 mm effluent outfall. • Least expensive alternative—costing considerably less than Alternatives 1 and 2.
4. Alternative 3B: Flow Attenuation at the Janet Avenue Pumping Station with Gravity Pipe	✓	✓	✓	✓	✓	✓	• **Proceed to detailed evaluation** because of these reasons: • This alternative will eliminate twinning of the 300 mm forcemain, minimizing cost and construction impacts. • This alternative will also eliminate twinning of the 450 mm effluent outfall sections. • The alternative wouldminimize the expansion of the Janet Avenue PS. Major civil and structural works may be avoided. • Providing flow attenuation at the Janet Avenue PS will result in the PS being expanded to a lesser capacity, 12,528 m^3/d (145 L/s), and also eliminate the twinning of the 4.5 km long forcemain and 668 m of constricted sections of the 450 mm effluent outfall. • Least expensive alternativ—costing considerably less than Alternatives 1 and 2.

4.3.1.3 Short List of Design Concepts

Alternatives 3A and 3B are based on gravity in and out from the flow attenuation tank and gravity pipe, respectively, an option that is inherently more robust and reliable than receiving the entire peak flow at the Janet Avenue PS and then pumping it downstream during a WWF event. Mechanical breakdowns, power outages, generator failure, fuel interruptions, etc., could occur that would increase the risk of an emergency overflow to the environment compared with Alternatives 3A and 3B. In addition, Alternatives 3A and 3B are substantially less expensive to implement compared to Alternatives 1 and 2 because they eliminate forcemain twinning and outfall twinning.

After the screening of the long list of alternative design concepts for the Janet Avenue PS, flow attenuation, King Street forcemain, and treated effluent outfall, the alternative design concepts of providing flow attenuation at the Janet Avenue PS, elimination of the forcemain twinning and elimination of the effluent outfall twinning were short-listed for further evaluation. The short-listed design concepts 3A and 3B were further evaluated. Refer to Table 4-5.

4.3.1.4 Evaluation of Alternative Design Concepts

Table 4-5 Short Listed Alternative Janet Avenue Pumping Station, Flow Attenuation, Forcemain, and Effluent Outfall Alternative Design Concepts - Detailed Evaluation

Evaluation Criteria	Alternative 3A: Offline Storage Tank At Janet Avenue PS	Alternative 3B: Inline Or Offline Gravity Pipe Upstream Of Janet Avenue PS
TECHNICAL		
A. CONSTRUCTABLITY • What are the major construction challenges and risks (e.g., crossing environmentally sensitive areas, noise, odour, dust, public safety, traffic, etc.) associated with the alternative? • To what extent does it impact the community? • How much volume and complexity of construction will be associated with the alternative? f	**LOW IMPACT** • Excavation required for a sizeable footprint (15.5 m X 12 m X 11 m deep) at the Janet Avenue PS site. • The Janet Avenue PS is in a residential area. Therefore, the community will be impacted by construction. • The existing Janet Avenue PS needs to be operational at firm capacity during the construction.	**MODERATE IMPACT** • Excavation required to install a large and deep pipe (3 m to 3.6 m diameter and up to 11 m deep) on the approach road to the Janet Avenue PS, and a chamber to connect the new pipe to the wet well. • The Janet Avenue PS is in a residential area. Therefore, the community will be impacted by construction. • The existing Janet Avenue PS needs to be operational at firm capacity during the construction. • Alternative access to the Janet Avenue PS will be needed during the construction of the big pipe on the approach road leading to the PS, causing further community impacts.
B. REDUNDANCY OF SUPPLY/SERVICE • Will the alternative be able to provide improvements in redundancy of supply or service?	**HIGH REDUNDANCY** • The PS firm capacity will increase by addition of larger and/or additional pumps, and/or increase in wet well capacity. • The flow attenuation tank will provide redundancy to divert flows to the tank if required during dry weather as well.	**HIGH REDUNDANCY** • The PS firm capacity will increase by addition of larger and/or additional pumps and/or increase in wet well capacity. • The big pipe will provide buffer to store flows if the pump station is operating at reduced capacity for preventative maintenance or breakdown.
C. RESILIENCE TO CLIMATE CHANGE • Will the alternative have the resilience against changing climate conditions, such as changes to water supply quantity and quality (e.g., high water demands, drought)?	**LOW RESILIENCE** • The facilities are sized based on Rainfall Derived Infiltration and Inflow (RDII) for a current 1 in 25 year wet weather event. Wet weather resilience has not been built into the volume calculation of the flow attenuation tank.	**LOW RESILIENCE** • The facilities are sized based on RDII for a current 1 in 25 year wet weather event. Wet weather resilience has not been built into the volume calculation of the flow attenuation tank.
D. O&M REQUIREMENTS • What will be the level of additional and new O&M resources (e.g., human resources) required for the alternative? • What will be the level of complexity and maintainability of new and optimized assets?	**MODERATE COMPLEXITY** • The expansion of the Janet Avenue PS will result in moderate increase of the O&M resources. • The new flow attenuation tank will need new equipment such as a coarse bubble aeration system, including blowers, in addition to tipping buckets and odour control and will result in moderate increase to the complexity of operation.	**MODERATE COMPLEXITY** • The expansion of the Janet Avenue PS will result in moderate increase of the O&M resources. • The new big pipe will not result in appreciable increase in the operation complexity. The key additional system envisaged for this infrastructure is a new odour control system.
E. ADAPTABILITY TO EXISTING INFRASTRUCTURE • What will be the level of modification required to the existing infrastructure to adapt to the alternative? What is the relative ease of connection to the existing alternative?	**MODERATE ADAPTABILITY** • Modest modifications will be needed to connect the new flow attenuation tank to the existing wet well. Moderate structural work will be needed.	**MODERATE ADAPTABILITY** • A new chamber will be needed to connect the new big pipe to the wet well. Moderate civil and structural work will be needed.
F. MAXIMIZING USE OF EXISTING INFRASTRUCTURE • Will the alternative be able to maximize the capacity of the existing infrastructure to reduce new assets needs?	**HIGH DEGREE** • This design concept will optimize the use of the existing facilities including the existing forcemain and outfall and eliminate their twinning. • This design concept will also limit the expansion of the Janet Avenue PS.	**HIGH DEGREE** • This design concept will optimize the use of the existing facilities including the existing forcemain and outfall and eliminate their twinning. • This design concept will also limit the expansion of the Janet Avenue PS.

Evaluation Criteria	Alternative 3A: Offline Storage Tank At Janet Avenue PS	Alternative 3B: Inline Or Offline Gravity Pipe Upstream Of Janet Avenue PS
OVERALL TECHNICAL RATING Based on all above technical criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Moderate constructability impact, O&M complexity, and adaptability to existing infrastructure. • High redundancy and high degree of maximizing existing infrastructure. • Low resilience to climate change.	• Moderate constructability impact, O&M complexity, and adaptability to existing infrastructure. • High redundancy and high degree of maximizing existing infrastructure. • Low resilience to climate change.
OVERALL TECHNICAL SUMMARY	Both Alternatives A and B rank similarly in the overall technical evaluation. They key differentiator is that the big pipe construction will prevent the existing approach road for the Janet Avenue PS to be utilized for day-to-day operations. An alternate temporary access will be needed for access during construction. For this reason, the tank option is preferred over the pipe option.	
ENVIRONMENTAL		
G. AQUATIC VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Streams and rivers. ○ Local aquatic species and habitat. ○ Environmentally sensitive areas, aquatic species at risk, and locally significant aquatic species.	**LOW IMPACT** • Increase in capacity and flow attenuation will have a positive impact, reducing the potential for emergency overflows into the water bodies. • The construction of the flow attenuation tank has the potential to allow sediment to flow into the nearest water body, which will be mitigated by taking control measures during construction.	**LOW IMPACT** • Increase in capacity and flow attenuation will have a positive impact, reducing the potential for emergency overflows into the water bodies. • The construction of the big pipe has the potential to allow sediment to flow into the nearest water body, depending on the method of construction, i.e., open trench versus trenchless methods. This will be mitigated by taking control measures during construction.
H. TERRESTRIAL VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Trees and vegetation. ○ Local terrestrial species and habitats. ○ Environmentally sensitive areas, species at risk, and locally significant species.	**LOW IMPACT** • Low risk expected to terrestrial vegetation and wildlife. Expansion of the PS and construction of the new tank is within the current footprint of the existing facility's property line. • Short term impacts during construction are possible, but non-damaging construction techniques would be employed to minimize impact.	**LOW IMPACT** • Low risk expected to terrestrial vegetation and wildlife. Expansion of the Pumping Station and construction of the big pipe expansion is within the current footprint of the existing facility's property line and existing easement. • Short term impacts during construction are possible, but non-damaging construction techniques would be employed to minimize impact.
I. GROUNDWATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on aquifers and groundwater resources such as groundwater quantity, groundwater recharge quality and flow regime and groundwater discharge to streams and wetlands?	**LOW IMPACT** • Low impact expected to groundwater resources.	**MODERATE IMPACT** • Low impact expected to groundwater resources.
J. SURFACE WATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on adjacent surface water resources (e.g., Humber River) and related biological communities?	**LOW IMPACT** • Due to excavation during construction, there is potential for silt and sediment finding its way into the nearby water course. Appropriate silt and sediment control measures will be taken during construction to minimize impact. • Minimum impact is expected during operation due to redundancy built into the system, which will minimize the potential for emergency overflows.	**LOW IMPACT** • Due to excavation during construction, there is potential for silt and sediment finding its way into the nearby water course. Appropriate silt and sediment control measures will be taken during construction to minimize impact. • Minimum impact is expected during operation due to redundancy built into the system, which will minimize the potential for emergency overflows.
K. GREENHOUSE GAS EMISSIONS • What will be the level of GHG emissions associated with the alternative? *(GHG emissions will be evaluation based on the alternative's energy intensity requirements.)*	**MODERATE IMPACT** • The PS expansion will result in greater energy demands due to increased power requirements. In addition, the flow attenuation tank will be equipped with a blower system which will place additional power demands.	**LOW IMPACT** • The PS expansion will result in similar energy demands as Alternative A. The big pipe will not be equipped with a blower system. Therefore, this alternative will have slightly lower greenhouse emission impact compared to Alternative 3A.

Evaluation Criteria	Alternative 3A: Offline Storage Tank At Janet Avenue PS	Alternative 3B: Inline Or Offline Gravity Pipe Upstream Of Janet Avenue PS
OVERALL ENVIRONMENTAL RATING Based on all above environmental criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• This alternative will have an overall low environmental impact except for the greenhouse emissions impact, which will be slightly greater due to the need for a blower system to supply air to the coarse bubble aeration system for the flow attenuation tank.	• This alternative will have an overall low environmental impact.
OVERALL ENVIRONMENTAL SUMMARY	Both the alternatives are expected to have a low overall environmental impact, and the environmental impact would not be a differentiating factor between the two alternatives.	
SOCIOECONOMIC		
L. SHORT-TERM COMMUNITY IMPACTS • Will the alternative have significant short-term impacts to the community during construction, including: o Noise, dust, and odour. o Local traffic.	**MODERATE IMPACT** • As the Janet Avenue Pumping Station is in a residential area, the construction of the tank and pumping station expansion will have typical construction impacts of traffic, noise and dust. These will be mitigated as much as possible by taking appropriate measures during construction.	**MODERATE IMPACT** • Alternative B will have similar construction impacts as Alternative A and will be mitigated by taking appropriate measures during construction.
M. LONG-TERM COMMUNITY IMPACT • Will the alternative have significant long-term impact to the community, including: o Benefit to community. o Impacts from facility operations. o Visual impact. o Public acceptance/resistance.	**MODERATE IMPACT** • The pumping station expansion is expected to increase the power requirements, as a result of which, a larger substation and a new, second standby power generator will be needed. • The new flow attenuation tank will be below ground and is not expected to cause adverse visual impact. However, the coarse bubble aeration system blowers will need additional footprint, building or enclosures and will create noise when in operation. • All new assets for system upgrade are within the current footprint of the existing facility. • The new flow attenuation tank will have the potential to cause adverse odours. This will be mitigated by providing odour control if required.	**LOW IMPACT** • The pumping station expansion is expected to increase the power requirements, as a result of which, a larger substation and a new, second standby power generator will be needed. • The new big pipe will not cause adverse visual impact. • All new assets for system upgrade are within the current footprint of the existing facility or the easement. • The new big pipe will have the potential to cause adverse odours. This will be mitigated by providing odour control if required.
N. ARCHAEOLOGICAL SITES • Will the alternative have significant impacts during construction and/or from ongoing operations on registered/known archaeological features?	**LOW IMPACT** • All construction activities take place on previously disturbed properties. Archeological potential not expected to be significant. • Stage 1 archeological assessment has not identified any significant risk of archaeological potential at any of the potentially expanded well facilities. As both Alternative A and B eliminate the forcemain twinning, Stage 2 archaeological assessment is not required.	**LOW IMPACT** • All construction activities take place on previously disturbed properties. Archeological potential not expected to be significant. • Stage 1 archeological assessment has not identified any significant risk of archaeological potential at any of the potentially expanded well facilities. A Stage 2 assessment is not required as forcemain twinning is eliminated under both alternatives.
O. CULTURAL/HERITAGE FEATURES • Will the alternative have significant impacts during construction and/or from ongoing operations on known cultural landscapes and built heritage features?	**LOW IMPACT** • All construction activities expected to take place on previously disturbed properties.	**LOW IMPACT** • All construction activities expected to take place on previously disturbed properties.
OVERALL SOCIOECONOMIC RATING Based on all above socioeconomic criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Alternative A provides low overall long-term community impact due to expansion of the existing Janet Avenue PS, and addition of the coarse bubble blower system.	• The coarse bubble aeration system is not needed for the big pipe alternative. However, the rest of the long-term community impacts would be very similar between the two alternatives.
OVERALL SOCIOECONOMIC SUMMARY	Alternatives A and B would both provide similar socioeconomic impact except for the coarse air blower system for the Flow Attenuation Tank, which is associated with noise impacts. Due to the occasional use anticipated for the blower system, this isn't considered to be the differentiating factor between the two alternatives	

Evaluation Criteria	Alternative 3A: Offline Storage Tank At Janet Avenue PS	Alternative 3B: Inline Or Offline Gravity Pipe Upstream Of Janet Avenue PS
FINANCIAL		
P. CAPITAL COST • What will be the relative capital cost for the alternative?	**LOW COST ALTERNATIVE** • The capital cost for the two alternatives is very similar. The pump station expansion requirement is essentially the same between Alternatives A and B. For the purpose of this comparison, the comparative cost difference between the Flow Attenuation Tank and the big pipe is negligible. As such, the capital cost is not a distinguishing factor between alternatives A and B. The reason both these alternatives are categorized under low cost is because both of these eliminate the forcemain and outfall pipe twinning.	**LOW COST ALTERNATIVE** • As discussed under Alternative A, the overall capital cost between the two alternatives is very similar.
Q. 20 YEAR LIFE-CYCLE COST • What will be the relative 20 year life-cycle cost for the alternative?	**MODERATE COST ALTERNATIVE** • The expanded Janet Avenue PS and flow attenuation tank is expected to increase the 20 year life-cycle cost primarily due to increased hydro requirement. Alternative A will need additional blower system, which is associated with additional power requirements. However, due to the use of the flow attenuation tank, only during WWF events, the life-cycle cost difference between alternatives A and B is expected to be negligible.	**MODERATE COST ALTERNATIVE** • This alternative is expected to have similar 20-year life cycle cost to Alternative A.
OVERALL FINANCIAL RATING Based on all above financial criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Expansion of the Janet Avenue PS and the new flow attenuation facility will increase the capital and 20 year life-cycle cost. However, these costs are similar between the two alternatives.	• No appreciable difference between the capital cost and the 20 year life-cycle cost between Alternatives A and B.
OVERALL FINANCIAL SUMMARY	Both the alternatives will result in moderate increase the 20 year life-cycle cost for the facility. However, these costs are expected to be very similar between the two alternatives and are not expected to be a differentiating factor between them.	
JURISDICTIONAL/REGULATORY		
R. LAND ACQUISITION COST • What will be the relative land acquisition cost for the alternative?	**LOW COST ALTERNATIVE** • No land acquisition expected.	**LOW COST ALTERNATIVE** • No land acquisition expected.
S. LAND REQUIREMENTS • What will be the level of area of non-regional land or easement required to construct the alternative?	**LOW REQUIREMENT** • No land requirement expected.	**LOW REQUIREMENT** • No land requirement expected.
T. ABILITY TO ACCOMMODATE POTENTIAL FUTURE REGULATORY CHANGES • Will the alternative have the ability to adapt to potential future changes in wastewater effluent quality requirements?	**NOT APPLICABLE** • The Janet Avenue PS and the storage is not expected to have any impact on the wastewater effluent quality requirements in the present or the future.	**NOT APPLICABLE** • The Janet Avenue PS and the storage is not expected to have any impact on the wastewater effluent quality requirements in the present or the future.
U. PERMITS AND APPROVALS • What will be the level of permits and approvals required to construct the alternative?	**LOW REQUIREMENT** • This alternative would need an amendment to the existing MECP ECA.	**LOW REQUIREMENT** • This alternative would need an amendment to the existing MECP ECA.
OVERALL JURISDICTIONAL/REGULATORY RATING Based on all above jurisdictional/regulatory criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• For Alternative 3A, it is expected that no additional land will be required to locate and construct the proposed infrastructure.	• For Alternative 3B, it is expected that no additional land will be required to locate and construct the proposed infrastructure.
OVERALL JURISDICTIONAL/REGULATORY SUMMARY	There isn't any difference in the overall jurisdictional/regulatory requirements between the two alternatives and as such, it is not the differentiating factor of selecting one over the other.	

4.3.1.5 Selection of Recommended Design Concept

The two alternative design concepts evaluated are generally on a par with each other. The following key differentiator was evident from the evaluation:

Construction of the large pipe along the approach road (easement) would necessitate an additional approach to the PS site during construction. This would result in community impacts.

Therefore, the design concept with a flow attenuation tank (Concept 3A) is recommended for further conceptual design.

4.3.2 Water Resource Recovery Facility

The existing WRRF will be expanded to provide treatment of wastewater discharged from the Janet Avenue PS. The treatment process includes multiple steps to remove pollution from the wastewater to satisfy the requirements of the ECA. The residuals generated are either landfilled, beneficially reused, or transferred to another collection system. A block flow diagram of the WRRF treatment process is shown on Figure 4-2. The treatment steps are described in Table 4-6. The technology options for each treatment process are screened and evaluated in the Technology Options Memo in Appendix A.

Table 4-6 Description of WRRF Treatment Processes

Treatment Process	Purpose	Types	Residuals / Destination	Required to Meet ECA Requirements?
Equalization	Equalize load, attenuate peak flow	Tanks, lagoons	No residuals	No
Preliminary	Remove bulk debris and grit to protect downstream equipment	Coarse Screening, Fine Screening, Grit Removal	Screenings and grit/landfill	Yes
Primary	Remove organic solids, reduce loading to secondary treatment	Sedimentation, Filtration	Primary (organic) sludge/sludge handling	No
Secondary (biological)	Remove oxygen demanding substances	Suspended growth, attached growth, hybrid suspended growth / attached growth	Biological sludge/sludge handling	Yes
Nutrient Removal	Remove phosphorus and/or nitrogen	Biological, chemical	Chemical sludge (captured in filters or clarifiers)/ sludge handling	Yes
Tertiary	Remove suspended solids	Filtration	Suspended solids/recycled to treatment	Yes
Disinfection	Eliminate pathogens	Chemical, ultraviolet	None	Yes
Effluent Outfall	Convey treated wastewater to Humber River	Channel, forcemain	None	Yes
Sludge Handling	Volume reduction, stabilization	Thickening, digestion, dewatering, storage	Sludge/landfill, beneficial reuse, regional facility	Yes

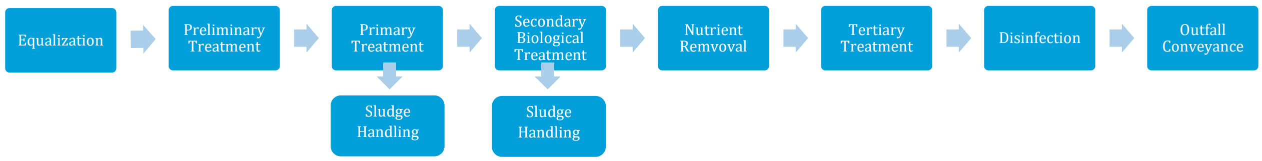

Figure 4-2 WRRF Treatment Process Block Flow Diagram

4.3.2.1 Development of Short List of Alternative Design Concepts

The alternative design concepts are based on the secondary biological treatment configuration and the WWF management design concept. Secondary biological treatment is the fundamental basis for municipal wastewater treatment. It has the largest impact on performance, operation, and cost of the WRRF. Each alternative design concept also consists of preliminary treatment, including screening and grit removal, nutrient removal, tertiary treatment, effluent disinfection, and sludge handling. Equalization and primary treatment are optional processes that can be considered for alternative design concepts to reduce the capacity of the other required processes. The secondary biological treatment technology used as the basis for screening and evaluation of WRRF alternative design concepts is extended aeration, the existing technology, based on the screening and evaluation in the Technology Options TM in Appendix A.

WWF management is the other critical factor. The WWF management design concept selected will dictate the maximum flow that requires treatment, the number of units required for each process, and the cost of the expansion project. The design basis of all treatment process components, except the aeration tanks, nutrient removal, sludge thickening, and sludge storage, is dependent on PIF or PHF (refer to Table 4-2).

4.3.2.2 Long List of WRRF Alternative Design Concepts

The long list of WRRF alternative design concepts is as follows:

- **Alternative 0 – No Flow Attenuation**: All treatment processes and the outfall sewer are expanded for the PIF or PHF without upstream flow attenuation.
- **Alternative 1 – Expand Capacity of Existing Secondary Biological Treatment Process**: Upstream flow attenuation would be provided to reduce PIF to not more than 12,528 m^3/d so that twinning of the outfall sewer is not required.
- **Alternative 1A – Enlarge Existing Aeration Tanks**: The existing aeration tanks would be enlarged to increase capacity for the design basis load.
- **Alternative 1B – Add Primary Treatment**: A primary treatment step would be added to reduce wastewater load. The existing aeration tanks would not be enlarged or reconfigured.
- **Alternative 2 – Intensify Secondary Biological Treatment System**: The existing secondary biological treatment process would be intensified by converting to a hybrid suspended growth/attached growth process to increase capacity for the design basis load without enlarging the existing aeration tanks. Upstream flow attenuation would be provided to reduce PIF to not more than 12,528 m^3/d so that twinning of the outfall sewer is not required.

- **Alternative 3 –Add Secondary Biological Treatment Process Train**: A parallel secondary biological treatment train would be added on the existing site to increase capacity for the design basis load. Upstream flow attenuation would be provided to reduce PIF to not more than 12,528 m^3/d so that twinning of the outfall sewer is not required.
- **Alternative 4 – Flow/Load Equalization**: Flow equalization would be added to reduce PIF and PHF so that the capacity of preliminary treatment, tertiary filtration, and UV disinfection would not be increased. The capacity of the secondary biological treatment process would be increased for the design basis load. Total flow equalization volume could be added in the collection system at the WRRF or split between both.

It should be noted that all of these options would also result in increased sludge generation and a need to expand the sludge handling facility. Solids thickening technologies identified and discussed in later sections can help address the level of impact and the significance of increase for the sludge handling facility.

4.3.2.2.1 Alternative 0 – No Upstream Flow Attenuation

Alternative 0 assumes no upstream flow attenuation. The existing capacity of preliminary treatment, tertiary treatment, UV disinfection, and outfall sewer would need to be essentially doubled. Furthermore, a third secondary clarifier is required for the peak hydraulic load.

The capacity of the secondary biological treatment system and sludge handling would need to be increased for the design basis load. Alternative 1A (enlarge the existing aeration tanks) design concept and gravity thickening design concepts are shown on Figure 4-3 but any of the secondary biological treatment or sludge handling conceptual design alternatives can be paired with this alternative.

The benefit of this alternative is that it would not require an offsite wastewater storage tank at the Janet Avenue PS. The main disadvantages are cost, construction impact, and performance robustness. Both the wastewater forcemain and effluent outfall sewer would need to be twinned. Furthermore, more and/or larger equipment would be required at the WRRF to treat the higher flow. Peak flows have the potential to destabilize treatment, which could result in reduced performance and the risk of effluent excursions.

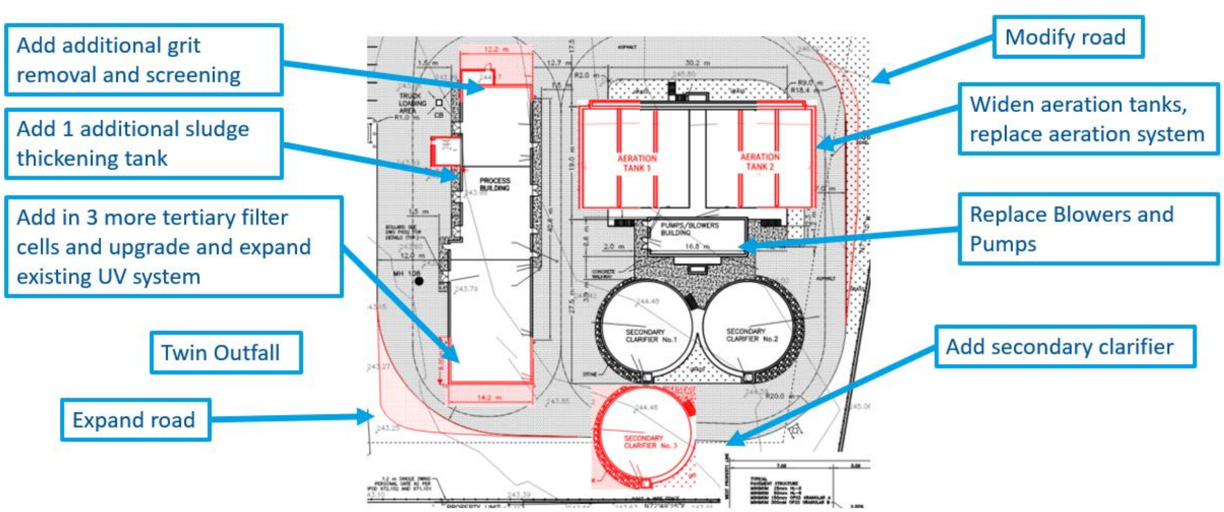

Figure 4-3 Alternative 0 - No Upstream Flow Attenuation

4.3.2.2.2 Alternative 1A – Enlarge Existing Aeration Tanks

Alternative 1A assumes flow attenuation to limit PIF to not more 12,528 m^3/d. This reduces the impact on the preliminary treatment, secondary treatment, tertiary treatment, and disinfection compared with Alternative 0. As shown on Figure 4-4, a third secondary clarifier is not needed, and the expansion of preliminary treatment, tertiary treatment, and UV disinfection is less than for Alternative 0.

The volume of each aeration tank would be increased by widening the tanks. The lanes would be widened to allow the same flow pattern. Alternatively, the flow pattern could be modified with the addition of a fourth pass. Aeration blower capacity and return activated sludge (RAS) pumping capacity would also need to be increased with replacement of the existing units. The aeration tanks would be reconfigured to provide unaerated selector zones at the upstream end of the first pass.

The benefit of this alternative is that it reduces the required twinning of the wastewater forcemain and effluent outfall sewer. Furthermore, the secondary biological treatment process and the other processes would not change limiting the need for additional O&M training and resources. The main disadvantage is the construction impact at the WRRF.

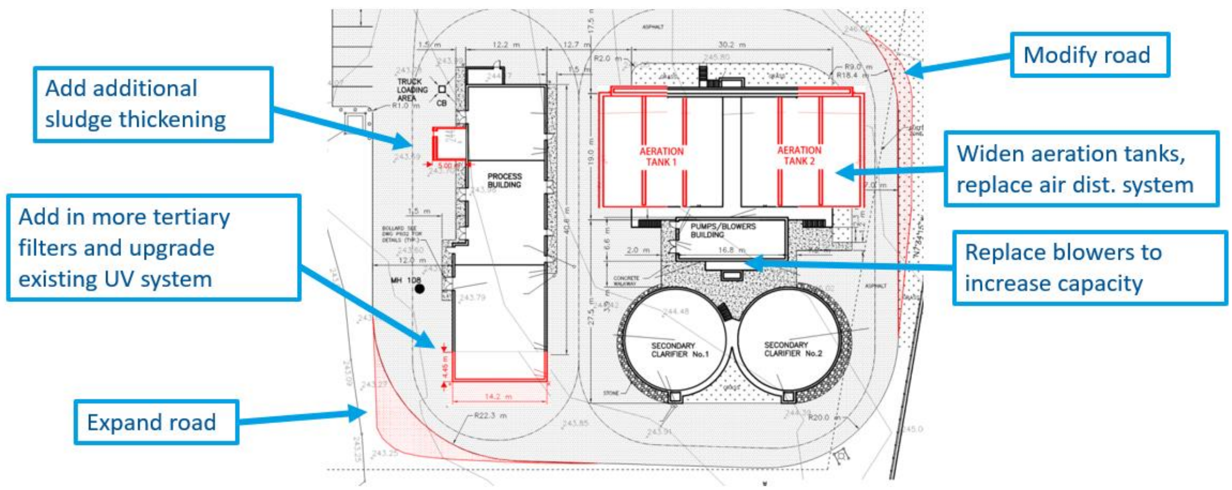

Figure 4-4 Alternative 1A - Enlarge Existing Aeration Tanks

4.3.2.2.3 Alternative 1B – Include Primary Treatment

Alternative 1B assumes flow attenuation to limit PIF to not more 12,528 m^3/d. This reduces the impact on the preliminary treatment, secondary treatment, tertiary treatment, and disinfection compared with Alternative 0. As shown on Figure 4-5, a third secondary clarifier is not needed, and the expansion of preliminary treatment, tertiary treatment, and UV disinfection is less than for Alternative 0.

Alternative 1B would add primary treatment to reduce loading on the secondary treatment process so that the existing aeration tanks are adequate for the increase in loading and the volume does not need to be increased. The aeration system would not need to be modified, but the RAS pumping capacity would need to be increased to satisfy MECP standards.

The benefit of this alternative is that it does not require structural modification or enlarging of the existing aeration tanks and the aeration system would not need to be modified. The disadvantage is that it would add a primary treatment process that would require additional O&M training and resources. Furthermore, a primary effluent pump station would be required. A new sludge stream would be generated that would increase the complexity of the sludge handling operation and increase potential for odours. No other York Region facility generates primary sludge, and this is also the case at Nobleton WRRF. For primary sedimentation, the primary sludge could be thickened and pumped to storage with WAS, or WAS could be redirected for co-thickening in the primary clarifier. For primary filtration alternatives, thickening and dewatering are integrated into the equipment. Thickened primary sludge could be stored with WAS in a liquid storage tank. Dewatered primary sludge would be hauled to landfill or directly to the Duffin Creek WPCP for incineration.

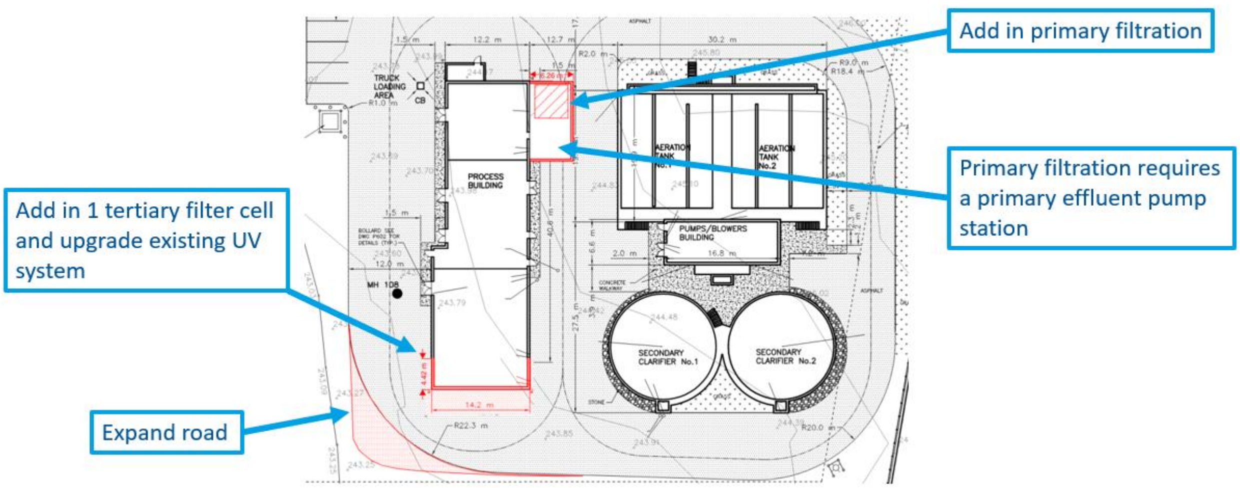

Figure 4-5 Alternative 1B - Expand Existing Treatment Process (Adding Primary Treatment)

4.3.2.2.4 Alternative 2 – Intensify Existing Biological Treatment

Alternative 2 assumes flow attenuation to limit PIF to not more 12,528 m^3/d. This reduces the impact on the preliminary treatment, secondary treatment, tertiary treatment, and disinfection compared with Alternative 0. As shown in Figure 4-6 , a third secondary clarifier is not needed, and the expansion of preliminary treatment, tertiary treatment, and UV disinfection is less than for Alternative 0.

The existing secondary biological treatment process would be intensified by converting to a hybrid suspended growth/attached growth process to increase capacity for the design basis load without enlarging the existing aeration tanks. The existing aeration system could be reused for the suspended growth component of treatment, but a new aeration system could be required for the attached growth component. RAS pumping capacity would need to be increased to satisfy MECP standards.

Intensification processes include membrane bioreactors (MBRs), membrane aerated bioreactors (MABRs), integrated fixed-film activated sludge (IFAS), and biological aerated filters (BAF). Biological treatment intensification processes are screened and evaluated in the Technology Options TM in Appendix A. Alternative 2 requires fine screens in preliminary treatment, which will require replacement of the existing duty mechanical screen with one or more fine screens. Fine screen technologies are screened and evaluated in the Technology Options TM in Appendix A. For some of these technologies, primary clarification can reduce the need for removing finer debris and having larger openings; however, Nobleton WRRF currently does not have primary clarifiers and, therefore, would likely need to install finer screens for intensification technologies. Sieve tests will need to be done to determine if the screenings are adequate for technologies continuing onward in design should the region move forward with Alternative 2.

The benefit of this alternative is that it does not require a major structural modification or enlargement of the existing aeration tanks. The media are added to the existing aeration tanks allowing a higher and more diverse inventory of microorganisms. Another benefit of this alternative is resiliency. Treatment capacity can be increased in the future or more functions added by adding more media without building more tankage. Energy intensity is also generally lower for intensification processes such that energy usage is lower compared with traditional activated sludge processes. This may not be the case for the MBR option.

The disadvantage of this alternative is that it will add more equipment and greater O&M complexity. Another disadvantage is that there are fewer of these types of systems installed in Canada and less widespread knowledge on operation. The technologies are proprietary which poses a risk of loss of support if the provider leaves the market or the product is discontinued.

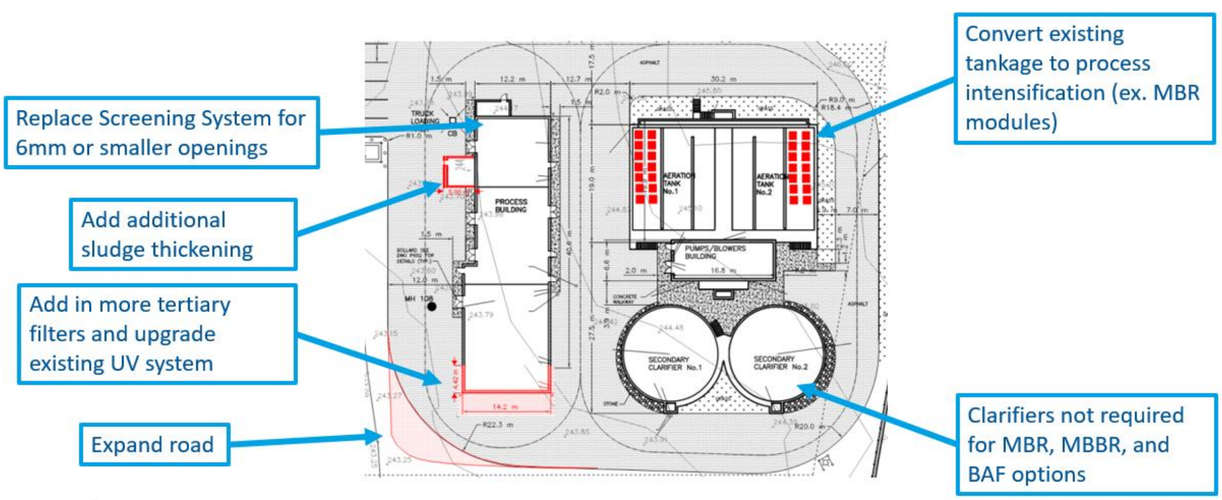

Figure 4-6 Alternative 2 - Intensify Existing Biological Treatment System

4.3.2.2.5 Alternative 3 – Build New Biological Treatment Train

Alternative 2 assumes flow attenuation to limit PIF to not more 12,528 m^3/d. This reduces the impact on the preliminary treatment, secondary treatment, tertiary treatment, and disinfection compared with Alternative 0. As shown on Figure 4-7, a third secondary clarifier is not needed, and the expansion of preliminary treatment, tertiary treatment, and UV disinfection is less than for Alternative 0.

Alternative 3 would include a new, independently operated treatment train on the west side of the property including a flow-splitting structure, preliminary treatment, secondary biological treatment and chemical addition (for phosphorus removal). An extended aeration process similar to the existing process is assumed based on the screening and evaluation of secondary treatment technologies in the Technology Options TM in Appendix A. New secondary clarifiers are assumed to eliminate the requirement to convey mixed liquor across the property and distribute it to the existing clarifiers and to simplify construction. The mixed liquor pipeline would have to cross the existing storm water infrastructure on the site and a new mixed liquor distribution structure would need to be constructed. It is assumed secondary effluents would be conveyed to the existing process building for filtration and disinfection. A headworks building including screening and grit removal is assumed to eliminate the requirement to pump up to the new treatment train. A new process building is assumed for the blowers and RAS pumps. Chemical coagulant would be fed from the existing chemical storage and feed facility to the new treatment train through small diameter chemical pipelines.

The benefit of this alternative is that the existing WRRF processes, except for tertiary filtration and UV disinfection, would not require modification, limiting construction impacts. Also, an additional level of redundancy would be provided from the addition of duplicate tanks and equipment.

The additional redundancy, however, would increase the construction cost of this alternative relative to the other alternatives. O&M complexity would also be increased from the requirement to operate two activated sludge systems, which would lead to a potential increase in O&M costs and labour requirements for the region.

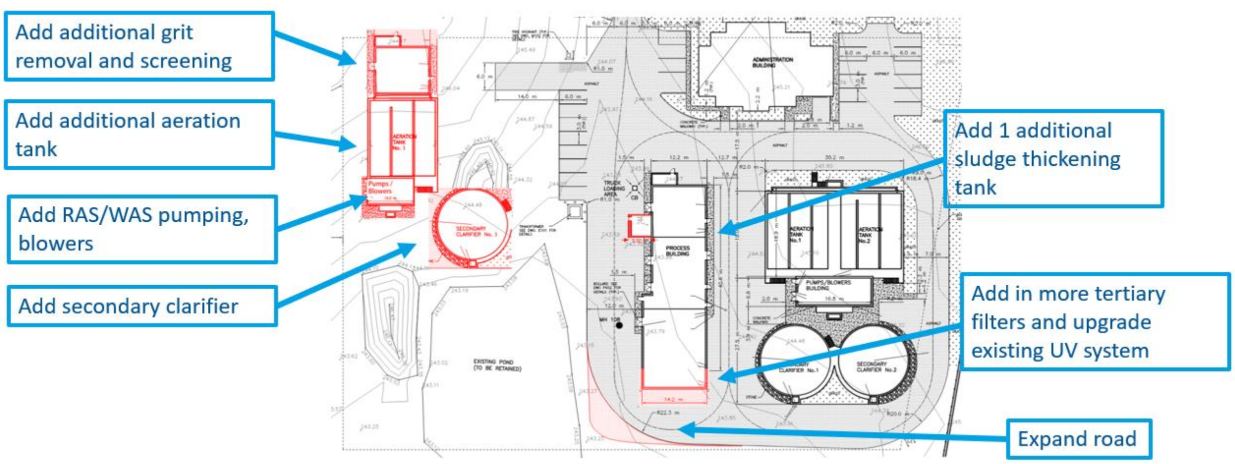

Figure 4-7 Alternative 3 - Build New Biological Treatment Train

4.3.2.2.6 Alternative 4 – Flow/Load Equalization

Alternative 4, depicted on Figure 4-8, assumes a new aerated equalization basin is installed downstream from preliminary treatment to balance the loading to treatment and attenuate peak flow. Upstream preliminary treatment and aeration are considerations to prevent accumulation of debris and organic solids. A new pump station would be provided to pump equalized flow to secondary treatment. A 2,300 m^3 is assumed to limit PHF to 9,177 m^3/d, which is the ECA rated capacity of the tertiary filters and UV disinfection processes. Preliminary treatment would be expanded for a PIF of 25,175 m^3/d. The equalization tank could be smaller, and the capacity of preliminary treatment reduced if upstream storage is constructed for the collection system. Alternatively, a larger tank or basin, could provide capability to store non-compliant effluent for returning to treatment.

The aeration tanks would need to be enlarged for the design basis load and the capacity of the aeration system and RAS pumping system would be increased. The capacity of the sludge thickening process would also be increased.

The benefit of this alternative is that it would eliminate the requirement to increase the capacity of tertiary treatment and UV disinfection. It would also increase the stability and reliability of the secondary treatment process by balancing load. On the other hand, it could reduce resiliency because peak flow treatment capacity would not be increased.

Features that could be added to this alternative to increase operational flexibility and resiliency include increasing effluent treatment capacity and providing a larger storage basin. Although this alternative does not increase effluent treatment capacity, it may be desirable to expand the capacity of tertiary filtration and effluent disinfection up to the limiting capacity of the outfall sewer, 12,528 m^3/d. York Region stakeholders have also expressed a desire for a larger basin that could allow storage of noncompliant effluent that could be recycled back into the treatment process. This feature would require a substantially larger volume.

One disadvantage of this alternative is life-cycle cost from the aeration of the aerated storage tank. Another disadvantage is that it could have negative community impact from the aesthetics of storing raw sewage. It would also increase O&M complexity from adding a new process.

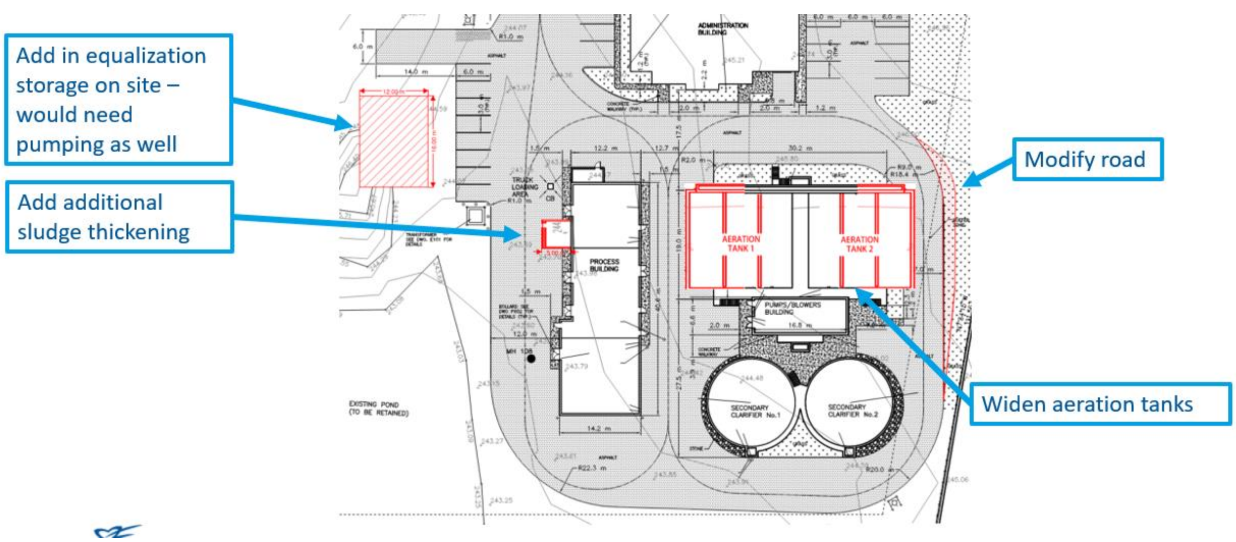

Figure 4-8 Alternative 4 - Build Equalization (Showing Alternative 1A for Biological Treatment)

4.3.2.3 Screening of Long List of WRRF Alternative Design Concepts

Screening of the alternatives was based on the information in Subsection 4.3.2.2 and feedback from York Region stakeholders from a meeting on 09 April 2021. The screening of the long list alternative WRRF design concepts is shown in Table 4-7.

Table 4-7 Screening of Long List of WRRF Alternative Design Concepts

Long List of WRRF Alternative Design Concepts	Screening Criria: Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	Notes
1. Alternative 0 – No Upstream Flow Attenuation	✓	✓	✗	✓	✗	✗	• **Eliminated** due to cost with significant construction impact both in the collection system and at the WRRF. With no upstream flow attenuation, this alternative will increase the capacity required for all processes with a PIF or PHF design basis relative to the other alternatives. Peak flow disturbances pose the risk of effluent excursions and permit violations.
2. Alternative 1A – Expand Existing Biological Treatment (Enlarge Existing Aeration Tanks)	✓	✓	✓	✓	✓	✓	• **Proceed to detailed evaluation.** Technology is compatible with existing WRRF, is a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.
3. Alternative 1B – Expand Existing Biological Treatment (Add Primary Treatment)	✗	✓	✓	✗	✓	✓	• **Eliminated** due to incompatibility with operation and hydraulics of the existing facility. It would add a new process and would require primary effluent pump station to fit into the existing hydraulic profile. It would require handling of a new sludge stream.
4. Alternative 2 – Intensify Existing Biological Treatment	✓	✓	✓	✓	✓	✓	• **Proceed to detailed evaluation.** Technology is compatible with existing WRRF and could be incorporated into existing treatment process without undue costs or construction impacts. Many of the intensification processes have a long track record.

Long List of WRRF Alternative Design Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
5. Alternative 3 – Build New Biological Treatment Train	×	✓	✓	✓	✓	×	• **Eliminated** due to construction impacts and cost. This alternative would require significant construction on the west side of the property and duplication of equipment at the facility to reduce pumping and piping. A new aeration tank and clarifier, auxiliary equipment, and a process building would need to be constructed.
6. Alternative 4 – Expand Existing Biological Treatment with Equalization Expansion	×	✓	×	✓	✓	✓	• **Eliminated** due to the need for a new process and a new pumping station. While this alternative would reduce the expansion on the existing downstream treatment processes dependent on peak flow, additional pumping would be required to get flows from the equalization basin to secondary treatment. Peak treatment capacity would not be increased which would reduce resiliency.

4.3.2.4 Short-List of WRRF Alternative Design Concepts

After the screening of the long list of WRRF alternative design concepts described in Subsection 4.3.2.3, two WRRF alternatives design concepts (shown in Table 4-8) were carried forward for evaluation.

Table 4-8 Short List of WRRF Alternative Design Concepts

Short Listed WRRF Alternative Design Concepts
A. Expand Capacity of Existing Secondary Biological Treatment Process by Enlarging Existing Aeration Tanks
B. Intensify Secondary Biological Treatment System

In addition, the screening of the long list of treatment technology alternative design concepts for each treatment process in Appendix A is summarized in Table 4-9 .

Table 4-9 Short-Listed Technology Alternatives for Each WRRF Treatment Process

WRRF Treatment Process	Short Listed Technology Alternative(S)	Notes
Coarse Screening	Climber screen	Existing technology. This option would be used with conventional secondary treatment processes
Fine Screening	Perforated plate (either belt or rotary drum)	This option would be used with secondary treatment in intensified secondary treatment processes
Grit Removal	Induced vortex	Existing technology
Primary Treatment	Primary filtration	Primary treatment applies only to alternative wastewater design concepts that include primary treatment
Secondary Treatment - Conventional	Extended aeration	Existing technology
Secondary Treatment - Intensification	MABR	
Tertiary Treatment	Two-stage sand filtration	Existing technology
Effluent Disinfection	Ultraviolet disinfection	Existing technology
Sludge Thickening	Gravity thickener Mechanical thickening No thickening	The short list is evaluated in this section

All treatment stages of the WRRF have a variety of technology considerations. These technologies make up long lists, proceed through screening, and create short lists that are used to develop the alternative WRRF design strategies that go through evaluation later in this technical memorandum. Screening of these technologies can be found in the Technology Options TM in Appendix A.

The technology short lists are summarized in the following sections as they fit into the high-level alternative design concepts presented to the Region.

4.3.2.4.1 Summary of Screening Short-List

For Alternative 2 (process intensification of the existing biological treatment process), fine screening would be required. The openings are dependent upon which process intensification alternatives are being considered. For example, MABRs would require 2 mm openings to protect the membranes, especially without primary treatment to help capture other materials that could get through preliminary treatment.

For all other alternatives, the existing screening technology, the climber/crawler bar screen is effective for the downstream processes and helps Nobleton WRRF meet effluent objectives. Therefore, replacing this equipment with a different technology is not recommended as it is not cost effective.

4.3.2.4.2 Summary of Grit Removal Short-List

The existing grit removal technology the forced vortex units is an effective means of removing grit from the influent raw wastewater. While these units are currently not in service, it is recommended to reuse and rehab the existing equipment, as this is more cost effective than replacing and adding additional grit removal in its place.

4.3.2.4.3 Summary of Primary Treatment Short-List

The following technology for primary treatment is carried over from screening in Appendix A. This technology is as follows:

- Primary Filtration – Primary filtration requires a primary effluent pumping station to assist with the additional headloss between the preliminary and secondary treatment stages and odour control technology.

4.3.2.4.4 Summary of Secondary Treatment Short-List

The following technologies for secondary treatment are carried over from screening in Appendix A:

- Extended Aeration (currently existing at Nobleton WRRF)
- Process Intensification – Membrane Aerated Bioreactor (MABR)

Extended aeration was considered for Alternative 1A (expanding the secondary treatment process) and MABR was considered for Alternative 2 (intensify the existing biological treatment process). Expanding the extended aeration process requires widening or adding an additional train to the existing aeration tanks; whereas, converting to an intensified process requires adding in framing and structural support for systems like MABR and MBR, and adding in media and auxiliary equipment for systems like IFAS and MBBR.

Both alternatives assume an anoxic selector/denitrification zone at the upstream end of the aeration tanks. For the MABR process intensification option, the membrane cassettes would be located within the anoxic zone. Anoxic zones have multiple benefits that can increase secondary biological treatment capacity, including improving settleability and restoring alkalinity destroyed by the nitrification process. Better settleability increases the capacity and efficiency of the secondary clarifiers. Restoring alkalinity through denitrification increases nitrification capacity without the need for chemicals.

4.3.2.4.5 Summary of Tertiary Treatment Short List

To meet the 12,528 m^3/d peak hour flow based on Alternatives 1A and 2, tertiary treatment will need to be expanded. To do so will require expansion of the existing process building.

The existing technology, deep bed sand filtration, is the only technology carried forward to the short-list for consideration for tertiary treatment. Three new filter cells with two modules each are proposed to be added to the existing four filter cells to increase firm capacity of tertiary filtration to 12,600 m^3/d.

This alternative should also include consideration for treatment enhancements to ensure reliable compliance with effluent limits and objectives. The System Capacity Optimization study identified increased effluent TP in2017. Effluent TP performance since 2017 should be evaluated and performance limiting factors identified. If reliable compliance with effluent limits at design conditions cannot be ensured, alternative remedies should be evaluated including, but not limited to, reducing filters hydraulic loading rate and adding additional treatment steps, e.g., rapid mixing. The evaluation should include considering the requirement for an additional pumping step to add additional treatment steps such as equalization or rapid mixing.

4.3.2.4.6 Summary of Disinfection Treatment Short List

To meet the 12,528 m^3/d peak hour flow for Alternatives 1A and 2, disinfection treatment will need to be expanded. To do so will require expansion of the existing channel and process building. Swapping to chemical disinfection will incur additional operating costs that the facility does not currently have. Due to cost considerations and easier constructability of expanding the channel rather than designing and building new contact basins, the existing technology, UV disinfection, is the only technology carried out to the short list for consideration for tertiary treatment. The existing system, Trojan 3000B, has a shallower channel than most UV disinfection systems. In this application, an upgrade to the Trojan 3000+ model provides more disinfection treatment capacity with few modifications and extension of the existing UV channel. The existing Trojan 3000B system is a low pressure, low output system, and swapping to the Trojan 3000+ system provides more output as a low-pressure system.

4.3.2.4.7 Summary of Solids Thickening Short List

Three solids thickening alternatives were short-listed:

- **Solids Thickening Alternative A- Gravity Thickening**: The gravity thickening alternative is an expansion of the current technology. WAS is thickened to 2 percent to 3 percent with decanting and transferred to the existing aerated solids storage tank. Alternatively, a new outdoor above-ground storage tank with decanting could be constructed.
- **Solids Thickening Alternative B - Mechanical Thickening**: The mechanical thickening alternative would thicken solids to 5 to 8 percent prior to transfer to solids storage. Thickened sludge concentration will be limited by the aeration and mixing alternative and the maximum allowable solids concentration for discharging to Aurora SPS. This alternative would require a new building for the mechanical thickening equipment. The mechanical thickener would operate intermittently, 2 to 4 days/week during the design period. Therefore, WAS storage and thickened WAS storage would be required. The existing aerated storage tank could be reused as the WAS storage tank and the existing sludge

thickener could remain as a short-term backup option to allow for servicing the mechanical thickener. The existing sludge thickener would be overloaded at design conditions reducing performance. Alternatively, a second thickener could be provided for redundancy.

- **Solids Thickening Alternative C – No Thickening**: The no solids thickening alternative would either result in a decreased or similar level of percent solids as the region has now. Due to increased flows, the sludge production will increase. With intermittent hauling of wet material, WAS storage would be required. The existing aerated storage tank could be reused as the WAS storage tank. This option provides the region with less demand on is O&M staff with the limited staffing hours currently set for O&M staff. While hauling costs would increase, there would be lower O&M requirements.

Alternative A and Alternative C scored better than Alternative B in the evaluation in Table 4-10. Operations has expressed a desire to discontinue sludge thickening. Therefore, the design should include provisions to allow WAS to be directly discharged to aerated sludge storage. The existing sludge storage tank is adequate to provide 3 days of WAS storage at future conditions, provided that sludge can be thicken ed to 1.0 percent by decanting. Modifications to the sludge storage tank are recommended to provide a separate decanting chamber.

Table 4-10 Evaluation of Short-Listed Sludge Thickening Alternatives

Evaluation Criteria	Alternative A: Gravity Thickening	Alternative B: Mechanical Thickening	Alternative C: No Thickening
TECHNICAL			
A. CONSTRUCTABLITY • What are the major construction challenges and risks (e.g. crossing environmentally sensitive areas, noise, odour, dust, public safety, traffic, etc.) associated with the alternative? • To what extent does it impact the community? • How much volume and complexity of construction will be associated with the alternative?	**LOW IMPACT** • A new thickening tank could be constructed adjacent to the existing thickening tank.	**MODERATE IMPACT** • A new thickening building would be constructed for the mechanical thickening equipment.	**LOW IMPACT** • This alternative would not require expansion of the existing building. The existing aerated storage tank could be used, and the existing thickening equipment could be repurposed if necessary to meet demand.
B. REDUNDANCY OF SUPPLY/SERVICE • Will the alternative be able to provide improvements in redundancy of supply or service?	**HIGH REDUNDANCY** • Two gravity thickeners would be required at design conditions. No redundancy is required or provided. The thickeners would have no mechanical components to service.	**HIGH REDUNDANCY** • One mechanical thickener would be provided. No redundancy is required or provided. Sludge can bypass the mechanical thickening equipment and go to the existing gravity thickening tank in the event that the unit must go offline for maintenance.	**HIGH REDUNDANCY** • No redundancy required. Wet material will be hauled off-site to be treated.
C. RESILIENCE TO CLIMATE CHANGE • Will the alternative have the resilience against changing climate conditions, such as changes to water supply quantity and quality (e.g., high water demands, drought)?	**HIGH RESILIENCE** • There is anticipated to be only negligible impacts due to climate change.	**HIGH RESILIENCE** • There is anticipated to be only negligible impacts due to climate change.	**HIGH RESILIENCE** • There is anticipated to be only negligible impacts due to climate change.
D. O&M REQUIREMENTS • What will be the level of additional and new O&M resources (e.g., human resources) required for the alternative? • What will be the level of complexity and maintainability of new and optimized assets?	**LOW COMPLEXITY** • This is the existing practice; no new O&M resources are required. • Simple operation with no mechanical components to fail.	**HIGH COMPLEXITY** • This would be new technology requiring additional and new O&M resources. • Modestly complex operation. Intermittent operation with chemical (polymer) feed. Multiple mechanical components,including upstream and downstream sludge storage. •	**LOW COMPLEXITY** • This would require little to no O&M resources.
E. ADAPTABILITY TO EXISTING INFRASTRUCTURE • What will be the level of modification required to the existing infrastructure to adapt to the alternative? What is the relative ease of connection to the existing alternative?	**HIGH ADAPTABILITY** • The existing process building will need to be extended for the new additional gravity thickener.	**MODERATE ADAPTABILITY** • The existing process building will need to be extended for the new mechanical thickener. • A new buffer tank with mixing would be required to store WAS to allow for intermittent operation of the mechanical thickener.	**HIGH ADAPTABILITY** • This alternative would need little to no modifications to existing infrastructure to accommodate it. • Modifications to convert the gravity thickener into an aerated storage tank might be required if there is a need to repurpose the equipment
F. MAXIMIZING USE OF EXISTING INFRASTRUCTURE • Will the alternative be able to maximize the capacity of the existing infrastructure to reduce new assets needs?	**HIGH DEGREE** • The existing gravity thickener and aerated sludge storage tank would continue to be used.	**HIGH DEGREE** • The existing gravity thickener would remain as a back-up option. The aerated sludge storage tank would be used for upstream WAS storage.	**HIGH DEGREE** • The existing aerated sludge storage tank would continue to be used. The existing gravity thickener could be abandoned, used in the event of emergencies or the need to thicken the WAS, or repurposed into another aerated sludge storage tank.

Evaluation Criteria			
OVERALL TECHNICAL RATING Based on all above technical criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Low impact constructability and low O&M complexity. • High redundancy, high resilience, high adaptability, and high degree of use of existing infrastructure.	• High complexity and moderate constructability and adaptability. • High redundancy, high resilience, and high degree of use of existing infrastructure.	• Low impact constructability and low O&M complexity. • High redundancy, high resilience, high adaptability, and high degree of use of existing infrastructure.
OVERALL TECHNICAL SUMMARY	Alternatives A and C rank highest overall due to low complexity regarding O&M requirements and high adaptability to the existing infrastructure. Alternative A and C provide technology that is familiar to the existing WRRF and plant staff and does not require additional training or operator attention since there are no mechanical components. Alternative B would also require more modifications of existing infrastructure and re-routing of piping in order to add in a mechanical thickening device whereas Alternative A would be easier in terms of connecting to the existing process. All alternatives have low construction impacts, are highly redundant and resilient, and reuse the existing infrastructure to a high degree.		
ENVIRONMENTAL			
G. AQUATIC VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Streams and rivers. ○ Local aquatic species and habitat. ○ Environmentally sensitive areas, aquatic species at risk, and locally significant aquatic species.	**LOW IMPACT** • This alternative would not impact aquatic vegetation and wildlife.	**LOW IMPACT** • This alternative would not impact aquatic vegetation and wildlife.	**LOW IMPACT** • This alternative would not impact aquatic vegetation and wildlife.
H. TERRESTRIAL VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Trees and vegetation. ○ Local terrestrial species and habitats. ○ Environmentally sensitive areas, species at risk, and locally significant species.	**LOW IMPACT** • This alternative would not impact terrestrial vegetation and wildlife.	**LOW IMPACT** • This alternative would not impact terrestrial vegetation and wildlife.	**LOW IMPACT** • This alternative would not impact terrestrial vegetation and wildlife.
I. GROUNDWATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on aquifers and groundwater resources such as groundwater quantity, groundwater recharge quality and flow regime and groundwater discharge to streams and wetlands?	**LOW IMPACT** • This alternative would not impact groundwater resources.	**LOW IMPACT** • This alternative would not impact groundwater resources.	**LOW IMPACT** • This alternative would not impact groundwater resources.
J. SURFACE WATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on adjacent surface water resources (e.g., Humber River) and related biological communities?	**LOW IMPACT** • This alternative would not impact surface water resources.	**LOW IMPACT** • This alternative would not impact surface water resources.	**LOW IMPACT** • This alternative would not impact surface water resources.
K. GREENHOUSE GAS EMISSIONS • What will be the level of greenhouse gas emissions associated with the alternative? *(Greenhouse GHG alternative's energy intensity requirements.)*	**MODERATE IMPACT** • This alternative would have higher GHG from the additional tanker truck trips to haul a higher volume of sludge.	**LOW IMPACT** • This alternative would reduce the number of tanker truck trips due to the more concentrated sludge.	**MODERATE IMPACT** • This alternative would have higher GHG from the additional tanker truck trips to haul a higher volume of sludge.
OVERALL ENVIRONMENTAL RATING Based on all above environmental criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Moderate GHG emissions impact. • Low impact for aquatic vegetation and wildlife, terrestrial vegetation and wildlife, and ground water resources.	• Low impact for aquatic vegetation and wildlife, terrestrial vegetation and wildlife, ground water resources, and greenhouse gas emissions.	• Moderate GHG emissions impact. • Low impact for aquatic vegetation and wildlife, terrestrial vegetation and wildlife, and groundwater resources.

Evaluation Criteria	Alternative A: Gravity Thickening	Alternative B: Mechanical Thickening	Alternative C: No Thickening
OVERALL ENVIRONMENTAL SUMMARY	Alternative B ranks highest overall due to low impact GHG emissions. With the higher concentrations that can be obtained from mechanical thickening equipment, Alternative B will reduce the number of tanker truck trips to and from the facility. Decreased traffic will have a lower impact on GHG emissions. Overall, all alternatives have low impact for aquatic vegetation and wildlife, terrestrial vegetation and wildlife, groundwater resources, and surface water resources.		
SOCIOECONOMIC			
L. SHORT-TERM COMMUNITY IMPACTS • Will the alternative have significant short-term impacts to the community during construction, including: ○ Noise, dust, and odour. ○ Local traffic.	**LOW IMPACT** • Limited community impact confined to the vicinity of the WRRF site. • Construction traffic should not impact local traffic because the facility is more than 1.6 kilometers from the Nobleton Urban Boundary. • Wastewater treatment services will not be interrupted. • Odour impacts from additional gravity thickening.	**LOW IMPACT** • Limited community impact limited to the vicinity of the WRRF site. • Construction traffic should not impact local traffic because the facility is more than 1.6 kilometers from the Nobleton Urban Boundary. • Wastewater treatment services will not be interrupted. • Minimal, but similar odour impacts as gravity thickening.	**LOW IMPACT** • Limited community impact confined to the vicinity of the WRRF site. • Construction traffic should not impact local traffic because the facility is more than 1.6 kilometers from the Nobleton urban boundary. • Wastewater treatment services will not be interrupted. • Odour impacts from aerated storage tank.
M. LONG-TERM COMMUNITY IMPACT • Will the alternative have significant long-term impact to the community, including: ○ Benefit to community. ○ Impacts from facility operations. ○ Visual impact. ○ Public acceptance/resistance.	**MODERATE IMPACT** • The number of tanker truck trips would increase from current levels to haul the additional sludge at the same concentrations.	**LOW IMPACT** • This alternative would require relatively fewer tanker truck trips from hauling more concentrated sludge.	**MODERATE IMPACT** • The number of tanker truck trips would increase from current levels to haul the additional sludge at the same or lower concentrations.
N. ARCHAEOLOGICAL SITES • Will the alternative have significant impacts during construction and/or from ongoing operations on registered/known archaeological features?	**LOW IMPACT** • This alternative would not impact archaeological sites.	**LOW IMPACT** • This alternative would not impact archaeological sites.	**LOW IMPACT** • This alternative would not impact archaeological sites.
O. CULTURAL/HERITAGE FEATURES • Will the alternative have significant impacts during construction and/or from ongoing operations on known cultural landscapes and built heritage features?	**LOW IMPACT** • All construction activities expected to take place on previously disturbed properties.	**LOW IMPACT** • All construction activities expected to take place on previously disturbed properties.	**LOW IMPACT** • There would be little to no construction activities for this alternative.
OVERALL SOCIOECONOMIC RATING Based on all above socioeconomic criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Low impact to traffic archaeological sites and cultural/heritage features. • Moderate long-term community impact due to increased tanker truck trips.	• Low impact to traffic archaeological sites and cultural/heritage features.	• Low impact to traffic archaeological sites and cultural/heritage features. • Moderate long-term community impact due to increased tanker truck trips.
OVERALL SOCIOECONOMIC SUMMARY	Alternative B ranked highest overall for the socioeconomic criteria. Because mechanical thickening equipment achieves higher solids concentrations than gravity thickening, there will be less tanker truck trips expected when using mechanical thickening equipment, and therefore, less of a long-term impact on the community. All construction activities are expected to take place on previously disturbed property. Traffic disruptions are expected to be minor as the site is located over 1.6 km from the current Nobleton urban boundary. The increased level of wastewater treatment services will allow for economic growth.		
FINANCIAL			
P. LAND ACQUISITION COST • What will be the relative land acquisition cost for the alternative?	**LOW IMPACT** • No land acquisition is required for this alternative.	**LOW IMPACT** • No land acquisition is required for this alternative.	**LOW IMPACT** • No land acquisition is required for this alternative.

Evaluation Criteria	Alternative A: Gravity Thickening	Alternative B: Mechanical Thickening	Alternative C: No Thickening
Q. CAPITAL COST • What will be the relative capital cost for the alternative?	**LOW COST ALTERNATIVE** • This alternative has additions of concrete tank for the new gravity thickener and add-on to the existing process building.	**HIGH COST ALTERNATIVE** • This alternative has the additions of a new mechanical thickening device and add-on to the existing process building. The mechanical thickening equipment would require WAS storage to allow for intermittent operation and thickened sludge storage prior to hauling.	**LOW COST ALTERNATIVE** • This alternative has no additions regarding constructions costs unless there is a need to repurpose the existing gravity thickener.
R. 20-YEAR LIFECYCLE COST • What will be the relative 20year life-cycle cost for the alternative?	**MODERATE COST ALTERNATIVE** • This alternative is expected to be have higher operating costs than the current operation due to increased sludge production as Nobleton grows.	**LOW COST ALTERNATIVE** • This alternative would expect to have lower operating costs because of higher concentrations of solids being hauled to incineration. This means fewer tanker truck trips and reduced hauling costs.	**V COST ALTERNATIVE** • This alternative is expected to have higher operating costs than the current operation due to increased sludge production as Nobleton grows.
OVERALL FINANCIAL RATING Based on all above financial criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Relatively low capital and moderate 20 year life-cycle costs. • No land acquisition is required.	• High capital cost. • Low operating cost for hauling.	• Relatively low capital and moderate 20year life-cycle costs. • No land acquisition is required.
OVERALL FINANCIAL SUMMARY	Neither alternative is superior to the other for financial criteria. While Alternative A and C do have a lower upfront anticipated capital cost, Alternative B is expected to have a lower 20-year lifecycle cost. The equipment for mechanical thickening will be more expensive than the gravity thickening equipment or no thickening option, but with the higher performance achieved by the mechanical thickening equipment, there would be fewertruck trips to haul solids and therefore, operating costs would be lower for Alternative B.		
JURISDICTIONAL/REGULATORY			
S. LAND REQUIREMENTS • What will be the level of area of non-regional land or easement required to construct the alternative?	**LOW REQUIREMENT** • All construction activities are expected to be on property already owned by the Region or within existing easements.	**LOW REQUIREMENT** • All construction activities are expected to be on property already owned by the Region or within existing easements.	**LOW REQUIREMENT** • All construction activities, if any, are expected to be on property already owned by the Region or within existing easements.
T. ABILITY TO ACCOMMODATE POTENTIAL FUTURE REGULATORY CHANGES • Will the alternative have the ability to adapt to potential future changes in wastewater effluent quality requirements?	**HIGH ADAPTABILITY** • Potential future regulatory options are not anticipated to impact sludge thickening.	**HIGH ADAPTABILITY** • Potential future regulatory options are not anticipated to impact sludge thickening.	**H ADAPTABILITY** • Potential future regulatory options are not anticipated to impact sludge thickening. •
U. PERMITS AND APPROVALS • What will be the level of permits and approvals required to construct the alternative?	**LOW REQUIREMENT** • This alternative would not require an amendment to the ECA permit.	**LOW REQUIREMENT** • This alternative would require approval to allow discharge of thickened sludge to the Aurora SPS. • This alternative could require a minor amendment to the ECA permit terms.	**LOW REQUIREMENT** • This alternative would not require an amendment to the ECA permit.
OVERALL JURISDICTIONAL/REGULATORY RATING Based on all above jurisdictional/regulatory criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Low requirement for land and permits and approvals • High adaptability	• Low requirement for land and permits and approvals • High adaptability	• Low requirement for land and permits and approvals. • Moderate adaptability.
OVERALL JURISDICTIONAL/REGULATORY SUMMARY	None of the alternatives is superior to the others. All construction for both alternatives is expected to be on the property already owned by the region or within the existing easement. All alternatives are proven technologies and would require minor or no amendments to the ECA permit terms. Both alternatives are also not anticipated to have impacts from potential future regulatory options		

4.3.2.5 Summary of Alternative WRRF Design Strategies

Based on the short lists in Appendix A and Subsection 4.3.2.4, the alternative WRRF design strategies are shown in Table 4-11. Each design strategy consists of a set of treatment processes. The secondary biological treatment process and preliminary treatment screening process differ between the two alternatives. All other processes are the same.

Table 4-11 Alternative WRRF Design Strategies

Treatment Process	Alternative WRRF Design Strategy A	Alternative WRRF Design Strategy B
Equalization (at WRRF)	None	None
Preliminary Treatment - Screening	Coarse screening – climber screen	Fine screening – perforated plate
Grit Removal	Vortex grit tanks	Vortex grit tanks
Primary Treatment	No primary treatment	No primary treatment
Secondary Treatment	Extended aeration – widen existing aeration tanks	Process intensification – Add MABR modules to the existing first pass
Tertiary Treatment	Two-stage sand filtration	Two-stage sand filtration
Disinfection	UV disinfection	UV disinfection
Solids Thickening	Gravity thickening/mechanical thickening/no thickening	Gravity thickening/mechanical thickening/no thickening
Outfall	Reuse existing outfall	Reuse existing outfall

4.3.2.6 Evaluation of Alternative WRRF Design Strategies

A detailed evaluation of the design strategy short lists is presented in this section.

Table 4-12 Short Listed Alternative Wastewater Servicing Design Concepts - Detailed Evaluation

Evaluation Criteria	Alternative A: Enlarge Existing Aeration Tanks	Alternative B: Process Intensification (MABR)
TECHNICAL		
A. CONSTRUCTABLITY • What are the major construction challenges and risks (e.g. crossing environmentally sensitive areas, noise, odour, dust, public safety, traffic, etc.) associated with the alternative? • To what extent does it impact the community? • How much volume and complexity of construction will be associated with the alternative?	**MODERATE IMPACT** • Modest excavation at aeration tanks – expand existing tanks and tie in additional volume • Currently, the WRRF uses only one of two aeration tanks – assume they'd be able to work at one tank at a time without disturbing the operation. • Assumes peak flow 12.6 million litres per day (MLD) due to attenuation upstream • Expansion of filtration and UV disinfection area of process building.	**LOW IMPACT** • No excavation at aeration tanks and limited, if any, structural modification required. • Currently, the WRRF uses only one of two aeration tanks – assume work could be done at one tank at a time without disturbing the operation. • Assumes peak flow 12.6 MLD due to attenuation upstream. • Expansion of filtration and UV disinfection area of process building. • More complicated renovation of the inlet works area of the process building to incorporate fine screens.
B. REDUNDANCY OF SUPPLY/SERVICE • Will the alternative be able to provide improvements in redundancy of supply or service?	**HIGH REDUNDANCY** • Firm capacity would be provided as required in MECP standards. • For secondary treatment, assumed conservative operational parameters (e.g., mixed liquor suspended solids (MLSS) concentration, such that each basin has spare capacity through operational modification.	**HIGH REDUNDANCY** • Firm capacity would be provided as required in MECP standards. • For secondary treatment, assumed conservative operational parameters (e.g., MLSS concentration), such that each basin has spare capacity through operational modification.
C. RESILIENCE TO CLIMATE CHANGE • Will the alternative have the resilience against changing climate conditions, such as changes to water supply quantity and quality (e.g. high water demands, drought)?	**MODERATE RESILIENCE** • This alternative does not include expanding the outfall. Higher rates of RDII than projected could require expanding upstream flow attenuation to limit peak flow through the WRRF.	**MODERATE RESILIENCE** • This alternative does not include expanding the outfall. Higher rates of RDII than projected could require expanding upstream flow attenuation to limit peak flow through the WRRF.
D. O&M REQUIREMENTS • What will be the level of additional and new O&M resources (e.g. human resources) required for the alternative? • What will be the level of complexity and maintainability of new and optimized assets?	**LOW COMPLEXITY** • This alternative is an expansion of the existing treatment system that would require minimum additional and new O&M resources.	**MODERATE COMPLEXITY** • This alternative is a hybrid attached growth/suspended growth system that would require new O&M resources to operate. • This alternative has more equipment and more complexity to operate and maintain the attached growth system.
E. ADAPTABILITY TO EXISTING INFRASTRUCTURE • What will be the level of modification required to the existing infrastructure to adapt to the alternative? What is the relative ease of connection to the existing alternative?	**HIGH ADAPTABILITY** • This alternative requires reconfiguration of the aeration tanks and system. There will be an expansion of concrete and adjustment of the piping.	**MODERATE ADAPTABILITY** • Some structural modifications may need to be made to add the frames that hold the MABR cassettes. The existing aeration system downstream from the MABR cassettes will not need to be modified • The existing coarse screens will need to be replaced with fine screens which may require reconfiguration of the screen channel.
F. MAXIMIZING USE OF EXISTING INFRASTRUCTURE • Will the alternative be able to maximize the capacity of the existing infrastructure to reduce new assets needs?	**MODERATE DEGREE** • This alternative will only require expansion of equipment and aeration tank.	**HIGH DEGREE** • This alternative will use the existing footprint of the aeration tanks and the aeration system.
OVERALL TECHNICAL RATING Based on all above technical criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Moderate climate change resilience, constructability impact, resilience, and maximizing use of existing infrastructure. • High redundancy, low complexity, and high adaptability.	• Moderate climate change resilience, complexity, and adaptability. • Low constructability impact, high redundancy, high degree of use of existing infrastructure.
OVERALL TECHNICAL SUMMARY	Neither alternative is superior to the other. Alternative B has a low constructability impact and a high degree of maximizing use of existing infrastructure. This would be achieved through intensification of the existing secondary biological treatment to a hybrid attached growth/suspended growth process. New additional concrete aeration tanks would not be required and the site plan around the tanks would not be modified. Alternative A has a moderate construction impact from excavation and installation of structural concrete to enlarge the aeration tanks. Alternative A has low complexity and a high adaptability. The current process would continue to be used and the existing types of equipment are the same, although the wastewater aeration system may need to be modified. Alternative B has a moderate complexity requiring new types of equipment to operate and maintain. The existing coarse screening system would need to be modified to a fine screening system. Both alternatives have moderate climate change resilience because wastewater capacity is limited to the capacity of the outfall sewer. Both alternatives have high redundancy.	

Evaluation Criteria	Alternative A: Enlarge Existing Aeration Tanks	Alternative B: Process Intensification (MABR)
ENVIRONMENTAL		
G. AQUATIC VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Streams and rivers. ○ Local aquatic species and habitat. ○ Environmentally sensitive areas, aquatic species at risk, and locally significant aquatic species.	**LOW IMPACT** • Proven technology to ensure that effluent quality meet requirements prior to discharge to Humber River to minimize impact.	**LOW IMPACT** • Proven technology to ensure that effluent quality meet requirements prior to discharge to Humber River to minimize impact.
H. TERRESTRIAL VEGETATION AND WILDLIFE • Will the alternative have significant impacts during construction and/or from ongoing operations on: ○ Trees and vegetation. ○ Local terrestrial species and habitats ○ Environmentally sensitive areas, species at risk, and locally significant species.	**LOW IMPACT** • Low risk expected to terrestrial vegetation and wildlife. System upgrade and expansion is within the current footprint of the existing facilities property line. • Short term impacts during construction are possible, but non-damaging construction techniques would be employed to minimize impact.	**LOW IMPACT** • Low risk expected to terrestrial vegetation and wildlife. System upgrade and expansion is within the current footprint of the existing facilities property line. • Short term impacts during construction are possible, but non-damaging construction techniques would be employed to minimize impact.
I. GROUNDWATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on aquifers and groundwater resources such as groundwater quantity, groundwater recharge quality and flow regime and groundwater discharge to streams and wetlands?	**LOW IMPACT** • Low impact expected to groundwater resources.	**MODERATE IMPACT** • Low impact expected to groundwater resources.
J. SURFACE WATER RESOURCES • Will the alternative have significant impacts during construction and/or from ongoing operations on adjacent surface water resources (e.g., Humber River) and related biological communities?	**LOW IMPACT** • Findings of assimilative capacity study would be used to determine final effluent quality to mitigate impact on the Humber River.	**LOW IMPACT** • Findings of assimilative capacity study would be used to determine final effluent quality to mitigate impact on the Humber River.
K. GREENHOUSE GAS EMISSIONS • What will be the level of GHG emissions associated with the alternative? *(GHG emissions will be evaluation based on the alternative's energy intensity requirements.)*	**MODERATE IMPACT** • Expansion of the aeration tanks will require more aeration capacity. Energy efficient blowers can be accounted for in system upgrades and expansion to reduce energy loads.	**LOW IMPACT** • MABR technology has more oxygen transfer efficiency than traditional secondary treatment processes. Less aeration energy will be required with this technology in comparison to Alternative A.
OVERALL ENVIRONMENTAL RATING Based on all above environmental criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Moderate GHG emissions impact. • Low impact for aquatic vegetation and wildlife, terrestrial vegetation and wildlife, and ground water resources.	• Low impact for aquatic vegetation and wildlife, terrestrial vegetation and wildlife, ground water resources, and greenhouse gas emissions.
OVERALL ENVIRONMENTAL SUMMARY	Alternative B ranks highest overall due to low impact GHG emissions. Energy intensity of the MABR process is lower than for extended aeration reducing energy demand, reducing or offsetting GHG emissions. Both Alternatives A and B present a minimal potential risk to Humber River, with increase in effluent discharge to the river. Furthermore, extended aeration has a relatively low energy efficiency, resulting in higher GHG emissions. Alternatives A and B are low impact with respect to terrestrial vegetation, ground water resources, and surface water resources.	
SOCIOECONOMIC		
L. SHORT-TERM COMMUNITY IMPACTS • Will the alternative have significant short-term impacts to the community during construction, including: ○ Noise, dust, and odour. ○ Local traffic.	**LOW IMPACT** • Limited community impact limited to the vicinity of the WRRF site. • Construction traffic should not impact local traffic because the facility is more than 1.6 kilometers from the Nobleton urban boundary. • Wastewater treatment services will not be interrupted.	**LOW IMPACT** • Limited community impact limited to the vicinity of the WRRF site. • Construction traffic should not impact local traffic because the facility is more than 1.6 kilometers from the Nobleton urban boundary. • Wastewater treatment services will not be interrupted.

Evaluation Criteria	Alternative A: Enlarge Existing Aeration Tanks	Alternative B: Process Intensification (MABR)
M. LONG-TERM COMMUNITY IMPACT • Will the alternative have significant long-term impact to the community, including: ○ Benefit to Community ○ Impacts from Facility Operations ○ Visual Impact ○ Public Acceptance/Resistance	**LOW IMPACT** • The new expanded facility will benefit the community by allowing economic growth. • Increase in sludge truck haulage from the WRRF should not impact local traffic. The facility is more than 1.6 kilometers from the current Nobleton urban boundary. • All new assets for system upgrade are within the current footprint of the existing facility.	**LOW IMPACT** • The new expanded facility will benefit the community by allowing economic growth. • Increase in sludge truck haulage from the WRRF should not impact local traffic. The facility is more than 1.6 kilometers from the current Nobleton urban boundary. • Increase in sludge truck haulage from the WRRF should not impact local traffic. The facility is more than 1.6 kilometers from the current Nobleton urban boundary. • All new assets for system upgrade are within the current footprint of the existing facility.
N. ARCHAEOLOGICAL SITES • Will the alternative have significant impacts during construction and/or from ongoing operations on registered/known archaeological features?	**LOW IMPACT** • All construction activities take place on previously disturbed properties. Archeological potential not expected to be significant. • Stage 1 archeological assessment has not identified any significant risk of archaeological potential at any of the potentially expanded well facilities.	**LOW IMPACT** • All construction activities take place on previously disturbed properties. Archeological potential not expected to be significant. • Stage 1 archeological assessment has not identified any significant risk of archaeological potential at any of the potentially expanded well facilities.
O. CULTURAL/HERITAGE FEATURES • Will the alternative have significant impacts during construction and/or from ongoing operations on known cultural landscapes and built heritage features?	**LOW IMPACT** • All construction activities expected to take place on previously disturbed properties.	**LOW IMPACT** • All construction activities expected to take place on previously disturbed properties.
OVERALL SOCIOECONOMIC RATING Based on all above socioeconomic criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Beneficial to economic growth. • Low impact to traffic archaeological sites and cultural/heritage features.	• Beneficial to economic growth. • Low impact to traffic archaeological sites and cultural/heritage features.
OVERALL SOCIOECONOMIC SUMMARY	Neither alternative is superior to the other and rank as low impact for all socioeconomic criteria. All construction activities are expected to take place on previously disturbed property. Traffic disruptions are expected to be minor as the site is located over 1.6 km from the current Nobleton urban boundary. The increased level of wastewater treatment services will allow for economic growth.	
FINANCIAL		
P. LAND ACQUISITION COST • What will be the relative land acquisition cost for the alternative?	**LOW IMPACT** • No land acquisition is required for this alternative.	**LOW IMPACT** • No land acquisition is required for this alternative.
Q. CAPITAL COST • What will be the relative capital cost for the alternative?	**MODERATE COST ALTERNATIVE** • Excavation and concrete work for the aeration tanks will be greater for Alternative A than Alternative B. • The cost for aeration system reconfiguration will be greater for Alternative A than Alternative B.	**LOW COST ALTERNATIVE** • Equipment costs for new screens and membrane equipment will be greater for Alternative B than Alternative A. • This alternative requires the same tertiary, disinfection, and solids thickening expansion as Alternative A. • Any concrete/excavation work will be lower for Alternative B.
R. 20 YEAR LIFE-CYCLE COST • What will be the relative 20 year life-cycle cost for the alternative?	**MODERATE COST ALTERNATIVE** • This alternative is expected to be similar to the current annual operating cost.	**LOW COST ALTERNATIVE** • This alternative would expect to have lower operating costs due to reduced energy intensity for aeration.
OVERALL FINANCIAL RATING Based on all above financial criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Relatively moderate capital and 20year life-cycle costs. • No land acquisition is required.	• Relatively low capital and 20 year life-cycle costs. • No land acquisition is required.
OVERALL FINANCIAL SUMMARY	Alternative B ranked highest overall for financial criteria. It has the lowest capital cost because the additional equipment is expected to cost less than increasing the volume of the existing aeration tanks. It has the lowest 20 year life-cycle cost for the anticipated lower energy consumption for wastewater aeration. Alternative A also has low resiliency as the capacity is set by the aeration tank volume.	

Evaluation Criteria	Alternative A: Enlarge Existing Aeration Tanks	Alternative B: Process Intensification (MABR)
JURISDICTIONAL/REGULATORY		
S. LAND REQUIREMENTS • What will be the level of area of non-regional land or easement required to construct the alternative?	**LOW REQUIREMENT** • All construction activities are expected to be on property already owned by the region or within existing easements.	**LOW REQUIREMENT** • All construction activities are expected to be on property already owned by the region or within existing easements.
T. ABILITY TO ACCOMMODATE POTENTIAL FUTURE REGULATORY CHANGES • Will the alternative have the ability to adapt to potential future changes in wastewater effluent quality requirements?	**MODERATE ADAPTABILITY** • The capacity of the extended aeration process is fixed by the volume of the aeration tanks. Aeration tank volume would need to be increased to add more functions, e.g., nitrogen removal.	**HIGH ADAPTABILITY** • This alternative has the ability to accommodate future more stringent nutrient requirements through operational modifications. Treatment capacity can be increased, or volume offset for additional functions, e.g. nitrogen removal, by addition of media without requiring major construction.
V. PERMITS AND APPROVALS • What will be the level of permits and approvals required to construct the alternative?	**LOW REQUIREMENT** • This alternative would require an amendment to the ECA permit.	**MODERATE REQUIREMENT** • This alternative would require an amendment to the ECA permit.
OVERALL JURISDICTIONAL/REGULATORY RATING Based on all above jurisdictional/regulatory criteria, what is the level of impact of the alternative, from low (most recommended) to high (least recommended) impact?	• Low requirement for land and permits and approvals. • Moderate adaptability.	• Low requirement for land and high adaptability to future regulatory changes. • Moderate requirement for permits and approvals for the MABR process due to the lack of full-scale operations.
OVERALL JURISDICTIONAL/REGULATORY SUMMARY	Neither alternative is superior to the other. Alternative B has high adaptability to future regulatory changes by adding more media to offset aeration tank volume for more functions, e.g. nitrogen removal. Alternative A would require construction of additional aeration tank volume. Alternative A has a low requirement for permits and approvals. The current compliant processes would continue to be used. Alternative A and B both will require an amended ECA and therefore are evaluated as a moderate requirement for permits and approvals..	

4.3.2.7 Selection of Recommended WRRF Conceptual Design Strategy

Based on the screening and evaluation, the WRRF conceptual design strategy consists of the following process components:

- Preliminary Treatment/Screening – Fine screens
- Secondary Biological Treatment – Process intensification with MABR
- Nutrient Removal – Chemical Phosphorus removal with Alum
- Tertiary Treatment – Two-stage sand filtration
- Disinfection – UV disinfection
- Sludge Thickening – None
- Aerated Sludge Storage

4.4 Selection of Recommended Wastewater Conceptual Design

Based on the screening and evaluation, the wastewater servicing conceptual design strategy consists of the following process components:

- Janet Avenue Pumping Station – Expand the Janet Avenue PS to a firm capacity of 12,500 m^3/d (145 L/s).
- Flow Attenuation – Provide a flow attenuation tank at the Janet Avenue PS for an operational capacity of 1,300 m^3.
- Forcemain – Twinning of existing forcemain not required.
- Effluent Outfall – Twinning of existing effluent outfall not required.

5.0 Summary and Recommendations

The key findings of this report are separated into water and wastewater servicing solutions.

Water Conceptual Design

For Well Site No. 2, the recommended solution is to do nothing other than replace the existing well pump because the existing infrastructure is suitable for the needed capacity increase. The existing well pump will require replacement to increase capacity.

For Well Site H, the detailed evaluation of the short-listed alternative water servicing solutions favored **Alternative 2** as the recommended servicing strategy because of the following considerations:

- **Technical –** Alternative 1 would require the existing Well Site No. 5 to be taken out of service for an extended period while modifications are made. This might impact the ability to meet demand under certain conditions. Temporary PVC piping could be constructed while the main line piping is expanded, but this option would have the potential to obstruct access and incur damage during construction.
- **Environmental** – Neither alternative is expected to have significant environmental impacts. However, Alternative 2 has a greater potential to impact the environment because of its larger footprint.
- **Socioeconomic** – Alternative 1 has a potential for significant short-term impacts to the community because Well Site No. 5 would be taken out of service. Long term, Alternative 2 has more potential to impact the community because of the additional buildings and equipment at the site. No other socioeconomic impacts are anticipated.
- **Financial** – Alternative 2 is likely to have higher capital and life-cycle cost requirements because of the addition of buildings and equipment not required in Alternative 1.
- **Jurisdictional** – Alternative 2 is likely to require more permits and jurisdictional interactions than Alternative 1.

Overall, because of the potential for Alternative 1 to impact the supply of drinking water to the community, it is recommended to proceed with Alternative 2.

Wastewater Conceptual Design

Wastewater Pumping, Flow Attenuation, Forcemain, and Outfall

Out of the three wastewater pumping, flow attenuation, forcemain, and outfall design concepts, two design concepts were brought forward for further evaluation. The other two were screened out. The two concepts, 3A and 3B, were generated for the flow attenuation storage option, i.e., a storage tank and a big pipe, respectively, and both of these concepts were carried forward for detailed evaluation. As an outcome of the evaluation, the tank concept was chosen as the preferred design concept, although the two concepts scored relatively evenly. This was primarily because of the requirement for a separate access road to the pumping station during construction of the pipe storage concept. This concept (3A) is described below:

- Provide flow attenuation storage at the Janet Avenue PS site for an operational volume of 1,300 m^3.
- Expand the Janet Avenue PS to a reduced capacity of 12,500 m^3/d (145 L/s). This would eliminate twinning of the forcemain and the constricted sections of the effluent outfall.

Wastewater Treatment

Out of six alternative wastewater treatment design concept solutions, four were screened out during the screening process. The following short list of alternative solutions proceeded into detailed evaluation:

- Alternative A: “Expand the existing biological treatment trains with upstream collection system flow attenuation to reduce peaking factor at the WRRF.”
- Alternative B: “Intensify the existing biological treatment trains (MABR technology) with upstream collection system flow attenuation to reduce peaking factor at the WRRF.”

The detailed evaluation of the two alternative wastewater design concept solutions favored **Alternative B: “Intensify the existing biological treatment trains (MABR technology) with upstream collection system flow attenuation to reduce peaking factor at the WRRF”** to be the recommended design concept solution under these considerations:

- **Technical** – Neither Alternative A nor B ranked over the other in the technical category. Alternative B has a low constructability impact and uses the existing infrastructure to a high degree, by converting the existing biological treatment process into a hybrid attached growth/suspended growth process. New additional concrete aeration tanks would not be required and the site plan around the tanks would not be modified. This alternative has moderate climate change resilience because wastewater capacity is limited to the capacity of the outfall sewer. This alternative is also highly redundant.
- **Environmental** – Alternative B ranked highest overall due to low impact GHG emissions. Energy intensity of the MABR process is lower than that for extended aeration. The reduction in energy demand of the system reduces or offsets GHG emissions. This alternative also presents minimal potential risk to the Humble River and low impact with respect to terrestrial vegetation and wildlife, aquatic vegetation and wildlife, groundwater resources, and surface water resources.
- **Socioeconomic** – Neither Alternative A nor B ranked over the other in the socioeconomic category. All construction activities are expected to take place on previously disturbed property. Traffic disruptions are expected to be minor as the site is located over 1.6 km from the current Nobleton urban boundary. The increased level of wastewater treatment services will allow for economic growth.
- **Financial** – Alternative B ranked highest overall for financial criteria; it has the lowest capital cost because of the additional equipment is expected to cost less than increasing the volume of the existing aeration tanks. It also has the lowest 20 year life-cycle cost for the anticipated lower energy consumption for wastewater aeration.
- **Jurisdictional/Regulatory** – Neither Alternative A nor B ranked over the other in the jurisdictional/regulatory category. Alternative B has high adaptability for future regulatory changes by adding more media to offset aeration tank volume for more functions, e.g., nitrogen removal. All construction activities are expected to be on property already owned by the region or within the existing easement.

6.0 Bibliography

Black & Veatch. (January 2019). *Class Environmental Assessment for Water and Wastewater Servicing in the Community of Nobleton – Study 1B: Wastewater System Capacity Optimization Study.* Regional Municipality of York

Black & Veatch. (June 2019). *Class Environmental Assessment for Water and Wastewater Servicing in the Community of Nobleton – Study 1A: Water System Capacity Optimization Study.* Regional Municipality of York

Black & Veatch. (June 2019). *Phase 1: Identify the Problem or Opportunity. Technical Memo No. 1.* Regional Municipality of York

Hutchinson. (May 2019). *Humber River Assimilative Capacity Study 2017-2018 (DRAFT).* Nobleton: Regional Municipality of York

TSH. (February 2007). *Nobleton Sewage Works Design Report (DRAFT).* Nobleton: Regional Municipality of York, Slokker Canada Corporation

York Region. (2016). *The Regional Municipality of York Water and Wastewater Master Plan*

York Region. (2019). *The Regional Municipality of York. Characterization and Comprehensive Review of Groundwater Supply Resources of the Regional Municipality of York (York Region Groundwater Supply Options Study). In Support of the 2021 Water and Wastewater Master Plan Update*

Nobleton Sewage Works Project *Forcemain and Effluent Sewer As-builts* (October 2012)

Nobleton Sewage Works Project *Water Pollution Control Plant As-builts* (October 2012)

Nobleton Water Supply System Upgrades Well No. 2 As-builts (December 2002)

Nobleton Well No. 5 and Township of King Sanitary Sewer and Watermain as Constructed (December 2015)

Nobleton Well 2 Operation Manual (August 2013)

Nobleton Well 5 Operation Manual (September 2015)

Appendix A. Wastewater Treatment Technology Options Memo

DRAFT

TECHNOLOGY OPTIONS TO MEET RECEIVING WATER QUALITY STUDY

Study 4

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

29 APRIL 2021

BLACK & VEATCH

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ADD	Average Day Demand
ADF	Average Day Flow (Annual)
EA	Environmental Assessment
I/I	Inflow and Infiltration
km	Kilometer
L/s	Liters per second
MECP	Ministry of Environment, Conservation and Parks
m^3/day	cubic meters per day
MDD	Max Day Demand
ML	Million Litres
MLD	million liters per day
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
RDII	Rainfall Derived Infiltration and Inflow
TM	Technical Memorandum
WWF	Wet Weather Flow
WRRF	Water Resource Recovery Facility

1.0 Introduction

Wastewater treatment consists of multiple processes in sequence to transform raw sewage into a treated effluent that satisfies all requirements of the ECA. The most critical process for achieving the desired effluent quality is the secondary biological treatment process. It is largely responsible for the quality of treated effluent discharged. Upstream processes remove debris and particulate matter through straining or sedimentation. Downstream processes remove particulate matter remaining after secondary treatment and eliminate pathogens.

The existing Nobleton WRRF consists of the following processes:

- Preliminary Treatment Screening – Coarse screens
- Preliminary Treatment Grit Removal – Induced vortex
- Secondary Biological Treatment – Extended Aeration
- Nutrient Removal – Chemical with alum
- Tertiary Treatment – Deep bed sand filtration
- Disinfection – UV disinfection
- Sludge Thickening – Gravity thickening
- Sludge Storage – Aeration sludge storage

Treated effluent is discharged to the Humber River. Residual solids are hauled to Aurora.

The existing wastewater treatment processes have performed well and produce an effluent in compliance with the requirements of the ECA. Furthermore, the equipment is functional and still within the expected service life. The main reason for the project is to service the projected population growth. Nonetheless, it is worthwhile to identify feasible alternatives to the existing technologies that will satisfy treatment requirements with the lowest overall cost.

1.1 Purpose of the Study

The purpose of this study is to screen the long list of technology alternatives for each wastewater treatment process. Screening and evaluation is performed according to the method described in Section 3 of TM3.

Each process is covered in sequence in the sections that follow. The long list of technology alternatives is described, and the alternatives are screened according to the method described in Section 3 of TM3. Technologies that pass the screening are evaluated in Section 5 of TM3.

2.0 Preliminary Treatment - Screening

The purpose of screening is to remove bulk materials from the wastewater to prevent interference with downstream equipment and to improve aesthetics of hauled residual materials.

2.1 Long List of Alternative Design Screening Technologies

Coarse screen technology is currently used at Nobleton WRRF. Fine screen technology is not used but may be required for some secondary biological treatment technologies.

2.1.1 Coarse Screening Equipment

The primary purpose of coarse screening is to remove objects and debris larger than ½ inch (12 mm) in size from the wastewater stream to protect the downstream influent pumps. Coarse screening options are largely dependent on the depth and configuration of downstream process as this dictates the depth and available space from which coarse screenings must be captured and lifted to the surface for handling and disposal. As such, shallower conveyance alternatives around 50 ft in depth or less, like the force main and gravity micro-tunnel alternatives, are better suited for conventional mechanically raked bar screens. Alternatives of greater depth, like those with a large diameter tunnel, are better suited for a deep tunnel bar screen with a specialized rake design. Each of these coarse screen technologies is described in more detail below.

Application depends on the downstream treatment processes. Coarse screens are adequate for conventional secondary biological treatment processes. Fine screens may be required for some secondary biological treatment technologies.

2.1.1.1 Climber/Crawler Bar Screens

A single 600-mm climber screen is installed in existing inlet works area of the Nobelton WRRF Process Building.

A climber/crawler bar screen is a conventional mechanically raked bar screen that uses a single mechanical raking mechanism (climber/crawler) to clean the screen. Climber/crawler bar screens for coarse screening applications can be provided with ½ to 3-inch (12 mm to 75 mm) spacing and have no mechanical components permanently located under water. In lieu of chains and a lower sprocket, these screens have wheels that move along a heavy-duty pin rack. As the rake assembly rotates around the lower end of the heavy-duty pin rack, the teeth on the raker heads engage the bar rack and collect debris as the rake assembly ascends back up the screen to the point of discharge. Once at the point of discharge, the wiper blade cleans the rake head and discharges screenings into a conveyor, compactor, or dumpster.

When compared to a multiple rake bar screen as described in the next section, a climber/crawler bar screen takes longer to clean because it only includes one rake; therefore, the travel time needs to be considered when utilizing this type of screen to ensure the screen doesn’t become blinded before the rake returns from its cleaning pass. Combined sewer overflow applications typically are more prone to a rapid influx of coarse debris (e.g. leaves) which could blind a climber/crawler screen in the time it takes for the raking mechanism to pass through an entire cleaning cycle.

Wastewater treatment applications not tied to a combined sewer system, while prone to traditional inflow and infiltration during wet weather, would likely be less susceptible to a rapid influx of coarse debris.

There are several manufacturers that offer climber/crawler bar screens including Infilco Degremont and Vulcan Industries, examples of which are shown on Figure 2-1, along with WSG & Solutions, and WTP Equipment Corporation, the supplier of the existing screen, and others.

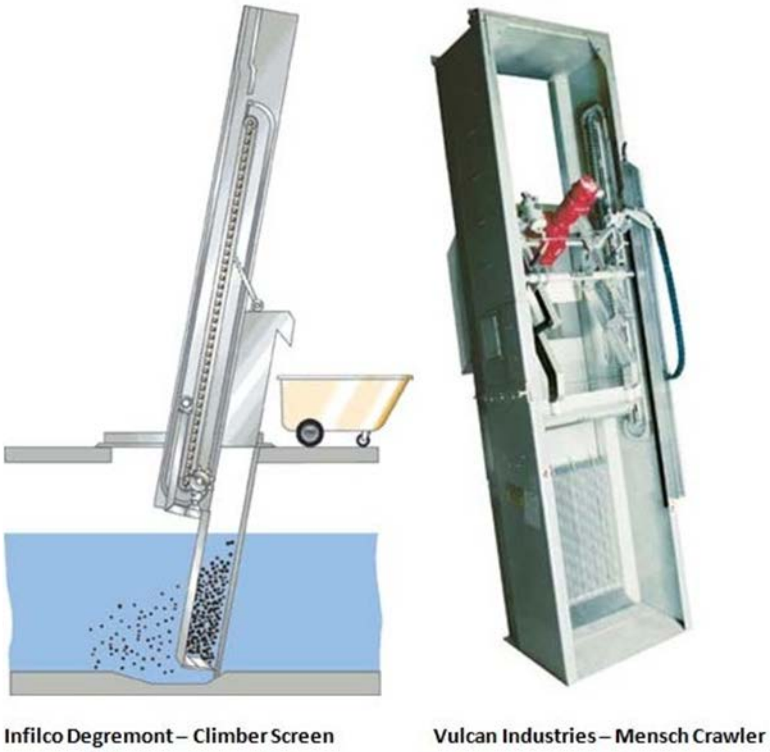
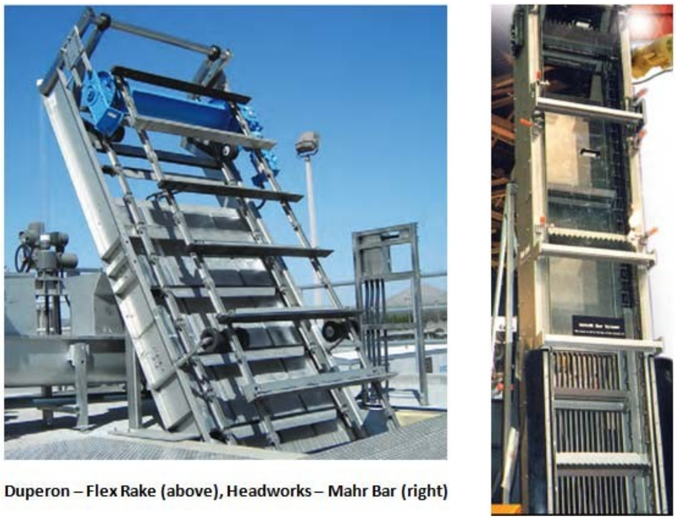


Figure 2-1 Climber/Crawler Bar Screens

2.1.1.2 Multi-Rake Bar Screens

A multi-rake bar screen is a conventional mechanically raked bar screen that uses a series of rakes to clean the screen. Multi-rake bar screens for coarse screening applications can be provided with ½ to 6-inch spacing. These types of screens are chain driven and include a lower submerged sprocket, with the exception of the Duperon Flex Rake as shown on Figure 2-2, which does not include a submerged sprocket. Multi-rake bar screens are less prone to blinding given their higher frequency of cleaning, with rakes engaging the screen as often as every 5 to 10 seconds. The rakes travel in a continuous circuit from the bottom of the channel, up the bar rack, and past the debris plate. The screenings are scraped off the rake into the discharge chute and dropped into a conveyor, compactor, or dumpster.

There are a number of manufacturers that offer this equipment including Duperon and Headworks International, which are shown on Figure 2-2, along with JWC Environmental (like those currently installed at DRPTP), Huber Technology Inc., HydroDyne, Vulcan Industries, and Wastetech.



Duperon – Flex Rake (above), Headworks – Mahr Bar (right)

Figure 2-2 Multi-Rake Bar Screens

2.1.1.3 Deep Raker Screen

A deep raker screen is a specialized mechanically raked bar screen designed for deep applications up to depths of 250 feet or greater. Deep raker screens can be provided with ½ to 6-inch bar spacing and range from 10 to 30 feet in height in single or double rack systems. The cleaning mechanism is operated by an overhead hoist and trolley system and consists of a gripper that engages with the bars and descends to the bottom of the screen while collecting debris in its jaws during the descent. When the gripper reaches the bottom of the screen, it closes and the hoist raises it back up to the trolley at grade. The trolley and gripper then travel to the discharge area where the gripper opens, releasing the debris directly into a dumpster.

There are a limited number of manufacturers that provide these types of specialized screens. Fairfield Service Company, Ovivo, and Kuenz are the known manufacturers operating in the U.S. Figure 2-3 depicts the Bosker Deep Raker screen by Ovivo.

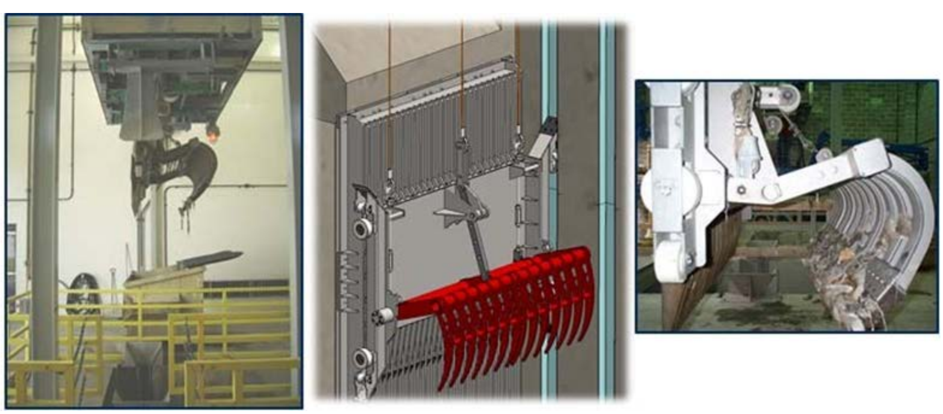


Figure 2-3 Deep Raker (Ovivo - Bosker)

2.1.1.4 Coarse Screening Equipment Advantages and Disadvantages

Table 2-1 summarizes the advantages and disadvantages of the coarse screening technologies described in this section.

Table 2-1 Advantages and Disadvantages of Coarse Screening Technologies

Technology	Status	Advantages	Disadvantages
Force Main or Micro Tunnel			
Climber/Crawler Bar Screen	Conventional: This is a mature technology that is widely used.	• No major submerged mechanical components • Rugged construction • Minimal operator attention required • Multiple manufacturers	• Requires higher overhead clearances • Can clog or be damaged by large and heavy debris • Single rake more prone to blinding during high solids loadings
Multi-Rake Bar Screen	Conventional: This is a mature technology that is widely used.	• Multiple manufacturers • Less prone to blinding during high solids loading • Rugged construction • Less headroom required • Minimal operator attention typically required • Duperon Flex Rake does not have a lower sprocket and can flex around large debris to prevent jamming	• Lower submerged sprocket (except Duperon Flex Rake) may require in-channel maintenance
Deep Micro Tunnel or Large Diameter Tunnel			
Deep Raker Screen	Conventional: This is a mature	• Material handling system included (raking, conveyance, and debris-loading) • Minimal operator attention • Rugged construction	• Limited number of manufacturers • Single gripper/rake more prone to blinding

2.1.2 Fine Screening Equipment

Fine screens are required for many of the secondary biological treatment intensification technologies. The existing coarse screens would be replaced in the event the selected Wastewater Design Concept includes biological treatment intensification.

2.1.2.1 Perforated Plate Screen

A perforated plate screen is a type of self-cleaning, in-channel screening device utilizing perforated plate media with 1/16-inch to ¼-inch spacing and no submerged bearings. All of the perforated plate screens are moving screens that trap media and transfer it up to the discharge point, with the exception of the Duperon FlexRake Perforated Fixed-Element screen. This screen operates similarly to a multi-rake bar screen, in which the actual screen is stationary and plate panels rotate to collect and transport the screenings to the discharge point. At the discharge point for perforated plate screens, the screenings are either discharged by gravity or cleaned with a brush assembly and water spray. The movement of the screen (or plate panels) can be continuous or intermittent, depending on the manufacturer. Some manufacturers have a continuous screening belt and some recommend intermittent movement of the belt or plate panels so solids are able to build up on the screen to increase capture rate. Perforated plate screens are more widely used than step screens, but typically introduce higher headloss.

There are several manufacturers of perforated plate screens including Duperon, Headworks Inc., Huber Technology, John Meunier, JWC Environmental, Parkson (shown on Figure 2-4), WSG & Solutions, WesTech, and others.

Parkson – "Aquaguard PF" Perforated Plate

Figure 2-4 Perforated Plate Screen

2.1.2.2 Step Screen

A stair/step screen is a type of self-cleaning, in-channel screening device that operates on a system of alternating fixed and movable stair-shaped screening elements with 1/32-inch to ¼-inch spacing and no submerged bearings. Debris is collected on the "steps" and forms a mat which acts as a filter to remove particles that would otherwise pass between the screens. When the headloss reaches a predetermined value, the movable steps are activated to rotate upward to lift the debris to the next highest step level. This slow progress from channel to discharge point allows the debris to shed water while suspended on the stair. Eventually the debris reaches the discharge point where it is mechanically forced off the screen by the movable screen without the need for brushes or spray systems. Screenings are then discharged to a conveyor, compactor, or dumpster. Step screens are not as widely used as perforated screens, but typically introduce lower headloss.

There are several manufacturers of step screens including John Meunier, Parkson, Vulcan, WesTech (shown on Figure 2-5), and others.

WesTech – Mono Screen Step Screen

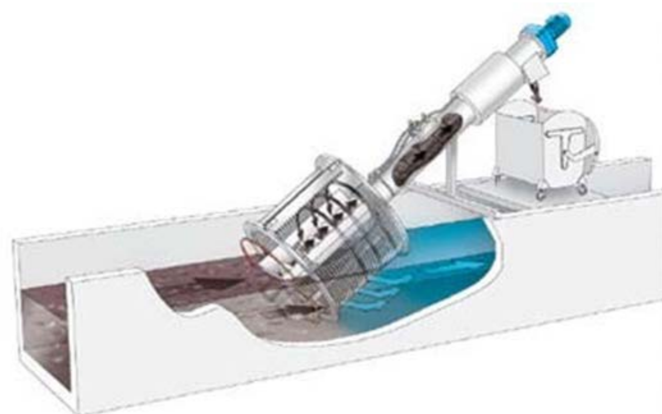
Figure 2-5 Step Screen

2.1.2.3 Rotary Drum Screens

A rotary drum screen is a type of self-cleaning fine screen in a drum arrangement with a perforated plate screen having 1/16-inch to 3/8-inch openings. Most manufacturers also offer a wedge wire type rotary drum screen with smaller openings down to 1/32-inch. Rotary drum screens are typically internally fed units similar to the JWC unit shown on Figure 2-6, where influent enters a headbox or distribution tray and then directed into the rotating drum screen. As the influent hits the rotating screen, the solids are caught inside the drum cylinder and the liquid passes through to the outside. Diverters on the drum screen move the solids along the length of the screen to the discharge end of the drum where they are discharged into a conveyor, compactor, or dumpster. Units are equipped with spray bars for cleaning.

Huber offers a unit that can be installed either directly in a channel, as shown on Figure 2-6, or in a separate tank. Wastewater influent flows into the open end of the inclined screen basket where screenings are captured and screened wastewater passes through. When the headloss reaches a predetermined value, the rake arm situated on the center axle starts to rotate. While rotating, its tines, which are extended completely through the screen bars, clean the basket to remove all the screenings from the drum. Screenings are collected into the center trough housing a screw conveyor and then transported out of the trough into an inclined pipe. As the screenings are pushed through the inclined pipe, they are dewatered and compacted prior to discharging into a conveyor or dumpster.

There are several manufacturers that supply rotary drum screens including Andritz, Huber Technology Inc., JWC Environmental, Parkson, and WesTech. The Huber and JWC Environmental units are shown on Figure 2-6.



Huber – Fine Screen ROTAMAT Ro1

JWC Environmental – IFO Internally Fed Rotary Screen

Figure 2-6 Rotary Drum Screens

2.1.2.4 Catenary Screens

A catenary screen (shown in Figure 2-7) is a variation of the traditional front-cleaned, front-return chain and rake screen. The catenary screen has the advantage of having no submerged sprockets that could be damaged or blocked by large solids that are common during high flow events. The headroom requirements for the catenary screen are also typically less than that for other screen types. The bar rake is held against the rack by weight of a heavy chain. If a large object does become lodged in the bars, the rakes pass over the objects instead of jamming. The downside is that catenary screens require a larger installation footprint compared to many other types of screens. Additionally, catenary screens are typically lighter duty compared to chain and rake screens.

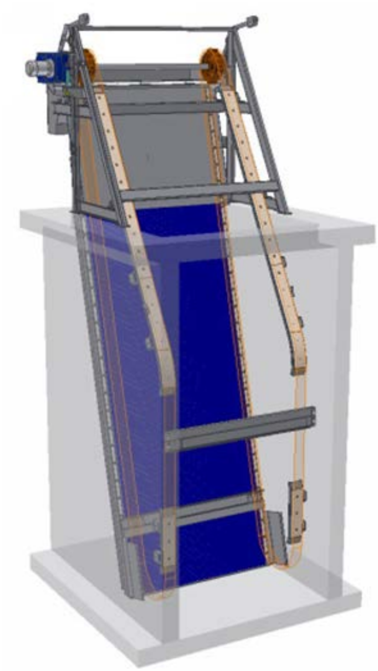


Figure 2-7 Catenary Screen

2.1.2.5 Continuous Belt Screens

The continuous belt screen (shown in Figure 2-8) is a relatively new type of screen used in the United States. Continuous belt screens are self-cleaning belts that can remove coarse and/or fine screenings. A large number of rakes are attached to the belt that clean the screen faster than single rake climber screens. The frequent cleanings also lowers the headloss through the screen. Most continuous belt screens have no major maintenance items located below the water level, which improves the ease of maintenance. The rake has multiple plastic pieces that can wear, especially in the presence of grit. Depending on the characteristics of the wastewater, these screens might not be suitable if there is a high concentration of grit. The rake may also be limited in handling large or heavy debris.

Figure 2-8 Continuous Belt Screen

2.1.2.6 Fine Screening Equipment Advantages and Disadvantages

Table 2-2 summarizes the advantages and disadvantages of the fine screening technologies.

Table 2-2 Advantages and Disadvantages of Fine Screening Technologies

Technology	Status	Advantages	Disadvantages
Combination Coarse and Fine Screens			
Climber/Crawler Bar Screen	Conventional: This is a mature technology that is widely used.	• No major submerged mechanical components • Rugged construction • Minimal operator attention required • Multiple manufacturers	• Requires higher overhead clearances • Can clog or be damaged by large and heavy debris • Single rake more prone to blinding during high solids loadings
Multi-Rake Bar Screen	Conventional: This is a mature technology that is widely used.	• Multiple manufacturers • Less prone to blinding during high solids loading • Rugged construction • Less headroom required • Minimal operator attention typically required • Duperon Flex Rake does not have a lower sprocket and can flex around large debris to prevent jamming	• Lower submerged sprocket (except Duperon Flex Rake) may require in-channel maintenance
Stand Alone Fine Screens			
Perforated Plate Screens	Conventional: This is a mature technology that is widely used.	• Major maintenance items located above water surface • Captures fine screenings and grit with opening sizes down to 1/16-inch • More widely used than step screens with a number of manufacturers • Can be installed in existing channel	• Frequent maintenance can be required for plate cleaning • More prone to blinding during high solids loading given fine solids capture • Less rugged construction than combination coarse and fine screens • Higher headloss than step screens • Compactor required due to wash water

Technology	Status	Advantages	Disadvantages
Step Screen	Conventional: This is a mature technology that is widely used.	• Less headloss than perforated plate screens • Captures fine screenings and grit with opening sizes down to 1/32-inch • Typically does not require separate wash water system • Can be installed in existing channel	• Frequent maintenance can be required for cleaning • More prone to blinding during high solids loading given fine solids capture • Less rugged construction than combination coarse and fine screens • Less widely used than perforated plate screens with a limited number of manufacturers
Rotary Drum Screen	Conventional: This is a mature technology that is widely used.	• Captures fine screenings and grit with opening sizes down to 1/32-inch • Some units provide additional dewatering and compaction • Huber version can be installed in existing channel • Lower required headroom	• Frequent maintenance can be required for cleaning • More prone to blinding during high solids loading given fine solids capture • Less rugged construction than combination coarse and fine screens
Catenary Screen	Conventional: This is a mature technology that is widely used.	• No submerged sprockets that could be blocked or damaged by large solids • Lower required headroom • Rakes pass over lodged large objects instead of jamming	• Larger installation footprint • Typically lighter duty compared to chain and rake screens
Continuous Belt Screens	Emerging: Relatively new type of screen used in the U.S.	• Self-cleaning belts that can remove coarse and/or fine screenings • Cleans belt faster than single rake climber screens • Lower headloss through the screen • No major submerged mechanical components	• Relatively new • Multiple plastic pieces that might wear, especially in the presence of grit • Rake may be limited in handling large or heavy debris

2.1.2.7 Combination Coarse and Fine Screening Options

The climber/crawler bar screen for fine screening applications are also identical to those for coarse screening applications with bar spacing of 1/4 to 5/8-inch.

The multi-rake bar screen for fine screening applications are also identical to those for coarse screening applications with bar spacing of 1/4 to 5/8-inch.

2.2 Screening of Long List of Alternative Screening Technologies

The screening of the long list alternatives of coarse and fine screening options is shown in Table 2-3 on the following page.

Table 2-3 Screening of the Long List of Alternative Screening Technologies

Long List of Alternative Screening Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
Coarse Screening Equipment							
1. Climber/Crawler Bar Screens	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. This is currently what Nobleton WRRF has installed and is still effective as a coarse screening technology. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.
2. Multi-Rake Bar Screens	✓	✓	✓	✗	✗	✓	Eliminated due to stakeholder acceptance and to reduce construction impacts.
3. Deep Raker Screen	✗	✓	✓	✓	✗	✓	Eliminated due to changes that would be required to the current channel and construction impacts.
Fine Screening Equipment							
4. Perforated Plate Screen	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.
5. Step Screen	✓	✓	✓	✗	✓	✓	Eliminated due to stakeholder acceptance.
6. Rotary Drum Screens	✗	✓	✓	✓	✗	✓	Eliminated due to incompatibility and construction impacts to the channel.
7. Catenary Screens	✓	✓	✓	✗	✓	✓	Eliminated due to stakeholder acceptance.
8. Continuous Belt Screens	✓	✓	✓	✗	✓	✓	Eliminated due to stakeholder acceptance.

2.3 Short-List of Screening Technologies

Coarse screening is recommended for conventional secondary biological treatment design concepts. Fine screening is required for secondary biological treatment intensification design concepts.

The following screening treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Coarse screening:
  - Climber/Crawler Bar Screen
- Fine screening:
  - Perforated Plate Screen

3.0 Preliminary Treatment - Grit Removal

The purpose of grit removal is to remove finer, dense solid material to reduce wear on downstream solids handling equipment. Grit removal systems can remove up to 95 percent of grit with a number of available technologies including channel type, detritor, aerated grit, forced vortex, and hydraulic vortex; each of these technologies are described below. In general, each of these technologies work on the principle of flow velocity control, whereby there is sufficient velocity to keep organic solids in suspension but is low enough to allow the denser, inorganic grit material to settle out. Once settled, the resulting grit slurry can then be pumped to a grit classifier for washing, dewatering, and disposal.

Two grit removal tanks are installed at the Nobelton WRRF.

3.1 Long List of Alternative Grit Removal Technologies

3.1.1 Channel

A grit removal channel is a configuration based on generating a desired velocity profile required to settle grit and keep organic solids in suspension. Along the top of the channel a series of grit pumps or a moving bridge with a single grit pump have a suction line that extend into the base of the sloped channel to lift the grit and directs it to a separate grit slurry channel.

Figure 3-1 shows a travelling bridge style grit and grease removal channel by Schreiber. For this unit, wastewater flows along a deep, narrow channel. Air is released into the bottom edge of the channel to create rolling water turbulence in an effort to wash the organics from the grit. The washed grit then settles to the bottom of the grit channel. A traveling bridge supported above the channel moves a grit pump the length of the channel to periodically pump the grit slurry from the channel bottom to a grit slurry trough for dewatering and disposal. The grease removal portion of this system consists of a grease channel parallel to the grit removal channel that is designed to allow grease to float to the top. The grit removal channel and the grease channel are separated by a baffle curtain wall to separate the rolling turbulence in the grit channel from the quiet pool needed for grease removal in the adjacent channel. Grit channels are not widely used and there are a limited number of manufacturers.

Figure 3-1 Grit Removal Channel

3.1.2 Detritor

A grit removal detritor is an older technology similar to channel grit removal in which flow is introduced to a velocity profile intended to keep organics in suspension and allow grit to settle to the bottom. In the case of a detritor, flow is distributed across a wide, shallow basin, similar to a clarifier, in a single direction to the outlet side. Flow enters the shallow basin/chamber via a series of inlet baffles designed to promote even flow distribution and uniform velocity across the entire width of the basin and promote grit settling. The outlet side is equipped with a sharp edged weir. As flow travels across the tank, grit settles on the bottom in a recessed, circular sump and is collected and transported into a collection hopper on the periphery of the tank by a slowly rotating scraper mechanism supported from above. From the collection hopper, a grit pump is typically used to transport settled grit slurry for dewatering and disposal.

There are a limited number of manufacturers of detritors, including Ovivo and Voltas Limited. The Ovivo J+A Crossflow unit is shown on Figure 3-2. New detritor systems are uncommon given the age of the technology, and a number of the original detritor systems have since been replaced with newer technologies; one example is for the Metropolitan Sewer District of Greater Cincinnati (MSDGC) Mill Creek WWTP, which recently replaced its detritors with vortex grit removal units.

Figure 3-2 Grit Removal Detritor

3.1.3 Aerated Grit Chamber

A grit removal aerated grit chamber is a technology in which air is introduced at the bottom of the chamber to keep organics in suspension and allows grit to settle to a sloped bottom. A dedicated blower introduces air flow into a tube which is located near the bottom of the chamber. The continuous rising air flow is intended to allow the grit to settle to the bottom of the chamber while keeping lighter organic material in suspension. Either a recessed-impeller grit pump or, more commonly, an air lift pump is used to lift settled grit slurry from the chamber bottom for dewatering and disposal. An aerated grit chamber is installed at the Eastern WRF, and as recently indicated by MCES staff, is not achieving the desired grit removal performance. This type of performance issue is not uncommon with this technology given the challenge of establishing and sustaining the right air and wastewater velocity and flow profile to effectively settle the grit. Similar to the detritor technology, MSDGC also replaced aerated grit at its Little Miami WWTP with vortex grit removal units.

There are several aerated grit chamber manufacturers including Fluidyne, Walker Process, WesTech (as shown on Figure 3-3), and others.

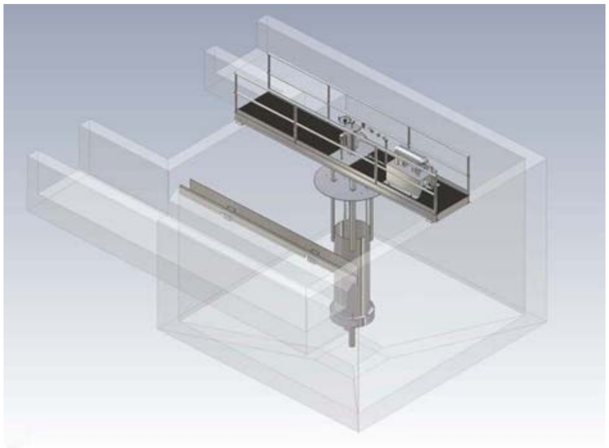


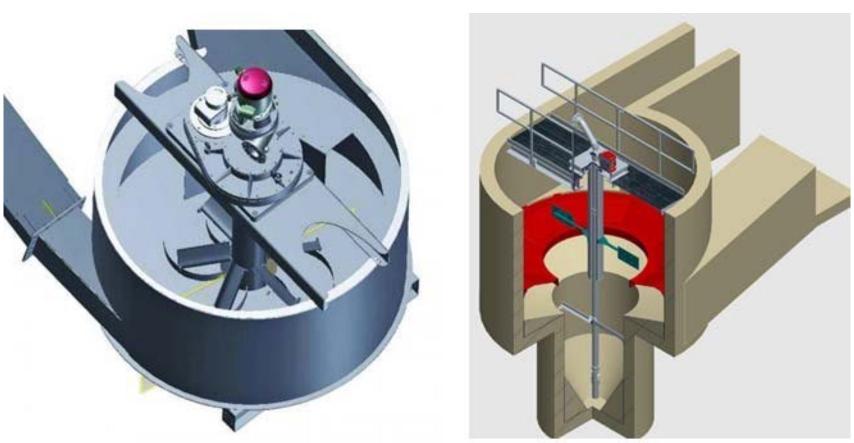
Figure 3-3 Aerated Grit Chamber

3.1.4 Vortex

3.1.4.1 Forced Vortex

Two forced vortex grit chambers manufactured by WTP Equipment Corporation are installed in the inlet works area of the Nobleton WRRF Process Building. Forced vortex grit removal chambers also work on the principle of establishing a desired velocity profile to settle grit to a collection point. Forced vortex introduces flow at a tangentially around a circular chamber with or without baffling and/or a rotating paddle to promote vortex flow. Effluent leaves the chamber tangentially in a separate channel and grit settles to the center of the chamber. Grit slurry is either lifted from a top-mounted grit pump or is pumped from a grit pump located in an adjacent dry pit to direct grit slurry to dewatering and disposal.

There are a number of vortex grit removal manufacturers including John Meunier, Ovivo, Smith and Loveless, Wastetech, WesTech , WTP Equipment Corporation, and others. The John Meunier and Smith and Loveless units are shown on Figure 3-4.



Smith and Loveless – Pista Grit Chamber

John Meunier – MECTAN V Grit Chamber

Figure 3-4 Forced Vortex

3.1.4.2 Hydraulic Vortex

A hydraulic vortex unit is similar to the more common forced vortex units, but is a proprietary technology manufactured by Hydro International, as shown on Figure 3-5. These units consist of stacked grit separator trays with no rotating parts. While these units are advertised to remove slightly finer grit than forced vortex (95 percent of grit greater than 75 microns versus 100 microns), they do introduce more headloss. Given the larger surface area provided by a stacked tray arrangement, a smaller footprint than forced vortex is required. A flow distribution header is provided to more evenly distributes influent flow tangentially over multiple conical trays and establish a vortex flow pattern where solids settled on each tray and are swept down into a center underflow collection chamber. A grit pump is installed at the underside of the unit, similar to some of the forced vortex units, to direct grit slurry to dewatering and disposal.

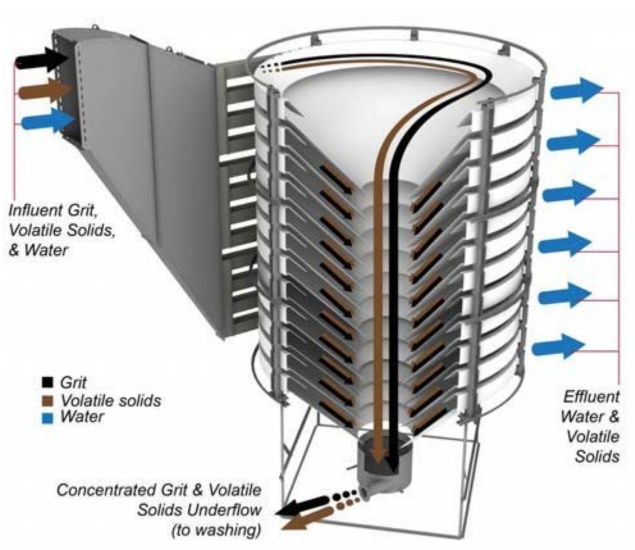


Figure 3-5 Hydraulic Vortex (Hydro International - Headcell)

3.1.5 Combined Rake and Clamshell System

Given the depths of a deep tunnel conveyance system, the above grit removal technologies are impractical for installation on the influent side of the tunnel pumps. As a means to remove coarse and some fine grit ahead of the deep tunnel pumps, a specialized rake and clamshell system can be used in this type of application to settle grit in a pit just upstream of the pump inlet header and then periodically lift it to the surface with a clamshell for disposal. Clamshell operation is typically manual and is initiated infrequently when the pumps aren't running or are running at low flow.

There are a limited number of manufacturers that provide these types of specialized rake and clamshell systems. Fairfield Service Company, Ovivo, and Kuenz are the known manufacturers operating in the U.S. Figure 3-6 depicts the Fairfield deep tunnel rake and clamshell system.

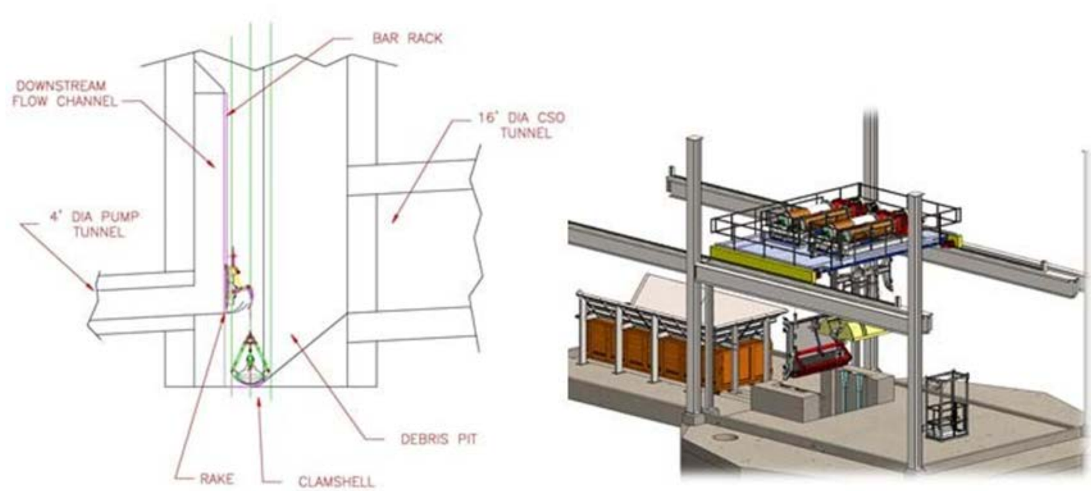


Figure 3-6 Combined Rake and Clamshell System

3.1.6 Grit Removal System Advantage and Disadvantages

Table 3-1 summarizes the advantages and disadvantages of the grit removal technologies.

Table 3-1 Advantages and Disadvantages of Grit Removal Systems

Technology	Status	Advantages	Disadvantages
Grit Removal Technologies			
Channel	Conventional: This is a mature technology that is widely used.	• No major mechanical components under water • Grease removal	• Limited system manufacturers
Detritor	Conventional: This is a mature technology that is widely used.	• Several facilities with long time detritor installations	• Lower grit removal performance when compared with newer vortex grit removal technology • Largest footprint to acoommodate wide, shallow detritor chambers • Limited system manufacturers
Aerated Grit	Conventional: This is a mature technology that is widely used.	• No moving parts below the water surface	• Requires dedicated blower system • Challenging air and wastewater flow arrangement
Forced Vortex	Conventional: This is a mature technology that is widely used.	• Widely used newer technology • Numerous system manufacturers • Designed to handle wide range of flows • Removal of ~95 percent of fine grit	• May require installations of dry pit to house grit pumps
Hydraulic Vortex	Conventional: This is a mature technology that is widely used.	• Designed to handle wide range of flows • Removal of ~95 percent of fine grit • No moving parts or external power needs	• Introduces more headloss than forced vortex units • Requires installation of dry pit to house grit pumps • Proprietary technology
Grit Removal Technologies for Deep Tunnels			
Combined Rake and Clamshell	Conventional: This is a mature technology that is widely used.	• Simple, infrequent clamshell operation • Offers coarse and some fine grit removal ahead of deep tunnel pumps for protection	• Additional grit removal system needed downstream of deep tunnel pumps if fine grit removal is desired

3.2 Screening of Long List of Alternative Grit Removal Technologies

The screening of the long list alternatives of grit removal technologies is shown on Table 3-2.

3.3 Short-List of Alternative Grit Removal Technologies

The following grit removal treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Forced Vortex

Table 3-2 Screening of the Long List of Alternative Grit Removal Technologies

Long List of Alternative Grit Removal Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Channel	✓	✓	✓	✓	✗	✓	Eliminated due to construction impacts. A new channel/basin would be required to be build and would require more footprint than existing technology.
2. Detritor	✓	✓	✗	✓	✗	✗	Eliminated due to construction impacts and performance robustness. A new channel/basin would be required to be build and would require more footprint than existing technology.
3. Aerated Grit	✓	✓	✓	✓	✗	✓	Eliminated due to construction impacts. A new channel/basin would be required to be build and would require more footprint than existing technology.
4. Forced Vortex	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. This is the current technology installed at Nobleton WRRF. While the existing technology is not currently in use, it is still an acceptable option and can be rehabbed.
5. Hydraulic Vortex	✗	✓	✓	✓	✓	✓	Eliminated due to hydraulic headloss imposed on WRRF's hydraulic profile between preliminary treatment and secondary treatment and could require pumping
6. Combined Rake and Clamshell	✗	✓	✗	✓	✓	✓	Eliminated due to performance robustness and the intermittent staffing at Nobleton WRRF. Combined Rake and Clamshell requires manual operation and would require more operator attention on preliminary treatment than currently provided.

4.0 Primary Treatment

The purpose of primary treatment is to remove settleable organic solids thereby decreasing the load on the secondary biological treatment process.

Primary treatment is not currently installed at Nobleton WRRF. It will be considered for design concepts to increase secondary biological treatment capacity.

4.1 Long List of Alternative Primary Treatment Technologies

4.1.1 Conventional Primary Sedimentation

Conventional primary treatment by sedimentation is to physically remove readily settleable solids and floating material found in the influent raw wastewater and reduce the suspended solids content. Primary sedimentation is typically the first step in further processing the wastewater following coarse/fine solids and grit removal in the preliminary treatment stage. Efficiently designed and operated treatment plants can achieve TSS removal from 50 to 70 percent and BOD removal from 25 to 40 percent in primary sedimentation tanks.

Almost all treatment plants that have primary sedimentation use mechanically cleaned sedimentation tanks that are of standard circular or rectangular design. The selection of type of sedimentation tank for a given application is typically governed by size of installation, local regulations, site conditions, stakeholder desires, and the experience and judgement of the design engineer.

4.1.2 Chemically Enhanced Primary Treatment

Chemically Enhanced Primary Treatment (CEPT) is often used to enhance settling of primary solids and subsequently increase the capacity of primary clarifiers. CEPT involves dosing chemicals, metal salts and a polymer, into the primary clarifiers to improve coagulation, flocculation and settling characteristics, thereby enhancing the removal of suspended solids and colloidal material in the primary clarifier. CEPT allows the primary clarifiers to be operated at higher overflow rates compared to conventional primary clarifiers. Importantly, through CEPT implementation the removal efficiency of total suspended solids (TSS) and biochemical oxygen demand (BOD) is enhanced by as much as 30%. Typical clarifiers, operating without CEPT achieve 50 to 60% removal of TSS and 20 to 35% removal of BOD.

CEPT also facilitates phosphorus removal. Metal ions in the dosed coagulant react with soluble ortho-phosphate present in the wastewater to form metal phosphates, which are then removed in the primary sludge. The two metal salts most commonly used in the CEPT process are ferric chloride (ferric) and aluminum sulfate (alum) although there are a number of other coagulants that are readily available on the market. The stoichiometric equations for the chemical precipitation of phosphorus using the previously highlighted metal salts are as shown in Table 4-1.

Table 4-1 Stoichiometric Equations for the Removal of TP

Metal Salt	Equation	Comments	
Ferric Chloride	$FeCl_3 + H_3PO_4 = FePO_4 + 3HCl_3$	• 1 mole of Iron III (Fe^{3+}) is theoretically required to remove 1 mole of P. • In practice however, more Fe (the molar ratio is typically in the range of 2:1 to 4:1 Fe to TP) is required due to the likelihood of competing reactions.	• For both metal salts and due to the acidic byproducts produced, alkalinity is consumed in the process.
Aluminum Sulfate	$Al_2(SO_4)_3.14H_2O + 2H_3PO_4 = 2AlPO_4 + 3H_2SO_4 + 18H_2O$	• The stoichiometric ratio for the removal of P is the same as that for Ferric. • As is the case with Ferric, the applicable dosage rate should exceed this stoichiometric ratio for effective TP removal	

4.1.3 Ballasted Flocculation

Ballasted flocculation, also known as high rate clarification, is a physical-chemical treatment process that uses continuously recycled media and a variety of additives to improve the settling properties of suspended solids through improved floc bridging. Typical ballasted flocculation removal efficiency is 85-95% TSS removal, and 50-80% BOD removal. The objective of this process is to form micro-floc particles with a specific gravity of greater than 2.0. Faster floc formation and decreased particle settling time allows the settlement process to proceed up to ten times faster than with conventional clarification, allowing treatment of flows at a significantly higher rate than possible with traditional unit processes. There are two types of ballasted flocculation systems on the market: (1) those that recycle sludge as a ballast (e.g., DensaDeg) and (2) those that add an exogenous material (e.g., ACTIFLO, CoMag). Possible ballasted flocculation technologies include:

- The Co-Mag process is a ballasted settlement technology that uses magnetite to weigh down solids and enhance solids capture in a settler at a much higher overflow rate. The ballast is recovered by shearing the floc and then separating the magnetite using a magnetic recovery drum.
- The Actiflo process combines ballasted settling using micro-sand with lamella settlers to provide high-rate settling. A hydro-cyclone separates out the micro-sand, which is re-injected into the maturation tank. This process has been used successfully for both water treatment and for wet weather excess flow treatment. It has not been used commonly for primary treatment.
- The DensaDeg process creates a floc using a coagulant and a polymer. The floc is settled by gravity using lamellas. A portion of this sludge is recycled to the flocculation step.
- The Rapisand process is similar to the Actiflo and Densadeg processes. A ballasted floc is created by mixing influent wastewater with a coagulant, polymer and microsand.

These technologies are depicted in Figure 4-1.

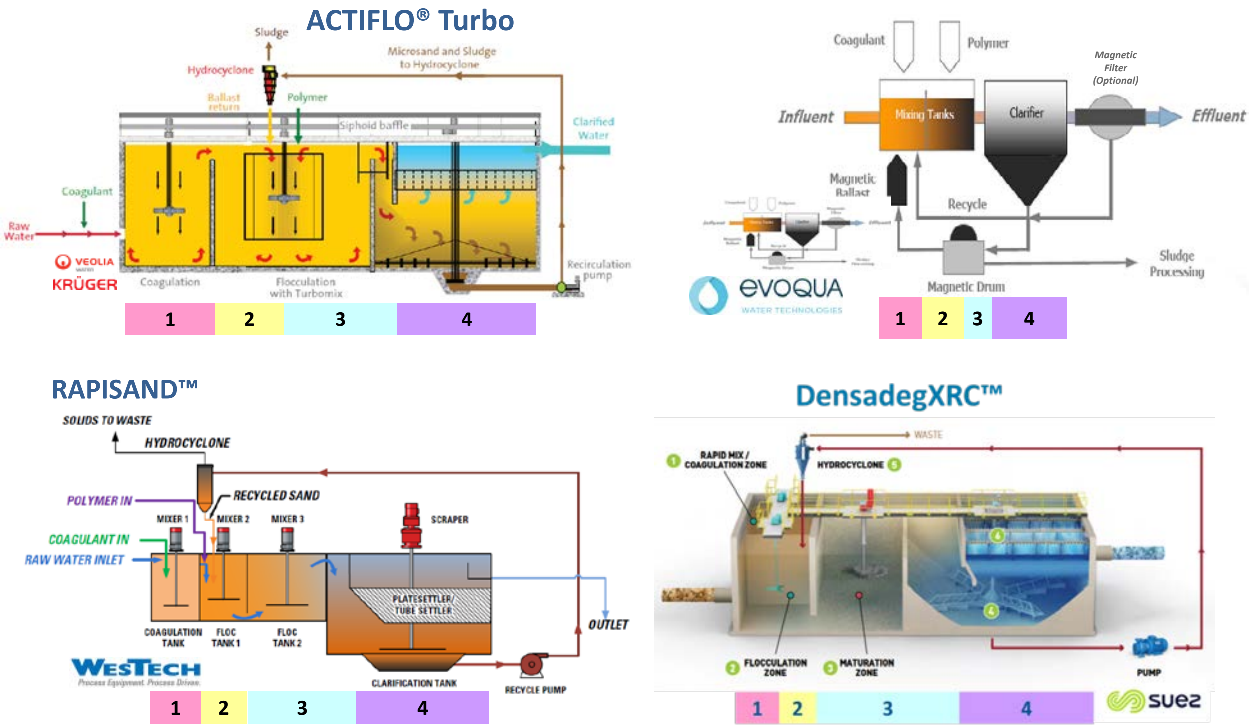


Figure 4-1 Examples of Ballasted Flocculation/Sedimentation Technologies

4.1.4 Primary Filtration Technologies

Direct filtration of raw wastewater is commonly not practiced in North America, but gaining ground with more and more full-scale installations. As an advanced primary treatment technology, primary filtration, specifically cloth media filtration, increases the removal of primary solids (approximately a 20% increase of removal efficiencies) in comparison to conventional primary sedimentation. Compressible media filters schematics are given in Figure 4-2.

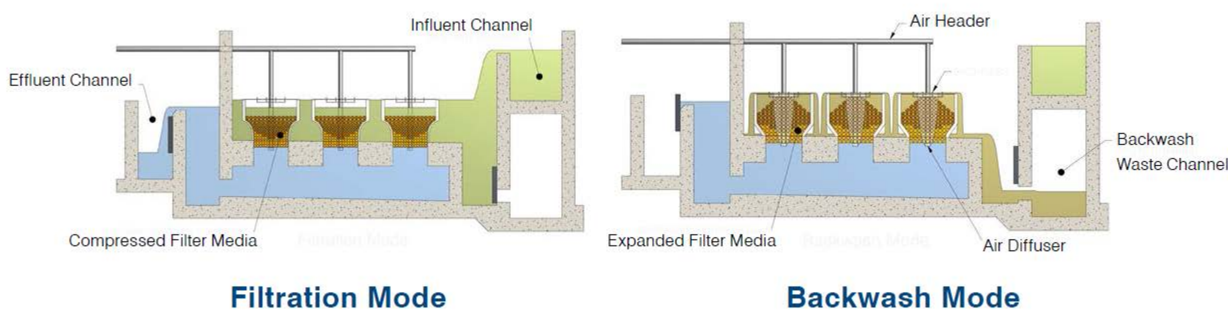


Figure 4-2 WWETCO Compressible Media Filter

Cloth Media Filtration

Cloth media filtration can be used for advanced primary treatment. There are several full-scale installations in place and soon to be installed across North America. When used in advanced primary treatment applications, cloth media filtration can achieve approximately 80% TSS removal and 50% total BOD removal. This kind of treatment application can help reduce the carbon load to the downstream secondary treatment process, which can also lead to aeration energy savings, increases in existing secondary treatment capacity or reduced basin size for the secondary treatment process. Primary filtration can also have a dramatically reduced footprint as compared to conventional primary sedimentation.

4.1.5 Primary Treatment Advantages and Disadvantages

Table 4-2 is a comparison of the primary treatment options evaluated for this project.

Table 4-2 Comparison of Primary Treatment Enhancement Technologies

Technology	Status	Advantages	Disadvantages
Conventional Primary Sedimentation	Conventional: Primary sedimentation is the standard for primary treatment in municipal wastewater facilities across North America.	• TSS and BOD removal prior to secondary treatment • Conventional removal efficiencies • Simple construction • Simple and easy operation	• Larger footprint • Increased headloss between preliminary and secondary treatment • Odour control technology required
CEPT	Conventional: Several facilities use CEPT year-round including San Diego, CA, Sydney - Australia, and Bloomington, NY.	• Improved TSS and BOD removal compared to conventional primary clarifiers (as high as 85% TSS removal, 65% BOD removal) • Consistent performance • Easy to retrofit into existing primary clarifiers • Simple and easy operation	• High chemical use resulting in high operating costs • Health and safety considerations for chemical handling • Required jar testing to determine proper water testing for correct chemicals • May remove too much carbon, requiring external source of carbon for BNR plants • Increased production of primary solids • Odour control technology required
Ballasted Flocculation	Emerging: Has been used successfully for both water treatment and for wet weather excess flow treatment. It has not been used commonly for primary treatment in North America. There are some primary treatment Actiflo installations in Europe.	• Improved TSS and BOD removal compared to conventional primary clarifiers (85-95% TSS removal, 50-80% BOD removal) • Small footprint	• May have higher construction cost than conventional primary clarifiers • Ballast may be expensive • More complex and mechanically intensive than conventional primary treatment • Proprietary technology • Increased production of primary solids • Odour control technology required
Primary Filtration (e.g., Compressed media filters, Salsnes Filters, Clear Cove, AquaPrime	Emerging: There have been several North American installations in recent years. These installations have been either used in place of primary treatment or used after primary treatment to further remove BOD and TSS before the secondary process.	• Improved TSS and BOD removal compared to conventional primary clarifiers • Can target a specific TSS removal, depending on particle size by selecting the type of media or mesh size • Smallest footprint	• More complex and mechanically intensive than conventional primary treatment. • Proprietary technology. • Headloss through filters may require additional pumping • Odour control technology required

4.2 Screening of Long List of Alternative Primary Treatment Technologies

The screening of the long list alternatives of primary treatment technologies is shown in Table 4-3.

4.3 Short-List of Alternative Primary Treatment Technologies

The following primary treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Primary Filtration

Table 4-3 Screening of the Long List of Alternative Primary Treatment Technologies

Long List of Alternative Primary Treatment Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Conventional Primary Sedimentation	✗	✓	✓	✗	✓	✗	Eliminated due to stakeholder acceptance and compatibility with the existing WRRF. Conventional primary sedimentation would also require the building of primary sedimentation basins and cost more than other alternatives. Primary equipment would require the construction of odour control technology as well.
2. CEPT	✗	✓	✓	✗	✓	✗	Eliminated due to stakeholder acceptance and compatibility with the existing WRRF. CEPT would also require the building of primary sedimentation basins and cost more than other alternatives. Primary equipment would require the construction of odour control technology as well.
3. Ballasted Flocculation	✗	✓	✓	✗	✓	✗	Eliminated due to stakeholder acceptance and compatibility with the existing WRRF. Primary equipment would require the construction of odour control technology as well.
4. Primary Filtration	✓	✓	✓	✓	✓	✓	Eliminated due to stakeholder acceptance and compatibility with the existing WRRF. Primary equipment would require the construction of odour control technology as well.

5.0 Secondary Treatment

The purpose of secondary treatment is to remove carbonaceous and nitrogenous oxygen demanding substances from wastewater.

Extended aeration is the current secondary biological treatment process at Nobleton WRRF.

5.1 Long List of Alternative Secondary Treatment Technologies

5.1.1 Conventional Nitrifying Activated Sludge Process

For a nitrifying conventional activated sludge (CAS) process at the Nobleton WRRF, primary clarifiers would be required between the headworks and the aeration tanks to reduce loadings onto the secondary treatment system. Based on the minimum month temperature of 12 °C, the solids retention time (SRT) for the CAS would be approximately 12 days to achieve the required level of nitrification. Typical design values are food to micro-organisms ratio (F/M) of 0.05-0.25 kgBOD/kgMLVSS.day, volumetric loading of 0.31-0.72 kgBOD/m3.d, MLSS concentration of 3,000-5,000 mg/L, and hydraulic retention time (HRT) of minimum 6 hours. Aeration should be 1 kg O2 per each kg of BOD in the influent, as well as an additional 4.6 kg O2 per kg of TKN influent for nitrification.

5.1.2 Extended Aeration

The extended aeration process is a modification of the CAS process which provides biological treatment for the removal of biodegradable organics under aerobic conditions. EA design solids retention time (SRT) is very high (20 to 30 d) and the hydraulic retention time (HRT) is typically 18 to 24 hours. Typical design values for extended aeration systems which provide nitrification are F/M of 0.05-0.15 kgBOD/kgMLVSS.day, organic loading of 0.17-0.24 kgBOD/m^3.day, MLSS concentration of 3,000-5,000 mg/L, and hydraulic retention time (HRT) of minimum 15 hours (if nitrification is required year-round, a longer detention time may be required). Because of the long solids retention time, aeration requirements should account for endogenous respiration, meaning that instead of 1 kg O_2 per kg BOD in the influent, 1.5 kg O_2 per daily average BOD should be considered for carbonaceous oxygen demand. If nitrification is provided, 4.6 kg O_2 per kg influent TKN is added as nitrogenous oxygen demand.

Because of the large tankage volume needed and relatively low volumetric oxygen demand rate, the aeration equipment design is used extensively for pre-engineered plants for small communities. Mechanical or diffused aeration provide the oxygen required to sustain the aerobic biological process. Mixing must be provided by aeration or mechanical means to maintain the microbial organisms in contact with the dissolved organics. The pH must also be controlled to optimize the biological process and essential nutrients must be present to facilitate biological growth and the continuation of biological degradation. Generally primary clarification is not used for EAs. Secondary clarifiers are designed at lower hydraulic loading rates than CAS clarifiers to better handle large flowrate variations. A flow equalization tank may be necessary at the WRRF prior to the EA tanks to prevent overloading of the system from inconsistent flow rates in the morning and evening.

The existing Nobleton WRRF extended aeration treatment system includes two aeration tanks, two clarifiers, and associated pumps, blowers, and air distribution equipment.

5.1.3 Sequencing Batch Reactor (SBR)

The sequencing batch reactor (SBR) is a variation of the activated sludge process. They act as a fill-and-draw type reactor system involving a single complete-mix reactor in which all steps of activated sludge processes occur. Mixed liquor remains in the reactor during all cycles and thus, eliminating the need for separate sedimentation tanks or clarifiers. For the Nobleton WRRF, at least 2 tanks are required so that one tank is in the fill mode while the other goes through react, solids settling, and effluent withdrawal. Decanting of effluent is accomplished by either fixed or floating decanter mechanisms. Based on influent flowrate and tank volume used, SBR hydraulic retention times generally range from 18 to 30 hours. An SBR goes through a number of cycles per day; a typical cycle may consist of 3-h fill, 2-h aeration, 0.5-h settle, and 0.5-h for withdrawal of supernatant. An idle step may also be included to accommodate peak flows. The aeration tank volumetric loading should not exceed 0.24 kg $BOD_5/(m^3 \cdot day)$, and design F/M ratios should be within the range of 0.05 to 0.1 kgBOD/(kgMLVSS.day).

Aeration may be provided by jet aerators or coarse/fine diffusers with submerged mixers. Dissolved oxygen (DO) should be monitored during this phase to ensure it is maintained above 2 mg/L so that nitrification can occur. For denitrification, DO level should be lowered to less than 0.5 mg/L. The treatment cycle can be adjusted to undergo aerobic, anaerobic, and anoxic conditions in order to achieve biological nutrient removal, including nitrification, denitrification, and some phosphorus removal. With SBRs, effluent BOD levels of less than 5 mg/L and NO_3-N concentrations of less than 5 mg/L are achievable. If the SBR provides denitrification, total nitrogen can reach to less than 5 mg/L. Low phosphorus limits of less than 2 mg/L can also be achieved by using a combination of biological treatment (anaerobic phosphorus absorbing organisms) and chemical addition (aluminum or iron salts) within the tank.

5.1.4 Rotating Biological Contactor

RBC is a fixed film biological treatment device in which microorganisms are grown on circular plastic disks mounted on a horizontal shaft that rotates slowly while partially immersed in wastewater. The rotating disks (known as the media) are contained in a tank or trough and rotate at between 2 to 5 revolutions per minute. The rotation helps to slough off excess solids. Commonly used plastics for the media are polyethylene, PVC and expanded polystyrene. The shaft is aligned with the flow of wastewater so that the discs rotate at right angles to the flow, with several packs usually combined to make up a treatment train. About 40% of the disc area is immersed in the wastewater. The disc system can be staged in series to obtain nearly any detention time or degree of removal required. Since the systems are staged, the culture of the later stages can be acclimated to the slowly degraded materials. Hydraulic loading to the RBCs should range between 75 to 155 $L/(m^2 \cdot d)$ of media surface area without nitrification and 30 to 80 $L/(m^2 \cdot d)$ with nitrification. Organic loading to the first stage of an RBC train should not exceed 0.03 to 0.04 kg $BOD_5/(m^2 \cdot d)$ or 0.012 to 0.02 kg $BOD_5/(m^2 \cdot d)$. Loadings in the higher end of these ranges will increase the likelihood of developing problems such as heavier than normal biofilm thickness, depletion of DO, nuisance organisms and deterioration of overall process performance. The optimum tank volume determined when treating municipal sewage of up to 300 mg/L BOD_5 is 0.042 L/m^2, which considers sewage displaced by the media and attached biomass. Based on a tank volume of 0.042 L/m^2, the detention time in each RBC stage should range between 40 to 120 minutes without nitrification and 90 to 250 minutes with nitrification.

The temperature of sewage entering any RBC should not drop below 5 °C unless there is sufficient flexibility to decrease the hydraulic loading rate. Otherwise, insulation or additional heating should be provided to the plant. Year-round operation requires that the RBC be covered to protect the biological growth from cold temperatures and the excessive loss of heat from the sewage with the resulting loss of performance.

RBCs need to be preceded by effective primary sedimentation tanks equipped with scum and grease removal devices or pretreatment devices which provide for effective removal of grit, debris and excessive oil and grease prior to the RBC units. Solids separation is an important part of the RBC process; accordingly, downstream secondary clarification is required.

5.1.5 Process Intensification Technologies

5.1.5.1 Moving Bed Bioreactor (MBBR)

MBBR is an integrated fixed film activated sludge (IFAS) or hybrid process. IFAS consists of an activated sludge system in which a material to support attached biomass growth has been added in addition to the suspended biomass growth in an activated sludge reactor. The MBBR process is similar to the IFAS process with mixed, suspended media contained within the reactor by effluent sieves, with the exception that there is no return activated sludge. The media fill volume is generally higher (up to 70 percent), and the suspended solids concentration in the flow to the secondary clarifier may be in the range of 100 to 250 mg/L versus 2,500 to 3,500 mg/L in an IFAS. Process design for MBBR can also include the suspended media in anoxic zones for fixed film biological denitrification. MBBR reactor effluent, filtration processes including granular media and membrane filtration, and dissolved air floatation can be used in lieu of gravity settling.

5.1.5.2 Biologically Active Filters (BAF)

The term biological aerated filter refers to the fact that the attached growth process is aerated to provide oxygen for BOD removal and nitrification. Biological aerated filter fall within a broader category called biological active filter (BAF). Biological active filter has the biological aerated filter design but working in anoxic conditions to provide denitrification for nitrogen removal.

Veolia is one of the vendors that provide this technology. Veolia's BIOSTYR® system is a very compact process combining fixed film biological treatment and filtration in a single unit operation with relatively high pollutant loads depending on the carbon and nitrogen requirements. BAF processes are very well suited when space is an important site constraint. During the last 25 years, more than 150 BIOSTYR® facilities have been built and operated to treat municipal wastewater around the world, thereby also demonstrating the wide-range of treatment applications in the marketplace. Figure 5-2 shows a schematic of a conventional BIOSTYR® cell.

The design and cost of BAF is impacted directly by hydraulic flow rate and flow equalizations should be considered for high hydraulic peak flows from wet weather events. Also, solids filtration may be implemented to produce a high-quality effluent.

As a case study, in 2014 a BAF unit was installed in a WWTP in New York, NY with a capacity of 94 MLD (280 MLD peak flow). This system was able to successfully reduce the effluent Total Nitrogen loading from 907 kg/day to 90.7 kg/day, and reach the tighter restriction of 4.0 mg/L TN regulated by the State Pollutant Discharge Elimination System.

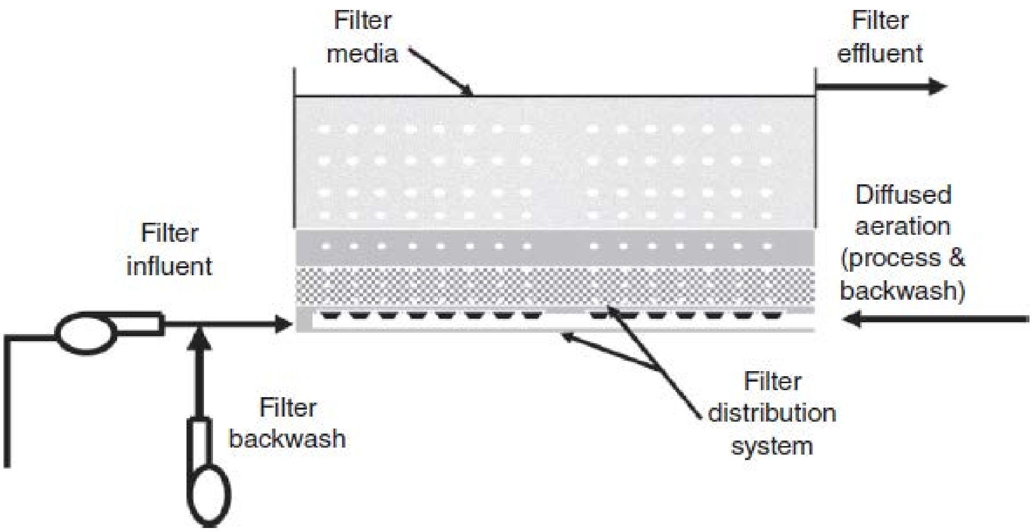


Figure 5-1 Biological Aerated Upflow Filter

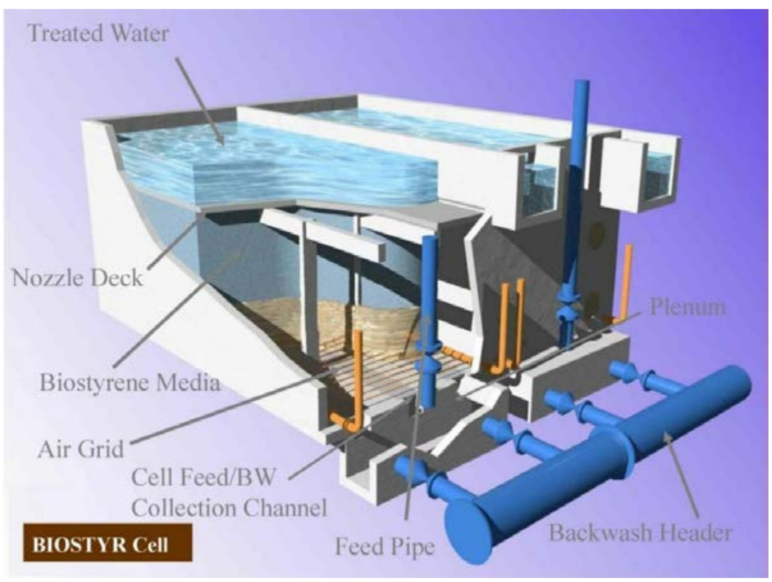


Figure 5-2 BIOSTYR® System Cell General Arrangement

5.1.5.3 Integrated Fixed-Film Activated Sludge (IFAS)

The IFAS process includes a RAS stream to provide for activated sludge as well as fixed film biomass for biological treatment.

Organic loading rates for these reactors are typically in the order of 3.5 to 7.0 g BOD_5/m^2 of media surface area/day for $CBOD_5$ removal and less than 3.5 g BOD_5/m^2 of media surface area/day for nitrification. For nitrification with the IFS process, the required media surface area will usually be dictated by TKN loading, TAN removal requirements and biological growth conditions in the reactor (e.g. temperature, pH, DO). Vendor should be consulted for design details.

A single-pass IFAS have continuously operating, non-cloggable fixed-film reactors with no need for backwashing or return sludge flows, low head-loss and high specific biofilm surface area. This is achieved by having the biomass grow on small carrier elements that move along with the sewage in the reactor or the attached growth support media may be immobile within the reactor for some designs. In the case of free-moving carrier elements, movement is normally induced by coarse bubble aeration in the aerated zone, although fine bubble aeration systems have also been used, while mechanical mixing is utilized in an anoxic/anaerobic zone. For small plants, mechanical mixers are omitted for simplicity reasons and pulse aeration for a few seconds a few times per day can be used to move the biofilm carriers in anoxic reactors.

Free-moving biofilm carrier elements are generally made of polyethylene or polypropylene. A screen is placed at the outlet of the reactor to keep the biofilm elements in the reactor. Agitation constantly moves the carrier elements over the surface of the screen and the scrubbing action prevents clogging. Upstream fine screening of raw sewage should also be considered for such designs. Also, downstream secondary clarification is required for IFAS systems

5.1.5.4 Membrane Bioreactor (MBR)

A membrane bioreactor (MBR) is an activated sludge system with membranes located at the end of the activated sludge tank(s) for liquid-solid separation instead of using secondary clarifiers. Low-pressure membranes (either microfiltration [0.07 to 2.0 µm] or ultrafiltration [0.008 to 0.2 µm]) are typically used in MBRs. The membranes are mounted in modules that can be lowered into the bioreactor. The modules are comprised of the membranes, support structure for the membranes, feed inlet and outlet connections, and an overall support structure. The membranes are subjected to a vacuum (less than 50 kPa) that draws water (permeate) through the membrane while retaining solids in the reactor. To minimize the accumulation of solids and fouling on the exterior side of the membranes, compressed air is introduced through a distribution manifold at the base of the membrane module. As the air bubbles rise to the surface, scouring of the membrane surface occurs; the air also provides oxygen to maintain aerobic conditions and solids suspension within the reactor.

There are two configurations for MBR systems: external (or submerged) and integrated. In the external system, membranes are a separate unit process requiring an intermediate pumping step. In the integrated MBR system, the key component is the microfiltration membrane that is immersed directly into the activated sludge reactor. The submerged configuration relies on coarse bubble aeration to produce mixing and limit fouling. Aeration also maintains solids in suspension, scours the membrane surface and provides oxygen to the biomass, leading to a better biodegradability and cell synthesis. The energy demand of the submerged system can be up to 2 orders of magnitude lower than that of the side stream systems and submerged systems operate at a lower flux, demanding more membrane area.

The principal operational problems with MBR systems are foaming and fouling. Similar to activated sludge and secondary clarifier systems, Nocardioform foaming can occur in MBR systems operated with fine pore diffused aeration. MBR systems must be operated in a preventative maintenance mode to avoid operating problems from fouled membranes. The WRRF capacity can be compromised due to the lower flux associated with fouled membrane. Membrane fouling is prevented by employing the cleaning and operating procedures provided by the membrane supplier, maintaining the upstream fine screening equipment, and operating the system within acceptable SRT and MLSS concentration limits. Improper screening would allow the accumulation of hair and fibrous material in the membranes, which cannot be removed by the normal membrane cleaning program. A lower SRT of about 0.8 d is normally recommended to prevent excessive fouling due to the release of microbial substances from a younger activated sludge. Excessively high SRTs may result in higher amount of free bacteria and floc fines to increase fouling rates.

Concentrations of MLSS in the range of 8,000 to 14,000 mg/L are normally within acceptable operating ranges. Very high MLSS concentrations require a much lower flux to maintain a balance between the amount of solids directed to the membrane surface versus the solids removal rate by the air scour. If excessive MLSS concentrations (>18,000 mg/L) exist under operation of normal design flux values, the membranes can become what is termed "sludged up" and special cleaning methods may be needed to regain the expected operation flux.

Certain wastewater substances must be prevented from entering the treatment facility or MBR system to maintain proper membrane operation. Cooking oils and grease can collect on membrane surfaces and lead to excessive fouling that can only be removed by special membrane cleaning methods.

The process performance of an MBR system is often regulated by effluent concentrations of BOD, COD, ammonia, TN, phosphorus, TSS, and turbidity. Membrane equipment can only control the concentration of the TSS and turbidity. The remaining criteria are governed by biological process design and area affected by SRT, dissolved-oxygen concentrations, recirculation rates within the process, volatile acid concentrations, and other design parameters.

5.1.5.5 Membrane Aerated Bioreactor (MABR)

The membrane aerated biofilm reactor (MABR) is a disruptive municipal wastewater treatment technology that reduces energy requirements for aeration by up to 40 percent, decreases tank requirements for nitrification and increases the level of simultaneous nitrification and denitrification (SND) occurring in the activated sludge process. The MABR relies on gas transferring membranes to deliver oxygen at the base of a nitrifying biofilm. This oxygen transfer is based on diffusion to the biofilm and not transfer from a gas bubble, resulting in transfer efficiencies up to 90%. This also results in a liquid around the membranes maintaining anoxic conditions, which results in nitrification in the biofilm and denitrification in the bulk liquid.

This technology has been in development since the 1980s, with significant bench-scale and pilot-scale work being completed in the 2000s. Initial attempts to incorporate membrane aeration into biological processes focused on using the membranes solely for gas transfer and not as a support structure for biofilms. However, gas transfer efficiency decreased rapidly due to biofouling of the membranes. Timberlake et al (1988) were the first to design a system to take advantage of the aeration membranes as a support for bacteria. By pressurizing hollow fiber membranes with air, Timberlake et al. found a significant amount of TN removal was achievable. Additional studies focused on achieving nitrification and denitrification in a stratified biofilm for TN removal. The thickness and density of the biofilm led to mass transfer and biofilm management concerns. Research began to examine a hybrid system, where a nitrifying biofilm was supported by the MABR, but suspended growth was maintained under anoxic conditions. Pilot-scale studies indicated that this hybrid system could achieve a high TN removal while maintaining a thinner biofilm. Even with all of the research investment since the 1980s, MABR technology has only been commercially available on the market in the past 8 years.

MABR technology is a suitable option for Nobleton WRRF due to limitations in the ability to build a new treatment train. While it can be done, there are hydraulic limitations to take into account with an additional treatment train. This would require additional pumping and piping, along with redundant equipment for the third treatment train and could make the capital costs comparable to MABR technology.

5.1.5.6 Granular Activated Sludge

5.1.6 Secondary Treatment Advantages and Disadvantages

Table 5-1 is a comparison of the secondary treatment options evaluated for this project.

Table 5-1 Comparison of Secondary Treatment Enhancement Technologies

Technology	Status	Advantages	Disadvantages
Conventional Nitrifying Activated Sludge Process (CAS)	Conventional: This technology has been applied in many wastewater treatment facilities in North America and around the world.	• Common and proven • Ability to treat BOD and ammonia in a single stage • Relatively uncomplicated design • Suitable for all kinds of aeration equipment	• Larger footprint required because of the need for primary clarifiers in this application • Larger footprint of aeration basins needed due to colder weather in this application • Stability linked to operation of secondary clarifier for biomass return (RAS)
Extended Aeration (EA)	Conventional: This technology has been applied in many wastewater treatment facilities in North America and around the world and is a modification of the CAS process.	• Relatively uncomplicated design and operation • Easy installation • Smaller footprint • Handles variability of organic loads and flow • High quality effluent • Low biosolids production	• Require large aeration tanks with long aeration periods • Does not achieve denitrification or phosphorus removal • Limited adaptability to changing effluent requirements • Possibility for filamentous sludge bulking and settling issues
Sequencing Batch Reactor (SBR)	Conventional: This is a mature technology that is widely used.	• Simple layout with littler operation and maintenance • Does not require final clarifiers/RAS pumping • Smaller footprint compared to EA • Easy installation • No need to optimize aeration and decanting to comply with power requirement and lower decant discharge rates • Consistently perform nitrification, denitrification, and phosphorus removal • Operational flexibility • Automatic and positive control of MLSS concentration and SRT • MLSS cannot be washed out by high flows because of flow equalization	• Process design and control complicated • Greater level of maintenance • High specific energy consumption and volumetric tankage requirements • Batch discharge may require equalization and secondary clarifiers primary to tertiary treatment and disinfection • High risk flows can disrupt operation • Sludge must be disposed of frequently • Effluent quality depends on operational reliability of decanting facility

Technology	Status	Advantages	Disadvantages
Rotating Biological Contactor (RBC)	Conventional: This is a mature technology that is widely used.	• Short retention time due to large active surface • Capability of handling wide range of flows • Good biomass settleability and easy solids separation • Ease of operation and excellent process control • Low power requirements	• Necessary to cover units to protect against freezing cold weather • Frequent maintenance of shaft bearings and mechanical drive units
Process Intensification			
Moving Bed Bioreactor (MBBR)	Conventional: Over 700 wastewater systems (both municipal and industrial) installed in over 50 countries that are operating.	• Similar BOD and nitrogen removal treatment performance as CAS • Small footprint • Simplicity of operation – no need for manual sludge wasting, SRT control, and sludge recycle • No sludge bulking • Can handle peak wet weather flow variations • Well suited for retrofit application with reduced time and little if any tank construction • More versatile and adaptable for BNR • Continuous operation that does not require special operation or interruption of treatment for biofilm thickness control or flushing out excess solids	• Higher energy demand • Potential issues caused by media removal for diffuser maintenance • High hydraulic profile headloss due to flow through the media screening devices • Limitations for phosphorus removal only by chemical addition
Biologically Active Filters (BAF)	Conventional: This is a mature technology that is widely used.	• Relatively small footprint • Ability to effectively treat dilute wastewaters • No issues with regard to sludge settling characteristics • Simplicity of operation	• More complex in terms of operations and maintenance of instrumentation and controls • Limitations of economies of scale for application to larger facilities • Higher capital cost unless land is at a premium or not available • Vulnerable to high headloss from high solids loadings

Technology	Status	Advantages	Disadvantages
Integrated Fixed-Film Activated Sludge (IFAS)	Conventional: This is a mature technology that is widely used.	• Retrofit flexibility – almost any size or shape of tank can be retrofitted • Carrier elements in the reactor may be decided for each case based on degree of treatment desired, BOD_5, TKN, hydraulic loadings, temperature, and oxygen transfer capability • Reactor volume completely mixed – no "dead" or unused space in reactor • Improved nitrification compared to simple suspended growth systems	• High energy requirements due to aeration • High costs for construction and operation • Challenges in finding mechanical spare parts locally
Membrane Bioreactor (MBR)		• Effluent qualities less dependent on MLSS concentration and sludge properties • Can be operated at higher MLSS concentrations (8,000 to 12,000 mg/L) • Reduction in reactor volume necessary to treat same loading rate • Enhanced ammonia removal • Can potentially reduce or eliminate need for secondary clarification and effluent filters – reduced footprint • Can be retrofitted into existing tankage • Higher SRTs – reduced sludge production • Capital cost can be offset by a lack of needing tertiary filtration • Ease of installation • Ease of flexibility and expansion potential for the future	• High capital costs – although have gotten less expensive • Hydraulic limitations – overloading can lead to fouling of membrane • Redundancy needs to due hydraulic limitations and availability of spare parts can limit flexibility of operations and maintenance staff in working on units or taking units out of service • Limited peaking availability • Optimization needed for chemical usage for membrane cleaning to limit effect of purchasing chemicals on operating costs • Membrane replacement cost affects life-cycle costs • Membrane equipment systems are unique, having different configurations and shapes depending on the manufacturer

Technology	Status	Advantages	Disadvantages
Membrane Aerated Bioreactor (MABR)	Emerging: MABR technology has gone through a lot of research, bench-scale, and pilot-scale testing, but has only been commercially available on the market for about 8 years. While there are many pilot-scale facilities, 1 full-scale facility is in operation since 2017 (Yorkville Bristol Sanitary District, US) and a full-scale facility in construction at Waterloo (expected completion 2021 and driving distance from Nobleton WRRF).	• Reduction in aeration energy by up to 40% • Increased nitrification reliability due to the retention time of attached biomass in the MABR biofilm • Ability to more readily control nitrite shunt for mainstream short cut nitrogen removal • Potential to reduce the SRT seasonally or year-round to increase wet weather treatment capacity • Adoption in the North America accelerating	• Limited manufacturers • Can have a higher capital cost when land is not at a premium, or when there is flexibility to build redundant train • Emerging technology, with more common pilot-scale demonstrations, and one full-scale operating facility in North America.
Granular Activated Sludge	Emerging: Background		

5.2 Screening of Long List of Alternative

The screening of the long list alternatives of secondary treatment technologies is shown in Table 5-2 on the following page. Supplemental to secondary treatment technologies, the various technologies that encompass process intensification are also screened in Table 5-3.

Secondary Treatment Technologies

Conventional: This is a mature technology that is widely used.

5.3 Short-List of Alternative Secondary Treatment Technologies

The following secondary treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Extended Aeration
- Process Intensification: Membrane Aerated Bioreactor (MABR)

Table 5-2 Screening of the Long List of Alternative Secondary Treatment Technologies

Long List of Alternative Secondary Treatment Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Conventional Nitrifying Activated Sludge Process (CAS)	✗	✓	✓	✓	✓	✓	Eliminated due to incompatibility with existing WRRF. More complex operation and therefore, generally applicable to large WRRFs that are continuously staffed. Higher sludge generation. This technology is generally applied to settled wastewater so a primary clarifier would be constructed.
2. Extended Aeration (EA)	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.
3. Sequencing Batch Reactor (SBR)	✓	✓	✓	✗	✓	✓	Eliminated due to stakeholder acceptance.
4. Rotating Biological Contactor (RBC)	✗	✓	✓	✓	✓	✓	Eliminated due to incompatibility with existing WRRF. RBC units have large footprints; therefore, they are not suitable when there is limited space availability. Moreover, these systems require effective primary sedimentation tanks equipped with scum and grease removal devices. This will add to space availability issue mentioned above.
5. Process Intensification	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.

Table 5-3 Screening of the Long List of Alternative Process Intensification Technologies

Long List of Alternative Process Intensification Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Moving Bed Bioreactor (MBBR)	×	✓	✓	✓	✓	✓	Eliminated due to potential for sieve used for catching media to induce more headlosses into the system
2. Biologically Active Filters (BAF)	×	✓	✓	✓	✓	✓	Proceed to detailed evaluation. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.
3. Integrated Fixed-Film Activated Sludge (IFAS)	×	✓	✓	✓	✓	✓	Eliminated due to potential for sieve used for catching media to induce more headlosses into the system
4. Membrane Bioreactor (MBR)	✓	✓	✓	×	✓	×	Eliminated due to high capital and lifecycle costs. Membrane replacement cost affects life-cycle cost analysis. Also, stakeholder acceptance.
5. Membrane Aerated Bioreactor (MABR)	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.
6. Granular Activated Sludge	×		✓	✓	✓	✓	Eliminated due to lack of full-scale application in North America. It is a batch process that would operate very different from the existing flow-through biological treatment process.

6.0 Tertiary Treatment

The main objective of secondary filtration is to reduce TSS and turbidity levels to comply with more stringent effluent requirements (compared to secondary effluent limitations). Filtration also further removes total (and in some technologies, even soluble) phosphorous remaining in secondary effluent.

Tertiary filtration is currently used at Nobleton WRRF.

6.1 Long List of Alternative Tertiary Treatment Technologies

6.1.1 Deep Bed Sand Filtration

Four deep bed Parkson Dynasand filters are installed in the Process Building at the Nobleton WRRF. Figure 6-1 shows Parkson Dynasand filter system shcematic.

This is a common filtration technology. Chemicals are added upstream to coagulate and flocculate solids containing phosphorus which are then removed by filtration in the sand matrix.

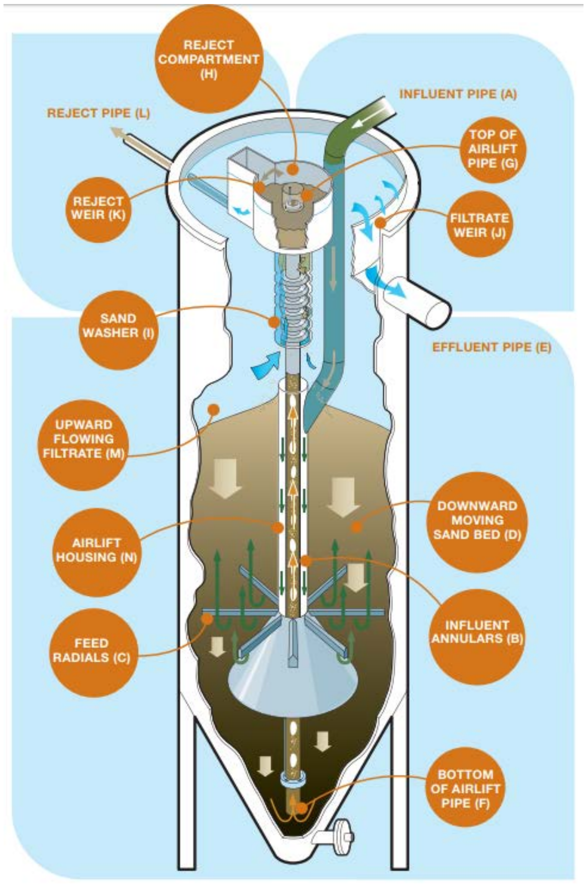


Figure 6-1 Parkson Dynasand Filter Process Schematic

6.1.2 Cloth Disk Filtration (CDF)

Cloth media filters are made of cloth woven or fiber pile (manufacturer dependent) with pores to filter TSS from the wastewater coming from the secondary system. Manufacturers may also offer different cloth media in order to address site-specific conditions (e.g., chemical resistance, different pore size characteristics). The use of woven pile cloth materials has emerged as the most common type of CDF due to improvements in backwash efficiency. Nominal pore size ranges between 5 and 10 μm for different type of cloth materials, but significant removals can be realized in smaller particle size ranges. The most common geometry for these filters is the disk configuration. Cloth disk filters are used as a pretreatment step prior to the membrane filtration system or for effluent TSS polishing, water reuse, and phosphorous removal.

According to the Water Environment Federation (WEF) manual of practice No. 8, typical maximum design filtration rates are between 240 to 280 L/(m^2·min). Although testing has shown that these filters can operate at hydraulic loading rates up to 800 L/(m^2·min) for short periods. The maximum hydraulic loading rate can also be limited by the influent TSS when the solids loading rate exceeds the manufacturer's recommendation.

During the filtration cycle, the wastewater flow is from the outside to the inside of the disks. Several cloth disks covered by cloth media are mounted vertically to a common hollow tube, which conveys filtered effluent from the filter. Wastewater passes through the cloth media by gravity and enters inside filter disks that are connected to the effluent line by the hollow tube. A total hydraulic head between 0.75 and 1.2 m is required for the operation of the disk filters.

Backwash cycle starts when the terminal headloss or a certain run time is reached. The disk filters backwash more frequently (e.g., compared to sand filters) because of the low head operational characteristics and low terminal headloss design values. Clean medium headloss ranges between 5 and 10 cm.

CFD technology was implemented in March 2014 by Nexom for a small municipal wastewater treatment plant for the community of Sundridge, ON (with the design flow of 0.45 MLD). After installing and having the two-tank infini-D system in operation for 18 months, effluent TP concentrations reduced from 8.3 mg/L to less than 0.1 mg/L.

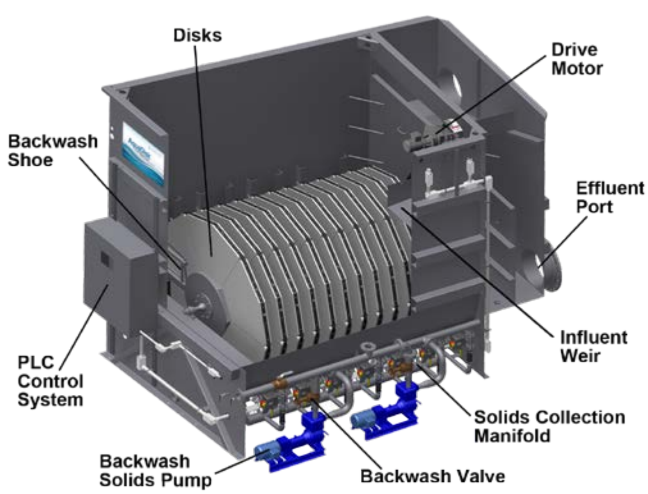


Figure 6-2 Cloth Media Filter with OptiFiber® Configuration

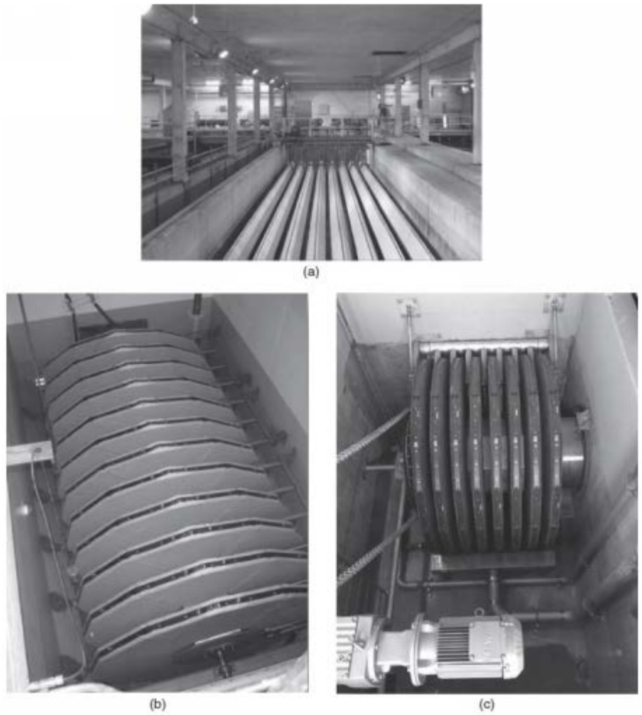


Figure 6-3 Different Cloth Depth Filter Configurations: (a) Aquadiamond Configuration; (b) Aquadisk

6.1.3 Blue PRO Filter System

The Blue PRO technology combines co-precipitation and sorption to remove both particulate and soluble phosphorus. It is similar to the deep bed sand filtration technology except that the media are coated with a chemical that adsorbs soluble phosphorus. Through these processes, some phosphorus is precipitated and removed from water as it moves upward though the sand media. At the same time, some phosphorus is adsorbed onto the hydrous ferric oxide coated sand. This adsorption mechanism allows the process to achieve very low concentrations of phosphorus in the effluent. The phosphorus is then removed from the sand through abrasion and separated in the sand washer at the top of the filter.

The Blue PRO process schematic is shown in Figure 6-4. An iron-based chemical is added to the wastewater before it passes into the rapid conditioning zone. The rapid conditioning zone allows the proper contact time for the mixture to optimize the adsorption process. The mixture enters the moving bed sand filter through distribution arms at the bottom of the sand bed, flowing upwards through the sand bed. The Blue PRO process uses ferric chloride or ferric sulphate for continuous regeneration of hydrous ferric oxide coated media for adsorption of phosphorus.

After filtration, treated water discharges from the top of the filter. Internally, the sand moves slowly from top to bottom, then returns to the top of the filter via an airlift located in the central assembly.

After adsorption, the iron and phosphorus are subsequently abraded off the sand both in the sand bed and in the airlift. A wash-box at the top of the filter separates sand from iron and phosphorus waste particulates. The sand is retained within the filter and falls back to the top of the bed; the residuals, including the iron and phosphorus or other contaminants, exit in a reject line.

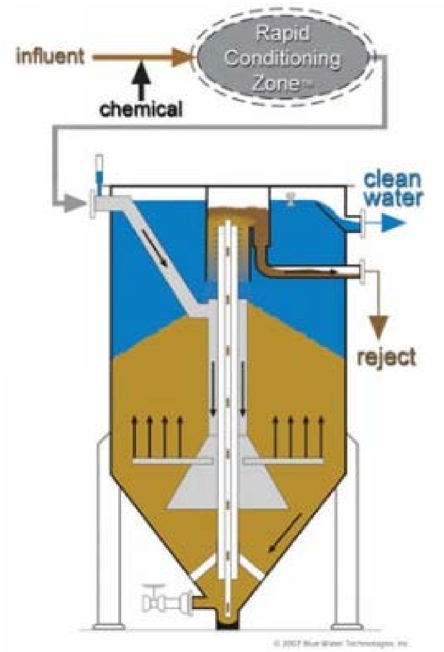


Figure 6-4 BluePRO Reactive Filtration System Process Schematic

6.1.4 Tertiary Low-Pressure Membrane Filtration (MF)

Membrane filtration is used to produce high quality effluent and serves as a pretreatment process for the reverse osmosis (RO) system. Membrane Filtration (MF) is a physical separation process sized based on the peak daily flow (PDF) and remove suspended/colloidal solids from the feed stream through a porous membrane. Figure 6-5 is a typical flow schematic that shows how membrane units and support systems are interrelated in an effluent filtration application.

Low-pressure membrane effluent filtration systems typically consist of the MF or Ultra-Filtration (UF) membrane system and various pretreatment and post-treatment systems. At a nominal size of 0.01µm, the UF membrane pores are approximately 1/10th the size of typical MF membrane pores. An MF membrane will reject particulates, including bacteria and suspended solids while the UF membranes can reject these solids as well as some macromolecules including emulsified oils. Compared with pressurized membrane systems, immersed membrane processes have significantly lower operating costs. For instance, the pumping energy needed for a 4,000 m^3/day immersed UF membrane system operating at 0.5 bar TMP and 65% pump efficiency is only 3.5 kW/h.

There are two types of membrane configurations: pressurized and immersed. Pressurized membrane configurations consist of membranes located within individual pressure vessels, with groupings of these pressure vessels housed in frames within buildings or on concrete pads. Immersed membrane configurations consist of membranes assembled into filter cells (also known as racks or cassettes) located within one or more tanks containing the wastewater to be treated. Ancillary systems for both configurations are typically located adjacent to the tanks or pressure vessels. Although the configurations are very different, the performance and filtrate water quality of the membranes are effectively the same.

Microfilter membranes operate by a surface removal mechanism and are similar to a fine screen or sieve. The pore size at the surface of most membranes is highly uniform and has a narrow pore size distribution. Particles larger than the size of the largest pore are rejected by the membrane surface and remain on the feed or concentrate side. The bulk carrier fluid, and any particles finer than the largest pore, can pass through the membrane to the filtrate side.

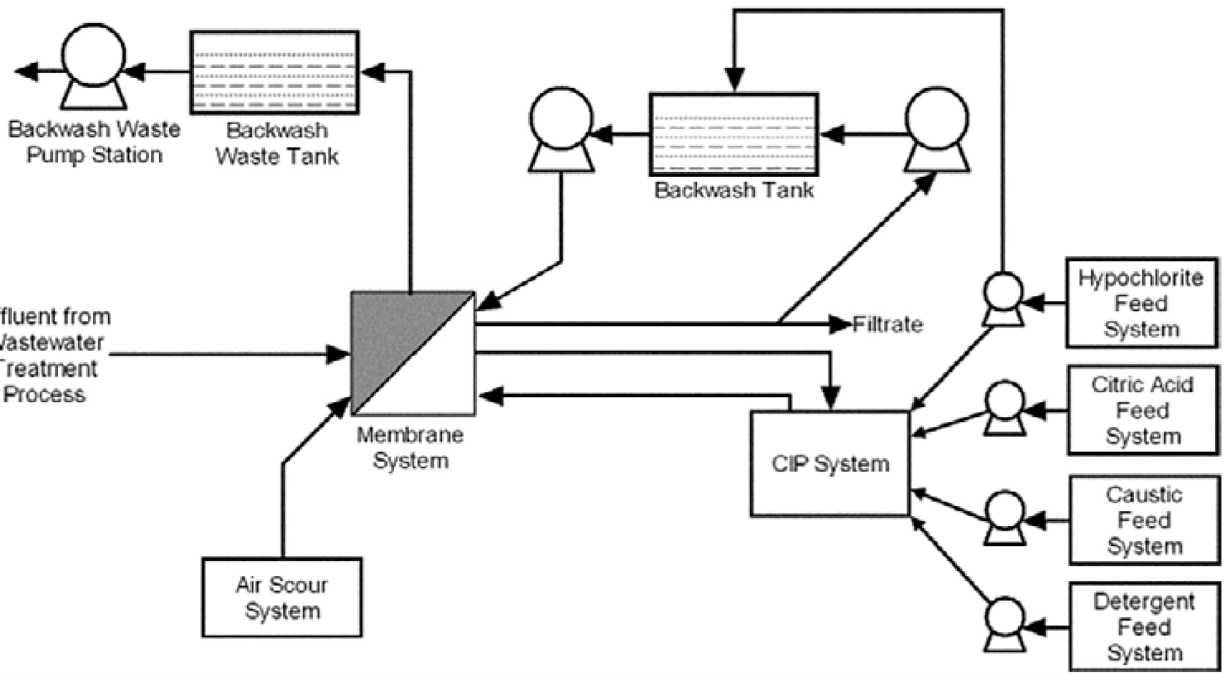


Figure 6-5 Diagram of a Typical Effluent Membrane Filtration System

6.1.5 Reverse Osmosis (High-Pressure Membrane Filtration)

Reverse Osmosis (RO) is a widely accepted unit operation for water purification. It is a high-pressure membrane filtration process with much smaller pores. This system consists of multiple components: 1) RO transfer pumps to pump the MF permeate through the Cartridge filters; 2) Cartridge filters for protection of the RO membranes; 3) RO high-pressure feed pumps to pump the water through the RO modules; 4) RO skids which hold the RO modules; and 5) Decarbonation system to raise the pH of the product water. The feedwater is treated by reverse osmosis after pretreatment and boosted to the required pressure by the high-pressure pump. The modules produce two process streams: (1) permeate, which is the product water, and (2) concentrate or reject, which is a waste stream. Figure 6-6 shows typical single-array and two-array reverse osmosis facility layouts. A significant advantage of the two-array configuration is that the product recovery is increased compared to single-stage operations.

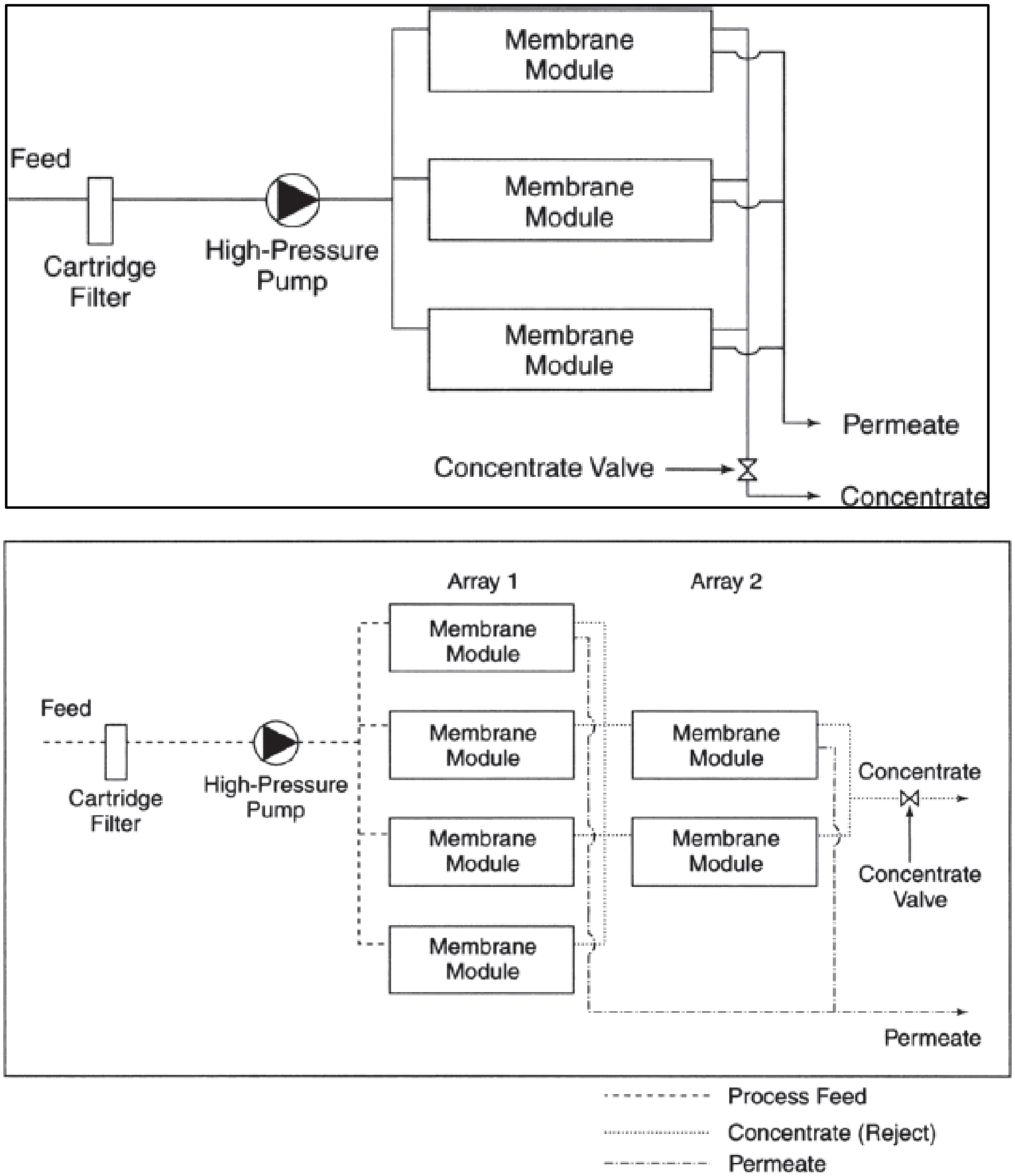


Figure 6-6 Simplified Schematic Diagram of a Single-Array (Top) and a Two-Array (Bottom) Reverse Osmosis Process

6.1.6 Tertiary Treatment Advantages and Disadvantages

Table 6-1 is a comparison of the secondary treatment options evaluated for this project.

Table 6-1 Comparison of Tertiary Treatment Enhancement Technologies

Technology	Status	Advantages	Disadvantages
Deep Bed Sand Filtration	Conventional: Well-established technology with numerous installations across North America	• Relatively common and able to meet tight effluent limits • Effective solids removal	• Relatively large footprint • Capital costs • Need for intermediate pumping
Cloth Disk Filtration (CDF)	Conventional: This is a mature technology that is widely used.	• Can reduce TSS concentrations down to 5 mg/L while removing TP down to less than 0.1 mg/L • Removal performance can be increased with chemical addition • Flexible in handling peal flows • Smaller footprint • Filtration operation is continuous due to small portion of media out of service during backwash – no need for backwash reject water storage basin • Filtered water used for backwash – no need for separate backwash water supply	• Chemical addition can prevent medium blinding if careful consideration not taken into account • Solids can sometimes pass through the pile media during high-pressure cleanings • Complicated system • Biological matter can grow on the filtrate side of the cloth • Filtration process must be taken offline to initiate high-pressure backwash cycle
Blue PRO Filter System	Emerging: 4 full-scale operations of Blue PRO Filter System.	• High efficiency and can remove 99+% of TP from municipal wastewater • Low chemical dose • No need for backwashing • Low capital, operating, and maintenance costs • Can reduce sludge handling costs • Works without pH adjustment • Highly tolerant of interfering water chemistry • Significantly lower turbidity and BOD.	• Large footprint • Large and tall building required over filters to allow for removal of air lift equipment • Proprietary equipment.

Technology	Status	Advantages	Disadvantages
Tertiary Low-Pressure Membrane Filtration (MF)	Conventional: This is a mature technology that is widely used.	• Smaller footprint • Automatically operated • Lower chemical usage • For Nobleton, a pressurized system will most likely be more cost effective • Membrane modules easily accessed	• Fouling • Membrane material properties, module hydrodynamic conditions, and feed water characteristics dictate the degree to which a membrane will foul
Reverse Osmosis (High-Pressure Membrane Filtration)	Conventional: This is a mature technology that is widely used.	• Removes nearly all contaminant ions and most dissolved non-ions • Capable of low effluent concentrations (especially TP) • Simplicity of operation • Automation allows for less operator attention • Demonstrated lowest of effluent phosphorous concentrations of current technologies	• High capital and operating costs • Permeate remineralization and brine disposal • Rejects charged species such as orthophosphate as well as large organic compounds • Consideration for reject brine disposal, permeate remineralization, and high energy cost in comparison to other alternatives

6.2 Screening of Long List of Alternative Wastewater Servicing Design Concepts

The screening of the long list alternatives of tertiary treatment technologies is shown in Table 6-2.

6.3 Short-List of Design Concepts

The following tertiary treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Deep Bed Sand Filtration

Table 6-2 Screening of the Long List of Alternative Tertiary Treatment Technologies

Long List of Alternative tertiary Treatment Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Deep Bed Sand Filtration	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. This is the current technology at Nobleton WRRF and is effective at obtaining Nobleton's effluent goals, and would require only a modest expansion for the future design.
2. Cloth Disk Filtration (CDF)	✗	✓	✓	✓	✓	✗	Eliminated due to cost of retrofitting or building a new filtration facility.
3. Blue PRO Filter System	✓	✓	✓	✓	✓	✗	Eliminated due to the relative higher cost of retrofitting the technology.
4. Tertiary Low-Pressure Membrane Filtration (MF)	✗	✓	✓	✗	✓	✗	Eliminated due to cost of retrofitting or building a new filtration facility and high operating costs. Membrane filtration is a higher level of treatment than required.
5. Reverse Osmosis (High-Pressure Membrane Filtration)	✗	✓	✓	✗	✓	✗	Eliminated due to cost of retrofitting or building a new filtration facility and high operating costs. Reverse osmosis is a higher level of treatment than required.

7.0 Disinfection

The purpose of disinfection is to eliminate pathogens from treated wastewater.

UV disinfection technology is currently used at the Nobleton WRRF.

7.1 Long List of Alternative Design Concepts

7.1.1 Chlorine Based Methods

Chlorine is one of the most widely used disinfectants for municipal wastewater. It destroys target organisms by oxidizing cell wall material, causing leakage of cellular constituents outside of the cell. Overall, chlorine disinfection is reliable and effective against a wide spectrum of pathogenic organisms.

However, due to the toxicity of chlorine residuals at extremely low concentrations (11 to 19 μg/l) it is difficult to control chlorine-induced toxicity to aquatic life in the receiving waters. This is not as critical as an issue at the plant as the current chlorine residual ranges from 0.5 to 0.6 mg/L. With this effluent chlorine residual concentration, the plant has been able to eliminate the use of the use of a dechlorinating agent. Chlorination can also produce undesirable by-products such as trihalomethanes (THMs) and haloacetic acids (HAAs).

Additionally, some parasitic species have shown resistance to low doses of chlorine, including oocysts, of Crptosporidium parvum, cysts of Endamoeba histolytica and Giardia lamblia, and eggs of parasitic worms.

Two of the main forms of using Chlorine for disinfection are presented below.

7.1.1.1 Chlorine Gas

Chlorine gas (Cl_2) is the most common means of disinfection in the United States. Since chlorine gas is frequently used, the design parameters and dosing requirements are well established. The equipment is fairly reliable and easy to operate. Typical gaseous chlorine facilities are comprised of a chlorine cylinder storage area equipped with storage cradles, scales, chlorine gas detectors, and an overhead crane or hoist. Chlorine feeders transfer the chlorine from the cylinders and disperse a dose of chemical into a stream of water.

The largest drawback to chlorine gas is the significant health hazard that an accidental release would incur on the surrounding community while in transport or at the plant. An emergency scrubber is commonly installed to capture and neutralize any chlorine gas leaks, but this is not full-proof.

7.1.1.2 Bulk Sodium Hypochlorite

Bulk Sodium Hypochlorite, commonly known as “liquid bleach”, is another common form of chlorine for disinfection. It is generally produced as a 12.5% w/v NaOCl diluted aqueous solution, and is increasing in water and wastewater treatment applications due to safety concerns associated with the use, storage and transport of chlorine gas. Caution has to be exercised in the handling and storage of sodium hypochlorite to prevent exposure and minimize degradation of the chemical. Due to the toxicity of chlorine residuals, bisulfite is used to quench the residual chlorine levels. Figure 7-1 shows a hypochlorite storage facility located in California.

Figure 7-1 Hypochlorite Storage Facility

7.1.2 Peracetic Acid

Peracetic Acid, or PAA, has been regularly used as a wastewater disinfectant in Europe and Canada for the past 30 years. It is a clear, colorless liquid that forms an equilibrium mixture with hydrogen peroxide and acetic acid. It is reported to be an inherently stronger oxidant and more rapid disinfectant than chlorine-based disinfectants. Additionally, it dissipates rapidly and does not generate harmful disinfectant byproducts even if overdosed. The largest drawback of PAA use in the plant is the absence of U.S. operation standards as it is still under investigation and testing by the EPA. Figure 7-2 shows a PPA storage tank facility.

Figure 7-2 PPA Storage Tank Facility

7.1.3 Ultraviolet Irradiation

Over the past several years, UV disinfection technology has grown in popularity, resulting in growth of new technology and more sophisticated and reliable systems that operate more cost effectively. It is a physical disinfecting agent, separating it from the chemical disinfectant options, using ~254 nm wavelength to penetrate cell walls and break apart the cellular DNA and RNA. UV light is effective as both a bactericide and virucide. Since UV light is not a chemical agent, no toxic residuals are produced. An example of an UV system is shown in Figure 7-3.

Figure 7-3 UV Disinfection System

The main water quality parameter used to specify UV disinfection systems and with which the performance is determined is UV transmittance (UVT). It is important to understand seasonal, wet-weather, and diurnal UVT trends. The importance of UVT is borne out of the fact that, for each 0.05 drop in UVT (on a zero to one scale), only half the volume of water can be disinfected using the same predetermined dosage rate.

Many UV disinfection systems have been installed in municipal wastewater treatment plants as effluent chlorine residual limits become tighter. There are multiple UV technology systems on the market today, and new advances are emerging as the market responds to user demands.

Two banks of low-pressure, low output bulbs are installed in a channel downstream from tertiary filtration in the Process Building

7.1.4 Disinfection Advantages and Disadvantages

Table 7-1 is a comparison of the disinfection technology options evaluated for this project.

Table 7-1 Comparison of Disinfection Treatment Enhancement Technologies

Technology	Status	Advantages	Disadvantages
Chemical Based Disinfection Technologies			
Chlorine Gas	Conventional: One of the most widely used disinfectants for municipal wastewater.	• Widely used • Reliable and effective against wide spectrum of pathogenic organisms • Dosing flexibility to handle peak flows • Ease of implementation • Chlorine scrubbing towers can mitigate the risk of chlorine gas	• Toxicity of chlorine residuals at extremely low concentrations – chlorine induced toxicity to aquatic life • Needs dichlorination agent if effluent chlorine residual concentrations are too high • Can produce undesirable by-products such as trihalomethanes and haloacetic acids • Some parasitic species have shown resistance to low doses of chlorine • Significant health hazard should an accidental release occur
Bulk Sodium Hypochlorite	Conventional: Another common form of chlorine for disinfection.	• Widely used • Reliable and effective against wide spectrum of pathogenic organisms • While more expensive per unit weight of chlorine than chlorine gas, aqueous form poses less health hazards – incur lower costs • Ease of implementation	• Toxicity of chlorine residuals at extremely low concentrations – chlorine induced toxicity to aquatic life • Needs dichlorination agent if effluent chlorine residual concentrations are too high • Can produce undesirable by-products such as trihalomethanes and haloacetic acids • Some parasitic species have shown resistance to low doses of chlorine • Can handle peak flows so long as chemicals are available
Peracetic Acid	Conventional: Regularly used as a wastewater disinfectant in Europe and Canada for the past 30 years.	• Widely used • Stronger oxidant and more rapid disinfectant the chlorine-based disinfectants • Dissipates rapidly and does not generate harmful disinfectant byproducts even if overdosed • Potential to be expanded for future growth/regulatory requirements • Ease of implementation	• Reliably proven for smaller facilities only (which is fine in this application as Nobleton is a smaller facility) • Operating cost highly dependent on market price for PAA

Technology	Status	Advantages	Disadvantages
Physical Based Disinfection Technologies			
Ultraviolet Irradiation	Conventional: Grown rapidly in the past several years, and is widely used across North America.	• Not a chemical agent – no toxic residuals are produced • Reliable • Operate more cost effectively • Potential to be expanded for future growth/regulatory requirements • Existing facility already has UV disinfection – capital cost would not be much compared to other alternatives having to replace/rehab existing infrastructure • Ease of implementation	• Capital cost – requires significant capital investment • Operating costs include electricity as a significant portion

7.2 Screening of Long List of Alternative Wastewater Servicing Design Concepts

The screening of the long list alternatives of disinfection treatment technologies is shown in Table 7-2.

7.3 Short-List of Design Concepts

The following disinfection treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- UV Irradiation

Table 7-2 Screening of the Long List of Alternative Disinfection Treatment Technologies

Long List of Alternative Disinfection Treatment Concepts	Screening Criria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Chlorine Gas	✗	✓	✓	✓	✓	✗	Eliminated due to UV disinfection already existing at Nobleton WRRF. By swapping to chemical-based disinfection, Nobleton WRRF would incur capital costs in order to change the existing channel to a contact basin and would add in operating costs.
2. Bulk Sodium Hypochlorite	✗	✓	✓	✓	✓	✗	Eliminated due to UV disinfection already existing at Nobleton WRRF. By swapping to chemical-based disinfection, Nobleton WRRF would incur capital costs in order to change the existing channel to a contact basin and would add in operating costs.
3. Peracetic Acid	✗	✓	✓	✓	✓	✗	Eliminated due to UV disinfection already existing at Nobleton WRRF. By swapping to chemical-based disinfection, Nobleton WRRF would incur capital costs in order to change the existing channel to a contact basin and would add in operating costs.
4. Ultraviolet Irradiation	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. This is the current technology existing at the Nobleton WRRF. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.

8.0 Sludge Thickening and Dewatering

The purpose of sludge thickening and dewatering is to reduce the volume and weight of sludge for hauling or downstream handling. The product of sludge thickeners is liquid, the product of sludge dewatering is cake.

A sludge thickener is installed in the Nobleton WRRF Process Building. Sludge dewatering

8.1 Long List of Alternative Sludge Thickening Technologies

8.1.1 Sludge Thickening - Gravity

8.1.1.1 Gravity Thickeners

Gravity thickening is one of the most common methods used for solids thickening and is accomplished in a tank similar in design to a conventional sedimentation tank. Feed sludge is allowed to settle and compact, and the thickened sludge is withdrawn from the bottom of the tank.

Gravity thickening is primarily used for primary sludge and mixtures of primary and waste activated sludge. Due to better performance of other thickening methods for WAS, gravity thickening has limited application for such sludges. Gravity thickening on untreated primary sludge, or primary sludge mixed with waste active sludge, is often used as it can achieve resulting sludge concentrations in the range of 4 to 6 percent.

A non-mechanical gravity thickener is currently used to thicken waste activated sludge prior to storage and hauling.

8.1.1.2 Dissolved Air Flotation (DAF)

Dissolved air flotation (DAF) thickening concentrates solids by attaching microscopic air bubbles to the suspended solids, increasing the buoyancy of the solids and causing them to float to the surface. A recycle stream from the DAF subnatant is super-saturated with air and discharge into the DAF influent. When this combined stream (whitewater) is released in the DAF, the dissolved air comes out of solution forming fine bubbles. A pressure tank (saturator) and compressor system has been typically used to make the whitewater; however, air handling recycle pumps are available that combine the pumping and air injection steps, eliminating the need for saturators and compressors. A DAF thickener is shown in Figure 8-1.

Figure 8-1 DAF Thickener (Courtesy of Envirex)

Dissolved air flotation thickeners are typically sized based on the solids loading rates and can be operated with or without polymer conditioning. Variables that can affect the performance of a DAF thickener include hydraulic loading, recycle flow, air-to-solids ratio, dissolution ratio, and the rate of removal of the float solids. The thickened solids concentrations range from 3 to 4 percent at greater than 90 percent capture efficiency. At this concentration, polymer is unlikely to be required, but the facility should be provided as a backup. DAF thickening technology is available from a number of manufacturers, including Evoqua/Envirex, Suez, and Ovivo.

8.1.2 Sludge Thickening - Mechanical

8.1.2.1 Centrifugation

Centrifuge thickening is commonly used for WAS thickening in medium- to large-capacity facilities. It is a self-contained process that uses high speed centrifugal forces to separate suspended solids from the liquid. The solids are forced to the perimeter of the bowl, conveyed by a scroll to one end of the unit and discharged. The liquid flows through ports at the opposite end of the unit and is typically returned to the headworks. The principle of operation is presented in Figure 8-2. An installed unit is shown in Figure 8-3. Centrifuge equipment is available from a number of manufacturers, including Westfalia, Andritz, and Alfa Laval.

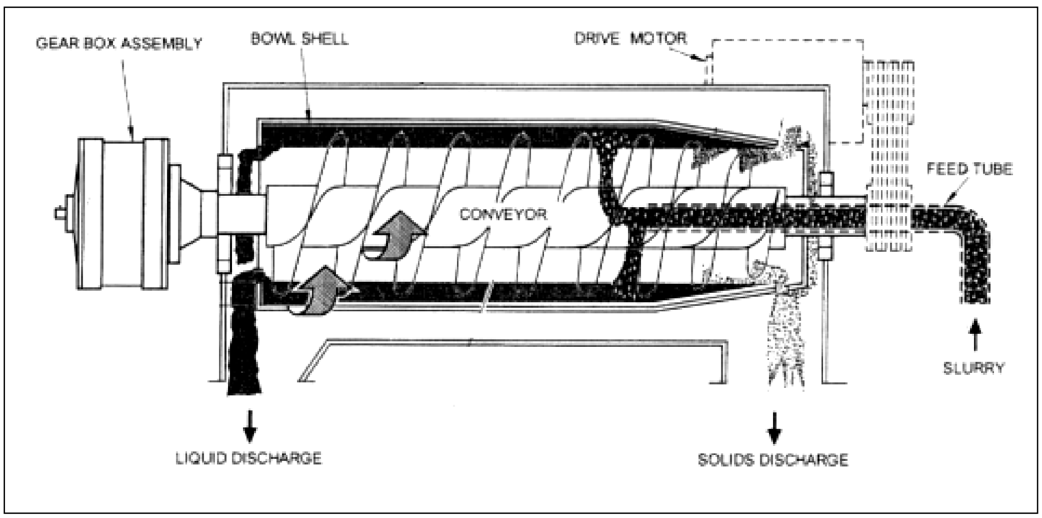


Figure 8-2 Centrifuge Principle of Operation (Courtesy of Alfa Laval)

Figure 8-3 Installed Centrifuge

In WAS thickening applications, centrifuge typically achieve solids concentrations ranging from 5 to 6 percent at solids capture efficiencies of 90 to 95 percent. Higher solids concentrations up to 8 percent TS are possible in co-thickening applications. Polymer addition can increase solids capture to approximately 95 percent, but generally does not increase the thickened solids concentration. Typically, facilities using centrifuges for WAS thickening feed up to 10 pounds of polymer per dry ton of solids; however, some installations have been able to operate thickening centrifuges with little or no polymer. Operational control of the process is possible through variation of hydraulic throughput, adjustment of scroll speed, pool depth, and polymer feed.

Centrifuges have higher power consumption than the other thickening technologies. Routine maintenance of centrifuges can be performed by the plant staff, but periodically the scroll/bowl assembly may have to be shipped to a maintenance facility. This can result in extended downtime for the equipment. Some centrifuge suppliers have started providing replacement scroll/bowl assemblies for use at the time the existing one is pulled to minimize downtime.

8.1.2.2 Gravity Belt Thickener (GBT)

Gravity belt thickeners have widespread use for WAS thickening applications. Gravity belt thickeners separate free water from the solids by gravity drainage through a porous belt. Dilute solids are introduced at the head end of a horizontal filter belt. As the solids move along the belt, free water drains through the porous belt into a collection tray and is returned to the headworks. Plows in the gravity zone break up the solids and aid the release of water. Thickened solids are discharged at the end of the horizontal filter belt. Gravity belt thickeners are available in belt widths ranging from 1 to 3 meters. Figure 8-4 and Figure 8-5 show the operation principle of a GBT and an installed unit, respectively.

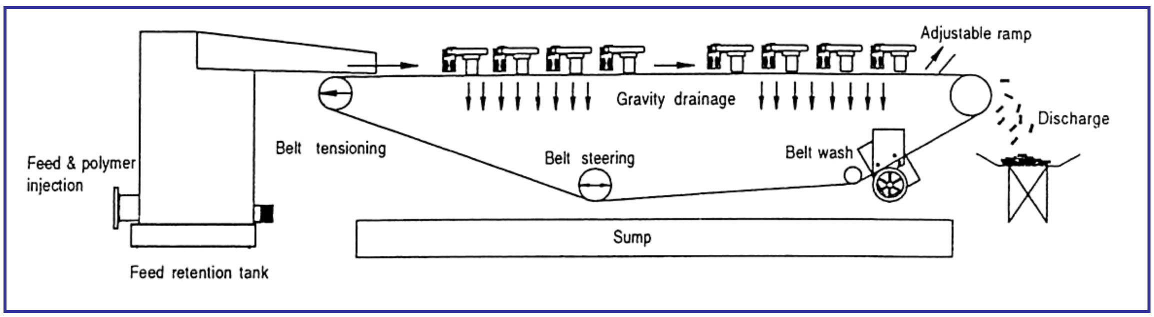


Figure 8-4 Gravity Belt Thickener Principle of Operations (Courtesy of Ashbrook)

Figure 8-5 Installed Gravity Belt Thickeners at the Bissell WWTP

The feed solids are conditioned with a polymer to form a stable floc before introduction to the belt. With the use of a polymer, GBTs can achieve solids captures of 95 percent. Operation of a GBT can be controlled by adjusting solids feed rate, polymer feed rate, belt speed to control solids retention time on the belt, and position of the solids plow.

Gravity belt thickeners have an open equipment design and can be difficult to capture odorous emissions for treatment, requiring odour control for the whole airspace. The belt has to be washed continuously to avoid blinding. They also require 1/2 hour operator attendance on startup and shutdown. Gravity belt thickeners are available from several manufacturers, including Bellmer, Komline-Sanderson, Ashbrook, and Siemens.

8.1.2.3 Rotary Drum Thickener/Rotary Screw Thickener

Rotary drum thickeners (RDT) and rotary screw thickeners (RST) are parallel technologies based on a similar premise. Both technologies use gravity to drain the solids as they pass through a mesh or perforated basket. Besides the need for polymer addition, a flocculation tank upstream, and a system of spray nozzles to keep the media clean, the main differences between the technologies are:

- RDTs:
  - Rotating shell made of wire or polyethylene mesh or perforated steel
  - Drum is differentiated into zones based on mesh size, with a finer mesh at the inlet where the feed solids contain more water and mesh size increases towards the drum outlet to facilitate drainage of the more concentrated solids
  - Feed solids are pumped into the drum, where drum rotation helps drive the filtrate through the perforations into a collection trough
  - Rings of varying heights inside the drum control the solids retention time in each zone

   - RDTs can produce 4-6 percent solids with 95 percent solids recovery with the use of polymer
   - Typically enclosed to contain odours
- RSTs:
   - Uses rotating screws with stationary drums
   - Flocculated solids overflow into the lower portion of the inclined drum with a static perforated basket
   - Equipped with a slowly rotating screw that conveys solids upward to the drum discharge while allowing water to drain through the basket
   - Basket is continuously cleaned with brushes to prevent solids accumulation and periodically cleaned with an automatic spray wash
   - RSTs can produce 4-8 percent solids with 95 percent solids capture

Figure 8-6 and Figure 8-7 show a rotary drum thickener and a rotary screw thickener, respectively.

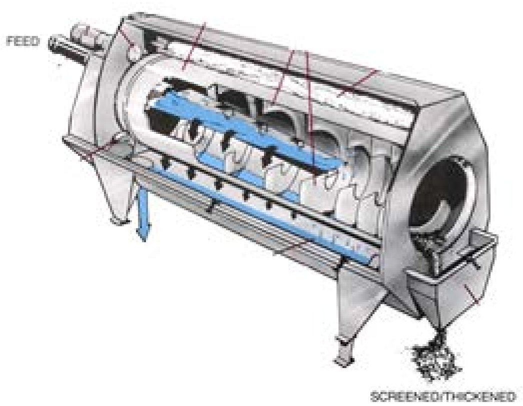


Figure 8-6 Rotary Drum Thickener Principle of Operation (Courtesy of Parkson)

Figure 8-7 Rotary Screw Thickener (Courtesy of Huber)

8.1.3 Solids Thickening Technologies Advantages and Disadvantages

Table 8-1 is a comparison of the solids thickening treatment options evaluated for this project.

Table 8-1 Comparison of Solids Thickening Technologies

Technology	Status	Advantages	Disadvantages
Non Mechanical Thickening			
Gravity Thickeners	Conventional: This is a mature technology that is widely used.	• Proven technology • Currently existing at facility	• WAS only sludge – performance only 2-3% solids
Dissolved Air Flotation (DAF)	Conventional: This is a mature technology that is widely used.	• Provides "wide spot" in line, minimizing need for WAS storage • Little operator attention • Can be designed for low or no polymer consumption • Relatively insensitive to hydraulic loading rate changes • Technology available from several manufacturers	• Relatively high power use – varies depending on saturation technology • Open tank, requiring odour control for the whole building airspace • Can achieve lower thickened solids concentration than other thickening technologies (WAS only DAFs) • Can have large footprint requirement • Higher capital costs compared to some of the other thickening technologies
Mechanical Thickening			
Centrifuge	Conventional: This is a mature technology that is widely used.	• High capacity equipment – well suited for larger plants • Higher solids concentrations (5-8% TS), depending on feed solids characteristics • Minimum space requirements • Little operator attention when operations are stable • Enclosed technology – good odour containment and housekeeping • Technology available from several manufacturers	• Higher capital costs compared to some of the other thickening technologies • Higher energy use • Major maintenance must be performed by the manufacturer • Polymer required • Closer operator attention is required to achieve thickened concentrations less than 5%

Technology	Status	Advantages	Disadvantages
Gravity Belt Thickener (GBT)	Conventional: This is a mature technology that is widely used.	• Moderate operational complexity; relatively low requirement for operator attention • Relatively high unit capacity • Relatively low initial capital cost • Low power requirements	• Open equipment design – potential for odours and high humidity • Require frequent belt washing to avoid blinding – high wash water flows • Requires operator intervention at startup • Closer operator attention is required to achieve thickened concentrations less than 5% • Polymer required
Rotary Drum Thickener/ Rotary Screw Thickener	Conventional: This is a mature technology that is widely used.	• Moderate operational complexity • Low initial capital cost • Low power usage • Good odour containment • Technology available from several manufacturers	• Higher polymer consumption – varies by manufacturer • High wash water requirements • Relatively low unit capacities • Closer operator attention is required to achieve thickened concentrations less than 5% • Requires operator intervention at startup

8.1.4 Sludge Dewatering

8.1.4.1 Centrifuges

Centrifugation is used widely in the industry as a means to separate liquids of different density, thickening slurries, or removing solids. Centrifuge types for dewatering applications include solid bowl, basket, and disc centrifuges. The most frequently used of these is the continuous countercurrent solids bowl centrifuge. In this type of centrifuge, sludge is fed at a constant flowrate into a rotating bowl, where the sludge separates into either a dense cake containing solids or a dilute liquid stream called "centrate."

Solid-bowl centrifuges are suitable for a number of dewatering applications and chemicals can be used to aid in conditioning to achieve the desired dewatering performance.

8.1.4.2 Belt Filter Presses

A belt filter press consists of two continuous, separate belts. One belt is a press belt and the other is a filter belt. The sludge is confined between the two belts with the press belt exerting pressure on the filter belt, therefore continuously dewatering the sludge.

For belt filter presses, there are generally three distinct dewatering zones. The first zone is the gravity drainage zone, the second is the pressure zone, and the third is the shear zone. Pressure is exerted by the rollers, conveying belts, or other external devices. The shear zone allows the cake to be further dewatered by deforming the sludge cake by passing the belts around rolls and/or between vertically offset rollers causing a serpentine-like configuration in the sludge cake movement.

8.1.4.3 Filter Presses

Filter presses are a conventional means of dewatering that were on the decline; however, recent changes in the design of filter presses, including the elimination of leakage problems, more automation, improved filter media, greater unit capacities, and the development of high molecular weight polymers and compatible polymer feed systems has resulted in a renewed interest in this sludge dewatering technology.

8.1.5 Solids Dewateromg Technologies Advantages and Disadvantages

Table 8-2 is a comparison of the solids dewatering treatment options evaluated for this project.

Table 8-2 Comparison of Solids Dewatering Technologies

Technology	Status	Advantages	Disadvantages
Centrifuges	Conventional: This is a mature technology that is widely used.	• Clean appearance • Minimal odour problems • Fast startup and shut down capabilities • Easy to install • Produces relatively dry sludge cake • Low capital cost-to-capacity ratio	• Scroll wear potentially a high maintenance problem • Requires grit removal and possibly sludge grinder in the feed stream • Skilled maintenance personnel required • Moderately high suspended solids content in centrate • Cannon observe dewatering zone to optimize/adjust performance
Belt Filter Presses	Conventional: This is a mature technology that is widely used.	• Low energy requirements • Relatively low capital and operating costs • Less complex mechanically and easier to maintain • High pressure machines are capable of producing very dry cake • Minimal effort required for a system shut down	• Hydraulically limited in throughput • Requires sludge grinder in feed stream • Very sensitive to incoming sludge feed characteristics • Short media life as compared to other devices using cloth media • Automatic operation generally not advised
Filter Presses	Conventional: This is a mature technology that is widely used.	• Highest cake solids concentration • Low suspended solids in filtrate • Simple operation • High solids capture rate	• Batch operation • High equipment cost • High labor cost • Special support structure requirements • Large floor area required for equipment • Skilled maintenance personnel required • Additional solids due to large chemical addition require disposal.

8.2 Screening of Long List of Alternative Sludge Thickening and Dewatering Technologies

The screening of the long list alternatives of solids treatment technologies is shown in Table 8-3 on the following page. Based on the screening completed in the following table, the only options that carry over are solids thickening by gravity thickening or by mechanical thickening. Based on the variety of solids thickening technologies, further screening is completed in Table 8-4 on various thickening technologies.

8.3 Short-List of Design Concepts

The following solids thickening treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Gravity Thickening
- Mechanical Thickening
  - Gravity Belt Thickener
  - Rotary Drum Thickener/Rotary Screw Thickener

Table 8-3 Screening of the Long List of Alternative Solids Alternatives

Long List of Alternative Solids Treatment Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Gravity Thickening	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation.
2. Mechanical Thickening	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation.
3. Dewatering	✗	✓	✓	✓	✗	✗	Eliminated due to incompatibility with the WRRF, construction impacts, and cost. In order to add in solids dewatering, the WRRF will be also required to upgrade solids thickening capacity prior to dewatering which will incur construction impacts and higher costs.

Table 8-4 Screening of the Long List of Alternative Solids Thickening Technologies

Long List of Alternative Solids Thickening Treatment Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Gravity Thickeners	✓	✓	✓	✓	✓	✓	Eliminated due to ....
2. Dissolved Air Flotation (DAF)	✗	✓	✓	✓	✗	✗	Proceed to detailed evaluation.
3. Centrifuge	✓	✓	✓	✗	✓	✓	
4. Gravity Belt Thickener (GBT)	✓	✓	✓	✓	✓	✓	
5. Rotary Drum Thickener/Rotary Screw Thickener	✓	✓	✓	✓	✓	✓	

9.0 Summary

The short lists for each of these stages of treatment will be carried over into the Technical Memo #3 to go through Stage 2 of the technology evaluation for the alternative design concepts for the Nobleton WRRF. The short lists for each stage are as follows in Table 9-1:

Table 9-1 Short-Listed Technology Alternatives for Each WRRF Treatment Process

WRRF Treatment Process	Short Listed Technology Alternative(s)	Notes
Coarse Screening	A. Climber Screen	Existing technology. This option would be used with conventional secondary treatment processes
Fine Screening	A. Perforated plate	This option would be used with secondary treatment in intensified secondary treatment processes
Grit Removal	A. Induced vortex	Existing technology
Primary Treatment	A. Primary Filtration	Primary treatment applies only to alternative wastewater design concepts that include primary treatment
Secondary Treatment - Conventional	A. Extended Aeration	Existing technology
Secondary Treatment - Intensification	A. Membrane-Aerated Biofilm Reactor	
Tertiary Treatment	A. Two-Stage sand filtration	Existing technology
Effluent Disinfection	A. Ultraviolet disinfection	Existing technology
Sludge Thickening	A. Gravity Thickener B. Mechanical Thickening	The short list is evaluated in this Section.

10.0 Bibliography

Black & Veatch. (January 2019). *Class Environmental Assessment for Water and Wastewater Servicing in the Community of Nobleton – Study 1B: Wastewater System Capacity Optimization Study.* Regional Municipality of York

Black & Veatch. (June 2019). *Class Environmental Assessment for Water and Wastewater Servicing in the Community of Nobleton – Study 1A: Water System Capacity Optimization Study.* Regional Municipality of York

Black & Veatch. (June 2019). *Phase 1: Identify the Problem or Opportunity. Technical Memo #1.* Regional Municipality of York

Hutchinson. (May 2019). *Humber River Assimilative Capacity Study 2017-2018 (DRAFT).* Nobleton: Regional Municipality of York

TSH. (February 2007). *Nobleton Sewage Works Design Report (DRAFT).* Nobleton: Regional Municipality of York, Slokker Canada Corporation

York Region. (2016). *The Regional Municipality of York Water and Wastewater Master Plan*

York Region. (2019). *The Regional Municipality of York. Characterization and Comprehensive Review of Groundwater Supply Resources of the Regional Municipality of York (York Region Groundwater Supply Options Study). In Support of the 2021 Water and Wastewater Master Plan Update*

Appendix B. Calculations for Storage Volume of the Flow Attenuation Tank at the Janet Avenue Pumping Station

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
1/1/2016	0:00:00	0.033045	0.033045	0	0.145	43.5	9.9135	-33.5865	
1/1/2016	0:05:00	0.033047	0.033047	0	0.145	43.5	9.9141	-33.5859	
1/1/2016	0:10:00	0.033031	0.03303	0	0.145	43.5	9.9093	-33.5907	
1/1/2016	0:15:00	0.032932	0.032929	0	0.145	43.5	9.8796	-33.6204	
1/1/2016	0:20:00	0.032679	0.032673	0	0.145	43.5	9.8037	-33.6963	
1/1/2016	0:25:00	0.032278	0.032268	0	0.145	43.5	9.6834	-33.8166	
1/1/2016	0:30:00	0.031225	0.031191	0	0.145	43.5	9.3675	-34.1325	
1/1/2016	0:35:00	0.02885	0.028797	0	0.145	43.5	8.655	-34.845	
1/1/2016	0:40:00	0.026621	0.026587	0	0.145	43.5	7.9863	-35.5137	
1/1/2016	0:45:00	0.0253	0.025278	0	0.145	43.5	7.59	-35.91	
1/1/2016	0:50:00	0.024415	0.0244	0	0.145	43.5	7.3245	-36.1755	
1/1/2016	0:55:00	0.025567	0.025685	0	0.145	43.5	7.6701	-35.8299	
1/1/2016	1:00:00	0.038756	0.038911	0	0.145	43.5	11.6268	-31.8732	
1/1/2016	1:05:00	0.03575	0.035609	0	0.145	43.5	10.725	-32.775	
1/1/2016	1:10:00	0.028486	0.028355	0	0.145	43.5	8.5458	-34.9542	
1/1/2016	1:15:00	0.023539	0.023466	0	0.145	43.5	7.0617	-36.4383	
1/1/2016	1:20:00	0.021031	0.020993	0	0.145	43.5	6.3093	-37.1907	
1/1/2016	1:25:00	0.019533	0.019506	0	0.145	43.5	5.8599	-37.6401	
1/1/2016	1:30:00	0.018448	0.018428	0	0.145	43.5	5.5344	-37.9656	
1/1/2016	1:35:00	0.017623	0.017607	0	0.145	43.5	5.2869	-38.2131	
1/1/2016	1:40:00	0.016914	0.0169	0	0.145	43.5	5.0742	-38.4258	
1/1/2016	1:45:00	0.018114	0.018241	0	0.145	43.5	5.4342	-38.0658	
1/1/2016	1:50:00	0.029965	0.030093	0	0.145	43.5	8.9895	-34.5105	
1/1/2016	1:55:00	0.027068	0.026936	0	0.145	43.5	8.1204	-35.3796	
1/1/2016	2:00:00	0.021362	0.021265	0	0.145	43.5	6.4086	-37.0914	
1/1/2016	2:05:00	0.017729	0.017665	0	0.145	43.5	5.3187	-38.1813	
1/1/2016	2:10:00	0.015076	0.015021	0	0.145	43.5	4.5228	-38.9772	
1/1/2016	2:15:00	0.013461	0.013439	0	0.145	43.5	4.0383	-39.4617	
1/1/2016	2:20:00	0.012902	0.012894	0	0.145	43.5	3.8706	-39.6294	
1/1/2016	2:25:00	0.01259	0.012583	0	0.145	43.5	3.777	-39.723	
1/1/2016	2:30:00	0.012319	0.012312	0	0.145	43.5	3.6957	-39.8043	
1/1/2016	2:35:00	0.012066	0.01206	0	0.145	43.5	3.6198	-39.8802	
1/1/2016	2:40:00	0.011839	0.011834	0	0.145	43.5	3.5517	-39.9483	
1/1/2016	2:45:00	0.011642	0.011637	0	0.145	43.5	3.4926	-40.0074	
1/1/2016	2:50:00	0.011471	0.011467	0	0.145	43.5	3.4413	-40.0587	
1/1/2016	2:55:00	0.011325	0.011321	0	0.145	43.5	3.3975	-40.1025	
1/1/2016	3:00:00	0.011613	0.011651	0	0.145	43.5	3.4839	-40.0161	
1/1/2016	3:05:00	0.020803	0.021082	0	0.145	43.5	6.2409	-37.2591	
1/1/2016	3:10:00	0.023196	0.023108	0	0.145	43.5	6.9588	-36.5412	
1/1/2016	3:15:00	0.01861	0.018518	0	0.145	43.5	5.583	-37.917	
1/1/2016	3:20:00	0.015089	0.015019	0	0.145	43.5	4.5267	-38.9733	
1/1/2016	3:25:00	0.01259	0.012549	0	0.145	43.5	3.777	-39.723	
1/1/2016	3:30:00	0.011551	0.011537	0	0.145	43.5	3.4653	-40.0347	
1/1/2016	3:35:00	0.011167	0.011161	0	0.145	43.5	3.3501	-40.1499	
1/1/2016	3:40:00	0.011018	0.011016	0	0.145	43.5	3.3054	-40.1946	
1/1/2016	3:45:00	0.010979	0.010979	0	0.145	43.5	3.2937	-40.2063	
1/1/2016	3:50:00	0.010979	0.010979	0	0.145	43.5	3.2937	-40.2063	
1/1/2016	3:55:00	0.010988	0.010989	0	0.145	43.5	3.2964	-40.2036	
1/1/2016	4:00:00	0.011003	0.011003	0	0.145	43.5	3.3009	-40.1991	
1/1/2016	4:05:00	0.011021	0.011022	0	0.145	43.5	3.3063	-40.1937	
1/1/2016	4:10:00	0.011046	0.011046	0	0.145	43.5	3.3138	-40.1862	
1/1/2016	4:15:00	0.011075	0.011076	0	0.145	43.5	3.3225	-40.1775	
1/1/2016	4:20:00	0.011113	0.011114	0	0.145	43.5	3.3339	-40.1661	
1/1/2016	4:25:00	0.011161	0.011162	0	0.145	43.5	3.3483	-40.1517	
1/1/2016	4:30:00	0.011227	0.011231	0	0.145	43.5	3.3681	-40.1319	
1/1/2016	4:35:00	0.014016	0.014235	0	0.145	43.5	4.2048	-39.2952	
1/1/2016	4:40:00	0.025514	0.02557	0	0.145	43.5	7.6542	-35.8458	
1/1/2016	4:45:00	0.022046	0.021941	0	0.145	43.5	6.6138	-36.8862	
1/1/2016	4:50:00	0.017715	0.017636	0	0.145	43.5	5.3145	-38.1855	
1/1/2016	4:55:00	0.014649	0.014586	0	0.145	43.5	4.3947	-39.1053	
1/1/2016	5:00:00	0.012712	0.012685	0	0.145	43.5	3.8136	-39.6864	
1/1/2016	5:05:00	0.012069	0.01206	0	0.145	43.5	3.6207	-39.8793	
1/1/2016	5:10:00	0.01189	0.011888	0	0.145	43.5	3.567	-39.933	
1/1/2016	5:15:00	0.011956	0.011961	0	0.145	43.5	3.5868	-39.9132	
1/1/2016	5:20:00	0.012287	0.012298	0	0.145	43.5	3.6861	-39.8139	
1/1/2016	5:25:00	0.012841	0.012855	0	0.145	43.5	3.8523	-39.6477	
1/1/2016	5:30:00	0.013473	0.013488	0	0.145	43.5	4.0419	-39.4581	
1/1/2016	5:35:00	0.014166	0.014183	0	0.145	43.5	4.2498	-39.2502	
1/1/2016	5:40:00	0.014939	0.014958	0	0.145	43.5	4.4817	-39.0183	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
1/1/2016	5:45:00	0.015805	0.015825	0	0.145	43.5	4.7415	-38.7585	
1/1/2016	5:50:00	0.01678	0.016803	0	0.145	43.5	5.034	-38.466	
1/1/2016	5:55:00	0.017911	0.017938	0	0.145	43.5	5.3733	-38.1267	
1/1/2016	6:00:00	0.019411	0.019455	5.829207	0.145	43.5	5.8233	-37.6767	
1/1/2016	6:05:00	0.028077	0.028448	2.502476	0.145	43.5	8.4231	-35.0769	
1/1/2016	6:10:00	0.037025	0.036985	2.4	0.145	43.5	11.1075	-32.3925	
1/1/2016	6:15:00	0.032644	0.032558	2.4	0.145	43.5	9.7932	-33.7068	
1/1/2016	6:20:00	0.029888	0.029866	2.4	0.145	43.5	8.9664	-34.5336	
1/1/2016	6:25:00	0.02999	0.030007	2.4	0.145	43.5	8.997	-34.503	
1/1/2016	6:30:00	0.031405	0.031442	3.56584	0.145	43.5	9.4215	-34.0785	
1/1/2016	6:35:00	0.033708	0.03376	3.600001	0.145	43.5	10.1124	-33.3876	
1/1/2016	6:40:00	0.038341	0.038504	2.434158	0.145	43.5	11.5023	-31.9977	
1/1/2016	6:45:00	0.057145	0.057473	3.56584	0.145	43.5	17.1435	-26.3565	
1/1/2016	6:50:00	0.059913	0.059831	3.600002	0.145	43.5	17.9739	-25.5261	
1/1/2016	6:55:00	0.054697	0.054648	3.599999	0.145	43.5	16.4091	-27.0909	
1/1/2016	7:00:00	0.053764	0.053777	5.931683	0.145	43.5	16.1292	-27.3708	
1/1/2016	7:05:00	0.057546	0.057686	3.668318	0.145	43.5	17.2638	-26.2362	
1/1/2016	7:10:00	0.078701	0.07896	3.599999	0.145	43.5	23.6103	-19.8897	
1/1/2016	7:15:00	0.083711	0.083661	3.599999	0.145	43.5	25.1133	-18.3867	
1/1/2016	7:20:00	0.076333	0.076262	3.599999	0.145	43.5	22.8999	-20.6001	
1/1/2016	7:25:00	0.073616	0.07364	5.931683	0.145	43.5	22.0848	-21.4152	
1/1/2016	7:30:00	0.086631	0.086768	2.502479	0.145	43.5	25.9893	-17.5107	
1/1/2016	7:35:00	0.100934	0.10092	5.897524	0.145	43.5	30.2802	-13.2198	
1/1/2016	7:40:00	0.093712	0.093552	3.668318	0.145	43.5	28.1136	-15.3864	
1/1/2016	7:45:00	0.087751	0.087754	2.434155	0.145	43.5	26.3253	-17.1747	
1/1/2016	7:50:00	0.102382	0.102609	2.400002	0.145	43.5	30.7146	-12.7854	
1/1/2016	7:55:00	0.114737	0.114732	2.400002	0.145	43.5	34.4211	-9.0789	
1/1/2016	8:00:00	0.107817	0.107749	10.560885	0.145	43.5	32.3451	-11.1549	
1/1/2016	8:05:00	0.098543	0.098525	9.634159	0.145	43.5	29.5629	-13.9371	
1/1/2016	8:10:00	0.108761	0.10891	17.760895	0.145	43.5	32.6283	-10.8717	
1/1/2016	8:15:00	0.123455	0.123464	56.472759	0.145	43.5	37.0365	-6.4635	
1/1/2016	8:20:00	0.120114	0.12008	58.765846	0.145	43.5	36.0342	-7.4658	
1/1/2016	8:25:00	0.115557	0.115568	75.121765	0.145	43.5	34.6671	-8.8329	
1/1/2016	8:30:00	0.134457	0.134499	86.092583	0.145	43.5	40.3371	-3.1629	
1/1/2016	8:35:00	0.162244	0.162251	68.912384	0.145	43.5	48.6732	5.1732	5.1732
1/1/2016	8:40:00	0.17309	0.173094	48.580708	0.145	43.5	51.927	8.427	13.6002
1/1/2016	8:45:00	0.182253	0.182256	42.170765	0.145	43.5	54.6759	11.1759	24.7761
1/1/2016	8:50:00	0.197177	0.197182	14.019817	0.145	43.5	59.1531	15.6531	40.4292
1/1/2016	8:55:00	0.217577	0.217582	7.3708	0.145	43.5	65.2731	21.7731	62.2023
1/1/2016	9:00:00	0.231932	0.231936	9.531677	0.145	43.5	69.5796	26.0796	88.2819
1/1/2016	9:05:00	0.240329	0.240331	9.599983	0.145	43.5	72.0987	28.5987	116.8806
1/1/2016	9:10:00	0.243231	0.243231	6.102473	0.145	43.5	72.9693	29.4693	146.3499
1/1/2016	9:15:00	0.239796	0.239794	2.502501	0.145	43.5	71.9388	28.4388	174.7887
1/1/2016	9:20:00	0.236582	0.236582	3.565818	0.145	43.5	70.9746	27.4746	202.2633
1/1/2016	9:25:00	0.238446	0.238447	2.434194	0.145	43.5	71.5338	28.0338	230.2971
1/1/2016	9:30:00	0.240574	0.240574	7.063329	0.145	43.5	72.1722	28.6722	258.9693
1/1/2016	9:35:00	0.241204	0.241204	23.521811	0.145	43.5	72.3612	28.8612	287.8305
1/1/2016	9:40:00	0.241073	0.241073	17.004936	0.145	43.5	72.3219	28.8219	316.6524
1/1/2016	9:45:00	0.241175	0.241175	20.297527	0.145	43.5	72.3525	28.8525	345.5049
1/1/2016	9:50:00	0.241999	0.241999	22.731676	0.145	43.5	72.5997	29.0997	374.6046
1/1/2016	9:55:00	0.241768	0.241767	7.644072	0.145	43.5	72.5304	29.0304	403.635
1/1/2016	10:00:00	0.237668	0.237667	0.204965	0.145	43.5	71.3004	27.8004	431.4354
1/1/2016	10:05:00	0.237147	0.237148	0	0.145	43.5	71.1441	27.6441	459.0795
1/1/2016	10:10:00	0.241782	0.241783	0	0.145	43.5	72.5346	29.0346	488.1141
1/1/2016	10:15:00	0.24557	0.245571	0	0.145	43.5	73.671	30.171	518.2851
1/1/2016	10:20:00	0.246727	0.246727	0	0.145	43.5	74.0181	30.5181	548.8032
1/1/2016	10:25:00	0.246032	0.246031	0	0.145	43.5	73.8096	30.3096	579.1128
1/1/2016	10:30:00	0.244153	0.244152	0	0.145	43.5	73.2459	29.7459	608.8587
1/1/2016	10:35:00	0.240824	0.240823	0	0.145	43.5	72.2472	28.7472	637.6059
1/1/2016	10:40:00	0.233123	0.233119	0	0.145	43.5	69.9369	26.4369	664.0428
1/1/2016	10:45:00	0.224503	0.224502	0	0.145	43.5	67.3509	23.8509	687.8937
1/1/2016	10:50:00	0.223622	0.223622	0	0.145	43.5	67.0866	23.5866	711.4803
1/1/2016	10:55:00	0.2251	0.2251	0	0.145	43.5	67.53	24.03	735.5103
1/1/2016	11:00:00	0.225619	0.225619	0	0.145	43.5	67.6857	24.1857	759.696
1/1/2016	11:05:00	0.224595	0.224594	0	0.145	43.5	67.3785	23.8785	783.5745
1/1/2016	11:10:00	0.218845	0.218842	0	0.145	43.5	65.6535	22.1535	805.728
1/1/2016	11:15:00	0.209593	0.20959	0	0.145	43.5	62.8779	19.3779	825.1059
1/1/2016	11:20:00	0.207718	0.207719	0	0.145	43.5	62.3154	18.8154	843.9213
1/1/2016	11:25:00	0.210372	0.210373	0	0.145	43.5	63.1116	19.6116	863.5329

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
1/1/2016	11:30:00	0.210928	0.210927	0	0.145	43.5	63.2784	19.7784	883.3113
1/1/2016	11:35:00	0.205854	0.205852	0	0.145	43.5	61.7562	18.2562	901.5675
1/1/2016	11:40:00	0.196566	0.196563	0	0.145	43.5	58.9698	15.4698	917.0373
1/1/2016	11:45:00	0.192808	0.192808	0	0.145	43.5	57.8424	14.3424	931.3797
1/1/2016	11:50:00	0.196879	0.19688	0	0.145	43.5	59.0637	15.5637	946.9434
1/1/2016	11:55:00	0.198257	0.198256	0	0.145	43.5	59.4771	15.9771	962.9205
1/1/2016	12:00:00	0.194056	0.194054	0	0.145	43.5	58.2168	14.7168	977.6373
1/1/2016	12:05:00	0.183703	0.183698	0	0.145	43.5	55.1109	11.6109	989.2482
1/1/2016	12:10:00	0.17792	0.177921	0	0.145	43.5	53.376	9.876	999.1242
1/1/2016	12:15:00	0.181841	0.181843	0	0.145	43.5	54.5523	11.0523	1010.1765
1/1/2016	12:20:00	0.184256	0.184256	0	0.145	43.5	55.2768	11.7768	1021.9533
1/1/2016	12:25:00	0.178775	0.17877	0	0.145	43.5	53.6325	10.1325	1032.0858
1/1/2016	12:30:00	0.167148	0.167144	0	0.145	43.5	50.1444	6.6444	1038.7302
1/1/2016	12:35:00	0.167776	0.167779	0	0.145	43.5	50.3328	6.8328	1045.563
1/1/2016	12:40:00	0.17325	0.173252	0	0.145	43.5	51.975	8.475	1054.038
1/1/2016	12:45:00	0.172551	0.172548	0	0.145	43.5	51.7653	8.2653	1062.3033
1/1/2016	12:50:00	0.159583	0.159572	0	0.145	43.5	47.8749	4.3749	1066.6782
1/1/2016	12:55:00	0.15201	0.152014	0	0.145	43.5	45.603	2.103	1068.7812
1/1/2016	13:00:00	0.163218	0.16322	0	0.145	43.5	48.9654	5.4654	1074.2466
1/1/2016	13:05:00	0.165506	0.165505	0	0.145	43.5	49.6518	6.1518	1080.3984
1/1/2016	13:10:00	0.15381	0.1538	0	0.145	43.5	46.143	2.643	1083.0414
1/1/2016	13:15:00	0.141819	0.141815	0	0.145	43.5	42.5457	-0.9543	1082.0871
1/1/2016	13:20:00	0.149443	0.149453	0	0.145	43.5	44.8329	1.3329	1083.42
1/1/2016	13:25:00	0.157511	0.15751	0	0.145	43.5	47.2533	3.7533	1087.1733
1/1/2016	13:30:00	0.148852	0.148844	0	0.145	43.5	44.6556	1.1556	1088.3289
1/1/2016	13:35:00	0.136139	0.136132	0	0.145	43.5	40.8417	-2.6583	
1/1/2016	13:40:00	0.138605	0.138616	0	0.145	43.5	41.5815	-1.9185	
1/1/2016	13:45:00	0.149422	0.149423	0	0.145	43.5	44.8266	1.3266	1.3266
1/1/2016	13:50:00	0.143481	0.143472	0	0.145	43.5	43.0443	-0.4557	
1/1/2016	13:55:00	0.130468	0.130459	0	0.145	43.5	39.1404	-4.3596	
1/1/2016	14:00:00	0.129848	0.129861	0	0.145	43.5	38.9544	-4.5456	
1/1/2016	14:05:00	0.142273	0.142318	0	0.145	43.5	42.6819	-0.8181	
1/1/2016	14:10:00	0.140738	0.140731	0	0.145	43.5	42.2214	-1.2786	
1/1/2016	14:15:00	0.127211	0.127224	0	0.145	43.5	38.1633	-5.3367	
1/1/2016	14:20:00	0.121126	0.121143	0	0.145	43.5	36.3378	-7.1622	
1/1/2016	14:25:00	0.133053	0.133066	0	0.145	43.5	39.9159	-3.5841	
1/1/2016	14:30:00	0.136168	0.136162	0	0.145	43.5	40.8504	-2.6496	
1/1/2016	14:35:00	0.123121	0.123072	0	0.145	43.5	36.9363	-6.5637	
1/1/2016	14:40:00	0.11333	0.113306	0	0.145	43.5	33.999	-9.501	
1/1/2016	14:45:00	0.122718	0.122774	0	0.145	43.5	36.8154	-6.6846	
1/1/2016	14:50:00	0.128574	0.128566	0	0.145	43.5	38.5722	-4.9278	
1/1/2016	14:55:00	0.117161	0.117087	0	0.145	43.5	35.1483	-8.3517	
1/1/2016	15:00:00	0.1058	0.105745	0	0.145	43.5	31.74	-11.76	
1/1/2016	15:05:00	0.112292	0.112414	0	0.145	43.5	33.6876	-9.8124	
1/1/2016	15:10:00	0.122749	0.122755	0	0.145	43.5	36.8247	-6.6753	
1/1/2016	15:15:00	0.113787	0.11373	0	0.145	43.5	34.1361	-9.3639	
1/1/2016	15:20:00	0.100818	0.100727	0	0.145	43.5	30.2454	-13.2546	
1/1/2016	15:25:00	0.100742	0.100841	0	0.145	43.5	30.2226	-13.2774	
1/1/2016	15:30:00	0.115688	0.115764	0	0.145	43.5	34.7064	-8.7936	
1/1/2016	15:35:00	0.113636	0.11358	0	0.145	43.5	34.0908	-9.4092	
1/1/2016	15:40:00	0.10091	0.100793	0	0.145	43.5	30.273	-13.227	
1/1/2016	15:45:00	0.092942	0.092923	0	0.145	43.5	27.8826	-15.6174	
1/1/2016	15:50:00	0.100861	0.101297	0	0.145	43.5	30.2583	-13.2417	
1/1/2016	15:55:00	0.11214	0.112132	0	0.145	43.5	33.642	-9.858	
1/1/2016	16:00:00	0.103051	0.10294	0	0.145	43.5	30.9153	-12.5847	
1/1/2016	16:05:00	0.091778	0.091717	0	0.145	43.5	27.5334	-15.9666	
1/1/2016	16:10:00	0.095035	0.095161	0	0.145	43.5	28.5105	-14.9895	
1/1/2016	16:15:00	0.108807	0.108834	0	0.145	43.5	32.6421	-10.8579	
1/1/2016	16:20:00	0.103115	0.103017	0	0.145	43.5	30.9345	-12.5655	
1/1/2016	16:25:00	0.091232	0.091136	0	0.145	43.5	27.3696	-16.1304	
1/1/2016	16:30:00	0.087836	0.087859	0	0.145	43.5	26.3508	-17.1492	
1/1/2016	16:35:00	0.102183	0.102526	0	0.145	43.5	30.6549	-12.8451	
1/1/2016	16:40:00	0.106115	0.106062	0	0.145	43.5	31.8345	-11.6655	
1/1/2016	16:45:00	0.095706	0.095603	0	0.145	43.5	28.7118	-14.7882	
1/1/2016	16:50:00	0.087332	0.087312	0	0.145	43.5	26.1996	-17.3004	
1/1/2016	16:55:00	0.094723	0.094907	0	0.145	43.5	28.4169	-15.0831	
1/1/2016	17:00:00	0.107205	0.107212	0	0.145	43.5	32.1615	-11.3385	
1/1/2016	17:05:00	0.099057	0.098944	0	0.145	43.5	29.7171	-13.7829	
1/1/2016	17:10:00	0.088453	0.088378	0	0.145	43.5	26.5359	-16.9641	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
1/1/2016	17:15:00	0.088002	0.088071	0	0.145	43.5	26.4006	-17.0994	
1/1/2016	17:20:00	0.104861	0.104952	0	0.145	43.5	31.4583	-12.0417	
1/1/2016	17:25:00	0.103451	0.103368	0	0.145	43.5	31.0353	-12.4647	
1/1/2016	17:30:00	0.091518	0.091387	0	0.145	43.5	27.4554	-16.0446	
1/1/2016	17:35:00	0.08692	0.086936	0	0.145	43.5	26.076	-17.424	
1/1/2016	17:40:00	0.100178	0.100298	0	0.145	43.5	30.0534	-13.4466	
1/1/2016	17:45:00	0.104255	0.104195	0	0.145	43.5	31.2765	-12.2235	
1/1/2016	17:50:00	0.093246	0.093059	0	0.145	43.5	27.9738	-15.5262	
1/1/2016	17:55:00	0.086222	0.086209	0	0.145	43.5	25.8666	-17.6334	
1/1/2016	18:00:00	0.09681	0.096967	0	0.145	43.5	29.043	-14.457	
1/1/2016	18:05:00	0.106381	0.106356	0	0.145	43.5	31.9143	-11.5857	
1/1/2016	18:10:00	0.097409	0.097357	0	0.145	43.5	29.2227	-14.2773	
1/1/2016	18:15:00	0.086565	0.086518	0	0.145	43.5	25.9695	-17.5305	
1/1/2016	18:20:00	0.085637	0.085712	0	0.145	43.5	25.6911	-17.8089	
1/1/2016	18:25:00	0.103276	0.10329	0	0.145	43.5	30.9828	-12.5172	
1/1/2016	18:30:00	0.103219	0.10315	0	0.145	43.5	30.9657	-12.5343	
1/1/2016	18:35:00	0.091525	0.091374	0	0.145	43.5	27.4575	-16.0425	
1/1/2016	18:40:00	0.084479	0.084443	0	0.145	43.5	25.3437	-18.1563	
1/1/2016	18:45:00	0.091588	0.091845	0	0.145	43.5	27.4764	-16.0236	
1/1/2016	18:50:00	0.106153	0.106189	0	0.145	43.5	31.8459	-11.6541	
1/1/2016	18:55:00	0.098859	0.098751	0	0.145	43.5	29.6577	-13.8423	
1/1/2016	19:00:00	0.087635	0.087568	0	0.145	43.5	26.2905	-17.2095	
1/1/2016	19:05:00	0.086328	0.086396	0	0.145	43.5	25.8984	-17.6016	
1/1/2016	19:10:00	0.10397	0.104022	0	0.145	43.5	31.191	-12.309	
1/1/2016	19:15:00	0.103292	0.103213	0	0.145	43.5	30.9876	-12.5124	
1/1/2016	19:20:00	0.090962	0.090815	0	0.145	43.5	27.2886	-16.2114	
1/1/2016	19:25:00	0.084225	0.084201	0	0.145	43.5	25.2675	-18.2325	
1/1/2016	19:30:00	0.094849	0.095034	0	0.145	43.5	28.4547	-15.0453	
1/1/2016	19:35:00	0.103697	0.103653	0	0.145	43.5	31.1091	-12.3909	
1/1/2016	19:40:00	0.093836	0.093659	0	0.145	43.5	28.1508	-15.3492	
1/1/2016	19:45:00	0.082249	0.082143	0	0.145	43.5	24.6747	-18.8253	
1/1/2016	19:50:00	0.083262	0.083363	0	0.145	43.5	24.9786	-18.5214	
1/1/2016	19:55:00	0.097333	0.097313	0	0.145	43.5	29.1999	-14.3001	
1/1/2016	20:00:00	0.095042	0.09492	0	0.145	43.5	28.5126	-14.9874	
1/1/2016	20:05:00	0.08122	0.081042	0	0.145	43.5	24.366	-19.134	
1/1/2016	20:10:00	0.07404	0.074018	0	0.145	43.5	22.212	-21.288	
1/1/2016	20:15:00	0.083953	0.084112	0	0.145	43.5	25.1859	-18.3141	
1/1/2016	20:20:00	0.092421	0.092406	0	0.145	43.5	27.7263	-15.7737	
1/1/2016	20:25:00	0.082941	0.082785	0	0.145	43.5	24.8823	-18.6177	
1/1/2016	20:30:00	0.071839	0.071752	0	0.145	43.5	21.5517	-21.9483	
1/1/2016	20:35:00	0.072433	0.072576	0	0.145	43.5	21.7299	-21.7701	
1/1/2016	20:40:00	0.087058	0.087145	0	0.145	43.5	26.1174	-17.3826	
1/1/2016	20:45:00	0.085037	0.084912	0	0.145	43.5	25.5111	-17.9889	
1/1/2016	20:50:00	0.072052	0.071941	0	0.145	43.5	21.6156	-21.8844	
1/1/2016	20:55:00	0.065929	0.065904	0	0.145	43.5	19.7787	-23.7213	
1/1/2016	21:00:00	0.075876	0.076145	0	0.145	43.5	22.7628	-20.7372	
1/1/2016	21:05:00	0.084416	0.084388	0	0.145	43.5	25.3248	-18.1752	
1/1/2016	21:10:00	0.07374	0.073616	0	0.145	43.5	22.122	-21.378	
1/1/2016	21:15:00	0.065583	0.065506	0	0.145	43.5	19.6749	-23.8251	
1/1/2016	21:20:00	0.066117	0.066241	0	0.145	43.5	19.8351	-23.6649	
1/1/2016	21:25:00	0.081648	0.081736	0	0.145	43.5	24.4944	-19.0056	
1/1/2016	21:30:00	0.076566	0.076442	0	0.145	43.5	22.9698	-20.5302	
1/1/2016	21:35:00	0.066866	0.066755	0	0.145	43.5	20.0598	-23.4402	
1/1/2016	21:40:00	0.062439	0.062457	0	0.145	43.5	18.7317	-24.7683	
1/1/2016	21:45:00	0.074755	0.075004	0	0.145	43.5	22.4265	-21.0735	
1/1/2016	21:50:00	0.079834	0.079745	0	0.145	43.5	23.9502	-19.5498	
1/1/2016	21:55:00	0.070081	0.069958	0	0.145	43.5	21.0243	-22.4757	
1/1/2016	22:00:00	0.062366	0.062297	0	0.145	43.5	18.7098	-24.7902	
1/1/2016	22:05:00	0.064022	0.064159	0	0.145	43.5	19.2066	-24.2934	
1/1/2016	22:10:00	0.080624	0.080721	0	0.145	43.5	24.1872	-19.3128	
1/1/2016	22:15:00	0.075722	0.075598	0	0.145	43.5	22.7166	-20.7834	
1/1/2016	22:20:00	0.06582	0.065703	0	0.145	43.5	19.746	-23.754	
1/1/2016	22:25:00	0.061196	0.061212	0	0.145	43.5	18.3588	-25.1412	
1/1/2016	22:30:00	0.073071	0.073311	0	0.145	43.5	21.9213	-21.5787	
1/1/2016	22:35:00	0.077954	0.077863	0	0.145	43.5	23.3862	-20.1138	
1/1/2016	22:40:00	0.06799	0.067858	0	0.145	43.5	20.397	-23.103	
1/1/2016	22:45:00	0.059699	0.05962	0	0.145	43.5	17.9097	-25.5903	
1/1/2016	22:50:00	0.060194	0.060311	0	0.145	43.5	18.0582	-25.4418	
1/1/2016	22:55:00	0.075818	0.075943	0	0.145	43.5	22.7454	-20.7546	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
1/1/2016	23:00:00	0.071739	0.071615	0	0.145	43.5	21.5217	-21.9783	
1/1/2016	23:05:00	0.061503	0.06137	0	0.145	43.5	18.4509	-25.0491	
1/1/2016	23:10:00	0.055623	0.055595	0	0.145	43.5	16.6869	-26.8131	
1/1/2016	23:15:00	0.062853	0.063089	0	0.145	43.5	18.8559	-24.6441	
1/1/2016	23:20:00	0.074521	0.074482	0	0.145	43.5	22.3563	-21.1437	
1/1/2016	23:25:00	0.066091	0.065957	0	0.145	43.5	19.8273	-23.6727	
1/1/2016	23:30:00	0.056998	0.056902	0	0.145	43.5	17.0994	-26.4006	
1/1/2016	23:35:00	0.053737	0.053731	0	0.145	43.5	16.1211	-27.3789	
1/1/2016	23:40:00	0.061704	0.061954	0	0.145	43.5	18.5112	-24.9888	
1/1/2016	23:45:00	0.074561	0.074536	0	0.145	43.5	22.3683	-21.1317	
1/1/2016	23:50:00	0.066653	0.066522	0	0.145	43.5	19.9959	-23.5041	
1/1/2016	23:55:00	0.057824	0.057732	0	0.145	43.5	17.3472	-26.1528	
2/1/2016	0:00:00	0.054821	0.054821	0	0.145	43.5	16.4463	-27.0537	
2/1/2016	0:05:00	0.063525	0.063779	0	0.145	43.5	19.0575	-24.4425	
2/1/2016	0:10:00	0.075785	0.075759	0	0.145	43.5	22.7355	-20.7645	
2/1/2016	0:15:00	0.067578	0.067446	0	0.145	43.5	20.2734	-23.2266	
2/1/2016	0:20:00	0.058598	0.058503	0	0.145	43.5	17.5794	-25.9206	
2/1/2016	0:25:00	0.055847	0.05587	0	0.145	43.5	16.7541	-26.7459	
2/1/2016	0:30:00	0.067031	0.0673	0	0.145	43.5	20.1093	-23.3907	
2/1/2016	0:35:00	0.074132	0.074051	0	0.145	43.5	22.2396	-21.2604	
2/1/2016	0:40:00	0.064542	0.064401	0	0.145	43.5	19.3626	-24.1374	
2/1/2016	0:45:00	0.056095	0.056015	0	0.145	43.5	16.8285	-26.6715	
2/1/2016	0:50:00	0.056485	0.056607	0	0.145	43.5	16.9455	-26.5545	
2/1/2016	0:55:00	0.072221	0.07236	0	0.145	43.5	21.6663	-21.8337	
2/1/2016	1:00:00	0.068624	0.068496	0	0.145	43.5	20.5872	-22.9128	
2/1/2016	1:05:00	0.058163	0.058025	0	0.145	43.5	17.4489	-26.0511	
2/1/2016	1:10:00	0.051982	0.051927	0	0.145	43.5	15.5946	-27.9054	
2/1/2016	1:15:00	0.054999	0.055173	0	0.145	43.5	16.4997	-27.0003	
2/1/2016	1:20:00	0.069962	0.070022	0	0.145	43.5	20.9886	-22.5114	
2/1/2016	1:25:00	0.063717	0.063574	0	0.145	43.5	19.1151	-24.3849	
2/1/2016	1:30:00	0.053155	0.05303	0	0.145	43.5	15.9465	-27.5535	
2/1/2016	1:35:00	0.047673	0.04762	0	0.145	43.5	14.3019	-29.1981	
2/1/2016	1:40:00	0.047143	0.04722	0	0.145	43.5	14.1429	-29.3571	
2/1/2016	1:45:00	0.061687	0.061892	0	0.145	43.5	18.5061	-24.9939	
2/1/2016	1:50:00	0.06137	0.061234	0	0.145	43.5	18.411	-25.089	
2/1/2016	1:55:00	0.050067	0.049899	0	0.145	43.5	15.0201	-28.4799	
2/1/2016	2:00:00	0.043557	0.043485	0	0.145	43.5	13.0671	-30.4329	
2/1/2016	2:05:00	0.040167	0.040122	0	0.145	43.5	12.0501	-31.4499	
2/1/2016	2:10:00	0.04217	0.042355	0	0.145	43.5	12.651	-30.849	
2/1/2016	2:15:00	0.055144	0.05521	0	0.145	43.5	16.5432	-26.9568	
2/1/2016	2:20:00	0.048882	0.048721	0	0.145	43.5	14.6646	-28.8354	
2/1/2016	2:25:00	0.040482	0.04037	0	0.145	43.5	12.1446	-31.3554	
2/1/2016	2:30:00	0.035131	0.035047	0	0.145	43.5	10.5393	-32.9607	
2/1/2016	2:35:00	0.032164	0.032131	0	0.145	43.5	9.6492	-33.8508	
2/1/2016	2:40:00	0.0315	0.031519	0	0.145	43.5	9.45	-34.05	
2/1/2016	2:45:00	0.040643	0.040943	0	0.145	43.5	12.1929	-31.3071	
2/1/2016	2:50:00	0.045653	0.045554	0	0.145	43.5	13.6959	-29.8041	
2/1/2016	2:55:00	0.038011	0.037866	0	0.145	43.5	11.4033	-32.0967	
2/1/2016	3:00:00	0.031415	0.031304	0	0.145	43.5	9.4245	-34.0755	
2/1/2016	3:05:00	0.027465	0.027416	0	0.145	43.5	8.2395	-35.2605	
2/1/2016	3:10:00	0.025807	0.025782	0	0.145	43.5	7.7421	-35.7579	
2/1/2016	3:15:00	0.024786	0.024768	0	0.145	43.5	7.4358	-36.0642	
2/1/2016	3:20:00	0.024049	0.02404	0	0.145	43.5	7.2147	-36.2853	
2/1/2016	3:25:00	0.028844	0.02912	0	0.145	43.5	8.6532	-34.8468	
2/1/2016	3:30:00	0.039348	0.039324	0	0.145	43.5	11.8044	-31.6956	
2/1/2016	3:35:00	0.033163	0.033021	0	0.145	43.5	9.9489	-33.5511	
2/1/2016	3:40:00	0.026562	0.026454	0	0.145	43.5	7.9686	-35.5314	
2/1/2016	3:45:00	0.022787	0.022732	0	0.145	43.5	6.8361	-36.6639	
2/1/2016	3:50:00	0.020784	0.020748	0	0.145	43.5	6.2352	-37.2648	
2/1/2016	3:55:00	0.019391	0.019366	0	0.145	43.5	5.8173	-37.6827	
2/1/2016	4:00:00	0.018527	0.018512	0	0.145	43.5	5.5581	-37.9419	
2/1/2016	4:05:00	0.017913	0.0179	0	0.145	43.5	5.3739	-38.1261	
2/1/2016	4:10:00	0.01735	0.017338	0	0.145	43.5	5.205	-38.295	
2/1/2016	4:15:00	0.016825	0.016815	0	0.145	43.5	5.0475	-38.4525	
2/1/2016	4:20:00	0.016655	0.01667	0	0.145	43.5	4.9965	-38.5035	
2/1/2016	4:25:00	0.024448	0.024762	0	0.145	43.5	7.3344	-36.1656	
2/1/2016	4:30:00	0.029203	0.02912	0	0.145	43.5	8.7609	-34.7391	
2/1/2016	4:35:00	0.023656	0.023549	0	0.145	43.5	7.0968	-36.4032	
2/1/2016	4:40:00	0.019632	0.019567	0	0.145	43.5	5.8896	-37.6104	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
2/1/2016	4:45:00	0.017011	0.016956	0	0.145	43.5	5.1033	-38.3967	
2/1/2016	4:50:00	0.015102	0.015076	0	0.145	43.5	4.5306	-38.9694	
2/1/2016	4:55:00	0.014518	0.014511	0	0.145	43.5	4.3554	-39.1446	
2/1/2016	5:00:00	0.014322	0.014319	0	0.145	43.5	4.2966	-39.2034	
2/1/2016	5:05:00	0.014202	0.014199	0	0.145	43.5	4.2606	-39.2394	
2/1/2016	5:10:00	0.014107	0.014105	0	0.145	43.5	4.2321	-39.2679	
2/1/2016	5:15:00	0.014026	0.014024	0	0.145	43.5	4.2078	-39.2922	
2/1/2016	5:20:00	0.013957	0.013955	0	0.145	43.5	4.1871	-39.3129	
2/1/2016	5:25:00	0.013897	0.013895	0	0.145	43.5	4.1691	-39.3309	
2/1/2016	5:30:00	0.013842	0.01384	0	0.145	43.5	4.1526	-39.3474	
2/1/2016	5:35:00	0.013941	0.013954	0	0.145	43.5	4.1823	-39.3177	
2/1/2016	5:40:00	0.019871	0.020161	0	0.145	43.5	5.9613	-37.5387	
2/1/2016	5:45:00	0.027206	0.027159	0	0.145	43.5	8.1618	-35.3382	
2/1/2016	5:50:00	0.022519	0.022418	0	0.145	43.5	6.7557	-36.7443	
2/1/2016	5:55:00	0.018565	0.018497	0	0.145	43.5	5.5695	-37.9305	
2/1/2016	6:00:00	0.01575	0.015692	0	0.145	43.5	4.725	-38.775	
2/1/2016	6:05:00	0.014106	0.014087	0	0.145	43.5	4.2318	-39.2682	
2/1/2016	6:10:00	0.013678	0.013673	0	0.145	43.5	4.1034	-39.3966	
2/1/2016	6:15:00	0.013573	0.013572	0	0.145	43.5	4.0719	-39.4281	
2/1/2016	6:20:00	0.013556	0.013556	0	0.145	43.5	4.0668	-39.4332	
2/1/2016	6:25:00	0.013562	0.013562	0	0.145	43.5	4.0686	-39.4314	
2/1/2016	6:30:00	0.013573	0.013573	0	0.145	43.5	4.0719	-39.4281	
2/1/2016	6:35:00	0.013588	0.013588	0	0.145	43.5	4.0764	-39.4236	
2/1/2016	6:40:00	0.013607	0.013608	0	0.145	43.5	4.0821	-39.4179	
2/1/2016	6:45:00	0.013632	0.013633	0	0.145	43.5	4.0896	-39.4104	
2/1/2016	6:50:00	0.013662	0.013663	0	0.145	43.5	4.0986	-39.4014	
2/1/2016	6:55:00	0.013714	0.013717	0	0.145	43.5	4.1142	-39.3858	
2/1/2016	7:00:00	0.015903	0.016065	0	0.145	43.5	4.7709	-38.7291	
2/1/2016	7:05:00	0.027849	0.027963	0	0.145	43.5	8.3547	-35.1453	
2/1/2016	7:10:00	0.025152	0.02504	0	0.145	43.5	7.5456	-35.9544	
2/1/2016	7:15:00	0.020499	0.02042	0	0.145	43.5	6.1497	-37.3503	
2/1/2016	7:20:00	0.017722	0.017673	0	0.145	43.5	5.3166	-38.1834	
2/1/2016	7:25:00	0.015895	0.01587	0	0.145	43.5	4.7685	-38.7315	
2/1/2016	7:30:00	0.015642	0.015649	0	0.145	43.5	4.6926	-38.8074	
2/1/2016	7:35:00	0.016129	0.016143	0	0.145	43.5	4.8387	-38.6613	
2/1/2016	7:40:00	0.016841	0.016858	0	0.145	43.5	5.0523	-38.4477	
2/1/2016	7:45:00	0.017699	0.01772	0	0.145	43.5	5.3097	-38.1903	
2/1/2016	7:50:00	0.018695	0.018718	0	0.145	43.5	5.6085	-37.8915	
2/1/2016	7:55:00	0.019841	0.019867	0	0.145	43.5	5.9523	-37.5477	
2/1/2016	8:00:00	0.021146	0.021174	0	0.145	43.5	6.3438	-37.1562	
2/1/2016	8:05:00	0.022546	0.022575	0	0.145	43.5	6.7638	-36.7362	
2/1/2016	8:10:00	0.02409	0.02413	0	0.145	43.5	7.227	-36.273	
2/1/2016	8:15:00	0.033024	0.033389	0	0.145	43.5	9.9072	-33.5928	
2/1/2016	8:20:00	0.04297	0.042951	0	0.145	43.5	12.891	-30.609	
2/1/2016	8:25:00	0.03882	0.038737	0	0.145	43.5	11.646	-31.854	
2/1/2016	8:30:00	0.035299	0.035261	0	0.145	43.5	10.5897	-32.9103	
2/1/2016	8:35:00	0.035235	0.035259	0	0.145	43.5	10.5705	-32.9295	
2/1/2016	8:40:00	0.037316	0.037368	0	0.145	43.5	11.1948	-32.3052	
2/1/2016	8:45:00	0.042287	0.042447	0	0.145	43.5	12.6861	-30.8139	
2/1/2016	8:50:00	0.061616	0.061835	0	0.145	43.5	18.4848	-25.0152	
2/1/2016	8:55:00	0.062249	0.062153	0	0.145	43.5	18.6747	-24.8253	
2/1/2016	9:00:00	0.056153	0.056096	0	0.145	43.5	16.8459	-26.6541	
2/1/2016	9:05:00	0.054869	0.05488	0	0.145	43.5	16.4607	-27.0393	
2/1/2016	9:10:00	0.060622	0.060837	0	0.145	43.5	18.1866	-25.3134	
2/1/2016	9:15:00	0.081568	0.081712	0	0.145	43.5	24.4704	-19.0296	
2/1/2016	9:20:00	0.079237	0.079147	0	0.145	43.5	23.7711	-19.7289	
2/1/2016	9:25:00	0.07164	0.071566	0	0.145	43.5	21.492	-22.008	
2/1/2016	9:30:00	0.071819	0.071931	0	0.145	43.5	21.5457	-21.9543	
2/1/2016	9:35:00	0.088338	0.088514	0	0.145	43.5	26.5014	-16.9986	
2/1/2016	9:40:00	0.091861	0.091775	0	0.145	43.5	27.5583	-15.9417	
2/1/2016	9:45:00	0.082006	0.081882	0	0.145	43.5	24.6018	-18.8982	
2/1/2016	9:50:00	0.0802	0.080311	0	0.145	43.5	24.06	-19.44	
2/1/2016	9:55:00	0.097258	0.097339	0	0.145	43.5	29.1774	-14.3226	
2/1/2016	10:00:00	0.099085	0.099016	0	0.145	43.5	29.7255	-13.7745	
2/1/2016	10:05:00	0.088101	0.087989	0	0.145	43.5	26.4303	-17.0697	
2/1/2016	10:10:00	0.084249	0.084282	0	0.145	43.5	25.2747	-18.2253	
2/1/2016	10:15:00	0.101237	0.101577	0	0.145	43.5	30.3711	-13.1289	
2/1/2016	10:20:00	0.107357	0.107306	0	0.145	43.5	32.2071	-11.2929	
2/1/2016	10:25:00	0.096745	0.096689	0	0.145	43.5	29.0235	-14.4765	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
2/1/2016	10:30:00	0.088423	0.088462	0	0.145	43.5	26.5269	-16.9731	
2/1/2016	10:35:00	0.103479	0.103719	0	0.145	43.5	31.0437	-12.4563	
2/1/2016	10:40:00	0.106595	0.106536	0	0.145	43.5	31.9785	-11.5215	
2/1/2016	10:45:00	0.095452	0.095338	0	0.145	43.5	28.6356	-14.8644	
2/1/2016	10:50:00	0.086671	0.086672	0	0.145	43.5	26.0013	-17.4987	
2/1/2016	10:55:00	0.098535	0.09862	0	0.145	43.5	29.5605	-13.9395	
2/1/2016	11:00:00	0.103146	0.103091	0	0.145	43.5	30.9438	-12.5562	
2/1/2016	11:05:00	0.091642	0.091465	0	0.145	43.5	27.4926	-16.0074	
2/1/2016	11:10:00	0.081409	0.081312	0	0.145	43.5	24.4227	-19.0773	
2/1/2016	11:15:00	0.090961	0.091285	0	0.145	43.5	27.2883	-16.2117	
2/1/2016	11:20:00	0.101387	0.101385	0	0.145	43.5	30.4161	-13.0839	
2/1/2016	11:25:00	0.09184	0.091659	0	0.145	43.5	27.552	-15.948	
2/1/2016	11:30:00	0.079471	0.079337	0	0.145	43.5	23.8413	-19.6587	
2/1/2016	11:35:00	0.081794	0.081961	0	0.145	43.5	24.5382	-18.9618	
2/1/2016	11:40:00	0.097003	0.097077	0	0.145	43.5	29.1009	-14.3991	
2/1/2016	11:45:00	0.094984	0.094855	0	0.145	43.5	28.4952	-15.0048	
2/1/2016	11:50:00	0.082243	0.082083	0	0.145	43.5	24.6729	-18.8271	
2/1/2016	11:55:00	0.075179	0.075202	0	0.145	43.5	22.5537	-20.9463	
2/1/2016	12:00:00	0.087611	0.087764	0	0.145	43.5	26.2833	-17.2167	
2/1/2016	12:05:00	0.094358	0.094298	0	0.145	43.5	28.3074	-15.1926	
2/1/2016	12:10:00	0.083327	0.083168	0	0.145	43.5	24.9981	-18.5019	
2/1/2016	12:15:00	0.073617	0.073583	0	0.145	43.5	22.0851	-21.4149	
2/1/2016	12:20:00	0.083354	0.083534	0	0.145	43.5	25.0062	-18.4938	
2/1/2016	12:25:00	0.092846	0.092855	0	0.145	43.5	27.8538	-15.6462	
2/1/2016	12:30:00	0.084336	0.084187	0	0.145	43.5	25.3008	-18.1992	
2/1/2016	12:35:00	0.072384	0.072302	0	0.145	43.5	21.7152	-21.7848	
2/1/2016	12:40:00	0.073745	0.073909	0	0.145	43.5	22.1235	-21.3765	
2/1/2016	12:45:00	0.088082	0.088154	0	0.145	43.5	26.4246	-17.0754	
2/1/2016	12:50:00	0.087208	0.087142	0	0.145	43.5	26.1624	-17.3376	
2/1/2016	12:55:00	0.073623	0.073486	0	0.145	43.5	22.0869	-21.4131	
2/1/2016	13:00:00	0.06809	0.068098	0	0.145	43.5	20.427	-23.073	
2/1/2016	13:05:00	0.081679	0.081879	0	0.145	43.5	24.5037	-18.9963	
2/1/2016	13:10:00	0.086295	0.08625	0	0.145	43.5	25.8885	-17.6115	
2/1/2016	13:15:00	0.073731	0.073579	0	0.145	43.5	22.1193	-21.3807	
2/1/2016	13:20:00	0.066007	0.065965	0	0.145	43.5	19.8021	-23.6979	
2/1/2016	13:25:00	0.074814	0.075055	0	0.145	43.5	22.4442	-21.0558	
2/1/2016	13:30:00	0.084188	0.084178	0	0.145	43.5	25.2564	-18.2436	
2/1/2016	13:35:00	0.073587	0.073444	0	0.145	43.5	22.0761	-21.4239	
2/1/2016	13:40:00	0.064254	0.064168	0	0.145	43.5	19.2762	-24.2238	
2/1/2016	13:45:00	0.067118	0.067329	0	0.145	43.5	20.1354	-23.3646	
2/1/2016	13:50:00	0.081372	0.081393	0	0.145	43.5	24.4116	-19.0884	
2/1/2016	13:55:00	0.072969	0.072824	0	0.145	43.5	21.8907	-21.6093	
2/1/2016	14:00:00	0.062495	0.062372	0	0.145	43.5	18.7485	-24.7515	
2/1/2016	14:05:00	0.058989	0.059048	0	0.145	43.5	17.6967	-25.8033	
2/1/2016	14:10:00	0.074062	0.074244	0	0.145	43.5	22.2186	-21.2814	
2/1/2016	14:15:00	0.072103	0.071979	0	0.145	43.5	21.6309	-21.8691	
2/1/2016	14:20:00	0.061715	0.061572	0	0.145	43.5	18.5145	-24.9855	
2/1/2016	14:25:00	0.05495	0.054911	0	0.145	43.5	16.485	-27.015	
2/1/2016	14:30:00	0.062428	0.062688	0	0.145	43.5	18.7284	-24.7716	
2/1/2016	14:35:00	0.074994	0.074959	0	0.145	43.5	22.4982	-21.0018	
2/1/2016	14:40:00	0.066354	0.066219	0	0.145	43.5	19.9062	-23.5938	
2/1/2016	14:45:00	0.056349	0.056234	0	0.145	43.5	16.9047	-26.5953	
2/1/2016	14:50:00	0.051783	0.051764	0	0.145	43.5	15.5349	-27.9651	
2/1/2016	14:55:00	0.058731	0.058981	0	0.145	43.5	17.6193	-25.8807	
2/1/2016	15:00:00	0.071292	0.071268	0	0.145	43.5	21.3876	-22.1124	
2/1/2016	15:05:00	0.063113	0.062968	0	0.145	43.5	18.9339	-24.5661	
2/1/2016	15:10:00	0.053896	0.053801	0	0.145	43.5	16.1688	-27.3312	
2/1/2016	15:15:00	0.05183	0.051889	0	0.145	43.5	15.549	-27.951	
2/1/2016	15:20:00	0.065316	0.065553	0	0.145	43.5	19.5948	-23.9052	
2/1/2016	15:25:00	0.067983	0.067885	0	0.145	43.5	20.3949	-23.1051	
2/1/2016	15:30:00	0.058194	0.058048	0	0.145	43.5	17.4582	-26.0418	
2/1/2016	15:35:00	0.051213	0.05115	0	0.145	43.5	15.3639	-28.1361	
2/1/2016	15:40:00	0.051573	0.051683	0	0.145	43.5	15.4719	-28.0281	
2/1/2016	15:45:00	0.066513	0.0667	0	0.145	43.5	19.9539	-23.5461	
2/1/2016	15:50:00	0.065946	0.065832	0	0.145	43.5	19.7838	-23.7162	
2/1/2016	15:55:00	0.055967	0.055835	0	0.145	43.5	16.7901	-26.7099	
2/1/2016	16:00:00	0.050231	0.050183	0	0.145	43.5	15.0693	-28.4307	
2/1/2016	16:05:00	0.053115	0.05329	0	0.145	43.5	15.9345	-27.5655	
2/1/2016	16:10:00	0.068428	0.068521	0	0.145	43.5	20.5284	-22.9716	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
2/1/2016	16:15:00	0.063717	0.063581	0	0.145	43.5	19.1151	-24.3849	
2/1/2016	16:20:00	0.053689	0.053573	0	0.145	43.5	16.1067	-27.3933	
2/1/2016	16:25:00	0.048858	0.048822	0	0.145	43.5	14.6574	-28.8426	
2/1/2016	16:30:00	0.052268	0.052461	0	0.145	43.5	15.6804	-27.8196	
2/1/2016	16:35:00	0.067301	0.067389	0	0.145	43.5	20.1903	-23.3097	
2/1/2016	16:40:00	0.06224	0.0621	0	0.145	43.5	18.672	-24.828	
2/1/2016	16:45:00	0.051971	0.051844	0	0.145	43.5	15.5913	-27.9087	
2/1/2016	16:50:00	0.046972	0.046931	0	0.145	43.5	14.0916	-29.4084	
2/1/2016	16:55:00	0.051514	0.051739	0	0.145	43.5	15.4542	-28.0458	
2/1/2016	17:00:00	0.064644	0.064682	0	0.145	43.5	19.3932	-24.1068	
2/1/2016	17:05:00	0.057698	0.057529	0	0.145	43.5	17.3094	-26.1906	
2/1/2016	17:10:00	0.048203	0.048101	0	0.145	43.5	14.4609	-29.0391	
2/1/2016	17:15:00	0.04405	0.044009	0	0.145	43.5	13.215	-30.285	
2/1/2016	17:20:00	0.043797	0.043866	0	0.145	43.5	13.1391	-30.3609	
2/1/2016	17:25:00	0.058524	0.058762	0	0.145	43.5	17.5572	-25.9428	
2/1/2016	17:30:00	0.060325	0.060206	0	0.145	43.5	18.0975	-25.4025	
2/1/2016	17:35:00	0.05004	0.049919	0	0.145	43.5	15.012	-28.488	
2/1/2016	17:40:00	0.044341	0.044281	0	0.145	43.5	13.3023	-30.1977	
2/1/2016	17:45:00	0.041978	0.041954	0	0.145	43.5	12.5934	-30.9066	
2/1/2016	17:50:00	0.043873	0.044019	0	0.145	43.5	13.1619	-30.3381	
2/1/2016	17:55:00	0.060214	0.06039	0	0.145	43.5	18.0642	-25.4358	
2/1/2016	18:00:00	0.058455	0.058317	0	0.145	43.5	17.5365	-25.9635	
2/1/2016	18:05:00	0.048837	0.048728	0	0.145	43.5	14.6511	-28.8489	
2/1/2016	18:10:00	0.04413	0.044082	0	0.145	43.5	13.239	-30.261	
2/1/2016	18:15:00	0.04249	0.042486	0	0.145	43.5	12.747	-30.753	
2/1/2016	18:20:00	0.05027	0.050564	0	0.145	43.5	15.081	-28.419	
2/1/2016	18:25:00	0.062774	0.062781	0	0.145	43.5	18.8322	-24.6678	
2/1/2016	18:30:00	0.055312	0.05516	0	0.145	43.5	16.5936	-26.9064	
2/1/2016	18:35:00	0.047843	0.047773	0	0.145	43.5	14.3529	-29.1471	
2/1/2016	18:40:00	0.044956	0.04493	0	0.145	43.5	13.4868	-30.0132	
2/1/2016	18:45:00	0.045556	0.045636	0	0.145	43.5	13.6668	-29.8332	
2/1/2016	18:50:00	0.061088	0.061333	0	0.145	43.5	18.3264	-25.1736	
2/1/2016	18:55:00	0.063754	0.063647	0	0.145	43.5	19.1262	-24.3738	
2/1/2016	19:00:00	0.054203	0.05408	0	0.145	43.5	16.2609	-27.2391	
2/1/2016	19:05:00	0.048907	0.048863	0	0.145	43.5	14.6721	-28.8279	
2/1/2016	19:10:00	0.047891	0.04791	0	0.145	43.5	14.3673	-29.1327	
2/1/2016	19:15:00	0.05896	0.059259	0	0.145	43.5	17.688	-25.812	
2/1/2016	19:20:00	0.06898	0.068933	0	0.145	43.5	20.694	-22.806	
2/1/2016	19:25:00	0.06051	0.060366	0	0.145	43.5	18.153	-25.347	
2/1/2016	19:30:00	0.052798	0.052724	0	0.145	43.5	15.8394	-27.6606	
2/1/2016	19:35:00	0.051533	0.05159	0	0.145	43.5	15.4599	-28.0401	
2/1/2016	19:40:00	0.065495	0.065759	0	0.145	43.5	19.6485	-23.8515	
2/1/2016	19:45:00	0.069344	0.069253	0	0.145	43.5	20.8032	-22.6968	
2/1/2016	19:50:00	0.059898	0.059758	0	0.145	43.5	17.9694	-25.5306	
2/1/2016	19:55:00	0.053258	0.053203	0	0.145	43.5	15.9774	-27.5226	
2/1/2016	20:00:00	0.054709	0.054845	0	0.145	43.5	16.4127	-27.0873	
2/1/2016	20:05:00	0.071383	0.071538	0	0.145	43.5	21.4149	-22.0851	
2/1/2016	20:10:00	0.068981	0.068867	0	0.145	43.5	20.6943	-22.8057	
2/1/2016	20:15:00	0.059304	0.05918	0	0.145	43.5	17.7912	-25.7088	
2/1/2016	20:20:00	0.054273	0.054255	0	0.145	43.5	16.2819	-27.2181	
2/1/2016	20:25:00	0.062579	0.06285	0	0.145	43.5	18.7737	-24.7263	
2/1/2016	20:30:00	0.074874	0.074833	0	0.145	43.5	22.4622	-21.0378	
2/1/2016	20:35:00	0.066602	0.066472	0	0.145	43.5	19.9806	-23.5194	
2/1/2016	20:40:00	0.05816	0.05808	0	0.145	43.5	17.448	-26.052	
2/1/2016	20:45:00	0.059427	0.05958	0	0.145	43.5	17.8281	-25.6719	
2/1/2016	20:50:00	0.076713	0.076822	0	0.145	43.5	23.0139	-20.4861	
2/1/2016	20:55:00	0.072336	0.072221	0	0.145	43.5	21.7008	-21.7992	
2/1/2016	21:00:00	0.063009	0.062896	0	0.145	43.5	18.9027	-24.5973	
2/1/2016	21:05:00	0.059393	0.059428	0	0.145	43.5	17.8179	-25.6821	
2/1/2016	21:10:00	0.074228	0.074508	0	0.145	43.5	22.2684	-21.2316	
2/1/2016	21:15:00	0.080187	0.080102	0	0.145	43.5	24.0561	-19.4439	
2/1/2016	21:20:00	0.069684	0.069551	0	0.145	43.5	20.9052	-22.5948	
2/1/2016	21:25:00	0.061127	0.061046	0	0.145	43.5	18.3381	-25.1619	
2/1/2016	21:30:00	0.062692	0.062849	0	0.145	43.5	18.8076	-24.6924	
2/1/2016	21:35:00	0.080237	0.080326	0	0.145	43.5	24.0711	-19.4289	
2/1/2016	21:40:00	0.074711	0.074566	0	0.145	43.5	22.4133	-21.0867	
2/1/2016	21:45:00	0.06379	0.063655	0	0.145	43.5	19.137	-24.363	
2/1/2016	21:50:00	0.057258	0.057228	0	0.145	43.5	17.1774	-26.3226	
2/1/2016	21:55:00	0.066084	0.066356	0	0.145	43.5	19.8252	-23.6748	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
2/1/2016	22:00:00	0.076859	0.076811	0	0.145	43.5	23.0577	-20.4423	
2/1/2016	22:05:00	0.067217	0.067077	0	0.145	43.5	20.1651	-23.3349	
2/1/2016	22:10:00	0.057089	0.056969	0	0.145	43.5	17.1267	-26.3733	
2/1/2016	22:15:00	0.052789	0.052788	0	0.145	43.5	15.8367	-27.6633	
2/1/2016	22:20:00	0.062647	0.062896	0	0.145	43.5	18.7941	-24.7059	
2/1/2016	22:25:00	0.069843	0.06977	0	0.145	43.5	20.9529	-22.5471	
2/1/2016	22:30:00	0.060558	0.060406	0	0.145	43.5	18.1674	-25.3326	
2/1/2016	22:35:00	0.052139	0.052052	0	0.145	43.5	15.6417	-27.8583	
2/1/2016	22:40:00	0.051562	0.051662	0	0.145	43.5	15.4686	-28.0314	
2/1/2016	22:45:00	0.066014	0.0662	0	0.145	43.5	19.8042	-23.6958	
2/1/2016	22:50:00	0.065195	0.065078	0	0.145	43.5	19.5585	-23.9415	
2/1/2016	22:55:00	0.054964	0.054828	0	0.145	43.5	16.4892	-27.0108	
2/1/2016	23:00:00	0.049038	0.048988	0	0.145	43.5	14.7114	-28.7886	
2/1/2016	23:05:00	0.051791	0.051975	0	0.145	43.5	15.5373	-27.9627	
2/1/2016	23:10:00	0.067153	0.067242	0	0.145	43.5	20.1459	-23.3541	
2/1/2016	23:15:00	0.062204	0.062064	0	0.145	43.5	18.6612	-24.8388	
2/1/2016	23:20:00	0.052242	0.052125	0	0.145	43.5	15.6726	-27.8274	
2/1/2016	23:25:00	0.047581	0.047542	0	0.145	43.5	14.2743	-29.2257	
2/1/2016	23:30:00	0.050254	0.050434	0	0.145	43.5	15.0762	-28.4238	
2/1/2016	23:35:00	0.066182	0.066299	0	0.145	43.5	19.8546	-23.6454	
2/1/2016	23:40:00	0.062112	0.061976	0	0.145	43.5	18.6336	-24.8664	
2/1/2016	23:45:00	0.052378	0.052265	0	0.145	43.5	15.7134	-27.7866	
2/1/2016	23:50:00	0.047822	0.047783	0	0.145	43.5	14.3466	-29.1534	
2/1/2016	23:55:00	0.048727	0.048839	0	0.145	43.5	14.6181	-28.8819	
3/1/2016	0:00:00	0.064851	0.06506	0	0.145	43.5	19.4553	-24.0447	
TOTAL STORAGE VOLUME REQUIRED (m^3)									**1088**
ADD 20% CONTINGENCY (m^3)									**218**
TOTAL OPERATIONAL VOLUME TO BE PROVIDED AT THE JANET AVENUE PUMPING STATION (m^3)									**1306**

FINAL

PHASE 3: CONCEPTUAL DESIGN

Technical Memo No. 4

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

21 SEPTEMBER 2021

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John Bourrie

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LIST OF TABLES

List of Abbreviations

ADD	Average Day Demand
ADF	Average Day Flow (Annual)
BOD_5	Biochemical Oxygen Demand
CMU	Concrete Masonry Unit
CT	Baffling Factor x Contact Time (min) x Concentration (mg/L)
DWWP	Drinking Water Works Permit
EA	Environmental Assessment
ECA	Environmental Compliance Approval
ESA	Electrical Safety Authority
hp	Horsepower
HRT	Hydraulic Retention Time
IFAS	Integrated Fixed-Film Activated Sludge
kg/h	Kilogram per Hour
km	Kilometer
L/min	Litres per Minute
L/s	Litres per Second
MABR	Membrane Aerated Biofilm Reactor
MCC	Motor Control Center
MECP	Ministry of Environment, Conservation and Parks
m^3/d	Cubic Meters per Day
MDD	Maximum Day Demand
MDWL	Municipal Drinking Water Licence
ML	Million Litres
MLD	Million Litres per Day
mg	Milligram
mJ/cm^2	Millijoule per Square Centimeter
MLSS	Mixed Liquor Suspended Solids
mm	Millimeter
m/s	Meters per Second
O&M	Operations and Maintenance
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
PW2	Production Well No. 2
RCC	Reinforced Concrete
RPU	Remote Processing Unit
SCADA	Supervisory Control and Data Acquisition
SPS	Sewage Pumping Station
TDH	Total Dynamic Head
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TP	Total Phosphorous
TRCA	Toronto and Region Conservation Authority
TSS	Total Suspended Solids
TSSA	Technical Standards and Safety Authority
WAS	Waste Activated Sludge
WRRF	Water Resource Recovery Facility

1.0 Introduction

Nobleton is a community in King Township in York Region. Currently, Nobleton is serviced by stand-alone water and wastewater systems to meet the needs of the current population. The York Region Water and Wastewater Master Plan (2016) indicated that the water and wastewater systems would require increased capacity to meet the requirements to support growth to the 2041 Master Plan population of 9,500. Therefore, the Master Plan recommended a Schedule C Class Environmental Assessment (EA) to identify servicing solutions to accommodate growth.

Taking into consideration the available land and the allowable population density, a future population of 10,800 has been estimated within the Nobleton urban boundary in the Nobleton Community Plan and the King Township Draft Official Plan.

Phases 1 and 2 of the current Class EA are complete, and the study is currently in Phase 3. As part of Phase 1, the problem and opportunity was identified, and as part of Phase 2, alternative solutions were identified, evaluated and preferred solutions for water and wastewater servicing were identified. These activities were documented in Technical Memoranda (TMs) 1 and 2, respectively.

As part of Phase 3, TM3 was prepared to document the development of alternative design concepts for the preferred water and wastewater servicing solutions identified in TM2 and carried out a rigorous evaluation of the alternative design concepts to recommend preferred design concepts for water and wastewater.

The next step, which is the focus of this TM, is to prepare conceptual designs for water and wastewater servicing using the preferred design concept.

1.1 Objective of Technical Memorandum

As part of TM3, different design concepts for the preferred water and wastewater solutions were identified using the preferred solutions developed as part of TM2. The objective of TM4 is to prepare conceptual designs from those preferred design concepts for water and wastewater servicing, which are as follows:

- Increase the capacity of the existing Well Site No. 2 pump to 34 liters per second (L/s) and provide a new production well (Well No. 6) and associated treatment system at Well Site No. 5 with a capacity of 34 L/s.
- Expand the firm capacity of the Janet Avenue Sewage Pumping Station (SPS) to 145 L/s, and provide a belowground flow attenuation tank at the Janet Avenue SPS site with approximate dimensions of 15.5 meters (m) by 12 m by 11 m (deep) tank with an approximate operational depth of 7 m and an operational volume of 1,300 cubic meters (m^3) to accommodate a 1 in 25 year storm.
- Intensify the existing biological treatment trains at the Nobleton Water Resource Recovery Facility (WRRF) using membrane aerated biofilm reactor (MABR) technology, expand and modify the headworks to be compatible with MABR technology, expand the tertiary treatment system, and provide additional sludge storage.

The purpose of TM4 is to develop conceptual designs for the preferred design concepts evaluated and recommended as part of TM3.

1.2 Summary of Work Completed in Tech Memo No. 3

Water and wastewater servicing opportunities and problems were identified in Phase 1. Preferred water and wastewater servicing solutions were identified in Phase 2. The current step, Phase 3, is to identify, screen, and evaluate recommended design concepts for the preferred servicing solutions and recommend preferred design concepts.

1.2.1 Preferred Design Concept for Water Servicing

The following preferred design concepts were carried forth from previous phases of the EA process. Solutions for implementing these design concepts are described herein.

- Increase capacity of Well No. 2 from 22.7 L/s to 34 L/s. Increasing the capacity of Well No. 2 will be accomplished by replacing the existing well pump at Well No. 2.
- Add new production well at site H (Well No. 6) with a capacity of 34 L/s. The new production well will be located on the same site as Well No. 5 and have a dedicated treatment train.

1.2.2 Preferred Design Concept for Wastewater Servicing

Phases 1 and 2 of the current Class EA study are complete. A brief description of the work performed during these phases is provided in the following sections.

Wastewater Pumping, Flow Attenuation, Forcemain, and Outfall

For the wastewater pumping, flow attenuation, forcemain, and outfall design concepts discussed in TM3, only one design concept was brought forward for further evaluation, and the others were screened out. The preferred design concept from TM3 that was carried into TM4 and conceptual design was the following:

- Provide flow attenuation storage at the Janet Avenue Pumping Station (PS) site for an operational volume of 1,300 m^3.
- Expand the Janet Avenue PS to a firm capacity of 12,500 cubic meters per day (m^3/d) (145 L/s). This would eliminate twinning of the forcemain and the constricted sections of the effluent outfall.

Two sub-concepts were generated for the flow attenuation storage option, i.e., a storage tank and a big pipe, and both of these sub-concepts were carried forward for detailed evaluation. As an outcome of the evaluation, the tank concept was chosen as the preferred design concept, although the two concepts scored relatively evenly. This was primarily because of the requirement for a separate access road to the pumping station during construction of the pipe storage concept.

Wastewater Treatment

For the wastewater treatment design concepts discussed in TM3, only two design concepts were brought forward for further evaluation, and the others were screened out. The detailed evaluation of the two alternative wastewater design concept solutions favored Alternative B: "Intensify the existing biological treatment trains (MABR technology) with upstream collection system flow attenuation to reduce peaking factor at the WRRF" to be the recommended design concept solution from TM3 that was carried into TM4.

2.0 Development of Conceptual Design for Wastewater Servicing

The Nobleton Water and Wastewater Schedule C Class EA developed, refined, and evaluated various potential servicing strategies (for both the water and wastewater systems) to address the problem statement using a two-stage process. A long list of servicing strategies (design concepts) was prepared and screened utilizing pass/fail criteria to obtain the short list of design concepts. The short-listed design concepts were further evaluated utilizing various criteria to recommend preferred design concepts. These preferred design concepts were documented in TM3. The current TM presents the conceptual level design for the preferred design concepts.

2.1 Design Criteria

2.1.1 Janet Avenue SPS and Flow Attenuation Tank

The design criteria and design basis listed in Table 2-1 were adopted for the Janet Avenue SPS and the flow attenuation tank.

Table 2-1 Design Criteria for the Janet Avenue SPS and the Flow Attenuation Tank

Design Element	Design Criterion/Design Basis
No. of Pumps	3 (2 duty + 1 standby)
Firm Capacity of the Janet Avenue SPS	145 L/s
Total Dynamic Head at the design point of 145 L/s	75 m (approximately)
Motor Power Required at the design point for each pump	140 kW (preliminary pump and motor selection by a vendor)
Operational Volume of the Flow Attenuation Tank (based on accommodating a 1 in 25 year storm in conjunction with 145 L/s capacity at Janet Avenue SPS)	1,300 m^3
Operational Depth of the Flow Attenuation Tank	7 m
Approximate Dimensions of the Flow Attenuation Tank	15.5 m by 12 m by 11 m deep

2.1.2 Water Resource Recovery Facility

Wastewater flow to the WRRF is limited by the capacity of the Janet Avenue SPS. Design pumping station capacity is 145 L/s (12,528m^3/d).

Wastewater load is based on the sum of current per capita pollutant loads and projected future loads from population and economic growth of the service area. The design criteria for the WRRF processes are shown in Table 2-2.

Table 2-2 Design Criteria for Water Resource Recovery Facility

Treatment Process	Design Basis Criterion[1]	Design Basis	Notes
Preliminary Treatment – Screening	Peak instantaneous flow (PIF) with 1 unit out of service	12,528 m^3/d	
Preliminary Treatment – Grit Removal	Peak hourly flow (PHF), peak hourly grit loading	12,528 m^3/d (522 m^3/h); 20 L grit/h	Grit loading assumed based on Ministry of Environment, Conservation and Parks (MECP) standards.
Secondary Treatment – Aeration	Average daily biochemical oxygen demandBOD_5 loading based on design average day flow (ADF), peak daily total Kjeldahl Nitrogen (TKN) Loading based on design PDF	683 kilogram (kg) BOD_5/d; 144 kg TKN/d	
Secondary Treatment – Sedimentation	PHF, Peak Daily Solids Loading	12,528 m^3/d; 14,010 kg/d	
Secondary Treatment (sludge return)	50 to 200% of design ADF	1,998 m^3/d (23 L/s) to 7,992 m^3/d (92.5 L/s)	
Secondary Treatment (Sludge Wasting)	0.5 to 25% of design ADF[2]	20 m^3/d (0.23 L/s) to 999 m^3/d (11.6 L/s)	
Chemical Phosphorus Removal	Total phosphorous (TP) load, molar ratio of coagulant to TP[3]	Max month wastewater TP load = 40.2 kg/day; the molar ratio of Al:TP is 6.5	
Chemical Storage	10 days, minimum	3,780 L/d of alum solution	
Disinfection	PHF	12,528 m^3/d	
Effluent Filtration	3.3 L/(m^2·min) at PHF with 1 unit out of service	12,528 m^3/d (8,700 L/min)	
Outfall Sewer	PIF	12,528 m^3/d	
Sludge Storage	4 days, minimum[4]	560 kg WAS/day[5]; 60 m^3/day[6]	Based on the historical data, the sludge production at the Nobleton WRRF is between 100 and 140 g TS/m^3 wastewater treated.

1. Design Guidelines for Sewage Works (Ontario MECP, 2021), unless otherwise noted.
2. Recommended Standards for Wastewater Facilities (GLUMRB , 2014).
3. General engineering knowledge.
4. Based on providing adequate storage over weekends and holidays.
5. Based on observed sludge production of 140 g / m^3 wastewater treated.
6. Based on 8,000 milligram (mg) waste activated sludge (WAS) total suspended solids (TSS)/L.

2.2 Process Design

2.2.1 Janet Avenue SPS

The Janet Avenue SPS will essentially retain the existing layout. The pump suction and discharge headers and the station header sizing will increase to accommodate the increased flows from the larger pumps. A larger flowmeter will also be needed to measure the increased pumped flows.

A new wet well to accommodate the increased flows will not be provided. It is expected that the larger pump units will result in the pumps cycling more often than the existing pumps. However, as the pumps will be equipped with variable frequency drives, the cycling will be reduced to reasonable limits. In addition, manufacturers are providing pump motors that can withstand more frequent starts and stops than before.

The existing emergency overflow pipe was evaluated for its capacity to convey 1 in 25 year wet weather flows in the event of a catastrophic failure at the pumping station. It has adequate capacity to convey the 1 in 25-year flow if the pumping station was not able to pump the received flow and the flow attenuation tank was full.

A preliminary flow schematic for the pumping station and the flow attenuation tank is included in Appendix B.

2.2.2 Flow Attenuation Tank

The flow attenuation tank will be a belowground cast-in-place structure with the plan dimensions of 15.5 m by 12 m and a depth of approximately 11 m. The operating depth of the tank will be 7 m. A new flow diversion chamber will be provided on the incoming gravity sewer immediately upstream of the wet well. In the event of a wet weather event, when the Janet Avenue SPS is unable to pump received wastewater, the flow diversion chamber will passively overflow wastewater into a gravity pipe conveying it into the flow attenuation tank. As the wet weather event subsides, the flow attenuation tank will be allowed to drain back into the flow diversion chamber by operator intervention.

A tank cleaning system in the form of tipping buckets will be provided in the flow attenuation tank. The cleaning cycle will be initiated by operators through the region's supervisory control and data acquisition (SCADA) system. The wash water will drain into the wet well.

2.2.3 Flow Diversion Chamber and Piping

A new flow diversion chamber will be constructed on the incoming gravity sewer at the wet well immediately upstream of it. This chamber will be equipped with an adjustable overflow weir, which will passively divert flow beyond the capacity of the Janet Avenue SPS into the flow attenuation tank through a new gravity sewer.

The flow diversion chamber will also receive flow drained from the flow attenuation tank and convey it to the wet well.

The flow diversion chamber top slab will be equipped with goosenecks to provide passive ventilation along with rising and falling liquid levels.

2.2.4 Water Resource Recovery Facility

The WRRF processes for the upgraded facility will be the same as the existing facility except that gravity thickening of WAS will be discontinued. The processes include screening, grit removal, secondary biological treatment, tertiary filtration, disinfection, and sludge storage.

2.2.4.1 Screening

The existing coarse screen system will be removed and replaced with a fine screen system to satisfy the requirements for the downstream secondary biological treatment system. Perforated plate fine screens with 2 millimeter (mm) openings will be provided to be compatible with technology at the region's other WRRFs. A minimum of two screens is required to provide firm capacity with one unit out of service.

One new screen will be located in the channel parallel to where the existing coarse screen is located. Additional screens will be located in a new channel(s) constructed in an extension of the process building to the north. The channels in the new extension will also be in parallel to each other. The width of the screens will be determined in the preliminary design stage.

The requirement for 2 mm openings should be evaluated in the preliminary design stage. A sieve analysis of the mixed liquor suspended solids (MLSS) can provide an objective evaluation of the screen openings size requirement. A larger opening size may be allowable, which would reduce screenings generation.

A summary of the fine screen design criteria is shown in Table 2-3.

Table 2-3 WRRF Wastewater Fine Screen Process Design Criteria

Parameter	Value
Number of New Screens	2 (1 duty / 1 standby) or 3 (2 duty / 1 standby)
Type of Screens	Perforated Plate (2 to 6 mm openings) *
Capacity (Each)	12,528 m^3/day (2 screens) or 6,264 m^3/day (3 screens)
* To be determined during preliminary design phase based on a sieve analysis of the mixed liquor.	

2.2.4.2 Grit Removal

The Environmental Compliance Approval (ECA)-rated capacity of the existing 2 m diameter vortex grit units is 9,177 m^3/d. Both units would be required to be in service for future design conditions. Therefore, a third 2 m unit is proposed to provide firm capacity with one unit out of service. A second grit classifier will be added to process the grit from the new unit. The new grit removal unit and classifier will be constructed in an extension of the process building to the north. A third grit pump will be added opposite the existing grit pumps in the process building.

A summary of the grit removal process design criteria is shown in Table 2-4.

Table 2-4 WRRF Wastewater Grit Removal Process Design Criteria

Parameter	Value
Type of Grit Removal	Induced vortex
Number of Grit Tanks	3
Number of New Grit Tanks	1
Size of Grit Tanks	2,000 mm diameter
Capacity (each)	9,177 m^3/d

2.2.4.3 Secondary Biological Treatment

The existing extended aeration activated sludge process will be converted to an MABR hybrid suspended growth/attached growth process with the addition of MABR media to the existing aeration basins. The MABR media will be located in new anoxic selector/denitrification zones constructed with the addition of a baffle wall in each existing aeration tank. The anoxic zone will be outfitted with mixers to keep MLSS suspended around the MABR media.

The proposed anoxic selector zone has multiple purposes:

- Improves settleability by selecting for microorganisms that create large flocs that settle fast.
- Recovers alkalinity by denitrification thereby stabilizing the biological treatment process and reducing demand for input of sources of chemical alkalinity.
- Reduces aeration demand by supplying nitrate (NO_3) as the dominant electron acceptor.
- Reduces total nitrogen in the final effluent

Aeration capacity will be increased to satisfy oxygen demand for the MABR media with the addition of dedicated blowers. The capacity of the existing aeration blowers is adequate for the suspended growth portion of the process. The required blower capacity will be confirmed in the preliminary design phase.

Return activated sludge (RAS) and WAS pumps will be replaced with larger pumps to satisfy design requirements for sludge recirculation and sludge wasting.

A dissolved oxygen monitoring and control system will be provided for the oxic zones for energy efficiency of the wastewater aeration system and process control benefit.

The WRRF secondary biological treatment system process design criteria are listed in Table 2-5.

Table 2-5 WRRF Secondary Biological Treatment System Process Design Criteria

Parameter	Value
Wastewater Temperature	12 ° C (minimum month)
Oxygen Transfer Rate	2,015 kg/d*
Solids Retention Time	> 15 days

Parameter	Value
MLSS Concentration	< 3,500 mg TSS/L
RAS Pumping	23 L/s to 92.5 L/s
F:M of Anoxic Selector Zone	0.5 to 1.0
Existing Alum Storage	20,000 L
Total Alum Storage Required	37,820 L
Membrane Oxygen Transfer Rate (OTR)	8 – 15 g/m^2/d**
Nit-Ammonia Removal Rate per m^2	1.5 – 3.5 g/m^2/d**
Film Thickness	0.1 – 0.6 mm**
Total SS/Area	10 – 50 g/m^2**
TSS at Film Bottom	> 30,000 mg/L**
OTR:NR Ratio	4.57 – 7**

* Calculated according to MECP standards assuming 1.5 kg O_2 / kg cBOD5, 4.6 kg O_2 / kg TKN, a PDF of 1.8 for TKN load, and assuming 90% of influent TKN is nitrified.
** MABR values not based on MECP design standards as there is not a category for MABRs. Values based on guidance from Suez for typical parameters.

2.2.4.4 Chemical Phosphorus Removal

The existing chemical phosphorus removal process will be retained. There are five alum metering pumps, including three with a capacity of 65 L/h serving the aeration tank inlets and clarifier inlets and two with a capacity of 17 L/h serving the aeration tanks outlets and filter inlet channel. The firm capacity is 164 L/h, which is adequate for future design conditions. Chemical dosing distribution should be evaluated in the preliminary design phase to match the desired dosing rates with the dosing locations.

One alum storage tank provides a storage volume of 20,000 L. Alum storage will be increased to provide a minimum 10 days of storage.

A summary of the chemical phosphorus removal process design criteria is shown in Table 2-6.

Table 2-6 WRRF Wastewater Chemical Phosphorus Removal Process Design Criteria

Parameter	Value
Phosphorus Removal Required	40 kg/d
Alum Dosing Capacity	158 L/h (3,792 L/d)
Dosing locations (existing)	Aeration basin inlet channel, mixed liquor outlet chambers, clarifier outlet chambers, filter inlet channel
Existing Alum Storage	20,000 L
Total Alum Storage Required	37,820 L

2.2.4.5 Tertiary Sand Filtration

The existing deep bed sand filtration system will be expanded with the addition of three additional cells to provide a total of seven cells and a total of 65 square meters (m^2) of filtration area. The new cells will be constructed in an extension of the existing process building to the south. The new cells will include an intermittent backwashing system which will also be retrofitted to the existing filter cells. The intermittent backwashing system will reduce backwashing volume and reject water such that the existing reject water sump and pumps will be adequate for design conditions without expansion.

The existing reciprocating compressors will be replaced with larger compressors to satisfy the increased air requirements. Two new screw compressors, each with its own receiver tank, will be provided in the same location as the existing compressors.

A summary of the tertiary filtration process design criteria is shown in Table 2-7

Table 2-7 WRRF Tertiary Filtration Process Design Criteria

Parameter	Value
Type of Filtration	Deep sand
Total Number of Filter Cells/Modules	7 / 14
Number of New Filter Cells	3 / 6
Total Filtration Area	65 m^2
Backwash Flow per Module	0.9 L/s (max) (intermittent)
Airlift Air Requirement	17.2 L/s

2.2.4.6 Effluent Disinfection

The existing ultraviolet disinfection system is a low-pressure, low intensity system installed in an 8,000 mm long by 245 mm wide channel in the process building. In order to increase capacity, the existing system will be replaced by a new low-pressure high output system. This will substantially reduce the number of lamps and length of channel required such that the replacement system will fit in the existing channel without an extension.

A summary of the effluent disinfection system design criteria is shown in Table 2-8.

Table 2-8 WRRF Wastewater Effluent Disinfection Process Design Criteria

Parameter	Value
Type of Disinfection	Ultraviolet irradiation (low-pressure, high intensity)
Design Dose	35 millijoule per square centimeter (mJ/cm^2) (minimum)
Capacity	12,528 m^3/d
Number of Banks of Lamps in Series	2 (minimum)
Level Control	Automatic level control gate
Cleaning System	Automatic

2.2.4.7 Sludge Storage

The objective of sludge storage is to provide short-term storage of waste sludge over weekends and holidays prior to hauling. Other important objectives include thickening to reduce the hauled volume and to provide a decant quality that does not interfere with achieving treatment goals in the main stream treatment process.

The existing sludge thickener and aerated sludge storage tank will be replaced with aboveground aerated sludge storage tanks. Two tanks will be provided for redundancy, each tank providing the design volume of storage. Separate aeration and mixing systems will be provided. The aeration system will keep sludge fresh and reduce odor potential. The mixing system is provided to allow recirculation of tank contents with aeration "off" to allow for reducing the nutrients recycled to the main stream through denitrification of the stored sludge. A sludge pumphouse will be provided between the two sludge storage tanks in order to seat the blowers for aerating the sludge and pumps for loading sludge to transport trucks.

A summary of the sludge storage process design criteria is shown in Table 2-9.

Table 2-9 WRRF Wastewater Sludge Storage Process Design Criteria

Parameter	Value
Sludge Disposal Method	Hauled to Aurora SPS
Type of Storage	Liquid (aerated)
Daily WAS Volume	70,000 L/d
Capacity	4 days (unthickened)
Number of tanks	2
Diameter of tanks, each	10 m
Height of tanks, each	5 m
Effective Volume of each Tank	280,000 L
Materials of tanks	Glass lined bolted steel tanks with aluminum geodesic dome fixed covers
Thickening	Decant
Mixing System	Pumped recirculation
Aeration System	Diffused air
Air Requirement	504 m^3/h*
*Air requirement based on MECP standard 18.2.3 for aerobically digested sludge storage, 30 m^3 / (1000 m^3 x min)	

2.3 Site Layout

2.3.1 Janet Avenue SPS and Flow Attenuation Tank

The Janet Avenue SPS site will accommodate a new belowground flow attenuation tank. The location of the tank will be kept close to the SPS to minimize pipe lengths and reduce friction losses. The location and dimensions of the tank are subject to be further refined during the preliminary design to obtain the most efficient layout and optimize cost.

Requirements relating to altering the site paving, fencing, yard piping etc. will be addressed during the preliminary design stage.

Appendix B includes a preliminary site layout for the Janet Avenue SPS and flow attenuation tank showing the SPS, flow attenuation tank, flow diversion chamber, and preliminary routing for the site piping connecting the flow diversion chamber and the flow attenuation tank.

2.4 Equipment Layout

2.4.1 Janet Avenue SPS and Flow Attenuation Tank

The equipment layout in the dry well will follow the existing layout. The existing pumps will be replaced with larger capacity dry pit submersible pumps. The existing suction and discharge piping will be replaced with larger sized piping suitable for the larger pumps. The existing valves will also be replaced for larger sized valves.

Floor plans with equipment, piping and valve layout will be prepared during the preliminary design stage. A preliminary schematic is included in Appendix B.

2.5 Electrical, Instrumentation and Control, SCADA Requirements

2.5.1 Janet Avenue Sewage Pumping Station

There are three existing pumps which need to be upsized and require larger starters. The existing motor control center (MCC) only has maximum ampacity of 400 ampere (A). It does not have enough power to accommodate the new power requirement. A larger MCC and generator will be installed to replace the existing MCC and generator. The new generator will be installed exterior to the building.

A higher power demand request needs to be submitted to local hydro company at the beginning of the project. During construction, a temporary or permanent generator will be installed before the existing generator is removed. The new MCC could be installed in the area vacated by removing the existing generator. The space will need to be repurposed for installation of the new MCC. A larger incoming transformer could be installed close to the existing transformer. A new transformer pad with ground grid shall be in place before transformer installation. A power study, including ground touch and step potential, should be provided before transformer pad installation. Ground resistance verification should be done after the ground grid has been installed. A new duct bank will be installed to extend to the new MCC incoming section. The new power line from the hydro company will connect to the new MCC. An additional main breaker will connect to the existing MCC during the incoming hydro power transfer.

Existing starters and control will transfer to the new MCC. After cabling transfer, the existing MCC associated with the concrete pad will be removed. The floor opening will be filled to prevent hazards.

All additional instrumentation and control will tie into the existing remote processing unit (RPU) panel. The number of additional signals and changes on the RPU and SCADA will be finalized during the detailed design stage.

2.5.2 Water Resource Recovery Facility

The existing resource recovery facility electrical distribution system demand load and emergency load should be verified before project detailed design. The current estimate from the as-built drawing indicates that the existing distribution system has sufficient power to accommodate the additional loads.

New indoor and exterior lights will be installed in the expanded building facility.

New equipment starters will be installed on the spare section of the MCC. All additional remote control and instrumentation signal will tie-in to existing RPU. All new signals will be tied to the existing RPU spare points. The existing RPU could be expanded if required. The existing RPU will be reprogrammed to accommodate additional equipment control and instrumentation sign. A SCADA program update will also be needed.

2.6 Structural and Architectural Requirements

2.6.1 Janet Avenue Sewage Pumping Station

The proposed upgrades at the Janet Avenue SPS include the following:

Equipment Pad for Pumps and Pipe Supports

Three of the existing pumps in the existing pumping station need to be upsized and, hence, require bigger equipment bases to seat the new pumps. The existing pump pads shall be demolished, and new pump base concrete pads shall be cast on the operating floor of the pump gallery. Alternatively, the existing pads may be reused with appropriate modifications to accommodate the new pumps.

The suction and discharge pipes also require replacement and may require a few pipe supports according to the pump manufacturer's criteria.

Flow Attenuation Tank and Flow Diversion Chamber

A new flow attenuation tank, approximately 15.5m by 12m by 11m deep, has to be constructed at a suitable location within the pumping station site to balance the flow between on-peak and off-peak hours sewage flow. This tank shall be a fully or partly buried type cast-in-place concrete tank.

Also, a buried concrete flow diversion chamber, approximately 0.9m by 1.4m in plan dimension, shall be constructed adjacent to the west side of the existing wet well.

Generator Pad and Transformer Pad

The existing generator capacity has to be increased because of the additional power requirement. This generator shall be replaced with a new higher capacity generator and shall be relocated to an exterior location on a separate concrete pad. Cast-in-place reinforced concrete slab-on-grade foundation shall be provided for the new generator. Frost heave below this foundation shall be prevented. The existing generator area will be repurposed for new MCC room.

Existing transformer pads may have to be resized if the existing transformers are upsized.

2.6.2 Water Resource Recovery Facility

Proposed upgrade works at the existing water resource recovery facility are enlisted below.

Existing Process Building Modifications:

New Fine Screens, Grit Tank, and Classifier at North End

The existing process building is a reinforced concrete structure up to the grade level and a concrete masonry unit (CMU) load bearing structural system above the grade, except at screen channels and grit tank, where the reinforced concrete walls are raised up to the upper floor, and the remainder is CMU walls with brick cladding up to the roof level. Hollow core slabs are provided at roof level to carry gravity loads and to transfer lateral loads to the supporting walls and foundations.

The north end of the existing process building has to be extended approximately 11 m further north in order to accommodate the proposed addition of new fine screen channels, grit classifiers, and grit tanks. There is no requirement to add or extend the existing sludge storage tank below grade. A structural system similar to the existing one (such as foundations and CMU wall load bearing superstructure) is proposed for the extension work. The existing stairwell at the north end shall be retained as a common access to the existing building and to the new north side extension.

New Tertiary Filters and Alum Storage Tanks Addition at South End

The existing tertiary filtration capacity has to be increased by adding six more filter beads to south side of the existing process building filtration units. Also, the existing effluent water tank adjacent to the existing filter units shall be extended along with the new filtration tank. This will involve construction of buried cast-in-place concrete tanks in continuation with the existing tanks.

Provisions for seating alum/sodium hydroxide storage tanks at operating level shall be provided on the roof slab of the new effluent storage tank. Sufficient bearing walls and/or beams shall be provided in the slab to transfer loads form these tanks to the foundation.

Blowers Room Upgrades

The existing blower room shall be modified to accommodate two new air compressor units and new blowers, one blower to be installed at the time of this upgrade works and the second as a future provision. Adequate equipment pads shall be provided to seat the new blower and air compressor units. The existing slab on grade foundation shall be verified for these additional loads.

Aeration Tanks 1 and 2 Upgrades

Five membrane cassettes shall be added to each of these existing aeration tanks. Some steel/stainless steel beams shall be added to support these additional new membrane cassettes in the aeration tanks. These beams may be supported from the existing baffle walls. Alternatively, options to support these membrane cassettes from the base slab of the existing aeration basin shall also be investigated. A feasible and economical supporting scheme shall be adopted in detailed design. In addition to supports for the membrane cassettes, a retrofit of the existing aeration tanks will also include a baffle wall for the anoxic selector zone.

New Sludge Storage Tanks and Sludge Pumphouse

Two new biosolids/sludge storage tanks, glass lined bolted steel, approximately 10m diameter by 5m height with aluminum geodesic dome fixed covers shall be constructed for the storage of sludge. A concrete base foundation shall be provided to seat these tanks by tank supplier.

A pumphouse is required between the two sludge storage tanks in order to seat the blowers for aerating sludge and pumps for loading sludge transport trucks. This pumphouse building may be a single storied CMU building with concrete base slab foundation and hollow core plank roof.

New Truck Loading Area Upgrade

A new truck loading area shall be provided adjacent to the proposed sludge pumphouse. Existing pavement shall be extended to facilitate this truck loading area. A concrete buried sump shall be centered on this pavement to collect the spillages and shall be connected to an existing sanitary line at this site. Adequate pipe supports shall be provided from the pumphouse structure to support the discharge header.

2.6.3 Design Codes

Structural design of these upgrades/modifications shall be in accordance with Ontario Building Code 2012 with 2020 amendments. Also, all liquid retaining concrete structures, for example, equalization tanks and overflow chamber, shall be designed in accordance with ACI 350 in order to ensure water tightness.

2.7 Permits and Approvals

The following permits and approvals are anticipated for the Janet Avenue Pumping Station and the Nobleton WRRF:

- Ministry of the Environment, Conservation and Parks (MECP) Environmental Compliance Approval (ECA) amendment.
- Township of King Site Plan Approval.
- Township of King Building Permit.
- Electrical Safety Authority (ESA) plan approval.
- Toronto and Region Conservation Authority (TRCA) Approval.

2.8 Opinions of Probable Cost

Black & Veatch has prepared opinions of probable cost suitable for this stage of the design (Tables 2-10 and 2-11). These should be considered indicative cost estimates (Class D Cost Estimates). These have not been developed from bottom up. As the design moves through the subsequent stages, where various design elements are firmed up, the cost estimates will be refined as well. Black & Veatch will prepare and present a more detailed cost estimate in the next stage, which is preliminary design.

Table 2-10 Opinion of Probable Cost for the Janet Avenue SPS and Flow Attenuation Tank

Discipline	Million Dollars (2021)
Site and Civil	$0.5 Million
Structural and Architectural	$2.3 Million
Process and Building Mechanical	$0.9 Million
Electrical, Instrumentation and Control, SCADA	$0.4 Million
Total Capital Cost of Infrastructure	$3.9 Million
General Requirements (@ 15% of Capital Cost)	$0.6 Million
Contingencies (@20% of Capital Cost + General Requirements)	$0.9 Million
Engineering, Legal and Administration (@ 20% of (Capital Cost + General Requirements + Contingencies))	$1.1 Million
Total Cost Including Engineering and Contingencies	**$6.7 Million**

Table 2-11 Opinion of Probable Cost for the WRRF

Discipline	Million Dollars (2021)
Site and Civil	$0.9 Million
Structural and Architectural	$1.0 Million
Process and Building Mechanical	$4.9 Million
Electrical, Instrumentation and Control, SCADA	$1.2 Million
Total Capital Cost of Infrastructure	$8.0 Million
General Requirements (@ 15% of Capital Cost)	$1.2 Million
Contingencies (@20% of Capital Cost + General Requirements)	$1.9 Million
Engineering, Legal and Administration (@ 20% of (Capital Cost + General Requirements + Contingencies))	$2.3 Million
Total Cost Including Engineering and Contingencies	**$13.4 Million**

3.0 Development of Conceptual Design for Water Servicing

The Nobleton Water and Wastewater Schedule C Class EA developed, refined, and evaluated various potential servicing strategies (for both the water and wastewater systems) to address the problem statement using a two-stage process. A long list of servicing strategies (design concepts) was prepared and screened utilizing pass/fail criteria to obtain the short list of design concepts. The shortlisted design concepts were further evaluated utilizing various criteria to recommend preferred design concepts. These preferred design concepts were documented in TM3. The current Tech Memo presents the conceptual level design for the preferred design concepts.

3.1 Design Criteria

Previous technical memoranda have identified, screened, and evaluated alternatives to service the increased population of 10,800. The following solutions have been selected for development of design concepts:

- Increase capacity of Well No. 2 from 22.7 L/s to 34 L/s. Increasing the capacity of Well No. 2 will be accomplished by replacing the existing well pump at Well No. 2, including new motor, starter, and cabling as required.
- Add new production well at site H (Well No. 6) with a capacity of 34 L/s. The new production well, including pump, motor, starter and cabling, will be located on the same site as Well No. 5 and have a dedicated treatment train. Equipment for Well No. 5 will be relocated and/or replaced as described herein to accommodate the installation of the treatment train for Well No. 6.

Per guidance provided in Technical Memorandum No. 2, no additional storage of potable water will be provided under the proposed solution. Storage deficits will be compensated through an additional 2.0 L/s supply above the peak demand of 32 L/s at both Well No. 2 and Well No. 6.

3.2 Process Design

The treatment process for Well No. 6 will consist of disinfection and iron and manganese sequestration. A process flow diagram showing the major components of the treatment process is shown in Appendix A.

Disinfection will be achieved using gas chlorine for 4-log virus inactivation. Chlorine gas will be delivered via 68 kg cylinders. Sufficient storage will be provided for 30 days of operation at the design dose. The chlorine feed system will be sized for a design dose of 8.5 mg/L of free chlorine. Contact time for primary disinfection will be accomplished in a below grade, chlorine contact chamber with superior baffling conditions for a baffle factor of 0.7 and will be sized for greater than 20 minutes of hydraulic retention time (HRT). Design criteria for the disinfection system are listed in Table 3-1.

Table 3-1 Well No. 6 Disinfection System

Parameter	Value
Chlorination System	
Disinfectant	Chlorine Gas
No. of Chlorinators	1 duty/1 standby Chlorinator will be fed from 2 duty/1 standby cylinders, each on separate weigh scale.
Design Dose	8.5 mg/L as free chlorine
Gas Feed Rate	1.04 kg/h (total) 0.52 kg/h (per duty cylinder)
Storage Volume	816 kg (12 full cylinders)
Storage Capacity	24 cylinders (12 full/12 empty)
Chlorine Contact Chamber	
Sizing Criteria	4-log virus inactivation (8 mg-min/L at 5°C)
Minimum Free Chlorine Residual	0.5 mg/L
Volume	46.8 m^3 (usable volume excluding freeboard in tank and bottom portion of tank)
Baffle Factor	0.7
HRT at Design Flow	23 min

Iron and manganese sequestration will be achieved through addition of 37.5% sodium silicate solution. Sufficient storage will be provided for 30 days of operation at the average dose. The sodium silicate feed system will be sized for a design dose of 25 mg/L. A water heater will be included for maintenance of the sodium silicate feed system. Flanged connections will be included for integration of future iron and manganese oxidation/filtration systems. Design criteria for the sodium silicate feed and storage system are listed in Table 3-2.

Table 3-2 Well No. 6 Sodium Silicate Feed and Storage System

Parameter	Value
Sodium Silicate Feed System	
Concentration	37.5%
Design Dose	25 mg/L
Average Dose	18 mg/L
No. of Pumps	1 duty/1 standby
Design Feed Rate	5.9 L/h

Parameter	Value
Sodium Silicate Storage System	
Storage Volume, Well No. 5	3,043 L (804 gal)
Storage Volume, Well No. 6	3,043 L (804 gal)
Storage Type	Two tanks (one per well, interconnected) with independent mixers

Finished water from Well No. 6 will combine with finished water from Well No. 5 downstream of the chlorine contact chambers and chlorine residual monitoring points.

3.3 Site Layout

A site layout showing the approximate location of the new Well No. 6, expansion of the existing building, location of the emergency generator, and location of the new chlorine contact chamber is shown in Appendix A. The upgrades required for Well No.2 are relatively minor and include pump and motor replacement and associated electrical and control upgrades if needed. As such, no change to the existing site layout is anticipated.

3.4 Equipment Layout

The existing building housing treatment equipment for Well No. 5 will be modified and expanded to include treatment equipment for Well No. 6. A preliminary equipment layout showing modifications to the building and new and relocated equipment is shown in Appendix A.

A new emergency power generator will be located outdoors in a dedicated acoustical enclosure and will include integrated fuel tank. The existing generator room will be converted to a new electrical room that will contain electrical switchgear for both Well No. 5 and Well No. 6. The existing electrical room will be converted to an operating room for the new Well No. 6 treatment train. The existing washroom and office area will be relocated as part of the building addition.

The disinfection systems for Well No. 5 and Well No. 6 will share a new, common chlorine room as part of an addition to the existing building. The existing disinfection equipment for Well No. 5 will be relocated.

The existing chlorine room will be expanded as part of the building addition and will be converted to a sodium silicate storage and feed room. The new sodium silicate storage and feed room will contain the sodium silicate feed and storage systems for Well No. 5 and Well No. 6. The existing sodium silicate feed equipment for Well No. 5 will be relocated. Sodium silicate storage for Well No. 5 will be converted to an above-ground tank storage system and the existing below grade storage tank will be demolished or abandoned.

3.5 Electrical, Instrumentation and Control, SCADA Requirements

A new electrical distribution system and communication system will be installed for Well No. 2 and Well No. 6 with a radio tower communication system, RPU, and MCC for all electrical equipment. Lighting and lighting control for the well will also be installed.

The generator will connect to well No.5 MCC to provide power for Wells No. 5 & No. 6. MCC will power to Well No.6 equipment. The existing Well 5 incoming feeder from Hydro has 200A (max) rating. It doesn't have sufficient ampacity to accommodate addition load from Well No. 6; Well No. 5 upgrade and potential future load. Hydro power feed upgrade request should be submitted to the local hydro company at the start of the project. Existing Well No. 2 and No.5 RPU; SCADA upgrades and programming will be implemented during the construction. Radio communication between well and master SCADA will also be established as part of the project.

Existing Well 2 and Well 5 well pump motor starters will be replaced with larger variable frequency drives and installed in the same location on the existing MCC of Wells No. 2 & 5. Demolition and replacement will be done during the construction.

3.6 Structural and Architectural Requirements

3.6.1 Water Servicing Pumping Station

The proposed upgrade works at the water servicing station includes the following.

Expanded Pumphouse for Well H

The existing pump house will be expanded to accommodate pumping of potable water from the proposed new Well No. 6 (Well Site H). The extensions to the existing pump house shall be in similar lines with the existing pump house.

Additions to the pump house building shall be a pitched roof CMU load bearing structure with brick veneer facing and pitched roof, matching the existing structure. Continuous concrete wall footing foundation at appropriate frost depth shall be provided below the exterior walls to prevent any frost heave underneath building foundation. A concrete slab-on-grade foundation shall be provided within the outer wall footing. Metal deck roof supported on steel trusses at appropriate intervals shall be provided to transfer gravity and lateral loads to the CMU load bearing walls and to the foundation.

New Outdoor Standby Generator Pad

A 300mm thick cast in place reinforced concrete slab on grade foundation pad with appropriate plan dimensions may be provided to support the new standby generator. Frost heave below this foundation shall be prevented.

3.7 Permits and Approvals

The following permits and approvals are anticipated for the expanded pumphouse for Well H:

- Amendment to the Ministry of the Environment Conservation and Parks (MECP) Drinking Water Works Permit (DWWP), Municipal Drinking Water Licence (MDWL), and Permit to Take Water (PTTW).
- Township of King Site Plan Approval.
- Township of King Building Permit.
- Electrical Safety Authority (ESA) plan approval.
- Toronto and Region Conservation Authority (TRCA) Approval.
- Technical Standards and Safety Authority (TSSA) Approval.

It is anticipated that the following permits and approvals will be needed for Well 2:

- Amendment to the Ministry of the Environment Conservation and Parks (MECP) Drinking Water Works Permit (DWWP), Municipal Drinking Water Licence (MDWL), and Permit to Take Water (PTTW).

3.8 Opinions of Probable Cost

Black & Veatch has prepared opinions of probable cost suitable for this stage of the design (Table 3-3). These should be considered indicative cost estimates (Class D Cost Estimates). These have not been developed from bottom up. As the design moves through the subsequent stages, where various design elements are firmed up, the cost estimates will be refined as well. Black & Veatch will prepare and present a more detailed cost estimate in the next stage, which is preliminary design.

Table 3-3 Opinion of Probable Cost for Well Site No. 6 and Well No.2

Discipline	Million Dollars (2021)
Site and Civil	$0.5 Million
Structural and Architectural	$0.8 Million
Process and Building Mechanical	$2.1 Million
Electrical, Instrumentation and Control, SCADA	$0.8 Million
Total Capital Cost of Infrastructure	$4.2 Million
General Requirements (@ 15% of Capital Cost)	$0.7 Million
Contingencies (@20% of Capital Cost + General Requirements)	$1.0 Million
Engineering, Legal and Administration (@ 20% of (Capital Cost + General Requirements + Contingencies))	$1.2 Million
Total Cost Including Engineering and Contingencies	**$7.1 Million**

Well No.2 will be associated with relatively minor cost as compared with Well Site No.6 construction. A cost allocation of $0.2 Million of the total cost allocation of $7.1 Million is considered appropriate for a new Well Pump and associated electrical and control upgrades for Well No.2. This cost will be further refined during the preliminary design stage.

4.0 Bibliography

Black & Veatch. (January 2019). *Class Environmental Assessment for Water and Wastewater Servicing in the Community of Nobleton – Study 1B: Wastewater System Capacity Optimization Study.* Regional Municipality of York

Black & Veatch. (June 2019). *Class Environmental Assessment for Water and Wastewater Servicing in the Community of Nobleton – Study 1A: Water System Capacity Optimization Study.* Regional Municipality of York

Black & Veatch. (June 2019). *Phase 1: Identify the Problem or Opportunity. Technical Memo No. 1.* Regional Municipality of York

Hutchinson. (May 2019). *Humber River Assimilative Capacity Study 2017-2018 (DRAFT).* Nobleton: Regional Municipality of York

TSH. (February 2007). *Nobleton Sewage Works Design Report (DRAFT).* Nobleton: Regional Municipality of York, Slokker Canada Corporation

York Region. (2016). *The Regional Municipality of York Water and Wastewater Master Plan*

York Region. (2019). *The Regional Municipality of York. Characterization and Comprehensive Review of Groundwater Supply Resources of the Regional Municipality of York (York Region Groundwater Supply Options Study). In Support of the 2021 Water and Wastewater Master Plan Update*

Nobleton Sewage Works Project *Forcemain and Effluent Sewer As-builts* (October 2012)

Nobleton Sewage Works Project *Water Pollution Control Plant As-builts* (October 2012)

Nobleton Water Supply System Upgrades Well No. 2 As-builts (December 2002)

Nobleton Well No. 5 and Township of King Sanitary Sewer and Watermain as Constructed (December 2015)

Nobleton Well 2 Operation Manual (August 2013)

Nobleton Well 5 Operation Manual (September 2015)