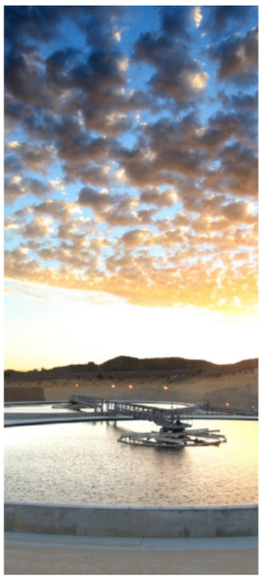
Appendix B: Full Scale 130 ML/d Test Report

(this page intentionally left blank)





Class EA Study to Rerate the Oakville WPP to 130 ML/d 130 ML/d Test Report

Region ref PR-2989A

Regional Municipality of Halton

GHD | 651 Colby Drive Waterloo Ontario N2V 1C2

8811884 | Report No 1 | September 12, 2016

Executive summary

This report documents the full scale hydraulic and limited process assessment test of the Oakville WPP that was conducted on May 5, 2015 and identifies hydraulic and performance limitations and provides recommendations to improve plant performance and support re-rating to the targeted net production rate of 130 ML/d. The results of the test are discussed in light of the background studies previously completed by GHD.

Section 1 provides an introduction to this report and background information on the project. Section 2 summarizes the results of the technical baseline review completed by GHD, including process systems, chemical systems, and plant hydraulics.

Section 3 presents the observations and data from the full scale test. During the initial ramp up of the full scale test, operators had to adjust process logic setpoints which resulted in the test time being restarted. A filter backwash was conducted approximately 2.5 hours into the test, which resulted in a drop in the clearwell and reservoir storage levels. In light of these challenges, a three hour period with average net production flows of 130 ML/d was achieved. Raw water flows during the three hour period were an average of 132 ML/d.

The following is a summary of the key findings from the 130 ML/d test.

1. No issues were discovered during the full scale test with the intake, travelling water screens, or low lift pumps during the performance test. Prior to the ramp up period, raw water turbidity levels were less than 1 NTU. Intake scouring during the ramp up period peaked raw water turbidity levels up to 30 NTU; average raw water turbidity during the three hour test period was 12 NTU.
2. The full scale test revealed that Actfilo® units performed satisfactorily and were able to achieve > 90% reduction in turbidity and total organic carbon.
3. Although the ozonation system exceeded the inactivation goals for Giardia (0.5-log inactivation), viruses (2.0 log inactivation), it did not meet satisfactorily the inactivation goal for Cryptosporidium (1-log inactivation). Average log inactivation values for Cryptosporidium ranged between 0.8-0.9 log. Furthermore, contactor 1 performed better than contactor 2 with respect to achieving required CT. However, the ozone system is capable of achieving the required performance ratio. Possible reasons for the performance differences may therefore include: the size difference between Contactor 1 and 2 (the latter being slightly larger volume); and that the ozone system was not run in AUTO mode and was not adjusted sufficiently during the test period to deliver the appropriate doses and meet CT requirements for each contactor. However, based on review of the ozone system it is capable of achieving the required performance ratio for 1.0 Log inactivation of *Cryptosporidium* in both cold and warm waters. CT calculations revealed that in warm waters (> 5 C°) up to 2.0 Log inactivation of *Cryptosporidium* is possible with the existing ozone system.
4. Headspace in the ozone contact chambers was surcharged when the low lift pumps increased flows during the initial ramp up.
5. The ozonated water conduits did not present any major hydraulic issues during the full scale test. Due to the fact that the tops of the conduits are unsealed, they must be operated at a water level below 100% to avoid risk of flooding parts of the facility. Since the level in the

ozonated water conduits controls the low lift pumps, it is therefore noted that these conduits pose a potential hydraulic bottleneck. Levels in the channel are particularly sensitive to water flow rates and will surge during raw water flow ramp ups and during quick changes to the combined filter effluent rate.

6. Filter 1 experiences significant hydraulic limitations under high flow conditions (generally above 100 ML/d). During the full scale test backwash cycle, Filter 1 level was only stabilized when its filter effluent rate was decreased to 90 L/s, while the remaining six filters operated at approximately 225 L/s. Field observations suggest that this is due to Filter 1's position along the filter inlet channel and its configuration of the inlet port relative to the trough structures within.

7. The filtration system, at 143 ML/d gross, will filter more particulate at an increased rate. If clearwell levels are also high, the available driving head will be reduced. The net effect will shorten filter run times between backwash cycles. This was not experienced during the full scale test but derived from desktop analysis and requires further investigation to determine filter performance (i.e. run time) under these conditions.

8. During the full scale test filter backwash, the clearwell levels dropped from 89% to 80%, while the reservoir levels dropped from approximately 79% to 50%, while the high lift pumps continued to pump 130 ML/d. Region operations have indicated that during a backwash under lower flow conditions (50 to 80 ML/d), storage levels typically drop from about 90-82% (clearwell) and 89-80% (reservoir), and that the levels are usually replenished in about 1.5 hours. The record drawings indicate that the floor elevation of the Reservoir is approximately 0.21 m higher than the upstream Clearwell. However, the hydraulic profile created from the SCADA data shows the downstream reservoir water surface elevation being higher than that of the clearwell with forward flow to the reservoir during pre-test flows, which is not possible. This therefore suggests an inaccuracy either in the elevations reported in the drawings or in the level monitoring system (e.g. monitoring range, instrument mounted level, etc.). It is recommended that the Region verify the entirety of the clearwell and reservoir monitoring systems

9. At high flows, the Region operates the Reservoir's two dual-channelled cells in series and not in parallel. This is done to increase the flow path through the serpentine configuration. In this configuration, this longer flow path may be contributing to hydraulic restrictions experienced during a filter backwash.

10. The chemical metering pumps operational during the test, with minor exceptions, performed satisfactorily and within their capacities (Chlorinators 1-3; Calcium thiosulphate pumps 1-3; Fluoride pump 2). The estimated average flow rates of Calcium thiosulphate pumps 2 and 3 were 43 L/hr and 35 L/hr, respectively, with estimated peak values of approximately 50 L/hr and 41 L/hr; the capacity of each of these pumps is 45.4 L/hr. Chemical systems not operational during the test included filter aid polymer (Type 2); Alum; and Hydrogen peroxide.

11. When all three HLPs were operating at 130 ML/d, participants felt vibrations while standing on the catwalks in the HLP building. Further investigation of this issue is recommended.

12. Bromate formation was below the maximum concentration level (MCL) of 10 ppb. Low bromide concentration in the raw water translated into lower bromate formation. Other disinfection byproducts (e.g. THMs and HAAs) in the finished water were well below

maximum concentration limits, many even below minimum detectable limits, and did not pose any problems.

13. During the full scale test the Davis Road Zone O1 bypass valve was throttled to 49% in order to reduce distribution system pressures in order to achieve the target production rate of 130 ML/d out of the high lift pumping station.

14. Net production volumes over a 24-hour period were estimated using data from the full scale test for three scenarios of filter operation. The estimates used the filters as they have been determined to be the rate limiting process. The scenarios included two Do Nothing scenarios (one minimizing the filter flow rate variable to achieve 130 ML/d net, the other maximizing filter flow rate variable to their rated maximum of 14.3 m/h) and one Improvement scenario (address Filter 1 hydraulic restrictions). The results indicate that 130 ML/d net production may be achieved without addressing Filter 1’s restrictions, and also that addressing Filter 1’s hydraulic restrictions would reduce the process burden placed on the remaining filters as well as increase confidence of operators in the filtration system when operating at the targeted production rate of 130 ML/d. It is further noted that limited headloss through the filter coupled with a 20% increase in gross flowrate through the filter (120 to 144 ML/d) will potentially reduce filter runtimes between backwashes.

Recommendations have been grouped as follows: Operations and protocol; Monitoring; Capital works; Additional investigations possibly requiring capital works; and Commissioning. After their implementation and in agreement with MOECC, perform a final full-scale site acceptance test to commission the plant at the newly rated capacity. The test should be longer than 3 hours, include a full filter backwash and post-backwash time to allow clearwell and storage levels to stabilize. Based on our review as documented in this report, the Oakville WPP has sufficient existing capacity to produce a net volume of 130 ML of water over a 24-hour period. Aside from installing the fourth high lift pump, the capital works recommendations are intended to increase operational flexibility and reliability with the identified bottlenecks in the ozone-to-filter conveyance and the filtration system itself.

This report is subject to, and must be read in conjunction with, the limitations set out in Section 1.3 Scope and limitations and the assumptions and qualifications contained throughout the Report.

Table of contents

Table index

Table index

Figure index

Appendices

No table of contents entries found.

1. Introduction

1.1 Purpose

The purpose of this report is to document the full scale test of hydraulic and process performance of the Oakville WPP that was conducted on May 5, 2015 and identify recommendations to improve plant performance and support re-rating to the targeted net production rate of 130 ML/d.

This report summarizes the desktop reviews of the plant's systems in Section 2, documents the results of the full scale test in Section 3, discusses the overall findings in Section 4, and closes with key findings and a list of recommendations in Section 5.

1.2 Background

The Oakville Water Purification Plant (WPP) is one of three surface water treatment plants within the Regional Municipality of Halton's (Region's) drinking water supply system. The WPP is currently rated for a net production capacity of 109 megaliters per day (ML/d). 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. was retained by the Region to provide engineering services for a Class EA study for the Oakville WPP including the overall re-rating of the plant to 130 ML/d (net).

The plant in its present form was constructed in the 1960s and has undergone numerous upgrades and expansions since that time. The most recent upgrades/expansion occurred from 2007 to 2014 under a two phased expansion/upgrade program. Phase 1 and 2 upgrades were completed in 2008 and late 2014, respectively.

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 up to 10 percent losses in the pre-treatment, 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 to account for losses including a filter backwash.

A Municipal Class Environmental Assessment (Class EA) is being undertaken to investigate alternative solutions to the problem definition to increase net production capacity to 130 ML/d while maintaining performance objectives, with one of the challenges being able to treat high raw water high turbidity that occasionally occurs. The alternative solutions will compare plant-based solutions (e.g. process optimizations, expansion or upgrades) with intake-based solutions (e.g. extension of the existing intake to deeper water or construction of a new intake) and coordinate approvals with the necessary regulatory agencies, including the Ontario Ministry of Environment and Climate Change (MOECC).

1.3 Scope and limitations

This report has been prepared by GHD for Regional Municipality of Halton and may only be used and relied on by Regional Municipality of Halton for the purpose agreed between GHD and the Regional Municipality of Halton as set out in Section 1 of this report.

GHD otherwise disclaims responsibility to any person other than Regional Municipality of Halton arising in connection with this report and as part of the Class Environmental Assessment Process that this report is intended.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described throughout this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by Regional Municipality of Halton and others who provided information to GHD, which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

2. Background and desktop studies

2.1 Process systems

This section provides an overview of the Oakville WPP's process systems. This information was first reported in detail in the Technical Baseline Review *(Memo #3, December 2014), completed as part of this Class EA study, which documented the findings from desktop studies and information gained from GHD's site visits and interviews with WPP operations staff on July 8 and 9, 2014. See Memorandum #3 for more detailed information on that which is reported in this section.*

An overview of the firm and rated capacities[1] of the WPP's process systems is given in Table 2-1 and Figure 2-1 below. Refer to Appendix A for this facility's Process Flow Diagram.

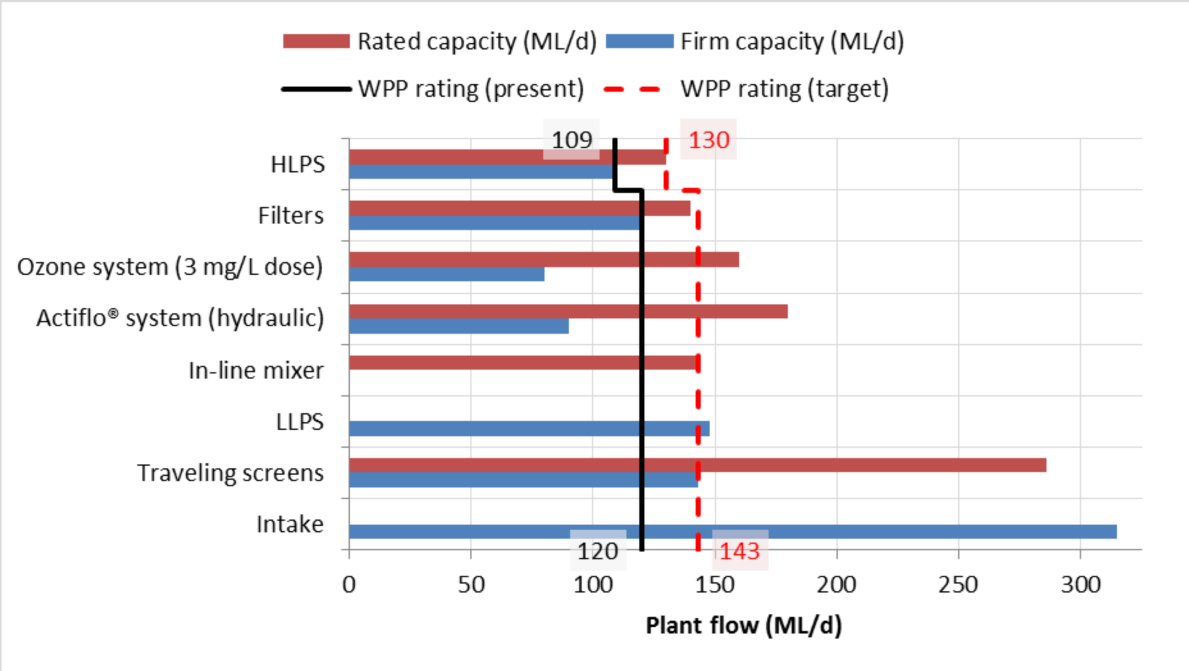
Table 2-1 Process system capacities – overview

Unit process	Firm capacity (ML/d)	Rated capacity (ML/d)	Ref.
Intake	315	(n/a)	(1)
Traveling screens	143	286	(2, 3)
LLPS	148	*	(7)
In-line mixer	(n/a)	143	(4, 5)
Actiflo® system - Treatment - Hydraulics	60 90	120 180[a]	(1)
Ozone system - 4 mg/L dose - 3 mg/L dose	60 80	120 160[a]	(1)
Filters – Hydraulic capacity	120 – six filters 140 – seven filters	140 - seven filters 160 – eight filters	(4, 6)
Reservoir[b]	-	-	**
HLPS[c]	109	130	(7)

(1) Tech. Memo 1: Oakville WPP Process Capacity Review, Associated Engineering, 2010.
(2) Manufacturer email communication; see discussion in relevant section below.
(3) Drinking Water Works Permit, South Halton, June 2014.
(4) MOECC Drinking Water Works Design Guidelines, 2008.
(5) Kawamura, 2000.
(6) Pilot testing results and calculations; see discussion in relevant section below.
(7) Equipment rating.
*System curve not available. To be verified.
**Table 2-4 provides minimum reservoir depths required for primary disinfection for *Giardia* inactivation using chlorine, however ozone is used as the primary disinfectant therefore is not applicable to this system.
a. Maximum hydraulic capacity of the indicated system.
b. Reservoir capacity is based on the concentration time (CT) calculation. See Section 2.1.9 for details.
c. Under the present pump arrangement of two duty/one standby. Planned installation of fourth HLP will increase both rated and firm capacities.

[1] The term "firm capacity" refers to a system's production capacity with the largest unit out of service, while the term "rated capacity" refers to a system's capacity with all units in service, as per MOE (2008).

Figure 2-1 Process system capacities



Note: Refer to Table 2-1 for the table of values, references, and notes.

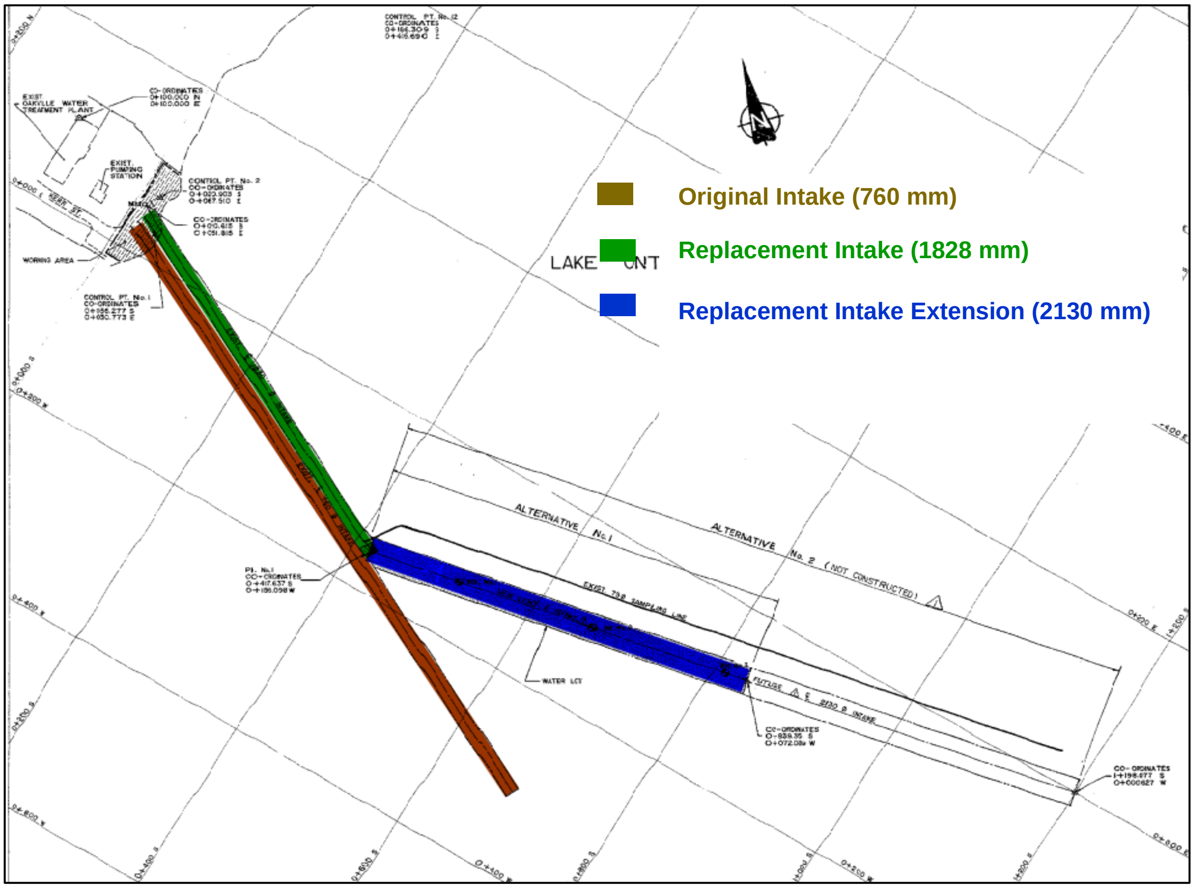
2.1.1 Intake

The current arrangement of the raw water intake piping for the Oakville WPP provides for two intake pipes with only Intake no. 2 being in service. The rated capacity of Intake no. 2 is noted as being 315 ML/d[2]. A short description of Intake nos. 1 and 2 is provided below. Refer to Figure 2-2 below.

- Intake no. 1 (shown in brown): Constructed around 1947, 760 mm diameter, extends approximately 725 meters +/- from shore, provides for 4.7 m of submergence at a low water lake level of 73.76 m, and is presently out of service.
- Intake no. 2: Constructed around 1977, extends an approximate total length of 858 meters +/- from the low lift pump station, and was constructed in two stages. Stage 1 (shown in green) 1828 mm diameter pipe extends 458 m +/- in length. The stage 2 extension (shown in blue) 2130 mm diameter pipe extends 400 m +/- to the intake crib location, at which point the average water depth is approximately 9.7 m assuming a low lake level elevation of 73.76 m (64.0 Lake bottom elevation). Intake no. 2 provides for a 30.0 meter wide water lot, and provides for future extension to Intake no. 2 for an additional length of 383 meters beyond the current intake crib location, providing for a water depth of approximately 13.5 m assuming the same low lake level elevation of 73.76 m.

[2] Associated Engineering, 2010, Technical Memo 1: Oakville WPP Process Capacity Review.

Figure 2-2 Plan View of Existing Intakes



The Region recently completed[3] a CCTV inspection of Intake no. 2 which reported no concerns with the buildup of zebra mussels on the interior surfaces and/or at the intake crib.

Summary of constraints and challenges (Technical Baseline Review Memo #3, GHD, Dec. 2014):

- The 50 mm dia. raw water sampling line at the that extends from the intake crib to the low lift pump station, which is used for raw water sampling and turbidity monitoring at the intake's crib, is reportedly plugged.
- The present location of the intake crib is located in an area subject to occasional high turbidity issues during stormwater runoff conditions, the solution for which is being explored under the present Class EA study.

2.1.2 Travelling water screens

Two automatic traveling water screens are used to prevent debris and fish from entering the low lift wetwell and damaging downstream equipment. The traveling screens are approximately 1.5 m wide with a stainless steel screen mesh size of 9.5 mm. The screens can be operated in a duty-standby mode or both in parallel, and are normally operated in parallel.

The June 2014 Drinking Water Works Permit (DWWP) states that the combined capacity of the intake screens is 137.5 ML/d. However, the screen manufacturer (now Evoqua Water Technologies

[3] Inspection completed on October 28, 2014 and report submitted on December 10, 2014 by ASI Group.

LLC) has confirmed that for each screen, when operating at flows of 143 ML/d (37.8 MGD), the velocity and differential are both within the design parameters[4].

Summary of challenges (Technical Baseline Review Memo #3, GHD, Dec. 2014):

- Oakville WPP operations staff previously noted that they sometimes have issues with algae build-up on the screens. Intermittent flushing with chlorinated water may help alleviate algae build-up, but would require dechlorination prior to disposal to Lake Ontario. Also it is possible that chlorine flushing would only kill the algae and not solve the build-up problem. Issue is recommended for further consideration.
- Combined with the manufacturer's recommendations noted above (that screens be as clean as possible under 143 ML/d flows), algae could present a challenge if algae were to buildup when one screen is offline and lake levels are at worst case (i.e. low level). Since Oakville WPP is rarely operated with one screen out of service, this is not considered to be a normal operating issue.

2.1.3 Low Lift Pumping Station (LLPS)

The low lift pump station consists of a raw water wetwell with four vertical turbine pumps (LLP1 through LLP4). Variable frequency drives (VFDs) are installed on LLP1, LLP2 and LLP3, with a soft start on LLP4. The low lift pumps operate in a three duty/one standby mode.

The following table summarizes the tested capacity of the low lift pumps. Note that the tested pumping capacities and efficiencies are lower than that indicated by the manufacturer at original installation. The test reports also recommend that pumps LLP1, LLP2, LLP3 be operated at less than 100% VFD speed so as to avoid overloading the motor (for LLP1) or to operate at or near peak efficiency (for LLP2 and LLP3).

Table 2-2 Low lift pumps, tested capacities of individual pumps

Pump	Tested best efficiency point	Typical operating point	Ref.
LLP1	701 L/s (60.6 ML/d) at 14.5 m TDH, 64.6% efficiency	Same as tested	(1)*
LLP2	449 L/s (38.8 ML/d) at 23.6 m TDH, 81.7% efficiency	636 L/s (55.0 ML/d) at 13.5 m TDH, 69.5% efficiency	(2)*
LLP3	446 L/s (38.5 ML/d) at 23.5 m TDH, 81.4% efficiency	624 L/s (53,9 ML/d) at 13.4 m TDH, 68.5% efficiency	(3)*
LLP4	594 L/s (51.3 ML/d) at 19.7 m TDH, 71.4% efficiency	743 L/s (64.2 ML/d) at 14.5 m TDH, 68.0% efficiency	(4)*

TDH = total dynamic head (m).
(1) Sept 22, 2015 LLP1 Performance Test, Report by HydraTek & Associates.
(2) Sept 22, 2015 LLP2 Performance Test, Report by HydraTek & Associates.
(3) Sept 22, 2015 LLP3 Performance Test, Report by HydraTek & Associates.
(4) July 31, 2014 LLP4 Performance Test, Report by HydraTek & Associates.
*See reports for full test curves, manufacturer curves, notes on test conditions, and full observations and recommendations.

[4] Email communication from representative Matthew Tyson on March 4, 2015. Previously, GHD's Technical Baseline Memo #3 (Dec. 2014) noted that at flows of 143 ML/d the maximum velocity was greater than the design parameters. However subsequent communication clarified to the contrary, and was also noted in GHD's Performance Test Plan (April 2015). Mr. Tyson further noted: "At the low water level of 8'-0" (2.45 m) (worst case scenario), the velocity would be 2.67 feet/sec (0.81 m/s) and the differential would be about 1.87 inches (47 mm), which are both within the design parameters for this type of screen. This is based on a 100% clean screen; and it was noted that at higher flow rates, debris will accumulate more quickly so the screen condition should be monitored."

At present the Region does not have a system curve for the low lift pumping station.

There are no challenges or constraints to note for meeting the target re-rated production capacity for the low lift pumping station.

2.1.4 In-line mixer (PACL injection)

The in-line vertical turbine mixer is installed within the 1200 mm diameter raw water pipeline and imparts mixing energy to a PACL coagulant.

The only constraint to note for meeting the target re-rated production capacity for the in-line mixer and PACL injection is that there is only one mixer and therefore no redundancy built into this system.

2.1.5 Actiflo® system

The Actiflo® Process is a high rate flocculation-sedimentation process , that introduces microsand/proprietary silica with an effective size of 85 microns to the water during the flocculation process in order to enhance both coagulation and settling. 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 feed system, a microsand injection system and a microsand recycling system.

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. Each unit is sized to treat a peak flow rate of 90 ML/d for a total peak treatment capacity of 180 ML/d[5]. Note that while the treatment capacity is less than the hydraulic capacity and less than the targeted re-rating of 143 ML/d gross, the Actiflo® units are operable above the treatment capacities permitting performance targets are met. The Actiflo® units became operational in 2006 and since then individual units have operated at 80 ML/d each without performance issues.

Summary of challenges and constraints (Technical Baseline Review Memo #3, GHD, Dec. 2014):

- In the past, full-scale testing was conducted at various flow rates with one Actiflo®, ozone, and filter train in service: 44 ML/d, 62 ML/d, and 85 ML/d. At a flow of 85 ML/d, it was observed that the Actiflo® maturation tank mixer became unsteady and vibrated. This requires further investigation. (No issues were noted during the May 5, 2015 full scale test; see discussion in Sections 3 and 4.)
- 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)
- Refer to Sections 2.2.2 Poly aluminium chloride (PACL) and 2.2.4 Dry polymer (Type 1) for related notes on the associated chemical systems.
- Gaps between lamella tube sections may exacerbate floc carryover

2.1.6 Ozone system

Ozone is used for primary disinfection and taste and odor control. Two 900 mm diameter lines convey clarified water from the collector channel of the Actiflo® process to two ozone contactors. A

[5] Associated Engineering, 2010, Technical Memo 1: Oakville WPP Process Capacity Review.

nominal design flow of 120 ML/d (based on 4 mg/L ozone dose) and a maximum hydraulic capacity of 160 ML/d (based on 3 mg/L ozone dose) has been assigned to the ozone system[6].

Summary of concerns (Technical Baseline Review Memo #3, GHD, Dec. 2014):

- 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 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 can meet this need as it has been designed to deliver an ozone dose of 3.0 mg/L at 160 Ml/d.

2.1.7 Ozonated water conduits (ozone-to-filter conveyance)

Two effluent channels (noted as North and South Channels by plant personnel) convey ozonated water to the existing filters. Each channel has a level transmitter, the signals from which control the flowrate of the low lift pumps. Present operational practice is to adjust LLPS flow based on the water level within the channels. Currently, maximum flow is based on 60% water level within the channels.

Summary of challenges and constraints (recent discussions with WPP operations):

- The top of the ozonated water conduits are not fully sealed. If the channels operate at 100% for a period greater than approximately 15 minutes, certain areas of the WPP facility ground level will flood (one of which is an office). Since these channels control the low lift pump flow rate at a desired maximum channel level of 60%, they present a hydraulic bottleneck.

2.1.8 Filtration system

The filtration process includes eight dual-media gravity filters equipped with a stainless steel lateral underdrain system. 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.

Backwash water is supplied from the Clearwell (central cell, high lift suction flume) via two new vertical turbine pumps with VFDs (duty-standby configuration), each with capacity of 680 L/s (58.5 ML/d at 11.4 m TDH. The air scouring blowers have a capacity of 650 SCFM, which provides sufficient capacity for air scour, based on MOECC guidelines. Therefore the filter backwash pumps and air scouring system were both noted as suitably sized.[7]

The Oakville WPP backwash sequence consists of the following. Note that there are additional periods of rest between each step; a normal backwash procedure can take between 60-90 minutes.

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 291 L/s (25 ML/d) for 2 minutes

[6] Associated Engineering, 2010, Technical Memo 1: Oakville WPP Process Capacity Review.
[7] GHD, Dec. 2014, Technical Baseline Memo #3.

3. High rate wash at 631 L/s (54 ML/d) for 3 minutes

4. Low rate wash at 291 L/s (25 ML/d) for 2 minutes

5. Settling for 2 minutes

6. Filter-to-waste (FTW) for 15 minutes

Operational parameters learned during GHD's site visit include the following (Technical Baseline Review Memo #3, Dec. 2014):

- Average filter run time is 80 to 100 hours (three to 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;[8]
- The filter backwash criteria is 0.1 NTU. The shutdown criteria is 0.15 NTU.

The following table shows the filtration capacities based on different filtration rates and numbers of filters (6, 7, and 8) on-line.

Table 2-3 Filtration rate and filtration capacity

Filtration rate, m/h (L/s*)	Runtime**	Filtration capacity, ML/d		
		6 of 8 filters on-line	7 of 8 filters on-line	8 of 8 filters on-line
11.7 (189)	122 hrs	98	114	131
12.8 (207)	111 hrs	107	125	143
13.7 (222)	104 hrs	115	134	153
14.3 (231)***	100 hrs	120 ML/d	140	160
14.6 (236)	98 hrs	123	143	163
15.7 (254)	91 hrs	132	154	176
16.7 (270)	86 hrs	140	163	187
17.1 (277)	84 hrs	143	167	191

This chart modified from GHD's Technical Baseline Review Memo #3, Dec. 2014. Darker shaded rows indicate filtration rates above the maximum filtration rate of 14.3 m/h.
*Volumetric flow rate (L/s) is for an individual filter based on the filter surface area of 58.25 m^2. The Oakville WPP's SCADA system measures filtration rate in L/s.
**Pro-rated based on 2011 pilot testing demonstrating a maximum filtration rate of 14.3 m/h (231 L/s) for up to 100 hours.
***Maximum filtration rate as per 2011 pilot tests.

Summary of challenges and constraints (Technical Baseline Review Memo #3, GHD, Dec. 2014):

- Maximum filtration rate: The MOECC guidelines indicate 11.7 m/h recommended as max unless confirmed through performance testing, which was done as part of the pilot test completed in 201. Filtration pilot testing was performed at the Oakville WPP in 2011 and demonstrated that the upgraded filters can operate at a filtration rate of 14.3 m/h (231 L/s/filter) and provide 100 hours of filter run time.[9] The Region's Drinking Water Works Permit

[8] The filter pilot study report indicates filter backwashing at 1.5 m of headloss (Oakville WPP Pilot Study, October 2011, Associated Engineering).

[9] The filter pilot test was conducted during Fall 2010 and Spring 2011, for approximately 1 month each. During the fall trial, rates of 11.7 m/h and 15.3 m/h were used and raw water turbidity was below 1 NTU except for one small increase to 2.6 NTU. During the spring trial, rates of 11.7 m/h and 14.3 m/h were used and raw water turbidity was generally below 10 NTU for the majority of the spring session, with two turbidity spikes >100 NTU and a third at 33 NTU (all three corresponded with rainy weather). Reference: Oakville WPP Pilot Study, October 2011, Associated Engineering.

(South Halton, June 2014) state a filtration rate of 14.3 m/hr with 6 filters in service (one filter offline and one in backwash mode).

2.1.9 Clearwell and Reservoir

Filtered water exits each of the individual filters via filter effluent piping and discharges into two clearwells below. The two clearwells provide a total storage volume of 1,660 m^3. The reservoir has a serpentine design with 0.3 baffling factor; 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 yard 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.

Summary of challenges and constraints (Technical Baseline Review Memo #3, GHD, Dec. 2014):

- In the event that the ozone system is off-line, the reservoir is required to provide chlorine contact time for primary disinfection. Refer to Table 2-4 for a summary of the minimum reservoir water depth required to meet the concentration time (CT) requirements for 0.5 log *Giardia* inactivation at various flows and water temperatures.[10] Limitations, if the ozone system is off-line, include the following:
  - *Giardia* and viruses: Primary disinfection using chlorine can be achieved within the reservoir for all flows with a water temperature 10 °C or greater. For 5 °C water temperatures or lower, primary disinfection is attainable for flows less than 120 ML/d. The minimum and average water temperatures for the Oakville WPP are 3 °C and 14 °C, respectively.[11] At these historic average and minimum temperatures, noting that peak demand is unlikely to coincide with minimum temperatures, the existing reservoir would be able to meet the secondary disinfection requirements at the re-rated plant capacity.
  - *Cryptosporidium*: Chlorine cannot meet the WPP's internal objective of 1.0 log inactivation.

Table 2-4 Minimum Reservoir depth required to provide primary disinfection for *Giardia* inactivation

Temperature (°C)	0.5	5	10	15
CT required for 0.5-log *Giardia* inactivation (mg x min/L)	48	33	25	17
Required contact time (min)*	34.3	23.6	17.9	12.1
Reservoir flow (ML/d)	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

*Assumes a reservoir baffling factor of 0.7 (Superior) and chlorine residual of 2.0 mg/L at the outlet.

[10] 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.

[11] Based on January 2010 to June 2014 operational data. See Table 2-1 in GHD's Technical Baseline Review Memo #3, Dec. 2014.

Darker shaded cells indicate insufficient conditions based on the stated parameters and maximum reservoir water depth of 3.2 m.

2.1.10 High Lift Pumping Station (HLPS)

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, the original capacity of each which was approximately 65 ML/d at 59 m TDH. The following table highlights the in-situ tested capacities as per the performance test that was completed in January 2015. The reports note that overall efficiencies of the pumps are within 4% of the original manufacturer's efficiency, and the overall head is relatively in-line with original manufacturer's head.

Table 2-5 High lift pumps, tested capacities of individual pumps

Pump	Tested best efficiency point	Typical operating point	Ref.
HLP1	618 L/s (53.4 ML/d) at 64.7 m TDH, 87.9% efficiency	808 L/s (69.8 ML/d) at 53.8 m TDH, 85.7% efficiency	(1)*
HLP2	650 L/s (56.2 ML/d) at 63.5 m TDH, 88.1% efficiency	817 L/s (70.6 ML/d) at 53.6 m TDH, 85.9% efficiency	(2)*
HLP3	619 L/s (53.5 ML/d) at 65.0 m TDH, 88.1% efficiency	816 L/s (70.5 ML/d) at 53.1 m TDH, 85.2% efficiency	(3)*

TDH = total dynamic head (m).
(1) January 13, 2015 HLP1 Performance Test (dated February 24, 2015), Report by HydraTek & Associates.
(2) January 13, 2015 HLP2 Performance Test (dated February 24, 2015), Report by HydraTek & Associates.
(3) January 13, 2015 HLP3 Performance Test (dated February 24, 2015), Report by HydraTek & Associates.
*See reports for full test curves, manufacturer curves, notes on test conditions, and full observations and recommendations.

The present rated capacity of the HLPS (two pumps online, one pump offline) is sufficient for 109 ML/d, and the present firm capacity (all three pumps online) is sufficient for 130 ML/d. A fourth high lift pump is scheduled for future installation.

Summary of challenges and constraints:

- The fourth high lift pump needs to be installed for contingency measures, which will therefore increase the HLPS rated capacity sufficiently to 130 ML/d.

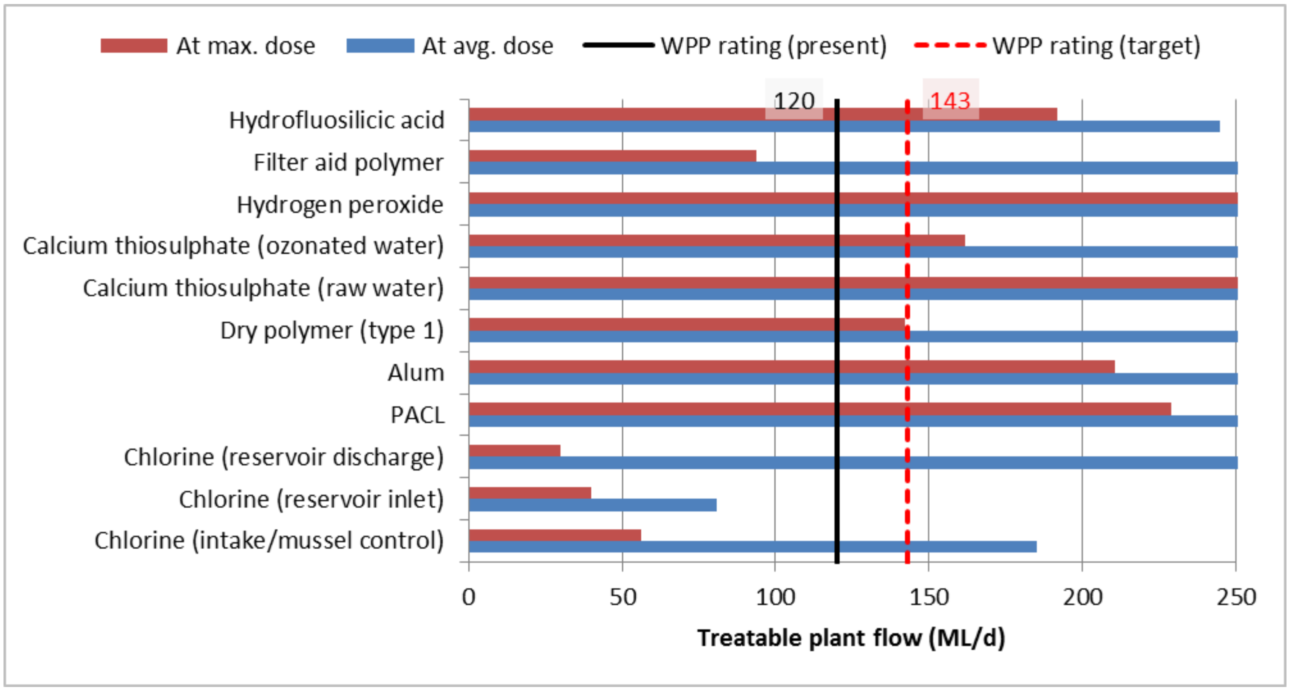
2.2 Chemical systems

This section provides an overview of the Oakville WPP's chemical systems. This information was first reported in detail in the Technical Baseline Review (Memo #3, December 2014), completed as part of this Class EA study, which documented the findings from desktop studies and information gained from GHD's site visits and interviews with WPP operations staff on July 8 and 9, 2014. See Memorandum #3 for more information on the baseline status of existing chemical systems at the Oakville WPP.

Table 2-6 below summarizes the capacities of the chemical systems in terms of the number of metering pumps and "treatable plant flow" at both average and maximum dosage (modified from GHD's Technical Baseline Review Memo #3, Dec. 2014).

Figure 2-3 below presents the "treatable plant flow" information in bar chart format.

Figure 2-3 Chemical systems capacity summary



Note: For clarity, the maximum treatable plant flow shown is 500 ML/d. For flows above this value, refer to Table 2-6 below.

Table 2-6 Chemical systems capacity summary

Chemical	Injection location(s)	No. of feed pumps	Treatable plant flow (ML/d)		Dose (mg/L)		Days of storage at 130 ML/d and avg. dose	Data source (dosages)
			At avg. dose	At max. dose	Avg.	Max.		
Chlorine (gas)	Raw water intake (zebra mussel control)	1 chlorinator	185	56	2.6	8.6	4.8	(3A)
	Reservoir[a] inlet (secondary disinfection)[b]	1 chlorinator	81	40	5.9	12.0	4.8	(3A)
	Reservoir* discharge (finished water trim)	1 chlorinator	728	30	0.33	8.1	4.8	(3A)
PACL	In-line injection flash mixer chamber	4 duty / 1 standby	807[d]	229[d]	2.0	7.2	107	(1)
Alum[c]	In-line injection flash mixer chamber	2 duty / 1 standby	674	211	14.9	47.6	15	(2)
Dry polymer (Type 1)	Actiflo® system	2 duty / 1 standby	538	142	0.14	0.55	5 hrs	(1)
Calcium thiosulphate	Raw water wetwell	1 duty / 1 standby	483	304	0.7	1.1	14	(2)
	Ozone contactor outlet	2 duty / 1 standby	384	162	1.7	4.1		(2)
Hydrogen peroxide	Cell 7 of ozone contactors 1 and 2 (two locations)	2 duty / 1 standby	575	411	1.3	1.8	25	(4)
Filter aid polymer (Type 2)	Upstream of filters	1 duty / 1 standby	376	94	0.25	1.0	13	(5)
Hydrofluosilicic acid	Reservoir* inlet	1 duty / 1 standby	245	192	0.57	0.73	78	(2)

Data in this table is modified from GHD's Technical Baseline Review, Memo #3, Dec 2014, and has been updated where appropriate and as indicated.

[a] Treated water

[b] Reservoir inlet chlorine dosages in 2014 were high due to necessary over quenching of the ozone residual. Prior to the completion of the Phase 2 upgrades in December 2014, the filter building did not have an HVAC system to handle ozone off-gassing. The reservoir inlet chlorine dosage will thus be reduced from 2015 onward. The data reported here has been updated from 2014 values using 2015 values.

[c] Alum is now strictly used as a contingency to PACL.

[d] Two additional PACL pumps have been installed since GHD's Dec. 2014 memo. The treatable plant flow values have been updated to reflect this increased capacity.

(1) 2012-2013 operational data.

(2) 2013-2014 operational data.

(3) 2014 operational data.

(3A) 2015 operational data (updated, previously reported using 2014 data).

(4) Anticipated based on 1:2 hydrogen peroxide to ozone dose ratio and ozone residuals of 2.5 mg/L (average) and 3.5 mg/L (maximum).

(5) Assumed dosage based on polymer type and application.

2.2.1 Chlorination systems

Gaseous chlorine is applied at the Oakville WPP at three locations: raw water intake (zebra mussel control when raw water temperature is greater than 7 °C), treated water reservoir inlet (for secondary disinfection), and high lift pump suction header (trim / distribution system residual maintenance chlorination).

Furthermore, the Region has noted that the chlorine system is aging and may be due for replacement.

Summary of constraints and challenges using GHD's Technical Baseline Review Memo #3 (Dec. 2014) and updated to account for more recent data and equipment upgrades:

- The treated reservoir inlet chlorine injection point does not meet the target re-rated production flows at average or maximum dosages (see Table 2-6). It is noted however that the dosage for this chlorinator is high due and is expected to be due to necessary over quenching of the ozone residual. Thus if chlorine was not needed to quench excess ozone it is expected that the present chlorinator capacity would be sufficient.
- The chlorine feed system does not have redundancy built into its chlorinator system.
- The number of days of storage provided by twelve ton gas chlorine cylinders, at 130 ML/d and average chlorine dosage, is approximately 4.8 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. It is noted again that if the chlorine dosages were not also functioning to quench ozone that the present storage capacity would suffice for a longer duration.

Additional note: Previously it was noted by Region operations staff that the Treated Reservoir Inlet chlorine dosages were higher than normal due to carry over of calcium thiosulphate into the finished water from ozone quenching. Upgrades and modifications made by the Region as part of the Phase 2 upgrades would result in decreasing the calcium thiosulphate dose required for ozone quenching and therefore decrease the Treated Water Reservoir Inlet chlorine dose. Monitoring the Treated Water Reservoir Inlet chlorine dose was planned throughout this year (2015) to determine any changes. Region operations staff has noted that monitoring has not been done to date. Presently, Region operations are dosing in compound loop on SCADA which takes the combined filter effluent flow and free chlorine residual at the inlet to achieve the SCADA setpoint input. To date the results have been satisfactory and no other adjustments were made.

2.2.2 Poly aluminium chloride (PACL)

PACL is added upstream of the Actiflo® process to assist with coagulation, flocculation and sedimentation. PACL is injected into the in-line injection mixer chamber on the common 1,200 mm diameter raw water feed line.

Two additional PACL metering pumps have been installed, thus addressing the issue that was previously flagged (Technical Baseline Review Memo #3, GHD, Dec. 2014).

2.2.3 Alum

The alum system is now only used as contingency, as a redundancy measure for the PACL system.

2.2.4 Dry polymer (Type 1)

Dry polymer (Type 1) can be injected into the Actiflo® system at three locations and, based on current operations, according to the following splits: microsand injection (hydrocyclone) (50%), injection tank (25%), and maturation tank (25%). 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.

Summary of constraints and challenges (Technical Baseline Review Memo #3, GHD, Dec. 2014):

- The existing Type 1 polymer feed system can treat flows up to 142 ML/d at the maximum 0.55 mg/L dosage. Approximately 4-5 hours of storage is provided by one 1.8 m^3 day/batching tank.
- A new actuator has been installed, resolving the previously noted issue that the hopper system has a tendency to clog (Technical Baseline Review Memo #3, GHD, Dec. 2014). The Region will also soon be installing a polymer batch density monitor.

2.2.5 Calcium thiosulphate

Calcium thiosulphate is used for dechlorination of the following chlorinated waters: raw water for zebra mussel control; 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 contactor (prior to filtration) and at ozone residual analyzer drain lines. The injection locations presently in service are the raw water wetwell, the ozone contactor outlets (cell 10 of each ozone train), and the ozone analyzer drain lines to waste tank.

There were no treatment or bulk storage constraints to note for meeting the target re-rated production capacities for the Calcium thiosulphate system.

2.2.6 Hydrogen peroxide

The WPP has the ability to inject Hydrogen peroxide to the ozonated water stream for advanced oxidation during extreme taste and odour events. Hydrogen peroxide is injected within Cell 7 (of 10) of each ozone contactor.

The Oakville WPP Hydrogen peroxide system has not been used so operations data is not available for the average and maximum dosages. Theory and standard practices were used to estimate the average and maximum dosages and corresponding treatable plant flow rates; refer to GHD’s Technical Baseline Review Memo #3 (Dec. 2014) for more information.

There were no treatment or bulk storage constraints to note for meeting the target re-rated production capacities for the Hydrogen peroxide system.

2.2.7 Filter aid (liquid) polymer (Type 2)

The filter aid polymer is used intermittently during high turbidity events, with one line to the gravity thickener and the second line added to the ozonated water conduit, upstream of filtration. The polymer feed system does not have dose control or dose feedback available so actual operating dosages are not available.

Summary of constraints and challenges (Technical Baseline Review Memo #3, GHD, Dec. 2014):

- At dosages greater than 0.6 mg/L, one PolyBlend unit cannot meet the feed requirements (average and maximum dosages are 0.25 mg/L and 1.0 mg/L, respectively). To accommodate higher dosages, the standby unit could be placed online to meet the targeted production requirements; however this leaves the system without redundancy. Whether dosages above 0.6 mg/L are required remains to be evaluated.

Previously the Region questioned whether the existing filter aid polymer (Type 2; cationic) is appropriate for the application. Region operations have confirmed that this is no longer a concern.

2.2.8 Hydrofluosilicic acid

Hydrofluosilicic acid is used at a low dose for fluoridation of the treated water prior to distribution and is injected at the treated water reservoir inlet.

There were no chemical feed or bulk storage constraints to note for meeting the target re-rated production capacities for the fluoride system.

2.3 Hydraulic analysis

2.3.1 Objective and data

A hydraulic model of the Oakville WPP was developed as a background study prior to the full scale test. The objectives of this model were to verify the engineering basis for 130 ML/d net production flows (up to 143 ML/d gross) and anticipate hydraulic restrictions within the WPP. Preliminary findings were presented in GHD's Technical Baseline Memo #3 (Dec. 2014).

Information was used from the following sources to develop the hydraulic model: Phase 1 Upgrades Oakville WPP record drawings (October 2008, Contract W-2062(A2)-04); Phase 2 Upgrades Oakville WPP design drawings (May 2012, Contract W-2062(B)-12); Oakville WPP Operations Manual (June 2009); and other consultant's technical memoranda/reports including 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; SCADA data from the May 5, 2015 hydraulic test at 130 ML/d net production. It should be noted that the elevations and dimensions of equipment and structures were solely obtained from the noted record sources, and that none were physically measured in the field; SCADA data was used for calibration purposes.

Based upon the records received from the Region, it should also be noted that information on the ozone-to-filter conveyances was limited. Our understanding of the four routes from the ozonated water vault (chamber between the ozone contact chambers and the filters) to the filters was previously described in GHD's Technical Baseline Memo #3 (Dec. 2014) and is supplemented in the following discussion as necessary. Simplifications were made in the hydraulic model for this portion of the WPP based on the information available and in line with the objectives for this task.

2.3.2 Model development

The hydraulic model was developed based on known inverts, pipe diameters, and estimated head losses. Refer to Appendix C for more details on the development of the model.

2.3.3 Key findings

The key findings from the hydraulic modeling of the plant are reported here. Some of these were originally reported in GHD's Technical Baseline Memo #3 (Dec. 2014), and an updated discussion is provided here.

The following WPP conditions have been observed as potentially having hydraulic limitations that may, at present, impact the targeted re-rated production capacity of 130 ML/d.

(a) Filter Inlet. Flow to the existing filters is provided through the “filter influent ring”. At each filter the water enters through a slide gate, rises over the inlet gullet wall and fills the top of the filter area. Figure 2-4 shows a cross section of Filter 2, showing the Top of Gullet Wall elevation.

Figure 2-4 Cross section of Filter 2[12]

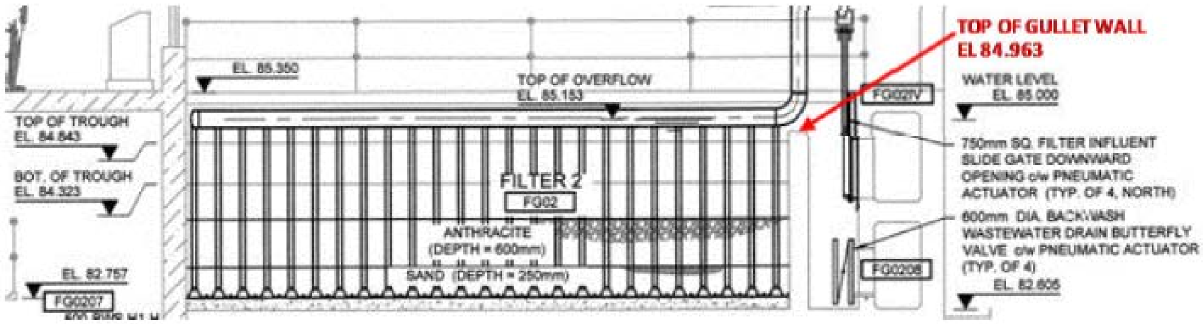


Figure 2-4 summarizes select elevations. Based on the noted elevations, the depth of water over the gullet wall during normal operation is only 0.037 m. [13] This is hydraulically inconsistent with the minimum depth that would be required due to the weir effect[14] of the wall alone. The maximum freeboard (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 below the filter overflow elevation (0.19 m above the gullet wall) will require precise upstream flow control and hence limited operational flexibility. Ideally, to allow for more operational flexibility, the noted maximum freeboard over the gullet wall is over 1 m. It was originally anticipated that this may be the root cause of Filter 1 (and sometimes Filter 8) being “starved” during high flows.

However, after observing filter performance during the full scale test, the draining of Filter 1 was not due to the maximum freeboard limitation noted, but appeared to result from (1) the geometry / position of the filter trough walls relative to the influent gate and (2) the high flow rate of the water in the filter inlet channel just outside of the Filter 1 influent gate causing the flow to pass by Filter 1’s inlet gate. This is discussed in more detail in the Discussion Section 4.1.8 of this report.

Table 2-7 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)	85.000	0.037
Top of Gullet Wall	84.963	-

[12] Oakville WPP Phase 2 Upgrades, Contract W-2062(B)-12, construction drawing P1004. Gullet wall elevation from drawing P1006 of the same contract.

[13] As reported in the referenced drawing. Field conditions observed by GHD indicate that this depth will vary above and below the noted 0.037 m.

[14] 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.

(b) Filter Outlet. Results from the hydraulic model at 143 ML/d indicated relatively high headloss from the filter effluent piping into the Clearwell below. Factors influencing the hydraulic head in the filter beds are as follows:

1. Headlosses due to filter effluent pipe sizes and fittings (increasing with increasing flows)
2. Headloss for proper rate-of-flow control (i.e. valve throttling)
3. Headloss of a clean filter media bed, and development of headloss as the filter media becomes dirty
4. Clearwell levels (increased clearwell level results in decreased hydraulic head in the filter bed)

Under high clearwell level conditions and at the targeted flow conditions of 143 ML/d, the available driving head in the filters may be reduced to less than 2 meters. Considered together, these factors reduce the driving head available for filter operation and will result in shorter filter run times between backwash cycles, (As noted, filter run times will also be shortened at an increased flow rate due to the associated increased sediment build-up.)

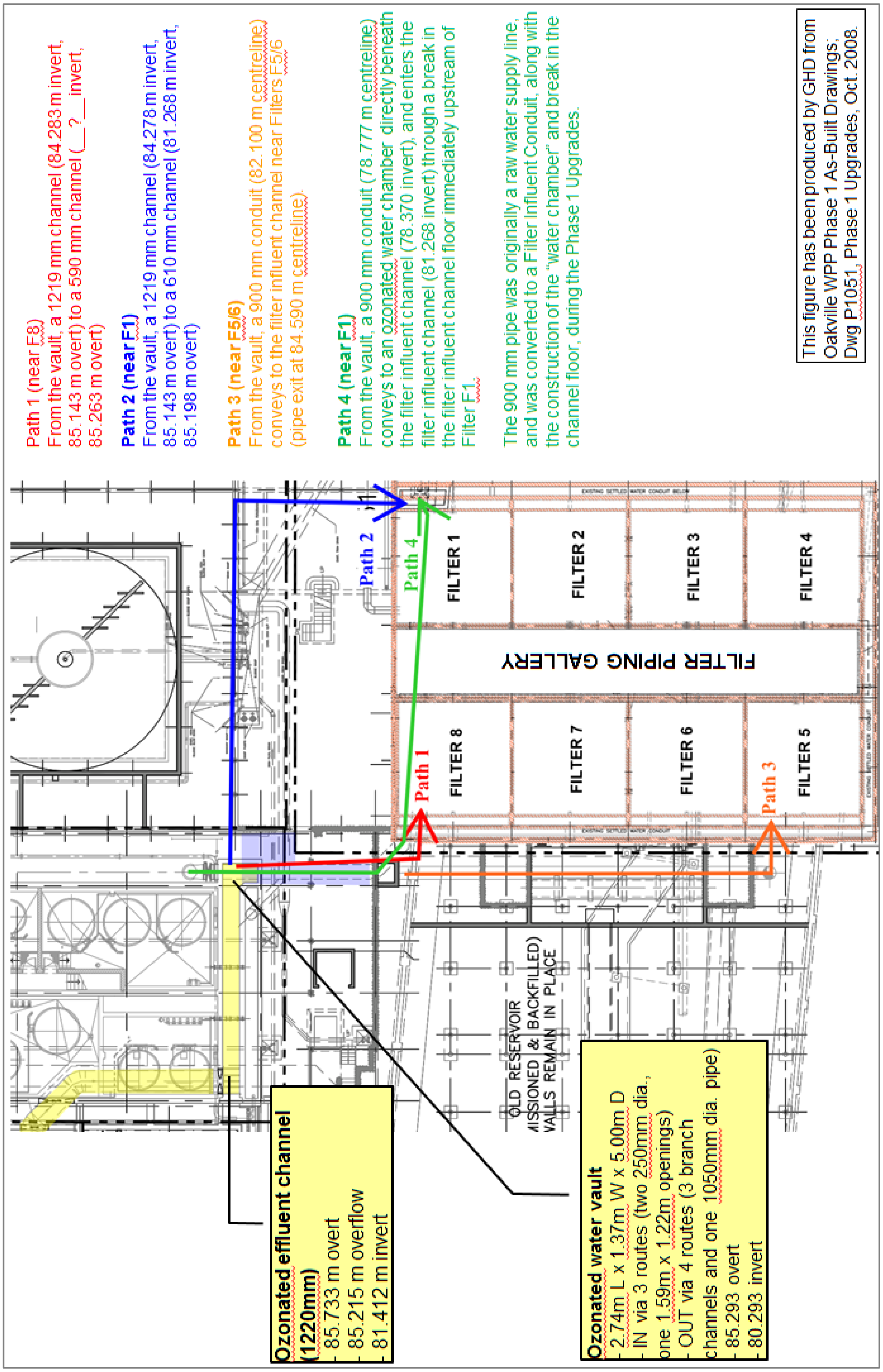
During the full scale test, the filter rate was easily controlled by throttling the filter effluent valves, and the operators were able to increase filtration rates as necessary. Thus, this did not present itself as an issue during the full scale test. See Section 4.1.8 for further discussion.

(c) 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 (see Figure 2-5). WPP operations staff have indicated that at flows above 120 ML/d, Filter 1 becomes starved. It was originally 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. It was also originally suspected that 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 channels to the filters (normally this would not result in a significant imbalance of flow to individual filters). Finally, it was also originally suspected that, due to the limited hydraulic freeboard (discussed above), this restriction cannot be overcome with adjustments to control logic alone as the tolerances are too tight.

While these items are noted, after observing filter performance during the full scale test, the draining of Filter 1 was not due to the maximum freeboard limitation noted, but appeared to result from (1) the geometry / position of the filter trough walls relative to the influent gate and (2) the high flow rate of the water in the filter inlet channel just outside of the Filter 1 influent gate causing the flow to pass by Filter 1's inlet gate. This is discussed in more detail in the Discussion Section 4.1.8 of this report.

(d) Clearwell and Reservoir. Both the clearwell and the reservoir have maximum operating depths of 2.8 m. However the floor elevation of the reservoir is approximately 0.21 m *above* that of the upstream clearwell (approximately 8% of the operating range). As discussed in Sections 3.4, during the full scale test and filter backwash, the Clearwell and Reservoir levels dropped, and more significantly so within the Reservoir. The model also shows the Reservoir operating at 10-20% lower than the Clearwell. Region operations have indicated that during a backwash under lower flow conditions (50 to 80 ML/d), storage levels typically drop from about 90-82% (clearwell) and 89-80% (reservoir), and that the levels are usually replenished in about 1.5 hours.

Figure 2-5 Ozone-to-filter flow paths



3. Full scale hydraulic and process test

3.1 Introduction and objective

A full scale hydraulic and process test of the WPP at the target re-rated production rate of 130 ML/d net (143 ML/d gross) was conducted on May 5, 2015. The detailed plan for this undertaking is contained in the Performance Test Plan document (GHD, May 2015) and was approved by the MOECC.

The objective of this test was to evaluate the WPP's ability to reliably produce 130 ML/d for a duration of 3 hours. If no major issues were encountered during the first two hours, the WPP would be further tested by attempting a filter backwash during the last hour of the test.

This section reports on the findings of the test, combining both field observations and post-test analysis of data (see Section 3.3 below).

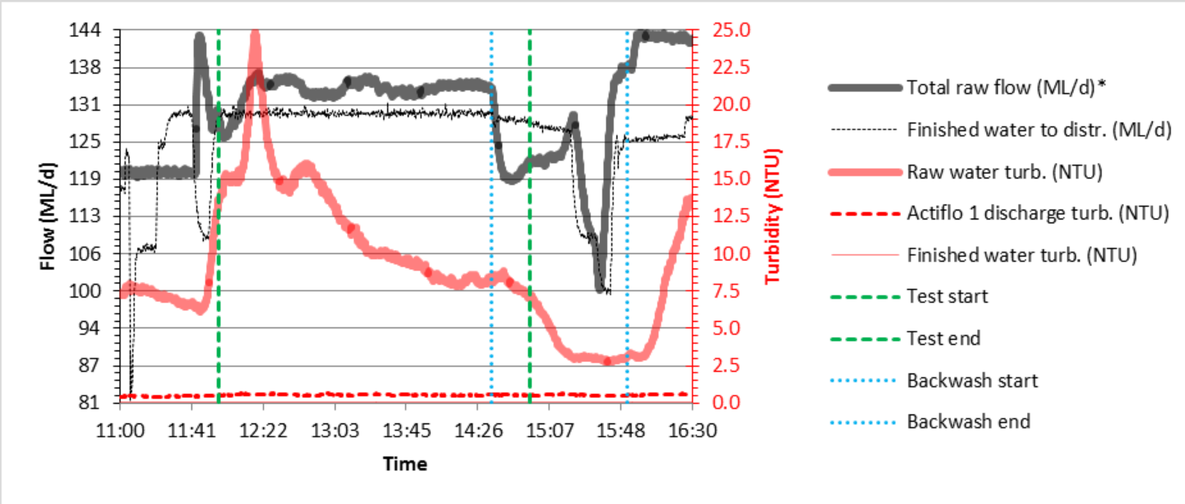
3.2 Overview

A brief overview of the May 5, 2015 events is listed in Table 3-1. An overview of the plant flows (raw and production) and turbidity levels is depicted in Figure 3-1. A summary of key SCADA parameter statistics, while running the WPP at 130 ML/d, is provided in Table 3-2.

Table 3-1 Overview of the full scale test day's events

Time	Event / note
07:00	GHD team arrived onsite. Plant operating at 60 ML/d.
08:10-11:30	Ramp up period: finished water production setpoint incrementally increased to 80, 100, 109, 115, 120, 125, and 130 ML/d.
11:40-11:55	Production issue discovered. Operations altered HLP3 duty max speed and LLP max flow interlocks. Finished water production temporarily reduced to 109 ML/d.
11:56	Production flows reach 130 ML/d again. Begin test period. Test time 0:00 h.
14:34	Commence backwash of Filter 6 (Test time 2:38 h). Filter 1 level begins to drop. Reservoir avg. level begins steady decline from 80%.
14:35-14:55	Production flow begins gradual decline from 130 ML/d to 129 ML/d.
14:56	Test time 3:00 h. End of test period.
14:56-16:36	Clearwell avg. level drops to 80% (14:56 h), rises, and drops again to 79% (15:35 h). Reservoir avg. level drops to 50% (15:39 h); does not begin recovery until raw water flow increases to 143 ML/d and production flow is decreased to 125 ML/d (16:00 h). Filter 1 level drops during backwash; operations decrease flow from an attempted 218 L/s (15:31 h) to ultimate 90 L/s (15:34-16:36 h). Other filters operating at approx. 223 L/s.
15:50	Filter 6 backwash complete, brought back online.

Figure 3-1 Overview of the full scale test flows and turbidity



*Measured by sum of Actiflo 1 and 2 inlet flows.

Table 3-2 Key SCADA parameter Test Period statistics

SCADA parameter	Mean	Min – Max	Range
Flows			
Raw water flow (ML/d)*	132.3	118.4 – 136.9	18.4
Combined filter effluent flow (ML/d)	132.6	115.1 - 136.9	21.8
Finished water to distribution (ML/d)	129.8	127.5 – 131.7	4.2
Turbidity			
Raw water (NTU)[15]	11.79	7.02 – 24.89	17.87
Actiflo 1 (NTU)	0.55	0.49 – 0.65	0.16
Actiflo 2 (NTU)	0.58	0.49 – 0.72	0.23
Filter (combined) (NTU)	0.047	0.045 – 0.050	0.005
Finished water (NTU)	0.058	0.054 – 0.064	0.010
Discharge pressure			
East header to distribution (kPa)	585.5	580.4 – 591.0	10.6
West header to distribution (kPa)	592.2	587.0 - 597.7	10.7
Levels			
Clearwell (average of cells 1 and 2) (%)	89.9	83.2 – 90.5	7.3
Reservoir (average of cells 1 and 2) (%)	78.8	72.3 – 80.9	8.6
Log inactivation (Plant overview)			
Cryptosporidium – contactor 1 – contactor 2	0.9 0.8	0.8 – 1.1 0.7 – 0.9	0.3 0.2
Giardia	7.2	6.8 – 7.7	0.90
Virus	14.1	13.7 – 15.0	1.3

*Measured by sum of Actiflo 1 and 2 inlet flows.
Note: Values in this table are based on the test duration time as indicated in Section 3.3.

[15] During the ramp up period, raw water turbidity had an ultimate peak of 27.4 NTU. The minimum raw water turbidity during the test day (outside of the test and ramp up periods) was 0.2 NTU.

3.3 Data and Methods

The data collected and used include the following (refer to Appendix B).

- WPP process data collected by the Region's SCADA system and provided by the Region. The dataset included 132 parameters trended at approximately 10 second intervals between 12:00:01 AM and 11:59:59 PM on May 5, 2015 (total of 8640 members). Data was received in Microsoft® Excel format (*.xls).
- Water quality data obtained from independent laboratory analysis of field samples collected.
- Field observations made onsite and in discussions with the operators during the full scale test, including information from the facility's operator log book.

Test time used for data analysis. Upon reviewing the SCADA data received, the official three hour Test Period was determined to be from **11:56:45 to 14:56:36** (~3.00 hours) (n=1080 members). All statistics from SCADA-obtained parameters are based on this subset of records unless otherwise stated.

This period was selected because it is three hours in duration and had a mean average net production-to-distribution flow rate of approximately 130 ML/d[16]. The start and end times were also affected by process challenges experienced at the beginning and end of the test which affected the net production flows outside of this period; see Section 3.4 below for details.

Toward the end of the test, Filter 6 underwent a backwash procedure between 14:34:16 and 15:52:26 (1 hour 18 minutes, the time during which the Filter 6 effluent valve position was 0% open). While there is overlap between start of the backwash event and the end of the official test time, the backwash event is not entirely contained within the Test Period (see Section 3.4 below for explanation), however both events will be shown in the figures for analysis purposes.

Data processing. Data processing, analysis, and figure generation was done using Microsoft® Excel (*.xlsm format).

Water quality data. Grab samples were collected by GHD personnel on the day of the test. Bottles supplied by SGS Canada Inc. were used for collection. Samples were stored on ice and couriered the next day to the laboratory for analysis. Refer to Appendix B for the original laboratory reports which include the lab's methods of analysis.

3.4 Note on challenges at beginning and end of the Test

As evidenced in Table 3-1 and Figure 3-1, both minor and moderate challenges were encountered at both the beginning and end of the test, which in turn affected the duration at which net production flows operated at the targeted 130 ML/d.

Ramp up period. Operators discovered that the High Lift Pump 3 maximum speed interlock and low lift pump maximum flow (interlocked at 120 ML/d) required adjustment[17]. At approximately 11:40 AM the production setpoint was reduced to 109 ML/d for approximately ten to fifteen minutes while this was resolved. The net effect only resulted in the test time being restarted.

[16] It is noted however that WPP clearwell and reservoir storage levels began to drop at the end of the noted period, thus actual average production flows are less; see discussion in following section.

[17] Many SCADA setpoints and interlocks required adjustment for operating the WPP at 130 ML/d net. There were a few of those that were overlooked during the initial SCADA adjustments for performance test preparation. The purpose of these interlocks in particular are to ensure the WPP remains within license requirements for raw water taking and finished water production.

Filter backwash. During the backwash of Filter 6, operators discovered that the master filter rate was set to 1400 L/s (200 L/s/filter with seven of eight filters online);[18] the master filter rate was automatically activated when the backwash commenced. The clearwell levels reportedly dropped from 89% to 80%, while the reservoir levels reportedly dropped from approximately 79% to 50%, while the high lift pumps continued to pump 130 ML/d. See Figure 3-2 below.

During a filter backwash, flows from the Clearwell are drawn to the clearwell flume for backwashing purposes; thus extra demand is placed on the clearwell and the feeding filters. Combined with the backwashing filter being offline, this resulted in the depletion of Clearwell and Reservoir levels (Figure 3-2), with the Reservoir levels being depleted much lower than the upstream Clearwell. Water surface elevations of the Clearwell and Reservoir were developed using floor elevations from the drawing record[19] and the SCADA data (Figure 3-3 and Figure 3-5). These water surface elevations demonstrate a very small difference in hydraulic head (+10 cm) during the test period and initially during the filter backwash (+10 cm), but increase substantially as the Reservoir level continues to drop (+70 cm). [20]

However, note that under lower flow conditions (~45 ML/d) prior to the ramp up period, the difference in the downstream reservoir hydraulic head was approximately +15 cm (see Figure 3-5), which is not possible (i.e. the reservoir level cannot have a higher surface elevation than the clearwell with forward moving flow). This strongly suggests an inaccuracy either in the elevations reported in the drawings or in the level monitoring systems. It is recommended that the Region verify the entirety of the clearwell and reservoir monitoring systems

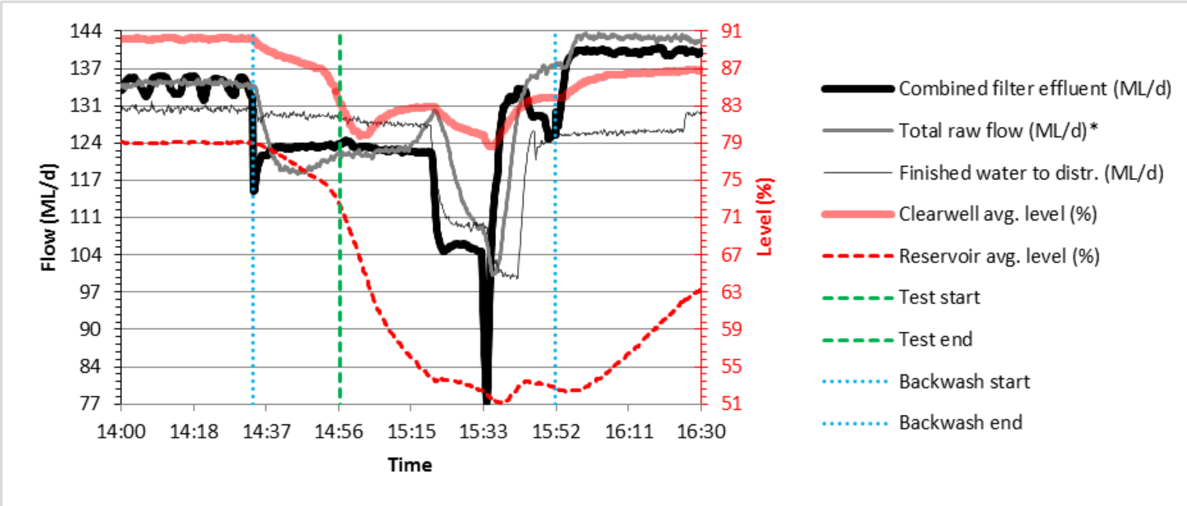
Additionally, we note that the floor elevation of the Reservoir is approximately 0.21 m higher than the upstream Clearwell (see footnote 19 for details). This has the potential to create a hydraulic preference either for flows to feed backward from the Reservoir to the Clearwell, or for the high lift pumps to draw down reservoir levels if the filtration rate is insufficient for high lift pumping needs. After further analysis and discussion with the Region, our opinion is that of the latter.

[18] Approximately 120 ML/d, the maximum raw water taking allowed under present facility licenses.

[19] Water surface elevations were calculated using the calibrated range of the level transmitters for the Clearwell and Reservoir (0-100% corresponds to 0.0-2.8 m for both, according to plant operations) and floor elevations of 76.821 m and 77.037 m for the Clearwell and Reservoir, respectively (references: Drawing S601 from Phase 1 upgrades; Drawing P1004 from Phase 2 upgrades). It is further noted that the drawings note the Clearwell floor elevation as "varying", and that 77.037 m corresponds to the Reservoir floor elevation near the inlet/outlet valves as this is where the level transmitter is located (per Region operations).

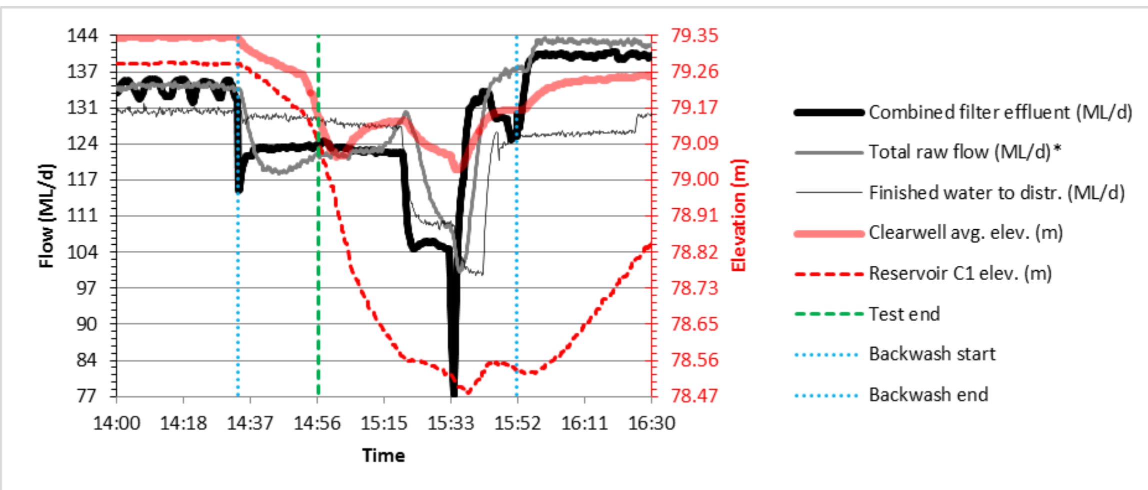
[20] Additional figures are provided for comparison purposes: Comparison of Reservoir and Clearwell levels in percent (Figure 3-2 and Figure 3-4) against the water surface elevations (Figure 3-3 and Figure 3-5) both during the backwash event (Figure 3-2 and Figure 3-3) and over a longer duration prior to the ramp up period (Figure 3-4 and Figure 3-5).

Figure 3-2 Plant flows with storage levels (%) during backwash of Filter 6



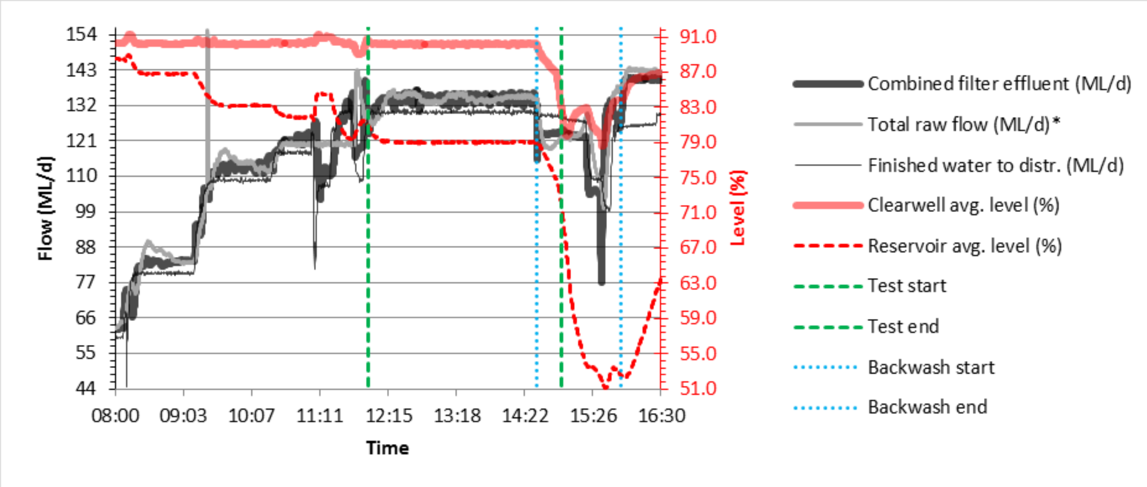
*Measured by sum of Actiflo 1 and 2 inlet flows.

Figure 3-3 Plant flows with storage level elevations during backwash of Filter 6



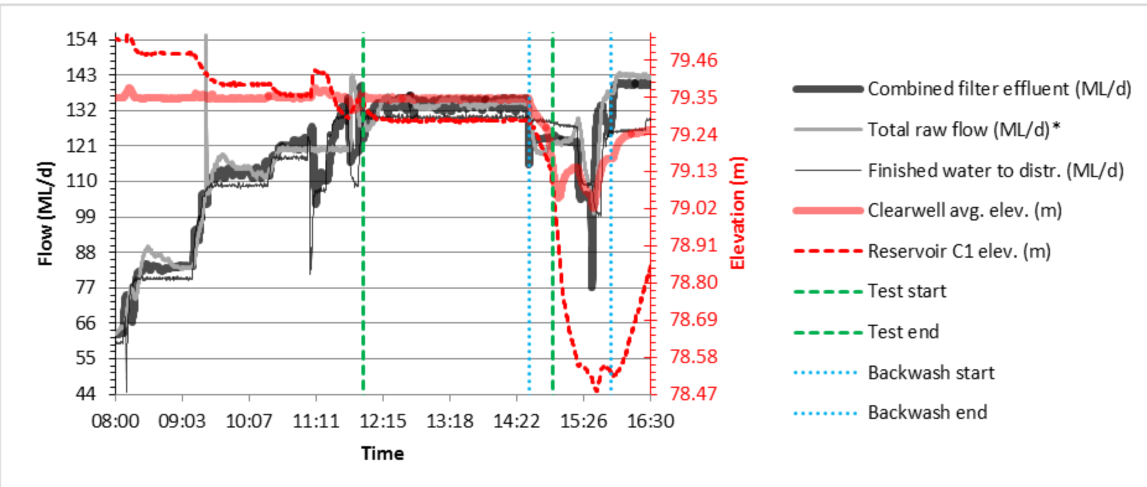
*Measured by sum of Actiflo 1 and 2 inlet flows.

Figure 3-4 Plant flows with storage levels (%) from 8:00 AM to 4:30 PM



*Measured by sum of Actiflo 1 and 2 inlet flows.

Figure 3-5 Plant flows with storage level elevations from 8:00 AM to 4:30 PM



*Measured by sum of Actiflo 1 and 2 inlet flows.

3.5 Process systems

This section discusses the WPP's performance for each process system.

3.5.1 Intake and Low Lift Pumping Station (LLPS)

A summary of the parameters monitored by SCADA are presented in Table 3-3 and Figure 3-6 below. The raw water intake and LLPS performed well without major issues during the full scale test. The only challenge encountered was a SCADA interlock as discussed below.

Table 3-3 Intake and LLPS SCADA parameter statistics during Test Period

SCADA parameter	Mean	Min – Max	Range
Raw water turbidity (NTU)*	11.79	7.02 - 24.89	17.87
Raw water pH*	8.06	8.04 - 8.08	0.04
Raw water temp (°C)*	5.9	5.8 - 6.0	0.2
Raw water free chlorine residual (mg/L)	0.27	0.15 - 0.46	0.31

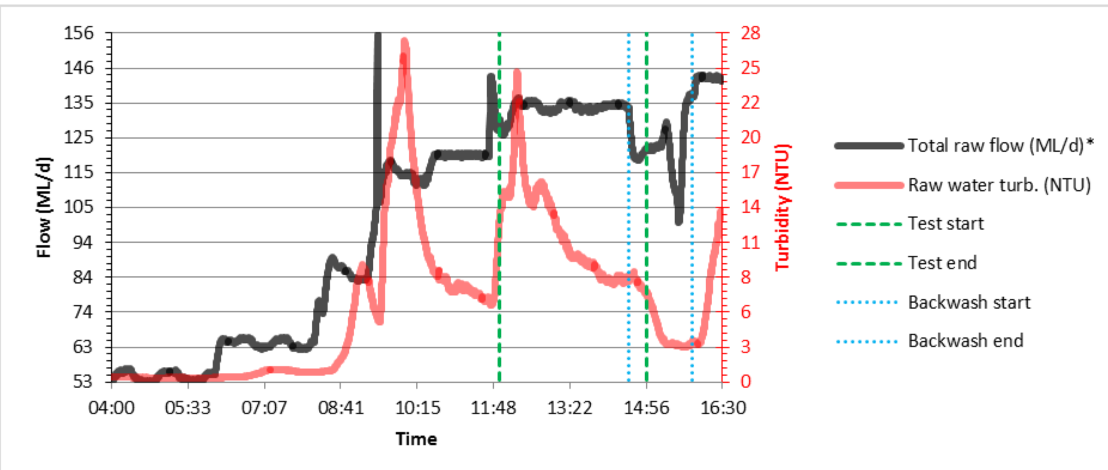
SCADA parameter	Mean	Min – Max	Range
Raw flow total (ML/d)**	132.3	118.4 - 136.9	18.5
Raw Water Wet well level (%)	84.1	83.4 - 85.2	1.8
LLP1 speed (%)	65.3	49.4 - 70.6	21.2
LLP2 speed (%)	67.5	51.2 - 72.8	21.6
LLP3 speed (%)	68.5	51.8 - 73.8	22.0

*Measured at the raw water wetwell in the Low Lift Pumping Station.
**Measured by sum of Actiflo 1 and 2 inlet flows.
Note: Values in this table are based on the test duration time as indicated in Section 3.3.

Prior to the ramp-up period (12:00 to 06:00), raw water turbidity and raw water flows averaged 0.41 NTU and 56 ML/d, respectively. At each stage in the ramp up period, the intake was scoured causing settled sediment to become suspended and thereby increasing the raw water turbidity measured in the wetwell. Raw water turbidity peaked in the ramp up stage at 27.4 NTU and at 24.9 NTU at the beginning of the Test Period after the final production ramp up. See Figure 3-6 below.

As discussed in Section 3.4 above, interlocks on the low lift pump maximum flow (interlocked at 120 ML/d) and HLP 3 maximum speed (see footnote 17) were discovered by Region operators at the end of the ramp up period when attempting to reach the targeted net production rate, which subsequently delayed the beginning of the three-hour test period.

Figure 3-6 Intake and LLPS: Raw water flow and turbidity



*Measured by sum of Actiflo 1 and 2 inlet flows.

Prior to the ramp-up period (12:00 to 06:00), the raw water wetwell average level was 88-89%. At each stage in the ramp up period, the extra demand placed on the wetwell slightly decreased its operating level, which stabilized at approximately 84%.

It should also be noted that both travelling water screens were online. The Performance Test Plan stated that one would be offline to field test their individual capacity. Furthermore, the travelling water screen differential level is not measured or trended through SCADA, and so no data was available for analysis. No issues were experienced during the test.

3.5.2 Actiflo® system

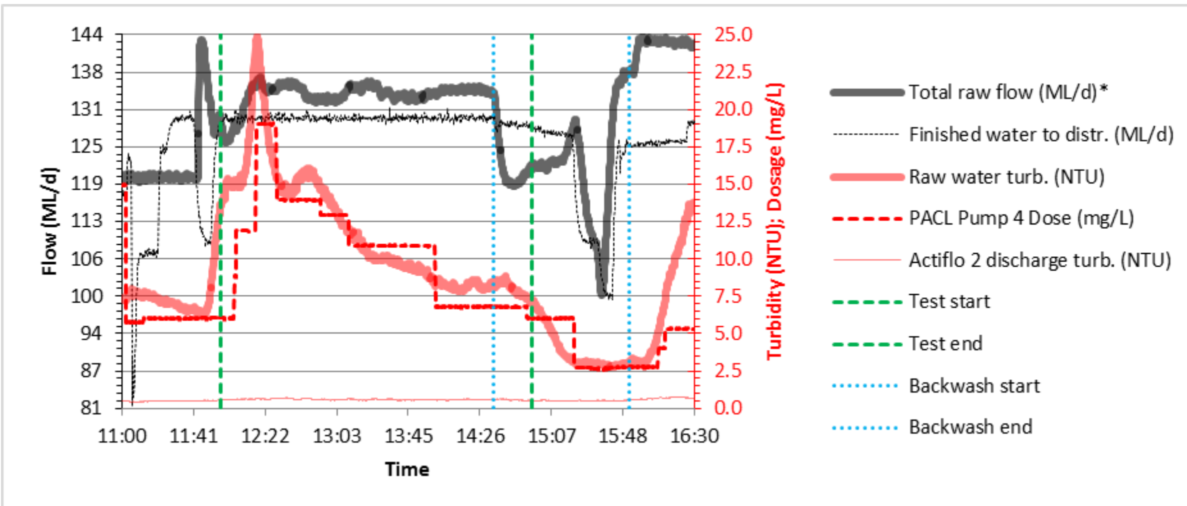
A summary of the parameters monitored by SCADA are presented in Table 3-4, Table 3-5 and Figure 3-7 below. The Actiflo® system performed well without any major issues during the full scale test.

Table 3-4 Actiflo® system SCADA parameter statistics

SCADA parameter	Mean	Min - Max	Range
PACL Pump 4 Dose (mg/L)*	10.58	5.99 - 19.08	13.08
Polymer (type 1) Pump 1 Dose (mg/L)**	0.12	0.12 - 0.12	0.00
Polymer (type 1) Pump 3 Dose (mg/L)**	0.12	0.12 - 0.13	0.01
Actiflo 1 pH	7.97	7.87 - 8.09	0.23
Actiflo 2 pH	7.66	7.58 - 7.76	0.17
Actiflo 1 discharge turbidity (NTU)	0.55	0.49 - 0.65	0.16
Actiflo 2 discharge turbidity (NTU)	0.58	0.49 - 0.72	0.23

*PACL Pumps 1 and 2 were not used during the test. PACL Pump 3 was only online for a brief time for post-maintenance testing and is thus not reported here.
**Polymer Pump 2 was not used during the test.
Note: Values in this table are based on the test duration time as indicated in Section 3.3.

Figure 3-7 Actiflo® system: Turbidity and PACL dosage



*Measured by sum of Actiflo 1 and 2 inlet flows.

Relevant notes from the WPP operator's log book and notes taken during the test are as follows:

- 08:10, added microsand to Actiflo unit 2 via the microsand handling unit, and sand to Actiflo unit 1 via sand bags (approximately 13 bags).
- 08:25, switched PACL pump duties from Pump 2 to Pump 4 (larger capacity) for increased plant flows.
- 08:30, increased coagulation (PACL) dose from 2 mg/L to 2.75 mg/L in preparation for intake scouring and turbidity spike.
- 09:02, Increased PACL pump stroke from 35% to 50%. Speed is in "plant auto" mode.

The following observations were made during the test:

- No inundation of the overflow weir in the Actiflo effluent (settled water) conduit was observed
- Some minor vibration of the effluent weirs in the settling tanks (one in each Actiflo) was noted at high flows. Some stiffeners may be required in these areas.
- No stability issues were noted for the maturation tank mixer.

- No floc carryover was observed.
- No ozone backfeeding into the Actiflo outlet channel and the associated risk of ozone off-gassing to the facility's atmosphere.

The goal for the Actiflo® system was to maintain the required post-Actiflo® (i.e. settled) water criteria at the net 130 ML/d flowrate through the coagulation, injection, and maturation tanks without inundating the overflow weir in the Actiflo effluent (settled water) conduit. The post-Actiflo® (i.e. settled) water performance criteria consist of: avoiding floc carryover to filters; settled water turbidity of < 1.0 NTU prior to filtration; and to meet filtered water turbidity objectives when settled water turbidity exceeds 1.0 NTU.

Table 3-5 shows the water quality parameters and their measured values during the testing.

Table 3-5 Actiflo® water quality parameters

Parameter at noted location	Mean	Min – Max	Range	Data source
Raw water				
Flow (ML/d)*	132.3	118.4 – 136.9	18.4	SCADA
pH	8.06	8.04 – 8.09	0.05	SCADA
Temperature (°C)	5.9	5.8 – 6.0	0.2	SCADA
Turbidity (NTU)	11.8	7.02 – 24.9	17.9	SCADA
Total organic carbon (mg/L)	2.3			Grab sample
Bromide (ppb or ng/L)	< 50			Grab sample
Conductivity (uS/cm)	324			Grab sample
Hardness (mg/L as $CaCO_3$)	105			Grab sample
Alkalinity (mg/L as $CaCO_3$)	95			Grab sample
Total dissolved solids (mg/L)	211			Grab sample
MIB (ng/L)	< 3			Grab sample
Geosmin (ng/L)	< 3			Grab sample
Actiflo settling				
pH (Actiflo 1)	7.98	7.87 – 8.09	0.22	SCADA
pH (Actiflo 2)	7.66	7.58 – 7.76	0.17	SCADA
Discharge turbidity (NTU) (Actiflo 1)	0.55	0.48 – 0.65	0.17	SCADA
Discharge turbidity (NTU) (Actiflo 2)	0.59	0.49 – 0.72	0.13	SCADA
Alkalinity (mg/L as $CaCO_3$)	93			Grab sample
Aluminium (total; dissolved) (mg/L)	0.23; 0.10			Grab sample
Colour (pcu)	< 3			Grab sample
Total dissolved solids (mg/L)	209			Grab sample
Conductivity (uS/cm)	331			Grab sample
Total organic carbon (mg/L)	< 0.2			Grab sample

Operating Conditions: Microsand dosage = 2.5 g/L; PACL=11 mg/L; Dry Polymer dosage = 0.12 mg/L
pcu = platinum-cobalt units
*Measured by the sum of Actiflo 1 and 2 inlet flows.

As seen from Table 3-5, average and highest raw water turbidity values were 11.8 NTU and 24.9 NTU respectively.[21] The raw water had low total organic carbon (2.3 mg/L), low bromide (50 ppb), moderate alkalinity (95 mg/L as $CaCO_3$), high pH (8.06) and low MIB and Geosmin (3 ng/l each).

[21] During the ramp up period, raw water turbidity had an ultimate peak of 27.4 NTU. The minimum raw water turbidity during the test day (outside of the test and ramp up periods) was 0.2 NTU.

Both Actiflo units maintained low effluent turbidities (97% removal; < 1 NTU) and removed total organic carbon by 91% indicating that the units performed very well in removing turbidity and total organic carbon during influent flows of 132 ML/d on average during the test period.

3.5.3 Ozone system

A summary of the parameters monitored by SCADA are presented in Table 3-6, **Error! Reference source not found.**, and Figure 3-8 below. Overall, the ozone system performed well with minor issues during the full scale test.

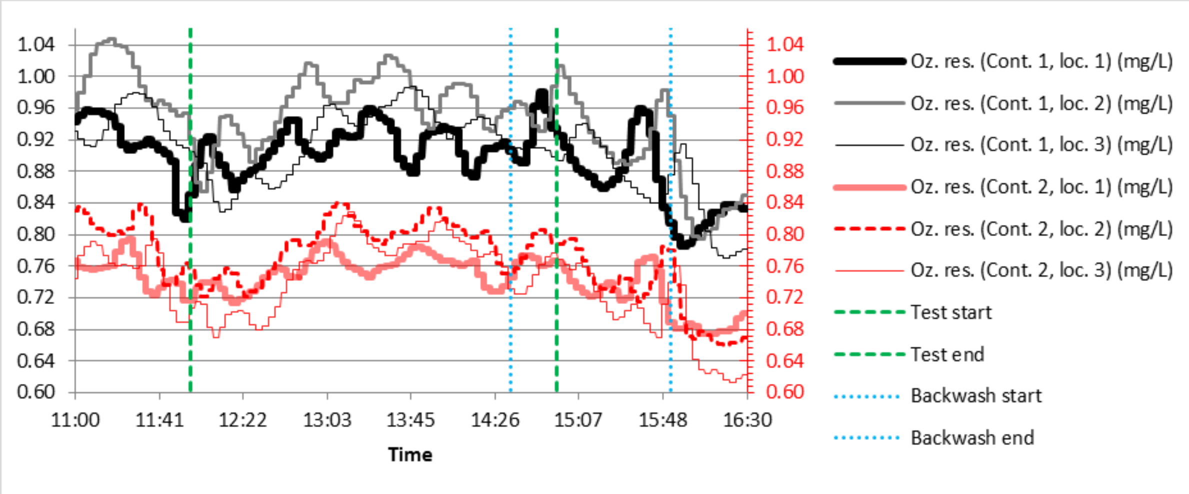
As can be seen from the table and figures below, the dissolved ozone residuals were 0.1-0.2 mg/L higher in Contactor 1 on average.

Table 3-6 Ozone system parameter statistics

Parameter	Mean	Min - Max	Range
Pre-ozone			
pH*/**	7.82	7.73 – 7.92	7.92
Temperature (°C)*	5.9	5.8 – 5.97	5.97
Alkalinity (mg/L as $CaCO_3$)**	93		
Total organic carbon (mg/L)**	<0.2		
Bromide (ppb)**	50		
Dissolved ozone residuals			
Dissolved ozone residual (Contactor 1, location 1) (mg/L)*	0.91	0.85 - 0.98	0.13
Dissolved ozone residual (Contactor 1, location 2) (mg/L)*	0.96	0.86 - 1.03	0.17
Dissolved ozone residual (Contactor 1, location 3) (mg/L)*	0.92	0.83 - 0.99	0.16
Dissolved ozone residual (Contactor 2, location 1) (mg/L)*	0.76	0.71 - 0.79	0.08
Dissolved ozone residual (Contactor 2, location 2) (mg/L)*	0.79	0.72 - 0.84	0.12
Dissolved ozone residual (Contactor 2, location 3) (mg/L)*	0.76	0.67 - 0.83	0.16
Log inactivation			
Crypto log inactivation (Contactor 1)*	0.95	0.84 - 1.13	0.30
Crypto log inactivation (Contactor 2)*	0.79	0.71 - 0.91	0.20
Giardia log inactivation (Contactor 1)*	7.72	7.24 - 8.42	1.18
Giardia log inactivation (Contactor 2)*	7.03	6.67 - 7.56	0.89
Virus log inactivation (Contactor 1)*	10.28	9.76 - 11.03	1.27
Virus log inactivation (Contactor 2)*	9.53	9.13 - 10.11	0.98
Post-Ozone			
Bromate (ppb)**	3.0	< 3.0 – 3.0	up to 3.0

Note: Values in this table are based on the test duration time as indicated in Section 3.3.
Data sources: *SCADA data; **Laboratory results from grab samples.

Figure 3-8 Dissolved ozone residual levels



The following observations were made during the test (discussion of these observations is provided in Section 4):

- Headspace in contactors: minimum requirement of 300 mm was met at all times during the test period, however during the ramp up period a few ozone offgas pressure HIGH and HIGH HIGH alarms for both contactors were triggered (setpoints of -40 mm H2O and -20 mm H2O, respectively).
- Ozone generators were able to produce the required ozone dose at the high flows (143 MLD) as they have been designed for a maximum flow of 160 MLD.
- Prior to the full scale test, the offset in the ozone levels between contactors was less pronounced than shown in Figure 3-8.

The treatment objectives for the ozonation system are summarized in the following table.

Table 3-7 Treatment objectives for the ozonation system

Parameter	Value	Reference standard
Taste and Odour	No telephone complaints	Internal standard set by Region;
Bromate	10 ppb	USEPA D/DBPR**
Primary disinfection		
Giardia	3.0 Log by removal/inactivation (0.5 Log inactivation by ozone)	MOECC
Viruses	4.0 Log removal/inactivation (2.0 Log inactivation by ozone)	MOECC
Cryptosporidium*	2.0 Log removal/inactivation (throughout the year)*	MOECC and 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 2.0 log inactivation by ozone in the summer months.
** US Environmental Protection Agency's Disinfectants/Disinfection Byproducts Rule.

The desired outcome for the ozone system was to maintain applicable concentration time (CT) requirements for pathogen inactivation (e.g. protozoa, bacteria, and viruses; particular protozoans of concern are Giardia lamblia and Cryptosporidium parvum) at the gross 143 ML/d flowrate through the ozone contactors. The design criteria for the ozone system is based upon providing a 0.5-log inactivation of Giardia, 2.0 log inactivation of viruses (both are regulatory requirements) and a 1-log (cold water) / 2-log (warm water) inactivation of Cryptosporidium (internal goal set by the Region). CT refers to the concentration of disinfectant residual (C) times the duration of disinfectant contact time (T).

Additional performance criteria consisted of keeping bromate formation below the MCL of 10 ppb and maintaining flow, within the ozone contactors, between the minimum and maximum water levels. Based on the Region's existing records, the minimum operating depth is 4.2 m, and the maximum operating elevation is 85.700 m. Assuming an invert elevation of 81.015 m as per Phase 1 as-built drawings, the maximum operating depth is 4.685 m.

The water quality parameters and their measured values during the testing are listed in Table 3-6. Water quality criteria that influence ozone demand and decay and hence the CT value include pH, temperature, alkalinity, organic parameters (e.g. total organic carbon), inorganic parameters (e.g. iron, manganese), bromide, taste and odour compounds. As seen from Table 3-5 and Table 3-6 the total organic carbon, bromide and taste and odour compounds are of low concentrations and hence pH and temperature are the most influential parameters that impact the CT value.

Ozone dosage and contact time were controlled through SCADA and CT was determined using the completely stirred tank reactor (CSTR) method which is incorporated in the SCADA system and is applicable for compartmentalized ozone contactors such as over-under or serpentine baffled basins. k* is the mathematical decay rate for ozone residual in water. This value is used to calculate the CT credit which is used to determine the CT log inactivation for *Cryptosporidium*, CT log inactivation for *Giardia*, and CT log inactivation for virus.

The Oakville WPP ozone system has three ozone residual analyzers installed in each one of the two ozone contactors. The first, second, and third ozone analyzers measure the ozone residuals from cells 1, 5, and 9, respectively. Table 3-6 shows the ozone residual values for both contactors 1 and 2, which exhibited very little ozone decay and can be attributed to less ozone demanding substances in the ozone influent. Historically, ozone demand varies between 0.5-1.0 mg/L in these waters with typical values of 1.0 mg/L.

As seen from Table 3-6 log inactivation of *Giardia* and virus exceeded the regulatory requirements. Contactor 1 exhibited better *Cryptosporidium* inactivation (0.8-1.0 Log) than Contactor 2 (0.71-0.85 Log). This can be attributed to the lower ozone residuals obtained in Contactor 2. Ozone dosing was set to MANUAL during the test, which is the reason for the required log inactivation not being achieved. (Normally, the plant's programmable logic controller (PLC) is programmed to increase the ozone dosage control rate automatically at auto-trim time intervals to achieve the desired CT credit).

Performance ratio (PR) is defined as the measured disinfection credit divided by the target disinfection credit and the goal is to maintain a PR> 1.0. This goal was not met during the full scale test. A PR < 1 means that the required performance has not been achieved and the ozone dose must be increased. Based on CT calculations, an ozone residual of ~1.12 mg/L would have met the PR goals.

3.5.4 Ozonated water conduits (conveyance from ozone to filters)

A summary of the parameters monitored by SCADA are presented in Table 3-8 and Figure 3-9. The ozonated water conduits performed well, without major issue, during the full scale test. It should be noted that this area represents a key hydraulic bottleneck and, as will be discussed, performed better than anticipated.

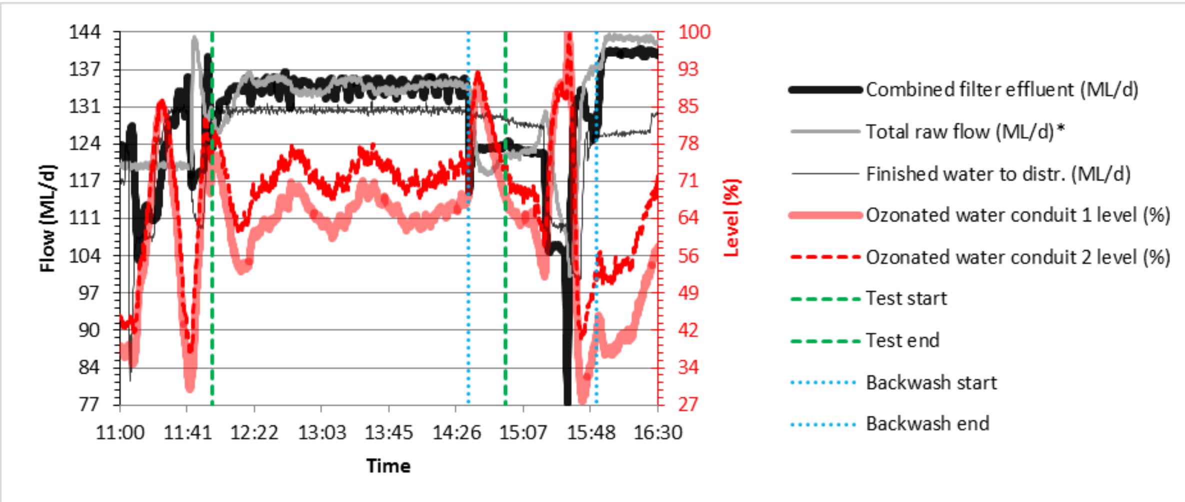
Table 3-8 Ozonated water conduit SCADA parameter statistics

SCADA parameter	Mean	Min - Max	Range
Ozonated water conduit 1 level (%)	66.4	53 - 89	36
Ozonated water conduit 2 level (%)	73.1	61 - 93	32

Note: Values in this table are based on the test duration time as indicated in Section 3.3.

The tops of the conduits are not sealed, and thus must operate below 100% capacity in order to avoid overflowing. The level transmitters for these conduits control the low lift pumps flowrate, and operations has determined that 60% channel level is the optimum value for controlling flow. The net result is that at high flows, or when attempting to increase flows, these channels surge which will then decrease LLP flows until the conduit levels subside to below 60%. At present the LOW LOW, LOW, and HIGH level alarms are set to 10%, 20%, and 85% respectively. A level of 100% represents the channel ceiling (not sealed).

Figure 3-9 Ozonated water conduit levels with plant flows



*Measured by sum of Actiflo 1 and 2 inlet flows.

As can be seen in Figure 3-9, the ozonated water conduits remained between 55% and 80% during the Test Period, with two exceptions. First, the surge in level to 89% and 93% in conduits 1 and 2, respectively, occurred when the backwash event was initiated and the master filter rate interlock at 1400 L/s was then initiated (thus decreasing filter flows), resulting in the brief level surge. As discussed previously, this cap was overlooked when adjusting the SCADA interlocks. The surge would not occur if the total filter rate is set to match to the low lift pumping maximum flow value (however sudden changes in raw water flow should not be directly translated to the filters to avoid turbidity spikes). Second, the channels surged momentarily to 100% near the conclusion of the filter backwash due to momentary flow rate adjustments to the filters (understood as normal during flow adjustments and seen only as a minor operational issue to be noted for future operation). During the test period, ozonated conduits 1 and 2 had mean average levels of 66% and 73%, respectively, and maximum values of 89% and 93%, respectively. With these levels, the ozonated conduit LITs were still able to request sufficient flows from the low lift pumps.

3.5.5 Filters

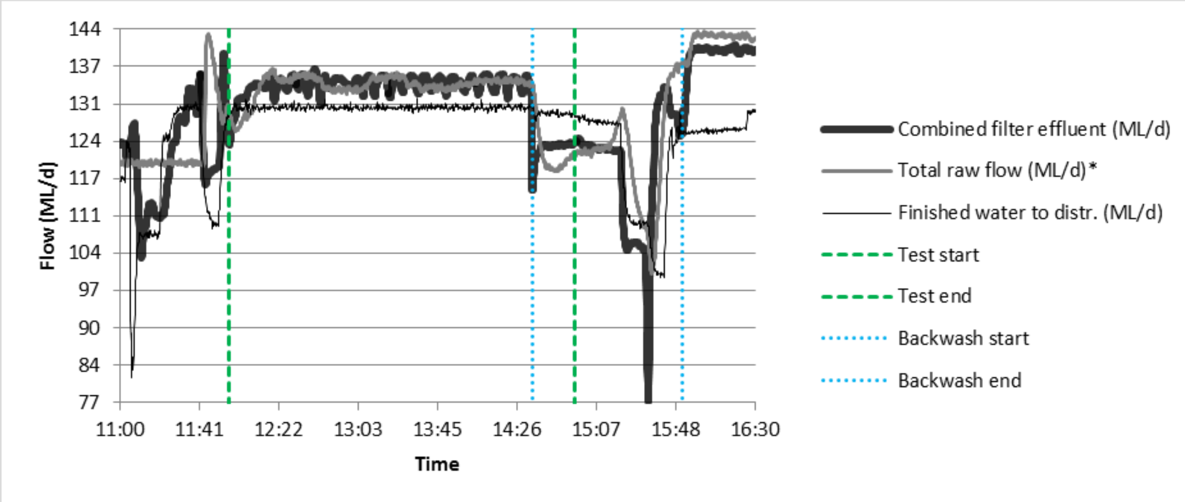
A summary of the parameters monitored by SCADA are presented in Table 3-9, Figure 3-10, and Figure 3-11 below. The filtration system presented challenges during the full scale test, both during the identified Test Period and specifically during a filter backwash event.

Table 3-9 Filter system SCADA parameter statistics

SCADA parameter	Mean	Min - Max	Range
Headloss			
Filter 1 headloss (m)	0.99	0.85 - 1.07	0.21
Filter 2 headloss (m)	1.07	0.96 - 1.14	0.19
Filter 3 headloss (m)	1.07	0.91 - 1.15	0.24
Filter 4 headloss (m)	0.92	0.80 - 1.00	0.20
Filter 5 headloss (m)	0.99	0.87 - 1.08	0.21
Filter 6 headloss (m)*	0.99	0.00 - 1.11	1.11
Filter 7 headloss (m)	1.06	0.98 - 1.14	0.16
Filter 8 headloss (m)	1.00	0.89 - 1.08	0.19
Effluent flow**			
Filter 1 effluent flow (L/s)	195.2	178.1 - 204.8	26.7
Filter 2 effluent flow (L/s)	195.0	179.4 - 206.2	26.8
Filter 3 effluent flow (L/s)	195.3	170.2 - 206.6	36.4
Filter 4 effluent flow (L/s)	195.1	173.9 - 207.8	33.9
Filter 5 effluent flow (L/s)	195.1	178.1 - 205.9	27.8
Filter 6 effluent flow (L/s)*	169.6	0.0* - 199.8	199.8
Filter 7 effluent flow (L/s)	195.0	185.0 - 204.4	19.4
Filter 8 effluent flow (L/s)	194.8	180.6 - 205.2	24.6
Combined (L/s)	*1535*	*1333 - 1585*	*252*
Combined (ML/d)	*132.6*	*115.1 - 136.9*	*21.8*
Effluent valve (EV) position (% open)			
Filter 1 EV position (%)	57.5	54.2 - 58.6	4.4
Filter 2 EV position (%)	64.4	60.7 - 65.9	5.2
Filter 3 EV position (%)	51.0	47.5 - 52.2	4.7
Filter 4 EV position (%)	49.5	43.2 - 54.5	11.3
Filter 5 EV position (%)	52.0	48.6 - 53.2	4.6
Filter 6 EV position (%)*	50.1	0.00 - 58.4	58.4
Filter 7 EV position (%)	56.4	48.5 - 57.8	9.3
Filter 8 EV position (%)	56.4	50.1 - 59.0	8.9
Turbidity			
Filter 1 turbidity (NTU)	0.05	0.05 - 0.06	0.02
Filter 2 turbidity (NTU)	0.04	0.04 - 0.05	0.01
Filter 3 turbidity (NTU)	0.06	0.06 - 0.07	0.01
Filter 4 turbidity (NTU)	0.07	0.06 - 0.07	0.01
Filter 5 turbidity (NTU)	0.05	0.04 - 0.10	0.06
Filter 6 turbidity (NTU)*	0.05	0.00 - 0.07	0.07
Filter 7 turbidity (NTU)	0.06	0.06 - 0.07	0.01
Filter 8 turbidity (NTU)	0.06	0.05 - 0.06	0.01
Combined (NTU)	*0.047*	*0.04 - 0.05*	*0.01*

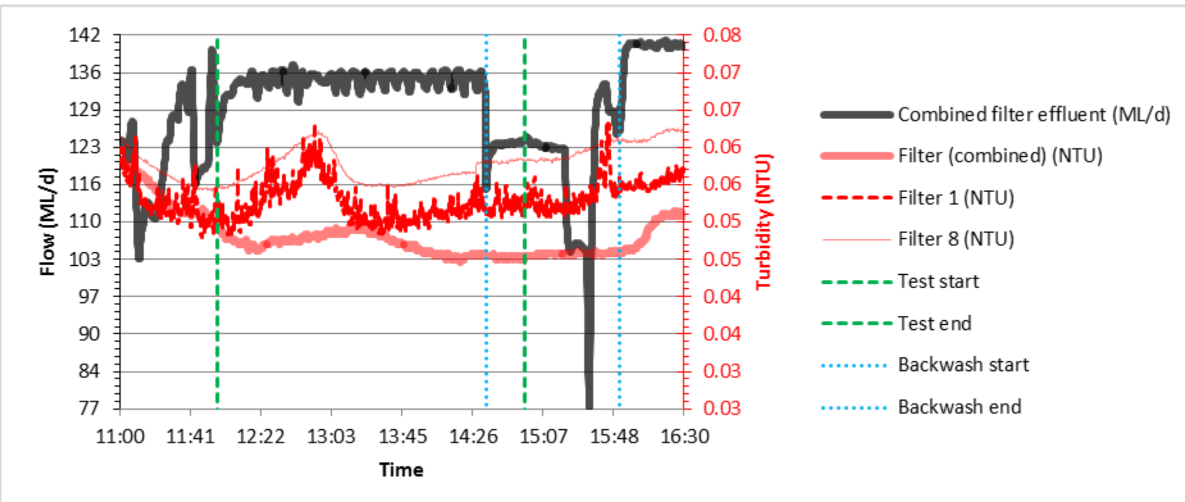
*Filter 6 backwash procedure commenced during the Test Period.
**Mean average effluent flows, reported here in L/s, ranged from 10.5-12.8 m/h.
Note: Values in this table are based on the Test Period time as indicated in Section 3.3.

Figure 3-10 Comparison of filtration and WPP flows



*Measured by sum of Actiflo 1 and 2 inlet flows.

Figure 3-11 Combined filtration flow and Filter 1, 8, and combined turbidity



Observations during the full scale test:

- Between approximately the time period 15:06-15:20 h, Reservoir cell 2 level drops below 60% (LOW alarm; LOW LOW alarm is 51%) while Filter 6 backwash cycle was in stratification phase. Clearwell levels were approximately 82% (operations staff noted normal levels are 90% which are to be confirmed during detailed design). Filter rates (200-204 L/s) did not increase to fill clearwell and reservoir levels. (It was later determined that the Master Filter Rate was set to 1400 L/s (~200 L/s with seven filters online), and that this is likely the reason why the filter rates were not increasing to fill Clearwell and Reservoir levels.) Filter biasing[22] would normally commence to reduce HLP flow.

- Water does not flow into Filter 1 sufficiently at high flows, to maintain its filter rate, without draining. Presently this is controlled by reducing Filter 1's filtration rate. Field observations suggest that this is due to Filter 1's position along the filter inlet channel and configuration of the inlet port relative to the trough structures within.
- Operator noted that the Master Filter Rate (L/s) should equal Maximum Low Lift Flowrate (ML/d). For future operation, set the Master Filter Rate to the max low lift flow to avoid this scenario.
- Water surface in filters are turbulent, especially in Filters 1, 2 and 8. The turbulence successively decreases downstream of Filter 1 and Filter 8. Filter 5 appeared more turbulent than Filter 4 likely because one of the four clarified water feeds is located near Filter 5.

3.5.6 Clearwell and Reservoir

A summary of the parameters monitored by SCADA are presented in Table 3-10 and Figure 3-12 below. Additional figures are provided in Section 3.4.

The Clearwell performed well with a minor challenge during a filter backwash (reduced levels). The Reservoir presented a major challenge during a filter backwash at the end of the Test Period.

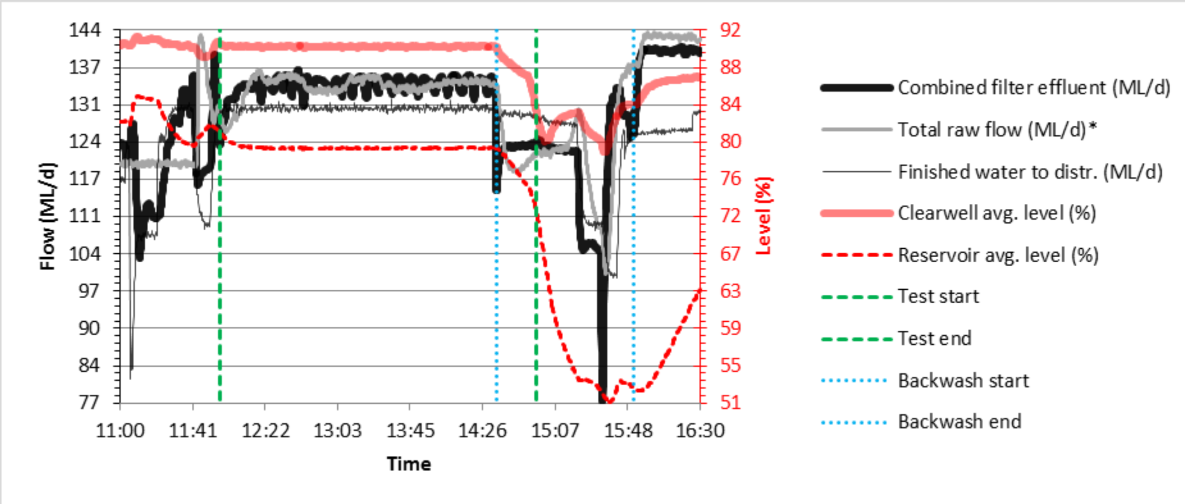
Table 3-10 Clearwell and Reservoir SCADA parameter statistics

SCADA parameter	Mean	Min - Max	Range
Clearwell			
Clearwell cell 1 level (%)	89.9	83.5 - 90.6	7.0
Clearwell cell 2 level (%)	89.9	82.9 - 90.5	7.6
Average (%)	*89.9*	*83.2 - 90.5*	*7.3*
Average (m)	*2.52*	*2.33 - 2.54*	*0.21*
Average (elevation, m)	*79.34*	*79.15 - 79.36*	*0.21*
Reservoir			
Reservoir cell 1 level (%)	79.9	73.3 - 82.0	8.7
Reservoir cell 2 level (%)	77.6	71.2 - 79.7	8.5
Average (%)	*78.8*	*72.3 - 80.8*	*8.6*
Average (m)	*2.20*	*2.02 - 2.26*	*0.24*
Average (elevation, m)	*79.24*	*79.06 - 79.30*	*0.24*

Note: Values in this table are based on the Test Period time as indicated in Section 3.3.

[22] "Filter bias mode" is a SCADA interlock that ensures that the high lift pump flows are kept equal to or less than the combined filter effluent flow. When on, the high lift pump flows will reduce below their target setpoint as dictated by reservoir levels.

Figure 3-12 Clearwell and Reservoir levels (%)



*Measured by sum of Actiflo 1 and 2 inlet flows.

Observations made during the full scale test:

- The reservoir's two cells were operating in series (as opposed to parallel). Region operations has confirmed this is typical procedure for the reservoir.
- During the filter backwash, the clearwell levels dropped from 89% to 80%, while the reservoir levels dropped from approximately 79% to 50%, while the high lift pumps continued to produce 130 ML/d. See discussion and figures in Section 3.4 for full details and Section 4.1.9 for further analysis.

3.5.7 High lift pumping station

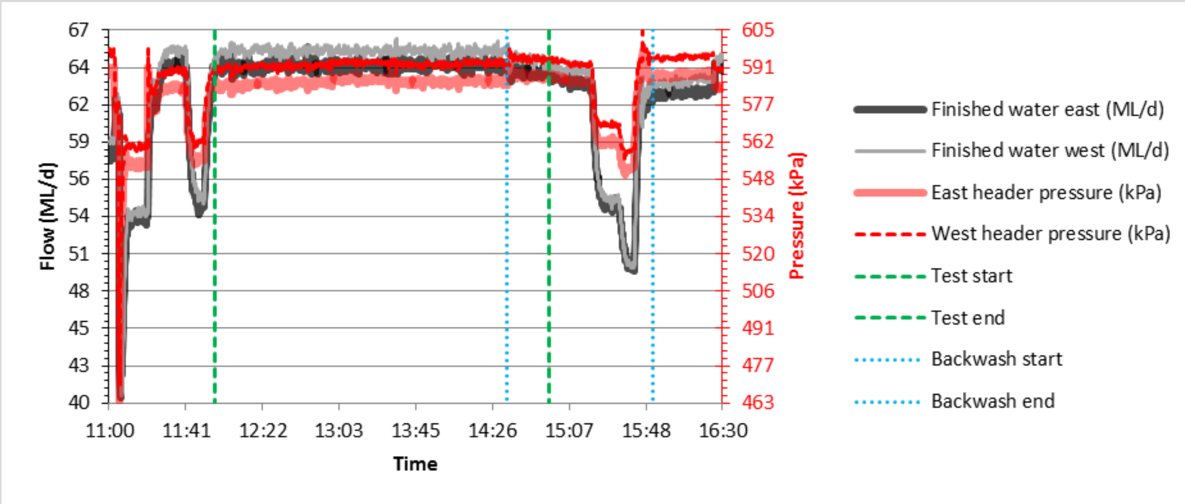
A summary of the parameters monitored by SCADA are presented in Table 3-11 and Figure 3-13 below. The high lift pumps performed well without any major issues during the full scale test.

Table 3-11 High lift pumping station SCADA parameter statistics

SCADA parameter	Mean	Min - Max	Range
High lift pump speeds			
HLP1 speed (%)	58.4	57.2 - 58.7	1.5
HLP2 speed (%)	56.7	55.3 - 56.9	1.5
HLP3 speed (%)	58.1	56.0 - 58.5	2.5
Finished water production			
To distribution – East header (ML/d)	64.4	63.3 - 65.8	2.5
To distribution – West header (ML/d)	65.4	63.9 - 66.4	2.5
To WPP – Service water (ML/d)	1.7	1.6 - 1.8	0.2
To distribution (East + West) (ML/d)	*129.8*	*127.5 - 131.7*	*4.2*
Pressure			
East header pressure (kPa)	585	580 - 591	11
West header pressure (kPa)	592	587 - 598	11

Note: Values in this table are based on the Test Period time as indicated in Section 3.3.

Figure 3-13 Finished water production flows and header pressures



Observations made during the test:

- As noted in Section 3.4 above, interlocks on the low lift pump maximum flow (interlocked at 120 ML/d) and HLP 3 maximum flow were discovered by Region operators at the end of the ramp up period when attempting to reach the targeted net production rate, which subsequently delayed the beginning of the three-hour Test Period.
- When all three HLPs were operating at 130 ML/d, vibrations were noted in the HLP building that are not typically noticed (i.e. could be felt while standing on the catwalks) and should be investigated further.
- Maximum speed of HLPs was set to 60% (of 51-60 Hz). This was optimized when operators were trying to get the HLPs to produce 130 ML/d.
- In the distribution system, the Davis Road Zone O1 valve[23] had to be throttled to 49% in order to reduce distribution system pressures. Since pumping rates are influenced by upstream system pressures, this permitted the HLPS to produce the targeted 130 ML/d net production rate.

3.6 Chemical systems

A summary of the parameters monitored by, and estimated using data from, the SCADA system are presented in the following table. The chemical systems performed without issue during the full scale test. Chemical systems not required / not operational during the test include filter aid polymer (Type 2); Alum; and Hydrogen peroxide.

All chemical feed rates were well below the chemical feed system capacities (refer to GHD's Technical Baseline Review Memo #3, Dec. 2014). Note that Calcium thiosulphate metering pumps 2 and 3 approached their capacity of 45.4 L/hr, with the suggestion that pump 2 slightly exceeded capacity. Note also that the metering pump flow rates are calculated using flow measured at the point nearest to the injection point and not at the injection point itself; this may explain the exceedance.

[23] This valve opens and closes based on the level of the McCraney Reservoir. The Davis Road Pumping Station inlet is between the Davis Road Zone O1 bypass valve and the McCraney Reservoir, and the Zone O1 bypass valve functions as a mixing valve between water from the Oakville WPP and McCraney Reservoir.

Table 3-12 SCADA data statistics for chemical equipment

SCADA parameter	Mean	Min – Max	Range
Flows			
Chlorinator 1 (kg/hr) (Reservoir outlet / trim)	0.014	0.014 - 0.015	0.01
Chlorinator 2 (kg/hr) (Clearwell outlet)	8.8	7.8 - 9.0	1.2
Chlorinator 3 (kg/hr) (Intake / zebra mussel control)	3.8	3.0 - 5.1	2.1
Calcium thiosulphate pump 1 (L/hr) (Raw water well)[a]	8.7	6.0 – 12.6	6.6
Calcium thiosulphate pump 2 (L/hr) (Contactor 1)[b]	42.7	36.9 – 49.6	12.7
Calcium thiosulphate pump 3 (L/hr) (Contactor 2)[c]	35.4	29.9 – 41.4	11.5
Dry polymer Type 1 pump 1 (L/hr)[d]	324	288 – 338	50
Dry polymer Type 1 pump 3 (L/hr)[e]	337	298 – 354	56
Fluoride pump 2 (L/hr)[f]	13.1	0.0 – 13.6	13.6
Dosages			
Chlorinator 1 (mg/L) (Reservoir outlet / trim)[g]	0.003	0.002 – 0.003	0.001
Chlorinator 2 (mg/L) (Clearwell outlet)[h]	1.6	1.4 – 1.7	0.3
Chlorinator 3 (mg/L) (Intake / zebra mussel control)[i]	0.70	0.55 – 0.97	0.42
Calcium thiosulphate pump 1 (mg/L) (Raw water well)	0.48	0.33 – 0.68	0.35
Calcium thiosulphate pump 2 (mg/L) (Contactor 1)	4.7	4.2 – 5.4	1.2
Calcium thiosulphate pump 3 (mg/L) (Contactor 2)	3.9	3.4 – 4.4	1.0
Dry polymer Type 1 pump 1 (mg/L)	0.115	0.112 – 0.120	0.005
Dry polymer Type 1 pump 3 (mg/L)	0.122	0.119 – 0.125	0.006
Fluoride pump 2 (mg/L)	0.69	0.00 – 0.71	0.71

Note: Values in this table are based on the Test Period time as indicated in Section 3.3. Unless otherwise noted, values were estimated using raw SCADA data.
[a] Calculated using Calcium thiosulphate pump 1 dosage, raw water flow rate (as measured at the Actiflo inlet), and chemical solution parameters (24% solution, specific gravity of 1.26).
[b] Calculated using Calcium thiosulphate pump 2 dosage, Actiflo 1 inlet flow rate, and chemical solution parameters (24% solution, specific gravity of 1.26).
[c] Calculated using Calcium thiosulphate pump 3 dosage, Actiflo 2 inlet flow rate, and chemical solution parameters (24% solution, specific gravity of 1.26).
[d] Calculated using dry polymer pump 1 dosage, total Actiflo flow rate, 0.2% solution, and specific gravity of 1.0.
[e] Calculated using dry polymer pump 3 dosage, total Actiflo flow rate, 0.2% solution, and specific gravity of 1.0.
[f] Calculated using Fluoride pump 2 dosage, high lift pumping station flow (production + service), and chemical solution properties (24% solution, specific gravity of 1.20).
[g] Calculated using chlorinator 1 flow rate and high lift pumping station flow (production + service).
[h] Calculated using chlorinator 2 flow rate and high lift pumping station flow (production + service).
[i] Calculated using chlorinator 3 flow rate and raw water flow (i.e. Actiflo flow rate).

3.7 Distribution system

The Region installed pressure loggers in the distribution system to monitor pressure. Monitoring locations were determined based on hydraulic modelling performed by the Region. The following table summarizes the data as measured by the pressure loggers.

Table 3-13 Select pressures within distribution system during full scale test

Location (Region ID)	Mean (kPa)	Min - Max (kPa)	Range (kPa)
Claremont Cres. (WHY7959)	359	248 - 435	188
Speers Rd. and Third Line (WHY10542)	304	221 - 350	129
Fourth Line and Maple Grove Dr. (WHY8708)	348	242 - 394	152
Allan St. and Galt Ave. (WHY7751)	368	298 - 417	119
Rebecca St., East of Bronte Rd. (WHY2458)	428	356 - 477	121
1151 Bronte Rd. (WHY7021)	75	37 - 116	78
Rebecca St. and Maurice Dr. (WHY8221)	459	353 - 500	147
Lakeshore Rd. E. and Winston Churchill Blvd. (WHY7813)	499	375 - 562	187
Devon Rd. and Maple Grove Dr. (WHY43467)	350	255 - 405	150
4257 New St. (WHY9995)	409	341 - 485	144

Data source: Region installed pressure-loggers, trending at 2 second intervals. Statistics represent readings from 9:00 AM to 9:00 PM (pre-, during, and post- full scale test).

Observations from full scale test:

- The Region reported no issues. At the monitored locations, the maximum pressure observed was 562 kPa at Lakeshore Road East and Winston Churchill Blvd; the remaining locations had maximum pressures at or below 500 kPa. These pressures are well below the recommended maximum of 700 kPa.
- Region operations did not have any issues managing flows from the plant. This was the result of a well-coordinated effort between plant and distribution operations as well as the proactive planning that was done by the Region.
- It is noted that Region operations began lowering selected distribution reservoirs near the Oakville WPP two days in advance of the test.
- Net production flow to the distribution system will fluctuate depending on distribution system pressure. Local distribution system pressure near the WPP can be lowered by throttling the Davis Road Zone O1 bypass valve, which was throttled to 49% during the full scale test. (Previously noted in Section 3.5.7 High lift pumping station; see Footnote 23 for more information on the Zone O1 bypass valve.)

3.8 Water quality parameters

The following table summarizes the water quality parameters from the grab samples taken during the full scale test. Overall, there were no reported issues with respect to disinfection byproducts, and we note one area for further investigation (turbidity and TOC in the filter effluent). It is further noted that lake (i.e. raw water) conditions were stable and that raw water turbidity was less than 1 NTU prior to flow increases and subsequent intake scouring. Refer to Section 4 for a detailed discussion.

Table 3-14 Water quality parameters from grab samples collected during the full scale test

Location	1-Raw (pre CL)	2-Actiflo	3-Ozone (loc. 1)	3-Ozone (loc. 2)	3-Ozone (loc. 3)	4-Filtered	5-Finished water		
Sample time	8:05	14:00	14:30	14:30	14:30	13:45	14:25	14:55	15:30
Aluminum (dissolved) [mg/L]		0.1				< 0.1			
Aluminum (total) [mg/L]		0.23				< 0.1			
BIO - E. coli [cfu/100 mL]							0		
BIO - Total coliform [cfu/100 mL]							0		
Bromate [mg/L]							< 0.003	0.003	0.003
Bromide [mg/L]	< 0.05								
Alkalinity [mg/L as CaCO3]	95	93				96	85		
Colour [TCU]	< 3	< 3				4	< 3		
Conductivity [uS/cm]	324	331				332			
Hardness [mg/L as CaCO3]	105								
pH		7.96	7.63	7.63	7.62		7.89		
Total dissolved solids [mg/L]	211	209				194			
Turbidity [NTU]						0.41			
HAA - Bromoacetic acid [ug/L]	< 2.9						< 2.9		
HAA - Bromochloroacetic acid [ug/L]	< 2.0						< 2.0		
HAA - Chloroacetic acid [ug/L]	< 4.7						< 4.7		
HAA - Dibromoacetic acid [ug/L]	< 2.0						< 2.0		
HAA - Dichloroacetic acid [ug/L]	< 2.6						< 2.6		
HAA - Haloacetic acids [ug/L]	< 5.3						< 5.3		
HAA - Trichloroacetic acid [ug/L]	< 5.3						< 5.3		
T&O - Geosmin [ng/L]	< 3						< 3		
T&O - MIB [ng/L]	< 3						< 3		
THM - Bromodichloromethane [ug/L]	< 0.26						1.4		
THM - Bromoform [ug/L]	< 0.34						< 0.34		
THM - Chloroform [ug/L]	< 0.29						2.2		
THM - Dibromochloromethane [ug/L]	< 0.37						0.86		
THM - Trihalomethanes (total) [ug/L]	< 0.37						4.5		
Total organic carbon [mg/L]	2.3	< 0.2				2.8	1.7		

Values noted as less than ("<") denote that the parameter was below the minimum detectable limit (MDL).

4. Discussion

The following discussion summarizes the findings from the desktop and background studies with the results of the full scale hydraulic and process assessment test that was conducted on May 5, 2015. The discussion will evaluate the WPP's systems with respect to the following parameters:

- Rated capacity (all units online) vs. firm capacity (largest unit out of service), as defined at the beginning of Section 2;
- Operability;
- Plant's ability to address the Class EA study's problem statements; and
- Water quality (especially pertaining to pathogen log inactivation and disinfection byproducts,).

4.1 Plant processes

The following is a summary of the constraints and/or challenges to meet the target production capacity of 130 ML/d (143 ML/d gross) incorporating findings from both the desktop and field assessments.

4.1.1 Intake

While the Actiflo® system is designed to process highly turbid water, it is challenged by rapid changes in raw water quality. In such instances, operators have to anticipate and respond quickly to adjust processes. The 50 mm dia. raw water sampling line at the intake crib has a tendency to clog and has no redundancy; this line is intended for raw water sampling and turbidity monitoring at the intake crib. Consideration to twinning this line and installing a backflushing system, in order to improve turbidity monitoring at the intake crib to provide operators additional time to anticipate and respond appropriately. (Recommendation #12)

4.1.2 Traveling water screens

(1) No challenges or constraints for meeting the target re-rated production capacity were noted in the background studies, or were encountered during the full scale test. It is noted that the test was completed with both screens in service.

(2) The one scenario that may pose as a potential challenge is during low lake levels, at the maximum demand of 143 ML/d, and when one screen is out of service (e.g. for maintenance). The manufacturer has verified that each screen operating alone can operate within its design parameters under these conditions, but noted that under this scenario the screen would need to be as clean as possible (i.e. near 100% clean), and furthermore that these conditions could result in faster debris accumulation. As noted in Section 2.1.2, this is not considered a normal operating issue since the screens are normally operated in parallel and only taken offline for maintenance.

The likelihood of occurrence for this scenario (low lake levels, one screen out of service, and 143 ML/d raw flow, requiring online screen to be 100% clean) could be further reduced if maintenance for the water screens is only scheduled at times other than peak water demand season (e.g. fall, winter, early spring). In the event that one screen needs to be taken offline during peak water demand season, operations can temporarily reduce the maximum production of Oakville WPP to its present rated capacity (109 ML/d net) and supplement supply with other sources (e.g. Burlington

WPP, Burloak WPP, or supply from neighbouring municipalities). This operational recommendation is made because a 100% clean screen should not relied upon. (Recommendation #1)

(3) Issues with algae build-up on the screens have been reported. Presently, the screens are flushed with non-chlorinated water automatically on a timed cycle (presently, 4 hours). Intermittent flushing with chlorinated water may help alleviate algae build-up, but would require dechlorination prior to disposal to Lake Ontario. Also it is possible that chlorine flushing would only kill the algae and not solve the build-up problem. Issue is recommended for further consideration.

Alone this does not present a challenge for an increase in raw water taking to a maximum of 143 ML/d, as both screens are normally operated in parallel. (Recommendation #1 and 18)

4.1.3 Low lift pumps

No challenges or constraints for meeting the target re-rated production capacity were noted in the background studies, or were encountered during the full scale test.

4.1.4 Inline mixer

The only constraint to note for meeting the target re-rated production capacity for the in-line mixer and PACL injection is that there is only one mixer and therefore no redundancy built into the PACL system; it is noted that the Alum system is available as a contingency. It is further noted that current research suggests that rapid mixing is not critical to coagulation performance; rather it is only important that the coagulant is dispersed (Edzwald, 2014).

4.1.5 Actiflo® system

(1) In the past, full-scale testing was conducted at various flow rates with one Actiflo®, ozone, and filter train in service: 44 ML/d, 62 ML/d, and 85 ML/d. At flow of 85 ML/d, it was observed that the Actiflo® maturation tank mixer became unsteady. During the full scale test on May 5, 2015, this issue was not observed, and no other issues related to the maturation tank mixer were noted. Region operations should monitor this issue during high flow conditions and, if necessary, conduct further investigation. (Recommendation #10)

(2) As the recirculation pumps have a fixed capacity, operating at higher flow rates could impact the Actiflo® performance in terms of effluent water quality (i.e. increased floc carryover). Furthermore, the gaps between lamella tube sections may exacerbate floc carryover. During the full scale test, no issues related to the recirculation pumps or floc carryover were noted. Region operations should monitor the Actiflo® effluent for floc carryover during high flow conditions. (Recommendation #10)

(3) As seen from Table 3-4 in Section 3.5.2 average and maximum raw water turbidity values were 11.6 NTU and 24.9 NTU respectively. The raw water had low total organic carbon (TOC) concentrations (2.3 mg/L), low bromide (50 ppb), moderate alkalinity, high pH and low taste and odour compounds MIB and Geosmin (3 ng/l each). Both Actiflo units maintained low effluent turbidities (97% removal; < 1 NTU) and removed TOC by 91% indicating that the units performed very well in removing turbidity and TOC during influent flows of 143 ML/d.

(4) Both Microsand and PACL dosages were increased during the full scale test. Increasing microsand, coagulant, or polymer dosages could increase carryover of sand, polymer and floc, thereby impacting filter performance. However, this is not foreseen to be an issue at this point. If the Region experiences carryover to the filters, consideration could be given to installing disc filters after the Actiflo® process.

(5) Minor vibration of the effluent weirs in the settling tanks (one in each Actiflo unit) was noted at high flows. Some stiffeners may be required in these areas. (Recommendation #10)

(6) It is noted that the full scale test was not performed under poor raw water quality conditions.

(7) Recently, Region operations noted that they are questioning if they are using the correct polymer for the Actiflo® system. The polymer presently in use is FLOPAM™ AN 934 SEP manufactured by SNF (UK) Ltd.; it is anionic and has been in use for approximately eight years on the advice of the manufacturer (John Meunier).

The initial bench scale tests completed in 2003 found that the anionic LT27AG polymer produced better clarified water turbidity than the cationic LT22S at various pHs, and doses of 0.2 mg/L. This is the reason the Actiflo® manufacturer recommended anionic polymer to the Region. We note however that the polymers tested in the bench-scale study were Maganfloc made by CIBA. Hence, there could be differences in polymer performance produced by different manufacturers.

If the Region would like to pursue this issue further, a bench-scale study should be conducted with various polymers from various manufacturers to determine the appropriate polymer and dosage to get optimal turbidity and total organic carbon removal. However, we note that the Region has done numerous such studies to date.

Our experience using an anionic polymer has the following general trends: anionic polymer provided greater iron removal when compared to the cationic polymer and has the potential to reduce floc carryover; organic removal was as good or better for anionic polymers than for cationic polymers; anionic polymers are generally effective at all pH ranges tested; and anionic polymers generally reduced turbidity better than cationic polymers.

4.1.6 Ozone system

(1) Peak production rates up to 143 ML/d will likely occur during the warmer summer months, and are not likely to occur during the colder winter months. CT calculations indicate that CT requirements for 1.0 log *Cryptosporidium* inactivation under such conditions (i.e. temperatures > 5 °C, flow = 143 ML/d), can be met by the ozonation system at lower ozone doses (< 2.0 mg/L). CT calculations indicate that under similar conditions, 1,5 Log inactivation of Cryptosporidium can be met with ozone doses of 2.5 mg/L and 2.0 Log inactivation of Cryptosporidium can be acieved with ozone dose of 3.1 mg/L. The existing ozone equipment has sufficient capacity to meet this need.

Because the ozone system was operated in MANUAL during the 130 ML/d test, the CT requirements for *Cryptosporidium* inactivation were not consistently achieved (goal of 1.0 Log inactivation); when in AUTO mode, the capacity of the ozone system is more than adequate to achieve 1.0 Log inactivation (as per Table 2-6 and Table 3-6), and so this is not a concern. Performance ratio (PR) is defined as PR = Measured disinfection credit divided by the target disinfection credit; the goal is to maintain a PR > 1.0. Based on CT calculations, an ozone residual of ~1.12 mg/L would have assisted in meeting the PR goals.

It is acknowledged that if this discussion was in the context of designing the Actiflo® system, it would not be expected to operate under peak flow and poor or worst case water quality conditions. The ozone system has a nominal rated capacity of 120 ML/d for a 4.0 mg/L ozone dose, and a maximum hydraulic capacity of 160 ML/d which corresponds with a 3.0 mg/L dose. The intent of this re-rating exercise is to utilize existing capacity that remains dormant at present. Secondly, it is acknowledged that planning to test the Actiflo units under high flows and poor water conditions would prove difficult to plan, since it takes 48-72 hours to lower distribution storage levels as well as coordinate SCADA changes, whereas a raw water turbidity event can come with 0-24 hours of notice. In light of these acknowledgements, it is suggested that the Region conduct additional performance tests for longer durations and poor water quality conditions for the Region's own awareness of plant limitations at the targeted flow rate. (Recommendation #9)

(2) As discussed in Section 3.5.3, all treatment objectives related to the ozone system were achieved during the full scale test, except the performance ratio (PR) goal for *Cryptosporidium*. The PR goals were not met because the plant's PLC ozone dosage control rate was turned off during the test and was running in MANUAL mode rather than AUTO mode. It is important that proper ozone dosages are applied to satisfactorily meet the inactivation goal for Cryptosporidium (1-log inactivation). It is recommended to maintain PR between 1.1-1.2 by automatically controlling ozone residual (adjusting ozone dose) and hence the CT value. (Recommendation #2)

(3) Although bromate has not exceeded the maximum contaminant level (10 ppb) in the past, higher ozone doses might lead to higher bromate formation. During the full scale test on May 5, 2015, bromate concentrations measured *** to 3 ppb for the three samples collected.

(4) During the ramp-up period of the full scale test a few ozone offgas pressure HIGH and HIGH HIGH alarms for both contactors were triggered (setpoints of -40 mm H_2O and -20 mm H_2O, respectively). The typical operating range of contactor off-gas pressure is -50 mm H_2O to -100 mm H_2O (a slight vaccum) and is controlled automatically by a pressure-relief valve or variable speed blower. The Oakville WPP has a minimum headspace offgas limit of 300 mm. Water level change in the ozone contactor can create significantly higher off-gas flow rates that over-pressurize the contactor. Based on the HIGH and HIGH HIGH offgas pressure alarms, we infer that the level in the contactors was above the allocated minimum headspace requirement of 300 mm. It should be noted that, in discussion with plant operations, these normally occur during flow increases and do not pose a significant concern in terms of process quality. However, sufficient headspace in the ozone contactors should be maintained for the following reasons:

- To prevent over-pressurization of the contactor; and
- Because ozone gas could also escape before or after the contactor leading to safety issues for operations.

It is therefore recommended that the minimum headspace requirement of 300 mm be maintained. To minimize the potential for headspace surges, consideration should be given to updating process logic (e.g. slower ramp up periods), implementing operations protocol (e.g. smaller low lift pump flow increases during ramp up periods), and additional monitoring (e.g. installing level monitors or glass viewing ports). (Recommendation #3)

Furthermore, it is important to note that the full scale test was only 3 hours, whereas in the United States similar tests must be conducted for a period of one to five days. If the Oakville WPP were to operate for a longer period of time, there is a potential for operational issues to arise (either those discussed thus far or otherwise). One problem with the contactors is that they have no glass ports, so one cannot visually observe what is going on.

(5) During the full scale test it was observed that the ozone generators were able to produce the required ozone dose at the high flows (143 ML/d) as they have been designed for a maximum flow of 160 ML/d. Contactor 1 did better than contactor 2 in terms of CT requirements. Possible reasons for the performance differences may therefore include: Contactor 2 being slightly larger than Contactor 1 and that ozone system was not run in AUTO mode and therefore ozone dosage was not adjusted sufficiently during the test period to meet CT requirements for each contactor. Monitoring of the variation in CT performance between the two contactors is recommended (Recommendation #11).

4.1.7 Ozonated water conduits (ozone-to-filter conveyance)

(1) As noted, the tops of the ozonated water conduits are not fully sealed and may overflow, causing flooding, if the channels operate at 100% level for a period greater than approximately 15 minutes. During the full scale full scale test the water levels in the ozonated water channels were maintained below 100% level for the majority of the ramp-up period, test period, and filter backwash. As discussed in Section 3.5.4, two brief level spikes resulted from operational adjustments which could be expected as this was the first time flows this high were being produced. This did not present a significant process issue; Region operators were able to stabilize flows, and no flooding occurred within the plant.

However, it should be noted that at flow rate of 143 ML/d, the optimal ozonated conduit levels for control of the LLPS may be around 70%, slightly higher than the present desired maximum level of 60%. Further observation from Figure 3-9 of the sinusoidal patterns of the raw water flow and the ozonated water conduit levels (conduit levels control raw water flows) suggest that raw water flows could be more stable and slightly increased if the ozonated conduit setpoint was increased. (Note that whenever conduit 1 level [controlling] decreased to 60%, raw flows would increase to ~136 ML/d, and when conduit 1 level reached ~70%, raw flows would decrease to ~132 ML/d). It is recommended that Region operations experiment with increased setpoints for verification of the optimal setpoint at 130 ML/d net production. The Region could also experiment with controlling the filter rate based on the Actiflo® level and raw water pumping based on the clearwell level, to help reduce the sinusoidal pattern. (Recommendation #4)

(2) The ozonated conduit levels are most sensitive to changes in the combined filter effluent flow rate. They are inversely proportional to one another (i.e. a net surge in filter rates will cause a drop in the conduit levels; and a net drop in filter rates will cause a surge in the conduit levels). It is recommended that the control logic be optimized to minimize rapid changes to the total filter effluent rate in order to avoid surging the ozonated water conduits to 100%. (Recommendation #5)

(3) To remove the hydraulic constraints noted above, it is recommended that the tops of the two ozonated water conduits be sealed. This recommendation is made under the assumption that at present the conduits are enclosed but that the enclosure is not sealed. If the two conduits do not have a top enclosure, a hydraulic assessment should be first conducted to compare open channel vs. full pipe flow conditions. Furthermore, sealing the conduits will also require adequate air and vacuum relief so to prevent over pressurization and to maintain an optimal hydraulic environment. (Recommendation #13)

(4) We further note that in a typical water treatment plant, it is the level of the Clearwell/Reservoir that controls the low lift pumps. If the flooding issue associated with the ozonated water conduits is resolved, the Region should consider this as an alternative control point for raw water flows.

4.1.8 Filters

(1) In regards to the filtration rate, previous pilot testing optimized and demonstrated a filtration rate of 14.3 m/h (231 L/s/filter), which is the rate stated on the Region's Drinking Water Works Permit (South Halton, June 2014) with six filters in service (one filter offline and one in backwash mode) for a total filtration rate of 120 ML/d. To achieve 143 ML/d, a filtration rate of 17.1 m/h (277 L/s/filter) would be required with 6 of 8 filters on line, and 14.6 m/h (236 L/s/filter) with 7 of 8 filters online.

However, we consider using only 6 of the 8 filters to determine the capacity of the filtration system to be overly conservative for determination of its rated capacity for the following reasons. First, filter backwashing occurs for an estimated average of 3 hours of a 24 hour period under peak flow conditions.[24] Secondly, if a filter was scheduled to be offline (e.g. maintenance), the plant would not be expected to operate at peak production rate. If a filter was taken offline for emergency maintenance purposes, the Region's distribution network has sufficient supply and storage capacity to ensure security of supply to its customers.

Acknowledging that the filters are a key rate limiting process, we have thus estimated the Oakville WPP production capacity by determining average maximum production volume over a 24-hour period using data and knowledge gained from the May 5, 2015 full scale test (Recommendation #21). Refer to Section 4.5 for determination of re-rated WPP capacity based on this approach.

(2) Filter 1. As discussed in Section 3.5.5, during the time frame 15:50-16:10 of the test , Filter 1 level dropped below the bottom of the trough. Operator had to reduce Filter 1 filter rate to 90 L/s to raise its level back to normal (other filters were operating around 225 L/s). At high flow rates, flow into Filter 1 cannot keep pace with flow out. Analysis suggests the following hydraulic limitations with Filter 1 at high flows.

- Filter 1 is at the beginning of the filter inlet channel that feeds filters 1-4. This channel also has a secondary feed coming up through the bottom of the channel just upstream of Filter 1. Thus, at high flow rates, at Filter 1, the water is flowing fast enough to partially bypass the inlet port into Filter 1. As water moves beyond Filter 1, flow rate is successively reduced to the remaining Filters 2, 3, and 4 due to both the reduced flow as water enters each filter friction losses in the channel.
- The filter 1 inlet port is directly aligned with the wall of one of the trough structures within (as opposed to being in line with the filter trough itself), thus water flowing in encounters immediate resistance. This is not the only filter configured as such, but combined with the preceding issue appears to produce a condition of hydraulic resistance which limits the flow entering Filter 1.

Accordingly, it is recommended that the Region further consider the options for adjustments (structural or otherwise) to Filter 1 to mitigate this issue. (See Recommendation #14)

(3) Filter Outlet. As noted in the hydraulic model discussion (Section 2.3.3), at the targeted flow conditions of 143 ML/d, if clearwell level conditions are high and combined with the headloss development as the filter media becomes dirty, the available driving head in the filters will be reduced and will result in shorten filter run times between backwash cycles.

This was not an issue during the full scale test due to its short duration. However, we recommend further assessing the impact on filter performance (i.e. time between backwash cycles) through testing at longer durations under the targeted gross rate of 143 ML/d. (Recommendation #21)

(4) Under present conditions, the average filter run time is 80 to 100 hours (four days) – this filter run time was achieved during raw water conditions with turbidity less than 100 NTU.[25] If demand of 130 ML/d is needed during an expected high raw water turbidity episode (e.g. wet weather, spring rains, initial snow melt), it is recommended that filters be cleaned prior as a mitigative measure. (Recommendation #6)

---

[24] Each filter has a runtime of 100 hours under peak filtration rates (14.3 m/h or 231 L/s) under typical water quality conditions. If filter backwashes are evenly spaced, with eight filters, a backwash interval occurs every 12.5 hours, or twice per day, each lasting approximately 1.5 hours, thus a total of 3 hrs per day.

[25] Operational parameters learned during GHD's site visit (Technical Baseline Review Memo #3, Dec. 2014).

4.1.9 Clearwell and reservoir

(1) During the full scale test filter backwash, the clearwell levels reportedly dropped from 89% to 80%, while the reservoir levels reportedly dropped from approximately 79% to 50%, while the high lift pumps continued to pump 130 ML/d. It was also discovered that the master filter rate, which was automatically activated when the backwash commenced, was set to 1400 L/s (approximately 120 ML/d). Region operations have indicated that during a backwash under lower flow conditions (50 to 80 ML/d), storage levels typically drop from about 90-82% (clearwell) and 89-80% (reservoir), and that the levels are usually replenished in about 1.5 hours.

As discussed in Section 3.4, however, under lower flow conditions (~45 ML/d) prior to the ramp up period, the difference in the downstream reservoir hydraulic head was approximately +15 cm (see Figure 3-5), which is not possible (i.e. the reservoir level cannot have a higher surface elevation than the clearwell with forward moving flow). This strongly suggests an inaccuracy either in the elevations reported in the drawings or in the level monitoring systems. It is recommended that the Region verify the entirety of the clearwell and reservoir monitoring systems.

Furthermore, the floor elevation of the Reservoir is approximately 0.21 m higher than the upstream Clearwell. Thus, when both the high lift pumps and the backwash pumps are running, the conditions are favourable for either the Reservoir to backfeed to the clearwell backwash pumps, or for the high lift pumps to draw down reservoir levels if the filtration rate is insufficient for high lift pumping needs. After further analysis, our opinion is that of the latter and no additional investigation is required at this time.

See Section 4.5 Estimation of re-rated WPP capacity for discussion on the plant’s ability to produce 130 ML/d net over a 24-hour period while maintaining clearwell and reservoir storage levels.

(2) In addition to the above point, during the full scale test (and normally at high flows), the Reservoir’s two dual-channelled cells were operating in series (i.e. one inlet/outlet valve pair open; one inlet/outlet valve pair closed; with interconnecting valve open) and not in parallel. This is done to increase the flow path through the serpentine configuration, achieving a baffling factor of 0.3.

This also could be a contributing factor to the noted depletion of reservoir levels. One means to attempt to preference flow from the Reservoir to the HLPS (as opposed to the Clearwell flume) would be to operate the Reservoir with both inlets and both outlets open. Another option previously discussed was to improve the hydraulics of the Reservoir-to-HLPS piping (as it is noted to have several horizontal and vertical bends), however since the high lift pumps continued to pump 130 ML/d through this piping, we have disregarded this suggestion; the clearwell and reservoir levels depleted during the backwash because the initial filter flows were insufficient due to a software interlock.

(3) It was previously noted in GHD’s Technical Baseline Review Memo #3 (Dec. 2014) that in the event that the ozone system is off-line, the Reservoir is required to provide chlorine contact time for primary disinfection.[26] The noted limitations included:

- *Giardia* and viruses: For 5 °C water temperatures or lower, primary disinfection using chlorine is attainable for flows less than 120 ML/d. The minimum and average water temperatures for

[26] Refer to Table 2-4 for a summary of the minimum reservoir water depth required to meet the concentration time (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.

the Oakville WPP are 3 °C and 14 °C, respectively.[27] At this average temperature the existing reservoir would be able to meet the secondary disinfection requirements at the re-rated plant capacity. Therefore, at plant flows of 130 ML/d net and water temperatures less than 10 °C, primary disinfection should be attained using ozone and not chlorine.

- *Cryptosporidium*: Chlorine is not able to meet the WPP's internal objective of 1.0 log inactivation at plant flows of 130 ML/d (net) at any temperature and therefore primary disinfection should be achieved using ozone.

4.1.10 High lift pumping station

(1) In order to operate with one pump in standby mode, the fourth high lift pump needs to be installed. The present configuration of two duty/one standby high lift pumps, the rated capacity of two pumps operating is 109 ML/d. During the full scale test, with all three pumps operating and system distribution pressures at the WPP interface was capable of operating at 130 ML/d net production. (Recommendation #15)

(2) When all three HLPs were operating at 130 ML/d, vibrations were noted in the HLP building that are not typically noticed (i.e could be felt while standing on the catwalks) and should be investigated further to determine the extent, causes, and mitigative measures. If vibrations are determined to be a significant issue, suggested recommendations may include: altering the catwalk's resonant frequency through structural adjustments or additional supports; and consideration of installing a decibel meter. (Recommendation #19**.**)

(3) During the full scale test the Davis Road Zone O1 bypass valve[28] was throttled to 49% in order to reduce distribution system pressures in order to achieve the target production rate of 130 ML/d. Throttling this valve at similar values may be required in order to enable 130 ML/d net flows to come from the WPP; this may vary under conditions, for instance, when future demand necessitates that the WPP produce 130 ML/d and the reservoir levels may have less storage available (i.e. operating at normal storage levels). The Region should monitor and further observe conditions to determine optimal distribution system parameters. (Recommendation #7)

4.2 Chemical systems

4.2.1 Chlorination systems

(1) The chlorine feed system does not have chlorinator redundancy.

(2) The dosages and chlorination capacities stated in GHD's Baseline Technical Memo (Dec. 2014) show that the plant does not have sufficient capacity at higher flows. Data in this report was updated using 2015 data which showed that now only the reservoir inlet chlorinator does not have sufficient capacity at average dosage to meet the targeted production rate of 130 ML/d (see Table 2-6). It is noted however that the dosage for this chlorinator is high and is expected to be due to necessary over quenching of the ozone residual, and thus if chlorine was not needed to quench excess ozone it is expected that the present chlorinator capacity would be sufficient.

Furthermore, it is noted that the feed rates during the full scale test, as per the SCADA data provided by the Region, indicate that the chlorinators were operating well below their respective

[27] Based on January 2010 to June 2014 operational data. See Table 2-1 in GHD's Technical Baseline Review Memo #3, Dec. 2014.

[28] The Davis Road Zone O1 valve splits flows from the Oakville WPP to a distribution reservoir and other parts of the local distribution system. See Footnote 23 for more information.

capacities (compare Table 2-6 and Table 3-12), which suggests that ozone quenching via chlorine at the reservoir inlet was not required.

(3) The number of days of storage provided by twelve ton gas chlorine cylinders, at 130 ML/d and average chlorine dosage, is approximately 4.8 days. (Region operations has confirmed that on average 6 tonners are received monthly.) 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. It is noted again that if the chlorine dosages were not also functioning to quench ozone that the present storage capacity would suffice for a longer duration.

During the full scale test, all three chlorinators provided sufficient capacities for chlorination of the WPP process water. Chlorinator feed rates, during the full scale test were well below the chlorinator capacities.

4.2.2 Dry polymer (Type 1)

(1) The existing Type 1 polymer feed system can treat flows up to 143 ML/d at the maximum 0.55 mg/L dosage. Approximately 4-5 hours of storage is provided by one 1.8 m^3 day/batching tank.

(2) At dosages greater than 0.6 mg/L, one PolyBlend unit cannot meet the feed requirements (average and maximum dosages are 0.25 mg/L and 1.0 mg/L, respectively). To accommodate higher dosages, the standby unit could be placed online to meet the targeted production requirements; however this leaves the system without redundancy.

(3) WPP operations staff noted that the hopper system has a tendency to clog.

(4) No issues associated with the Dry Polymer Type 1 were noted during the full scale test.

4.2.3 Average chemical dosages

Based on our analysis of the data received from the full scale test, the majority of the chemical systems operated without issue and demonstrated sufficient feed capacities at average dosages. It is noted that raw water flows averaged at 136 ML/d during the three hour test and not at 143 ML/d, the latter of which was used to evaluate equipment capacities in GHD’s Technical Baseline Review Memo #3 (Dec. 2014).

The exceptions to the above paragraph, based on the data in Table 3-12, are as follows:

- Calcium thiosulphate pump 2 (contactor 1), estimated average feed rate was calculated to be 42.7 L/hr with a peak value of near 50 L/hr during the test. The capacity of the unit is 45.4 L/hr.
- Calcium thiosulphate pump 3 (contactor 2) operated at an estimated average of 35 L/hr during the test, peaking at 41-42 L/hr, the latter of which approaches the unit’s 45.4 L/hr capacity.
- Filter aid polymer (Type 2) pumps were off during the test, and therefore could not be evaluated. However, no capacity concerns were noted during our baseline review.

4.2.4 Maximum chemical dosages

The WPP will not have sufficient capacity to feed chemicals at the maximum dosages at 143 ML/d (see also Figure 2-3 and Table 2-6): Filter aid polymer (Type 2) and the chlorinators.

4.2.5 Storage

Based on the full scale test, all chemical systems provide sufficient storage capacity for a gross flowrate of 143 ML/d. Dry Polymer Type 1 provides approximately 4 hours of storage capacity at 143 ML/d based on the quantity of dry chemical stored. Additional storage for Dry Polymer Type 1 should be investigated when re-rating the WPP.

4.3 Water quality parameters

4.3.1 Disinfection byproducts

As seen from Table 3-14, bromate formation was well below the MCL of 10 ppb at the ozone doses applied during the test. The TTHMs in the finished water were below the MCL (actual value= 4.5 ppb < MCL=80 ppb). The HAAs in the finished water were also below the MCL (actual value= 24.8 ppb < MCL = 60 ppb). As reported in these tables, many of the disinfection byproducts (DBPs) were below the minimum detectable limit (MDL). We conclude that DBP formation was not an issue during the full scale test.

4.3.2 Filtered water quality

The only concern that we note from the grab samples collected during the test is regarding the quality of filtered water (i.e. filter effluent water). The laboratory results from our grab samples returned a turbidity reading of 0.41 NTU and also a total organic carbon (TOC) reading of 2.8 mg/L (up from the <0.2 mg/L reading from the Actiflo® effluent) (see Table 3-14). Possible explanations include:

- Lab error, some or all of these values have some QA/QC issue,
- Sampling error (bottle contamination), or
- Resuspension of previously removed organic carbon due to the increase in filter flow rates.

Since the filters are one of the rate limiting process steps and were operating near filtration capacity, we note that this is worthy of additional investigation.

4.4 Distribution system

During the full scale test the Davis Road Zone 1 valve[29] had to be throttled to 49% in order to reduce distribution system pressures in order to achieve the target production rate of 130 ML/d. Throttling this valve at similar values may be required in order to enable 130 ML/d net flows to come from the WPP. This may change under conditions where in instance when future demand necessitates that the WPP produce 130 ML/d and the reservoir levels may have less storage available. The Region should monitor and further observe conditions to determine optimal distribution system parameters.

[29] The Davis Road Zone 1 valve splits flows from the Oakville WPP to a distribution reservoir and other parts of the local distribution system. See Footnote 23 for more information.

4.5 Estimation of re-rated WPP capacity

As discussed in Section 4.1.8 Filters, below are estimates showing the average maximum production volume over a 24-hour period using the data and knowledge gained from the May 5, 2015 full scale performance test.

4.5.1 Approach

The plant production capacity will be based on that of the filtration system subject to all losses between the filter-clearwell discharge and the high lift pumping to distribution. Reasons for using the filter capacity to calculate plant production capacity are as follows:

- Excluding storage and chlorine addition, the filters are the final process prior to high lift pumping to distribution. Storage, chlorine contact time, and high lift pumping requirements for 130 ML/d net production are met as discussed in this report, and hence the filters are the rate-limiting step.
- As evidenced during this full scale test, during a filter backwash, high lift pumping rates to the distribution system drew on reservoir and clearwell storage volumes which were not being replenished satisfactorily by the filters. Using the production capacity of the filters will simplify and clarify the analysis and can be reliably done by accounting for losses that occur between filter effluent flows and high lift pumping flows to distribution (i.e. service water and hydraulic headloss) and assuming a net zero change to these storage levels.

Three scenarios will be considered:

1. *Do Nothing 'A': Target 130 ML/d net production.* This scenario will extrapolate the field test results over a 24-hour period while optimizing filter flow rates to achieve the targeted 130 ML/d net production. A conservative approach will be taken to Filter 1 flows due to its noted hydraulic issues.
2. *Do Nothing 'B': Maximum production.* This scenario will extrapolate the field test results over a 24-hour period to estimate the maximum net production volume by maximizing filter flow rates within their constraints. A conservative approach will be taken to Filter 1 flows due to its noted hydraulic issues.
3. *Fix Filter 1 and Target 130 ML/d net production.* This scenario will consider the hydraulic issues of Filter 1 to be resolved in order to evaluate the benefit of implementing the corresponding recommendation. The filter flow rates will be optimized to achieve the targeted 130 ML/d net production.

4.5.2 Hydraulic flow balance model

The production capacity of the Oakville WPP over a 24-hour period may be expressed as:

$$Q = Q_F + Q_{F*} - Q_B - Q_L$$

Where **Q** is the total net production capacity of the plant, $\mathbf{Q_F}$ is the production capacity of filters during non-backwash times, $\mathbf{Q_{F*}}$ is the production capacity of the filters during a filter backwash, $\mathbf{Q_B}$ accounts for filter backwash losses, and $\mathbf{Q_L}$ accounts for other losses, all over a 24-hour period, thus having units of ML/d.

Based on the configuration of the Oakville WPP, Q_F, Q_{F*}, Q_B and Q_L can then be stated as

$$Q_F = t_F(q_1 + q_2 + \cdots + q_8)$$

$$Q_{F*} = nt_{F*}(q_{1*} + q_{2*} + \cdots + q_{8*})$$

$$Q_B = n(t_{low}q_{low} + t_{high}q_{high} + t_{FTW}q_{FTW})$$

$$Q_L = t(q_{service} + q_{HL} + q_{other})$$

Where $\mathbf{q_1}$ through $\mathbf{q_8}$ are individual filter flows, $\mathbf{q_{1*}}$ through $\mathbf{q_{8*}}$ are individual filter flows during a backwash cycle, $\mathbf{q_{low}}$ is the low backwash rate, $\mathbf{q_{high}}$ is the high backwash rate, $\mathbf{q_{FTW}}$ is the filter-to-waste flow rate that occurs at the end of a backwash cycle, $\mathbf{q_{service}}$ is the portion of flow that is diverted to service water, $\mathbf{q_{HL}}$ accounts for general headlosses between the filter flow meters and the high lift pumping station flow meters, $\mathbf{q_{other}}$ is a buffer to account for other unanticipated losses (a conservative measure), **n** is the number of backwashes per day, $\mathbf{t_F}$ is the duration of non-backwash filter flows, $\mathbf{t_{F*}}$ is the duration of filter flows during a backwash, **t** is the total 24-hour period, and $\mathbf{t_{low}}$, $\mathbf{t_{high}}$, $\mathbf{t_{FTW}}$ are the low wash, high wash, and filter-to-waste runtimes, respectively.

Determination of q_1, q_2, …, q_8 and q_{1*}, q_{2*}, …, q_{8*}

During the 2.5 hours of the full scale test before backwashing, all filters, including Filter 1, had a mean average flow rate of 194 L/s (combined 1552 L/s or 134 ML/d). During the backwash of Filter 6, flow rates for all filters, including Filter 1, had an initial mean average of 204 L/s (combined 1428 L/s or 123 ML/d), and was increased to approximately 218 L/s (combined 1526 L/s or 132 ML/d), at which point Filter 1's level dropped significantly and operations reduced the rate to 90 L/s to restore its level while the flow of the remaining six filters were increased to 222 L/s to compensate (combined 1422 L/s or 123 ML/d). The drop in Filter 1's level is understood to be triggered by quick changes to flow rates coupled with Filter 1's hydraulic issues as discussed in this report.

Therefore, Filter 1 flow rates for the various scenarios will be as follows:

- Scenarios 1 and 2, maximum q_1 of 190 L/s and maximum q_{1*} of 190 L/s.
- Scenario 3, q_1 and q_{1*} will be equal to the flow rate of the other online filters.

During normal production, individual filter flows for filters 2 through 8 will be identical. During production while backwashing, filters 2 through 7 will be identical and filter 8 will be considered in backwash mode. Individual filtration rates will be limited to a maximum of 218 L/s (13.5 m/h) as per test results and as a conservative measure. Recall maximum filtration rate per filter is 231 L/s (14.3 m/h).

Determination of q_{low}, q_{high} and q_{FTW}

As per Region operations low rate and high rate backwash flows are 25 ML/d (291 L/s) and 54 ML/d (631 L/s), respectively. The SCADA data records the high rate wash during the test day to be only approximately 43 ML/d (500 L/s); the larger value of 54 ML/d will be used as a conservative measure. Therefore, $\mathbf{q_{low}}$=25 ML/d and $\mathbf{q_{high}}$=54 ML/d.

The filter-to-waste rate will be taken to be that of the remaining online filters (other than Filter 1) as a conservative measure. Thus, $\mathbf{q_{FTW}}=q_{2*}$, provided that Filter 2 is not the filter being backwashed.

Determination of n and t variables

Since we are calculating production capacity over a 24-hour period, the total production time is equal to the sum of all normal (i.e. non-backwash) production and production during a filter backwash:

$$t = t_F + nt_{F*} = 24\ hours$$

Each filter has a runtime of 100 hours under peak filtration conditions (14.3 m/h or 231 L/s) under typical water quality conditions. If filter backwashes are evenly spaced, with eight filters, a backwash interval occurs every 12.5 hours, or twice per day. A typical backwash procedure takes approximately 1.5 hours to complete. Over a 24-hour period then, an average of two 1.5 hour backwashes will occur, leaving the remaining non-backwash filter run time to be 21 hours. In summary, $\mathbf{t}$=24 hours, $\mathbf{n}$=2, $\mathbf{t_{F*}}$=1.5 hours, and $\mathbf{t_F}$=21 hours.

As per Region operations and SCADA data from the test day, the high rate backwash as indicated from SCADA records lasts 4.5 minutes and occurs once per backwash cycle, the low rate backwash lasts 3 minutes and occurs twice per backwash cycle, and the filter-to-waste lasts approximately 15 minutes[30] and occurs once per cycle. Therefore, $\mathbf{t_{low}}$=6 minutes, $\mathbf{t_{high}}$=4.5 minutes, and $\mathbf{t_{FTW}}$=15 minutes.

Determination of Q_L

$\mathbf{Q_L}$ can be determined using the SCADA data during the full scale test period. The service flow water ($\mathbf{q_{service}}$) had a mean average flow of 1.7 ML/d during the test (1.3% of 130 ML/d). The headloss between the filter flow meters and those of the high lift pumps ($\mathbf{q_{HL}}$) can be calculated as the difference between the combined filtration rate and the total flow out of the high lift pumps (i.e. east header, west header, and service water), which had a consistent mean average of 2.7 ML/d (2.1% of 130 ML/d). Therefore, $\mathbf{Q_L}$ can be expressed as:

$$Q_L = (q_{service} + q_{HL}) + q_{other} = (1.7 + 2.7) + q_{other} = 4.4\frac{ML}{d} + q_{other}$$

Other unanticipated losses may occur and should be accounted for as a conservative measure. These losses may include additional service water or hydraulic headlosses, the need to reduce filtration rates to stabilize filter levels (e.g. Filter 1), etc. It is also reasonable to say that the potential and magnitude of unanticipated losses will decrease if the Filter 1 hydraulic issues are resolved. Consideration should also be given that hydraulic and process losses for the entire plant are estimated at 10%, and 3.3% of the losses are accounted for with $q_{service}$ and q_{HL} alone. Therefore, a value of 2% and 1% of the target net production flows (2.6 ML/d and 1.3 ML/d) will be used for other losses for scenarios 1 and 2, and for scenario 3, respectively. $\mathbf{Q_L}$ can then be expressed as:

$$Q_L = 4.4\frac{ML}{d} + q_{other} = 4.4\frac{ML}{d} + 2.6\frac{ML}{d} = 7\frac{ML}{d} \qquad \text{for Scenarios 1 and 2}$$

$$Q_L = 4.4\frac{ML}{d} + q_{other} = 4.4\frac{ML}{d} + 1.3\frac{ML}{d} = 5.7\frac{ML}{d} \qquad \text{for Scenario 3}$$

4.5.3 Results

The results for the production capacity of Oakville WPP over a 24 hour period for the scenarios described above are summarized in the table below.

[30] In actual practice, at the present, Region operations will backwash filters until the turbidity reading is below 0.15 NTU. Per previous discussions with the Region, this generally takes about 15 minutes.

Table 4-1 Summary of daily WPP production scenarios

Variable	S1: Do Nothing ‘A’ and target 130 ML/d	S2: Do Nothing ‘B’ and maximize production	S3: Fix Filter 1 and target 130 ML/d
Variables			
q_1	190 L/s (11.7 m/h)	190 L/s (11.7 m/h)	200 L/s (12.4 m/h)
q_2 through q_8	204 L/s (12.6 m/h)	218 L/s (13.5 m/h)	200 L/s (12.4 m/h)
$\sum q_i$	1618 L/s (139.8 ML/d)	1716 L/s (148.3 ML/d)	1600 L/s (138.2 ML/d)
q_{1*}	190 L/s (11.7 m/h)	190 L/s (11.7 m/h)	208 L/s (12.9 m/h)
q_{2*} through q_{7*}	209 L/s (12.9 m/h)	218 L/s (13.5 m/h)	208 L/s (12.9 m/h)
q_{8*}	0 L/s	0 L/s	0 L/s
$\sum q_{i*}$	1444 L/s (124.8 ML/d)	1498 L/s (129.4 ML/d)	1456 L/s (125.8 ML/d)
q_{other}	2.6 ML/d	2.6 ML/d	1.3 ML/d
Calculated volumes over 24 hour period			
Q_F	122.3 ML	129.8 ML	120.9 ML
Q_{F*}	15.6 ML	16.2 ML	15.8 ML
Q_B	0.9 ML	0.9 ML	0.9 ML
Q_L	7.0 ML	7.0 ML	5.7 ML
Net Production (QF + QF* - QB - QL)			
Q	130.0 ML	138.1 ML	130.1 ML

See discussion above for calculation details. Summary of assumptions: Filter 1 maximum rate = 190 L/s for scenarios 1 and 2; Filters 2-8 maximum rate = 218 L/s; Filter 8 is used as backwash filter; other unaccounted losses (q-other) is taken as 2% and 1% of 130 ML/d for scenarios 1 and 2, and scenario 3, respectively.
***Note that this table represents the data obtained during the 130 ML/d full scale test, and accordingly is based on test conditions and average raw water quality conditions.*

Discretion was used in selecting filter flow rates for both normal production and production during a filter backwash. Attempt was made to minimize both combined filtration rates ($\sum q_i$ and $\sum q_{i*}$). Recall that individual filtration rates had a limited maximum of 218 L/s (13.5 m/h) as discussed above.

Do Nothing scenarios. Calculations show that the filters have sufficient capacity to produce 130 ML/d net to distribution over a 24-hour period using flow rates and losses experienced during the May 5 full scale test. By running the filters at 218 L/s (13.5 m/h), except for Filter 1 and the backwash filter, the net production to distribution is 138 ML per day. This may also be interpreted as an extra 8 ML/d of buffer to account for additional losses (e.g. hydraulic, extra backwashes, process interruptions) when aiming for the target production rate of 130 ML/d; the extra 8 ML/d is also equivalent to 1.5 hours process time. The filter rates for scenarios 1 and 2 may also serve as target rates for operational purposes.

Fix Filter 1 scenario. If the hydraulic issues for Filter 1 are addressed, this will allow a reduction of all filter rates, particularly during backwash events, which is desirable from an operations perspective.

In summary, the results indicate that 130 ML/d net production may be achieved without addressing Filter 1’s restrictions, and also that addressing Filter 1’s restrictions would reduce the process burden placed on the remaining filters as well as increase confidence of operators in the filtration system when operating at the targeted production rate of 130 ML/d.

5. Key Findings and Recommendations

5.1 Key findings

The following is a summary of the key findings from the 130 ML/d test.

1. No issues were discovered during the full scale test with the intake, travelling water screens, or low lift pumps during the performance test. Prior to the ramp up period, raw water turbidity levels were less than 1 NTU. Intake scouring during the ramp up period peaked raw water turbidity levels up to 30 NTU; average raw water turbidity during the three hour test period was 12 NTU.
2. The full scale test revealed that Actfilo® units performed satisfactorily and were able to achieve > 90% reduction in turbidity and total organic carbon.
3. Although the ozonation system exceeded the inactivation goals for Giardia (0.5-log inactivation), viruses (2.0 log inactivation), it did not meet satisfactorily the inactivation goal for Cryptosporidium (1-log inactivation). Average log inactivation values for Cryptosporidum ranged between 0.8-0.9 log. Furthermore, contactor 1 performed better than contactor 2 with respect to achieving required CT. However, the ozone system is capable of achieving the required performance ratio. Possible reasons for the performance differences may therefore include: the size difference between Contactor 1 and 2 (the latter being slightly larger volume); and that the ozone system was not run in AUTO mode and was not adjusted sufficiently during the test period to deliver the appropriate doses and meet CT requirements for each contactor. However, based on review of the ozone system it is capable of achieving the required performance ratio for 1.0 Log inactivation of *Cryptosporidium* in both cold and warm waters. CT calculations revealed that in warm waters (> 5 C°) up to 2.0 Log inactivation of *Cryptosporidium* is possible with the existing ozone system.
4. The ozonated water conduits did not present any major hydraulic issues during the full scale test. Due to the fact that the tops of the conduits are unsealed, they must be operated at a water level below 100% to avoid risk of flooding parts of the facility. Since the level in the ozonated water conduits controls the low lift pumps, it is therefore noted that these conduits pose a potential hydraulic bottleneck. Levels in the channel are particularly sensitive to water flow rates and will surge during raw water flow ramp ups and during quick changes to the combined filter effluent rate.
5. Filter 1 experiences significant hydraulic limitations under high flow conditions (generally above 100 ML/d). During the full scale test backwash cycle, Filter 1 level was only stabilized when its filter effluent rate was decreased to 90 L/s, while the remaining six filters operated at approximately 225 L/s. Field observations suggest that this is due to Filter 1's position along the filter inlet channel and its configuration of the inlet port relative to the trough structures within.
6. The filtration system, at 143 ML/d gross, will filter more particulate at an increased rate. If clearwell levels are also high, the available driving head will be reduced. The net effect will shorten filter run times between backwash cycles. This was not experienced during the full scale test but derived from desktop analysis and requires further investigation to determine filter performance (i.e. run time) under these conditions.

7. During the full scale test filter backwash, the clearwell levels dropped from 89% to 80%, while the reservoir levels dropped from approximately 79% to 50%, while the high lift pumps continued to pump 130 ML/d. Region operations have indicated that during a backwash under lower flow conditions (50 to 80 ML/d), storage levels typically drop from about 90-82% (clearwell) and 89-80% (reservoir), and that the levels are usually replenished in about 1.5 hours. The record drawings indicate that the floor elevation of the Reservoir is approximately 0.21 m higher than the upstream Clearwell. However, the hydraulic profile created from the SCADA data shows the downstream reservoir water surface elevation being higher than that of the clearwell with forward flow to the reservoir during pre-test flows, which is not possible. This therefore suggests an inaccuracy either in the elevations reported in the drawings or in the level monitoring system (e.g. monitoring range, instrument mounted level, etc.). It is recommended that the Region verify the entirety of the clearwell and reservoir monitoring systems.

8. At high flows, the Region operates the Reservoir's two dual-channelled cells in series and not in parallel. This is done to increase the flow path through the serpentine configuration. In this configuration, this longer flow path may be contributing to hydraulic restrictions experienced during a filter backwash.

9. The chemical metering pumps operational during the test, with minor exceptions, performed satisfactorily and within their capacities (Chlorinators 1-3; Calcium thiosulphate pumps 1-3; Fluoride pump 2). The estimated average flow rates of Calcium thiosulphate pumps 2 and 3 were 43 L/hr and 35 L/hr, respectively, with estimated peak values of approximately 50 L/hr and 41 L/hr; the capacity of each of these pumps is 45.4 L/hr. Chemical systems not operational during the test included filter aid polymer (Type 2); Alum; Hydrogen peroxide.

10. When all three HLPs were operating at 130 ML/d, participants felt vibrations while standing on the catwalks in the HLP building. Further investigation of this issue is recommended.

11. Bromate formation was below the maximum concentration level (MCL) of 10 ppb. Low bromide concentration in the raw water translated into lower bromate formation.Other disinfection byproducts (e.g. THMs and HAAs) in the finished water were well below maximum concentration limits, many even below minimum detectable limits, and did not pose any problems.

12. During the full scale test the Davis Road Zone O1 bypass valve was throttled to 49% in order to reduce distribution system pressures in order to achieve the target production rate of 130 ML/d.

13. Net production volumes over a 24-hour period were estimated using data from the full scale test for three scenarios of filter operation. The estimates used the filters as they have been determined to be the rate limiting process. The scenarios included two Do Nothing scenarios (one minimizing the filter flow rate variable to achieve 130 ML/d net, the other maximizing filter flow rate variable to their rated maximum of 14.3 m/h) and one Improvement scenario (address Filter 1 hydraulic restrictions). The results indicate that 130 ML/d net production may be achieved without addressing Filter 1's restrictions, and also that addressing Filter 1's hydraulic restrictions would reduce the process burden placed on the remaining filters as well as increase in confidence of operators in the filtration system when operating at the targeted production rate of 130 ML/d. It is further noted that limited headloss through the filter coupled with a 20% increase in gross flowrate through the filter (120 to 144 ML/d) will potentially reduce filter runtimes between backwashes.

5.2 Recommendations

The following recommendations are proposed to facilitate the Oakville WPP re-rating to 143 ML/d gross and 130 ML/d net production.

Operations and protocol recommendations

1. Travelling water screens should be operated in parallel during peak raw water flows (130 to 143 ML/d). This is particularly important during low lake levels. If one screen needs to be taken out of service, instantaneous raw water flows should not exceed the present water taking allowance (120 ML/d). Maintenance for the travelling water screens should be scheduled during periods where 143 ML/d raw water pumping is not anticipated or can be avoided (e.g. not during peak summer demand). In the event that one screen needs to be taken offline during peak water demand season, temporarily reduce the maximum production of Oakville WPP to not exceed the present water taking allowance (120 ML/d) and supplement supply with other sources (e.g. Burlington WPP, Burloak WPP, or supply from neighbouring municipalities). (Section 2.1.2 and 4.1.2)
2. Maintain a Performance Ratio of 1.1 to 1.2 for inactivation of Cryptosporidium by controlling ozone residual (i.e. adjusting ozone dosage).
3. Maintain the 300 mm minimum required headspace in the ozone and that headspace surge frequencies be reduced. To minimize the potential for headspace surges, consideration should be given to updating process logic (e.g. slower ramp up periods), implementing operations protocol (e.g. smaller low lift pump flow increases during ramp up periods), and additional monitoring (e.g. installing level monitors or glass viewing ports). (Section 4.1.6)
4. Optimize the ozonated water conduit level setpoint for raw water flow control for flows above 120 ML/d (gross) for the targeted production of 130 ML/d (net). Considerations should also be given to experiment with controlling the filter rate based on the Actiflo® level and raw water pumping based on the clearwell level in order to help reduce the sinusoidal pattern. (Section 4.1.7)
5. Optimize the control logic for changes to the total filter effluent rate to prevent instantaneous changes in order to avoid surging the ozonated water conduits to 100%. (Section 4.1.7)
6. Continue protocol of backwashing filters prior to an expected high raw turbidity episode (e.g. wet weather, spring rains, initial snow melt) if demand is also expected to approach 130 ML/d (net). (Section 4.1.8)
7. Optimize and determine the operable range at which the Davis Road Zone O1 bypass valve is throttled to reduce local distribution system pressures near Oakville WPP to permit net production flows of 130 ML/d. (Sections 3.5.7, 4.1.10, and 4.4)
8. Update/revise the process control narrative for the WPP for net production flows of 130 ML/d to better manage flows and overflows. This may include rearranging how processes are controlled. Consideration should be given to how other similar water treatment plants are controlled.
9. Suggestion to conduct full scale testing (130 ML/d net) for longer duration and under poorer water quality conditions to evaluate the performance of both Actiflo® ,Ozone and filtration systems and clearwell at targeted flow rate for operational knowledge only. This is not a re-rating requirement. It is acknowledged that these processes have the capacity under typical operating conditions to produce 130 ML/d net over a 24 hour period, as well as the difficulty

associated with coordinating a full scale test to coincide with poor raw water quality (Sections 4.1.5 and 4.1.6).

Monitoring recommendations

10. Actiflo® monitoring:
    - Monitor the Actiflo® maturation tank mixer's stability during high flow conditions. Conduct further investigation if an issue arises. (Section 2.1.5 and 4.1.5)
    - Monitor the Actiflo® effluent for floc carryover during high flow conditions (influencing factors are the fixed capacity of the recirculation pumps and the gaps between lamella tube sections). (Section 2.1.5 and 4.1.5)
    - Monitor the Actiflo® settling tank weirs for vibrations during high flows. If noted, and if further investigation warrants, the issue may be mitigated by installation of stiffeners. (Section 2.1.5 and 4.1.5)
11. Monitoring of the variation in CT performance between ozone contactors 1 and 2. During the full scale test, with respect to CT requirements, contactor 1 performed better than contactor 2. It is suspected that this was due to Contactor 2 being larger than the Contactor 1 and ozone dosages not adjusted accordingly to meet CT requirements. In the future, this fact should be kept in mind while running performance tests. (Section 4.1.6)

Capital works recommendations

12. Twin the 50 mm dia. raw water sampling line at the intake crib and install a backflushing system to mitigate line clogging which has been known to happen. Value of upgrade: Ensure monitoring of high turbidity at the intake crib's location as the present line is known to clog; monitoring is needed to give operators extra time to prepare and adjust processes. (Section 2.1.1 and 4.1.1)
13. Seal the top of the two ozonated water conduits or consider replacing conduits with pipe. Another option could be to make the channel walls higher. This would remove the hydraulic bottleneck that limits raw water flows when conduit levels are high, which generally occur during flow adjustments, particularly with adjustments to the master filter rate. This is made under the assumption that at present the conduits are enclosed but that the enclosure is not sealed. If the two conduits do not have a top enclosure, a hydraulic assessment should be first conducted to compare open channel vs. full pipe flow conditions. Furthermore, sealing the conduits will also require adequate air and vacuum relief so to prevent over pressurization and to maintain an optimal hydraulic environment. Further discussion with operations and field investigation required. (Section 4.1.7)
14. Investigate options for mitigating the flow restrictions into Filter 1, which may include adjustments to its structure. With respect to the flows bypassing the Filter 1 inlet, suggestions include installing a bulkhead in the filter inlet channel between Filters 1 and 2 and/or increasing the size of the inlet gate. With respect to the position of the internal filter troughs relative to the inlet port, it is suggested to realign the inlet port with one of the trough channels to reduce the hydraulic resistance. Our estimate suggests that the plant may have sufficient capacity to produce 130 ML/d net over a 24-hour period without implementing this recommendation, however doing so will improve reliability of the filtration system at 130 ML/d net flows and provide additional peace of mind to the operators and is therefore recommended. (Sections 2.3.3, 3.5.5, 4.1.8, and 4.5)

15. Install the fourth high lift pump to provide redundancy for flows at 130 ML/d (net). (Section 4.1.10)

Investigation recommendations

16. Verify the entirety of the clearwell and reservoir monitoring systems, since the SCADA data and the record drawings suggest an inaccuracy as discussed above. Verification should account for at minimum: floor elevations, location of level monitoring instruments, instrument calibration and bandwidth verification, SCADA tags, and monitoring check using field water level measurements.

17. Investigate the turbidity and total organic carbon levels in the filtered water effluent during low production flows and during the targeted production flows. (Section 4.3.2)

Additional investigation recommendations possibly requiring capital works

18. Investigate options and to alleviate the occasional algae build-up on the travelling water screens. One consideration includes intermittent flushing with chlorinated water (noting that this would require dechlorination prior to disposal to Lake Ontario). This recommendation is made based on past algae build up issues, and in recognizing that increased raw water flows may result in an increased accumulation rate during an algae build up episode. Also it is possible that chlorine flushing would only kill the algae and not solve the build-up problem. Further investigation is required. (Sections 2.1.2 and 4.1.2)

19. Investigate the presence, extent, and mitigative measures of vibrations in the High Lift Pumping Station during net production flows at 130 ML/d. If vibrations are determined to be a significant issue, suggested recommendations may include: altering the catwalk's resonant frequency through structural adjustments or additional supports; and consideration of installing a decibel meter.(Sections 3.5.7 and 4.1.10)

20. Investigate and consult with the Region to consider either adding redundancy by installing an additional chlorinator or replacing the chlorination system altogether (operators have noted that it is aging). The 2015 data suggests that the chlorine is still being used to quench excess ozone at the reservoir inlet injection location, and thus both the ozone and chlorine systems should be optimized to ensure efficient use of chemical resources as well as of the capacity of the chlorine system.

Commissioning recommendation

21. After implementation of recommendations, and in agreement with MOECC, perform a final full-scale site acceptance test to commission the plant. Ideally, the test should should allow for a minimum of two filter backwashes and necessary recovery time to replenish clearwell and reservoir storage levels and allow the system to stabilize.. The test should also include turning on all chemical systems, particularly those that were not operational during this test (i.e. Filter Aid Polymer Type 2, Alum, and Hydrogen peroxide). The duration of the commissioning test should be determined during the detailed design phase and coordiationed with the distribution system operation to ensure demand requirements are met. Based on our review as documented in this report, the Oakville WPP has sufficient existing capacity to produce 130 ML/day of water. Aside from installing the fourth high lift pump, the capital works recommendations are intended to increase operational flexibility and reliability with the identified bottlenecks in the ozone-to-filter conveyance and the filtration system itself. (Sections 4.1.8 and 4.5)

6. References

ASI Group. Dec. 2014. Oakville WPP Intake Inspection Report. Submitted to Halton Region.

Associated Engineering. Sep. 2003. Oakville WPP Preliminary Design Report. Submitted to Halton Region.

_____. Jun. 2009. Oakville WPP Operations Manual. Submitted to Halton Region.

_____. Jun. 2010. Technical Memorandum 1: Oakville WPP Process Capacity Review. Submitted to Halton Region.

_____. Mar. 2011. Oakville WPP Phase 2 Upgrades Predesign Report. Submitted to Halton Region.

_____. Oct. 2011. Oakville WPP Pilot Study. Submitted to Halton Region.

Edzwald, J.K. March 2014. “Rapid mixing tanks for coagulation: are they needed?” Published in the Journal for the New England water Works Association (vol. 128, no. 1, pp. 1-11).

Finnemore, E.J., J.B. Franzini. 2002. Fluid Mechanics with Engineering Applications (Tenth edition). McGraw Hill.

GHD. Dec. 2014. Technical Baseline Review Memorandum 3 for the Oakville WPP. Submitted to Halton Region.

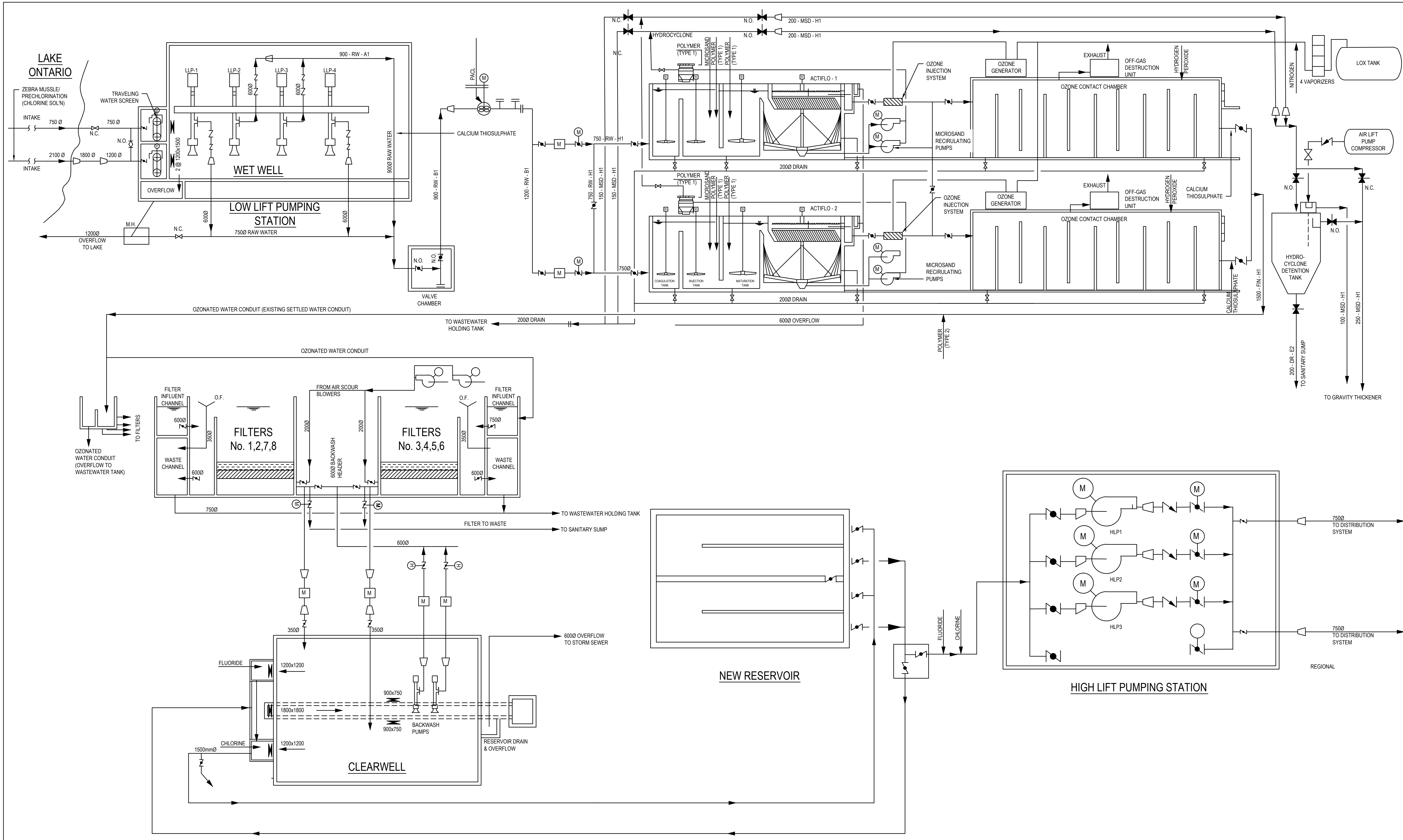
Kawumara, S. 2000. Integrated Design and Operation of Water Treatment Facilities (second edition). John Wiley & Sons, Inc.

Ontario Ministry of the Environment (MOE). 2008. Drinking Water Works Design Guidelines.

_____. 2014. Optimization Guidance Manual for Drinking Water Systems.

Appendices

Appendix A
Process Flow Diagram



PRELIMINARY

No	Revision	Drawn	Job Manager	Project Director	Date

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

GHD Consulting Services Inc.
200 John James Audubon Parkway Suite 101
Amherst NY 14228 USA
T 1 716 748 6620 F 1 716 748 6621
E amhmail@ghd.com W www.ghd.com

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

Appendix B
Lab Reports for Full Scale Test Water Quality Samples

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

0086979-GD-GHD

Project : 086979

13-May-2015

Conestoga Rovers & Associates
Attn : Ariesse MacPhee

651 Colby Drive
Waterloo, Ontario
N2V 1C2, Canada

Phone: 519-884-0510
Fax:519-725-1394

Date Rec. : 06 May 2015
LR Report: **CA14139-MAY15**
Reference: 086979-PO-20-020436
Ozone

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Temperature Upon Receipt °C	pH no unit
1: Analysis Start Date		---	08-May-15
2: Analysis Start Time		---	08:38
3: Analysis Approval Date		--	08-May-15
4: Analysis Approval Time		--	15:51
5: AO/OG		--	6.5-8.5
6: MDL		---	0.05
7: NR 11884-OZ-001	05-May-15 14:30	13.0	7.63
8: NR 11884-OZ-002	05-May-15 14:30	13.0	7.63
9: NR 11884-OZ-003	05-May-15 14:30	13.0	7.62

```
MDL - SGS Method Detection Limit
NR - Not reportable under applicable Provincial drinking water regulations as per client.
```

Ozone sample points 1, 2, 3

Brian Graham B.Sc.
Project Specialist
Environmental Services, Analytical

OnLine LIMS

0000407853

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Project : 086979

LR Report : **CA14139-MAY15**

Method Descriptions

Parameter	SGS Method Code	Reference Method Code
pH	ME-CA-[ENV]EWL-LAK-AN-001	SM 4500

OnLine LIMS

0000407853

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Project : 086979

13-May-2015

Conestoga Rovers & Associates
Attn : Ariesse MacPhee

651 Colby Drive
Waterloo, Ontario
N2V 1C2, Canada

Phone: 519-884-0510
Fax:519-725-1394

Date Rec. : 06 May 2015
LR Report: **CA14140-MAY15**
Reference: 086979-PO-20-020436
Actiflo

Copy: #1

Actiflo

CERTIFICATE OF ANALYSIS

Final Report

Analysis	**1: Analysis Start Date**	**2: Analysis Start Time**	**3: Analysis Approval Date**	**4: Analysis Approval Time**	**8: MDL**	**9: NR 11884-AF-001-005**
Sample Date & Time						05-May-15 14:00
Temperature Upon Receipt [°C]	---	---	--	--	---	10.0
pH [no unit]	08-May-15	13:07	12-May-15	15:57	0.05	7.96
Alkalinity [mg/L as CaCO3]	08-May-15	13:07	12-May-15	15:57	2	93
Conductivity [uS/cm]	08-May-15	13:07	12-May-15	15:57	2	331
Colour [TCU]	08-May-15	08:47	08-May-15	16:14	3	< 3
Total Dissolved Solids [mg/L]	07-May-15	15:09	11-May-15	11:54	30	209
Total Organic Carbon [mg/L]	07-May-15	19:56	08-May-15	13:25	0.2	< 0.2
Aluminum (total) [mg/L]	08-May-15	14:10	11-May-15	15:05	0.01	0.23
Aluminum (dissolved) [mg/L]	08-May-15	14:10	11-May-15	15:05	0.01	0.10

MDL - SGS Method Detection Limit
NR - Not reportable under applicable Provincial drinking water regulations as per client.

Brian Graham B.Sc.
Project Specialist
Environmental Services, Analytical

OnLine LIMS

0000407878

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Project : 086979

LR Report : **CA14140-MAY15**

Method Descriptions

Parameter	SGS Method Code	Reference Method Code
Alkalinity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2320
Carbon by SFA	ME-CA-[ENV]SFA-LAK-AN-009	SM 5310
Colour	ME-CA-[ENV]EWL-LAK-AN-002	SM 2120
Conductivity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2510
Metals in aqueous samples - ICP-OES	ME-CA-[ENV]SPE-LAK-AN-003	SM 3030/EPA 200.7
pH	ME-CA-[ENV]EWL-LAK-AN-001	SM 4500
Solids Analysis	ME-CA-[ENV]EWL-LAK-AN-005	SM 2540C

OnLine LIMS

0000407878

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

0086979-GD-GHD

Project : 086979

LR Report : **CA14140-MAY15**

Quality Control Report

Inorganic Analysis												
Parameter	Reporting Limit	Unit	Method Blank				LCS / Spike Blank			Matrix Spike / Reference Material		
					RPD	Acceptance Criteria	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
						%		Low	High		Low	High
Alkalinity - QCBatchID: EWL0136-MAY15												
Alkalinity	2	mg/L as Ca	< 2		3	10	105	90	110	NA		
Carbon by SFA - QCBatchID: SKA0073-MAY15												
Total Organic Carbon	0.2	mg/L	0.14		3	10	109	90	110	108	75	125
Colour - QCBatchID: EWL0125-MAY15												
Colour	3	TCU	< 3		0	10	100	90	110	NA		
Conductivity - QCBatchID: EWL0136-MAY15												
Conductivity	2	uS/cm	< 2		2	10	98	90	110	NA		
Metals in aqueous samples - ICP-OES - QCBatchID: ESG0033-MAY15												
Aluminum (dissolved)	0.01	mg/L	<0.01		0	20	100	90	110	NV	70	130
Aluminum (total)	0.01	mg/L	<0.01		0	20	100	90	110	NV	70	130
pH - QCBatchID: EWL0136-MAY15												
pH	0.05	no unit	NA		1		100			NA		
Solids Analysis - QCBatchID: EWL0111-MAY15												
Total Dissolved Solids	30	mg/L	0		8	20	99	90	110	NA		

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

0086979-GD-GHD

Project : 086979

15-May-2015

Conestoga Rovers & Associates
Attn : Ariesse MacPhee

651 Colby Drive
Waterloo, Ontario
N2V 1C2, Canada

Phone: 519-884-0510
Fax:519-725-1394

Date Rec. : 06 May 2015
LR Report: **CA14141-MAY15**
Reference: 086979-PO-20-020436
Filtered

Copy: #1

RAW2 denotes filtered water

CERTIFICATE OF ANALYSIS

Final Report

Analysis	**1: Analysis Start Date**	**2: Analysis Start Time**	**3: Analysis Approval Date**	**4: Analysis Approval Time**	**8: MDL**	**9: NR 11884-RAW2-001-005**
Sample Date & Time						05-May-15 13:45
Temperature Upon Receipt [°C]	---	---	--	--	---	10.0
Alkalinity [mg/L as CaCO3]	08-May-15	08:22	08-May-15	14:05	2	96
Colour [TCU]	08-May-15	08:47	08-May-15	16:14	3	4
Turbidity [NTU]	07-May-15	09:48	08-May-15	14:05	0.10	0.41
Conductivity [uS/cm]	07-May-15	15:42	08-May-15	13:59	2	332
Total Dissolved Solids [mg/L]	07-May-15	15:09	11-May-15	11:55	30	194
Total Organic Carbon [mg/L]	14-May-15	16:00	15-May-15	11:13	0.2	2.8
Aluminum (total) [mg/L]	08-May-15	14:10	11-May-15	15:05	0.01	< 0.01
Aluminum (dissolved) [mg/L]	08-May-15	14:10	11-May-15	15:05	0.01	< 0.01

MDL - SGS Method Detection Limit
NR - Not reportable under applicable Provincial drinking water regulations as per client.

Brian Graham B.Sc.
Project Specialist
Environmental Services, Analytical

OnLine LIMS

0000410856

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Method Descriptions

Parameter	SGS Method Code	Reference Method Code
Alkalinity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2320
Carbon by Combustion/Oxidation	ME-CA-[ENV]EWL-LAK-AN-023	SM 5310B
Colour	ME-CA-[ENV]EWL-LAK-AN-002	SM 2120
Conductivity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2510
Metals in aqueous samples - ICP-OES	ME-CA-[ENV]SPE-LAK-AN-003	SM 3030/EPA 200.7
Solids Analysis	ME-CA-[ENV]EWL-LAK-AN-005	SM 2540C
Turbidity	ME-CA-[ENV]EWL-LAK-AN-003	SM 2130

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Quality Control Report

Inorganic Analysis												
Parameter	Reporting Limit	Unit	Method Blank				LCS / Spike Blank			Matrix Spike / Reference Material		
					RPD	Acceptance Criteria	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
						%		Low	High		Low	High
Alkalinity - QCBatchID: EWL0122-MAY15												
Alkalinity	2	mg/L as Ca	< 2		0	10	107	90	110	NA		
Carbon by Combustion/Oxidation - QCBatchID: EWL0237-MAY15												
Total Organic Carbon	1.0	mg/L	< 0.2		0	20	102	90	110	107	75	125
Carbon by SFA - QCBatchID: SKA0073-MAY15												
Total Organic Carbon	0.2	mg/L	0.14		3	10	109	90	110	108	75	125
Colour - QCBatchID: EWL0125-MAY15												
Colour	3	TCU	< 3		0	10	100	90	110	NA		
Conductivity - QCBatchID: EWL0110-MAY15												
Conductivity	2	uS/cm	< 2		0	10	100	90	110	NA		
Metals in aqueous samples - ICP-OES - QCBatchID: ESG0033-MAY15												
Aluminum (dissolved)	0.01	mg/L	<0.01		0	20	100	90	110	NV	70	130
Aluminum (total)	0.01	mg/L	<0.01		0	20	100	90	110	NV	70	130
Solids Analysis - QCBatchID: EWL0111-MAY15												
Total Dissolved Solids	30	mg/L	0		8	20	99	90	110	NA		
Turbidity - QCBatchID: EWL0097-MAY15												
Turbidity	0.10	NTU	< 0.10		2	10	99	90	110	NA		

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

0086979-GD-GHD

Project : 086979

15-May-2015

Conestoga Rovers & Associates
Attn : Ariesse MacPhee

651 Colby Drive
Waterloo, Ontario
N2V 1C2, Canada

Phone: 519-884-0510
Fax:519-725-1394

Date Rec. : 06 May 2015
LR Report: **CA14142-MAY15**
Reference: 086979-PO-20-020436 Raw Water

Copy: #1

Raw water (prior to ramp-up)

CERTIFICATE OF ANALYSIS

Final Report

Analysis	1: Analysis Start Date	2: Analysis Start Time	3: Analysis Approval Date	4: Analysis Approval Time	8: MDL	9: NR 11884-RAW-001-005	10: NR 11884-RAW-003A
Sample Date & Time						05-May-15 08:05	05-May-15 08:05
Temperature Upon Receipt [°C]	---	---	--	--	---	8.0	8.0
Alkalinity [mg/L as CaCO3]	08-May-15	13:07	12-May-15	15:57	2	95	---
Conductivity [uS/cm]	08-May-15	13:07	12-May-15	15:57	2	324	---
Colour [TCU]	08-May-15	08:47	08-May-15	16:14	3	3 <MDL	---
Total Dissolved Solids [mg/L]	07-May-15	15:09	11-May-15	11:55	30	211	---
Bromide [mg/L]	07-May-15	21:58	08-May-15	13:13	0.05	0.05 <MDL	---
Total Organic Carbon [mg/L]	14-May-15	16:00	15-May-15	11:13	0.2	2.3	---
Hardness [mg/L as CaCO3]	08-May-15	10:15	11-May-15	14:32	0.05	105	---
Geosmin [ng/L]	08-May-15	14:57	13-May-15	14:29	3	3 <MDL	---
MIB [ng/L]	08-May-15	14:57	13-May-15	14:29	3	3 <MDL	---
Trihalomethanes (total) [ug/L]	07-May-15	15:44	11-May-15	12:35	0.37	---	0.37 <MDL
Bromodichloromethane [ug/L]	07-May-15	15:44	11-May-15	12:35	0.26	---	0.26 <MDL
Bromoform [ug/L]	07-May-15	15:44	11-May-15	12:35	0.34	---	0.34 <MDL
Chloroform [ug/L]	07-May-15	15:44	11-May-15	12:35	0.29	---	0.29 <MDL
Dibromochloromethane [ug/L]	07-May-15	15:44	11-May-15	12:35	0.37	---	0.37 <MDL
Haloacetic Acids [ug/L]	08-May-15	08:03	11-May-15	12:01	5.3	5.3 <MDL	---
Chloroacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:01	4.7	4.7 <MDL	---

OnLine LIMS

0000410867

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Analysis	1: Analysis Start Date	2: Analysis Start Time	3: Analysis Approval Date	4: Analysis Approval Time	8: MDL	9: NR 11884-RAW-001-005	10: NR 11884-RAW-003A
Bromoacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:01	2.9	2.9 <MDL	---
Dichloroacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:01	2.6	2.6 <MDL	---
Dibromoacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:01	2.0	2.0 <MDL	---
Trichloroacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:01	5.3	5.3 <MDL	---
Bromochloroacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:01	2.0	2.0 <MDL	---

MDL - SGS Method Detection Limit
NR - Not reportable under applicable Provincial drinking water regulations as per client.

Brian Graham B.Sc.
Project Specialist
Environmental Services, Analytical

OnLine LIMS

0000410867

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Method Descriptions

Parameter	SGS Method Code	Reference Method Code
Alkalinity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2320
Anions by IC	ME-CA-[ENV]IC-LAK-AN-001	EPA300/MA300-Ions1.3
Carbon by Combustion/Oxidation	ME-CA-[ENV]EWL-LAK-AN-023	SM 5310B
Colour	ME-CA-[ENV]EWL-LAK-AN-002	SM 2120
Conductivity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2510
Haloacetic Acids	ME-CA-[ENV]GC-LAK-AN-013	EPA 552.3
Metals in aqueous samples - ICP-OES	ME-CA-[ENV]SPE-LAK-AN-003	SM 3030/EPA 200.7
Solids Analysis	ME-CA-[ENV]EWL-LAK-AN-005	SM 2540C
Taste & Odour	ME-CA-[ENV]GC-LAK-AN-012	In-House
Volatile Organics	ME-CA-[ENV]GC-LAK-AN-004	EPA 5030B/8260C

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

0086979-GD-GHD

Project : 086979

LR Report : **CA14142-MAY15**

Quality Control Report

Organic Analysis												
Parameter	Reporting Limit	Unit	Method Blank				LCS / Spike Blank			Matrix Spike / Reference Material		
					RPD	Acceptance Criteria	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
						%		Low	High		Low	High
Haloacetic Acids - QCBatchID: GCM0056-MAY15												
Bromoacetic Acid	2.9	ug/L	2.9#<MDL		ND	30	108	70	130	118	70	130
Bromochloroacetic Acid	2	ug/L	2.0#<MDL		13	30	95	70	130	112	70	130
Chloroacetic Acid	4.7	ug/L	4.7#<MDL		ND	30	107	70	130	113	70	130
Dibromoacetic Acid	2	ug/L	2.0#<MDL		ND	30	96	70	130	103	70	130
Dichloroacetic Acid	2.6	ug/L	2.6#<MDL		10	30	88	70	130	91	70	130
Trichloroacetic Acid	5.3	ug/L	5.3#<MDL		ND	30	80	70	130	109	70	130
Taste & Odour - QCBatchID: GCM0067-MAY15												
Geosmin	3	ng/L	3#<MDL		ND	30	96	60	140	NSS	60	140
MIB	3	ng/L	3#<MDL		ND	30	78	60	140	NSS	60	140
Volatile Organics - QCBatchID: GCM0054-MAY15												
Bromodichloromethane	0.26	ug/L	0.26#<MDL		NSS	30	96	60	130	NSS	50	140
Bromoform	0.34	ug/L	0.34#<MDL		NSS	30	99	60	130	NSS	50	140
Chloroform	0.29	ug/L	0.29#<MDL		NSS	30	98	60	130	NSS	50	140
Dibromochloromethane	0.37	ug/L	0.37#<MDL		NSS	30	101	60	130	NSS	50	140
Inorganic Analysis												
Parameter	Reporting Limit	Unit	Method Blank				LCS / Spike Blank			Matrix Spike / Reference Material		
					RPD	Acceptance Criteria	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
						%		Low	High		Low	High
Alkalinity - QCBatchID: EWL0136-MAY15												
Alkalinity	2	mg/L as Ca	< 2		3	10	105	90	110	NA		
Anions by IC - QCBatchID: DIO0115-MAY15												
Bromide	0.05	mg/L	<0.05		ND	20	102	80	120	100	75	125
Carbon by Combustion/Oxidation - QCBatchID: EWL0237-MAY15												
Total Organic Carbon	1.0	mg/L	< 0.2		0	20	102	90	110	107	75	125
Carbon by SFA - QCBatchID: SKA0073-MAY15												
Total Organic Carbon	0.2	mg/L	0.14		3	10	109	90	110	108	75	125
Colour - QCBatchID: EWL0125-MAY15												
Colour	3	TCU	< 3		0	10	100	90	110	NA		
Conductivity - QCBatchID: EWL0136-MAY15												
Conductivity	2	uS/cm	< 2		2	10	98	90	110	NA		
Solids Analysis - QCBatchID: EWL0111-MAY15												
Total Dissolved Solids	30	mg/L	0		8	20	99	90	110	NA		

0086979-GD-GHD

Project : 086979

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

14-May-2015

Conestoga Rovers & Associates
Attn : Ariesse MacPhee

651 Colby Drive
Waterloo, Ontario
N2V 1C2, Canada

Phone: 519-884-0510
Fax:519-725-1394

Date Rec. : 06 May 2015
LR Report: **CA14143-MAY15**
Reference: 086979-PO-20-020436 Finished

Copy: #1

Finished water

CERTIFICATE OF ANALYSIS

Final Report

Analysis	1: Analysis Start Date	2: Analysis Start Time	3: Analysis Approval Date	4: Analysis Approval Time	6: MDL	7: NR 11884-FIN-Brom-2	8: NR 11884-FIN-Brom-3
Sample Date & Time						05-May-15 14:55	05-May-15 15:30
Temperature Upon Receipt [°C]	---	---	--	--	---	12.0	12.0
Bromate [mg/L]	09-May-15	23:48	13-May-15	15:26	0.003	0.003	0.003

MDL - SGS Method Detection Limit
NR - Not reportable under applicable Provincial drinking water regulations as per client.

Brian Graham B.Sc.
Project Specialist
Environmental Services, Analytical

OnLine LIMS

0000408514

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Method Descriptions

Parameter	SGS Method Code	Reference Method Code
Disinfection Byproducts by IC	ME-CA-[ENV]IC-LAK-AN-006	EPA317

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Quality Control Report

Inorganic Analysis												
Parameter	Reporting Limit	Unit	Method Blank				LCS / Spike Blank			Matrix Spike / Reference Material		
					RPD	Acceptance Criteria	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
						%		Low	High		Low	High
Disinfection Byproducts by IC - QCBatchID: DIO0138-MAY15												
Bromate	0.003	mg/L	<0.003		ND	20	104	80	120	120	75	125

OnLine LIMS

0000408514

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

0086979-GD-GHD

Project : 086979

14-May-2015

Conestoga Rovers & Associates
Attn : Ariesse MacPhee

651 Colby Drive
Waterloo, Ontario
N2V 1C2, Canada

Phone: 519-884-0510
Fax:519-725-1394

Date Rec. : 06 May 2015
LR Report: **CA14144-MAY15**
Reference: 086979-PO-20-020436
Finished

Copy: #1

Finished water

CERTIFICATE OF ANALYSIS

Final Report

Analysis	1: Analysis Start Date	2: Analysis Start Time	3: Analysis Approval Date	4: Analysis Approval Time	8: MDL	9: NR 11884-FIN-001-005	10: NR 11884-FIN-006
Sample Date & Time						05-May-15 14:25	05-May-15 14:25
Temperature Upon Receipt [°C]	---	---	--	--	---	9.0	9.0
Total Coliform [cfu/100mL]	07-May-15	10:15	08-May-15	10:26	---	0	---
E. Coli [cfu/100mL]	07-May-15	10:15	08-May-15	10:26	---	0	---
pH [no unit]	08-May-15	13:07	12-May-15	15:57	0.05	7.89	---
Alkalinity [mg/L as CaCO3]	08-May-15	13:07	12-May-15	15:57	2	85	---
Colour [TCU]	08-May-15	08:47	08-May-15	16:14	3	< 3	---
Bromate [mg/L]	09-May-15	23:48	13-May-15	15:26	0.003	---	0.003 <MDL
Total Organic Carbon [mg/L]	07-May-15	19:56	08-May-15	13:26	0.2	1.7	---
Geosmin [ng/L]	08-May-15	14:57	13-May-15	14:27	3	3 <MDL	---
MIB [ng/L]	08-May-15	14:57	13-May-15	14:27	3	3 <MDL	---
Trihalomethanes (total) [ug/L]	07-May-15	15:44	11-May-15	12:36	0.37	4.5	---
Bromodichloromethane [ug/L]	07-May-15	15:44	11-May-15	12:36	0.26	1.4	---
Bromoform [ug/L]	07-May-15	15:44	11-May-15	12:36	0.34	0.34 <MDL	---
Chloroform [ug/L]	07-May-15	15:44	11-May-15	12:36	0.29	2.2	---
Dibromochloromethane [ug/L]	07-May-15	15:44	11-May-15	12:36	0.37	0.86	---
Haloacetic Acids [ug/L]	08-May-15	08:03	11-May-15	12:02	5.3	5.3 <MDL	---
Chloroacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:02	4.7	4.7 <MDL	---
Bromoacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:02	2.9	2.9 <MDL	---
Dichloroacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:02	2.6	2.6 <MDL	---
Dibromoacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:02	2.0	2.0 <MDL	---
Trichloroacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:02	5.3	5.3 <MDL	---
Bromochloroacetic Acid [ug/L]	08-May-15	08:03	11-May-15	12:02	2.0	2.0 <MDL	---

MDL - SGS Method Detection Limit
NR - Not reportable under applicable Provincial drinking water regulations as per client.

OnLine LIMS

0000409392

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Project : 086979

LR Report : **CA14144-MAY15**

Brian Graham B.Sc.
Project Specialist
Environmental Services, Analytical

OnLine LIMS

0000409392

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Method Descriptions

Parameter	SGS Method Code	Reference Method Code
Alkalinity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2320
Carbon by SFA	ME-CA-[ENV]SFA-LAK-AN-009	SM 5310
Colour	ME-CA-[ENV]EWL-LAK-AN-002	SM 2120
Disinfection Byproducts by IC	ME-CA-[ENV]IC-LAK-AN-006	EPA317
Haloacetic Acids	ME-CA-[ENV]GC-LAK-AN-013	EPA 552.3
Microbiology	ME-CA-[ENV]MIC-LAK-AN-001	OMOE MICROMFDC-E3407A
pH	ME-CA-[ENV]EWL-LAK-AN-001	SM 4500
Taste & Odour	ME-CA-[ENV]GC-LAK-AN-012	In-House
Volatile Organics	ME-CA-[ENV]GC-LAK-AN-004	EPA 5030B/8260C

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Quality Control Report

Parameter	Reporting Limit	Unit	Method Blank		RPD	Acceptance Criteria	LCS / Spike Blank: Spike Recovery (%)	LCS / Spike Blank: Recovery Limits (%)		Matrix Spike / Reference Material: Spike Recovery (%)	Matrix Spike / Reference Material: Recovery Limits (%)	
						%		Low	High		Low	High
Organic Analysis												
Haloacetic Acids - QCBatchID: GCM0056-MAY15												
Bromoacetic Acid	2.9	ug/L	2.9#<MDL		ND	30	108	70	130	118	70	130
Bromochloroacetic Acid	2	ug/L	2.0#<MDL		13	30	95	70	130	112	70	130
Chloroacetic Acid	4.7	ug/L	4.7#<MDL		ND	30	107	70	130	113	70	130
Dibromoacetic Acid	2	ug/L	2.0#<MDL		ND	30	96	70	130	103	70	130
Dichloroacetic Acid	2.6	ug/L	2.6#<MDL		10	30	88	70	130	91	70	130
Trichloroacetic Acid	5.3	ug/L	5.3#<MDL		ND	30	80	70	130	109	70	130
Taste & Odour - QCBatchID: GCM0067-MAY15												
Geosmin	3	ng/L	3#<MDL		ND	30	96	60	140	NSS	60	140
MIB	3	ng/L	3#<MDL		ND	30	78	60	140	NSS	60	140
Volatile Organics - QCBatchID: GCM0054-MAY15												
Bromodichloromethane	0.26	ug/L	0.26#<MDL		NSS	30	96	60	130	NSS	50	140
Bromoform	0.34	ug/L	0.34#<MDL		NSS	30	99	60	130	NSS	50	140
Chloroform	0.29	ug/L	0.29#<MDL		NSS	30	98	60	130	NSS	50	140
Dibromochloromethane	0.37	ug/L	0.37#<MDL		NSS	30	101	60	130	NSS	50	140
Inorganic Analysis												
Parameter	Reporting Limit	Unit	Method Blank		RPD	Acceptance Criteria	LCS / Spike Blank: Spike Recovery (%)	LCS / Spike Blank: Recovery Limits (%)		Matrix Spike / Reference Material: Spike Recovery (%)	Matrix Spike / Reference Material: Recovery Limits (%)	
						%		Low	High		Low	High
Alkalinity - QCBatchID: EWL0136-MAY15												
Alkalinity	2	mg/L as Ca	< 2		3	10	105	90	110	NA		
Carbon by SFA - QCBatchID: SKA0073-MAY15												
Total Organic Carbon	0.2	mg/L	0.14		3	10	109	90	110	108	75	125
Colour - QCBatchID: EWL0125-MAY15												
Colour	3	TCU	< 3		0	10	100	90	110	NA		
Disinfection Byproducts by IC - QCBatchID: DIO0138-MAY15												
Bromate	0.003	mg/L	<0.003		ND	20	104	80	120	120	75	125
pH - QCBatchID: EWL0136-MAY15												
pH	0.05	no unit	NA		1		100			NA		

Microbiological		
Parameter	Method Blank	Duplicate
Microbiology - QCBatchID: BAC9092-MAY15		
E. Coli	ACCEPTED	ACCEPTED
Total Coliform	ACCEPTED	ACCEPTED

Appendix C
Hydraulic Model Development

Appendix C Hydraulic model development

The following provides information on how the hydraulic model for the WPP was developed.

The purpose of the model was to conduct a pre-full scale test analysis of plant hydraulics to determine where hydraulic restrictions would be anticipated at higher flows, which then informed the full scale test plan that was developed. This was accomplished by developing the plant's hydraulic grade line. Due to incomplete information regarding the four ozone-to-filter feeds, the filtration system was modelled as a whole, and as such the hydraulic performance of individual filters was not modelled.

After the full scale test was conducted, hydraulic grade elevations from the model were compared with surface water elevations as measured by SCADA instrumentation during the test. The model resembled the trends observed during the full scale test (e.g. that the reservoir tends to operate at a lower percentage full than the clearwell). Since individual filters were not modelled, the hydraulic issues noted with Filter 1 during the full scale test was not modelled (this issue was already anticipated based on discussion with plant operations).

The equations used in the model were as follows.

For pipe frictional losses, the Hazen-Williams equation was used; this empirical equation is widely used for these purposes and the conditions under which it can be used were satisfied (full pipe flow, fluid is water, 50 mm minimum pipe diameter, and at velocities less than 3 m/s).[1] A roughness coefficient of C=100 was used.

Hazen-Williams (Full pipe flow)
Pipe frictional losses (Metric units)

$$h_L = 10.675 * \frac{L * Q^{1.852}}{C^{1.852} * D^{4.870}}$$

R_H = hydrualic radius = Area / Wetted perimeter
L = length (feet)
D = diameter (in)
Q = flow (gpm)
h_L = headloss (ft)

For open channel frictional losses, the Manning formula was used; this empirical equation is also widely used for these purposes and the conditions under which it can be used (flow ranges similar to that of the Hazen-Williams equation) were also satisfied.[2] A roughness coefficient of n=0.016 was used.

Manning Open Channel Flow
Pipe Frictional losses (Metric units)

$$h_L = V^2 L n^2 / R_H^{4/3}$$

R_H = hydraulic radius
V = velocity (m/s)
n = roughness coefficient

For frictional losses due to fittings (e.g. valves, bends), the minor fitting loss equation was used.

---

[1] Finnemore and Franzini, 2002, p. 299.

[2] Finnemore and Franzini, 2002, p. 299.

Fitting Losses
$h_L = KV^2/2g$

K = fitting loss coefficient
V = velocity (m/s or fps)
g = gravity (9.81 m/s^2)

Sharp and broad-crested weir equations[3] used as appropriate to model depth of flow over a weir under free falling conditions.

Sharp Weir Equation
(Suppressed Rectangular Weir)
Flow depth (Metric units)

$Q \approx 3.32\ L\ H^{3/2}$ (US units)
$Q \approx 1.83\ L\ H^{3/2}$ (metric units)

$H = (Q / 1.83L)^{2/3}$ (metric)

Broad-crested Weir Equation
Flow depth (Metric units)

$Q = L * \text{sqrt}(g) * (2/3)^{3/2} * (H + V/2g)^{3/2}$

$$H = \frac{3Q^{2/3}}{2L\text{sqrt}(g)} - \frac{V}{2g}$$

A hydraulic profile of the plant for the processes between the Low Lift Pumping Station and the High Lift Pumping Station is provided in the figure below.

[3] Finnemore and Franzini, 2002, pp. 530, 534.

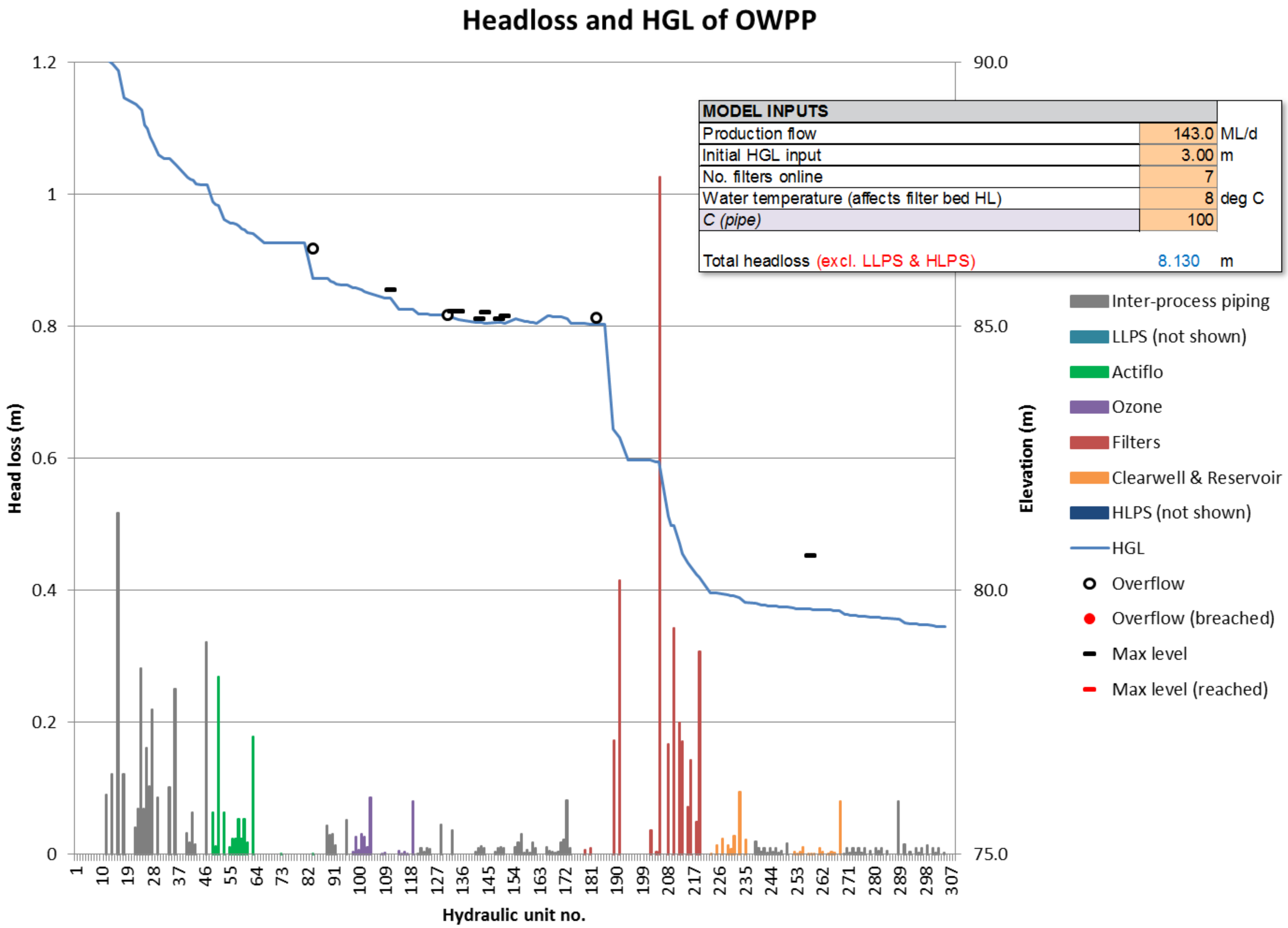


Figure 1 Hydraulic profile of Oakville WPP at 143 ML/d raw water flow

www.ghd.com