Appendix A: Baseline Plant Capacity Review

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Region of Halton

Oakville WPP Re-Rating Class EA
Memo #3: Technical Baseline Review Memo

December 2014

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1. Introduction and Purpose

1.1 Introduction

The Oakville Water Purification Plant (WPP) is one of three surface water treatment plants within the Regional Municipality of Halton's (Region's) system. The WPP is rated for a production capacity of 109 ML/d and is currently undergoing plant upgrades.

The plant was originally constructed in the 1960s and has undergone numerous upgrades and expansions since that time. The most recent upgrades/expansion has occurred from 2007 to present under a two phased expansion/upgrade program. Phase 1 upgrades were completed in 2008 and included pre-chlorination for zebra mussel control, pre-treatment using Actiflo©, taste and odor (T&O) control and primary disinfection through intermediate ozonation, dual media filtration, air scour (two new blowers) backwash system, secondary disinfection by gaseous chlorination, and process residuals treatment by gravity thickening/clarification.

Phase 2 upgrades are currently under construction and are scheduled to be complete in 2014. Phase 2 upgrades include the replacement of the four vertical turbine low lift pumps, filter underdrain replacement, filter backwash pump replacement, a new high lift pumping station, modifications to the waste holding tank, and other miscellaneous upgrades.

To achieve a net finished water production capacity of 109 ML/d, up to 120 ML/d (gross) must be withdrawn from Lake Ontario to compensate for 10 percent losses in the pre-treatment sludge production, filter backwash waste, filter-to-waste (FTW) flows, and other minor plant water uses. To provide a finished water production rate of 130 ML/d (net), approximately 143 ML/d of raw water must be withdrawn from Lake Ontario and treated by the Oakville WPP.

In addition to re-rating the WPP, the Region has requested an investigation into the ability of the WPP to treat high turbidity source water due to storm events. The Region would like to evaluate alternatives for treating the high turbidity water, including: optimizing existing WPP processes or extending the raw water intake.

1.2 Purpose

Due to increased growth throughout the Region's service area it is the Region's intent to officially re-rate the Oakville WPP to a net finished water production capacity of 130 ML/d. GHD Inc., in association with exp Services Inc. and Hatch Mott McDonald (HMM), was retained by the Region to provide engineering services for a Municipal Class Environmental Assessment study for the Oakville WPP including the overall re-rating of the plant to 130 ML/d (net).

A Municipal Class Environmental Assessment (Class EA) is being undertaken to evaluate the potential for optimizing current WPP treatment processes to treat the storm-event turbidity issue versus extending the raw water intake; as well as to facilitate the overall re-rating of the Plant's capacity to 130 ML/d (net) for approval with the Ministry of Environment and Climate Change.

The project has been subdivided into Part A and Part B project components. Part A of the project includes an evaluation of the plant for potential re-rating to a net finished water production capacity of 130 ML/d through process optimization and changes to operation of the facility. Part A will also include any operational recommendations for plant capacity improvements which would satisfy a capacity increase to 130 ML/d. The Part A Class EA work will also evaluate plant based versus intake extension based alternatives to address the high raw water turbidity resulting from storm events. Depending on the complexity of the in-plant improvements required to meet the desired capacity increase or the need to extend the intake; the study may be upgraded to a Schedule C Class EA process.

In the event that the preferred alternative or required alternative to increase the plant capacity is to modify/replace/extend the existing intake, the requirements for a Schedule B or C Class EA and preliminary design of the intake will be completed. This effort is considered to be Part B of the Project.

The purpose of this technical memorandum pertains to only Part A of the project, which is to assess the baseline status of the existing processes at the Oakville WPP and document information gained from GHD's site visit and interviews with WPP operations staff on July 8 and 9, 2014. The memorandum presents the following:

- Overview of the Oakville WPP process, operation and hydraulic parameters;
- Existing equipment capacity calculations;
- Information compiled from GHD's site visit and interviews with WPP operations staff.

GHD utilized information gained during the July 8th and 9th , 2014 site visits, follow-up discussions with the Region, Ministry of Environment and Climate Change (MOECC) design guidelines, the Drinking Water Works Permit – South Halton (June 2014), Drinking Water License – South Halton (September 2014), Permit To Take Water – Oakville OWPP (February 2012), Phase 1 (October 2008) Oakville WPP Record Drawings, Phase 2 (May 2012) Oakville WPP design drawings, and other consultant's technical memoranda/reports (Oakville WPP Preliminary Design Report, September 2003; Technical Memo 1 – Oakville WPP Process Capacity Review, June 2010, and Oakville WPP Phase 2 Upgrades Predesign Report, March 2011) to prepare this memorandum.

2. Oakville WPP Process Overview

Lake Ontario serves as the raw water source for the Oakville WPP. Raw water is withdrawn by way of two (2130 mm extension and 1828 mm diameter main intake) intake pipes with intake cribs leading to two traveling screen units in the raw water pump station, which empty into a below grade wetwell. Four vertical turbine low lift pumps move raw water from the wet well and pump to the Actiflo© trains. Pre-chlorination is performed seasonally at the intake crib for zebra mussel control. The low lift branch piping (600 mm diameter) combines into a 900 mm diameter header upstream of the in-line flash mixer where PACL/Alum is injected for coagulation. Downstream of the flash mixer the header increases to 1200 mm and then splits into two 750 mm diameter inlets feeding each of two Actiflo© units that operate in parallel.

Actiflo© is a high-rate ballasted-flocculation upflow clarification process that takes the place of conventional flocculation-sedimentation. The Actiflo© system consists of coagulation, injection, maturation and settling tanks. Water exits each 750 mm diameter pipe and enters at the bottom of each coagulation tank. Following coagulation, coagulated water overflows into the top of the injection tank where microsand and polymer are injected simultaneously to form ballasted floc with high settling velocities. Flocculated water underflows from the injection tank into the maturation tank. The maturation tank incorporates slow mixing to promote floc formation, aggregation, and settling. As the flow containing ballasted floc exits the flocculation tank the weight of the microsand causes the ballasted floc to settle upon entering the settling tank. The flow continues through lamella tubes and exits the Actiflo© process for further treatment. The material that settles in the settling tank is continuously withdrawn by way of a scraper mechanism and two recirculation pumps. The settled material is sent to the hydrocyclone where the microsand is separated from the sludge and re-injected into the flocculation tank while the sludge is gravity fed to the hydrocyclone detention tank and then pumped to the plant's residuals treatment (gravity thickener) system.

From the Actiflo© units, clarified water flows through two 900 mm diameter lines to two ozone contactors. Ozone is applied by sidestream injection to provide primary disinfection and taste and odour control. The ozone system consists of one liquid oxygen (LOX) storage tank and four vaporizers, a supplemental nitrogen boost system, two Fuji ozone generators, an ozone contact/dissolution system, and an ozone off-gas destruct system. During extreme taste and odor events, hydrogen peroxide can be injected downstream of ozonation (within Cell No. 7 of the ozone contactor) to generate additional free radicals for advanced oxidation. Calcium thiosulphate is added downstream of the ozone contactors for quenching any excess ozone residual in the ozonated water prior to filtration. Exhaust gas from the ozone contactors is pulled by vacuum through three ozone destruct units.

A 1500 mm diameter ozonated water conduit delivers water to a flow splitting structure with four outlet pipes. The outlet pipes feed a filter influent channel at four different locations. Eight dual-media (sand and anthracite) filters comprise the plant's filtration system. Filter backwash involves air scour and low/high rate backwashing with treated water from the clearwell. The filtration system is capable of operating in biological mode with the provision to add calcium thiosulphate at the backwash header for dechlorination of the backwash water. Filter discharge from the first 15 minutes of a newly backwashed filter at 50% of the filtration rate, is sent as FTW. The WPP has the ability to inject filter aid polymer upstream of the filters during high turbidity events to aid in turbidity removal.

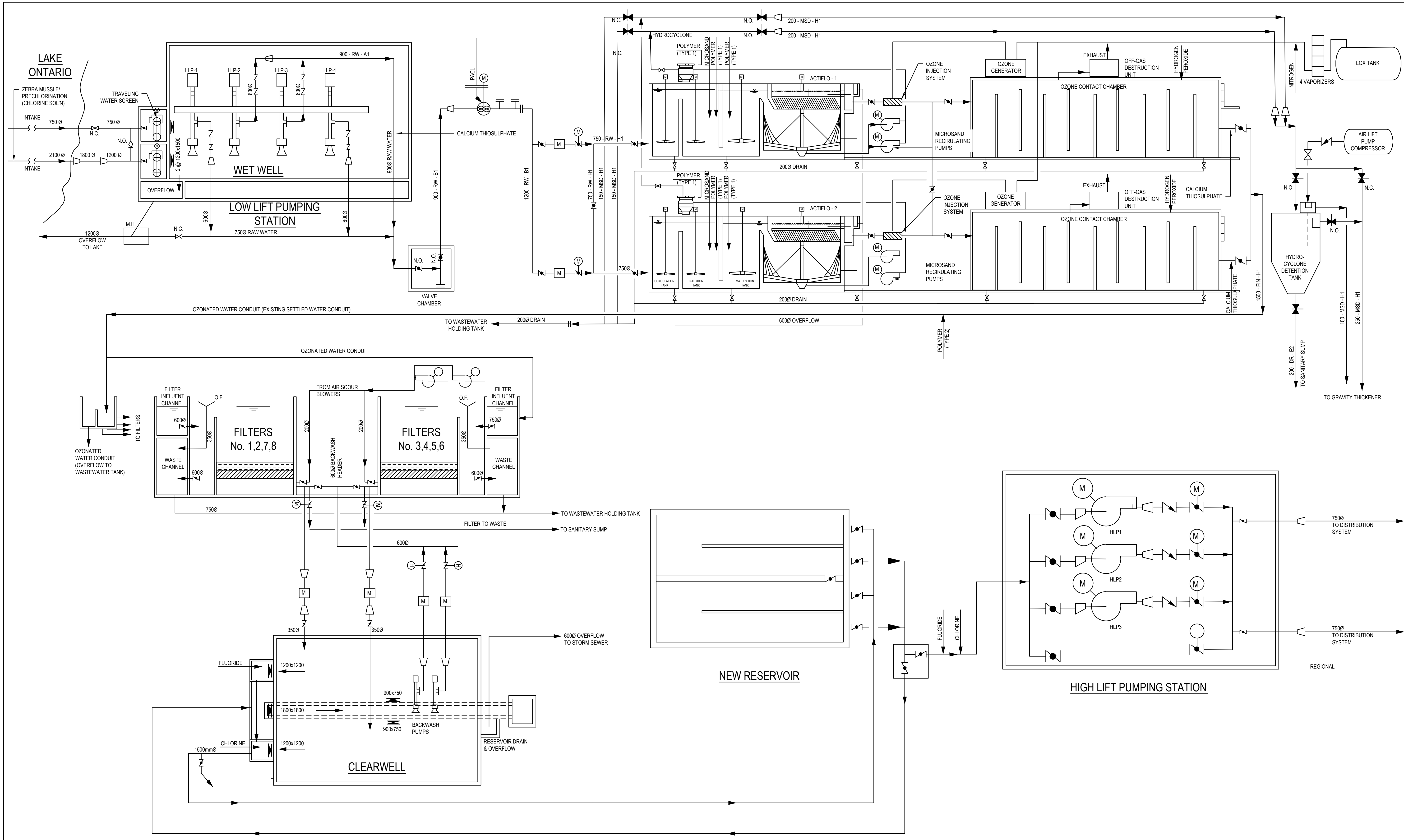
The filtered water is collected in two clearwells, located below the filters, and conveyed via a 1500 mm diameter transfer pipe to the underground 2,000 m^3 treated water reservoir that is divided in half to form two channels. Chlorine solution is added to the filtered water in the transfer pipe to provide disinfection (CT) credit and a chlorine residual within the distribution system.

A 1500 mm diameter pipe connects the reservoir to the high lift pumping station. Fluoridation is provided by the addition of hydrofluosilicic acid to the treated water upstream of the high lift pumps. Three horizontal split-case centrifugal pumps comprise the high lift pumping station and provide water to the distribution system. The distribution system is fed via two 900 mm discharge lines from the high lift pumping station (east and west), which are interconnected along Lakeshore Blvd.

Figure 2-1 is a process flow diagram showing the major processes at the Oakville WPP, excluding the Residuals Treatment System.

2.1 Water Quality

Table 2-1 provides a summary of select raw and finished water quality for the Oakville WPP from January 2008 through June 2014.



PRELIMINARY

No	Revision	Drawn	Job Manager	Project Director	Date

Note: * indicates signatures on original issue of drawing or last revision of drawing

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Client **REGIONAL MUNICIPALITY OF HALTON**
Project **CLASS EA STUDY FOR OAKVILLE WPP**
Title **PROCESS FLOW DIAGRAM**

FIGURE 2-1

Plot Date: 19 December 2014 - 2:49 PM Plotted by: Barry Beebe Cad File No: G:\86\16793\CADD\FIG\WPPP Process Flow Diagram.dwg

Table 2-1 Oakville WPP Water Quality Data January 2008 – June 2014

Parameter	Raw Water			Finished Water		
	Min.	Avg.	Max.	Min.	Avg.	Max.
Turbidity (NTU)	0.12	1.44	34.60	0.07	0.17	0.86
Dissolved Organic Carbon (mg/L)	1.8	2.1	2.9	1.4	1.6	2.4
pH	7.7	8.1	8.6	7.1	7.5	7.9
Temperature (^{o}C)	3	14	22	3	14	22
Free Chlorine (mg/L)	-	-	-	0.7	1.5	2.2
Total Chlorine (mg/L)	-	-	-	0.8	1.7	2.2
Geosmin (ng/L)	2	4	9	2	3	7
MIB (ng/L)	0	<2	3	0	<2	3
Total Coliform (CFU/100 mL)	0	146	24,700	0	0	0
Total Trihalomethanes (ug/L)	-	-	-	0.6	8.1	18.8
Bromide (ug/L)	9	34.0	71.0	<4	8.8	17.0
Bromate (ug/L)	<3	<3	<3	<3	4	7

As shown in the table, the maximum raw water turbidity is 34.60 nephelometric turbidity units (NTU) because the WPP was historically removed from service during high turbidity events (turbidity > 100 NTU). Past WPP operational practice of removing the plant from service during high turbidity events is a legacy issue because it was challenging to operate the Actiflo© units and keep settled water turbidities below 1 NTU, thus affecting performance of the downstream units; and was not necessarily related to hydraulic performance of the plant. Since June 2014, the WPP has remained on line during high turbidity events.

3. Oakville WPP Process Systems

This section provides information on each major unit process comprising the Oakville WPP, except the chemical systems, which are covered under Section 4. Lessons learned from GHD's July 8th and 9th site visit are included for some of the major processes. The lessons learned were compiled from discussions with Oakville WPP operations staff and pertain to issues related to the particular process.

3.1 Intake

The current arrangement of the raw water intake piping for the Oakville WPP provides for two intake pipes. In reference to Figure 3-1 shown below, the plan information as presented was prepared from the As-Built drawing information for Region Contract SPW-502-80(A) for the construction of the 2130 mm diameter intake. The Water Intake Extension as was provided by the Region for this assignment.

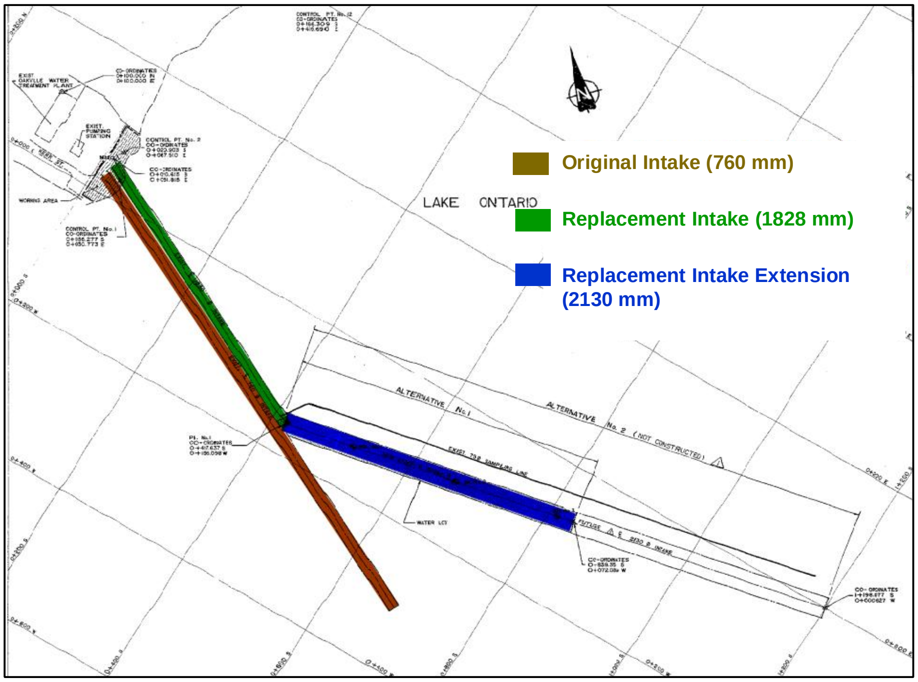


Figure 3-1 Plan View of Existing Intakes

The original smaller 760 mm diameter Intake No. 1 (shown in brown) extends an approximate length of 725 meters +/- from shore, was constructed around 1947 and is **out of service**. Intake No. 1 provides for a 4.7 m submergence at a low water lake level of 73.76 m.

Intake No. 2, which currently provides the Lake Ontario raw water supply to the plant was constructed to replace Intake No. 1. Intake No. 2 extends an approximate total length of 858 meters +/- from the low lift pump station, and was constructed in two stages. Stage 1 (i.e., 458 m +/- length shown in 'green') was constructed around 1977 and consists of an 1828 mm diameter pipe. The stage 2 extension (i.e., 400 m +/- length as shown in blue) to the Intake Crib Alternative #1 location was increased in size to 2130 mm diameter in size. The average water depth at the end of extended length of Intake No. 2 to crib location Alternative #1 provides for a water depth in the order of 9.7 m assuming a low lake level elevation of 73.76 m (64.0 Lake bottom elevation). The rated capacity of Intake No. 2 is noted as being 315 ML/d.

Intake No. 2 provides for a 30.0 meter wide water lot. In addition, the water lot for Intake No. 2 provides for future extension to Intake No. 2 for an additional length of 383 meters beyond the current intake crib location, which would provide for a water depth in the order of 13.5 m using the low lake level elevation of 73.76 m (60.2 m lake bottom elevation). Also shown on the As-Built drawings is a 75 mm diameter raw water sampling line which is reported to be plugged, whereby raw water samples are collected at the low lift chamber.

Available Data from Recent Lake Ontario Studies undertaken on behalf of Halton Region for the Mid-Halton WWTP Expansion

As part of the Class Environmental Assessment for the expansion of the Mid Halton WWTP, 'Baird & Associates' were retained by the Region to undertake numerical modelling to assess the impacts of the effluent from the Mid-Halton and Oakville South West WWTPs on the water quality in Lake Ontario, at specific locations as required by the MOECC, and in particular addressing the locations of the Burloak and Oakville WPP's intakes. In reference to Figure 3-2 shown below, the plan information as presented was prepared by 'Baird & Associates' and was included as part of their

November 2009 – Assimilative Capacity Study for the Mid Halton WWTP Phase IV & Phase V Expansion Report. Figure 3-2 is provided herewith to illustrate the proximity of the Mid-Halton Outfall to the Burloak & Oakville Intakes.

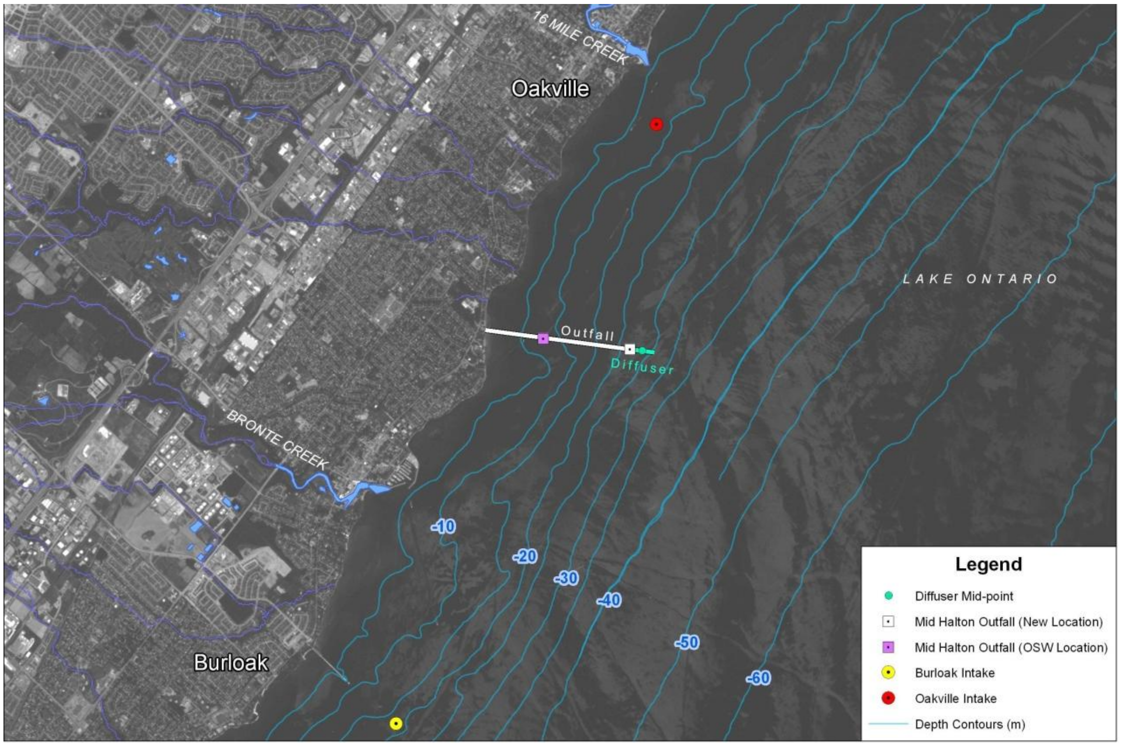


Figure 3-2 Overview of Existing Outfall (courtesy of Baird and Associates)

The Danish Hydraulic Institute's "Mike3 Model" was used and run by Baird to simulate twenty conditions to reflect different times of the year to assess the effect of the effluent plume in the far-field zone (i.e., at beach locations and water treatment plant intakes in the proximity of the Mid-Halton WWTP outfall). The modelling results showed the Provincial Water Quality Objectives (PWQO) will be met at the edge of the 900 meter mixing zone and at the Oakville and Burloak WPP intakes and at the shoreline.

The Oakville WPP intake is located 2.9 km north of the proposed location for the Mid-Halton Outfall and differs in a water depth of 9 +/- meters. The Burloak WPP intake is situated 5.6 km southwest of the proposed Mid-Halton diffuser at a water depth of 19 m. A summary of the estimated dilution near lake bottom, where the intake cribs are located is presented in Table 4.6 of the 2009 Baird Report, along with the minimum dilution predicted in the water column. The Baird report noted that for all conditions simulated, the results showed that the effluent from the Mid-Halton WWTP achieved the PWQOs at both intake locations as referenced in Table 3.7 of the report for the required dilutions.

In terms of additional Geophysical Surveys and/or Bathymetry Studies to be carried out in proximity of the Oakville WPP intake, it is anticipated that the need for these studies will be determined by the project team, following the completion of 'Part A' of the project that relates to the completion of the Hydraulic Assessment of the Oakville WPP, whereby it has been determined that the preferred solution for re-rating of the Oakville WPP is to modify and/or replace the existing intake.

Available Geotechnical Data from Recent Lake Ontario Studies undertaken on behalf of Halton Region for Mid Halton WWTP Expansion

By review of the As-Built drawing information for Region Contract SPW-502-80(A) for the construction of the 2130 mm diameter Oakville Water Intake Extension, three borehole (BH) locations were noted on the contract drawings, with notation that a series of probes were also undertaken for the purpose of design of the intake extension. The As-Built drawings also provided a lake bottom profile to the end of the water lot to reflect the Alternative #2 location for the intake crib; however the detail of information did not clearly identify the anticipated bed rock profile.

In advance of a detailed design phase for a possible further extension to the Oakville Intake, and only if and when this requirement is clearly identified, it is recommended that additional geotechnical investigations be undertaken along the alignment for the length of the affected new section(s) of the intake. The extent as to the number of and spacing and depths of these future BHs will be dictated by considerations as to alternative methods of intake construction (i.e., cut & cover dredging of the lake bottom or by tunnel construction methods below the lake bottom).

In reference to the Burloak WPP intake works, a Geotechnical Data Report dated February 2005 was prepared by Geo-Canada Ltd. (Report # 15961.06.05). In addition a Volume 2 Geotechnical Interpretation and Recommendations Report dated February 2005 was also prepared by Geo-Canada Ltd. (Report # G-04.1003A).

In reference to the Mid-Halton outfall works, a Geotechnical Data Report titled 'Geotechnical Data Report – 'Offshore Portion of the Mid Halton Waste Water Treatment Plant Outfall' dated December 2012 was prepared by Golder Associates (Report # 10-1117-0047). In addition a Geotechnical Interpretation Report dated February 2013 was also prepared by Golder Associated (Report # 10-1117-0047).

Although the Burloak Intake and the Mid-Halton Outfall are in relatively close proximity to the Oakville WPP Intake location (i.e., 8.5 km from the Burloak intake & 2.9 km from the Mid-Halton outfall), it is suggested that the recent Geotechnical Investigations undertaken for both the Burloak WPP intake and the Mid-Halton WWTP outfall should only be used as reference documents as indicators to forecast anticipated geotechnical conditions of the lake bottom at increased water depths and the longer distances from the shoreline to reflect a potential extension to the length of the Oakville WPP intake.

Additional Information Requirements:

Video Inspection:
Additional investigation (i.e., CCTV) is required to inspect the condition of the existing Oakville intake.

From the internal CCTV video, a 'Condition Assessment' of the intake piping can be carried out to determine if the capacity is compromised by the build-up of zebra mussels on the interior surfaces and/or at the intake crib.

Review of Raw Water Quality Records:
It is understood that Region Plant Operations have approximately five years of Raw Water Data records (i.e., measured NTU levels) from samples taken at the Low Lift Chamber.

An analysis of the Raw Water Data should be undertaken to cross reference this data to historic records of storm events and hence runoffs via nearby Sixteen Mile Creek.

The constraints that pertain to the location of Intake No. 2 are as follows:

- Lake water turbidity is influenced by storm water runoff from Sixteen Mile Creek post storm water events.
- Hydraulic capacity of the intake is reduced as a result of the smaller 1828 mm diameter section.
- The internal diameter of the intake is suspected to be further reduced by the potential buildup of zebra mussels. Further investigation into the condition of the intake is required.
- Lake Ontario water quality in the approximate area of the intake could potentially be impacted once the Mid-Halton WWTP comes on line and the new outfall for the plant is fully in service.

3.1.1 Site Visit Lessons Learned

The 50 mm diameter turbidity sample line runs from the intake crib to a turbidimeter located in the low lift pumping station. Operations staff noted that the sample line is plugged and has been out of service for over a year. Turbidity is currently sampled within the low lift pump station wetwell. Sampling turbidity at the intake provides operations an early warning for turbidity spikes (water quality upsets) and not knowing the raw water turbidity in the Lake makes it difficult for operators to react to the changing water quality in advance of a turbidity event. In the past the WPP was taken out of service when the raw water turbidity was above 100 NTU at the intake crib. Since June 2014, operations staff have kept the WPP on-line to assess the plant's ability to treat higher turbidities. GHD will continue to review plant performance under high turbidity conditions, after June 2014, as this study progresses.

Chlorine is added within the intake crib for zebra mussel control. Operations maintains a 0.2-0.3 mg/L chlorine residual through the intake piping and quenches the residual with calcium thiosulphate at the low lift pump station upstream of coagulation and the Actiflo© system. A 50 mm chlorine feed line is routed inside the 2130 mm intake pipe to the crib, and passes through a 2130 mm butterfly isolation valve.

3.2 Traveling Screens

Two USFilter Model 45A Automatic Traveling Water Screens are used to prevent debris and fish from entering the low lift wetwell and damaging the low lift pumps and downstream equipment. The mechanical screens are operated in one duty and one stand-by mode. The traveling screens are approximately 1.5 m wide with a stainless steel screen mesh size of 9.5 mm. MOECC's Design Guidelines for Drinking Water Systems (MOECC, 2008) recommends sizing a screen for a maximum velocity of water through the screen of 0.6 m/s. The screen manufacturer recommends a maximum velocity of 0.76 m/s. The traveling screens were designed for a water depth in the screen well of approximately 2.4 m (minimum water level) to 6.4 m (maximum water level), based on information from the screen manufacturer. Based on the minimum water level (2.4 m) in the screen well, at a flow of 143 ML/d, the velocity through the screen is 0.81 m/s, which is greater than the manufacturer recommended maximum velocity. The June 2014 DWWP states that the combined capacity of the intake screens is 137.5 ML/d.

3.2.1 Site Visit Lessons Learned

The screens non-chlorinated flushing system was upgraded as part of the Phase 2 Upgrades Project. Flushing water from the screens flows back to Lake Ontario. Oakville WPP operations staff noted that they sometimes have issues with algae building up on the screens. Intermittent flushing with chlorinated water would help alleviate algae buildup, but would require dechlorination prior to disposal to Lake Ontario.

The flushing water for the screens does not pass through the raw water meter, and thus is not accounted for in the total plant water. The flushing water should be metered to account for this water. The flushing water requirement is provided by two screen flushing pumps (one duty / one standby) and requires 8.6 L/s (0.74 ML/d) at 414 (kilopascals) kPa.

3.3 Low Lift Pumps

The low lift pump station consists of a pump wet well with four vertical turbine pumps, which were upgraded as part of the Phase 2 Upgrades. Two of the low lift pumps (LLP1 and LLP4) had their motors rebuilt and LLP2 and LLP3 were replaced with new larger pumps as part of the Phase 2 Upgrades. Variable frequency drives (VFDs) have been installed on LLP1, LLP2 and LLP3, with a soft start on LLP4 Table 3-1 summarizes the capacity of the low lift pumps.

Table 3-2 Low Lift Pumps

Pump	Flow (ML/d)	TDH (m)
LLP1	54.5	18.29
LLP2	47.0	18.59
LLP3	47.0	18.59
LLP4	54.5	18.29

The low lift pumps operate in a three duty/one standby mode to draw raw water from the low lift pump wet well. The capacity of the low lift pumps with the largest pump out of service is 148.5 ML/d.

3.4 In-Line Mixer (PACL Injection)

An in-line Lightnin Model 75-S-25 vertical turbine mixer installed within the 1200 mm diameter raw water pipeline imparts mixing energy to a PACL coagulant. The mixer motor is 25 hp, 230 rpm with variable speed. The MOECC drinking water system guidelines recommends a mixing intensity velocity gradient (G-value) in the order 1000 s^{-1}. Kawamura (2000) recommends a G x t (time) range between 500-1600 for a flash mixing system. The formula for calculating the G-value is shown below.

$$G = \left(\frac{P}{\mu V}\right)^{0.5}$$

Where, P = power input = 15000 J/s assuming 80% mixer efficiency

μ = absolute viscosity of water = 0.001 Ns/m^2 at 20^oC

V = volume of mixing zone = 0.7 m^3

If the in-line mixer provides 0.5 seconds of mixing time, the mixer provides a G-value of 3,200 s^{-1} and a G x t value of approximately 1300 at 143 ML/d, which is sufficient according to the MOECC design guidelines and Kawamura's recommendations.

3.5 Actiflo© System

Table 3-2 shows the current plant production rates during different times of the year.

Table 3-2 Current Plant Production Rates

Flows	Flow Rate (ML/d)
Rated Capacity	109
Normal Winter Flows (October-April)	60
Normal Summer Flow (April-September)	109

The Actiflo© Process operates similarly to a conventional flocculation-sedimentation design, with the exception that microsand/proprietary silica with an effective size of 60-130 microns is added to the water during the flocculation process in order to enhance both coagulation and settling. The microsand adds surface area in the coagulation process, which significantly improves the frequency of collision of dispersed or colloidal particles in the raw water with oppositely charged coagulated floc. This action accelerates the coagulation and flocculation processes. The microsand also provides "ballast" to the floc, resulting in floc settling velocities that are 25 to 35 times faster than floc produced in conventional flocculation sedimentation processes. Figure 3-3 is a schematic of the Actiflo© process.

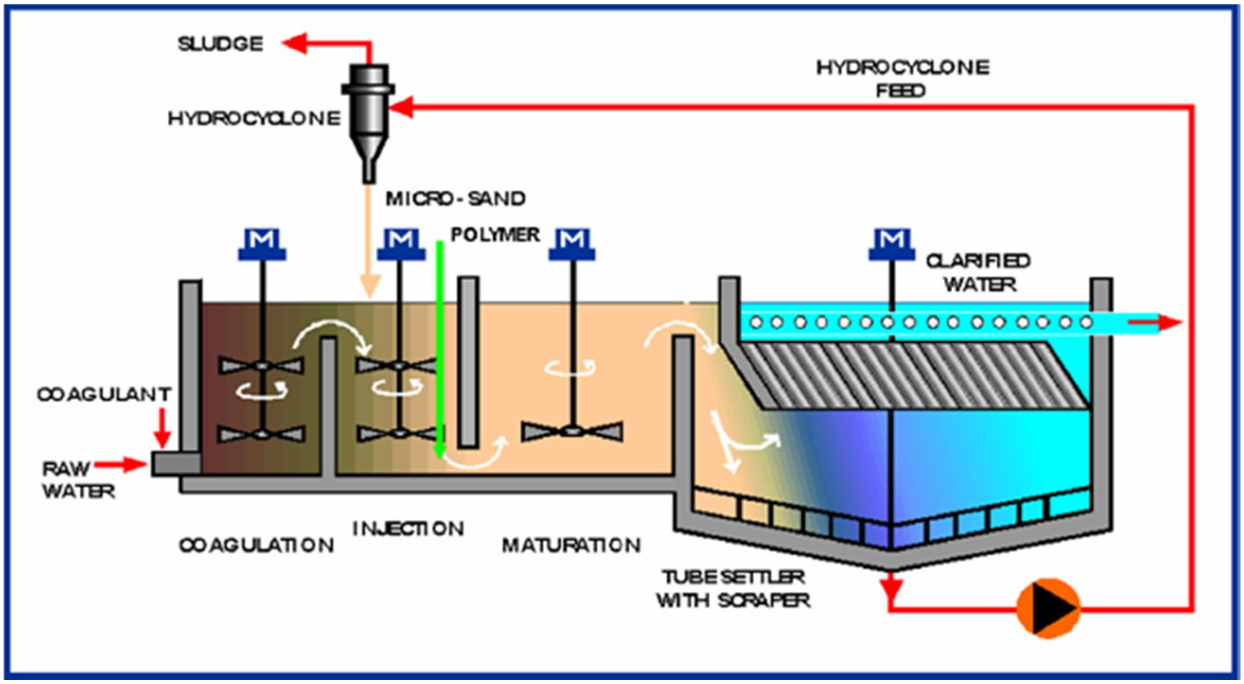


Figure 3-3 Actiflo© Schematic

At the Oakville WPP, there are two parallel treatment trains with one Actiflo© unit per train. Each Actiflo© unit consists of

- Coagulation
- Injection
- Maturation and
- Settling tanks

In addition, each Actiflo© unit is equipped with a polymer system, a microsand injection system and a microsand recycling system. An additional coagulation injection point is provided in the coagulation tank for injecting similar or alternative coagulant. Microsand and polymer are added in the injection tank and there is an additional injection point for polymer addition in the maturation tank. Settling is achieved in the lamella clarifier. The settling tanks consist of a conventional arrangement of tube settlers, collection weir troughs and solids collection system. Flow enters the “settling basin” through an underflow baffle and flows through the bottom of the 60 degree tube settlers. Solids are separated from the solution based on lamella principles of settling and clarified water egresses out the top of the tubes and over the V-notched weirs into an effluent trough (collector channel). Settled sludge from the sedimentation basins collects in a sloped sump at the bottom of the basin. Each train has two dedicated recirculation pumps under the respective basins (one operational and one standby) that recycle the sludge to hydrocyclones. Energy from pumping is effectively converted to centrifugal forces within the body of the hydrocyclone causing aluminum humate and hydroxide floc to be separated from the higher density microsand. Once separated, the microsand is concentrated and discharged from the bottom of the hydrocyclone and re-injected into the process for reuse. The lighter density aluminum humate and hydroxide floc exit the top of the hydrocyclone with a major fraction of process residuals through the hydrocyclone vortex overflow to the hydrocyclone detention tank by gravity and are finally pumped to the residuals treatment process.

Dry Polymer and Microsand Feeding System: Polymer (anionic) mixing is conducted in an automated batch process i.e. dry polymer is fed to a hopper and is mixed with cold water in a tray. The batched polymer is fed to two 1.8 m^3 aging tanks. Metering pumps (3 pumps, 2 active, 1 stand-by) convey the aged polymer solution which is added into the injection basin of the Actiflo© unit. Polymer can be fed at three locations: injection tank, maturation tank and at the hydrocyclone. Plant staff noted that the hopper has a tendency to clog with the polymer.

The microsand handling system is comprised of microsand (silica) bags that are added manually into a hopper and mixed with water, resulting in a sand slurry. Metering pumps convey the sand slurry through a single line to the injection basin of the Actiflo© unit.

Each Actiflo© train has been sized to provide an individual treatment capacity of 60 ML/d for a total treatment capacity of 120 ML/d. Also, each unit is sized to treat a peak flow rate of 90 ML/d i.e. a total treatment capacity of 180 ML/d (Technical Memo 1-Oakville WPP, Process Capacity Review, Associated Engineering, 2010). During the site visit it was noticed that the plant was operating only one Actiflo© train with a plant flow of 40 ML/d. Table 3-3 summarizes the existing system design and operational criteria.

Table 3-3 Existing Design and Operational Criteria for Actiflo© System*

Unit Process	Parameter and Units	Value
Actiflo© Units (Hydraulic Basis of Design)	Total Nominal Design Flow (ML/d)	120
	Number of Trains	2
	Nominal Design Flow per Train (ML/d)	60
	Total Max. Design Flow Rate (ML/d)	180
	Max. Design Flow Rate per Train (ML/d)	90
Coagulation Tank Design	Number of Tanks per Train	1
	Length, m	4.067
	Width, m	3.83
	Side Water Depth, m	5.409
	Total Depth, m	5.706
	Speed Mixer Number	1
	Mixer Motor, HP and RPM	7.5; 31.27
	HRT[1] at Nominal Design Flow, min	2.02
	HRT at Peak Design Flow Rate, min.	1.35
Injection Tank Design	Number of Tanks per Train	1
	Length, m	4.067
	Width, m	3.83
	Side Water Depth, m	5.409
	Total Depth, m	5.706
	Speed Mixer	1
	Mixer Motor, HP and RPM	7.5; 31.37
	Number of Tanks per Train	1
	Hydrocyclone Return Flow per Tank*, ML/d	0.36
	Total Flow per Injection Tank at Peak Flow Rate, ML/d	90.36
	Total Flow per Injection Tank at Nominal Design Flow, ML/d	60
	HRT at Nominal Design Flow, min	2.02
	HRT at Maximum Design Flow Rate, min.	1.34
Maturation Tank Design	Number of Tanks per Train	1
	Length, m	7.484
	Width, m	5.92
	Side Water Depth, m	5.266
	Total Depth, m	5.706
	Speed Mixer (VFD)	1
	Mixer Motor, HP and RPM	20; 15.27
	Number of Tanks per Train	1
	Total Flow per Maturation Tank at Peak Design Flow Rate, ML/d	90.36
	Total Flow per Maturation Tank at Nominal Design Flow, ML/d	60
	HRT at Nominal Design Flow, min	5.60
	HRT at Peak Flow Rate, min.	3.72

Unit Process		
Settling Tank Design	Number of Tanks per Train	1
	Total Length, m	8.434
	Total Width, m	9.31
	Side water Depth, m	5.265
	Total Depth, m	5.706
	Projected Surface Area, m^{2} '	62.5
	Projected Overflow Rate at Nominal Design Flow , m/hour	40
	Projected Overflow Rate at Max. Flow Rate, m/hour	60
	Sludge Underflow per Tank, ML/d	1.8
	Total Influent Flow per Settling Tank at Peak Design Flow Rate, ML/d	90.36
	Total Influent Flow Per Settling Tank at Nominal Design Flow, ML/d	60
	Total Effluent Flow per Settling Tank at Peak Flow Rate, ML/d	88.56
	Total Effluent Flow per Settling Tank at Nominal Design Flow, ML/d	60
	Type of Sedimentation	Laminar Upflow through lamella settling tubes
	Variable Speed Sludge Scraper Assembly	One per tank
	Scraper Motor, HP	3
Sand-Sludge Recirculation Circuit	Total number of Hydrocyclones	4
	Number of Hydrocyclones per Injection Tank	2 (1 per pump)
	Number Sand Recirculation Pumps per Injection Tank	2 (1 active, 1 standby)
	Pump Capacity (cubic meter/hour)	75
	Total Pump Capacity, ML/d	1.8
	Pump Motor Size, HP	20
	Ratio of underflow sludge and microsand recycle stream at Nominal Design Flow (%)	3
	Ratio of underflow sludge and microsand recycle stream at Maximum Design Flow Rate (%)	2
	Total Sludge Flow Rate, MLD (2-3% of design capacity)	3.6
	Estimated Sludge Solids, percent	0.2 to 0.3
Coagulation Aid and Sand/Sludge Separation Polymer	Chemical	Anionic Polymer
	Strength (%)	0.2
	Average Dose, mg/L	
	Maximum Dose, mg/L	0.4
Microsand Replenishment System**	Sand Slurry Concentration, g/l	2-4

* Source: Technical Memo 1-Oakville WPP, Process Capacity Review, Associated Engineering, 2010; Drawings at the WPP; Communication with Plant Staff and personal observations during the site visit to the WPP
1. Hydraulic Retention Time (HRT)
** Data from May-July 2014 showed that sand dosages were min= 0.61 g/L; max= 3.24 g/L with an average dose of 1.79 g/L; Flow varied from 31-48 ML/d

As observed from Table 3-3, the hydraulic retention time (HRT) for coagulation-flocculation at the nominal design flow rate of 120 ML/d is 9.6 minutes. At the maximum design flow of 180 ML/d the HRT reduces to 6.4 minutes, a reduction of 33%. At the re-rated plant flow of 143 ML/d (net 130 ML/d), the HRT reduces to 7.1 minutes, a reduction of 26%. Surface loading rate for the clarifiers are 40 meters/hour and 60 meters/hour corresponding to the nominal design flow rate of 120 ML/d and the maximum design flow of 180 ML/d. At the re-rated plant flow of 143 ML/d (net 130 ML/d) the surface loading rate is 46.7 m/h which is within the lower and upper design criteria of the system. Table 3-3 shows that operating at 180 ML/d will decrease the relative ratio of the underflow sludge and microsand recycle stream to 2% (3% @ 120 ML/d). Operating at 143 ML/d will result in this ratio lowering to 2.6%. This could lead to more floc carryover to the filtration process.

As the re-rated plant flow of 143 ML/d would most likely be experienced during the warmer months i.e. high demand period, this may not pose a problem to coagulation processes as coagulants are more effective at warmer temperatures. The planned re-rating study will confirm if the targeted flow range (143 ML/d) can be treated by the Actiflo© units while still meeting the clarified water quality objectives (turbidity < 1 NTU) and the filter performance objectives. The study should also help in determining from an operations perspective if the Actiflo© unit can be operated at 143 ML/d (e.g. efficient addition of chemicals e.g. polymer, microsand).

3.5.1 Site Visit Lessons Learned

- When the raw water turbidity exceeds 100 NTU (high turbidity events), the past procedure was to shut down the plant as it was challenging to operate the Actiflo© units and keep settled water turbidities below 1 NTU, thus affecting performance of the downstream units. Since June 2014, the plant is now operated under high turbidity conditions;
- The existing polymer mixing method is an obsolete technology and is showing signs of aging as witnessed during the site visit. It is estimated that storage capacity of this system varies from 6 to 24 hours. While operating under higher flows this system would impact performance of the Actiflo© system.
- As there is inadequate mixing of the dry polymer with water in the automated batch process, a poorly mixed polymer solution with varying concentrations is fed to the Actiflo© unit. It was observed that the discharge lines from the Hydrocyclone going into the injection basin were plugged due to solids clumping (mixture of sand, polymer and sludge; inadequate mixing of the polymer appears to be a major contributor).
- Operators add water to dissolve the solids in the discharge lines or switch to alternate hydrocyclones to maintain performance;
- Operators conduct sand test regularly (3 times /week) to determine the sand dosage and replenish the injection basin by adding sand manually if desired dosages are not met (typical sand dosages for good performance for Actiflo© systems are 3-6 gram/L)
- Gaps in lamella tubes might be exacerbating the floc carryover problem
- As the recirculation pumps have a fixed capacity, operating at higher flow rates could impact the Actiflo© performance in terms of effluent water quality (increased floc carryover)

In relation to the clarification process, the information provided in Table 2-1 reveals that the Actiflo© system generally delivers the expected removal effectiveness related to turbidity i.e. when raw water turbdity is 1.44 NTU, filtered water turbidity in 0.17 NTU (average values). However, excursions from acceptable removal rates may have occurred when raw water quality conditions deteriorated (when raw water turbidity approached 35 NTU, maximum filtered water turbidity was 1.4 NTU). This is likely related to Actiflo© performance. It is likely that the turbidity removal levels are insufficient when compared to the typical Actiflo© advertised process guarantee. Further assistance from the manufacturer (JMI) in optimizing the Actiflo© units to handle high turbidities may be required.

3.6 Ozone System

Two 900 mm diameter lines convey settled water from the collector channel of the Actiflo© process to two ozone contactors. Ozone is used for primary disinfection and taste and odor control. Table 3-4 is a summary of treatment objectives for the ozonation system.

Table 3-4 Treatment Objectives for the Ozonation System

Parameter	Value	Description
Taste and Odor	No Telephone complaints	Internal Standard set by Region
DBP: Bromate	10 ppb	USEPA D/DBPR
Primary Disinfection		
A) Giardia	3.0 Log by Removal/Inactivation (0.5 Log inactivation by ozone)	MOECC
B) Viruses	4.0 Log Removal/Inactivation (2.0 Log inactivation by ozone)	MOECC
C) Cryptosporidium*	2.0 Log Removal/Inactivation (throughout the year)	MOECC/Internal Standard set by Region
	1.0 Log inactivation (Winter)	

* Minimum of 1-log inactivation has to be achieved by ozone at all times; 2-log by a combination of filtration and inactivation; current goal at the WPP is to achieve 1.5 log inactivation by ozone in the summer months

There are provisions to add hydrogen peroxide in the seventh cell of the ozonation contactors for advanced oxidation to combat extreme taste and odor events. However, according to plant staff, hydrogen peroxide has not been utilized in the last few years. Advanced oxidation refers to the condition where hydrogen peroxide is used with ozone to generate free hydroxyl radicals which have a very high oxidation potential and non-selectively oxidize contaminants they come in contact with (e.g. taste and odor compounds). Calcium thiosulphate is added for quenching ozone residuals in the effluent from the ozone contactors. Two effluent channels (noted as North and

South Channels by plant personnel) convey ozonated water to existing filters. Table 3-5 summarizes the existing system design and operational criteria. A nominal design flow of 120 ML/d and a maximum capacity of 160 ML/d (80 ML/d per train) has been assigned to the ozone system (Technical Memo 1-Oakville WPP, Process Capacity Review, Associated Engineering, 2010). The existing ozone generation system can provide for a maximum dose of 3.0 mg/L at the maximum flow (160 ML/d).

Table 3-5 Existing Design and Operational Criteria for the Ozonation System

Unit Process	Parameter and Units	Value
Hydraulic Basis of Design	Total Nominal Design Flow (ML/d)	120
	Total Max. Design Flow Rate (ML/d)	160
	Number of Independent Ozone Contactors	2
	Nominal Design Flow Rate per Contactor, (ML/d)	60
	Max. Flow Rate per Contactor (ML/d)	80
	Typical Ozone Dose (mg/L)	1.5 to 2.0
Ozone Generation System	Ozone demand (mg/L)	0.5-1.0
	Number of medium-frequency high concentration ozone generators	2 (1 duty, 1 stand by)
	Ozone dosage at 10% weight for a nominal design flow of 120 ML/d, mg/L	4
	Ozone dosage at nominal design flow kg/day	472.0
	Max. Flow Rate (ML/d) accommodated by Generator	160.0
	Maximum Ozone Dose (mg/L) at 160 ML/d	3
	Size of LOX Tank (cubic meter) to meet maximum Ozone dose	62.5
	Number of days supply of LOX (days)	15
	Number of Vaporizers	4 (3 duty, 1 standby)
Ozone Contactor	Number of Total Cells (after ozone dissolution)	10
	Number of Cells Prior to Hydrogen Peroxide Addition	9

	Basin Length (Inner Dimensions), m	17.25
	Basin Width (Inner Dimensions), m	11.915
	Minimum Water Depth, m	4.6
	Total Contactor Volume, ML	1.89
	Total Effective Ozone Contact Volume, ML (T10/T=0.7)	1.324
	Ozone contact time at Nominal Design Flow , min	15.9
	Ozone contact time at Maximum Design Flow , min	11.9
	Number of Ozone Residual Analyzers per Contactor	3
	Locations of the Ozone Residual Analyzers	Cell # 1, 5, 9
	Volume of Hydrogen Peroxide Tank (m^3)	12.0
	Number of Hydrogen Peroxide Metering Pumps	3 (2 duty, 1 standby)
Unit Process	Parameter and Units	Value
Ozone Dissolution	Dissolution Method	Sidestream Injection
	Number of Injection Pumps	3 (2 active, 1 standby)
	Number of degasification Units	2
	Number of Injectors and injection points	2, 4
	Number of stainless steel injector spool pieces	2
	Centrifugal side-stream injection pumps (HP, RPM)	75, 1800
	Ozone dose (mg/L) and pump speed (rpm) at 160 ML/d and 1 degree C	3, 1400
	Ozone dose (mg/L) and pump speed (rpm) at 160 ML/d and 5 degree C	2, 1300
	Number of Pre-Dispersion Mixer Helical Elements	4
Ozone Destruction	Type of Destruct Unit	Thermal Catalytic Units with Off-gas Blowers
	Number of Destruct Units	3 (2 active, 1 standby)

As seen from Table 3-5, the HRT at the maximum design flow of 160 ML/d would be reduced by about 25% when compared to the HRT at the nominal design flow rate of 120 ML/d. At the re-rated plant flow of 143 ML/d (net 130 ML/d) the HRT would be reduced by about 16% when compared to the HRT at the nominal design flow rate of 120 ML/d. At a shorter HRT, a higher ozone dose would be required to meet inactivation of *Cryptosporidium*. To meet the *Cryptosporidium* inactivation objective, under all water quality conditions, the longest contact time will be required and will be the limiting factor with respect to ozone contactor capacity.

The Oakville WPP operates to achieve 1.0 log inactivation of *Cryptosporidium* in winter. According to plant personnel, starting August 2014, the WPP currently is operating at higher doses to meet the goal of 1.5 Log inactivation of *Cryptosporidium* during the summer months.

CT refers to the concentration of disinfectant residual (C) times the duration of disinfectant contact time (T). Preliminary computations have revealed that meeting the CT requirements for 1.5 log *Cryptosporidium* inactivation under the re-rated plant flow of 143 ML/d and warmer temperatures (temperatures > 5°C) is possible by the ozonation system using an ozone dose 2.0 mg/L. The existing ozone equipment can easily meet this need as it has been designed to provide a 3.0 mg/L ozone dose at 160 ML/d (maximum capacity).

Table 3-2 shows that during the winter months the plant treats only 60 ML/d. Due to the lower flow, contact time will increase and preliminary computations show that an ozone dose of ~1.5 mg/L will be required to meet 1.0 Log inactivation of *Cryptosporidium.* If re-rated plant flow conditions (143 ML/d) are experienced during the winter months (worst scenario), then meeting the CT requirements for 1.0 log *Cryptosporidium* inactivation under worst-case conditions for disinfection (i.e., 1°C), would necessitate an increase in ozone dose (~2.5 mg/L). The existing equipment should also be able to meet this need as it has been designed to deliver a maximum dose of 3 mg/L at 160 ML/d.

3.6.1 Site Visit Lessons Learned

- Operations staff have noticed that at high gas/liquid ratios (> 0.1), ozone migrates back to the Actiflo© process indicating there could be some issues with transfer of ozone into the water (mixing, and degassing);
- At higher ozone doses, normal practice is to operate at a higher ozone concentration to lower the gas flows. Plant staff noted that this practice essentially helps in better ozone transfer leading to higher residuals.
- Operating staff have also noted that there is no backup power in case of power failure.

3.6.2 Constrains/Challenges to Meet Desired Production Capacity of 130 MLD (143 MLD gross)

A total capacity of 180 ML/d was assigned to the Actiflo© system and a total capacity of 160 ML/d was assigned to the Ozone system (Technical Memo 1-Oakville WPP, Process Capacity Review, Associated Engineering, 2010). The existing ozone generation system can provide for a maximum dose of 3.0 mg/L at this maximum flow of 160 ML/d. Full-scale testing has been conducted in the past at various flow rates: 44 ML/d, 62 ML/d, and 85 ML/d. During the hydraulic testing, when a flow of 85 ML/d was treated by one Actiflo© train, it was observed that the maturation tank mixer became unsteady and this requires further investigation. No capacity issues were observed with the ozonation system at this flow rate.

Based on the site visit, review of background data, work done by the previous Consultant, and some preliminary calculations, it has been determined that there are no major issues or infrastructure modifications needed that could limit the net production of 130 ML/d (143 MLD gross). Some minor issues that have been identified for both the Actiflo© and ozone systems include:

Actiflo©

Although each Actiflo© unit is sized to treat a peak flow rate of 90 ML/d (a total treatment capacity of 180 ML/d), desired treatment and performance targets will need to be met:

- Avoiding floc carryover to filters
- Settled water turbidity of < 1.0 NTU (prior to filtration)
- Meet filtered water turbidity objectives when settled water turbidity exceeds 1.0 NTU

- Structural integrity of the Actiflo© units (e.g. maturation tank mixers) to handle high flow rates needs to be investigated during full-scale stress testing

- As the recirculation pumps have a fixed capacity, operating at higher flow rates could impact the Actiflo© performance in terms of effluent water quality (increased floc carryover)

OZONE

- It is very likely that a net production of 130 ML/d (143 ML/d gross) will be required coinciding with the higher demand periods (summer months). Preliminary computations have revealed that CT requirements for 1.0 log *Cryptosporidium* inactivation under such conditions (temperatures > 5°C, flow = 143 ML/d), can be met by the ozonation system at lower ozone doses (< 2.0 mg/L). The existing ozone equipment can also sufficiently meet this need.

- If high flow conditions (143 ML/d) are experienced during the winter months, then meeting the CT requirements for 1.0 log *Cryptosporidium* inactivation under worst-case conditions for disinfection (i.e., 1°C), would necessitate an increase in ozone dose (~2.5 mg/L). The existing equipment should also be able to meet this need as it has been designed to deliver an ozone dose of 3.0 mg/L at a flow of 160 Ml/d.

- Although bromate has not exceeded the maximum contaminant level (10 ppb) in the past, higher ozone doses might lead to higher bromate formation which needs to be monitored during performance testing.

3.7 Filtration System

The filters were recently modified as part of the Phase 2 Upgrade Project. The upgrades included replacing the underdrain system with a lower profile underdrain, increasing media depth (due to the low profile underdrain system), filter media replacement, replacement of the backwash pumps with larger pumps on VFDs, replacement of the filter effluent piping, and filter instrumentation/controls improvements. The upgraded filtration process at the Oakville WPP includes eight dual-media gravity filters equipped with a stainless steel lateral underdrain system (AWI). The media meets AWWA B100 standards and consists of 600 mm of anthracite over 250 mm of sand. The filters are 7.632 m x 7.632 m with a depth of 2.575 m and a surface area of 58.25 m^2 per filter. According to the Region's Drinking Water Works Permit – South Halton (June 2014) the proposed filtration rate is 14.3 m/hr with 6 filters in service (one filter offline and one in backwash mode). Further confirmation from the MOECC on the increased filtration rate is required. The MOECC recommends a maximum filtration rate of 11.7 m/h for traditional dual media filter designs, although the MOECC design guidelines mention that filter rates up to 20 m/h have been achieved. The MOECC design guidelines state that filtration rates greater than 11.7 m/h should be confirmed through pilot testing and the pilot testing effort should include cold water conditions. Filtration pilot testing was performed at the Oakville WPP in 2011. The pilot testing showed that the upgraded filters can operate at a filtration rate of 14.3 m/h and provide 100 hours of filter run time. Table 3-6 shows the filtration rate based on different filtration rates and numbers of filters (6, 7, and 8) on-line.

Table 3-6 Filtration Capacity vs. Filtration Rate

Filtration Rate (m/h)	Filtration Capacity - 6 of 8 Filters On-Line (ML/d)	Filtration Capacity - 7 of 8 Filters On-Line (ML/d)	Filtration Capacity - 8 of 8 Filters On-Line (ML/d)
11.7	98	114	131
12.7	107	124	142
13.7	115	134	153
14.7	123	144	164
15.7	132	154	176
16.7	140	163	187

Based on Table 3-6 and the current number of filters, to meet the required 143 ML/d gross taking of water, a filtration rate of 14.7 is required with 7 of 8 filters on-line. With all eight filters on-line, a filtration rate of 12.7 m/h is required to meet the gross production capacity. GHD recommends calculating the filtration capacity with 7 of 8 filters on-line to account for one filter in backwash mode. Pilot testing should be performed to confirm the filter performance (time between backwashing) and filter effluent quality are acceptable at a higher than recommended filtration rate (11.7 m/h) at full-scale.

The Oakville WPP backwash sequence consists of the following:

1. Air scour at 8 L/s/m^2 (0.5 m^3/(min x m^2)) for 2 minutes
2. Low rate wash at 37 ML/d (backwash rate of 26 m/h) for 2 minutes
3. High rate wash at 70 ML/d (backwash rate of 50 m/h) for 3 minutes
4. Low rate wash at 37 ML/d for 2 minutes
5. Settling for 2 minutes
6. FTW for 15 minutes

Backwash water is supplied from the high lift suction flume via two new vertical turbine pumps with VFDs, one with capacity of 70 ML/d at 12.85 m TDH (high wash) and the second with 21 ML/d at 5.23 m TDH (low wash). The pumps were replaced as part of the Phase 2 Upgrade Project. The existing backwash pumps were replaced due to their age and because there was no redundancy (two pumps were required to meet the high rate wash condition).

MOECC design guidelines recommend an air scour flowrate of 0.9 to 1.5 m^3/(min x m^2). The air scour blowers, along with piping, valving, and instrumentation, were replaced during the Phase 1

Upgrade Project. The blowers have a capacity of 650 SCFM, which provides sufficient capacity for air scour, based on MOECC guidelines.

3.7.1 Site Visit Lessons Learned

- Average filter run time is 80 to 100 hrs (four days) – this filter run time was achieved during raw water conditions with turbidity less than 100 NTU
- If filter headloss is greater than 2 m then a filter backwash is commenced
- Filters are taken out of service if filter effluent turbidity gets above 0.3 NTU
- Filter backwash pumps are suitably sized
- Air scour blowers are suitably sized
- Operations would like to look into running the filters in a biological mode

3.8 Clearwell and Reservoir

Filtered water exits each of the individual filters via filter effluent piping and discharges into two clearwells below. Filters 1, 2, 7 and 8 discharge into the east clearwell, while filters 3, 4, 5 and 6 discharge into the west clearwell. The two clearwells provide a total storage volume of 1,660 m^3. The clearwells are not baffled and are not relied upon for contact time for chlorine disinfection. Filtered water from the clearwells is transferred to the two-celled below grade treated water reservoir via 1500 mm diameter transfer piping. Chlorine is injected at the reservoir inlet to provide secondary disinfection. The reservoir has a surface area of approximately 600 m^2 and the depth varies based on flowrate.

In the event that the ozone system is off-line and the reservoir is required to provide chlorine contact time for primary disinfection, Table 3-7 summarizes the minimum reservoir water depth required to meet the CT requirements for 0.5 log *Giardia* inactivation at various flows and water temperatures. The filters receive credit for 2.5 log removal of *Giardia* and 2.0 log removal of viruses. Chlorine therefore needs to provide 0.5 log inactivation of *Giardia* and 2.0 log inactivation of viruses. Chlorine is not able to meet the WPP's internal objective of 1.0 log inactivation of *Cryptosporidium*. The required reservoir levels are based upon a 2.0 mg/L chlorine residual maintained throughout the reservoir and a 0.7 baffling factor. The maximum water depth within the reservoir is approximately 3.2 m so primary disinfection for giardia and viruses, if necessary, can be achieved within the reservoir for all flows with a water temperature 10^{o}C or greater. For 5^{o}C water temperatures or lower, primary disinfection is attainable for flows less than 120 ML/d. The minimum water temperature reported for the Oakville WPP is 3^{o}C (refer to Section 2.1), but the average water temperature is 14^{o}C. At average temperatures the existing reservoir would be able to meet the secondary disinfection requirements at the re-rated plant capacity.

Table 3-7 Minimum Reservoir Depth Required to Provide Primary Disinfection for Giardia Inactivation

Temperature (^oC)	0.5	5	10	15
CT Required for 0.5-log Giardia Inactivation (mg x min/L)	48	33	25	17
Chlorine Residual (mg/L)	2.0	2.0	2.0	2.0
Required Contact Time (min)[(1)]	34.3	23.6	17.9	12.1
Reservoir Flow (ML/d)	Minimum Reservoir Depth Required (m)	Minimum Reservoir Depth Required (m)	Minimum Reservoir Depth Required (m)	Minimum Reservoir Depth Required (m)
100	3.97	2.73	2.07	1.41
110	4.37	3.00	2.27	1.55
120	4.76	3.27	2.48	1.69
130	5.16	3.55	2.69	1.83
140	5.56	3.82	2.89	1.97
150	5.95	4.09	3.10	2.11

[(1)]Assumes a reservoir baffling factor of 0.7 (Superior).

Discussions with WPP operations staff indicated that only one cell (half) of the reservoir is typically in service.

3.9 High Lift Pumps

As part of the Phase 2 Upgrades Project, the high lift pump station (HLPS) was relocated and retrofitted into the north section of the existing waste holding tank on the north side of the plant adjacent to the treated water storage reservoir. The HLPS receives water from the on-site storage reservoir and discharges into the distribution system. The HLPS consists of three 700 kW horizontal centrifugal pumps each with a capacity of 54.5 ML/d at 61.6 m TDH. The firm capacity of the HLPS (with one pump off line) is currently 109 MLD.

4. Oakville WPP Chemical Systems

The following section describes the chemical systems used throughout the Oakville WPP. Chemical dosages are based on information from WPP operations staff and past reports. For each chemical system, the treatable plant flow is presented to highlight any chemical system limitations to re-rating the WPP to 130 ML/d (net) capacity. Bulk chemical storage calculations are based projected average daily flows (90% of gross plant rated capacity 143 ML/d) and average chemical dosages. The projected average daily flow is based on information from the Region during the Project Chartering Meeting on June 19, 2014.

4.1 Chlorination System

Gaseous chlorine is applied at the Oakville WPP at three locations:

- Raw water intake - for zebra mussel control

- Treated water reservoir inlet – secondary disinfection
- High Lift Pump Suction Header - trim (distribution system residual maintenance) chlorination

The major components of the gaseous chlorine system include: ton chlorine gas cylinders, weight scales, chlorinators, and an automatic switchover system. The chlorine storage room has space for twelve ton cylinders, but only six are connected to the feed system at any one time. Severn Trent Services Model Captrol 1450 chlorinators, are used to control the flow rate of chlorine gas and chlorine dosage. Table 4-1 presents information on the WPP's chlorination system. Average and maximum chlorine dosages are based on information from WPP operations staff and past reports.

Table 4-1 Chlorination System

	Injection Location		
Parameter	Raw Water Intake	Treated Water Reservoir Inlet	High Lift Pump Suction Header
Chemical Form (solution strength, %)	100%		
Number of Chlorinators	1	1	1
Max. Chlorinator Capacity (kg/hr)(1)	20	20	10
Avg. Dose (mg/L)(2)	4.5	6.9	0.1
Treatable Plant Flow @ Avg. Dose (ML/d)	108	69	2,925
Max. Dose (mg/L)(2)	9.0	12.6	1.4
Treatable Plant Flow @ Max. Dose (ML/d)	53	38	166
Bulk Storage(3)	6, 1-Ton Gas Cylinders (5,400 kg)		
Days of Storage at 129 ML/d and Avg. Dose	4		

(1)Based on information from Oakville WPP operations staff.
(2)Based on year 2014 Oakville WPP operations data.
(3)Based on South Halton Drinking Water System DWWP (June 2014).

The row titled "Treatable Plant Flow" in Table 4-1 shows the maximum WPP flowrate that can be treated by the chlorination system (at each injection location) at average and maximum dosages. Based on the treatable plant flows in Table 4-1, the Raw Water Intake and Treated Reservoir Inlet chlorine injection points do not meet the re-rated plant capacity requirements at average or maximum dosages. Also, at this time the chlorine feed system does not have redundancy built into its chlorinator system. Based on information from Region operations staff, it should be noted that the Treated Reservoir Inlet chlorine dosages are higher than normal due to carry over of calcium thiosulphate into the finished water from ozone quenching. Upgrades and modifications have been made by the Region as part of the Phase II upgrades that will decrease the calcium thiosulphate dose required for ozone quenching and therefore decrease the Treated Water Reservoir Inlet chlorine dose. The Treated Water Reservoir Inlet chlorine dose will be monitored throughout this year (2015) to determine any changes.

The last row in Table 4-1 presents the number of days of bulk storage provided by the chlorination system at the projected WPP average daily flowrate (129 ML/d) and average chlorine dose at all three injection locations. The number of days of storage provided by twelve ton gas chlorine cylinders is approximately 4 days. Although six, 1-ton chlorine cylinders are connected to the chlorine feed system at one time; there is storage space in the chlorine bulk storage room for twelve 1-ton cylinders. Twelve 1-ton cylinders provide approximately 7 days of storage at the WPP average daily flowrate and average chlorine dose.

4.2 Poly Aluminum Chloride (PACL)

The WPP is in the process of changing coagulants from Alum to PACL. The WPP will soon switch to PACL year round. The PACL system is housed in the old potassium hydroxide storage room, since potassium hydroxide is no longer used at the WPP. PACL is added upstream of the Actiflo© process to assist with the coagulation, flocculation and sedimentation through the pre-treatment process. PACL can currently be injected at three locations: into the in-line injection mixer chamber on the common 1,200 mm diameter raw water feed line, or into each of the individual 750 mm diameter raw water feed lines immediately upstream of each of the Actiflo© process trains. The injection location at the individual 750 mm diameter raw water feeder lines are not used and are scheduled to be removed.

PACL is stored in three PVC-lined wood stave tanks, each with a nominal volume of 20 m^3. PACL from the storage tank flows through a single line to the two PACL metering pumps. Three PACL Prominent Delta metering pumps are available, but two are installed and the third pump is an onsite (shelf) spare. The two pumps are operated in a two duty mode. Each metering pump has a capacity of 49 L/hr. The Region is in the process of installing two additional PACL metering pumps in the feed system, giving a total of four installed pumps and one shelf spare.

Currently, the WPP uses Kemira's PAXL 50 as their PACL. WPP operations staff noted that the coagulant performs better in colder water.

Information pertaining to the WPP's PACL storage and feed system is presented in Table 4-2.

Table 4-2 PACL System

Parameter	Injection Location In-Line Injection Mixer
Chemical Form (solution strength, %)	38%
Number of Metering Pumps(1)(2)	2 duty / 1 standby (onsite spare)
Max. Metering Pump Capacity - per Pump (L/hr)	49
Avg. Dose (mg/L)(3)	2.0
Treatable Plant Flow @ Avg. Dose (ML/d)	269
Max. Dose (mg/L)(3)	7.2
Treatable Plant Flow @ Max. Dose (ML/d)	76
Bulk Storage(2)	3 - 20 m^3 Tanks
Days of Storage at 129 ML/d and Avg. Dose	107

(1)Based on information from Oakville WPP operations staff.
(2)Based on South Halton Drinking Water System DWWP (June 2014).
(3)Based on years 2012-2013 Oakville WPP operations data.

The PACL system can treat up to 269 ML/d and 76 ML/d at the average and maximum dosage, respectively. An additional metering pump is required to meet the maximum PACL dosage at the re-rated WPP flowrate of 143 ML/d. It should be noted that the Region is in the process of installing two additional PACL metering pumps in the feed system, giving a total of four installed pumps and one shelf spare.

The bulk storage system as significant storage capacity and provides 107 days of storage at the average dose (2 mg/L) or 30 days of storage at a dose of 7.2 mg/L.

4.3 Alum

The WPP is in the process of switching to PACL, but still has the ability to inject Alum. The PACL and Alum feed systems are completely independent systems, except they share a common feed point to the in-line mixer. Alum can be added at the same injection points as PACL (discussed in Section 4.2). Alum is stored in two PVC-lined wood stave bulk storage tanks, each with a nominal volume of 30 m^3. Three Wallace and Tiernan Encore 700 diaphragm metering pumps, each with a capacity of 290 L/hr, are provided and operated in a two duty / one standby mode. Average Alum dosage is 15 mg/L, with peak dosages up to 48 mg/L, depending on raw water quality.

Table 4-3 Alum System

Parameter	Injection Location In-Line Injection Mixer
Chemical Form (solution strength, %)	36%
Number of Metering Pumps(1)(2)	2 duty / 1 standby
Max. Metering Pump Capacity - per Pump (L/hr)	290
Avg. Dose (mg/L)(3)	15
Treatable Plant Flow @ Avg. Dose (ML/d)	673
Max. Dose (mg/L)(3)	48
Treatable Plant Flow @ Max. Dose (ML/d)	210
Bulk Storage(2)	2 - 30 m^3 Tanks
Days of Storage at 129 ML/d and Avg. Dose	15

(1)Based on information from Oakville WPP operations staff.
(2)Based on South Halton Drinking Water System DWWP (June 2014).
(3)Based on years 2013-2014 Oakville WPP operations data.

The Alum system can treat up to 673 ML/d and 210 ML/d at the average and maximum dosages, respectively. The bulk storage system provides 15 days of storage at the average dose (15 mg/L).

4.4 Dry Polymer (Type 1)

As discussed in Section 3.5 (Actiflo© System), an anionic (Type 1) polymer can be injected into the Actiflo© system at three locations: microsand injection (hydrocyclone), injection tank, and maturation tank. Based on current operations, the polymer feed is split between the three injection locations according to: 50% to microsand injection (hydrocyclone), 25% to injection tank, and 25% to maturation tank.

Dry polymer (anionic) is fed to a hopper and is mixed with cold water in a tray using an automated batching process. The batched polymer is fed to two 1.8 m^3 aging tanks where one tank (after proper aging) feeds aged polymer to three Wallace and Tiernan Chem Tube 2000 polymer metering pumps operating in a two duty / one standby mode to supply the mixed polymer to the Actiflo© units. The other 1.8 m^3 tank is used to age the batched polymer and is put in service when the other tank reaches a low level setpoint.

Table 4-4 shows information on the WPP's dry polymer (Type 1) system.

Table 4-4 Dry Polymer (Type 1) System

Parameter	Injection Location Actiflo© System
Chemical Form (solution strength, %)	0.2%
Number of Metering Pumps(1)(2)	2 duty / 1 standby
Max. Metering Pump Capacity - per Pump (L/hr)	810
Avg. Dose (mg/L)(3)	0.14
Treatable Plant Flow @ Avg. Dose (ML/d)	537
Max. Dose (mg/L)(3)	0.55
Treatable Plant Flow @ Max. Dose (ML/d)	142
Bulk Storage(2)	2 - 1.8 m^3 Aging Tanks w/hopper system
Hours of Storage at 129 ML/d and Avg. Dose	4

(1)Based on information from Oakville WPP operations staff.
(2)Based on South Halton Drinking Water System DWWP (June 2014).
(3)Based on years 2012-2013 Oakville WPP operations data.

The existing Type 1 polymer feed system can treat a WPP flow up to 142 ML/d at the maximum 0.55 mg/L dosage. Approximately 4 hours of storage is provided by one 1.8 m^3 aging tank.

The Type 1 polymer is a critical chemical in the WPP process and if the feed system is out of service then the entire WPP must be shut down. WPP operations staff noted that the hopper system has a tendency to clog.

4.5 Calcium Thiosulphate

Calcium thiosulphate is used for dechlorination of chlorinated (for zebra mussel control) raw water upstream of Actiflo©, dechlorination of filter backwash for operating the filters in biological mode, and dechlorination of chlorinated process water prior to discharge into Lake Ontario. Calcium thiosulphate is also used for quenching ozone at the exit of each ozone contractor (prior to filtration) and at ozone residual analyzer drain lines. The five injection locations are listed below:

- low lift effluent (raw water wetwell),
- last cell (10^{th}) cell of each ozone contactor (two injection locations),
- ozone analyzer drain lines to waste tank
- individual 500 mm diameter backwash pump discharge lines, and/or
- Lake Ontario storm sewer manhole.

Based on information from WPP operations staff, the last two injection locations listed above (backwash pump discharge and Lake Ontario storm sewer manhole) are no longer in service. Two 15 m^3 PVC-lined wood stave tanks provide storage for calcium thiosulphate. Six Wallace and Tiernan Encore 700 pumps (CTP1, CTP2, CTP3, CTP4, CTP5 and CTP6), each having a capacity of 45.4 L/hr are used to feed calcium thiosulphate to the four injection locations. The metering pumps are operated in a four duty / two standby mode, as shown in Table 4-5. A summary of the calcium thiosulphate feed system is presented in Table 4-5.

Table 4-5 Calcium Thiosulphate System

Parameter	Injection Location	
	Raw Water Wetwell	Ozone Contactor Outlet
Chemical Form (solution strength, %)	24%	
Number of Metering Pumps[(1)(2)]	1 (CTP1; CTP5 standby)	CTP2 (Ozone Contactor No. 1) duty and CTP3 (Ozone Contactor No. 2) duty; CTP 6 standby
Max. Metering Pump Capacity - per Pump (L/hr)	45	45
Avg. Dose (mg/L)[(3)]	0.7	1.7
Treatable Plant Flow @ Avg. Dose (ML/d)	483	384
Max. Dose (mg/L)[(3)]	1.1	4.1
Treatable Plant Flow @ Max. Dose (ML/d)	304	162
Bulk Storage[(2)]	2- 15 m^3 Tanks	
Days of Storage at 129 ML/d and Avg. Dose	14	

(1)Based on information from Oakville WPP operations staff.
(2)Based on South Halton Drinking Water System DWWP (June 2014).
(3)Based on years 2013-2014 Oakville WPP operations data.

Based on the information in Table 4-5, the existing calcium thiosulphate feed system can accommodate the re-rated plant capacity for raw water dechlorination and ozone quenching (ozone contactor outlet) at the average and maximum calcium thiosulphate dosages. Approximately 14 days of bulk storage capacity is provided by the existing calcium thiosulphate feed system.

There is one additional chemical metering pump (CTP-4) that is used to quench ozone from the dissolved ozone analyzer drain lines. The Prominent Gamma/L metering pump has a capacity of 2.1 L/hr. and is located along the back wall of the calcium thiosulphate storage room. The feed rate for this pump is manually paced and is so low that it has minimal impact on the bulk storage capacity. The feed rate is not expected to change for the re-rated flowrate so the existing Prominent pump has sufficient capacity to accommodate quenching ozone in the analyzer drain lines when the WPP is re-rated to 143 ML/d.

4.6 Hydrogen Peroxide

The WPP has the ability to inject hydrogen peroxide to the ozonated water stream for advanced oxidation during extreme T&O events. Hydrogen peroxide is injected within Cell No. 7 (of 10) of each ozone contactor. Hydrogen peroxide is stored in an aluminum storage tank with a nominal volume of 12 m^3. Three Wallace and Tiernan Encore 700 diaphragm metering pumps are provided and operated in a two duty / one standby mode. Each pump has a capacity of 45.4 L/hr. Table 4-6 shows information pertaining to the Oakville WPP's hydrogen peroxide system.

The Oakville WPP hydrogen peroxide system has not been used so operations data is not available for the average and maximum dosages. Assuming a 0.5:1.0 hydrogen peroxide to ozone dose ratio (Rakness, 2005) and a maximum ozone residual of 3.5 mg/L (ozone dose of 4.0 mg/L and ozone demand of 0.5 mg/L), the maximum hydrogen peroxide dose would be 1.75 mg/L. At a more typical ozone dose and residual of 3 mg/L and 2.5 mg/L, a hydrogen peroxide dose of 1.25 mg/L would be required.

Table 4-6 Hydrogen Peroxide System

Parameter	Injection Location Cell No. 7 of Ozone Contactors
Chemical Form (solution strength, %)	30%
Number of Metering Pumps(1)(2)	2 duty / 1 standby
Max. Metering Pump Capacity - per Pump (L/hr)	45
Avg. Dose (mg/L)(3)	1.25
Treatable Plant Flow @ Avg. Dose (ML/d)	625
Max. Dose (mg/L)(4)	1.75
Treatable Plant Flow @ Max. Dose (ML/d)	436
Bulk Storage(2)	1 - 12 m^3 Tank
Days of Storage at 129 ML/d and Avg. Dose	25

(1)Based on information from Oakville WPP operations staff.
(2)Based on South Halton Drinking Water System DWWP (June 2014).
(3)Anticipated dosage based on 0.5:1.0 hydrogen peroxide to ozone dose ratio and ozone residual of 2.5 mg/L.
(4)Anticipated dosage based on 0.5:1.0 hydrogen peroxide to ozone dose ratio and ozone residual of 3.5 mg/L.

At both average and maximum dosages, the existing hydrogen peroxide feed pumps provide sufficient capacity to accommodate the re-rated plant flow of 143 ML/d. The available bulk storage capacity is also sufficient.

Operations staff indicated that the hydrogen peroxide system and advanced oxidation is not typically used because any carryover to the treated water reservoir reacts with chlorine added during disinfection and exerts a chlorine demand.

4.7 Filter Aid (Liquid) Polymer (Type 2)

A liquid cationic polymer (ClearTech CL2410) is used as a filter aid. The filter aid polymer is used intermittently during high turbidity events. Two USFilter PolyBlend units activate the polymer by mixing it with water and deliver it to the injection location. Liquid polymer is added to the ozonated water conduit, upstream of filtration. Liquid polymer is delivered and stored in two 200 L drums located on a containment skid. Table 4-7 shows information pertaining to the Oakville WPP's filter aid polymer feed system.

The polymer feed system does not have dose control or dose feedback available so actual operating dosages are not available. A typical average dose of 0.25 mg/L and maximum dose of 1.0 mg/L have been assumed based on the type of polymer and application.

Table 4-7 Filter Aid Polymer (Type 2) System

Parameter	Injection Location Ozonated Water Conduit
Chemical Form (solution strength, %)	100.0%
Number of Metering Pumps(1)(2)	1 duty / 1 standby
Max. Metering Pump Capacity - per Pump (L/hr)	3.8
Avg. Dose (mg/L)(3)	0.25
Treatable Plant Flow @ Avg. Dose (ML/d)	376
Max. Dose (mg/L)(3)	1.0
Treatable Plant Flow @ Max. Dose (ML/d)	94
Bulk Storage(2)	2 – 200 L Drums
Days of Storage at 129 ML/d and Avg. Dose	13

(1)Based on information from Oakville WPP operations staff.
(2)Based on South Halton Drinking Water System DWWP (June 2014).
(3)Assumed dosage based on type of polymer and application.

The existing PolyBlend units provide sufficient capacity to accommodate the re-rated plant flow of 143 ML/d at average dosages. At higher dosages (greater than 0.6 mg/L), one PolyBlend unit cannot meet the feed requirements. Two units could be put in operation at higher dosages to meet the feed requirements. The bulk storage capacity provides approximately 13 days of storage (assuming 2, 200 L drums are available) at the proposed average flow of 129 ML/d.

The polymer blending system was previously used for feeding polymer aid in thickening the process residuals waste. Operations staff has concerns whether the existing polymer is the proper polymer for a filter aid application.

4.8 Hydrofluosilicic Acid

Hydrofluosilicic acid is used at a low dose for fluoridation of the treated water prior to distribution. Hydrofluosilicic acid is injected at the treated water reservoir inlet. Hydrofluosilicic acid is stored in one PVC-lined wood stave tank with a nominal volume of 19.8 m^3. Two Prominent Delta fluoride metering pumps, each with a capacity of 20.2 L/h, provide fluoride to the injection point. The fluoride metering pumps are provided and operated in a one duty / one standby mode. Table 4-8 provides the details of the hydrofluosilicic acid feed system.

Table 4-8 Hydrofluosilicic Acid System

Parameter	Injection Location Treated Water Reservoir Inlet
Chemical Form (solution strength, %)	24%
Number of Metering Pumps(1)(2)	1 duty / 1 standby
Max. Metering Pump Capacity - per Pump (L/hr)	20.2
Avg. Dose (mg/L)(3)	0.6
Treatable Plant Flow @ Avg. Dose (ML/d)	245
Max. Dose (mg/L)(3)	0.7
Treatable Plant Flow @ Max. Dose (ML/d)	192
Bulk Storage(2)	1 - 19.8 m^3 Tank
Days of Storage at 129 ML/d and Avg. Dose	77

(1)Based on information from Oakville WPP operations staff.
(2)Based on South Halton Drinking Water System DWWP (June 2014).
(3)Based on years 2013-2014 Oakville WPP operations data.

Based on the information in Table 4-8, the existing hydrofluosilicic acid system provides sufficient feed pump and storage capacity for the re-rated (143 ML/d) WPP.

4.9 Potassium Hydroxide

The potassium hydroxide system is no longer used and the wood stave storage tanks are being modified to store PACL. The previous potassium hydroxide feed room has been converted into the PACL storage and feed room.

4.10 Chemical System Summary

Refer to Table 4-9 for a summary of the results of the capacity evaluation of the chemical systems at the Oakville WPP. All chemical systems, except the chlorine system (Raw water intake and Treated water reservoir inlet injection locations), have sufficient feed capacities to meet the re-rated gross capacity of the Oakville WPP of 143 ML/d at average chemical dosages. As discussed in Section 4.1, the Treated Reservoir Inlet chlorine dosages are higher than normal due to carry over of calcium thiosulphate into the finished water from ozone quenching. Upgrades and modifications have been made by the Region as part of the Phase II upgrades that will decrease the calcium thiosulphate dose required for ozone quenching and therefore decrease the Treated Water Reservoir Inlet chlorine dose. For the majority of the chemicals (except for Dry Polymer Type 1, calcium thiosulphate, hydrogen peroxide, and hydrofluosilicic acid), the WPP will not have sufficient capacity to feed chemicals at the maximum dosages and 143 ML/d.

All chemical systems, except the chlorine system, provide sufficient storage capacity for a gross flowrate of 143 ML/d. The storage capacities shown in Table 4-9 were calculated based on the projected average flow (129 ML/d) and average chemical dosages.

As discussed in Section 4.4, the Type 1 polymer is a critical chemical in the WPP process and if the feed system is out of service then the entire WPP must be shut down.

Table 4-9 Oakville WPP Chemical System Summary

Chemical	Injection Location(s)	Treatable Plant Flow @ Avg. Dose (ML/d)	Treatable Plant Flow @ Max. Dose (ML/d)	Days of Storage at 129 ML/d and Avg. Dose
Chlorine	1) Raw water intake - zebra mussel control	108	53	4
	2) Treated water reservoir inlet - secondary disinfection	69	38	
	3) Treated water reservoir discharge - finished water trim	2925	166	
PACL	1) In-line injection flash mixer chamber	269	76	107
Alum(1)	1) In-line injection flash mixer chamber	673	210	15
Dry Polymer - Type 1	1) Actiflo© system	537	142	4 hrs.
Calcium Thiosulphate	1) Raw water wetwell	483	304	14
	2) Ozone contactor outlet	384	162	
Hydrogen Peroxide	1) Cell No. 7 of ozone contactors 1 and 2 (2 locations)	625	436	25
Filter Aid Polymer (Type 2)	1) Upstream of filters	376	94	13
Hydrofluosilicic Acid	1) Treated Water Reservoir Inlet	245	192	77

(1)WPP is in the process of changing to PACL year round

5. Oakville WPP Hydraulics

5.1 Existing System

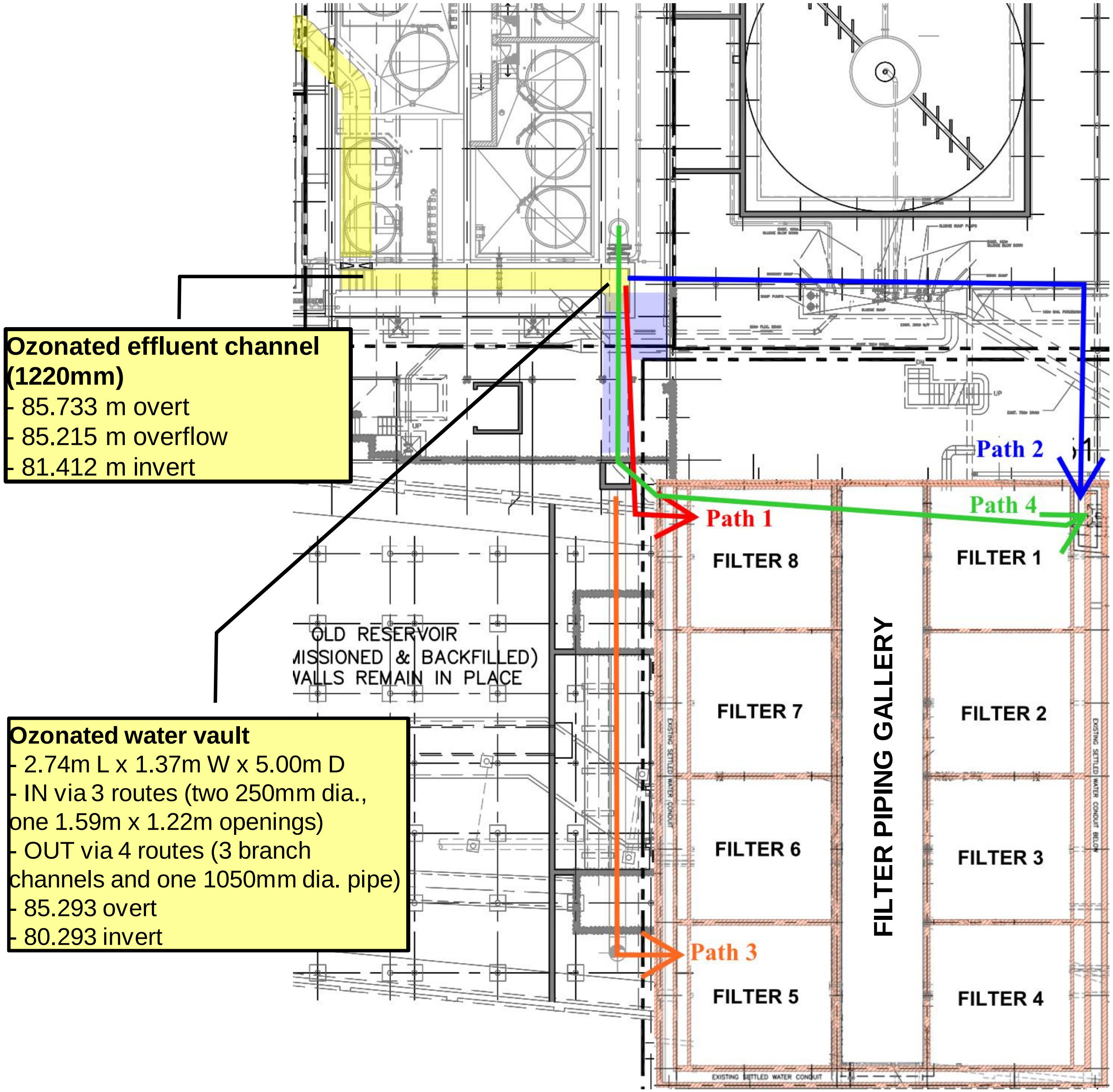
During normal operation, raw water is drawn in from Lake Ontario through the 2130 /1828 mm dia. Intake No. 2 into the wet well of the Low Lift Pumping Station. Four Low Lift Pumps pump raw water through 900 mm dia. yard piping to a 1200 mm dia. concrete pipe, where the flow is then split through two 750 mm dia. pipes (which are connected via a bypass line and valve). From here, raw water enters the two parallel Actiflo© trains at the south end, and flows sequentially through the coagulation, injection, maturation and settling tanks of the Actiflo© system. Water discharges from the settling tanks through six weir troughs into the Actiflo© settled water conduits (one per train), having an invert elevation of 83.427 m and an overflow elevation of 86.468 m.

From each Actiflo© settled water conduit, water flows south through its respective 900 mm diameter stainless steel pipe to the north end of the two ozone contact chambers, each measuring 17.25 m long by 11.92 m wide and fitted with nine baffles in an east-west orientation, which provides a serpentine flow pattern to give the required ozone contact time. The discharge weir elevation at the south end of each contact tank is 85.215 m and is 4.2 m in height (thus the minimal operating depth), thus making the invert elevation of the contact chambers at the south end 81.015 m. According to existing documentation (Oakville WPP Process Capacity Review, Associated Engineering 2010), the maximum operating depth of the ozone contact chambers is 85.700 m (depth of 4.685 m), providing 300 mm of head space.

From the ozone contact chambers, ozonated water is discharged through two 1500 mm diameter stainless steel pipes (each fitted with butterfly valves) which combine to a common 1500 mm diameter pipe. From this pipe, ozonated water discharges into a 1220 mm wide ozonated effluent channel (invert, overt and overflow elevations of 81.412 m, 85.733 m, and 85.215 m) which runs south, west-adjacent to the chemical storage rooms.

The following bulleted information regarding flow from the ozonated effluent channel to the filter inlet ring/channel is drawn from Technical Memoranda 1 – Oakville WPP Process Capacity Review (Associated Engineering, 2010). Figure 5-1 shows the assumed layout of the filter inlets and conduits connecting the filters to the ozone contactors.

- The ozonated effluent channel discharges into a 2.74 m long by 1.37 m wide by 5.00 m deep ozonated water vault (invert and overt elevations 80.923 m and 85.293 m, respectively) through three penetrations (two 250 mm diameter openings and one 1.59 m by 1.22 m opening). From the ozonated water vault, three separate branch channels and one 900 mm diameter pipe feed filters F1 through F8 via four different routes. One route enters the filter inlet channel at Filter F8, one (900 mm diameter pipe) enters near Filters F5/F6, and two routes enter the filter inlet channel at Filter F1.
- The filter influent channels for all eight filters are hydraulically connected with an additional 610 mm wide conduit at the west end of the filters linking the north and south filter inlet conduits. As these filters are constant rate – constant level filters, the water level in all filters is maintained and operated at an elevation of approximately 85.030 m. Flow from each filter is controlled by throttling the flow control valve on each filter effluent pipe.
- Filter inlet route 1 (Filter F8 channel): The channel feeding the north bank of filters near Filter F8 exits the water vault and begins as a 1219 mm wide channel (invert and overt elevation of 84.283 m and 85.143 m, respectively). This channel reduces to a 590 mm wide filter influent channel adjacent to the bank of Filters F5 to F8 (overt elevation of 85.263 m at Filter 8 with a slight slope towards Filter 5.)
- Filter inlet route 2 (Filter F1 channel): The channel feeding the south bank of filters near Filter F1 exits the water vault and begins as a 1219 mm wide channel (invert and overt elevation of 84.278 m and 85.143 m, respectively). This channel reduces to a 610 mm wide filter inlet channel adjacent to Filters F1 to F4 (invert and overt elevation of 81.268 m and 85.198 m, respectively). Filter inlet route 3 (Filters F5/F6 pipe): A 900 mm diameter pipe (centerline elevation of 82.100 m) conveys water from the water vault (assumed) and discharges into the filter influent channel from the 900 mm dia. pipe (centerline elevation of 84.590 m) near Filters F5 and F6. This route was purportedly added to relieve hydraulic constraints.
- Filter inlet route 4 (Filter F1 pipe): A 900 mm diameter pipe (centerline elevation of 78.777 m) conveys water from the water vault and discharges into a water chamber below the filter



Path 1 (near F8)
From the vault, a 1219 mm channel (84.283 m invert, 85.143 m overt) to a 590 mm channel (__?__ invert, 85.263 m overt)

Path 2 (near F1)
From the vault, a 1219 mm channel (84.278 m invert, 85.143 m overt) to a 610 mm channel (81.268 m invert, 85.198 m overt)

Path 3 (near F5/6)
From the vault, a 900 mm conduit (82.100 m centreline) conveys to the filter influent channel near Filters F5/6 (pipe exit at 84.590 m centreline).

Path 4 (near F1)
From the vault, a 900 mm conduit (78.777 m centreline) conveys to an ozonated water chamber directly beneath the filter influent channel (78.370 invert), and enters the filter influent channel (81.268 invert) through a break in the filter influent channel floor immediately upstream of Filter F1.

The 900 mm pipe was originally a raw water supply line, and was converted to a Filter Influent Conduit, along with the construction of the "water chamber" and break in the channel floor, during the Phase 1 Upgrades.

Figure has been produced from Oakville WPP Phase 1 As-Built Drawings; Dwg P1051, Phase 1 Upgrades, Oct. 2008

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Region of Halton – Oakville WPP Re-Rating

Figure 5-1
Ozone Contactor to Filter Flow Paths and Filter Inlet Locations

influent channel (chamber invert elevation 78.370 m). From here, water flows up through a 1.2 m by 0.6 m break in the filter influent channel floor to the filter influent channel adjacent to Filter F1. Based on information from the Oakville WPP operations staff, this route was added to relieve hydraulic constraints during the Phase 1 upgrades completed in 2007. During the Phase 1 upgrades the 900 mm diameter pipe was converted from a raw water pipe to a filter inlet pipe (route 4).

From the filter influent channel, water flows into one of the eight filter gullet areas via 750 mm by 750 mm slide gate valves. The gullet area has an invert and overflow elevations of 82.605 m and 85.153 m, respectively. The invert elevation of the gullet wall, over which water flows into the filter beds, is 84.963 m. The dual media filter beds are comprised of 600 mm of anthracite, 250 mm of sand. Filter effluent flows out through the underdrain, flume feed box, 500 mm dia. flume, and exits the flume into a 500 mm diameter effluent pipe. This pipe then tees, with a 300 mm diameter branch out to the remainder of the filter effluent piping to the clearwell below, and a 500 mm diameter line out to the backwash water supply.

Filter effluent from F1, F2, F7, and F8 is discharged into Clearwell 1, and filter effluent from F3, F4, F5, F6 is discharged into Clearwell 2. The clearwells are hydraulically connected through a central clearwell area (which contain two backwash pumps) via 900 mm by 900 mm slide/sluice gate valves (normally in the open position). Each of the filter effluent pipes discharges into a weir box, each with reported maximum and minimum elevations of 79.944 m and 78.030 m, respectively; the operating level is set by adjustable weirs (aluminum stop logs) located on one side of each weir box, and are reportedly raised to an elevation of 78.830 m. The clearwells have an invert elevation of 76.821 m (varies) and an overflow elevation of 81.262 m.

From each clearwell, filtered water enters an associated transfer chamber on the north side via 1200 mm by 1200 mm sluice gates. Flow from the Clearwell 1 transfer chamber flows through a 1200 mm transfer pipe to the Clearwell 2 transfer chamber. The combined flow (now full plant flow) is then directed north a 1,500 mm diameter transfer pipe to the treated water reservoir. The flow then splits through a 1500 x 1500 x 1500 mm tee to divert water into Cells 1 and 2 of the Reservoir. Each reservoir cell consists of two channels, with invert elevations of 77.037 m and 77.357 m at the south and north ends, respectively (1% grade), and a maximum operating elevation of 80.649 m. Note that there is no overflow in the Reservoir. Flow then exits from Reservoir Cells 1 and 2 (reservoir effluent, potable water) via respective 1,500 mm dia. piping, which then combines to a 1,500 mm pipe. Water then travels through yard piping and a 1,500 mm valve chamber (with a tee, line out to the Clearwell backwash area) and flows to the High Lift Pumping Station.

The maximum water level in the clearwell is set by an overflow at an elevation of 80.649 m. Although the maximum water elevation in the reservoir can reach 80.649 m during static condition with a freeboard of approximately 880 mm, the actual water level will be a function of the flow rate from the clearwell, which depends on the filtration rate and the high lift pumping rate. In the event that the water level rises in the reservoir and clearwell, a flap valve in the existing clearwell allows overflow to be wasted through a 600 mm storm sewer discharge line back to Lake Ontario.

5.2 Hydraulic Modeling

To determine the maximum hydraulic capacity of the Plant, hydraulic modeling is being undertaken and will be presented in a future technical memorandum. The objectives of the hydraulic model are as follows.

1. To determine hydraulic capacity of the Plant

2. To identify the hydraulic restrictions within the Plant
3. To identify the areas of highest loss of hydraulic head

These objectives will be achieved by modeling the hydraulic grade line (HGL) based on known inverts and calculated head loss. The hydraulic analysis will be broken down into the following segments.

1. From the High Lift Pumping Station influent piping back to the Clearwell.
2. From the Clearwell back to the Filters.
3. From the Filter Influent Gullet weir back to the effluent of the Ozone Contact Chambers (note: this area includes the Ozonated Effluent Channel, eg. Settled Water Conduit).
4. From the Ozone Contact Chambers back to the Actiflo© Settling Tank effluent weir.
5. From the Actiflo© Settling Tank effluent back to the Actiflo© influent piping.
6. From the Actiflo© influent piping back to the Raw Water Pumps.
7. From the Raw Water Pumps back to the Intake.

Flow through the Plant is governed by the following physical elevation constraints.

1. Actiflo© Settling Tank process weir at elevation 85.83 m (approximate).
2. Actiflo© effluent (settled water) conduit overflow weir at elevation 86.468 m.
3. Ozone Contact Chambers: (a) The broad-crested weir at the south (downstream) end of each Ozone Contact Chamber, measuring 4.2 m in height. Assuming a floor elevation of 80.790 m, the invert elevation of these weirs is 84.990 m. (b). The maximum operating level of each Ozone Contact Chambers is 85.700 m.
4. Ozonated effluent channel overflow weir at elevation of 85.215 m.
5. Filter feed routes:
    - Route 1: Channel feeding the north bank of filters near Filter 8 is reported to have upstream invert and overt elevations of 84.283 m and 85.143 m (where channel width is 1219 mm), respectively, and a downstream overt elevation of 85.263 m (where channel width is reduced to 590 mm).
    - Route 2: Channel feeding the south bank of filters near Filter 1 is reported to have upstream invert and overt elevations of 84.278 m and 85.143 m (where channel width is 1219 mm), respectively, and downstream invert and overt elevations of 84.278 m and 85.198 m (where channel width is reduced to 610 mm), respectively.
    - Route 3: The 900 mm dia. pipe conveying water from the water vault (assumed) to the filter influent channel near Filters F5 and F6 has a centerline elevation of 82.100 m out of the water vault, and a centerline elevation of 84.590 m at its discharge location.
    - Route 4: The 900 mm diameter pipe conveying water from the water vault to the filter influent channel near Filter F1 has a centerline elevation of 78.777 m and discharges into a water chamber below the filter influent channel (invert elevation 78.370 m) where water then flows up through a 1.2 m by 0.6 m break in the filter influent channel floor.
    - Filter gullet wall, top of gullet wall elevation of 84.963 m.
    - Filter overflow elevation of 85.153 m.

6. Clearwell:
   - Filter effluent discharge to weir box in Clearwells, minimum and maximum weir elevation of 78.030 m and 79.944 m, respectively. Typical elevation is 78.830 m (according to 1980 MacLaren Drawings).
   - Overflow elevation of 81.262 m.
7. Reservoir maximum operating water level at elevation 80.649 m.

5.3 Hydraulic capacity – Preliminary observations

GHD is completing the development of the hydraulic model for the Oakville WPP. At this stage of model development several conditions in the facility have been observed as potentially having hydraulic limitations that may impact ultimate capacity. The most notable conditions are identified below.

Filter Inlet. The existing filters are provided influent flow through the “filter influent ring”. At each filter the water enters through a slide gate, rises over the inlet gullet wall and fills the filter area. Figure 5-2 shows a cross section of Filter No. 2 from Drawing P1004 (Contract W-2062(B)-12, Issued for Construction 31/05/2012) and shows the Top of Gullet Wall elevation, from Drawing P1006 of the same contract.

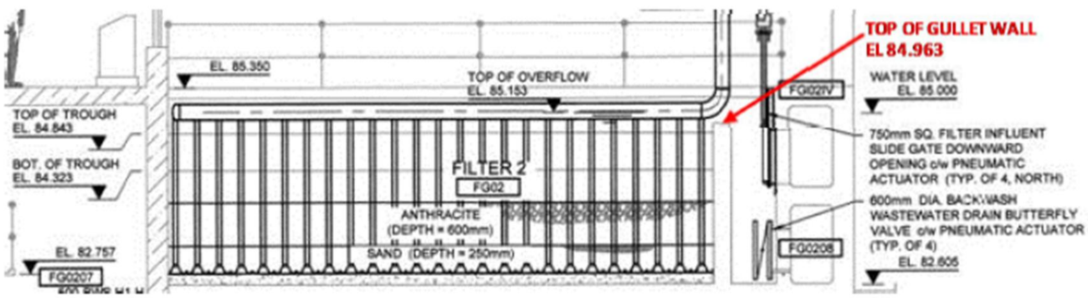

Figure 5-2 Cross section of Filter 2

Table 5-1 summarizes select elevations apparent in Figure 5-2. Based on the noted elevations, the depth of water over the gullet wall during normal operation is only 0.037 m. This is hydraulically inconsistent with the minimum depth that would be required due to the weir effect[1] of the wall alone. If the water level noted in the influent channel is accurate, this may be the root cause of select filters “being starved” during high plant flow operations. In the event that the water surface in the influent channel drops slightly, the influent feed capacity of a filter can be severely reduced. This condition would be mitigated by the raw water pumps and filter rate of flow controllers attempting to maintain their water level control set points. As such, when this condition begins to occur, adjustments within the control loops in the plant resolve it. Hence why multiple filters do not ultimately experience this condition.

The maximum freeboard (depth of flow) over the gullet wall into the filters is restricted to a maximum of 0.190 m. To maintain flow into the filters within this range will require precise upstream flow control and hence limited operational flexibility. Typically, to allow for more operational

[1] Water flowing over the top of a wall will experience a depth of flow over the wall due to the effect of the wall acting as a sharp crested weir. Until the up or downstream depth of flow greatly exceeds the height of the wall, the minimum water surface elevation over the wall must be at minimum equivalent to the head over said weir.

flexibility, the noted maximum freeboard over the gullet wall is typically over 1 m. The filter inlet configuration will be further reviewed during GHD's site visit.

Table 5-1 Notable Filter Elevations

Item	Elevation (m)	Depth from Top of Gullet Wall (m)
Top of Floor Above	85.35	0.387
Top of Overflow	85.153	0.19
Water Level (per referenced drawing)	85	0.037
Top of Gullet Wall	84.963	-

Filter Outlet. Preliminary results from the hydraulic model indicate significant headloss from the filter effluent piping into the Clearwell below and thus potentially pose a hydraulic limitation. A combination of the headloss through the filter effluent piping and fittings, combined with the headloss necessary for proper rate-of-flow control operation (valve throttling) and the headloss of a clean filter media bed results in a hydraulic grade (theoretical water surface elevation) above the top of media elevation. This reduces the driving head available for filter operation and for the development of headloss as the filter media becomes dirty. During high clearwell levels and peak flow conditions the available driving head in the filters may be reduced to less than 2 meters. This available driving head will considerably shorten filter run lengths. These results will be discussed further in the hydraulic assessment technical memorandum.

Ozone Effluent. Water from the ozone contact chambers purportedly flows into an ozonated water vault, from which flow to the filter influent channel is fed through a combination of four separate channels and pipes of varying diameter, width, depth and slope. WPP Operations have indicated that at flows above 120 ML/d Filter 1 becomes starved. It is suspected that, at high flows, the interconnectedness of the four filter feeds may result in preferential flow paths which drain or short circuit the flow to Filter 1, sufficiently to exploit the condition identified in the Filter Inlet section above. A combination of preferential flow paths and the effect of dynamic losses within the filter inlet ring causes inconsistent water surface elevation in the inlet channel to the filters. Normally this would not result in a significant imbalance of flow to individual filters. However, due to the limited hydraulic freeboard discussed in the Filter Inlet Section above, it is anticipated that this restriction cannot be overcome with adjustments to control logic alone as the tolerances are too tight. These results will be discussed further in the hydraulic assessment report.

5.4 Lessons Learned from Site Visit

5.4.1 Flow Control Scheme

Flows to the WPP processes are controlled according to:

1. Low Lift Pumps - controlled off of filter inlet channel level. Two level transmitters (north and south section of filter channel) operate in a one duty / one standby mode. The filter inlet level controls the low lift pump VFDs. Operations tries to maintain a level in the filter inlet channel of 60%-70% full.

2. Filters - controlled by a master filter rate within SCADA. This master filter rate equal to the flow setpoint for the high lift pumps. However, the HI clearwell level of 93% full will trip the filters to shut their effluent valves to prevent the clearwell from overfilling. The high lift pump flow setting can be overridden by operations to maintain a flow setpoint through SCADA. Flow is intended to be equally divided between all on-line filters.
3. High Lift Pumps - controlled based on a flow setpoint that is input by operations and varies on a daily basis.

The Oakville WPP consists of two parallel Actiflo© and ozone process trains and one train is on line for flows up to 80 ML/d; above 80 ML/d, two trains are on line with an equal flow split. Hydraulic limiting factors include:

- Flows above 120 ML/d begin to starve Filter No. 1 of feed water and the filter eventually shuts down. This may be due to a bottleneck within the ozonated water vault, but further investigation is required to confirm the cause.
- Hydraulic constraints will be confirmed/identified as part of the hydraulic assessment.

6. Miscellaneous

Operations staff has concerns that grit and sand within the waste sludge from filter backwashes is deteriorating the 250 mm diameter carrier piping. Operations staff noted that the velocities through the piping are high (up to 70 L/sec). The high velocities and abrasive material within the waste sludge may be compromising the integrity of the pipe. Operations staff noted one instance in which a connection on the 250 mm piping failed and had to be replaced. Operators also noted that they previously observed anthracite carryover into the waste holding tank. Modifications to the filters as part of the Phase 2 Upgrade Project may correct the anthracite carryover.

7. Summary

The following section presents a summary of the lessons learned from investigations into each WPP process as detailed in Sections 1 through 6.

7.1 Oakville WPP Process Systems

INTAKE

- Lake water turbidity is influenced by storm water runoff from Sixteen Mile Creek post storm water events.
- Hydraulic capacity of the intake is reduced as a result of the smaller 1828 mm diameter section.
- The internal diameter of the intake is suspected to be further reduced by the potential buildup of zebra mussels. Further investigation into the condition of the intake is required.

- Lake Ontario water quality in the approximate area of the intake could potentially be impacted once the Mid-Halton Wastewater Treatment Plant comes on line and the new outfall for the plant is fully in service.
- Replace the plugged 50 mm diameter turbidity sample line that runs to the intake to provide operators an early warning for turbidity events.
- Continue to review plant performance under high turbidity conditions, after June 2014, as this study progresses.
- Relocate 50 mm chlorine feed line (for zebra mussel control) outside of the 2130 mm intake pipe to allow use of the butterfly isolation valve.
- Consideration should be given to the use of chlorine dioxide for zebra mussel control to reduce the formation of disinfection byproducts in the WPP feed water.

TRAVELING SCREENS

- Make provisions for chlorinated flushing system for help control algae buildup. This would require dechlorination prior to disposal to Lake Ontario.
- Screen flushing water should be metered to account for this water.

Actiflo©

- The existing polymer mixing method is an obsolete technology and is showing signs of aging as witnessed during the site visit. It is estimated that storage capacity of this system varies from 6 hours-24 hours. While operating under higher flows this system would impact performance of the Actiflo© system.
- As there is no good mixing of the polymer, a poorly mixed polymer solution is fed to the Actiflo© unit. The discharge lines from the Hydrocyclone going into the injection basin can become plugged due to solids clumping (mixture of sand, polymer and sludge; inadequate mixing of the polymer appears to be a major contributor).
- Operators add water to dissolve the solids in the discharge lines or switch to alternate hydrocyclones to keep up with the performance
- Gaps in lamella tubes might be exacerbating the floc carryover problem
- As the recirculation pumps have a fixed capacity, operating at higher flow rates could impact the Actiflo© performance in terms of effluent water quality (increased floc carryover)
- Integrity of maturation tank mixer at high flows (85 ML/d or greater) requires further investigation.
- Although each Actiflo© unit is sized to treat a peak flow rate of 90 ML/d (a total treatment capacity of 180 ML/d), desired treatment and performance targets will need to be met:
  - Avoiding floc carryover to filters
  - Settled water turbidity of < 1.0 NTU (prior to filtration)
  - Meet filtered water turbidity objectives when settled water turbidity exceeds 1.0 NTU
- Structural integrity of the Actiflo© units (e.g. maturation tank mixers) to handle high flow rates needs to be investigated during full-scale testing

Based on the site visit, review of background data, work done by the previous Consultant and some preliminary calculations, it has been determined there are no major issues or infrastructure modifications needed that could limit the net production of 130 ML/d.

Recommended improvements include:

- Modifying the sand and polymer injection process,
  - o Switch from dry to liquid polymer to ensure consistency in the feed
  - o Convert to a silo system for adding microsand with volumetric screw feeder
- Replacing the sludge/sand recycle pumps,
- Replacement of instrumentation, and
- Eliminating the gaps in the tube settlers.

OZONE

- Plant staff noted that the South channel does not carry enough ozonated water to the filters as compared to the North Channel; (uneven flow splitting)
- Operations staff have noticed that at high gas/liquid ratios (> 0.1), ozone migrates back to the Actiflo© process indicating there could be some issues with transfer of ozone into the water (mixing, and degassing)
- At higher ozone doses, normal practice is operate at a higher ozone concentration to lower the gas flows. Plant staff noted that this practice essentially helps in better ozone transfer leading to higher residuals.
- There is no backup power in case of power failure
- It is very likely that a net production of 130 ML/d (143 ML/d gross) will be required coinciding with the higher demand periods (summer months). Preliminary computations have revealed that meeting the CT requirements for 1.0 log Cryptosporidium inactivation under such conditions (temperatures > 5°C, flow of 143 ML/d) can be accommodated by the ozonation system at lower ozone doses (< 2.0 mg/L). The existing ozone equipment can sufficiently meet this need.
- If high flow conditions (143 ML/d) are experienced during the winter months, then meeting the CT requirements for 1.0 log Cryptosporidium inactivation under worst-case conditions for disinfection (i.e., 1°C), would necessitate an increase in ozone dose (~2.5 mg/L). The existing equipment should also be able to meet this need as it has been designed to deliver an ozone dose of 3 mg/L for a flow of 160 ML/d.
- Although bromate has not exceeded the MCL (10 ppb) in the past, higher ozone doses might lead to higher bromate formation which needs to be monitored during performance testing.

It is recommended that optimization/performance testing be combined with the planned hydraulic and rerating study to determine capacity limitations over more sustained operation at the desired flow rate (130 ML/d net, 143 ML/d gross) both from a water quantity and quality perspective, while meeting the MOECC regulatory requirements and the internal water quality objectives as desired by the Region.

FILTRATION

- MOECC accepted filtration rate needs to be confirmed
- Average filter run time is 80 to 100 hrs (four days)

- If filter headloss is greater than 2 m then a filter backwash is commenced
- Filters are taken out of service if filter effluent turbidity gets above 0.3 NTU
- Filter backwash pumps are suitably sized
- Air scour blowers are suitably sized
- Operations would like to look into running the filters in a biological mode

7.2 Oakville WPP Chemical Systems

- Additional chlorinators or upsizing two chlorinators (Raw water intake and Treated water reservoir inlet) is recommended to provide additional capacity and redundancy for the WPP's chlorine feed system.
- All chemical systems, except the chlorine system, provide sufficient storage capacity for a gross flowrate of 143 ML/d at average dosages.
- Additional chemical metering pumps are required to meet the 143 ML/d gross flowrate at maximum chemical dosages.
- An additional PACL metering pump should be installed to provide capacity for increased PACL dosages (greater than 7.2 mg/L). The Region is in the process of installing two additional PACL metering pumps.
- As discussed in Section 4.4, the Type 1 polymer is a critical chemical in the WPP process and if the feed system is out of service then the entire WPP must be shut down. Installation of an upgraded feed system or the addition of another aging tank would provide additional redundancy within the Type 1 polymer feed system.
- Confirmation on whether the existing filter aid polymer is the proper polymer for the application, is required.

7.3 Oakville WPP Hydraulics

- Filtration system requires further investigation to determine hydraulic limitations. Preliminarily, it has been observed that flows above 120 ML/d begin to starve Filter No. 1 of feed water and the filter eventually shuts down. This may be due to a bottleneck within the ozonated water vault, but further investigation is required to confirm the cause.
- Hydraulic constraints will be confirmed/identified after calibration of the hydraulics model and full-scale testing.

7.4 Miscellaneous

- Review velocities through the waste sludge line to determine whether the flows are compromising the integrity of the pipe.
- Monitor anthracite carryover into the waste holding tank to determine whether the modifications to the filters as part of the Phase 2 Upgrade Project correct the anthracite carryover.

8. References

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