XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

XCG File No.: 3-252-57-01
December 6, 2016

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY
SUMMARY OF CAPACITY ASSESSMENT AND RE-RATING POTENTIAL

Draft

Prepared for:

TOWN OF GRAND VALLEY
5 Main Street, North
Grand Valley, Ontario
L9W 5S6

Attention: Jane Wilson

Prepared by:

XCG CONSULTING LIMITED
Suite 300, 2620 Bristol Circle
Oakville, Ontario
L6H 6Z7

TABLE OF CONTENTS

TABLES

FIGURES

APPENDICES

Draft

1. INTRODUCTION

1.1 Background

The Grand Valley WPCP provides treatment for wastewater generated in the community of Grand Valley within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of Environment and Climate Change (MOECC) Certificate of Approval (C of A) No. 9706-7KWQ57, issued on February 2, 2009. The quality and quantity of effluent currently discharged by the existing WPCP is regulated by the C of A. The Grand Valley WPCP has a rated average capacity of 1,244 m^3/d.

XCG recently completed an update to the Assimilative Capacity Study to propose effluent limits associated with an increase in the rated capacity to 2,547 m^3/d. The proposed effluent limit associated with total phosphorus (TP) for this increased capacity was very low at 0.073 mg/L. Consistently achieving such low TP concentrations requires enhanced tertiary treatment, such as dual-stage tertiary filtration or membrane ultrafiltration. Upgrading the Grand Valley WPCP to provide this level of treatment would require a significant capital expenditure.

At this time, the Town would like to investigate the potential to re-rate the existing WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. As such, the Town has retained XCG to undertake a capacity assessment of the Grand Valley WPCP to evaluate the potential for plant re-rating.

1.2 Approach

Re-rating of the Grand Valley WPCP could be completed as a Schedule A activity under the requirements of the Municipal Class Environmental Assessment (Class EA) process (MEA, 2015) as defined in the Class EA document, provided it can meet the following conditions:

"Increase sewage treatment plant capacity beyond existing rated capacity through improvements to operations and maintenance activities only, but without construction of works to expand, modify or retrofit the plant or the outfall to the receiving water body, with no increase to total mass loading to receiving water body as identified in the Certificate of Approval."

As such, final effluent design requirements were developed to establish the effluent concentrations that the existing facility must produce to maintain effluent loadings that are equal to or less than the existing C of A effluent loadings. The capacity of the existing treatment processes was evaluated based on its ability to treat future projected flows and loads while achieving projected effluent quality requirements.

Draft

1.3 Objectives

XCG was retained by the Town to undertake a capacity assessment of the Grand Valley WPCP to investigate a plant capacity re-rating. The specific objective of this report is to provide a brief summary of the estimated treatment capacity of the Grand Valley WPCP, and to discuss the feasibility of re-rating of the Grand Valley WPCP, including implications of the Municipal Class EA process.

Draft

2. DESIGN BASIS

The future design basis was developed to project raw wastewater flows and loads transferred to the Grand Valley WPCP from the collection system via the Emma St. SPS at several future annual average day flow scenarios. For the purposes of developing this design basis, flows and loadings were developed for three scenarios, details of which are presented briefly below.

- Scenario I: Full completion of planned residential developments;
- Scenario II: A 15% increase above the current C of A rated ADF (1,430 m^3/d); and,
- Scenario III: A 25% increase above the current C of A rated ADF (1,555 m^3/d).

The original design basis, completed November 2015, considered plant operational data collected between 2012 and 2014 (XCG, 2015). This design basis was subsequently updated with additional plant operational data collected between January 2015 and May 2016 (XCG, 2016). A summary of the previous and updated design basis is provided in Table 2.1.

Table 2.1 Summary of Design Basis

Parameter	Scenario I		Scenario II		Scenario III	
	Previous	Updated	Previous	Updated	Previous	Updated
Population	2,919	2,919	3,260	3,252	3,536	3,527
ADF	1,276 m^3/d	1,279 m^3/d	1,430 m^3/d		1,555 m^3/d	
MDF	5,828 m^3/d	5,839 m^3/d	6,165 m^3/d	6,169 m^3/d	6,439 m^3/d	6,442 m^3/d
MDF Factor	4.6		4.3		4.1	
PIF	7,811 m^3/d	7,811 m^3/d	8,303 m^3/d	8,291 m^3/d	8,695 m^3/d	8,684 m^3/d
PIF Factor	6.1		5.8		5.6	
BOD_5 Avg. Load Max Load Avg. Conc.	 186 kg/d 353 kg/d 146 mg/L	 200 kg/d 379 kg/d 156 mg/L	 211 kg/d 402 kg/d 148 mg/L	 225 kg/d 427 kg/d 157 mg/L	 232 kg/d 441 kg/d 149 mg/L	 245 kg/d 466 kg/d 158 mg/L
TSS Avg. Load Max Load Avg. Conc.	 239 kg/d 453 kg/d 187 mg/L	 268 kg/d 509 kg/d 210 mg/L	 269 kg/d 512 kg/d 188 mg/L	 298 kg/d 566 kg/d 208 mg/L	 294 kg/d 559 kg/d 189 mg/L	 322 kg/d 613 kg/d 208 mg/L
TKN Avg. Load Max Load Avg. Conc.	 47.9 kg/d 91.1 kg/d 37.6 mg/L	 49.3 kg/d 93.7 kg/d 38.6 mg/L	 53.4 kg/d 104 kg/d 37.4 mg/L	 54.7 kg/d 104 kg/d 38.2 mg/L	 57.9 kg/d 110 kg/d 37.2 mg/L	 59.1 kg/d 112 kg/d 38.0 mg/L
TP Avg. Load Max Load Avg. Conc.	 5.72 kg/d 12.6 kg/d 4.48 mg/L	 6.21 kg/d 13.7 kg/d 4.85 mg/L	 6.43 kg/d 14.2 kg/d 4.50 mg/L	 6.91 kg/d 15.2 kg/d 4.83 mg/L	 7.01 kg/d 15.4 kg/d 4.51 mg/L	 7.48 kg/d 16.5 kg/d 4.81 mg/L

Draft

It is important to note that the projected peak instantaneous flow for each scenario is in excess of the rated capacity of the Emma St. SPS. Analysis suggests the Emma St. SPS may require upgrades to accommodate future flows if peak flows cannot be abated by any I/I reduction strategies. An extensive review of the Emma St. SPS was not conducted as part of this analysis.

Final effluent design requirements were developed to establish the effluent concentrations that the existing facility must produce to maintain effluent loadings that are equal to or less than the existing C of A effluent loadings. Table 2.2 presents the existing effluent loading limits for the C of A rated capacity of 1,244 m^3/d. Also shown are the associated effluent concentration limits for the Grand Valley WPCP at the each of the three scenarios.

Table 2.2 Effluent Concentration Limits for a Re-rated Grand Valley WPCP

Parameter	Existing C of A	Scenario I	Scenario II	Scenario III
	Loading Limit (kg/d)	Concentration Limit (mg/L)	Concentration Limit (mg/L)	Concentration Limit (mg/L)
ADF	1,244 m^3/d	1,273 m^3/d	1,430 m^3/d	1,555 m^3/d
$cBOD_5$	12.4	9.7	8.7	8.0
TSS	12.4	9.7	8.7	8.0
TP	0.19	0.15	0.13	0.12
TAN				
Winter	4.98	3.9	3.5	3.2
Spring	1.24	1.0	0.9	0.8
Summer	0.87	0.7	0.6	0.6
Fall	1.24	1.0	0.9	0.8

Notes:
Existing loading and concentration limits based on monthly average values.

The C of A defines compliance limits for *E. coli* and pH. The limit for *E. coli* is 200 organisms/100 mL and pH must be maintained within the range of 6.0 to 9.5. It is expected that these requirements would remain the same for a re-rated Grand Valley WPCP.

Draft

3. CAPACITY ASSESSMENT SUMMARY

3.1 Capacity of the Existing Grand Valley WPCP

To facilitate comparison between treatment units, the equivalent average day flow capacity of all treatment processes was calculated using information from the updated projected design basis. The attenuation of future peak flows by the existing storm tank was considered, where applicable.

A summary of the equivalent ADF capacity of each treatment processes is given in Table 3.1. A visual representation of this information is included as Figure 3.1. Complete details of the Grand Valley WPCP capacity assessment is included in Appendix A.

Table 3.1 Capacity Assessment Summary

Treatment Unit	Capacity Assessment			
	Average Day Flow	**Maximum Day Flow**	**Peak Flow**	**Equivalent Average Day Flow**
Screens	-	-	9,650 m^3/d	1,555 m^3/d
Grit Removal	-	-	7,680 m^3/d	1,371 m^3/d
Biological Treatment	1,582 m^3/d	-	-	1,582 m^3/d
Oxygenation	1,713 m^3/d	-	-	1,713 m^3/d
Secondary Clarifiers (SOR)	-	-	4,388 m^3/d	952 m^3/d
Secondary Clarifiers (SLR)	-	5,203 m^3/d	-	1,146 m^3/d
Tertiary Filters	-	-	5,300 m^3/d	1,169 m^3/d
UV Disinfection	-	-	7,680 m^3/d	1,371 m^3/d

Draft

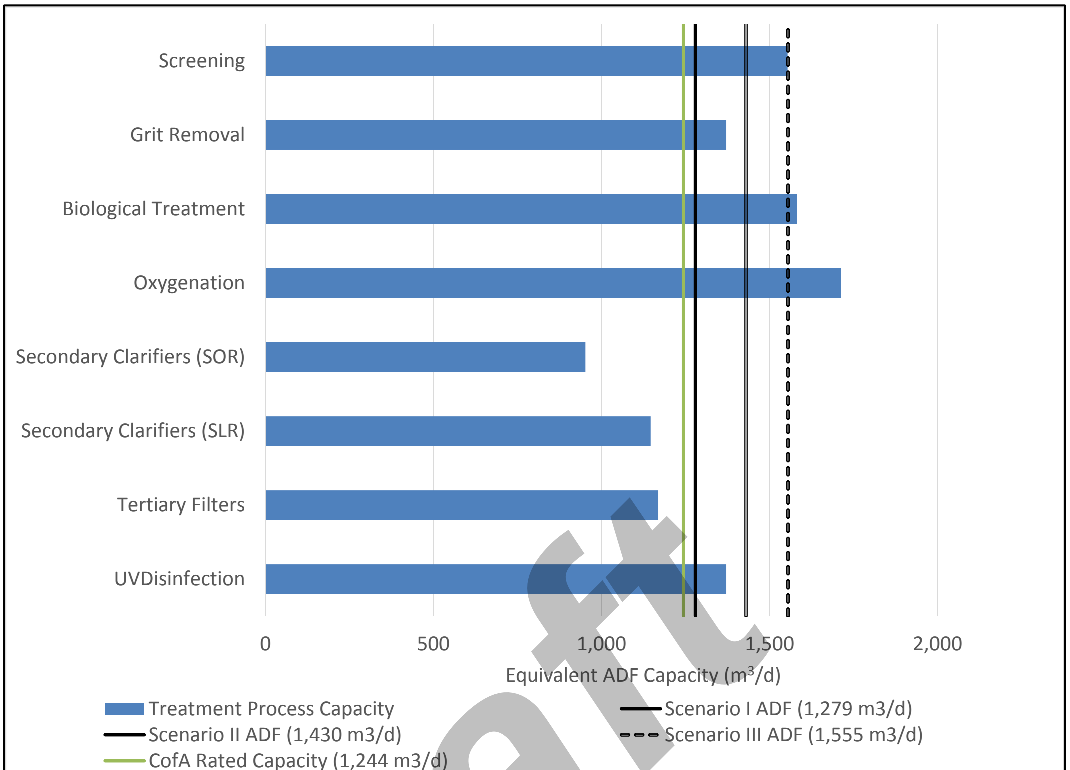


Figure 3.1 Summary of Grand Valley WPCP Capacity

Based on results presented above, the capacity of several treatment processes at the Grand Valley WPCP may be limited by maximum day and peak hour flows to the treatment plant. Projected peak flows are driven by a single extreme peak flow event recorded during the review period (April 2014). Although significantly greater in magnitude that other peak flow events over the review period, this peak flow event cannot be excluded from analysis due, in part, to uncertainty in flow data collected by OCWA at the Grand Valley WPCP, the limited data set which was available for analysis (dating back to only 2012), and the increasing frequency of extreme weather events. As such, based on the estimated capacity of existing treatment processes, re-rating of the Grand Valley WPCP as a Schedule A activity under the Municipal Class EA process is not feasible.

3.2 Impact of Additional Equalization

The construction of additional equalization volume in Grand Valley would reduce peak flows to the Grand Valley WPCP. There are two locations which additional equalization could be constructed in Grand Valley; at the Emma St. SPS and/or onsite at the Grand Valley WPCP. Construction of additional equalization at the Emma St. SPS reduces peak flow in the forcemain between the pumping station and the treatment plant, and through the headworks at the treatment plant. Therefore, to avoid the potential of additional required upgrades to the forcemain, it was assumed equalization volume would be installed at the Emma St. SPS. A thorough analysis and conceptual level design of the construction of additional equalization at the Emma St.

Draft

SPS is included as Appendix B. It is important to note that optimization of the equalization location and volume would be completed during the detailed design.

The possible impact of additional equalization on the estimated equivalent ADF capacity of each treatment process is summarized in Table 3.2. This information is shown visually in Figure 3.2. Results show that the construction of additional equalization can provide sufficient capacity to treat projected Scenario III flows and loads thereby making it feasible to pursue a plant re-rating to increase the rated capacity up to an ADF capacity of 1,555 m^3/d.

It is important to note that this analysis has evaluated the capacity of treatment processes in the liquid treatment train. If plant re-rating is pursued, additional analysis of the solids treatment train would be required, including evaluation of the existing treatment capacity and strategies to handle future sludge flows.

Table 3.2 Impact of Additional Equalization on the Grand Valley WPCP Capacity Assessment

Treatment Unit	Capacity Assessment	
	Existing Equivalent ADF	Equivalent ADF with Additional Equalization
Screens	1,555 m^3/d	3,466 m^3/d
Grit Removal	1,371 m^3/d	2,758 m^3/d
Biological Treatment	1,582 m^3/d	1,582 m^3/d
Oxygenation	1,713 m^3/d	1,713 m^3/d
Secondary Clarifiers (SOR)	952 m^3/d	1,576 m^3/d
Secondary Clarifiers (SLR)	1,146 m^3/d	1,728 m^3/d
Tertiary Filters	1,169 m^3/d	1,763 m^3/d
UV Disinfection	1,371 m^3/d	2,758 m^3/d

Draft

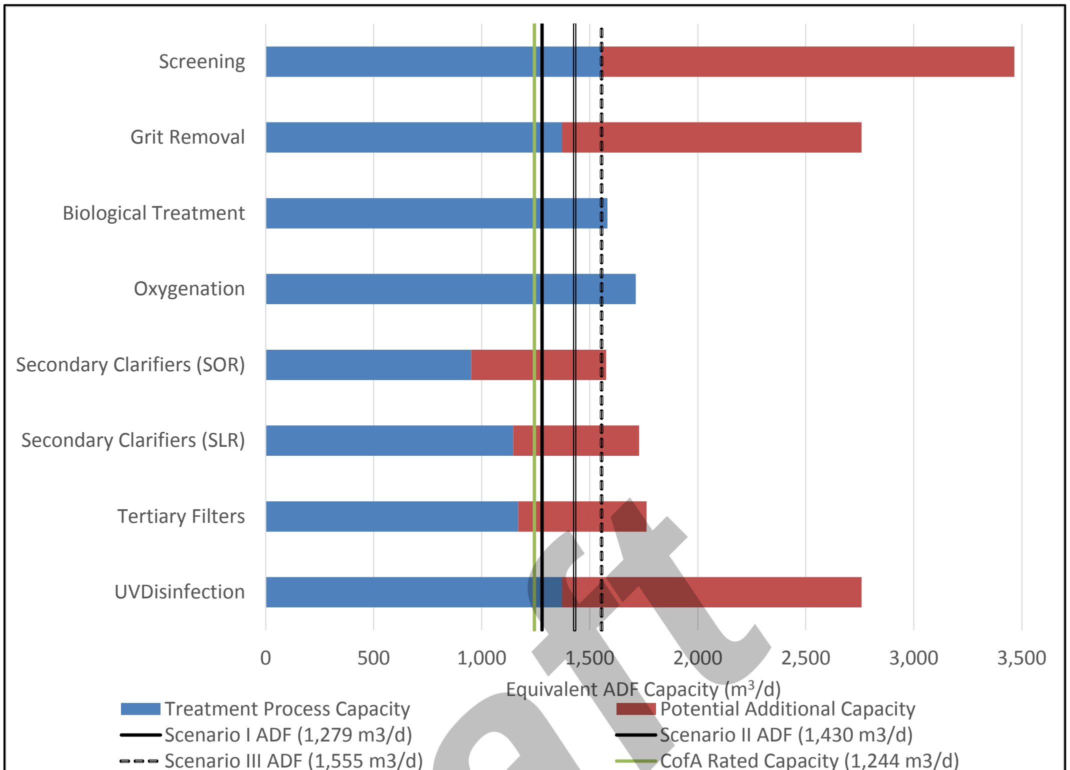


Figure 3.2 Impact of Additional Equalization on the Estimated Treatment Capacity at the Grand Valley WPCP

Installation of additional equalization volume can be carried out as a Schedule B activity under the Municipal Class EA Process as per the following text:

"Establish sewage flow equalization tankage in existing sewer system or at existing sewage treatment plants, or at existing pumping stations for influent and/or effluent control"

As a Schedule B project, Phase 1 and Phase 2 of the Class EA process must be completed prior to implementation of the project (i.e. construction). Brief requirements of each Phase are given below.

Phase 1

During this phase, the problem or opportunity must be identified and described. Projects which are expected to generate significant public interest can also begin the public consultant process.

Phase 2

During this phase, potential alternative solutions will be identified and evaluated. Solutions will consider the size (volume) and location of additional equalization. This Phase will also include mandatory consultation with relevant review agencies and other stakeholders (e.g. MOECC, GRCA, First Nations, etc.) and the public.

At the completion of Phase 2, the entire planning process (i.e. Phase 1 and Phase 2 activities) will be summarized and placed on file for a period of 30 days. A notice of completion will be issued to review agencies and to the public.

Assuming no request for an Order is received during the review period, the Town may proceed with the design and construction of the equalization tank. Detailed design of the equalization tank would need to consider the integration of the equalization tank into the existing infrastructure in the Town of Grand Valley. Specifically, detailed design would establish the following:

- Type and location of the tank (e.g. glass fused steel storage tank located primarily above ground, rectangular cement tank located above ground or below ground, etc.);
- Additional treatment processes required upstream of the equalization tank (e.g. communitor, etc.);
- Regular maintenance required of the equalization tank (e.g. washing, etc.) and provisions to allow for required maintenance;
- Integration into the existing infrastructure, including the reuse of existing pumps and piping where possible; and,
- Evaluation of existing utilities and standby power on the site.

For purposes of this conceptual level design, it is assumed a circular glass fused steel storage tank would be installed at the Emma St. SPS. A conceptual level site layout of equalization at the Emma St. SPS is included as Figure 3.3 and indicates that the site has sufficient space for construction of the equalization tank. Exact dimensions of the equalization tank and the optimal location on the site would be finalized during the detailed design.

Draft

Figure 3.3 Overview of Conceptual Level Layout for Equalization at the Emma St. SPS

Conceptual level capital costs were estimated for the installation of additional equalization volume at the Emma St. SPS. Conceptual level capital costs include installation the equalization tank, as well as allowances for excavation, piping, installation of a tank cleaning mechanism, and electrical works. These additional considerations are critical for the integration of the equalization tank into the existing infrastructure and SCADA system.

For purposes of this investigation, two equalization options were developed and evaluated. Details of each equalization option is included in Table 3.3.

Table 3.3 Summary of Equalization Options

Option	Details
Option 1	• Provide sufficient equalization volume to facilitate re-rating of the Grand Valley WPCP to the Scenario I flows and loads.
Option 2	• Provide sufficient equalization volume to facilitate re-rating of the Grand Valley WPCP to the Scenario III flows and loads.

Conceptual level costs are generally considered to be accurate to -25% to +40%. Actual costs will depend on site specific factors, such as soil and groundwater conditions, the engineering design applied, construction conditions at the time of tendering, and the extent of additional upgrades to the works that may be included in the final design. Capital costs include a 30% allowance for contingency and a 12%

Draft

allowance for engineering and approvals. A summary of conceptual level capital costs for the two equalizations options are summarized in Table 3.4.

Table 3.4 Summary of Conceptual Level Capital Cost Estimates for Equalization at the Emma St. SPS

Item	Option 1 (Sufficient Capacity for Scenario I Flows)	Option 2 (Sufficient Capacity for Scenario III Flows)
General/Miscellaneous	$130,000	$155,000
Equalization Tank	$1,302,000	$1,545,000
Sub Total	$1,432,000	$1,700,000
Contingency (30%)	$429,000	$510,000
Engineering (12%)	$172,000	$204,000
Estimated Equalization Capital Costs (1)	**$2,033,000**	**$2,414,000**

Notes:

1. All costs are conceptual level opinions of probable costs and are considered to be accurate to within -25 to +40 percent and are exclusive of HST.

Draft

4. SUMMARY AND CONCLUSIONS

Based on the capacity assessment of the Grand Valley WPCP, and on projections of future flows and loadings, the capacity of the liquid treatment train is limited by the peak flow treatment capacity. Due to these existing limitations, re-rating the Grand Valley WPCP is not a feasible option at this time.

Through installation of additional equalization at the Emma St. SPS, peak flows to the plant could be reduced, thereby making it feasible to pursue a plant re-rating, potentially up to an ADF capacity of 1,555 m^3/d. Additional analysis of the solids treatment train would be required if plant re-rating is pursued.

Construction of additional equalization volume would be carried out as a Schedule B activity under the Municipal Class EA process, therefore requiring an evaluation of alternative solutions and consultation with the public and with relevant review agencies.

A high level assessment of equalization options was completed, and there appears to be sufficient space at the existing Emma St. SPS to construct additional equalization. Estimated costs for equalization will depend on several factors, including the type of equalization tank selected and additional equipment required to integrate the equalization tank into existing infrastructure.

The estimated costs for equalization ranged from approximately $2.03 million to $2.41 million.

Draft

APPENDIX A
GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY CAPACITY EVALUATION

Draft

XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

XCG File No.: 3-252-57-01
December 6, 2016

GRAND VALLEY WATER POLLUTION CONTROL PLANT CAPACITY EVALUATION

Draft

Prepared for:

TOWN OF GRAND VALLEY
5 Main Street, North
Grand Valley, Ontario
L9W 5S6

Attention: Jane Wilson

Prepared by:

XCG CONSULTANTS LTD.
2620 Bristol Circle, Suite 300
Oakville, Ontario
L6H 6Z7

TABLE OF CONTENTS

TABLES

Draft

FIGURES

APPENDICES

Draft

1. INTRODUCTION

1.1 Background

The Grand Valley WPCP provides treatment for wastewater generated in the community of Grand Valley within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of Environment and Climate Change (MOECC) Certificate of Approval (C of A) No. 9706-7KWQ57, issued on February 2, 2009. The quality and quantity of effluent currently discharged by the existing WPCP is regulated by the C of A. The Grand Valley WPCP has a rated average capacity of 1,244 m^3/d.

XCG recently completed an update to the Assimilative Capacity Study to propose effluent limits associated with an increase in the rated capacity to 2,547 m^3/d. The proposed effluent limit associated with total phosphorus (TP) for this increased capacity was very low at 0.073 mg/L. Consistently achieving such low TP requirements requires enhanced tertiary treatment, such as dual-stage tertiary filtration or membrane ultrafiltration. Upgrading the Grand Valley WPCP to provide this level of treatment would require a significant capital expenditure.

At this time, the Town would like to investigate the potential to re-rate the existing WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. As such, the Town has retained XCG to undertake a capacity assessment of the Grand Valley WPCP to support a plant capacity re-rating.

1.2 Objectives

XCG was retained by the Town to undertake a capacity assessment of the Grand Valley WPCP to investigate a plant capacity re-rating. The specific objectives of this technical memorandum are to:

1. Conduct a review of historic plant performance.
2. Assess the capacity of treatment processes at the Grand Valley WPCP using typical design guideline values, desktop analytical methods, a BioWin™ process model, and results from field testing.
3. Determine the overall capacity of the Grand Valley WPCP.

Draft

2. EXISTING TREATMENT PROCESS

Raw sewage flows from the collection system are conveyed to the Grand Valley WPCP from the Emma St. sewage pumping station (SPS) via a forcemain. The Emma St. SPS is equipped with the following equipment:

- Two variable frequency drive (VFD) pumps (one duty and one standby), each with a rated capacity of 88.9 L/s (7,680 m^3/d).
- One VFD jockey pump with a rated capacity of 29.5 L/s (2,550 m^3/d).
- One wet will, with approximate volume of 125 m^3.

The jockey pump will not operate at peak flows. As such, the capacity of the Emma St. SPS is approximately 7,680 m^3/d. Over the review period (2012 to May 2016) there are no records of raw sewage bypassing at the Emma St. SPS or at the Grand Valley WPCP.

The Grand Valley WPCP receives septage at the septage receiving station. The septage receiving station removes solids from the raw septage using a comgination of grinding, washing, and dewatering. The septage is then discharged to the plant headworks, upstream of the plant screens.

Plant influent raw wastewater flow consists of wastewater from the following sources:

- Raw wastewater from the Emma St. SPS;
- Septage from the onsite receiving station;
- Tertiary filter backwash; and
- Digester supernatant.

Tertiary filter backwash and digester supernatant are transferred back to the head of the plant via an onsite pumping station. All flows are combined at the head of the plant, upstream of the plant headworks.

Headworks at the Grand Valley WPCP consists of a mechanical bar screen and two vortex grit separators. A manual screen also exists in parallel to the mechanical screen, and can be used as required. Headworks effluent flow is discharged to a splitter box, where flow is directed to the aeration tanks, or to a bypass channel. Sustained peak flows in excess of 64 L/s (5,530 m^3/d) for greater than 10 minutes will be directed to the bypass channel and into the 400 m^3 equalization tank. From the equalization tank, flow can be returned to the head of the plant through the onsite pumping station. Flows in excess of the equalization tank capacity are disinfected and discharged. There have been no recorded plant bypasses at the Grand Valley WPCP.

Secondary treatment at the Grand Valley WPCP consists of three aeration tanks and two secondary clarifiers. Oxygen is provided to each aeration tank through fine bubble diffusers. Alum is added immediately upstream of the secondary clarifiers for chemical phosphorus removal. Activated sludge is separated from the treated stream in the secondary clarifiers. Return activated sludge (RAS) is returned to the raw wastewater upstream of the aeration tanks. Waste activated sludge (WAS) is pumped

Draft

to the aerobic digester located onsite. RAS and WAS are pumped from the same location in the secondary clarifier. Overflow from the secondary clarifiers is passed through one of four tertiary filters at the plant. Filter effluent is disinfected using ultraviolet (UV) radiation, then discharged to the Grand River. Waste activated sludge is digested and thickened onsite in the aerobic digester. Thickened sludge is pumped to the onsite biosolids storage tank, then trucked offsite for disposal.

Wastewater flow is measured at several locations at the plant. Raw wastewater from the collection system is metered at the Emma St. SPS. Wastewater flows from septage and the onsite pumping station are separately metered. Collectively, they represent the plant influent flow. Effluent flow from the Grand Valley WPCP is measured by a V-notch weir, downstream of the UV disinfection.

A summary of unit processes is included in Table 2.1, and flow schematic is presented in Figure 2.1.

Table 2.1 Grand Valley WPCP Unit Process Design Information

Unit Process	Design Parameter (1)
Preliminary Treatment	
Screening	
Type	Mechanical and Manual Bar
Number	1 mechanical (duty) 1 bar (standby)
Peak Flow Capacity (mechanical screen)	7,680 m^3/d
Grit Removal	
Type	Vortex
Number	2
Capacity	3,840 m^3/d (each) 7,680 m^3/d (total)
Flow Equalization Tank	
Number	1
Volume	400 m^3
Secondary Treatment	
Bioreactor Tanks	
Type	Rectangular, with fine bubble diffusers
Number	3
Dimensions (each)	25.0 m x 4.0 m x 4.0 m SWD
Operating Liquid Volume	400 m^3 (each) 1,200 m^3 (total)
Secondary Clarifiers	
Number	2
Surface Area	75.4 m^2 (each) 150.8 m^2 (total)
Return Activated Sludge Pumping	
Number	3
Capacity	1,244 m^3/d (each) 3,732 m^3/d (total)

Draft

Unit Process	Design Parameter [1]
Waste Activated Sludge Pumping	
Number	2
Capacity	1,244 m^3/d (each) 2,488 m^3/d (total)
Tertiary Treatment	
Filters	
Type	Continuous up-flow, deep bed, granular media
Backwash	Continuous
Number	Four (4)
Filtration Area	4.65 m^2 (each) 18.6 m^2 (total)
Peak Flow Capacity	5,300 m^3/d
Aeration	
Blowers (Air Supply to Aeration Tanks)	
Number	3 (2 duty, 1 standby)
Capacity	858 m^3/h (each)
Type of Aeration	Fine bubble
Blowers (Air Supply to Primary and Secondary Digester)	
Number	2
Capacity	1,349 m^3/h (each)
Type of Aeration	Coarse bubble
Chemical Treatment	
Phosphorus Removal	
Chemical	Alum
Chemical Storage Tanks	1 x 240 L (day tank) 1 x 9,600 L (main storage tank)
Chemical Dosing Pumps	2 x 13.8 L/h (one duty, one standby) for dosage upstream of the secondary clarifiers 1 x 13.8 L/h for dosage to the equalization tank (when required) 2 x 2.5 L/h for dosage to the tertiary filtration feed channel (when required)
Disinfection	
Disinfection	
Type	UV Disinfection
Capacity	7,680 m^3/d
Sludge Management	
Aerobic Digestion	
Volume	500 m^3 (Primary Digester) 250 m^3 (Secondary Digester)
Digested Sludge Storage Tank	
Number	1
Capacity	2,200 m^3

Notes:
SWD - side water depth
TDH - total dynamic head
1. Based on Amended Certificate of Approval Number 9706-7KWQ57, issued February 2, 2009 and Grand Valley Wastewater Treatment Plant Operations Manual (RJ Burnside, 2015).

Draft

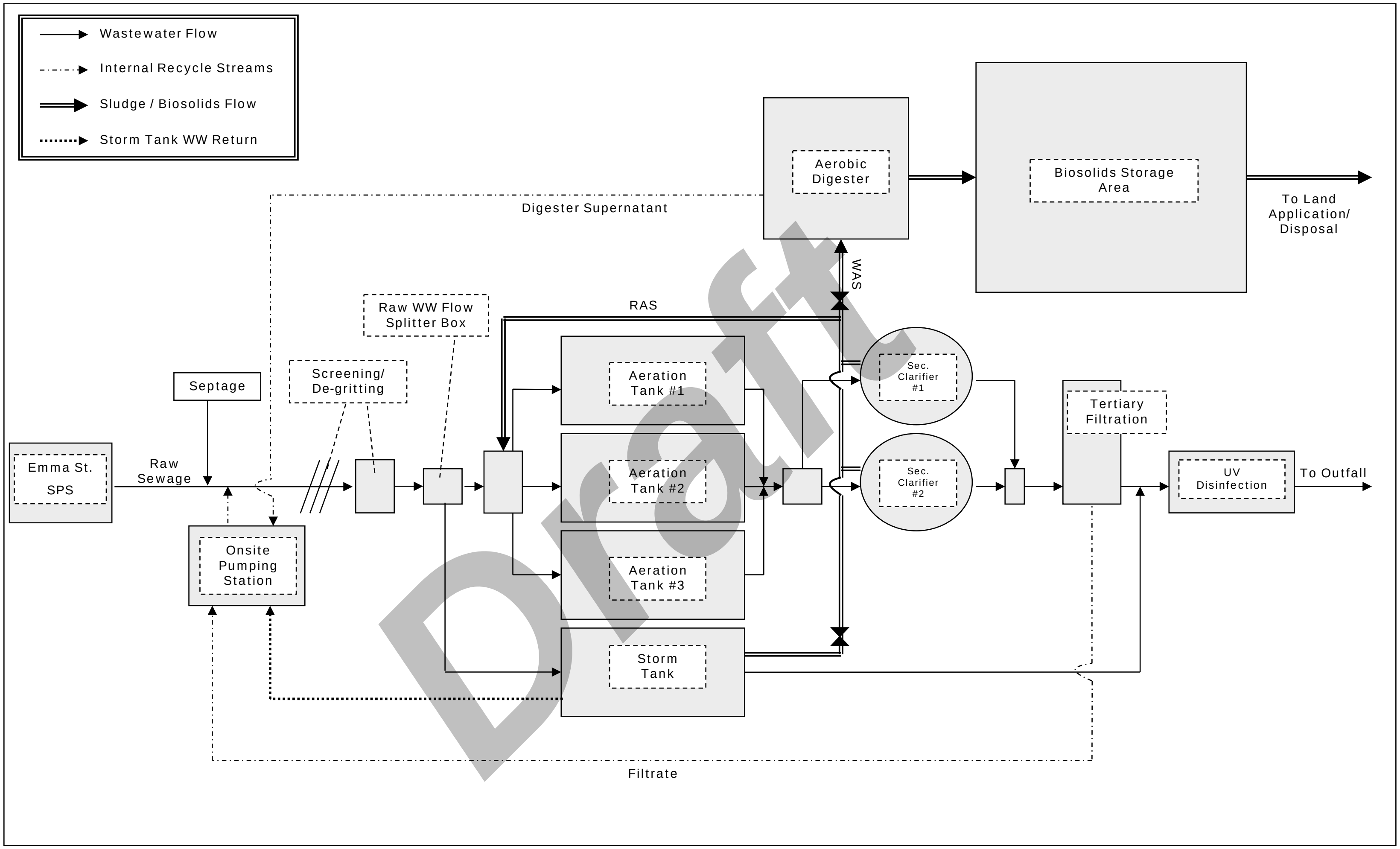


Figure 2.1 Process Flow Schematic – Grand Valley WPCP

3. FINAL EFFLUENT QUALITY

3.1 Treatment Objectives and Compliance Requirements

The Grand Valley WPCP has a rated ADF capacity of 1,244 m^3/d. It is operated under C of A No. 9706-7KWQ57 issued on February 2, 2009. The C of A specifies concentration objectives for carbonaceous biochemical oxygen demand ($cBOD_5$), total suspended solids (TSS), total phosphorus (TP), total ammonia nitrogen (TAN), and *E. coli*. Final effluent is also subject to monthly concentration compliance limits for $cBOD_5$, TSS, TP, TAN, *E. coli*, and pH. Monthly loading compliance limits are also specified for $cBOD_5$, TSS, TP, and TAN. Table 3.1 presents the C of A effluent requirements for the Grand Valley WPCP.

Table 3.1 Amended C of A Objectives and Compliance Limits

Parameter	Effluent Objectives	Effluent Compliance Limits	
	Concentration	Concentration	Total Loading
$cBOD_5$ (1)	8.0 mg/L	10 mg/L	12.4 kg/d
TSS (1)	8.0 mg/L	10 mg/L	12.4 kg/d
TP (1)	0.13 mg/L	0.15 mg/L	0.19 kg/d
TAN (1)			
Winter (Dec. 1 - Mar. 31)	3.0 mg/L	4.0 mg/L	4.98 kg/d
Spring (Apr. 1 - May 31)	0.8 mg/L	1.0 mg/L	1.24 kg/d
Summer (June 1 - Sep. 30)	0.6 mg/L	0.7 mg/L	0.87 kg/d
Fall (Oct. 1 - Nov. 30)	0.8 mg/L	1.0 mg/L	1.24 kg/d
E. coli (2)	100 organisms / 100 mL		
pH	6.0 – 9.5		

Notes:
1. Based on monthly average values.
2. Based on monthly geometric mean density.

3.2 Historical Final Effluent Quality

Table 3.2 and Table 3.3 present historical final effluent concentrations and loadings, respectively, from the Grand Valley WPCP, with maximum monthly average values shown in parentheses. For purposes of this evaluation, data collected between 2012 and May 2016 was analyzed. It is important to note, however, that the accuracy of influent and effluent flow data collected in 2015 cannot be confirmed. As such, effluent loads in 2015 cannot be calculated and have not been presented in Table 3.3. Additional details regarding the accuracy of flow measurement at the Grand Valley WPCP is included in the Updated Design Basis located in Appendix B.

Draft

Table 3.2 Final Effluent Quality over the Review Period (2012 to May 2016)

Parameter	2012	2013	2014	2015	2016 [1]	Effluent Limit	
						Obj.	Limit
$cBOD_5$ (mg/L)	2.06 (2.50)	2.18 (3.75)	2.16 (3.40)	2.04 (2.25)	2.10 (2.50)	8.0	10.0
TSS (mg/L)	2.91 (4.25)	3.16 (7.00)	4.29 (24.8)	2.19 (2.50)	2.00 (2.00)	8.0	10.0
TAN (mg/L)							
Winter (Dec.1 - Mar.31)	0.11 (0.12)	0.56 (2.15)	0.11 (0.13)	0.10 (0.10)	0.10 (0.10)	3.0	4.0
Spring (Apr.1 - May31)	0.10 (0.10)	0.10 (0.10)	0.72 (1.18)	0.10 (0.10)	0.14 (0.18)	0.8	1.0
Summer (June1 - Sep.30)	0.11 (0.13)	0.12 (0.20)	0.11 (0.13)	0.10 (0.10)	- (-)	0.6	0.7
Fall (Oct.1 - Nov.30)	0.10 (0.10)	0.11 (0.13)	0.10 (0.10)	0.10 (0.10)	- (-)	0.8	1.0
TP (mg/L)	0.06 (0.10)	0.07 (0.14)	0.10 (0.32)	0.06 (0.10)	0.05 (0.07)	0.13	0.15
E. coli (organisms / 100 mL)	2.00 (2.00)	2.03 (2.40)	2.28 (9.60)	2.00 (2.00)	2.49 (6.00)	100	100

Notes:

Values in parentheses represent maximum monthly average concentrations.

All samples measured below the detection limit were assumed at the detection limit for purposes of average concentration calculation.

1. Considers data collected from January to May.

Draft

Table 3.3 Final Effluent Loads over the Review Period (2012 to May 2016)

Parameter	2012	2013	2014	2015 [1]	2016	Effluent Compliance Limit [2]
$cBOD_5$ (kg/d)	1.33 (2.47)	1.79 (3.25)	1.82 (6.55)	-	1.73 (2.19)	12.4
TSS (kg/d)	1.91 (3.59)	2.68 (6.08)	5.51 (47.8)	-	1.67 (2.19)	12.4
TAN (kg/d)				-		
Winter (Dec.1 - Mar.31)	0.08 (0.09)	0.52 (1.87)	0.07 (0.07)		0.08 (0.10)	4.98
Spring (Apr.1 - May31)	0.09 (0.11)	0.11 (0.14)	1.21 (2.27)		0.13 (0.19)	1.24
Summer (June1 - Sep.30)	0.05 (0.06)	0.09 (0.13)	0.07 (0.08)		- (-)	0.87
Fall (Oct. 1 - Nov. 30)	0.06 (0.06)	0.10 (0.13)	0.06 (0.07)		- (-)	1.24
TP (kg/d)	0.04 (0.07)	0.06 (0.12)	0.10 (0.62)	-	0.04 (0.05)	0.19

Notes:
Values in parentheses represent maximum monthly loading conditions.
1. Accuracy of 2015 flow data could not be confirmed. As such, effluent loading could not be calculated.
2. Effluent loading compliance evaluated based on the monthly average loading.

Over the review period (2012 to May 2016), effluent concentrations were consistently below the C of A effluent concentration and loading limits, with the exception of one month (April 2014). During this month, the plant reported exceedances in TSS, TP, and TAN.

Figure 3.1, Figure 3.2, Figure 3.3, and Figure 3.4 present the average final effluent concentrations for $cBOD_5$, TSS, TAN, and TP, respectively. The objectives and compliance limits as outlined in the C of A are provided for reference.

Draft

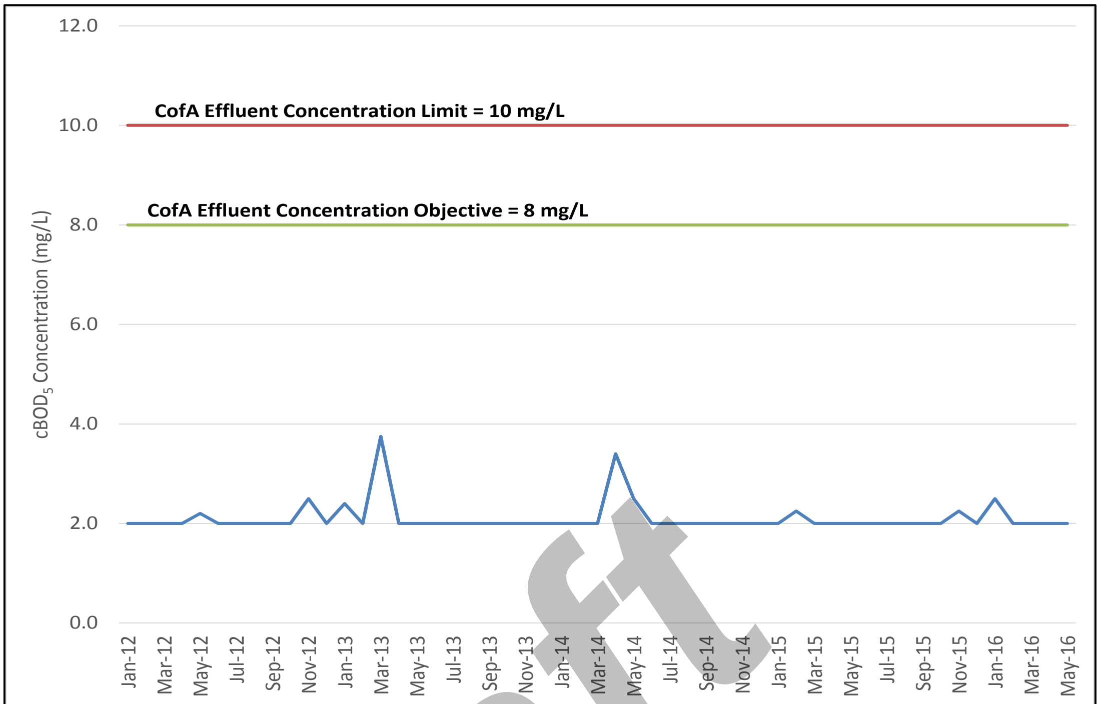


Figure 3.1 Average Monthly Final Effluent cBOD$_5$ Concentration

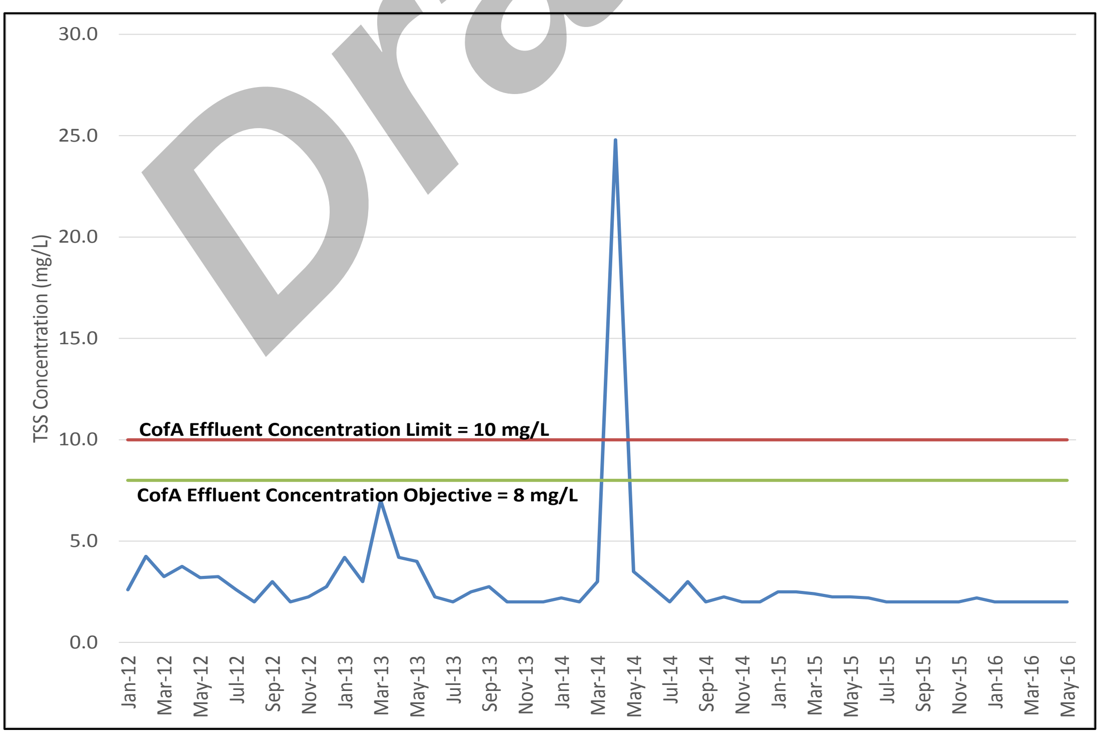


Figure 3.2 Average Monthly Final Effluent TSS Concentration

Draft

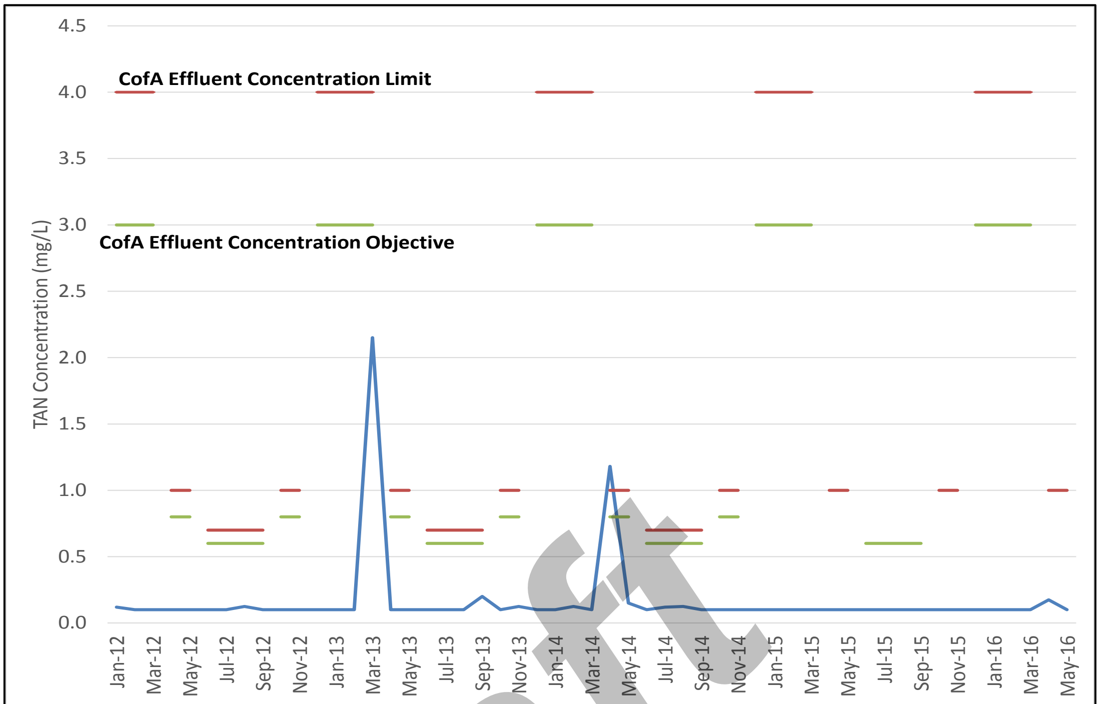


Figure 3.3 Average Monthly Final Effluent TAN Concentration

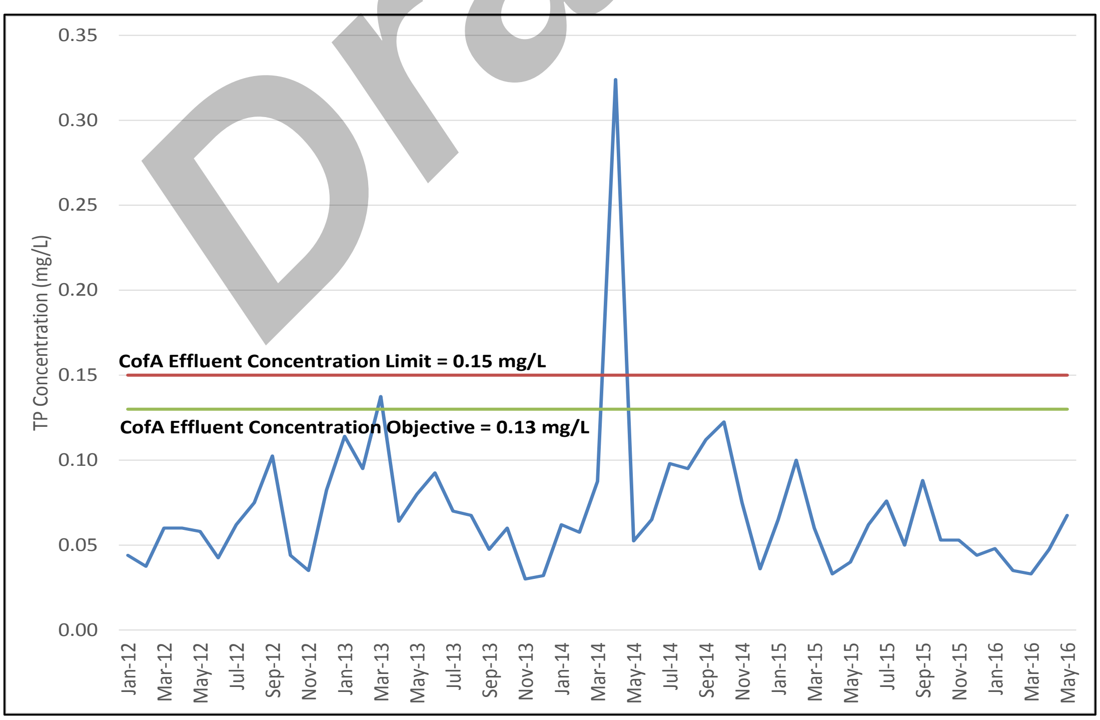


Figure 3.4 Average Monthly Final Effluent TP Concentration

Draft

4. DESIGN BASIS

The future design basis was developed to project raw wastewater flows and loads transferred to the Grand Valley WPCP from the collection system via the Emma St. SPS at several future annual average day flow scenarios. For the purposes of developing this design basis, flows and loadings were developed for three scenarios, details of which are presented briefly below.

- Scenario I: Full completion of planned residential developments;
- Scenario II: A 15% increase above the current C of A rated ADF (1,430 m^3/d); and,
- Scenario III: A 25% increase above the current C of A rated ADF (1,555 m^3/d).

The original design basis, completed November 2015, considered plant operational data collected between 2012 and 2014 (XCG, 2015). This design basis was subsequently updated with additional plant operational data collected between January 2015 and May 2016 (XCG, 2016). A summary of the previous and updated design basis is provided as Table 4.1. Additional details regarding the development of the previous design basis and the updated design basis are provided in Appendix A and Appendix B, respectively.

Draft

Table 4.1 Summary of Design Basis

Parameter	Scenario I		Scenario II		Scenario III	
	Previous	Updated	Previous	Updated	Previous	Updated
Population	2,919	2,919	3,260	3,252	3,536	3,527
ADF	1,276 m^3/d	1,279 m^3/d	1,430 m^3/d		1,555 m^3/d	
MDF	5,828 m^3/d	5,839 m^3/d	6,165 m^3/d	6,169 m^3/d	6,439 m^3/d	6,442 m^3/d
MDF Factor	4.6		4.3		4.1	
PIF	7,811 m^3/d	7,811 m^3/d	8,303 m^3/d	8,291 m^3/d	8,695 m^3/d	8,684 m^3/d
PIF Factor	6.1		5.8		5.6	
BOD_5						
Avg. Load	186 kg/d	200 kg/d	211 kg/d	225 kg/d	232 kg/d	245 kg/d
Max Load	353 kg/d	379 kg/d	402 kg/d	427 kg/d	441 kg/d	466 kg/d
Avg. Conc.	146 mg/L	156 mg/L	148 mg/L	157 mg/L	149 mg/L	158 mg/L
TSS						
Avg. Load	239 kg/d	268 kg/d	269 kg/d	298 kg/d	294 kg/d	322 kg/d
Max Load	453 kg/d	509 kg/d	512 kg/d	566 kg/d	559 kg/d	613 kg/d
Avg. Conc.	187 mg/L	210 mg/L	188 mg/L	208 mg/L	189 mg/L	208 mg/L
TKN						
Avg. Load	47.9 kg/d	49.3 kg/d	53.4 kg/d	54.7 kg/d	57.9 kg/d	59.1 kg/d
Max Load	91.1 kg/d	93.7 kg/d	104 kg/d	104 kg/d	110 kg/d	112 kg/d
Avg. Conc.	37.6 mg/L	38.6 mg/L	37.4 mg/L	38.2 mg/L	37.2 mg/L	38.0 mg/L
TP						
Avg. Load	5.72 kg/d	6.21 kg/d	6.43 kg/d	6.91 kg/d	7.01 kg/d	7.48 kg/d
Max Load	12.6 kg/d	13.7 kg/d	14.2 kg/d	15.2 kg/d	15.4 kg/d	16.5 kg/d
Avg. Conc.	4.48 mg/L	4.85 mg/L	4.50 mg/L	4.83 mg/L	4.51 mg/L	4.81 mg/L

Draft

It is important to note that the projected peak instantaneous flow for each scenario is in excess of the rated capacity of the Emma St. SPS. Analysis suggests the Emma St. SPS may require upgrades to accommodate future flows if peak flows cannot be abated by any I/I reduction strategies. An extensive review of the Emma St. SPS was not conducted as part of this analysis. Additional details regarding projected peak flow analysis is available in Appendix A and Appendix B.

Re-rating of the Grand Valley WPCP could be completed as a Schedule A activity under the requirements of the Municipal Class Environmental Assessment (Class EA) process (MEA, 2015) as defined in the Class EA document, provided it can meet the following conditions:

"Increase sewage treatment plant capacity beyond existing rated capacity through improvements to operations and maintenance activities only, but without construction of works to expand, modify or retrofit the plant or the outfall to the receiving water body, with no increase to total mass loading to receiving water body as identified in the Certificate of Approval."

As such, final effluent design requirements were developed to establish the effluent concentrations that the existing facility must produce to maintain effluent loadings that are equal to or less than the existing C of A effluent loadings.

Table 4.2 presents the existing effluent loading limits for the C of A rated capacity of 1,244 m^3/d. Also shown are the associated effluent concentration limits for the Grand Valley WPCP at each of the three scenarios.

Table 4.2 Effluent Concentration Limits for a Re-rated Grand Valley WPCP

Parameter	Existing C of A	Scenario I	Scenario II	Scenario III
	Loading Limit (kg/d)	Concentration Limit (mg/L)	Concentration Limit (mg/L)	Concentration Limit (mg/L)
ADF	1,244 m^3/d	1,273 m^3/d	1,430 m^3/d	1,555 m^3/d
$cBOD_5$	12.4	9.7	8.7	8.0
TSS	12.4	9.7	8.7	8.0
TP	0.19	0.15	0.13	0.12
TAN				
Winter	4.98	3.9	3.5	3.2
Spring	1.24	1.0	0.9	0.8
Summer	0.87	0.7	0.6	0.6
Fall	1.24	1.0	0.9	0.8

Notes:
Existing loading and concentration limits based on monthly average values.

The C of A defines compliance limits for *E. coli* and pH. The limit for *E. coli* is 200 organisms/100 mL and pH must be maintained within the range of 6.0 to 9.5. It is expected that these requirements would remain the same for a re-rated Grand Valley WPCP.

Draft

5. HISTORICAL REVIEW AND CAPACITY ASSESSMENT

5.1 Basis for Evaluation

A review of the current performance of each unit process at the Grand Valley WPCP, along with typical design guideline values, were used to assess the capacity and performance of each major unit process. The unit process review incorporated the plant operations manual, plant design brief, and plant performance communicated through annual reports and operational data from the period of 2012 to May 2016.

The process capacity assessment was performed using traditional desktop analytical methods, historical plant operational data, plant design criteria, process modelling, and approved C of A capacities, as well as typical design guidelines. For the purposes of the desktop capacity assessment, the design influent raw wastewater characteristics used are those developed in the design basis presented in Table 4.1.

The capacity assessment of the Grand Valley WPCP unit processes were conducted using the following assumptions:

- All tanks and treatment equipment will be online;
- Treated effluent must meet the effluent requirements defined in Table 4.2;
- Final effluent must meet the existing C of A treatment requirements for pH and *E. coli*; and
- Future alum dosages will be consistent with historic values.

5.2 Preliminary Treatment

Preliminary treatment at the Grand Valley WPCP consists of screening and grit removal. This section details the performance and capacity assessment of both treatment processes.

Screening Performance and Design Information

Screening is provided by one perforated plate type mechanical screen operating as the duty screen and one manually raked bar screen operating in stand-by. The mechanical screen has a rated capacity of 7,680 m^3/d based on the CofA and operations manual (RJ Burnside, 2015). Screenings are collected and compacted then transferred to a bin and disposed off-site. The quantity of screenings generated at the Grand Valley WPCP is not measured; therefore the performance of the screens in terms of screenings generation per m^3 of wastewater treated could not be assessed as part of this study.

Grit Removal Performance and Design Information

Grit removal is provided by two vortex grit separators, each 1.83 m in diameter. The rated capacity of each vortex grit separator is 3,840 m^3/d, for a total peak capacity of 7,680 m^3/d. Grit from both separators is collected and compacted then transferred to a bin and disposed off-site. The quantity of grit generated at the Grand Valley WPCP is not measured; therefore the performance of the grit separators in terms of volume generation per m^3 of wastewater treated could not be assessed as part of this study.

Draft

Capacity Assessment of the Grand Valley WPCP Headworks

As previously noted, the rated peak flow capacity of the mechanical screen is approximately 7,680 m^3/d, and the rated capacity of each vortex grit separator is 3,840 m^3/d, providing a total capacity of 7,680 m^3/d.

To evaluate the treatment capacity of the screening and grit removal processes, a detailed hydraulic analysis of the Grand Valley WPCP headworks was completed at projected Scenario III flows. It is important to note that projected peak flows presented in Table 4.1 exceed the existing rated capacity of the Emma St. SPS. Therefore, the Emma St. SPS may require upgrades to accommodate future flows if peak flows cannot be abated by any I/I reduction strategies. An extensive review of the Emma St. SPS capacity was not conducted as part of this review. Further, it was assumed that future peak flows to the Grand Valley WPCP will not be inhibited by the pumping capacity of the Emma St. SPS. Complete results of the hydraulic analysis are included as Appendix C. A brief summary of key points is as follows:

- Due to the existing bypass around the grit removal process, future hydraulic capacity of the plant headworks is expected to be limited by the hydraulic capacity of the mechanical screen channel.
- A detailed relationship between peak flow and headloss across the grit removal process was not available from the manufacturer. It is possible that a portion of future un-equalized Scenario III peak flows will bypass the grit removal process. However, possible bypass around the grit removal treatment process is expected to have a negligible impact on downstream treatment processes.
- There is sufficient hydraulic capacity in the mechanical screening channel to treat un-equalized Scenario III peak flows.

Overall, the estimated treatment capacity of the existing headworks treatment processes exceeds the projected Scenario III peak flows.

5.3 Biological Treatment

Performance and Design Information

The Grand Valley WPCP has three rectangular bioreactors providing a total liquid volume of approximately 1,200 m^3 at the operating water depth of 4.0 m. Over the review period (2012 to May, 2016), only two bioreactors were used, providing a total liquid volume of approximately 800 m^3. The tanks are operated in parallel. RAS is combined with raw wastewater upstream of the bioreactor, and the combined stream is equally split between reactors. Channels exist along the length of each bioreactor which allow for the wastewater to be added at several locations. Currently these channels are closed, and all wastewater is charged to the head each bioreactor. Each bioreactor is equipped with a fine bubble diffuser for the provision of oxygen.

For purposes of this evaluation, plant operating data between 2012 and May 2016 was available for analysis. However, the accuracy of both influent and effluent flow measurements in 2015 could not be confirmed and, as such, this operating data has been excluded from the historical analysis of biological treatment at the Grand Valley

Draft

WPCP. Additional details are included in the updated design basis located in Appendix B.

Table 5.1 presents a summary of the bioreactor operating conditions between 2012 and 2016. Where applicable, each value is compared to typical operating values based on the MOECC Design Guidelines for an extended aeration process. It should be noted that operating data were not available for MLVSS concentrations. Where required, the MLVSS:MLSS ratio was assumed to be 0.70 based on the range observed from samples collected during the intensive sampling program (0.67 - 0.70).

Key findings of the bioreactor process review are summarized below:

- Over the review period, the average daily WAS flow rate significantly decreased. As a result, increased solids were retained within the bioreactors, leading to an increase in the observed MLSS concentration, WAS solids concentration, and estimated solids retention time (SRT).
- In 2014 and 2016, the average MLSS concentration (6,459 mg/L and 5,096 mg/L, respectively) was outside typical operating MLSS concentrations of an extended aeration plant (2,000 to 5,000 mg/L). Although MLSS concentrations were high, there was no observed negative impact on the final effluent TSS concentrations.
- The estimated SRT over the review period was calculated from plant records of WAS flows and solids concentrations. During the review period, the estimated SRT ranged from 21.8 days (2012) to 58.2 days (2014). Increased estimated SRT values is a direct result of reduced solids wasting at the plant. High SRTs can contribute to low food to microorganism (F/M_v) conditions in the bioreactor.
- Due to high MLSS concentrations and low influent loads, the average F/M_v ratio over the review period was 0.03 d^{-1}, which is slightly less than the typical design range for an extended aeration treatment plant. Low F/M_v conditions in the bioreactor can promote the growth of filamentous bacteria, which can lead to issues related to sludge bulking.
- The settling characteristics of the mixed liquor, as measured by the SVI, is similar between bioreactors. Despite the high estimated SRT and low F/M_v ratio, mixed liquor in both bioreactors was readily settleable over the review period. There were no significant changes to the settleability over the review period.

Draft

Table 5.1 Summary of Bioreactors Operation during the Review Period (January 2012 to May 2016)

Parameter	2012	2013	2014	2015 (7)	2016	Typical Design Values
Flow to Bioreactors (m^3/d)	735	910	847	-	918	-
Operating Volume (m^3)	800 (1)					-
BOD_5 Load (kg/d)	89.2	96.0	74.1	-	90.5	-
MLSS (mg/L)	3,223	4,525	6,459	-	5,096	2,000 - 5,000 (2) 3,000 - 5,000 (3)
MLVSS (mg/L)	2,256	3,168	4,521	-	3,567	-
Estimated MLVSS:MLSS (4)	0.7					0.7
HRT (hrs)	26.1	21.1	22.7	-	20.9	> 15 (3)
OLR (kg BOD_5/(m^3·d))	0.11	0.12	0.09	-	0.11	0.10 -0.30 (2) 0.17 -0.24 (3)
F/Mv (d^{-1}) (4)	0.05	0.04	0.02	-	0.03	0.04 - 0.10 (3) 0.05 - 0.15 (3)
RAS:ADF Ratio (%)	99	86	41	-	34	50 - 150 (2) 50 - 200 (3)
Estimated WAS Flow (m^3/d)	14.05	10.93	4.05	-	4.73	n/a
WAS Production (kg/d)	118	133	88.8	-	108	n/a
Estimated Yield (kg TSS/kg BOD_5)	1.32	1.39	1.20	-	1.19	-
SRT (days) (5)	21.8	27.2	58.2	-	37.7	20 - 40 (2) > 15 (3)
Effluent TAN (mg/L) (6)	0.10	0.26	0.21	-	0.13	-
Bioreactor 1 SVI	58	47	46	-	-	-
Bioreactor 2 SVI	56	47	46	-	-	-

Notes:

F/Mv – food to micro-organisms ratio
HRT – hydraulic retention time
MLSS / MLVSS – mixed liquor suspended solids / mixed liquor volatile suspended solids
OLR – organic loading rate
RAS – return activated sludge
SRT – solids retention time
SS – suspended solids
WAS – waste activated sludge

1. Only two bioreactors in operation during the review period (2012 to May 2016).
2. Metcalf & Eddy, 2003.
3. MOECC Design Guidelines for Sewage Works (MOECC, 2008) for extended aeration.
4. Assumes a MLVSS:MLSS ratio of 0.70, based on samples collected during the intensive sampling program.
5. Estimated based on available plant solids concentrations and wasting records.
6. The minimum detection limit was 0.1 mg/L. All samples below the minimum detection limit were assumed equal to the minimum detection limit to calculate the average concentration.
7. Accuracy of flow data could not be confirmed. Therefore, 2015 data has not been included in the analysis above.

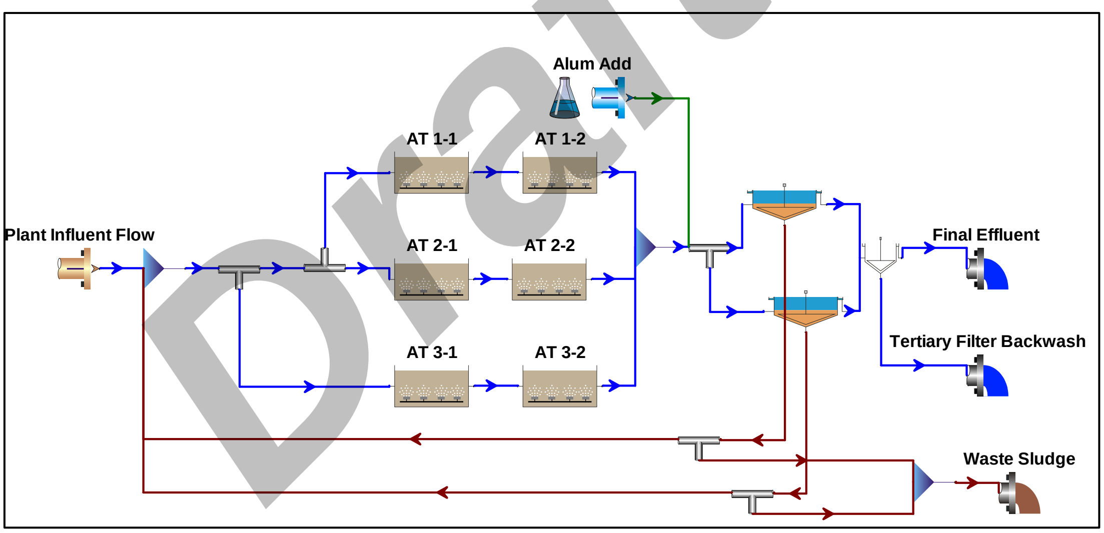
Draft

Capacity Assessment

The biological treatment capacity assessment of the Grand Valley WPCP was completed using BioWin™ process modelling, and based on historic operating conditions, typical design guidelines, and the following assumptions:

- At the biological treatment capacity, all secondary treatment processes (i.e. three aeration tanks and two secondary clarifiers) will be online, and flow will be equally split between all treatment processes;
- Typical DO concentrations of 2.0 mg/L will be maintained in all aeration tanks;
- RAS flow is approximately 100% of the raw influent flow; and
- Future recycle stream flow is approximately 11% of the projected raw influent flow, as estimated from historical plant records.

BioWin™ modelling of the Grand Valley WWTP was conducted to verify the potential biological treatment capacity of the secondary treatment train at the projected Scenario III flows and loads. The BioWin™ model of the existing plant was configured as shown in Figure 5.1.



Draft

Figure 5.1 Schematic of the BioWin™ Model Setup of the Grand Valley WPCP

Using a calibrated and validated BioWin™ model of the Grand Valley WPCP, a minimum design SRT was developed to meet future projected effluent requirements of TAN. Applying a safety factor of 2.3, a design SRT of 15 days was established.

The biological treatment capacity of the Grand Valley WPCP was estimated given the design SRT and given the following assumptions:

- Design yield of 0.96 kg TSS/kg BOD_5, estimated from BioWin™ simulations;
- Target operating MLSS concentration of 3,000 mg/L, estimated to maximize secondary clarifier treatment capacity;
- A bioreactor operating volume of 1,200 m^3, assuming all three bioreactors (at 400 m^3 each) will be online at future flows; and

- A future influent BOD_5 concentration of 158 mg/L, as per projected Scenario III design basis.

Given the above assumptions, the ADF biological treatment capacity of the Grand Valley WPCP was estimated to be 1,582 m^3/d.

To verify this calculation, the calibrated BioWin™ model of the Grand Valley WPCP was tested to evaluate its ability to treat projected average day and maximum month flows and loads at Scenario III. Complete details of the plant modelling and analysis are included in Appendix D. Briefly, results indicate the Grand Valley WPCP is capable of meeting all projected effluent ECA limits at the projected average day and maximum month Scenario III flow, BOD_5 load, and TKN load while operating at an MLSS concentration of approximately 3,000 mg/L.

The following key points should also be highlighted from the assessment of biological treatment performance:

- Results presented in the appendix depend on the accuracy of future projections of BOD_5 and TKN to the plant.
- The capacity of downstream treatment processes (i.e. secondary clarifiers, tertiary filters, UV disinfection) will be impacted by operation of the biological treatment train. Specifically, the biological treatment capacity will increase with increasing MLSS concentrations. However, the secondary clarifier treatment capacity, based on the SLR, will decrease with increasing MLSS concentrations. The specific relationship between the operating MLSS concentration and secondary clarifier treatment capacity was not explored as part of this evaluation. In order to maximize the potential capacity of the secondary clarifiers, a target operating MLSS concentration of 3,000 mg/L was assumed.
- The biological capacity assessment was based on achieving effluent objectives for TAN at projected Scenario III flows and loads. Future effluent targets for all parameters are presented in Table 4.2. Future effluent TP requirements may be approaching the removal limit of existing tertiary filtration equipment installed at the plant.

Draft

5.4 Secondary Clarification and Tertiary Filtration

Secondary Clarifier Historic Performance and Design Information

Secondary clarification at the Grand Valley WPCP is provided by two circular clarifiers. Each clarifier has a diameter of 9.8 m and operates with a side water depth of 4.2 m. The total surface area for settling is approximately 150 m^2. The clarifier is equipped with a sludge collector mechanism, a scum removal system, and covers to prevent the growth of algae on the clarifier surface. RAS and WAS are both pumped from a single pipe at the bottom of the sludge hopper located in the centre of each clarifier.

Table 5.2 summarizes operation of the online secondary clarifier over the review period. As previously noted, the accuracy of both influent and effluent flow measurements in 2015 could not be confirmed and, as such, the summary presented in

Table 5.2 has excluded data collected during 2015. Additional details regarding the plant operating data are included in the updated design basis located in Appendix B.

Table 5.2 Summary of Secondary Clarifier Operation during the Review Period (January 2012 to May 2016)

Parameter	2012	2013	2014	2015 (7)	2016	Typical Design Values
Clarifier Surface Area (m^2)	75 (1)					-
Flow to Secondary Clarifiers (m^3/d)	735	910	847	-	918	-
MDF (m^3/d)	2,780	2,361	4,630	-	2,508	-
PHF (m^3/d)	4,003 (4)	3,400 (4)	5,011 (5)	-	3,612 (4)	-
MLSS (mg/L)	3,223	4,525	6,459	-	5,096	-
RAS:ADF Ratio (%)	99	88	41	-	34	50 - 150 (2) 50 - 200 (3)
Peak Hour SOR ($m^3/(m^2 \cdot d)$)	53.4	45.3	66.8	-	48.2	< 37 (3)
Maximum Day SLR ($kg/(m^2 \cdot d)$) (6)	152	191	429	-	192	< 170 (3)

Notes:

ADF – Average Day Flow
MDF – Maximum Day Flow
PHF – Peak Hour Flow
SOR – Surface Overflow Rate
SLR – Solids Loading Rate
RAS – Return Activated Sludge
MLSS – Mixed Liquor Suspended Solids

1. Operators have indicated only one secondary clarifier in operation during the review period (2012 to May 2016).
2. Metcalf & Eddy, 2003.
3. MOECC Design Guidelines for Sewage Works (MOECC, 2008) for settling after an extended aeration process.
4. Estimated based on the observed MDF and a typical PHF peaking factor of 1.44 (WEF, 2010).
5. Estimated based on effluent flow records from a peak flow event in April 2014.
6. Estimated based on plant records of the MLSS concentration and RAS flow rates.
7. Accuracy of flow information could not be confirmed. Therefore, secondary clarifier performance could not be accurately evaluated.

Draft

Over the review period, estimated peak hour SORs and maximum day SLRs have exceeded typical design values. Secondary clarifier effluent is not currently sampled. As such, the performance of the secondary clarifier during peak flow events cannot be quantified. High estimations of SOR and SLR are due in part to high peak flows observed through the plant and, in 2014, high MLSS concentrations.

Due to tertiary filters located downstream of the secondary clarifiers, final effluent TSS concentrations remained below the C of A compliance limits over the duration of the review period, with the exception of April, 2014. During this month, simultaneous snow melt and rainfall events led to estimations of peak SOR (66.8 $m^3/(m^2 \cdot d)$) and

SLR (429 kg/(m^2·d)) in excess of typical design values. Due to operational issues, the second secondary clarifier could not be brought online during the peak flow event observed in April 2014. Operations staff have indicated that plugging or blinding of filters due to high TSS concentrations has not been a consistent issue during the review period (January 2012 to May 2016).

Tertiary Filtration Historic Performance and Design Information

Tertiary filtration at the Grand Valley WPCP is accomplished by four continuous up-flow, deep bed, granular media filters. Each filter has a filtration area of 4.65 m^2, for a total filtration area of 18.6 m^2.

The filters have a design peak flow capacity of 5,300 m^3/d, as detailed in the operations manual (R.J. Burnside, 2015). The design filter influent TSS and TP concentrations are 20 mg/L and 1 mg/L, respectively.

Each filter is backwashed continually. Filter backwash water is collected at the onsite pumping station, and pumped back to the plant headworks. Filters are designed to provide tertiary effluent quality of 10 mg/L or less total suspended solids, and 0.15 mg/L or less total phosphorus. Tertiary filter influent quality was not monitored over the review period. As such, the performance of tertiary filters over the review period could not be evaluated.

Secondary Clarifier and Tertiary Filter Capacity Assessment

The capacity of the secondary clarifiers and tertiary filters was evaluated through stress testing which was conducted at the Grand Valley WPCP from July 12 to 18, 2016. During testing, flows and solid loading to the secondary clarifier and tertiary filters was artificially increased while the performance of each treatment process was carefully monitored. Only half of the secondary clarifier and tertiary filter treatment capacity was brought online during the stress test (i.e. one secondary clarifier and two tertiary filters, respectively). It was assumed capacity between equal unit treatment processes was identical.

Complete results and analysis of the stress testing program is included as Appendix E. A summary of key observations and conclusions is as follows:

- Capacity evaluations of the secondary clarifier typically consist of a peak hour capacity (determined by the SOR) and a maximum day capacity (determined by the SLR). However, as a result of attenuation by the storm tank, peak hour and max day flows at the Grand Valley WPCP are expected to be similar. As such, a 'peak day' capacity of the secondary clarifier based on both SOR and SLR was made using measurements of secondary clarifier effluent TSS and TP concentrations, and on the height and stability of sludge blanket level measurements.
- Using results from both Day 2 and Day 3, capacity of the secondary clarifier was found to be limited by the SOR. Detailed analysis of results from Day 3 of testing identified a period of stable clarifier operation between 10:00 am and 11:00 am, and was characterized by stable secondary clarifier effluent concentrations of TSS and TP, and stable measurements of sludge height. The SOR capacity, estimated from this period of stable operation, is approximately 29.1 m^3/m^2·d.

Draft

- Capacity evaluations of tertiary filters were based on tertiary effluent TSS and TP concentrations. Capacity was found to be limited by the filtration rate, and was estimated to be 3.30 L/m^2·s.

Based on the results of the stress testing, Table 5.3 summarizes the estimated capacities of the selected treatment units.

It is important to note that the clarifier capacity calculated based on the measured SLR assumed an operating MLSS concentration of 3,000 mg/L. This is consistent with previous evaluations of the biological treatment capacity at the Grand Valley WPCP. Operating MLSS concentrations in excess of 3,000 mg/L would simultaneously increase the biological treatment capacity and decrease the secondary clarifier treatment capacity as evaluated by the SLR. Historically, the plant has operated at MLSS concentrations from approximately 2,500 mg/L to greater than 8,000 mg/L. As flows increase, operating at high MLSS concentrations in the future may result in the clarifier being limited by the SLR to a peak capacity less than 4,388 m^3/d.

Table 5.3 Estimated Secondary Clarifier and Tertiary Filter Operating Capacity

Treatment Process	Limiting Factor	Estimated Capacity
Secondary Clarification		
Peak Hour	SOR (29.1 m^3/m^2·d)	4,388 m^3/d
Maximum Day	SLR (153 kg/m^2·d)	5,203 m^3/d (1)
Tertiary Filtration		
Peak Hour	Filtration Rate (3.30 L/m^2·s)	5,300 m^3/d

Notes:
1. Assuming a future target operating MLSS concentration of 3,000 mg/L, an ADF of 1,244 m^3/d, and a RAS:ADF ratio of 2:1.

5.5 Oxygenation

Historic Performance and Design Information

Air is supplied to the three bioreactors from three positive displacement air blowers (two duty, one standby). Each blower has a rated capacity of 858 m^3/h.

Each bioreactor is equipped with a fine bubble diffuser assembly. Diffusers are arranged in three identical grids along the bioreactor floor. Piping to each grid has its own butterfly valve to control the amount of air delivered to the grid. Therefore, tapered aeration is possible, but is not practiced at the Grand Valley WPCP.

Currently, the Grand Valley WPCP operates only two of the three existing bioreactors. The target DO concentration in each bioreactor is 4.5 mg/L.

According to the MOECC Design Guidelines (MOECC, 2008), the field oxygen transfer efficiency (FOTE) of fine bubble diffusers is 6 to 15 percent. For the purposes of this report, a FOTE of 9 percent was assumed for the bioreactors. The oxygen demand for the bioreactors was calculated based on the oxygen required for the removal of BOD_5 and for complete nitrification. Table 5.4 presents the historic operating conditions of the aeration system at average and peak loadings. Peak TKN

Draft

loads were estimated from average historical TKN loads and a dry weather peaking factor of 2.1, which was estimated from historical meteorological data. As previously noted, the accuracy of raw influent and final effluent flows from 2015 cannot be confirmed and, as such, Table 5.4 has excluded this data.

Table 5.4 Aeration System Operating Conditions during the Review Period (2012 to May 2016)

Design Parameter	Oxygen Demand	Air Requirement
Average Loading		
Process Requirement (1)	260 kg O_2/d	430 m^3/h
Mixing Requirement (3)	-	439 m^3/h
Bioreactor Air Requirement	439 m^3/h	
Peak Loading		
Process Requirement (2)	401 kg O_2/d	665 m^3/h
Mixing Requirement (3)	-	439 m^3/h
Bioreactor Air Requirement	962 m^3/h	

Notes:
1. Based on an oxygen demand of 1.5 kg O_2/kg BOD_5 + 4.6 kg O_2/kg TKN (MOECC, 2008). Based on average BOD_5 and TKN loadings of 87 kg/d and 27.9 kg/d, respectively.
2. Based on an oxygen demand of 1.5 kg O_2/kg BOD_5 + 4.6 kg O_2/kg TKN (MOECC, 2008). Based on average BOD_5 loading of 87 kg/d and a peak day TKN loading of 58.7 kg/d.
3. Mixing requirements are based on 0.61 L/(m^2·s) for fine bubble diffusers (MOECC, 2008), and considers only two bioreactors in operation.

Results presented in Table 5.4 suggest that two existing blowers have sufficient capacity to handle oxygen demands over the review period.

Capacity Assessment

Table 5.5 presents the equivalent ADF capacity of the Grand Valley WPCP based on the design organic loadings, aeration zone oxygenation requirements, and an assumed FOTE of 9 percent. Based on MOECC Design Guidelines (MOECC, 2008), the aeration capacity is estimated based on maintaining a minimum DO concentration of 2.0 mg/L at the average BOD_5 loading and peak daily TKN loading.

Table 5.5 Oxygenation – Capacity Assessment

Parameter	Estimated Total Plant Capacity
Existing Blowers Firm Capacity	1,716 m^3/h (1)
Equivalent ADF Capacity	1,713 m^3/d (2,3)

Notes:
1. Assuming two blowers operating at the design capacity.
2. Based on an oxygen demand of 1.5 kg O_2/kg BOD_5 + 4.6 kg O_2/kg TKN (MOECC, 2008).
3. Based on design average raw wastewater BOD_5 and TKN concentrations of 158 mg/L and 38.0 mg/L, respectively, and the design raw wastewater dry weather flow factor of 2.1 applied to TKN.

Therefore, the equivalent ADF capacity of the existing blowers is approximately 1,713 m^3/d based on an assumed FOTE of 9 percent.

Draft

5.6 Phosphorus Removal

Historic Performance and Design Information

Currently, the plant uses aluminum sulphate (alum) for phosphorus precipitation and removal. The alum is dripped into the wastewater stream following the aeration tanks, upstream of the secondary clarifiers. The alum is stored in a chemical storage tank with a volume of 9,600 L. Alum from the storage tank is pumped to a 240 L day storage tank prior to dosage into the wastewater stream. From the 2015 Operations Manual, the alum day tank has five chemical feed pumps:

- Two (2) pumps, each with a capacity of 13.8 L/h, to dose upstream of the secondary clarifier;
- One (1) pump with a capacity of 13.8 L/h to dose the equalization tank, as required; and,
- Two (2) pumps, each with a capacity of 2.5 L/h, to dose the filter influent stream.

Currently, alum is only dosed upstream of the secondary clarifiers on a regular basis.

Alum dosage data collected from the annual reports was used for this evaluation. Over this period, the monthly average alum dosages ranged from 47 mg/L to 82 mg/L as $Al_2(SO_4)_3.14H_2O$, with an overall average of 70 mg/L as $Al_2(SO_4)_3.14H_2O$. The MOECC Design Guidelines recommends an alum dosage of 110 mg/L to 225 mg/L as $Al_2(SO_4)_3.14H_2O$. Therefore, alum dosages have been lower than the MOECC Design Guidelines typical range. During the review period, the monthly average final effluent TP concentration exceeded the CofA limit on only one occasion (April 2014). The average effluent TP concentration between January 2012 and May 2016 was 0.07 mg/L, indicating that, on average, the plant has operated with a chemical dosage sufficient to meet the current effluent phosphorus objective.

Capacity Assessment

The equivalent ADF capacity of the Grand Valley WPCP based on the alum feed system capacity is presented in Table 5.6. The table shows the estimate equivalent ADF capacity at the historical average dosage of 70 mg/L as $Al_2(SO_4)_3 \cdot 14H_2O$.

Table 5.6 Phosphorus Removal – Capacity Assessment

Parameter	Estimated Capacity
Existing Feed Pumps Total Capacity	16.3 L/h (1)
Equivalent ADF Capacity at historical Alum Dose (2)	3,670 m^3/d

Notes:
1. Combined capacity of the chemical feed pumps upstream of the secondary clarifiers and upstream of the tertiary filters.
2. Based on the historic alum dosage of 70 mg/L, as $Al_2(SO_4)_3 \cdot 14H_2O$ (MOECC, 2008) and alum concentration in solution of 48.5 percent with a specific gravity of 1.335.

Based on Table 5.6, the alum dosage pumps at the Grand Valley WPCP have an equivalent ADF capacity of approximately 3,670 m^3/d at historical dosage rates. This capacity assessment assumes alum will be dosed upstream of both the secondary clarifiers and tertiary filters.

Draft

The alum storage tank has a volume of 9,600 L. At the total feed pump capacity of 16.3 L/h, the storage tank can provided a total of 24.5 days of storage time.

It is important to note that, during the secondary clarifier and tertiary filter stress testing, it was found that alum dosing restrictions at the Grand Valley WPCP had a negative impact on final effluent concentrations of orthophosphate and TP. Specifically, the capacity of the dosing pump at the secondary clarifier limited the alum concentration to approximately 55 mg/L Future removal of orthophosphate can be optimized by increasing the alum dosing capacity to achieve historical (70 mg/L) or typical (110 to 225 mg/L) dosage rates (MOE, 2008) at projected peak flows.

5.7 Disinfection

Historic Performance and Design Information

The existing UV disinfection system is a Trojan UV 3000B Model consisting of two (2) banks of seven (7) modules. Each module contains eight (8) low pressure high intensity UV lamps. The design UV dose is 30.0 mJ/cm^2 at a minimum UV transmittance of 55%. The existing UV disinfection system has a rated capacity of 7,680 m^3/d.

There were no exceedances of the monthly effluent *E. coli* compliance limit over the review period (2012 to May 2016).

Capacity Assessment

Capacity evaluations of the UV disinfection system were based on secondary clarifier and tertiary filter effluent UVT measurements taken during this test, and on previous work which measured the UVT of final effluent and raw influent samples combined in different volumetric ratios. Capacity of the UV disinfection system was estimated to be in excess of the design peak capacity of 7,680 m^3/d.

Draft

6. CAPACITY ASSESSMENT SUMMARY

6.1 Capacity of the Existing Grand Valley WPCP

Results presented in the preceding sections can be used to estimate the treatment capacity of all unit treatment processes at the Grand Valley WPCP. It is important to note, however, that the capacity of different treatment units is determined by different measurements of plant flow (i.e. average day, maximum day, or peak hour). To facilitate comparison between treatment units, the equivalent average day flow capacity of all treatment processes was calculated using information from the updated projected design basis. The attenuation of future peak flows by the existing storm tank was considered where applicable.

A summary of the equivalent ADF capacity of each treatment processes is given in Table 6.1. A visual representation of this information is included as Figure 6.1.

Table 6.1 Capacity Assessment Summary

Treatment Unit	Capacity Assessment			
	Average Day Flow	Maximum Day Flow	Peak Flow	Equivalent Average Day Flow
Screens	-	-	9,650 m^3/d	1,555 m^3/d
Grit Removal	-	-	7,680 m^3/d	1,371 m^3/d
Biological Treatment	1,582 m^3/d	-	-	1,582 m^3/d
Oxygenation	1,713 m^3/d	-	-	1,713 m^3/d
Secondary Clarifiers (SOR)	-	-	4,388 m^3/d	952 m^3/d
Secondary Clarifiers (SLR)	-	5,203 m^3/d	-	1,146 m^3/d
Tertiary Filters	-	-	5,300 m^3/d	1,169 m^3/d
UV Disinfection	-	-	7,680 m^3/d	1,371 m^3/d

Draft

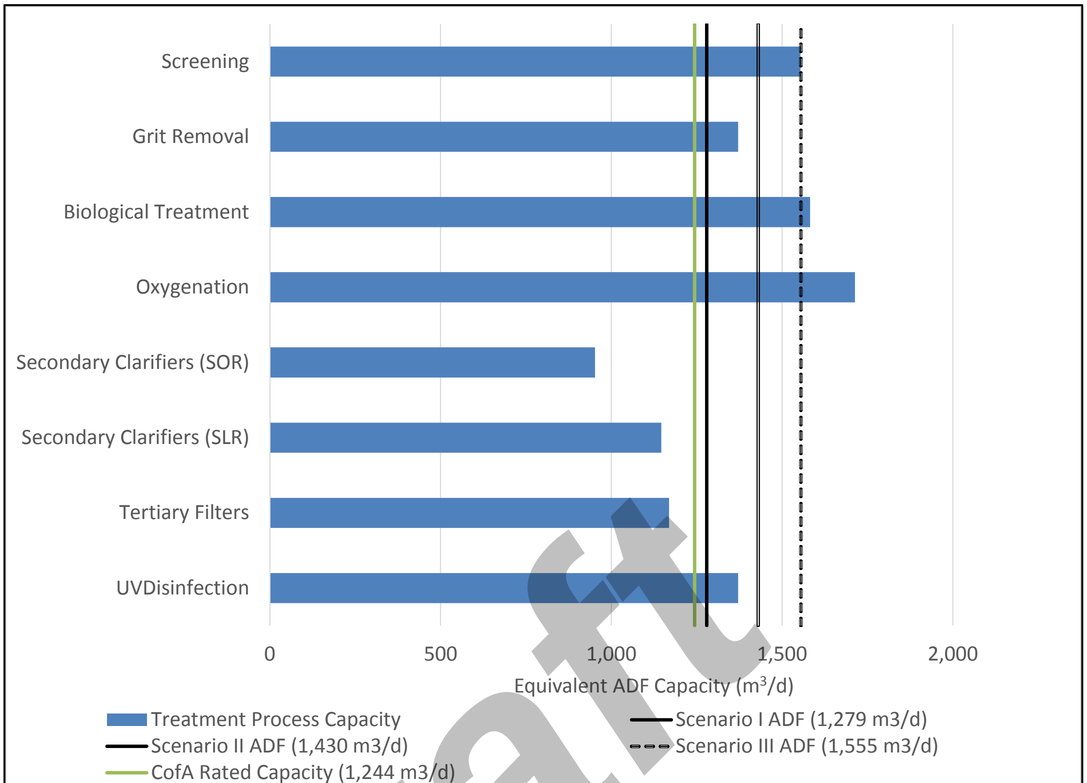


Figure 6.1 Summary of Grand Valley WPCP Capacity

Draft

Based on results presented above, the capacity of several treatment processes at the Grand Valley WPCP may be limited by maximum day and peak hour flows to the treatment plant. Projected peak flows are driven by a single extreme peak flow event recorded during the review period (April 2014). Although significantly greater in magnitude that other peak flow events over the review period, this peak flow event cannot be excluded from analysis due, in part, to uncertainty in flow data collected by OCWA at the Grand Valley WPCP, the limited data set which was available for analysis (dating back to only 2012), and the increasing frequency of extreme weather events. As such, based on the estimated capacity of existing treatment processes, re-rating of the Grand Valley WPCP as a Schedule A activity under the Municipal Class EA process is not feasible.

6.2 Impact of Additional Equalization

Through installation of additional equalization at the Emma St. SPS, peak flows to the plant may be reduced, thereby making it feasible to pursue a plant re-rating to increase the rated capacity, potentially up to an ADF capacity of 1,555 m^3/d. Construction of additional equalization can be completed as a Schedule B activity under the Municipal Class EA process. A thorough analysis and conceptual level design of the construction of additional equalization at the Emma St. SPS is included as Appendix F.

The impact of additional equalization on the estimated equivalent ADF capacity of each treatment process is summarized in Table 6.2. This information is shown visually in Figure 6.2. Results show that the construction of additional equalization at the

Grand Valley WPCP can provide sufficient capacity to treat projected Scenario III flows and loads in the liquid treatment train.

Table 6.2 Impact of Additional Equalization on the Grand Valley WPCP Capacity Assessment

Treatment Unit	Capacity Assessment	
	Existing Equivalent ADF	Equivalent ADF with Additional Equalization
Screens	1,555 m^3/d	3,466 m^3/d
Grit Removal	1,371 m^3/d	2,758 m^3/d
Biological Treatment	1,582 m^3/d	1,582 m^3/d
Oxygenation	1,713 m^3/d	1,713 m^3/d
Secondary Clarifiers (SOR)	952 m^3/d	1,576 m^3/d
Secondary Clarifiers (SLR)	1,146 m^3/d	1,728 m^3/d
Tertiary Filters	1,169 m^3/d	1,763 m^3/d
UV Disinfection	1,371 m^3/d	2,758 m^3/d

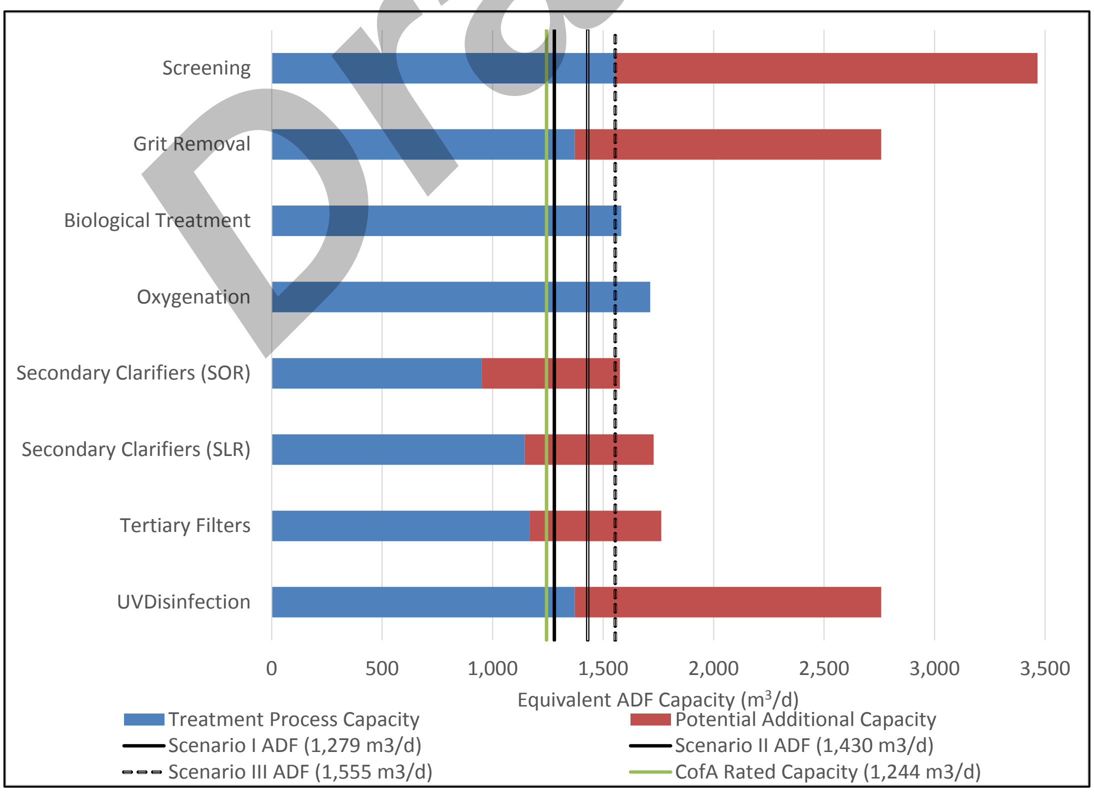


Figure 6.2 Impact of Additional Equalization on the Estimated Treatment Capacity at the Grand Valley WPCP

Draft

7. REFERENCES

1. R.J. Burnside & Associates Limited. Grand Valley Wastewater Treatment Plant Operations Manual. 2015.
2. Ontario Ministry of the Environment. Design Guidelines for Sewage Works. 2008.
3. Metcalf & Eddy. Wastewater Engineering: Treatment and Resource Recovery. Fourth Edition. Toronto. 2003.

Draft

APPENDIX A

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY PROPOSED DESIGN FLOWS AND LOADS

Draft

XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

XCG File No.: 3-252-57-01
November 17, 2015

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY
PROPOSED DESIGN FLOWS AND LOADS

Draft

Prepared for:

TOWN OF GRAND VALLEY
5 Main Street, North
Grand Valley, Ontario
L9W 5S6

Attention: Jane Wilson

Prepared by:

XCG CONSULTING LIMITED
Suite 300, 2620 Bristol Circle
Oakville, Ontario
L6H 6Z7

TABLE OF CONTENTS

TABLE

APPENDICES

Draft

1. INTRODUCTION

1.1 Background

The Grand Valley WPCP provides treatment for wastewater generated in the community of Grand Valley within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of Environment and Climate Change (MOECC) Certificate of Approval (CofA) No. 9706-7KWQ57, issued on February 2, 2009. The quality and quantity of effluent currently discharged by the existing Water Pollution Control Plant (WPCP) is regulated by the CofA. The Grand Valley WPCP has a rated average capacity of 1,244 m^3/d.

XCG Consulting Limited (XCG) recently completed an update to the Assimilative Capacity Study to propose effluent limits associated with an increase in the rated capacity to 2,547 m^3/d. The proposed effluent limit associated with total phosphorus (TP) for this increased capacity was very low at 0.073 mg/L. Consistently achieving such low TP requirements requires enhanced tertiary treatment, such as dual-stage tertiary filtration or membrane ultrafiltration. Upgrading the Grand Valley WPCP to provide this level of treatment would require a significant capital expenditure.

At this time, the Town would like to investigate the potential to re-rate the existing WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. As such, the Town has retained XCG to undertake a capacity assessment of the Grand Valley WPCP to support a plant capacity re-rating.

1.2 Objectives

The specific objectives of this technical memorandum are to:

- Conduct a review of plant raw wastewater flows and loads; and,
- Develop a design basis for future raw wastewater flows and loads.

1.3 Data Sources

The following data sources were used in part to develop projections of plant flows and loads:

- 2012 to 2014 plant flow and quality information;
- Memorandum completed by R.J. Burnside regarding the existing and future service populations of the Grand Valley WPCP (May, 2015);
- East Luther Grand Valley (ELGV) Wastewater Treatment Plant Design Brief (2008);
- ELGV Inflow and Infiltration (I/I) Study Report (July, 2009);
- Grand Valley Wastewater Treatment Plant Operations Manual (July, 2015); and,
- Grand Valley WPCP facility tour (September, 2015).

Draft

2. REVIEW OF RAW WASTEWATER FLOW AND QUALITY

Raw sewage flows from the collection system are conveyed to the Grand Valley WPCP from the Emma St. sewage pumping station (SPS) via a forcemain. The Emma St. SPS is equipped with the following equipment:

- Two variable frequency drive (VFD) pumps (one duty and one standby), each with a rated capacity of 88.9 L/s (7,680 m^3/d);
- One VFD jockey pump with a rated capacity of 29.5 L/s (2,550 m^3/d); and,
- One wet well, with an approximate volume of 125 m^3.

Only one of the above pumps is in operation at a time. As such, the existing peak capacity of the Emma St. SPS is approximately 7,680 m^3/d. Over the review period (2012 - 2014) there were no records of raw sewage bypasses at the Emma St. SPS or at the Grand Valley WPCP.

It is important to note that a condition assessment of the Emma St. SPS was not completed as part of this study. Although the existing capacity of the Emma St. SPS was taken into consideration as part of the review of historic operating conditions, its capacity was not assumed to be a limiting factor when developing future anticipated peak flows at the Grand Valley WPCP.

Draft

2.1 Review of Raw Wastewater Flow over the Review Period (2012 - 2014)

The Grand Valley WPCP currently serves a residential population of approximately 1,752 persons. Influent flow to the Grand Valley WPCP is comprised of:

- Raw wastewater from the Grand Valley sanitary collection system, pumped to the plant via the Emma St. SPS;
- Septage flow from the onsite septage receiving station; and,
- Plant recycle flow (i.e. digester supernatant and filter backwash flow), pumped to the head of the plant from the onsite pumping station.

Flow from each source above is metered separately. Reported total influent flow to the plant is calculated as the sum of flow from each source. In addition, effluent flow is monitored using a V-notch weir. During a tour of plant treatment facilities, operators indicated the accumulation of grit within the magnetic flowmeter measuring flows from the Emma St. SPS led to false high measurements during the review period. As such, plant effluent flow measurements were used as the basis for the evaluation of average raw wastewater flows from the Grand Valley sanitary collection system over the review period (2012 - 2014).

Table 2.1 presents a summary of the estimated collection system raw influent flow and per capita flows to the Grand Valley WPCP. The table includes an estimation of dry weather plant flow and per capita flows, and quantification of the historical I/I observed at the plant. Meteorological data was obtained from the Environment Canada station at Fergus, Ontario. Days were considered dry when no precipitation occurred for that day and three days prior. Only data from May to October was used for dry weather flow analysis.

Table 2.1 Summary of Treated Flow over the Review Period (2012 - 2014)

	Units	2012	2013	2014	Overall (1)
Estimated Service Population	Persons	1,494	1,683	1,752	-
Average Daily Flow	m^3/d	643	821	776	746
Per Capita Flow	L/cap/d	430	488	443	454
Estimated Dry Weather Flow (2)	m^3/d	554	658	620	603
Estimated Per Capita Dry Weather Flow	L/cap/d	371	391	354	372
Estimated Per Capita I/I	L/cap/d	59	97	89	82

Notes:
Estimated flows are based on flow measurements taken at the effluent flow meter over the review period.
1. Represents the average flow over the entire review period (2012 - 2014).
2. Days were considered dry when no precipitation occurred for that day, and two days prior from May to September.

Draft

Results in Table 2.1 indicate that the overall average per capita flow to the Grand Valley WPCP over the review period was 454 L/cap/d, inclusive of I/I. The estimated dry weather per capita flow (372 L/cap/d) is consistent with the typical range of per capita flows of 225 to 450 L/cap/d, exclusive of extraneous flows (MOE, 2008). The calculated per capita I/I was 82 L/cap/d, which is slightly less than the typical design I/I flow of 90 L/cap/d (MOE, 1985).

Summary of Maximum Day Flows during the Review Period (2012 - 2014)

Similar to average day flow analysis, maximum day flows for 2012 and 2013 were estimated from effluent flow meter measurements. In 2014, the maximum day flow event (April 14, 2014) was caused by simultaneous rainfall and snow melt events, and required use of the storm tank to equalize peak flows through the secondary treatment train. Volume accumulated in the storm tank was returned to the head of the plant in the days following the peak flow event. As such, the measured flow at the effluent flow meter is not an accurate representation of total maximum day influent flow in 2014.

As previously discussed, the accumulation of grit at the Emma St. SPS flow meter has caused false high flow measurements over the review period (2012 - 2014). However, during the seven days preceding the peak flow event in 2014, the average percent difference between flows measured at the Emma St. SPS and at the effluent flow meter was 3%. Therefore, it was assumed that flow measured at the Emma St. SPS represents an accurate estimation of total influent flow to the Grand Valley WPCP during the peak flow event recorded in April 2014. A summary of maximum day flows and calculated maximum day factors (MDF) during the review period is shown as Table 2.2.

Results in Table 2.2 indicate the Grand Valley WPCP has been subject to significant peak flows over the review period. Specifically, the extreme peak flows observed in 2014 are attributed to simultaneous snow melt and rain fall events in April 2014. There have been no recorded observations of raw wastewater bypass during the review period.

Table 2.2 Summary Maximum Day Flows over the Review Period (2012 - 2014)

	Units	2012	2013	2014	Overall
Average Daily Flow	m^3/d	643	821	776	746
Maximum Day Flow	m^3/d	2,601	2,254	4,671 (1)	4,671 (1)
MDF	-	4.0	2.8	6.0	6.3

Notes:
Unless otherwise indicated, flows are based on flow measurements taken at the effluent flow meter over the review period (2012 - 2014)
1. Based on Emma St. SPS flow measurements.

Summary of Peak Flows during the Review Period (2012 - 2014)

As discussed, operators have indicated that the accumulation of grit within the magnetic flow meter has contributed to false high measurements of flow from the Emma St. SPS. In 2015, operators began periodically operating the Emma St. SPS pump at capacity to flush any accumulated solids from the magnetic flow meter. Since beginning this practice, operators have reported consistent agreement between influent and effluent flow measurements.

Similarly, it is likely that peak flow periods which occurred during the review period, and which required pumps at the Emma St. SPS to run at or near capacity, would remove any accumulated grit at the magnetic flow meter. Therefore, it was assumed that peak flow data collected from the Emma St. SPS represents an accurate representation of peak flows to the Grand Valley WPCP during the review period (2012 - 2014).

For selected days with high measured effluent flows, measured flow from the Emma St. SPS was further analyzed to understand the existing peak flows to the plant. Specifically, several days from the peak flow event in April 2014 were examined. A SCADA screenshot of Emma St. SPS flows on April 13 and April 14, 2014 is included as Appendix A.

During these days, the observed peak flow from the Emma St. SPS reached approximately 88 L/s, which is near the rated capacity of the SPS. However, detailed analysis of these figures suggests that the observed peak flows are likely related to pump operation at the Emma St. SPS rather than actual raw influent flow to the wet well. Plant operations staff have indicated that the VFD of the large duty pump was programmed to operate between 60 L/s and 90 L/s. As indicated, the capacity of the jockey pump is approximately 29.5 L/s. Influent flow greater than the jockey pump capacity, but less than the minimum programmed operation of the large duty pump is likely the cause of unstable periods of pump operation, characterized by rapid changes in pumping output and cycling of pump on/off cycles. These unstable periods are detailed in the screenshots included in Appendix A. During the morning of April 14, 2014, operations staff modified operation of the VFD control to allow the large pump to operate between 40 L/s and 89 L/s in an attempt to smooth pump output during this high flow event. This can be clearly seen on Figure A.2 in Appendix A. It is recommended the Town conduct further investigation into the PLC programming at the Emma St. SPS to optimize pumping control if required.

Excluding periods of unstable pump operation, the peak flow from the collection system was estimated to be approximately 70 L/s (6,048 m^3/d) during the review period (2012 - 2014).

Evaluation of Plant Recycle and Septage Flows over the Review Period (2012 - 2014)

Decant flow from the aerobic digester and backwash flow from the tertiary filters are directed to the onsite pumping station, which pumps flow to the head of the plant, upstream

Draft

of the plant headworks. Flow from the pumping station is measured with a magnetic flow meter. Over the review period, measured flow from the onsite pumping station represented approximately 12% of the final effluent measured flow. On an average monthly basis, there was a positive linear correlation between the measured final effluent flow and the measured flow from the onsite pumping station. As such, plant recycle flow is expected to increase as raw wastewater flows increase.

Flow from the onsite septage receiving tank is also metered. Plant operators have indicated there are some drains and rain water which are directed to the onsite septage receiving tank. Over the review period, the plant has received an average of approximately 11 m^3/d of flow from the septage receiving tank. However, due to the contributions from the connected drains, this value overestimates the actual volume of septage received at the Grand Valley WPCP.

Plant operators also indicated that issues were experienced with solenoids associated with wash water for the screening and grit removal system sticking in the open position, resulting in potable water flowing directly into the liquid stream. This flow is not measured directly, however it contributes to the measured effluent flow from the WPCP. The impact of these valves on total effluent wastewater flow is expected to be negligible.

2.2 Analysis of Inflow / Infiltration in the Collection System

The Town has recently conducted an investigation of I/I in the collection system (RJ Burnside, 2009). The investigation found significant volumes of I/I in the Grand Valley collection system. The investigation identified structural deficiencies at several manholes, but observed that the overall structural integrity of the collection system was not a significant factor contributing to I/I. Instead, it identified that significant I/I flows are generated on private property, specifically from the direct connection of footings to the sanitary collection system. Historically, the implementation of I/I reduction strategies on private property is difficult. The Town and R.J. Burnside have indicated they are currently pursuing provincial funding assistance to conduct an I/I reduction program.

Overall, I/I in the Grand Valley collection system impacts the magnitude of peak flows to the Emma St. SPS, and flow to the Grand Valley WPCP. It is important to note that several treatment processes at the Grand Valley WPCP are dependent on the maximum day and peak raw wastewater flows. As such, I/I may directly impact the available treatment capacity at the Grand Valley WPCP. Implementation of an I/I reduction strategy may reduce the intensity of peak flows to the Grand Valley WPCP in the future.

2.3 Plant Influent Raw Wastewater Quality during the Review Period (2012 - 2014)

Over the review period, grab samples of the raw wastewater stream were collected monthly. Samples were collected immediately upstream of the influent screens, and are representative of the plant influent raw wastewater flow. It includes contributions from the collection system raw wastewater, septage, tertiary filter backwash, and digester supernatant.

Table 2.3 presents a summary of the plant influent raw wastewater quality over the review period (2012 - 2014).

Generally, the combined influent was found to be of low strength with respect to biological oxygen demand (BOD_5), total suspended solids (TSS), and TP, and of low to medium strength with respect to total Kjeldahl nitrogen (TKN).

As discussed, only grab samples of the combined influent stream were collected during the review period (2012 - 2014). These samples are a representation of influent quality at the

Draft

moment they are collected, but may not be an accurate representation of the average influent quality over 24 hours. Therefore, the results presented in Table 2.3 may not accurately represent average combined influent quality.

Table 2.3 Plant Influent Raw Wastewater Characteristics

Parameter	Units	Plant Influent Raw Wastewater (1)	Typical Wastewater Strength (2)
BOD_5	(mg/L)	105	110 *(Low)* 190 *(Med)* 350 *(High)*
TSS	(mg/L)	133	120 *(Low)* 210 *(Med)* 400 *(High)*
TKN	(mg/L)	33.4	20 *(Low)* 40 *(Med)* 70 *(High)*
TP	(mg/L)	3.45	4 *(Low)* 7 *(Med)* 12 *(High)*

Notes:
BOD - Biochemical Oxygen Demand
TSS - Total Suspended Solids
TKN - Total Kjeldahl Nitrogen
TP - Total Phosphorus
1. Includes filter backwash and digester supernatant recycle streams.
2. Metcalf and Eddy (2003).

2.4 Liquid Train Influent Loadings during the Review Period

Using results presented in Table 2.3 and the estimated average day plant flow over the review period, Table 2.4 presents a summary of the average day liquid train loading and per capita loading from data collected during the review period. This assumes a current service population of approximately 1,752.

Table 2.4 Summary of Plant and Per Capita Loading over the Review Period (2012 - 2014)

Parameter	Average Daily Load (kg/d) (1)	Historic Per Capita Load (g/cap/d)	Typical Per Capita Load (g/cap/d)
BOD_5	88.2	50.4	75 (2)
TSS	112	64.0	90 (2)
TKN	28.2	16.1	13.3 (3)
TP	2.91	1.66	2.1 (3)

Notes
1. Includes loading from recycle streams (digester supernatant and tertiary filter backwash), and from septage.
2. As per Design Guidelines for Sewage Works (MOE, 2008).
3. As per Metcalf and Eddy, 2010.

From the table above, the estimated per capita loading during the review period was below typical per capita loading rates for BOD_5, TSS, and TP. However, the estimated per capita TKN loading rate was greater than typical.

Draft

3. DEVELOPMENT OF DESIGN BASIS

The following subsections outline the future design basis in terms of raw wastewater flows and loadings for the Grand Valley WPCP. This design basis will be used to evaluate the capacity of the Grand Valley WPCP from both a hydraulic and biological treatment perspective in subsequent phases of this study.

For the purposes of this evaluation, flows and loads were developed for three future scenarios as follows:

- Scenario I: Full completion of planned residential developments;
- Scenario II: A 15% increase above the current CofA rated average day flow (ADF) (1,430 m^3/d); and,
- Scenario III: A 25% increase above the current CofA rated ADF (1,555 m^3/d).

3.1 Raw Wastewater Flows from the Collection System

3.1.1 Design Average Day Flow

Population projections for the Town were based on a recently completed review of future planned residential developments for the Town (R.J. Burnside, 2015). Specifically, future planned developments consist of:

- 321 housing units constructed as part of three residential developments (Mayberry Phase 1 and 2, and Hollenbeck); and,
- The 'Moco Allocation', consisting of 7 residential units and 15.3 hectares of developable land.

A summary of these planned residential developments is presented in Table 3.1.

Table 3.1 Summary of Serviced New Developments

Development	Population
Moco Allocation (1)	192
Mayberry Phase 1	190
Mayberry Phase 2	507
Hollenbeck	278
Total Additional Equivalent Service Population from New Developments	1,167

Notes:
1. Equivalent service population, consisting of serviced residential lots, and developed land.

New services corresponding to an equivalent population of 1,167 persons have been allocated by the Township, for a total equivalent service population of approximately 2,919.

Draft

Projected future wastewater flows from planned developments (Scenario I) were based on a design dry weather per capita flow of 372 L/cap/d, and an average I/I allowance of 82 L/cap/d. Both values are based on a review of 2012 - 2014 plant operating records. The overall design per capita wastewater flow for future development (454 L/cap/d) is identical to the 3-year average observed at the plant.

From Table 3.1, the estimated new equivalent service population associated with completion of all planned developments is 1,167 and is projected to contribute approximately 529 m^3/d on average to the plant. The existing average day flow is 746 m^3/d. Therefore, the overall projected average day flow is 1,276 m^3/d, which is only 32 m^3/d more than the CofA rated average day flow for the Grand Valley WPCP of 1,244 m^3/d.

Table 3.2 presents a summary of the ADF design basis for each presented scenario. For Scenario II and Scenario III, growth service populations were estimated from the increase in ADF and the design per capita flow.

Table 3.2 Design Per Capita Flows, Populations, and ADFs

Source	Per Capita Flow (L/cap·d)(1)		Design Serviced Population			Design ADF (m^3/d) (2)
	Existing	New Growth	Existing	New Growth	Total	
Scenario I				1,167	2,919	1,276
Scenario II	454	454	1,752	1,508	3,260	1,430
Scenario III				1,784	3,536	1,555

Notes:
1. Inclusive of I/I flow allowance.
2. Raw wastewater from the collection system.

Draft

3.1.2 Design Maximum Day Flow

The design MDF is based on the historic base MDF for the existing service area, plus a MDF allowance for future residential development.

To calculate the MDF allowance for new growth, a MDF peaking factor for the new growth flows was determined. This was done by applying the historic dry weather flow (DWF) factor to the non-I/I portion of the per capita flow rate, and applying a typical per capita generation rate of 227 L/cap/d for I/I flows (MOE, 2008).

A dry weather flow analysis was completed to determine the historic DWF factor. The analysis of DWF was conducted based on flow data from 2012 to 2014 and meteorological data from Environment Canada. Days were considered to be "dry" when no precipitation occurred for that day and three days prior between the months of May and October, inclusive. Based on the flow analysis, the historic DWF peaking factor for the existing service area was 2.1. In addition, the existing per capita DWF for the residential service area was estimated to be 372 L/cap/d, based on a service population of 1,752, and the existing I/I flow was estimated to be 82 m^3/d. Details of existing flows are presented in Table 2.1.

By applying the historic DWF peaking factor of 2.1 to the dry weather flow portion of the per capita flow, and the I/I flow peak factor to the I/I portion of the per capita flow, the overall MDF peaking factor for new growth was determined to be 2.2.

To determine the conceptual level design MDF for each phase, the new growth MDF factors were applied to the increase in average day design flows for each phase, and these growth MDF values were added to the existing base MDF. The conceptual level design MDF values for each phase are presented in Table 3.3.

Table 3.3 Design Maximum Day Flows

Parameter	Scenario I	Scenario II	Scenario III
Design ADF			
Existing	746 m^3/d	746 m^3/d	746 m^3/d
Growth	529 m^3/d	684 m^3/d	809 m^3/d
Overall (1)	**1,276 m^3/d**	**1,430 m^3/d**	**1,555 m^3/d**
MDF Factor			
Existing	6.3	6.3	6.3
Growth	2.2	2.2	2.2
Overall (1)	**4.6**	**4.3**	**4.1**
Design MDF			
Existing	4,671 m^3/d	4,671 m^3/d	4,671 m^3/d
Growth	1,157 m^3/d	1,494 m^3/d	1,768 m^3/d
Overall (1)	**5,828 m^3/d**	**6,165 m^3/d**	**6,439 m^3/d**

Notes:
1. Projected maximum day raw wastewater flow from the collection system.

Therefore, the conceptual level design MDF flows are 5,828 m^3/d, 6,165 m^3/d, and 6,439 m^3/d for Scenario I, Scenario II, and Scenario III, respectively.

3.1.3 Design Peak Flows

As previously noted, peak flow data indicate that peak flow of raw wastewater from the collection system via the Emma St. SPS has approached 6,048 m^3/d. This peak flow was observed during a peak flow event in April 2014, resulting from both a large snow melt and precipitation event.

Future peak instantaneous flow (PIF) values were calculated based on the PIF observed over the review period, plus a peak flow allowance for new growth. To calculate the PIF allowance for new growth, a PIF peaking factor for the new growth flows was determined for each design scenario. This was done by applying the Harmon peaking factor to the non-I/I portion of the per capita flow value, and applying a typical per capita peak I/I flow rate of 227 L/cap/d (MOE, 2008). The Harmon peaking factor was calculated for each phase based on the overall design equivalent populations of 2,919 for Scenario I; 3,260 for Scenario II; and 3,536 for Scenario III. Accordingly, the Harmon peaking factors for Scenarios I, II, and III were determined to be 3.5, 3.4, and 3.4, respectively.

By applying the appropriate Harmon peaking factor to the dry weather flow portion of the per capita flow, and the I/I flow peak factor to the I/I portion of the per capita flow, the overall PIF peaking factor for new growth was determined to be 3.3 for all three scenarios.

Draft

To determine the conceptual level design PIF for each scenario, the new growth PIF peaking factors were applied to the increase in average day design flows for each phase, and these growth PIF values were added to the existing base PIF. For the purposes of this conceptual level design basis, the PIF factor for new growth was applied to the growth flows. The conceptual level design PIF values for each phase are presented in Table 3.4.

Table 3.4 Design Peak Instantaneous Flows

Parameter	Scenario I	Scenario II	Scenario III
Design ADF			
Existing	746 m^3/d	746 m^3/d	746 m^3/d
Growth	529 m^3/d	684 m^3/d	809 m^3/d
Overall	**1,276 m^3/d**	**1,430 m^3/d**	**1,555 m^3/d**
PIF Factor			
Existing	10.2	10.2	10.2
Growth	3.3	3.3	3.3
Overall	**6.1**	**5.8**	**5.6**
Design PIF			
Existing	6,048 m^3/d	6,048 m^3/d	6,048 m^3/d
Growth	1,763 m^3/d	2,255 m^3/d	2,647 m^3/d
Overall	**7,811 m^3/d**	**8,303 m^3/d**	**8,695 m^3/d**

The conceptual level design PIF values are 7,811 m^3/d for Scenario I; 8,303 m^3/d for Scenario II; and 8,695 m^3/d for Scenario III.

The following important observations can be made based on results in Table 3.4:

- The overall design PIF factor for all scenarios is in excess of a typical peak factor given the equivalent service population of the Grand Valley WPCP. This is primarily a result of the large peak instantaneous flow observed in April 2014. Excessive peaking factors suggest the collection system may be susceptible to high extraneous flows during wet weather events; and,
- The projected PIF for all scenarios is in excess of the CofA rated Emma St. SPS capacity (7,680 m^3/d). This analysis suggests the Emma St. SPS may require upgrades at future flows provided that existing peak flows are not abated by any I/I reduction strategies. An extensive review of the Emma St. SPS capacity was not conducted as part of this review.

Draft

3.2 Raw Wastewater Loads

For purposes of developing loading projections, typical per capita loading rates were assumed for BOD_5, TSS, and TP. This is a conservative approach that accounts for the uncertainty of future development, and the uncertainty in grab sample data collected during the review period. Future per capita TKN loadings were assumed to be identical to per capita loadings observed during the review period (2012 - 2014).

Estimations of maximum month loading factors were established from plant records of effluent flows and influent concentrations. Data from April 2014 was found to be outlying due to high observed flows, and was excluded from analysis. Maximum month factors were estimated to be 1.9, 1.9, 1.9, and 2.2 for BOD_5, TSS, TKN and TP, respectively. Typical maximum month loading factors are much less than those observed at the Grand Valley

WPCP, and range from 1.4 to 1.6. As previously discussed, raw influent quality data over the review period (2012 - 2014) represents results from a single grab sample, collected on a monthly basis. This sampling technique may result in increased variability in results. The discrepancy between typical maximum month loading factors and those observed at the Grand Valley WPCP may be in part related to the type and frequency of raw influent sample collection. In order to develop a conservative design basis, maximum month factors developed from plant data were used.

Base raw wastewater loading included contributions from the following sources:

- Raw wastewater from the collection system;
- Recycle flow from the onsite pumping station; and,
- Septage.

Wastewater from all three sources are combined at the plant headworks, upstream of the grab sample location. As such, it is assumed that raw wastewater quality collected over the review period is a representation of all three streams and, therefore, base wastewater loadings include contributions from all three sources.

Septage receiving facilities at the Grand Valley WPCP were designed to treat an average day septage flow of 3.6 m^3/d. Plant operators have indicated that the septage receiving tank also receives drain water and some rain water from the plant. As such, accurate records of septage flow over the review period (2012 - 2014) are not available. Currently, the plant is operating at approximately 60% of its CofA rated ADF capacity of 1,244 m^3/d. For purposes of loading projections, it is assumed the plant also receives 60% of its designed septage capacity (i.e. approximately 2.2 m^3/d), and will receive the full design volume of septage when raw wastewater flows from the collection system reach the full projected capacity. Septage quality was assumed from typical values reported in literature (US EPA, 1984/1994).

Table 3.5 presents the projected future average day loadings to the Grand Valley WPCP.

Draft

Table 3.5 Design Average Raw Wastewater Loadings

Parameter	Base Raw Wastewater Loading	Loading Due to Growth [1,2,3]	Total Design Average Loading	Average Design Concentration
Scenario I				
BOD_5	88.2 kg/d	97.6 kg/d	186 kg/d	146 mg/L
TSS	112 kg/d	127 kg/d	239 kg/d	187 mg/L
TKN	28.2 kg/d	19.8 kg/d	47.9 kg/d	37.6 mg/L
TP	2.91 kg/d	2.81 kg/d	5.72 kg/d	4.48 mg/L
Scenario II				
BOD_5	88.2 kg/d	123 kg/d	211 kg/d	148 mg/L
TSS	112 kg/d	157 kg/d	269 kg/d	188 mg/L
TKN	28.2 kg/d	25.3 kg/d	53.4 kg/d	37.4 mg/L
TP	2.91 kg/d	3.53 kg/d	6.43 kg/d	4.50 mg/L
Scenario III				
BOD_5	88.2 kg/d	144 kg/d	232 kg/d	149 mg/L
TSS	112 kg/d	182 kg/d	294 kg/d	189 mg/L
TKN	28.2 kg/d	29.7 kg/d	57.9 kg/d	37.2 mg/L
TP	2.91 kg/d	4.11 kg/d	7.01 kg/d	4.51 mg/L

Notes:
1. Based on an assumed per capita loading of 75 g/cap/d for BOD5, 90 g/cap/d for TSS, 15.86 g/cap/d for TKN, and 2.1 g/cap/d for TP.
2. Based on an assumed population growth of 1,167 for Scenario 1, 1,515 for Scenario 2, and 1,793 for Scenario 3.
3. Assumed approximate 1.4 m^3/d increase in septage flows. Assumed septage quality (7,000 mg/L BOD_5, 15,000 mg/L TSS, 700 mg/L TKN, and 250 mg/L TP) as reported in literature (EPA 1984/1994)

Draft

The maximum monthly loadings were based on the maximum month loading peak factors observed over the review period for each parameter. The peak factors were 1.9 for BOD_5, 1.9 for TSS, 1.9 for TKN, and 2.2 for TP. Table 3.6 presents the design maximum monthly loadings to the Grand Valley WPCP.

Table 3.6 Design Maximum Month Raw Wastewater Loadings

Parameter	Average Design Wastewater Loading	Maximum Month Loading Peak Factor	Design Maximum Month Loading
Scenario I			
BOD_5	186 kg/d	1.9	353 kg/d
TSS	239 kg/d	1.9	453 kg/d
TKN	47.9 kg/d	1.9	91.1 kg/d
TP	5.72 kg/d	2.2	12.6 kg/d
Scenario II			
BOD_5	211 kg/d	1.9	402 kg/d
TSS	269 kg/d	1.9	512 kg/d
TKN	53.4 kg/d	1.9	101 kg/d
TP	6.43 kg/d	2.2	14.2 kg/d
Scenario III			
BOD_5	232 kg/d	1.9	441 kg/d
TSS	294 kg/d	1.9	559 kg/d
TKN	57.9 kg/d	1.9	110 kg/d
TP	7.01 kg/d	2.2	15.4 kg/d

Draft

4. SUMMARY OF PLANT FLOW AND LOAD PROJECTIONS

Table 4.1 contains a summary of the projected plant design basis flows and loads to the Grand Valley WPCP for all three scenarios. Projections of future plant loads were made using typical per capita loading rates, or based on the estimated historical per capita loading rate, whichever resulted in the more conservative estimate of future loads. Plant data collected from 2012 to 2014 was used as part of this review.

Table 4.1 Summary of Design Basis

Parameter	Scenario I	Scenario II	Scenario III
Population	2,919	3,260	3,536
ADF	1,276 m^3/d	1,430 m^3/d	1,555 m^3/d
MDF	5,828 m^3/d	6,165 m^3/d	6,439 m^3/d
MDF Factor	4.6	4.3	4.1
PIF	7,811 m^3/d	8,303 m^3/d	8,695 m^3/d
PIF Factor	6.1	5.8	5.6
BOD_5			
Average Loading	186 kg/d	211 kg/d	232 kg/d
Maximum Month Loading	353 kg/d	402 kg/d	441 kg/d
Average Concentration	146 mg/L	148 mg/L	149 mg/L
TSS			
Average Loading	239 kg/d	269 kg/d	294 kg/d
Maximum Month Loading	453 kg/d	512 kg/d	559 kg/d
Average Concentration	187 mg/L	188 mg/L	189 mg/L
TKN			
Average Loading	47.9 kg/d	53.4 kg/d	57.9 kg/d
Maximum Month Loading	91.1 kg/d	101 kg/d	110 kg/d
Average Concentration	37.6 mg/L	37.4 mg/L	37.2 mg/L
TP			
Average Loading	5.72 kg/d	6.43 kg/d	7.01 kg/d
Maximum Month Loading	12.6 kg/d	14.2 kg/d	15.4 kg/d
Average Concentration	4.48 mg/L	4.50 mg/L	4.51 mg/L

Draft

5. REFERENCES

Ministry of the Environment. Design Guidelines for Sewage Works. 2008.

Metcalf & Eddy. Wastewater Engineering: Treatment and Resource Recovery. Fifth Edition. Toronto. 2014.

R.J. Burnside & Associates Limited. Capacity at the Wastewater Plant. May 2015.

R.J. Burnside & Associates Limited. Township of East Luther Grand Valley - Grand Valley Wastewater Treatment Plant Operations Manual. July 2015.

R.J. Burnside & Associates Limited. Township of East Luther Grand Valley - ELGV Wastewater Treatment Plant Design Brief. October 2008.

R.J. Burnside & Associates Limited. East Luther Grand Valley - I and I Study Report Township of East Luther Grand Valley. July 2009.

US EPA. Handbook: Septage Treatment and Disposal. EPA-625/6-84-009. 1984.

US EPA. Guide to Septage Treatment and Disposal. EPA-625/R-94-002. 1994.

Draft

APPENDIX A
SCREENSHOTS OF EMMA ST. SPS MEASURED FLOW

Draft

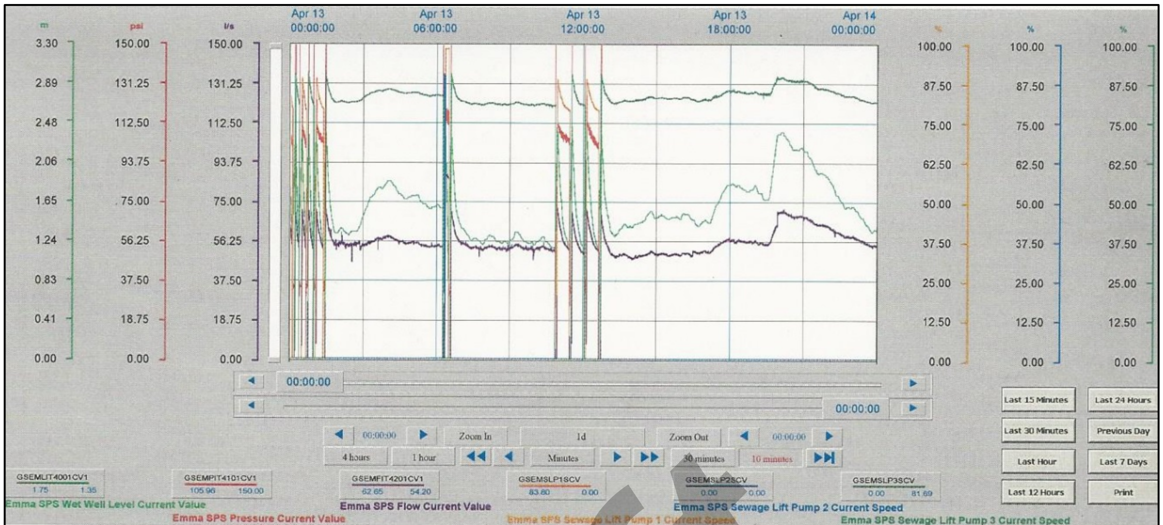


Figure A.1 Emma St. SPS Measured Flows - April 13, 2014

Draft

Figure A.2 Emma St. SPS Measured Flows - April 14, 2014

Draft

APPENDIX B

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY UPDATED DESIGN BASIS

XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

XCG File No.: 3-252-57-01
September 6, 2016

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY
UPDATED DESIGN BASIS

Draft

Prepared for:

TOWN OF GRAND VALLEY
5 Main Street, North
Grand Valley, Ontario
L9W 5S6

Attention: Jane Wilson

Prepared by:

XCG CONSULTING LIMITED
Suite 300, 2620 Bristol Circle
Oakville, Ontario
L6H 6Z7

TABLE OF CONTENTS

TABLES

FIGURE

APPENDICES

Draft

1. INTRODUCTION

1.1 Background

The Grand Valley WPCP provides treatment for wastewater generated in the community of Grand Valley within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of Environment and Climate Change (MOECC) Certificate of Approval (C of A) No. 9706-7KWQ57, issued on February 2, 2009. The quality and quantity of effluent currently discharged by the existing Water Pollution Control Plant (WPCP) is regulated by the C of A. The Grand Valley WPCP has a rated average capacity of 1,244 m^3/d.

The Town has initiated an investigation to analyze the potential to re-rate the existing Grand Valley WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. The Town has retained XCG Consulting Limited (XCG) to undertake a capacity assessment of the Grand Valley WPCP to evaluate the potential to re-rate the plant. As part of this assessment, XCG recently completed a review of plant raw wastewater flows and loads, and developed a design basis for future raw wastewater flows and loads (XCG, 2015). This review was completed using historic plant operating data, collected between 2012 and 2014. The purpose of this document is to update the design basis using additional raw wastewater flow and load information collected at the plant between January 2015 and May 2016.

1.2 Objectives

The specific objectives of this technical memorandum are to:

- Conduct an updated review of plant raw wastewater flows and loads; and
- Develop an updated design basis for future raw wastewater flows and loads.

1.3 Data Sources

The following data sources were used in part to develop projections of plant flows and loads:

- Plant flow and quality information (2012 - May 2016);
- Memorandum completed by R.J. Burnside regarding the existing and future service populations of the Grand Valley WPCP (May, 2015);
- East Luther Grand Valley (ELGV) Wastewater Treatment Plant Design Brief (2008);
- ELGV Inflow and Infiltration (I/I) Study Report (July, 2009);
- Grand Valley Wastewater Treatment Plant Operations Manual (July, 2015); and,
- Grand Valley WPCP facility tour (September, 2015).

Draft

2. REVIEW OF RAW WASTEWATER FLOW AND QUALITY

Raw sewage flows from the collection system are conveyed to the Grand Valley WPCP from the Emma St. sewage pumping station (SPS) via a forcemain. Complete details of equipment and operation of the Emma St. SPS are given in the design basis developed earlier in this study (XCG, 2015) that used historic operating data over the period 2012 to 2014 (a copy of this document is provided in Appendix A).

2.1 Review of Raw Wastewater Flow over the Review Period (2012 - May 2016)

As of 2015, the Grand Valley WPCP serves a residential population of approximately 1,807 persons. Influent flow to the Grand Valley WPCP liquid treatment train is comprised of:

- Raw wastewater from the Grand Valley sanitary collection system, pumped to the plant via the Emma St. SPS;
- Septage flow from the onsite septage receiving station; and,
- Plant recycle flows (i.e. digester supernatant and filter backwash flow), pumped to the head of the plant from the onsite pumping station.

Flow from each source above is metered separately. In addition, effluent flow from the plant is measured using a V-notch weir. Although the recycle flows are metered and impact flows through the liquid treatment train, they do not contribute to the recorded plant influent and effluent flows since they simply recirculate internally within the process. A summary of the recorded plant influent (Emma St. SPS + septage) and recorded effluent flow (effluent V-notch weir) to the Grand Valley WPCP is shown as Table 2.1. For reference, the ADF as given in the annual report has also been included. The following points must be considered for purposes of flow analysis:

- Raw influent flow to the Grand Valley WPCP was calculated as the sum of flow from the Emma St. SPS and the onsite septage receiving station.
- Plant operators reported that the accumulation of grit in the magnetic flow meter at the Emma St. SPS led to false high measurements from 2012 - 2014. Beginning in July 2014, operators began regular flushing to prevent grit accumulation at the Emma St. SPS.
- In 2015, plant operators noted that malfunctioning solenoid valves at the plant headworks resulted in a larger volume of potable flushing water being added to the WPCP downstream of the influent flow measurement devices. Although this flushing water did not impact reported influent flow, it contributed to the final effluent flow readings, artificially increasing them. Unfortunately, potable water use at the WPCP is not metered, so it is not possible to estimate the volume of flushing water added to the process. The malfunctioning solenoid valves were replaced in early January 2016, and therefore this excess source of potable water would not have impacted effluent flows from February 2016 on.
- The final effluent V-notch weir was recalibrated in January 2016, approximately two weeks after the solenoid valves were replaced. As such, there is insufficient

Draft

data available to quantify the impact of replacing the malfunctioning solenoid valves on effluent flow measurements.

- The effluent flow meter calibration record indicates the 'zero' reading was adjusted during the calibration process in January 2016. Records do not detail the magnitude of the adjustment. Plant operators have indicated that the effluent meter was calibrated using influent flow data. Overall, details of calibration process and its impact on measured effluent flow from the Grand Valley WPCP are not clear from the available information and should be further investigated. The Town should also consider performing an additional assessment and calibration of the effluent flow meter, as required, to ensure the accuracy of the recorded final effluent flow.
- At the time of this analysis, 2016 flow data was only available for the months of January to May. To project annual 2016 ADF values, historic operating data were used to develop a ratio of (average January to May flow):(annual ADF). This method was used to account for the typically high flows experienced during the spring freshet. 2016 flows shown in Table 2.1 represent the projected 2016 annual ADF values.

Table 2.1 Summary of Plant Influent and Effluent Flow (2012 - 2016)

Average Day Flow	2012	2013	2014	2015	Projected 2016 (1)	Overall
Service Population	1,494	1,683	1,752	1,807	1,807 (2)	-
Raw Influent Flow (m^3/d) (3)	- (4)	- (4)	- (4)	471	675	573
Plant Effluent Flow (m^3/d)	643	821	776	710	719	734
OCWA Reported ADF (m^3/d) (5)	718	815	772	473	-	

Notes:
1. Flows measured from January to May 2016. Average daily influent flow (777 m^3/d) and effluent flow (828 m^3/d) have been adjusted here to account for the spring freshet.
2. Population data not available. Assumed equal to the 2015 service population.
3. Includes flows from the Emma St. SPS and the onsite septage receiving station.
4. Measured flow not available as a result of grit accumulation at the Emma St. SPS magmeter.
5. As reported in the Grand Valley WPCP Annual Report.

In 2016, raw influent and final effluent flow measurements from January to May were within 10%, indicating good agreement between the flow meters. The adjusted 2016 ADF as measured by either the influent or effluent flow meters is consistent with flows reported from 2012 to 2014.

Based on the available information, raw influent flow measured in 2015 (471 m^3/d) is not consistent with the range of effluent flows measured from 2012 - 2014 (643 m^3/d to 821 m^3/d) or ADF values reported in the Annual Reports over the same period (718 m^3/d to 815 m^3/d). Further, the 2015 raw influent flow also appears to be inconsistent with projected 2016 influent and effluent measurements at the Grand Valley WPCP (675 m^3/d and 719 m^3/d, respectively). Therefore, the accuracy of the 2015 raw influent data cannot be confirmed and, as such, these flows were not used as part of this design basis update.

Draft

As previously noted, measured final effluent flow in 2015 was impacted by malfunctioning solenoid valves in the headworks. However, the increase in final effluent flow resulting from the solenoid valves cannot be determined using the available information. Further, a dry weather flow analysis conducted using the 2015 final effluent data was found to be inconsistent with historical dry weather flows observed from 2012 to 2014. Therefore, the accuracy of the 2015 final effluent data could also not be confirmed and the data set was similarly excluded from the design basis update.

Table 2.2 presents a summary of the estimated final effluent flow and per capita flows to the Grand Valley WPCP. For comparison, projected flows from 2016 are included in the table. However, since the 2016 data set is not complete (i.e. only flows to May have been considered), it has not been used to develop flow projections. As previously noted, 2015 flows have also been excluded since their accuracy cannot be confirmed.

The table includes an estimate of dry weather plant flow and per capita flows, and quantification of the historical I/I observed at the plant for the period 2012 to 2014. Meteorological data was obtained from the Environment Canada station at Fergus, Ontario. Days were considered dry when no precipitation occurred for that day and three days prior. Only data from May to October was used for dry weather flow analysis. Since a complete data set is not available, dry weather flow analysis was not conducted on 2016 data.

Table 2.2 Summary of Treated Flow over the Review Period

	Units	2012	2013	2014	Projected 2016	Overall (3)
Estimated Service Population	Persons	1,494	1,683	1,752	1,807 (2)	-
Average Daily Flow (1)	m^3/d	643	821	776	719	746
Per Capita Flow	L/cap/d	430	488	443	398	454
Estimated Per Capita Dry Weather Flow	L/cap/d	371	391	354	-	372
Estimated Per Capita I/I	L/cap/d	59	97	89	-	82

Notes:
1. Based on flow measurements taken at the effluent flow meter over the review period.
2. Assumed population is unchanged from 2015.
3. Overall flows consider data collected from 2012 - 2014 only.

Results presented in Table 2.2 are unchanged from the design basis developed earlier in this study. The overall average per capita flow to the Grand Valley WPCP over the review period was 454 L/cap/d, inclusive of I/I. The estimated dry weather per capita flow (372 L/cap/d) is consistent with the typical range of per capita flows of 225 to 450 L/cap/d, exclusive of extraneous flows (MOE, 2008). The calculated per capita I/I was 82 L/cap/d, which is slightly less than the typical design I/I flow of 90 L/cap/d (MOE, 1985).

Draft

Summary of Maximum Day Flows during the Review Period (2012 - May 2016)

Table 2.3 provides an updated summary of the maximum day flows observed over the review period as measured at the final effluent flow meter. In 2014, the maximum day flow event required use of the storm tank. As such, the maximum day flow was estimated from influent flow as measured by the magmeter at the Emma St. SPS. Additional details are given in Appendix A.

For comparison, data collected between January and May 2016 has also been included in the table. However, as previously noted, the accuracy of flow data from 2015 cannot be confirmed. As such, 2015 flow information has been excluded from this review.

Table 2.3 Summary Maximum Day Flows over the Review Period (2012 - May 2016)

	Units	2012	2013	2014	2016	Overall
ADF	m^3/d	643	821	776	719 (2)	734
MDF	m^3/d	2,601	2,254	4,671 (1)	2,370 (3)	4,671 (1)
MDF Factor	-	4.0	2.8	6.0	3.3	6.3

Notes:
Unless otherwise indicated, flows are based on flow measurements taken at the effluent flow meter over the review period (2012 - May 2016)
1. Based on Emma St. SPS flow measurements on April 13, 2014.
2. Projected 2016 ADF.
3. Maximum day flow recorded over the period January to May 2016.

Summary of Peak Flows during the Review Period (2012 - May 2016)

Peak flows were estimated from flow records at the Emma St. SPS. Additional details of the flow analysis are included in the design basis developed earlier in this study (XCG, 2015) and the analysis remains unchanged for this updated design basis. The peak flow from the collection system was estimated to be approximately 70 L/s (6,048 m^3/d).

Evaluation of Plant Recycle and Septage Flows over the Review Period (2012 - May 2016)

Decant flow from the aerobic digester and backwash flow from the tertiary filters are directed to the onsite pumping station, which pumps flow to the head of the plant, upstream of the plant headworks. Flow from the pumping station is measured with a magnetic flow meter. Over the review period, measured flow from the onsite pumping station represented approximately 11% of the final effluent measured flow. On an average monthly basis, there was a positive linear correlation between the measured final effluent flow and the measured flow from the onsite pumping station. As such, plant recycle flow is expected to increase as raw wastewater flows increase. As noted above, plant recycle flows impact flows to the liquid treatment train, but do not impact raw influent or final effluent flows.

Flow from the onsite septage receiving tank is also metered. From 2012 to 2014, plant operators indicated the annual average volume of septage received and treated at the Grand Valley WPCP was 75 m^3, or an equivalent daily flow of approximately 0.2 m^3/d. However, from 2012 to 2014, the plant received an average of approximately 11 m^3/d of flow from the septage receiving tank, significantly greater than the

Draft

estimated equivalent daily septage flow. Exact reason for the discrepancy is not known, but plant operators have indicated there are some drains and rain water which are directed to the onsite septage receiving tank. The design average day septage treatment capacity is 3.6 m^3/d (R.J.Burnside, 2015).

2.2 Plant Influent Raw Wastewater Quality during the Review Period (2012 - May 2016)

Over the review period, grab samples of the raw wastewater stream were collected monthly. Samples were collected immediately upstream of the influent screens, and are representative of the plant influent raw wastewater flow. It includes contributions from the collection system raw wastewater, septage, tertiary filter backwash, and digester supernatant.

Table 2.4 presents a summary of the plant influent raw wastewater quality over the review period (2012 - May 2016). For purposes of comparison, plant influent quality as reported in the previously developed design basis (XCG, 2015) is also reported in the table.

Generally, the combined influent was found to be of low strength with respect to biological oxygen demand (BOD_5), total suspended solids (TSS), and TP, and of low to medium strength with respect to total Kjeldahl nitrogen (TKN). Inclusion of additional historical data had little impact on the average quality of the influent stream.

It is important to note that only one grab sample per month of the combined influent stream was collected during the review period (2012 - May 2016). These samples are a representation of influent quality at the moment they are collected, but may not be an accurate representation of the average influent quality over 24 hours. Therefore, the results presented in Table 2.4 may not accurately represent average combined influent quality.

Draft

Table 2.4 Plant Influent Raw Wastewater Characteristics

Parameter	Units	Plant Influent Raw Wastewater (1)				Typical Wastewater Strength (2)
		(2012 - 2014)	(2015)	(January - May, 2016)	Overall (2012 - May 2016)	
BOD_5	(mg/L)	105	134	99	111	110 *(Low)* 190 *(Med)* 350 *(High)*
TSS	(mg/L)	133	147	90	134	120 *(Low)* 210 *(Med)* 400 *(High)*
TKN	(mg/L)	33.4	38.7	31.2	34.4	20 *(Low)* 40 *(Med)* 70 *(High)*
TP	(mg/L)	3.45	4.02	3.02	3.54	4 *(Low)* 7 *(Med)* 12 *(High)*

Notes:
BOD - Biochemical Oxygen Demand
TSS - Total Suspended Solids
TKN - Total Kjeldahl Nitrogen
TP - Total Phosphorus
1. Includes filter backwash and digester supernatant recycle streams.
2. Metcalf and Eddy (2003).

Draft

Results presented in Table 2.4 indicate that raw wastewater in 2015 was slightly stronger than the 2012 - 2014 average raw wastewater strength. Conversely, raw wastewater samples collected from January to May 2016 were slightly weaker than the 2012 - 2014 average.

Due to the sampling method, there is significant variability expected in the quality results which impact the average concentration observed in a given year. For example, Figure 2.1 plots the measured BOD_5 concentration in the raw influent stream from 2012 - May 2016. Results show that, in 2015, the measured BOD_5 concentration was significantly greater than other measurements in the months of February, November, and December. However, over all other months, the BOD_5 concentration was comparable to other historical measurements. This figure is representative of other influent parameters (i.e. TSS, TKN, and TP). As such, there is no apparent trend in the raw influent concentrations, and data collected between January 2015 and May 2016 agrees with previous characterization of raw influent flow using data collected between 2012 and 2014.

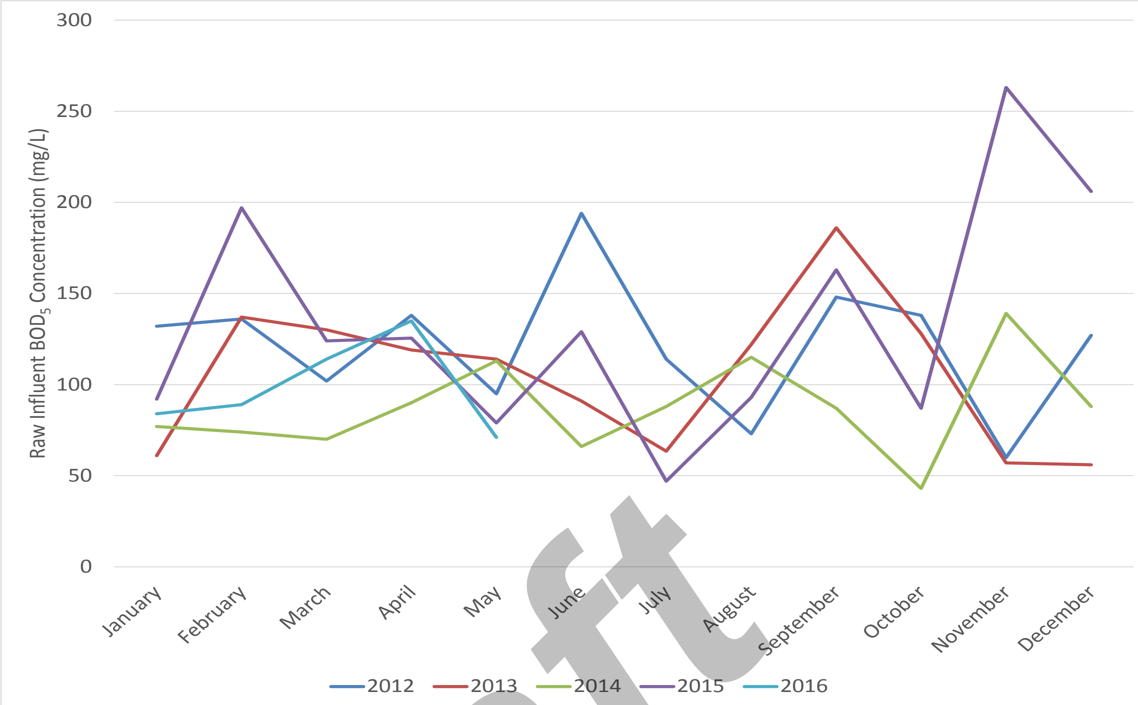


Figure 2.1 Raw Influent BOD_5 Concentrations (2012 - May 2016)

2.3 Liquid Train Influent Loadings during the Review Period

As previously presented, the accuracy of 2015 influent and effluent flows cannot be confirmed and have been excluded from consideration as part of this review. Further, raw wastewater quality information collected in 2015 and 2016 is consistent with previous data collected between 2012 and 2014.

As such, the estimated plant and per capita loading considers data collected from 2012 to 2014, and therefore is identical to the design basis which was previously developed. This information is reproduced in Table 2.5.

Table 2.5 Summary of Plant and Per Capita Loading over the Review Period (2012 - 2014)

Parameter	Average Daily Load (kg/d) (1)	Historic Per Capita Load (g/cap/d)	Typical Per Capita Load (g/cap/d)
BOD_5	88.2	50.4	75 (2)
TSS	112	64.0	90 (2)
TKN	28.2	16.1	13.3 (3)
TP	2.91	1.66	2.1 (3)

Notes
3. Includes loading from recycle streams (digester supernatant and tertiary filter backwash), and from septage.
4. As per Design Guidelines for Sewage Works (MOE, 2008).
5. As per Metcalf and Eddy, 2010.

From the table above, the calculated per capita loading during the review period was below typical per capita loading rates for BOD_5, TSS, and TP. However, the calculated per capita TKN loading rate was greater than typical.

Draft

3. DEVELOPMENT OF DESIGN BASIS

The following subsections outline the updated design basis in terms of raw wastewater flows and loadings for the Grand Valley WPCP. Similar to the previous design basis, flows and loads were developed for three future scenarios as follows:

- Scenario I: Full completion of planned residential developments;
- Scenario II: A 15% increase above the current CofA rated average day flow (ADF) (1,430 m^3/d); and,
- Scenario III: A 25% increase above the current CofA rated ADF (1,555 m^3/d).

3.1 Raw Wastewater Flows from the Collection System

3.1.1 Design Average Day Flow

Population projections for the Town were based on a recently completed review of future planned residential developments for the Town (R.J. Burnside, 2015).

New services corresponding to an equivalent population of 1,167 persons have been allocated by the Township, for a total equivalent service population of 2,974 based on the estimated 2015 existing service population. Details of planned developments were included in the design basis developed earlier in this study (XCG, 2015).

Projected future wastewater flows from planned developments (Scenario I) were based on a design dry weather per capita flow of 372 L/cap/d, and an average I/I allowance of 82 L/cap/d. Both values are based on the updated review of 2012 - May 2016 plant operating records. The overall design per capita wastewater flow for future development is 454 L/cap/d, contributing approximately 529 m^3/d on average to the plant. The existing average day flow is approximately 746 m^3/d, including septage contributions. For purposes of these projections, it is assumed future septage flows to the plant will be equal to the design treatment capacity (3.6 m^3/d). Plant records indicate the equivalent average daily septage flow treated at the plant is approximately 0.2 m^3/d, and therefore projections must consider an additional septage flow of 3.4 m^3/d.

The overall projected average day flow is approximately 1,279 m^3/d, which comparable to the CofA rated average day flow for the Grand Valley WPCP of 1,244 m^3/d.

Table 3.1 presents a summary of the ADF design basis for each presented scenario. For Scenario II and Scenario III, growth service populations were estimated from the increase in ADF and the design per capita flow of 454 L/cap/d (inclusive of I/I).

Draft

Table 3.1 Design Per Capita Flows, Populations, and ADFs

Source	Per Capita Flow (L/cap·d)[1]		Design Serviced Population			Added Septage Flow (m^3/d)	Design ADF (m^3/d)[2]
	Existing	New Growth	Existing	New Growth	Total		
Scenario I	454	454	1,752	1,167	2,919	3.4	1,279
Scenario II				1,508	3,260		1,430
Scenario III				1,784	3,536		1,555

Notes:
1. Inclusive of I/I flow allowance. Represents the average per capita flow observed over the review period.
2. Sum of base flow from the collection system (746 m^3/d from plant records), and growth flows from the collection and from received septage at the treatment plant.

3.1.2 Design Maximum Day Flow

The design MDF is based on the historic base MDF for the existing service area, plus a MDF allowance for future residential development. Details regarding the development of design maximum day flows are presented in the design basis developed earlier in this study (XCG, 2015). Design MDFs must also consider design maximum day septage flows of 11 m^3/d (R.J.Burnside, 2015). All design MDFs were based on the historic MDF observed at the Grand Valley WPCP. The updated conceptual level design MDF values for each phase are presented in Table 3.2.

Table 3.2 Design Maximum Day Flows

Parameter	Scenario I	Scenario II	Scenario III
Design ADF Existing Growth **Overall** [1]	 746 m^3/d 533 m^3/d **1,279 m^3/d**	 746 m^3/d 684 m^3/d **1,430 m^3/d**	 746 m^3/d 809 m^3/d **1,555 m^3/d**
MDF Factor Existing Growth **Overall** [1]	 6.3 2.2 **4.7**	 6.3 2.2 **4.3**	 6.3 2.2 **4.1**
Design MDF Existing Growth **Overall** [1]	 4,671 m^3/d 1,168 m^3/d **5,839 m^3/d**	 4,671 m^3/d 1,498 m^3/d **6,169 m^3/d**	 4,671 m^3/d 1,771 m^3/d **6,442 m^3/d**

Notes:
1. Projected maximum day raw wastewater flow from the collection system.

Therefore, the conceptual level design MDF flows are 5,839 m^3/d, 6,169 m^3/d, and 6,442 m^3/d for Scenario I, Scenario II, and Scenario III, respectively.

3.1.3 Design Peak Flows

As previously noted, peak flow data indicate that peak flow of raw wastewater from the collection system via the Emma St. SPS has approached 6,048 m^3/d. This peak flow was observed during a peak flow event in April 2014, resulting from both a large snow melt and precipitation event.

Future peak instantaneous flow (PIF) values were calculated based on the PIF observed over the review period, plus a peak flow allowance for new growth. Details regarding the development of peak instantaneous flows are presented in the design basis developed earlier in this study (XCG, 2015). The updated conceptual level design PIF values for each scenario are presented in Table 3.3.

Table 3.3 Design Peak Instantaneous Flows

Parameter	Scenario I	Scenario II	Scenario III
Design ADF			
Existing	746 m^3/d	746 m^3/d	746 m^3/d
Growth	533 m^3/d	684 m^3/d	809 m^3/d
Overall	**1,279 m^3/d**	**1,430 m^3/d**	**1,555 m^3/d**
PIF Factor			
Existing	10.2	10.2	10.2
Growth	3.3	3.3	3.3
Overall	**6.1**	**5.8**	**5.6**
Design PIF			
Existing	6,048 m^3/d	6,048 m^3/d	6,048 m^3/d
Growth	1,763 m^3/d	2,255 m^3/d	2,647 m^3/d
Overall	**7,811 m^3/d**	**8,303 m^3/d**	**8,695 m^3/d**

The conceptual level design PIF values are 7,811 m^3/d for Scenario I; 8,303 m^3/d for Scenario II; and 8,695 m^3/d for Scenario III.

The following important observations can be made based on results in Table 3.3:

- The overall design PIF factor for all scenarios is in excess of a typical peak factor given the equivalent service population of the Grand Valley WPCP. This is primarily a result of the large peak instantaneous flow observed in April 2014. Excessive peaking factors suggest the collection system may be susceptible to high extraneous flows during wet weather events; and,
- The projected PIF for all scenarios is in excess of the CofA rated Emma St. SPS capacity (7,680 m^3/d). This analysis suggests the Emma St. SPS may require upgrades at future flows provided that existing peak flows are not abated by any I/I reduction strategies. An extensive review of the Emma St. SPS capacity was not conducted as part of this review.

3.2 Raw Wastewater Loads

For purposes of developing loading projections, typical per capita loading rates were assumed for BOD_5, TSS, and TP. This is a conservative approach that accounts for the uncertainty of future development and the uncertainty in grab sample data collected during the review period. Future per capita TKN loadings were assumed to be identical to per capita loadings observed during the review period (2012 - 2014).

Estimations of maximum month loading factors were established from plant records of effluent flows and influent concentrations. Data from April 2014 was found to be outlying due to high observed flows, and was excluded from analysis. Maximum month factors were estimated to be 1.9, 1.9, 1.9, and 2.2 for BOD_5, TSS, TKN and TP,

Draft

respectively. Typical maximum month loading factors are much less than those observed at the Grand Valley WPCP, and range from 1.4 to 1.6. As previously discussed, raw influent quality data over the review period (2012 - 2014) represents results from a single grab sample, collected on a monthly basis. This sampling technique may result in increased variability in results. The discrepancy between typical maximum month loading factors and those observed at the Grand Valley WPCP may be in part related to the type and frequency of raw influent sample collection. In order to develop a conservative design basis, maximum month factors developed from plant data were used.

Base raw wastewater loading included contributions from the following sources:

- Raw wastewater from the collection system;
- Recycle flow from the onsite pumping station; and,
- Septage.

Wastewater from all three sources are combined at the plant headworks, upstream of the grab sample location. As such, it is assumed that raw wastewater quality collected over the review period is a representation of all three streams and, therefore, base wastewater loadings include contributions from all three sources.

Septage receiving facilities at the Grand Valley WPCP were designed to treat an average day septage flow of 3.6 m^3/d (R.J.Burnside, 2015). Plant operators have indicated that the septage receiving tank also receives drain water and some rain water from the plant. As such, accurate records of septage flow over the review period (2012 - May 2016) are not available. Using annual septage received records from plant operators, the estimated equivalent daily septage flow is 0.2 m^3/d. For purposes of loading projections, it is assumed the plant will receive the full design volume of septage when raw wastewater flows from the collection system reach the full projected capacity. Septage quality was assumed from typical values reported in literature (US EPA, 1984/1994).

Table 3.4 presents the projected future average day loadings to the Grand Valley WPCP.

Draft

Table 3.4 Design Average Raw Wastewater Loadings

Parameter	Base Raw Wastewater Loading	Loading Due to Growth (1,2,3)	Total Design Average Loading	Average Design Concentration
Scenario I				
BOD_5	88.2 kg/d	111 kg/d	200 kg/d	156 mg/L
TSS	112 kg/d	156 kg/d	268 kg/d	210 mg/L
TKN	28.2 kg/d	21.1 kg/d	49.3 kg/d	38.6 mg/L
TP	2.91 kg/d	3.30 kg/d	6.21 kg/d	4.85 mg/L
Scenario II				
BOD_5	88.2 kg/d	136 kg/d	225 kg/d	157 mg/L
TSS	112 kg/d	186 kg/d	298 kg/d	208 mg/L
TKN	28.2 kg/d	26.5 kg/d	54.7 kg/d	38.2 mg/L
TP	2.91 kg/d	4.00 kg/d	6.91 kg/d	4.83 mg/L
Scenario III				
BOD_5	88.2 kg/d	157 kg/d	245 kg/d	158 mg/L
TSS	112 kg/d	211 kg/d	322 kg/d	208 mg/L
TKN	28.2 kg/d	30.9 kg/d	59.1 kg/d	38.0 mg/L
TP	2.91 kg/d	4.58 kg/d	7.48 kg/d	4.81 mg/L

Notes:
1. Based on an assumed per capita loading of 75 g/cap/d for BOD5, 90 g/cap/d for TSS, 16.1 g/cap/d for TKN, and 2.1 g/cap/d for TP.
2. Based on an assumed population growth of 1,167 for Scenario 1, 1,500 for Scenario 2, and 1,775 for Scenario 3.
3. Assumed approximate 3.4 m^3/d increase in septage flows. Assumed septage quality (7,000 mg/L BOD_5, 15,000 mg/L TSS, 700 mg/L TKN, and 250 mg/L TP) as reported in literature (EPA 1984/1994)

Draft

The maximum monthly loadings were based on the maximum month loading peak factors observed over the review period for each parameter. The peak factors were 1.9 for BOD_5, 1.9 for TSS, 1.9 for TKN, and 2.2 for TP. Table 3.5 presents the design maximum monthly loadings to the Grand Valley WPCP.

Table 3.5 Design Maximum Month Raw Wastewater Loadings

Parameter	Average Design Wastewater Loading	Maximum Month Loading Peak Factor	Design Maximum Month Loading
Scenario I			
BOD_5	200 kg/d	1.9	379 kg/d
TSS	268 kg/d	1.9	509 kg/d
TKN	49.3 kg/d	1.9	93.7 kg/d
TP	6.21 kg/d	2.2	13.7 kg/d
Scenario II			
BOD_5	225 kg/d	1.9	427 kg/d
TSS	298 kg/d	1.9	566 kg/d
TKN	54.7 kg/d	1.9	104 kg/d
TP	6.91 kg/d	2.2	15.2 kg/d
Scenario III			
BOD_5	245 kg/d	1.9	466 kg/d
TSS	322 kg/d	1.9	613 kg/d
TKN	59.1 kg/d	1.9	112 kg/d
TP	7.48 kg/d	2.2	16.5 kg/d

Draft

4. SUMMARY OF PLANT FLOW AND LOAD PROJECTIONS

Table 4.1 contains a summary of the projected plant design basis flows and loads to the Grand Valley WPCP for all three scenarios. Projections of future plant loads were made using typical per capita loading rates, or based on the estimated historical per capita loading rate, whichever resulted in the more conservative estimate of future loads. Plant data collected from 2012 to May 2016 was used as part of this review.

Table 4.1 Summary of Design Basis

Parameter	Scenario I		Scenario II		Scenario III	
	Previous	Updated	Previous	Updated	Previous	Updated
Population	2,919	2,919	3,260	3,252	3,536	3,527
ADF	1,276 m^3/d	1,279 m^3/d	1,430 m^3/d		1,555 m^3/d	
MDF	5,828 m^3/d	5,839 m^3/d	6,165 m^3/d	6,169 m^3/d	6,439 m^3/d	6,442 m^3/d
MDF Factor	4.6		4.3		4.1	
PIF	7,811 m^3/d	7,811 m^3/d	8,303 m^3/d	8,291 m^3/d	8,695 m^3/d	8,684 m^3/d
PIF Factor	6.1		5.8		5.6	
BOD_5						
Avg. Load	186 kg/d	200 kg/d	211 kg/d	225 kg/d	232 kg/d	245 kg/d
Max Load	353 kg/d	379 kg/d	402 kg/d	427 kg/d	441 kg/d	466 kg/d
Avg. Conc.	146 mg/L	156 mg/L	148 mg/L	157 mg/L	149 mg/L	158 mg/L
TSS						
Avg. Load	239 kg/d	268 kg/d	269 kg/d	298 kg/d	294 kg/d	322 kg/d
Max Load	453 kg/d	509 kg/d	512 kg/d	566 kg/d	559 kg/d	613 kg/d
Avg. Conc.	187 mg/L	210 mg/L	188 mg/L	208 mg/L	189 mg/L	208 mg/L
TKN						
Avg. Load	47.9 kg/d	49.3 kg/d	53.4 kg/d	54.7 kg/d	57.9 kg/d	59.1 kg/d
Max Load	91.1 kg/d	93.7 kg/d	104 kg/d	104 kg/d	110 kg/d	112 kg/d
Avg. Conc.	37.6 mg/L	38.6 mg/L	37.4 mg/L	38.2 mg/L	37.2 mg/L	38.0 mg/L
TP						
Avg. Load	5.72 kg/d	6.21 kg/d	6.43 kg/d	6.91 kg/d	7.01 kg/d	7.48 kg/d
Max Load	12.6 kg/d	13.7 kg/d	14.2 kg/d	15.2 kg/d	15.4 kg/d	16.5 kg/d
Avg. Conc.	4.48 mg/L	4.85 mg/L	4.50 mg/L	4.83 mg/L	4.51 mg/L	4.81 mg/L

Draft

5. REFERENCES

1. Ministry of the Environment. Design Guidelines for Sewage Works. 2008.
2. Metcalf & Eddy. Wastewater Engineering: Treatment and Resource Recovery. Fifth Edition. Toronto. 2014.
3. R.J. Burnside & Associates Limited. Capacity at the Wastewater Plant. May 2015.
4. R.J. Burnside & Associates Limited. Township of East Luther Grand Valley - Grand Valley Wastewater Treatment Plant Operations Manual. July 2015.
5. R.J. Burnside & Associates Limited. Township of East Luther Grand Valley - ELGV Wastewater Treatment Plant Design Brief. October 2008.
6. R.J. Burnside & Associates Limited. East Luther Grand Valley - I and I Study Report Township of East Luther Grand Valley. July 2009.
7. US EPA. Handbook: Septage Treatment and Disposal. EPA-625/6-84-009. 1984.
8. US EPA. Guide to Septage Treatment and Disposal. EPA-625/R-94-002. 1994.

Draft

APPENDIX A
COPY OF PREVIOUSLY DEVELOPED DESIGN BASIS

Draft

XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

XCG File No.: 3-252-57-01
November 17, 2015

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY
PROPOSED DESIGN FLOWS AND LOADS

Draft

Prepared for:

TOWN OF GRAND VALLEY
5 Main Street, North
Grand Valley, Ontario
L9W 5S6

Attention: Jane Wilson

Prepared by:

XCG CONSULTING LIMITED
Suite 300, 2620 Bristol Circle
Oakville, Ontario
L6H 6Z7

TABLE OF CONTENTS

TABLE

APPENDICES

Draft

1. INTRODUCTION

1.1 Background

The Grand Valley WPCP provides treatment for wastewater generated in the community of Grand Valley within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of Environment and Climate Change (MOECC) Certificate of Approval (CofA) No. 9706-7KWQ57, issued on February 2, 2009. The quality and quantity of effluent currently discharged by the existing Water Pollution Control Plant (WPCP) is regulated by the CofA. The Grand Valley WPCP has a rated average capacity of 1,244 m^3/d.

XCG Consulting Limited (XCG) recently completed an update to the Assimilative Capacity Study to propose effluent limits associated with an increase in the rated capacity to 2,547 m^3/d. The proposed effluent limit associated with total phosphorus (TP) for this increased capacity was very low at 0.073 mg/L. Consistently achieving such low TP requirements requires enhanced tertiary treatment, such as dual-stage tertiary filtration or membrane ultrafiltration. Upgrading the Grand Valley WPCP to provide this level of treatment would require a significant capital expenditure.

At this time, the Town would like to investigate the potential to re-rate the existing WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. As such, the Town has retained XCG to undertake a capacity assessment of the Grand Valley WPCP to support a plant capacity re-rating.

1.2 Objectives

The specific objectives of this technical memorandum are to:

- Conduct a review of plant raw wastewater flows and loads; and,
- Develop a design basis for future raw wastewater flows and loads.

1.3 Data Sources

The following data sources were used in part to develop projections of plant flows and loads:

- 2012 to 2014 plant flow and quality information;
- Memorandum completed by R.J. Burnside regarding the existing and future service populations of the Grand Valley WPCP (May, 2015);
- East Luther Grand Valley (ELGV) Wastewater Treatment Plant Design Brief (2008);
- ELGV Inflow and Infiltration (I/I) Study Report (July, 2009);
- Grand Valley Wastewater Treatment Plant Operations Manual (July, 2015); and,
- Grand Valley WPCP facility tour (September, 2015).

Draft

2. REVIEW OF RAW WASTEWATER FLOW AND QUALITY

Raw sewage flows from the collection system are conveyed to the Grand Valley WPCP from the Emma St. sewage pumping station (SPS) via a forcemain. The Emma St. SPS is equipped with the following equipment:

- Two variable frequency drive (VFD) pumps (one duty and one standby), each with a rated capacity of 88.9 L/s (7,680 m^3/d);
- One VFD jockey pump with a rated capacity of 29.5 L/s (2,550 m^3/d); and,
- One wet well, with an approximate volume of 125 m^3.

Only one of the above pumps is in operation at a time. As such, the existing peak capacity of the Emma St. SPS is approximately 7,680 m^3/d. Over the review period (2012 - 2014) there were no records of raw sewage bypasses at the Emma St. SPS or at the Grand Valley WPCP.

It is important to note that a condition assessment of the Emma St. SPS was not completed as part of this study. Although the existing capacity of the Emma St. SPS was taken into consideration as part of the review of historic operating conditions, its capacity was not assumed to be a limiting factor when developing future anticipated peak flows at the Grand Valley WPCP.

2.1 Review of Raw Wastewater Flow over the Review Period (2012 - 2014)

The Grand Valley WPCP currently serves a residential population of approximately 1,752 persons. Influent flow to the Grand Valley WPCP is comprised of:

- Raw wastewater from the Grand Valley sanitary collection system, pumped to the plant via the Emma St. SPS;
- Septage flow from the onsite septage receiving station; and,
- Plant recycle flow (i.e. digester supernatant and filter backwash flow), pumped to the head of the plant from the onsite pumping station.

Flow from each source above is metered separately. Reported total influent flow to the plant is calculated as the sum of flow from each source. In addition, effluent flow is monitored using a V-notch weir. During a tour of plant treatment facilities, operators indicated the accumulation of grit within the magnetic flowmeter measuring flows from the Emma St. SPS led to false high measurements during the review period. As such, plant effluent flow measurements were used as the basis for the evaluation of average raw wastewater flows from the Grand Valley sanitary collection system over the review period (2012 - 2014).

Table 2.1 presents a summary of the estimated collection system raw influent flow and per capita flows to the Grand Valley WPCP. The table includes an estimation of dry weather plant flow and per capita flows, and quantification of the historical I/I observed at the plant. Meteorological data was obtained from the Environment Canada station at Fergus, Ontario. Days were considered dry when no precipitation occurred for that day and three days prior. Only data from May to October was used for dry weather flow analysis.

Draft

Table 2.1 Summary of Treated Flow over the Review Period (2012 - 2014)

	Units	2012	2013	2014	Overall [1]
Estimated Service Population	Persons	1,494	1,683	1,752	-
Average Daily Flow	m^3/d	643	821	776	746
Per Capita Flow	L/cap/d	430	488	443	454
Estimated Dry Weather Flow [2]	m^3/d	554	658	620	603
Estimated Per Capita Dry Weather Flow	L/cap/d	371	391	354	372
Estimated Per Capita I/I	L/cap/d	59	97	89	82

Notes:
Estimated flows are based on flow measurements taken at the effluent flow meter over the review period.
1. Represents the average flow over the entire review period (2012 - 2014).
2. Days were considered dry when no precipitation occurred for that day, and two days prior from May to September.

Results in Table 2.1 indicate that the overall average per capita flow to the Grand Valley WPCP over the review period was 454 L/cap/d, inclusive of I/I. The estimated dry weather per capita flow (372 L/cap/d) is consistent with the typical range of per capita flows of 225 to 450 L/cap/d, exclusive of extraneous flows (MOE, 2008). The calculated per capita I/I was 82 L/cap/d, which is slightly less than the typical design I/I flow of 90 L/cap/d (MOE, 1985).

Summary of Maximum Day Flows during the Review Period (2012 - 2014)

Similar to average day flow analysis, maximum day flows for 2012 and 2013 were estimated from effluent flow meter measurements. In 2014, the maximum day flow event (April 14, 2014) was caused by simultaneous rainfall and snow melt events, and required use of the storm tank to equalize peak flows through the secondary treatment train. Volume accumulated in the storm tank was returned to the head of the plant in the days following the peak flow event. As such, the measured flow at the effluent flow meter is not an accurate representation of total maximum day influent flow in 2014.

As previously discussed, the accumulation of grit at the Emma St. SPS flow meter has caused false high flow measurements over the review period (2012 - 2014). However, during the seven days preceding the peak flow event in 2014, the average percent difference between flows measured at the Emma St. SPS and at the effluent flow meter was 3%. Therefore, it was assumed that flow measured at the Emma St. SPS represents an accurate estimation of total influent flow to the Grand Valley WPCP during the peak flow event recorded in April 2014. A summary of maximum day flows and calculated maximum day factors (MDF) during the review period is shown as Table 2.2.

Results in Table 2.2 indicate the Grand Valley WPCP has been subject to significant peak flows over the review period. Specifically, the extreme peak flows observed in 2014 are attributed to simultaneous snow melt and rain fall events in April 2014. There have been no recorded observations of raw wastewater bypass during the review period.

Draft

Table 2.2 Summary Maximum Day Flows over the Review Period (2012 - 2014)

	Units	2012	2013	2014	Overall
Average Daily Flow	m^3/d	643	821	776	746
Maximum Day Flow	m^3/d	2,601	2,254	4,671 (1)	4,671 (1)
MDF	-	4.0	2.8	6.0	6.3

Notes:
Unless otherwise indicated, flows are based on flow measurements taken at the effluent flow meter over the review period (2012 - 2014)
1. Based on Emma St. SPS flow measurements.

Summary of Peak Flows during the Review Period (2012 - 2014)

As discussed, operators have indicated that the accumulation of grit within the magnetic flow meter has contributed to false high measurements of flow from the Emma St. SPS. In 2015, operators began periodically operating the Emma St. SPS pump at capacity to flush any accumulated solids from the magnetic flow meter. Since beginning this practice, operators have reported consistent agreement between influent and effluent flow measurements.

Similarly, it is likely that peak flow periods which occurred during the review period, and which required pumps at the Emma St. SPS to run at or near capacity, would remove any accumulated grit at the magnetic flow meter. Therefore, it was assumed that peak flow data collected from the Emma St. SPS represents an accurate representation of peak flows to the Grand Valley WPCP during the review period (2012 - 2014).

For selected days with high measured effluent flows, measured flow from the Emma St. SPS was further analyzed to understand the existing peak flows to the plant. Specifically, several days from the peak flow event in April 2014 were examined. A SCADA screenshot of Emma St. SPS flows on April 13 and April 14, 2014 is included as Appendix A.

During these days, the observed peak flow from the Emma St. SPS reached approximately 88 L/s, which is near the rated capacity of the SPS. However, detailed analysis of these figures suggests that the observed peak flows are likely related to pump operation at the Emma St. SPS rather than actual raw influent flow to the wet well. Plant operations staff have indicated that the VFD of the large duty pump was programmed to operate between 60 L/s and 90 L/s. As indicated, the capacity of the jockey pump is approximately 29.5 L/s. Influent flow greater than the jockey pump capacity, but less than the minimum programmed operation of the large duty pump is likely the cause of unstable periods of pump operation, characterized by rapid changes in pumping output and cycling of pump on/off cycles. These unstable periods are detailed in the screenshots included in Appendix A. During the morning of April 14, 2014, operations staff modified operation of the VFD control to allow the large pump to operate between 40 L/s and 89 L/s in an attempt to smooth pump output during this high flow event. This can be clearly seen on Figure A.2 in Appendix A. It is recommended the Town conduct further investigation into the PLC programming at the Emma St. SPS to optimize pumping control if required.

Excluding periods of unstable pump operation, the peak flow from the collection system was estimated to be approximately 70 L/s (6,048 m^3/d) during the review period (2012 - 2014).

Evaluation of Plant Recycle and Septage Flows over the Review Period (2012 - 2014)

Decant flow from the aerobic digester and backwash flow from the tertiary filters are directed to the onsite pumping station, which pumps flow to the head of the plant, upstream

Draft

of the plant headworks. Flow from the pumping station is measured with a magnetic flow meter. Over the review period, measured flow from the onsite pumping station represented approximately 12% of the final effluent measured flow. On an average monthly basis, there was a positive linear correlation between the measured final effluent flow and the measured flow from the onsite pumping station. As such, plant recycle flow is expected to increase as raw wastewater flows increase.

Flow from the onsite septage receiving tank is also metered. Plant operators have indicated there are some drains and rain water which are directed to the onsite septage receiving tank. Over the review period, the plant has received an average of approximately 11 m^3/d of flow from the septage receiving tank. However, due to the contributions from the connected drains, this value overestimates the actual volume of septage received at the Grand Valley WPCP.

Plant operators also indicated that issues were experienced with solenoids associated with wash water for the screening and grit removal system sticking in the open position, resulting in potable water flowing directly into the liquid stream. This flow is not measured directly, however it contributes to the measured effluent flow from the WPCP. The impact of these valves on total effluent wastewater flow is expected to be negligible.

2.2 Analysis of Inflow / Infiltration in the Collection System

The Town has recently conducted an investigation of I/I in the collection system (RJ Burnside, 2009). The investigation found significant volumes of I/I in the Grand Valley collection system. The investigation identified structural deficiencies at several manholes, but observed that the overall structural integrity of the collection system was not a significant factor contributing to I/I. Instead, it identified that significant I/I flows are generated on private property, specifically from the direct connection of footings to the sanitary collection system. Historically, the implementation of I/I reduction strategies on private property is difficult. The Town and R.J. Burnside have indicated they are currently pursuing provincial funding assistance to conduct an I/I reduction program.

Overall, I/I in the Grand Valley collection system impacts the magnitude of peak flows to the Emma St. SPS, and flow to the Grand Valley WPCP. It is important to note that several treatment processes at the Grand Valley WPCP are dependent on the maximum day and peak raw wastewater flows. As such, I/I may directly impact the available treatment capacity at the Grand Valley WPCP. Implementation of an I/I reduction strategy may reduce the intensity of peak flows to the Grand Valley WPCP in the future.

2.3 Plant Influent Raw Wastewater Quality during the Review Period (2012 - 2014)

Over the review period, grab samples of the raw wastewater stream were collected monthly. Samples were collected immediately upstream of the influent screens, and are representative of the plant influent raw wastewater flow. It includes contributions from the collection system raw wastewater, septage, tertiary filter backwash, and digester supernatant.

Table 2.3 presents a summary of the plant influent raw wastewater quality over the review period (2012 - 2014).

Generally, the combined influent was found to be of low strength with respect to biological oxygen demand (BOD_5), total suspended solids (TSS), and TP, and of low to medium strength with respect to total Kjeldahl nitrogen (TKN).

As discussed, only grab samples of the combined influent stream were collected during the review period (2012 - 2014). These samples are a representation of influent quality at the

Draft

moment they are collected, but may not be an accurate representation of the average influent quality over 24 hours. Therefore, the results presented in Table 2.3 may not accurately represent average combined influent quality.

Table 2.3 Plant Influent Raw Wastewater Characteristics

Parameter	Units	Plant Influent Raw Wastewater (1)	Typical Wastewater Strength (2)
BOD_5	(mg/L)	105	110 *(Low)* 190 *(Med)* 350 *(High)*
TSS	(mg/L)	133	120 *(Low)* 210 *(Med)* 400 *(High)*
TKN	(mg/L)	33.4	20 *(Low)* 40 *(Med)* 70 *(High)*
TP	(mg/L)	3.45	4 *(Low)* 7 *(Med)* 12 *(High)*

Notes:
BOD - Biochemical Oxygen Demand
TSS - Total Suspended Solids
TKN - Total Kjeldahl Nitrogen
TP - Total Phosphorus
1. Includes filter backwash and digester supernatant recycle streams.
2. Metcalf and Eddy (2003).

2.4 Liquid Train Influent Loadings during the Review Period

Using results presented in Table 2.3 and the estimated average day plant flow over the review period, Table 2.4 presents a summary of the average day liquid train loading and per capita loading from data collected during the review period. This assumes a current service population of approximately 1,752.

Table 2.4 Summary of Plant and Per Capita Loading over the Review Period (2012 - 2014)

Parameter	Average Daily Load (kg/d) (1)	Historic Per Capita Load (g/cap/d)	Typical Per Capita Load (g/cap/d)
BOD_5	88.2	50.4	75 (2)
TSS	112	64.0	90 (2)
TKN	28.2	16.1	13.3 (3)
TP	2.91	1.66	2.1 (3)

Notes
1. Includes loading from recycle streams (digester supernatant and tertiary filter backwash), and from septage.
2. As per Design Guidelines for Sewage Works (MOE, 2008).
3. As per Metcalf and Eddy, 2010.

From the table above, the estimated per capita loading during the review period was below typical per capita loading rates for BOD_5, TSS, and TP. However, the estimated per capita TKN loading rate was greater than typical.

Draft

3. DEVELOPMENT OF DESIGN BASIS

The following subsections outline the future design basis in terms of raw wastewater flows and loadings for the Grand Valley WPCP. This design basis will be used to evaluate the capacity of the Grand Valley WPCP from both a hydraulic and biological treatment perspective in subsequent phases of this study.

For the purposes of this evaluation, flows and loads were developed for three future scenarios as follows:

- Scenario I: Full completion of planned residential developments;
- Scenario II: A 15% increase above the current CofA rated average day flow (ADF) (1,430 m^3/d); and,
- Scenario III: A 25% increase above the current CofA rated ADF (1,555 m^3/d).

3.1 Raw Wastewater Flows from the Collection System

3.1.1 Design Average Day Flow

Population projections for the Town were based on a recently completed review of future planned residential developments for the Town (R.J. Burnside, 2015). Specifically, future planned developments consist of:

- 321 housing units constructed as part of three residential developments (Mayberry Phase 1 and 2, and Hollenbeck); and,
- The 'Moco Allocation', consisting of 7 residential units and 15.3 hectares of developable land.

A summary of these planned residential developments is presented in Table 3.1.

Table 3.1 Summary of Serviced New Developments

Development	Population
Moco Allocation (1)	192
Mayberry Phase 1	190
Mayberry Phase 2	507
Hollenbeck	278
Total Additional Equivalent Service Population from New Developments	1,167

Notes:
1. Equivalent service population, consisting of serviced residential lots, and developed land.

New services corresponding to an equivalent population of 1,167 persons have been allocated by the Township, for a total equivalent service population of approximately 2,919.

Draft

Projected future wastewater flows from planned developments (Scenario I) were based on a design dry weather per capita flow of 372 L/cap/d, and an average I/I allowance of 82 L/cap/d. Both values are based on a review of 2012 - 2014 plant operating records. The overall design per capita wastewater flow for future development (454 L/cap/d) is identical to the 3-year average observed at the plant.

From Table 3.1, the estimated new equivalent service population associated with completion of all planned developments is 1,167 and is projected to contribute approximately 529 m^3/d on average to the plant. The existing average day flow is 746 m^3/d. Therefore, the overall projected average day flow is 1,276 m^3/d, which is only 32 m^3/d more than the CofA rated average day flow for the Grand Valley WPCP of 1,244 m^3/d.

Table 3.2 presents a summary of the ADF design basis for each presented scenario. For Scenario II and Scenario III, growth service populations were estimated from the increase in ADF and the design per capita flow.

Table 3.2 Design Per Capita Flows, Populations, and ADFs

Source	Per Capita Flow (L/cap·d)(1)		Design Serviced Population			Design ADF (m^3/d) (2)
	Existing	New Growth	Existing	New Growth	Total	
Scenario I	454	454	1,752	1,167	2,919	1,276
Scenario II				1,508	3,260	1,430
Scenario III				1,784	3,536	1,555

Notes:
1. Inclusive of I/I flow allowance.
2. Raw wastewater from the collection system.

Draft

3.1.2 Design Maximum Day Flow

The design MDF is based on the historic base MDF for the existing service area, plus a MDF allowance for future residential development.

To calculate the MDF allowance for new growth, a MDF peaking factor for the new growth flows was determined. This was done by applying the historic dry weather flow (DWF) factor to the non-I/I portion of the per capita flow rate, and applying a typical per capita generation rate of 227 L/cap/d for I/I flows (MOE, 2008).

A dry weather flow analysis was completed to determine the historic DWF factor. The analysis of DWF was conducted based on flow data from 2012 to 2014 and meteorological data from Environment Canada. Days were considered to be "dry" when no precipitation occurred for that day and three days prior between the months of May and October, inclusive. Based on the flow analysis, the historic DWF peaking factor for the existing service area was 2.1. In addition, the existing per capita DWF for the residential service area was estimated to be 372 L/cap/d, based on a service population of 1,752, and the existing I/I flow was estimated to be 82 m^3/d. Details of existing flows are presented in Table 2.1.

By applying the historic DWF peaking factor of 2.1 to the dry weather flow portion of the per capita flow, and the I/I flow peak factor to the I/I portion of the per capita flow, the overall MDF peaking factor for new growth was determined to be 2.2.

To determine the conceptual level design MDF for each phase, the new growth MDF factors were applied to the increase in average day design flows for each phase, and these growth MDF values were added to the existing base MDF. The conceptual level design MDF values for each phase are presented in Table 3.3.

Table 3.3 Design Maximum Day Flows

Parameter	Scenario I	Scenario II	Scenario III
Design ADF			
Existing	746 m^3/d	746 m^3/d	746 m^3/d
Growth	529 m^3/d	684 m^3/d	809 m^3/d
Overall (1)	**1,276 m^3/d**	**1,430 m^3/d**	**1,555 m^3/d**
MDF Factor			
Existing	6.3	6.3	6.3
Growth	2.2	2.2	2.2
Overall (1)	**4.6**	**4.3**	**4.1**
Design MDF			
Existing	4,671 m^3/d	4,671 m^3/d	4,671 m^3/d
Growth	1,157 m^3/d	1,494 m^3/d	1,768 m^3/d
Overall (1)	**5,828 m^3/d**	**6,165 m^3/d**	**6,439 m^3/d**

Notes:
1. Projected maximum day raw wastewater flow from the collection system.

Therefore, the conceptual level design MDF flows are 5,828 m^3/d, 6,165 m^3/d, and 6,439 m^3/d for Scenario I, Scenario II, and Scenario III, respectively.

3.1.3 Design Peak Flows

As previously noted, peak flow data indicate that peak flow of raw wastewater from the collection system via the Emma St. SPS has approached 6,048 m^3/d. This peak flow was observed during a peak flow event in April 2014, resulting from both a large snow melt and precipitation event.

Future peak instantaneous flow (PIF) values were calculated based on the PIF observed over the review period, plus a peak flow allowance for new growth. To calculate the PIF allowance for new growth, a PIF peaking factor for the new growth flows was determined for each design scenario. This was done by applying the Harmon peaking factor to the non-I/I portion of the per capita flow value, and applying a typical per capita peak I/I flow rate of 227 L/cap/d (MOE, 2008). The Harmon peaking factor was calculated for each phase based on the overall design equivalent populations of 2,919 for Scenario I; 3,260 for Scenario II; and 3,536 for Scenario III. Accordingly, the Harmon peaking factors for Scenarios I, II, and III were determined to be 3.5, 3.4, and 3.4, respectively.

By applying the appropriate Harmon peaking factor to the dry weather flow portion of the per capita flow, and the I/I flow peak factor to the I/I portion of the per capita flow, the overall PIF peaking factor for new growth was determined to be 3.3 for all three scenarios.

Draft

To determine the conceptual level design PIF for each scenario, the new growth PIF peaking factors were applied to the increase in average day design flows for each phase, and these growth PIF values were added to the existing base PIF. For the purposes of this conceptual level design basis, the PIF factor for new growth was applied to the growth flows. The conceptual level design PIF values for each phase are presented in Table 3.4.

Table 3.4 Design Peak Instantaneous Flows

Parameter	Scenario I	Scenario II	Scenario III
Design ADF			
Existing	746 m^3/d	746 m^3/d	746 m^3/d
Growth	529 m^3/d	684 m^3/d	809 m^3/d
Overall	**1,276 m^3/d**	**1,430 m^3/d**	**1,555 m^3/d**
PIF Factor			
Existing	10.2	10.2	10.2
Growth	3.3	3.3	3.3
Overall	**6.1**	**5.8**	**5.6**
Design PIF			
Existing	6,048 m^3/d	6,048 m^3/d	6,048 m^3/d
Growth	1,763 m^3/d	2,255 m^3/d	2,647 m^3/d
Overall	**7,811 m^3/d**	**8,303 m^3/d**	**8,695 m^3/d**

Draft

The conceptual level design PIF values are 7,811 m^3/d for Scenario I; 8,303 m^3/d for Scenario II; and 8,695 m^3/d for Scenario III.

The following important observations can be made based on results in Table 3.4:

- The overall design PIF factor for all scenarios is in excess of a typical peak factor given the equivalent service population of the Grand Valley WPCP. This is primarily a result of the large peak instantaneous flow observed in April 2014. Excessive peaking factors suggest the collection system may be susceptible to high extraneous flows during wet weather events; and,
- The projected PIF for all scenarios is in excess of the CofA rated Emma St. SPS capacity (7,680 m^3/d). This analysis suggests the Emma St. SPS may require upgrades at future flows provided that existing peak flows are not abated by any I/I reduction strategies. An extensive review of the Emma St. SPS capacity was not conducted as part of this review.

3.2 Raw Wastewater Loads

For purposes of developing loading projections, typical per capita loading rates were assumed for BOD_5, TSS, and TP. This is a conservative approach that accounts for the uncertainty of future development, and the uncertainty in grab sample data collected during the review period. Future per capita TKN loadings were assumed to be identical to per capita loadings observed during the review period (2012 - 2014).

Estimations of maximum month loading factors were established from plant records of effluent flows and influent concentrations. Data from April 2014 was found to be outlying due to high observed flows, and was excluded from analysis. Maximum month factors were estimated to be 1.9, 1.9, 1.9, and 2.2 for BOD_5, TSS, TKN and TP, respectively. Typical maximum month loading factors are much less than those observed at the Grand Valley

WPCP, and range from 1.4 to 1.6. As previously discussed, raw influent quality data over the review period (2012 - 2014) represents results from a single grab sample, collected on a monthly basis. This sampling technique may result in increased variability in results. The discrepancy between typical maximum month loading factors and those observed at the Grand Valley WPCP may be in part related to the type and frequency of raw influent sample collection. In order to develop a conservative design basis, maximum month factors developed from plant data were used.

Base raw wastewater loading included contributions from the following sources:

- Raw wastewater from the collection system;
- Recycle flow from the onsite pumping station; and,
- Septage.

Wastewater from all three sources are combined at the plant headworks, upstream of the grab sample location. As such, it is assumed that raw wastewater quality collected over the review period is a representation of all three streams and, therefore, base wastewater loadings include contributions from all three sources.

Septage receiving facilities at the Grand Valley WPCP were designed to treat an average day septage flow of 3.6 m^3/d. Plant operators have indicated that the septage receiving tank also receives drain water and some rain water from the plant. As such, accurate records of septage flow over the review period (2012 - 2014) are not available. Currently, the plant is operating at approximately 60% of its CofA rated ADF capacity of 1,244 m^3/d. For purposes of loading projections, it is assumed the plant also receives 60% of its designed septage capacity (i.e. approximately 2.2 m^3/d), and will receive the full design volume of septage when raw wastewater flows from the collection system reach the full projected capacity. Septage quality was assumed from typical values reported in literature (US EPA, 1984/1994).

Table 3.5 presents the projected future average day loadings to the Grand Valley WPCP.

Draft

Table 3.5 Design Average Raw Wastewater Loadings

Parameter	Base Raw Wastewater Loading	Loading Due to Growth (1,2,3)	Total Design Average Loading	Average Design Concentration
Scenario I				
BOD_5	88.2 kg/d	97.6 kg/d	186 kg/d	146 mg/L
TSS	112 kg/d	127 kg/d	239 kg/d	187 mg/L
TKN	28.2 kg/d	19.8 kg/d	47.9 kg/d	37.6 mg/L
TP	2.91 kg/d	2.81 kg/d	5.72 kg/d	4.48 mg/L
Scenario II				
BOD_5	88.2 kg/d	123 kg/d	211 kg/d	148 mg/L
TSS	112 kg/d	157 kg/d	269 kg/d	188 mg/L
TKN	28.2 kg/d	25.3 kg/d	53.4 kg/d	37.4 mg/L
TP	2.91 kg/d	3.53 kg/d	6.43 kg/d	4.50 mg/L
Scenario III				
BOD_5	88.2 kg/d	144 kg/d	232 kg/d	149 mg/L
TSS	112 kg/d	182 kg/d	294 kg/d	189 mg/L
TKN	28.2 kg/d	29.7 kg/d	57.9 kg/d	37.2 mg/L
TP	2.91 kg/d	4.11 kg/d	7.01 kg/d	4.51 mg/L

Notes:
1. Based on an assumed per capita loading of 75 g/cap/d for BOD5, 90 g/cap/d for TSS, 15.86 g/cap/d for TKN, and 2.1 g/cap/d for TP.
2. Based on an assumed population growth of 1,167 for Scenario 1, 1,515 for Scenario 2, and 1,793 for Scenario 3.
3. Assumed approximate 1.4 m^3/d increase in septage flows. Assumed septage quality (7,000 mg/L BOD_5, 15,000 mg/L TSS, 700 mg/L TKN, and 250 mg/L TP) as reported in literature (EPA 1984/1994)

The maximum monthly loadings were based on the maximum month loading peak factors observed over the review period for each parameter. The peak factors were 1.9 for BOD_5, 1.9 for TSS, 1.9 for TKN, and 2.2 for TP. Table 3.6 presents the design maximum monthly loadings to the Grand Valley WPCP.

Draft

Table 3.6 Design Maximum Month Raw Wastewater Loadings

Parameter	Average Design Wastewater Loading	Maximum Month Loading Peak Factor	Design Maximum Month Loading
Scenario I			
BOD_5	186 kg/d	1.9	353 kg/d
TSS	239 kg/d	1.9	453 kg/d
TKN	47.9 kg/d	1.9	91.1 kg/d
TP	5.72 kg/d	2.2	12.6 kg/d
Scenario II			
BOD_5	211 kg/d	1.9	402 kg/d
TSS	269 kg/d	1.9	512 kg/d
TKN	53.4 kg/d	1.9	101 kg/d
TP	6.43 kg/d	2.2	14.2 kg/d
Scenario III			
BOD_5	232 kg/d	1.9	441 kg/d
TSS	294 kg/d	1.9	559 kg/d
TKN	57.9 kg/d	1.9	110 kg/d
TP	7.01 kg/d	2.2	15.4 kg/d

Draft

4. SUMMARY OF PLANT FLOW AND LOAD PROJECTIONS

Table 4.1 contains a summary of the projected plant design basis flows and loads to the Grand Valley WPCP for all three scenarios. Projections of future plant loads were made using typical per capita loading rates, or based on the estimated historical per capita loading rate, whichever resulted in the more conservative estimate of future loads. Plant data collected from 2012 to 2014 was used as part of this review.

Table 4.1 Summary of Design Basis

Parameter	Scenario I	Scenario II	Scenario III
Population	2,919	3,260	3,536
ADF	1,276 m^3/d	1,430 m^3/d	1,555 m^3/d
MDF	5,828 m^3/d	6,165 m^3/d	6,439 m^3/d
MDF Factor	4.6	4.3	4.1
PIF	7,811 m^3/d	8,303 m^3/d	8,695 m^3/d
PIF Factor	6.1	5.8	5.6
BOD_5			
Average Loading	186 kg/d	211 kg/d	232 kg/d
Maximum Month Loading	353 kg/d	402 kg/d	441 kg/d
Average Concentration	146 mg/L	148 mg/L	149 mg/L
TSS			
Average Loading	239 kg/d	269 kg/d	294 kg/d
Maximum Month Loading	453 kg/d	512 kg/d	559 kg/d
Average Concentration	187 mg/L	188 mg/L	189 mg/L
TKN			
Average Loading	47.9 kg/d	53.4 kg/d	57.9 kg/d
Maximum Month Loading	91.1 kg/d	101 kg/d	110 kg/d
Average Concentration	37.6 mg/L	37.4 mg/L	37.2 mg/L
TP			
Average Loading	5.72 kg/d	6.43 kg/d	7.01 kg/d
Maximum Month Loading	12.6 kg/d	14.2 kg/d	15.4 kg/d
Average Concentration	4.48 mg/L	4.50 mg/L	4.51 mg/L

Draft

5. REFERENCES

Ministry of the Environment. Design Guidelines for Sewage Works. 2008.

Metcalf & Eddy. Wastewater Engineering: Treatment and Resource Recovery. Fifth Edition. Toronto. 2014.

R.J. Burnside & Associates Limited. Capacity at the Wastewater Plant. May 2015.

R.J. Burnside & Associates Limited. Township of East Luther Grand Valley - Grand Valley Wastewater Treatment Plant Operations Manual. July 2015.

R.J. Burnside & Associates Limited. Township of East Luther Grand Valley - ELGV Wastewater Treatment Plant Design Brief. October 2008.

R.J. Burnside & Associates Limited. East Luther Grand Valley - I and I Study Report Township of East Luther Grand Valley. July 2009.

US EPA. Handbook: Septage Treatment and Disposal. EPA-625/6-84-009. 1984.

US EPA. Guide to Septage Treatment and Disposal. EPA-625/R-94-002. 1994.

Draft

APPENDIX A
SCREENSHOTS OF EMMA ST. SPS MEASURED FLOW

Draft

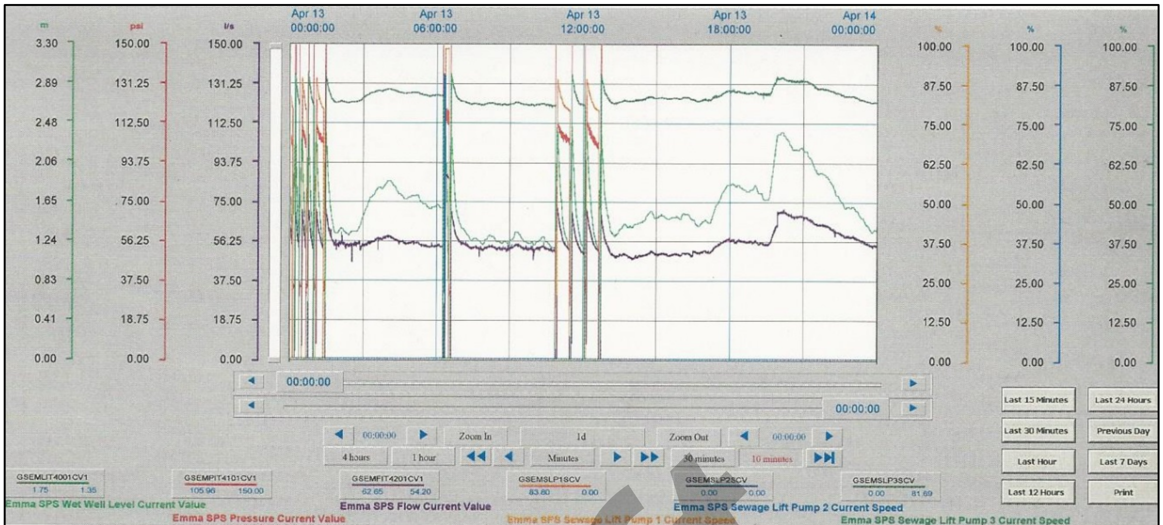


Figure A.1 Emma St. SPS Measured Flows - April 13, 2014

Figure A.2 Emma St. SPS Measured Flows - April 14, 2014

Draft

APPENDIX C
GRAND VALLEY WPCP HEADWORKS HYDRAULICS ANALYSIS

Draft

XCG Environmental Engineers & Scientists
XCG CONSULTANTS LTD.
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

Date:	December 6, 2016	**XCG File No.: 3-252-57-02**
To:	Jane Wilson, Town of Grand Valley	
From:	XCG Consultants Ltd (XCG)	
Re:	Grand Valley WPCP Headworks Hydraulics Analysis	

Draft

1. INTRODUCTION

The Grand Valley Water Pollution Control Plant (WPCP) provides treatment for wastewater generated in the community of Grand Valley, within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) and has a rated average day flow (ADF) capacity of 1,244 m^3/d.

The town has initiated an investigation to analyze the potential to re-rate the existing Grand Valley WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. The Town has retained XCG Consultants Ltd. (XCG) to conduct a capacity evaluation and re-rating study of the Grand Valley WPCP to potentially defer the next required plant expansion.

The purpose of this memorandum is to present the methodology and results of the hydraulic analysis of the Grand Valley WPCP headworks facilities.

2. BACKGROUND

2.1 Future Design Basis

For purposes of this capacity evaluation, three future design scenarios are being considered:

- Scenario I: Full completion of planned residential developments;
- Scenario II: A 15% increase above the current C of A rated ADF (1,430 m^3/d); and
- Scenario III: A 25% increase above the current C of A rated ADF (1,555 m^3/d).

A summary of the Grand Valley WPCP flow design basis is included in Table 2.1. This table includes flow details as presented in the updated design basis (XCG, 2016), but does not include comparison to previous design basis projections nor projections of parameter loads. Flows shown in Table 2.1 represent the projected raw influent flow from the collection system to the Grand Valley WPCP. It is important to note the projected peak flows for all three scenarios exceed the existing rated capacity of the Emma St. SPS (7,680 m^3/d). Therefore, the Emma St. SPS may require upgrades at future flows provided that existing peak flows are not abated by any I/I reduction strategies. An extensive review of the Emma St. SPS capacity was not conducted as part of this review. Further, it is assumed that future peak flows to the Grand Valley WPCP will not be inhibited by the pumping capacity of the Emma St. SPS.

Table 2.1 Summary of Raw Influent Flow from the Collection System (XCG, 2016)

Parameter	Scenario I	Scenario II	Scenario III
Population	2,919	3,252	3,527
ADF	1,279 m^3/d	1,430 m^3/d	1,555 m^3/d
MDF	5,839 m^3/d	6,169 m^3/d	6,442 m^3/d
MDF Factor	4.6	4.3	4.1
PIF	7,811 m^3/d	8,291 m^3/d	8,684 m^3/d
PIF Factor	6.1	5.8	5.6

However, backwash flow from the tertiary filters is discharged to the on-site pumping station where it is pumped to the head of the plant upstream of the plant headworks. As such, hydraulic analysis of the plant headworks must also consider peak flow from the onsite pumping station.

The on-site pumping station is equipped with two pumps, one duty and one standby. However, records of plant operation indicate that both pumps will operate under peak flow conditions. Both pumps have a rated capacity of 8.0 L/s (691 m^3/d), but the peak pumping rate when both pumps are in operation is approximately 11 L/s (950 m^3/d).

Headworks at the Grand Valley WPCP consists of screening and grit removal. The capacity of these processes is evaluated based on peak instantaneous and peak hour flows, respectively. Table 2.2 summarizes the projected peak flow through the plant headworks considering contributions from the Emma St. SPS (i.e. raw influent from the collection system) and from the onsite pumping station (i.e. tertiary filter backwash flow).

Table 2.2 Summary of Peak Flow through the Grand Valley WPCP Headworks

Peak Flow	Scenario I	Scenario II	Scenario III
Emma St. SPS (Collection System)	7,811 m^3/d	8,291 m^3/d	8,684 m^3/d
Onsite Pumping Station (Filter Backwash)		950 m^3/d	
Total Projected Peak Instantaneous Flow	8,761 m^3/d	9,241 m^3/d	9,634 m^3/d
Total Projected Peak Hour Flow (1)	7,885 m^3/d	8,317 m^3/d	8,670 m^3/d

Notes:
1. Assumed to be 90% of the peak instantaneous flow.

2.2 Existing Plant Headworks

As previously noted, headworks at the Grand Valley WPCP consists of screening and grit removal processes. Screening is provided by one perforated plate type mechanical screen, operating as the duty screen, and one manually raked bar screen operating in stand-by. The mechanical screen has a rated capacity of 7,680 m^3/d based on the C of A and the plant

Draft

operations manual (RJ Burnside, 2015). Screenings are collected and compacted then transferred to a bin and disposed off-site. The quantity of screenings generated at the Grand Valley WPCP is not measured; therefore the performance of the screens in terms of screenings generation per m^3 of wastewater treated could not be assessed.

Flow to the manual screen channel is controlled by a gate. Under typical flow conditions, the gate remains closed, thereby directing all flow through the mechanical screen. In the closed position, the top elevation of the gate is well below the elevation at the top of the channel. As such, in the closed position, the gate serves as an emergency bypass weir. Peak flows which exceed the elevation at the top of the gate will automatically bypass the mechanical screen through the manual screen channel.

Grit removal is provided by two vortex grit separators, each 1.83 metres in diameter. The rated capacity of each vortex grit separator is 3,840 m^3/d, for a total peak capacity of 7,680 m^3/d. Grit from both separators is collected and compacted then transferred to a bin and disposed off-site. The quantity of grit generated at the Grand Valley WPCP is not measured; therefore the performance of the grit separators in terms of volume generation per m^3 of wastewater treated could not be assessed.

A bypass exists around the vortex grit separators which transports screened raw influent wastewater to the raw wastewater flow splitter box located upstream of biological treatment at the Grand Valley WPCP. Grit bypass is controlled by an overflow weir which has a set elevation. It is assumed the height of the weir controls flow through the grit removal process to the design peak flow (7,680 m^3/d).

A summary of the Grand Valley WPCP headworks treatment process design information is included in Table 2.3.

Table 2.3 Grand Valley WPCP Headworks Process Design Information

Unit Process	Design Parameter [(1)]
Preliminary Treatment	
Screening	
Type	Mechanical and Manual Bar
Number	1 mechanical (duty) 1 bar (standby)
Peak Flow Capacity (mechanical screen)	7,680 m^3/d
Grit Removal	
Type	Vortex
Number	2
Capacity	3,840 m^3/d (each) 7,680 m^3/d (total)

Notes:
1. Based on Amended Certificate of Approval Number 9706-7KWQ57, issued February 2, 2009 and Grand Valley Wastewater Treatment Plant Operations Manual (RJ Burnside, 2015).

A plan view of influent channel, screening, and grit removal is shown as Figure 2.1. The figure has been modified from available plant as-built drawings (R.J. Burnside, 2012).

Draft

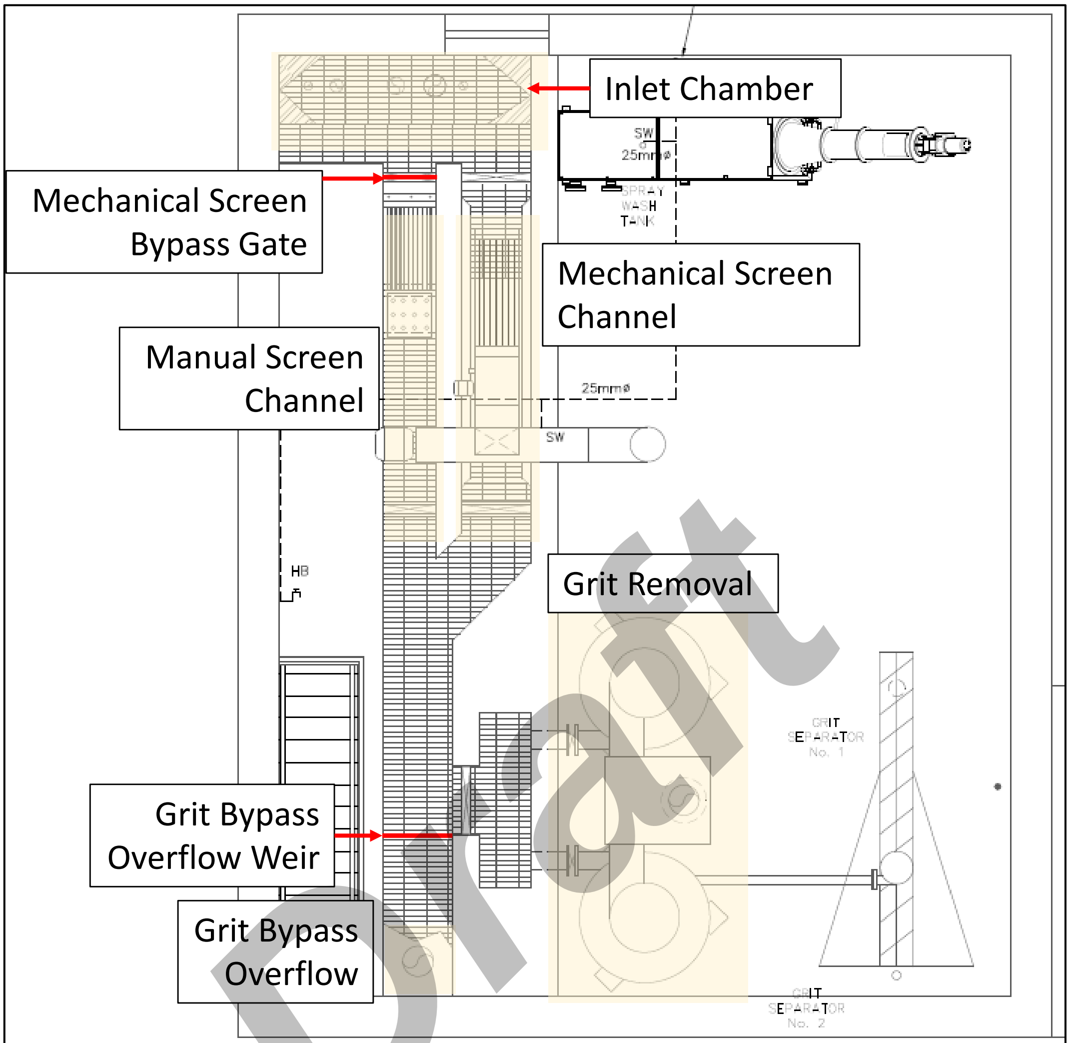


Figure 2.1 Plan View of Grand Valley WPCP Headworks

3. HYDRAULIC ANALYSIS OF GRAND VALLEY WPCP HEADWORKS

As previously discussed, peak flow through the grit removal process is limited by a fixed-height grit bypass overflow weir. For purposes of this analysis, it is assumed the weir-controlled peak flow through the grit removal process is equal to the design peak capacity of the grit removal process (7,680 m^3/d) and that excess flows will bypass the grit removal process. As such, the grit bypass weir controls the hydraulic level in the screen channel immediately upstream of the grit removal process.

It is important to note that a hydraulic analysis of the grit removal process was not completed as part of this work. As such, the exact relationship between the raw influent flow rate and grit removal performance is not known.

Overall, it is acknowledged that grit removal performance may decrease at future peak flows as a result of operation in excess of the rated capacity and/or bypass of the grit removal treatment processes. However, the existing grit removal processes have the rated capacity to treat approximately 89% of the projected peak hour flow for Scenario III. Therefore, the impact of grit removal performance on the estimated capacity of downstream treatment processes is expected to be negligible.

As such, this analysis focused on estimating the headloss in the mechanical screen channel upstream of the grit removal process. Headloss in the channel was estimated from three distinct sources:

- Headloss due to friction between the wastewater and channel walls;
- Headloss due to form changes (i.e. corners) in the channel; and
- Headloss across the mechanical screening process.

From plant as-built drawings, the channel width was noted to be 0.8 metres, and was assumed unchanged along the length of the channel.

Headloss due to friction was estimated using the process described by Nicklow & Boulos (2005). For this calculation, a reference hydraulic head level is required at a downstream location. The process then calculates the hydraulic level at upstream locations given the projected flow rate and characteristics of the channel (e.g. width, construction material, slope, etc.). The reference head level at the grit bypass weir was estimated from weir flow equations given the known height of the bypass weir and the estimated grit bypass flow at Scenario III peak flows.

Headloss due to form changes was estimated as described by Hager (1999). Headloss due to form changes depends the configuration of the form change, the estimated velocity in the channel, and a headloss coefficient which is estimated based on the geometry of the channel.

Headloss across the mechanical screen was estimated by the screen supplier (John Meunier). Headloss across the screen will depend on the volumetric flow rate and screen blockage. For purposes of this work, a conservative assumption of 70% screen blockage was used for calculations. A summary of the estimated headloss across the mechanical screen from the supplier is included as Appendix A.

A summary of estimated headloss in the mechanical screen channel from each source is given in Table 3.1.

Draft

Table 3.1 Summary of Estimated Headloss in the Screen Channel at a Peak Flow of 9,634 m^3/d

Parameter	Headloss (m)	Percentage of Total (%)
Friction Losses	0.005	2.6%
Form Losses	0.004	2.4%
Across the Mechanical Screen	0.175	95.0%
Total Headloss	0.184	-

Based on results presented in Table 3.1, the majority of headloss in the screen channel occurs across the mechanical screen. At the conservative estimation of screen blockage (70%), the headloss is approximately 175 millimetres (0.175 metres), or approximately 95% of the total estimated headloss in the mechanical screen channel.

Given the estimated downstream head level at the grit removal bypass weir (474.26 metres) and the estimated headloss in the mechanical screen channel (0.184 m), the estimated hydraulic level at the mechanical screen channel inlet at projected peak flows for Scenario III is approximately 474.45 metres. Therefore, the estimated head level at peak flows is less than both the current high level alarm in the influent chamber (474.49 metres) and the mechanical screen bypass (474.59 metres).

A visual representation of the estimated hydraulic level in the mechanical screen channel is given as Figure 3.1. The hydraulic levels immediately upstream and downstream of the mechanical screen have been modified from the hydraulic profile given as part of the plant as-built drawings. Modified hydraulic levels are shown in red text.

Therefore, based on preceding discussion and results presented in Figure 3.1, the estimated hydraulic level in the mechanical screen channel at projected peak flows for Scenario III is below both the high-level float in the inlet chamber and mechanical screen bypass levels. As such, the headworks appear to have sufficient hydraulic capacity to treat flows the projected Scenario III peak flows.

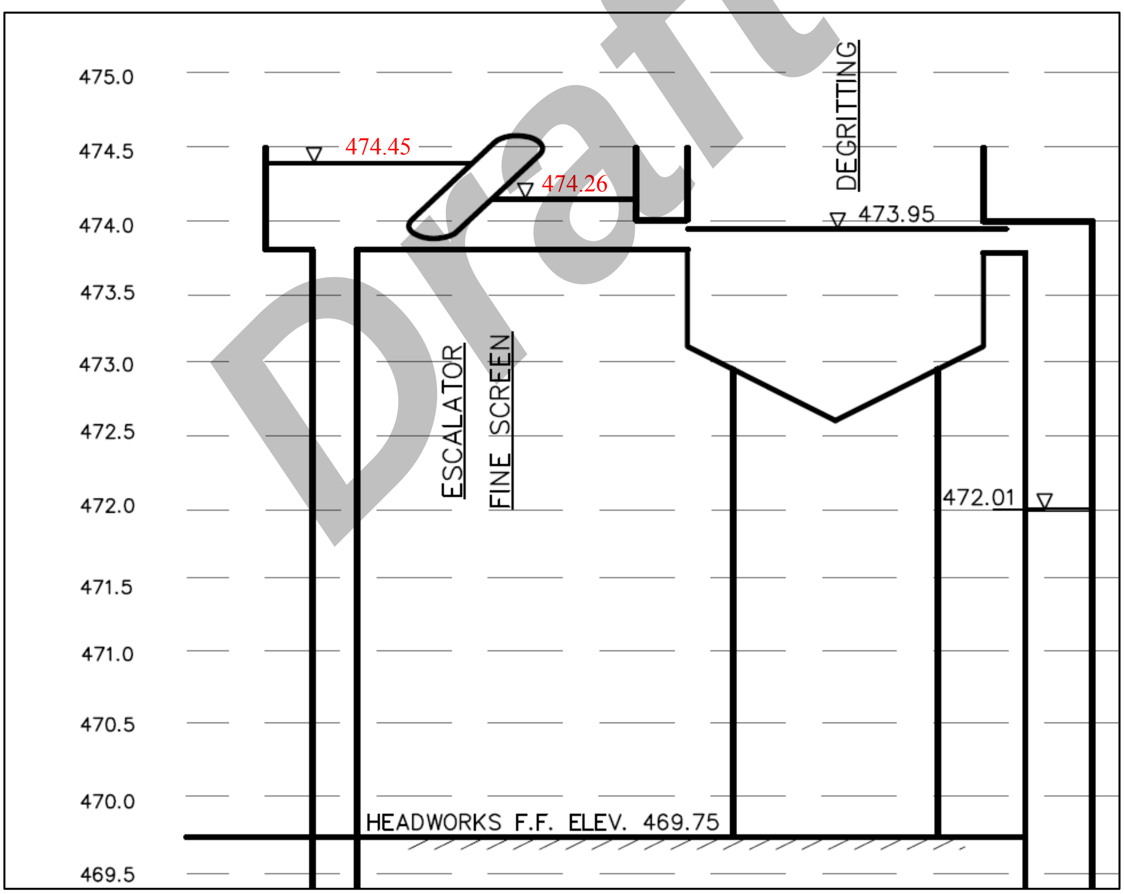


Figure 3.1 Projected Hydraulic Level in the Grand Valley WPCP Headworks at Scenario III Peak Flows

Draft

4. REFERENCES

1. Nicklow, J.D. & Boulos, P.F. Comprehensive Water and Wastewater Treatment Plant Hydraulics Handbook for Engineers and Operators. 2005.
2. Hager, W.H. Wastewater Hydraulics Theory and Practice. 2010.
3. R.J. Burnside & Associates Limited. Grand Valley Wastewater Treatment Plant Operations Manual. 2015.
4. R.J. Burnside & Associates Limited. As-built Drawings for the Grand Valley Water Pollution Control Plant. 2012.
5. XCG Consulting Limited. Grand Valley WPCP Re-rating Feasibility Study Updated Design Basis. 2016.

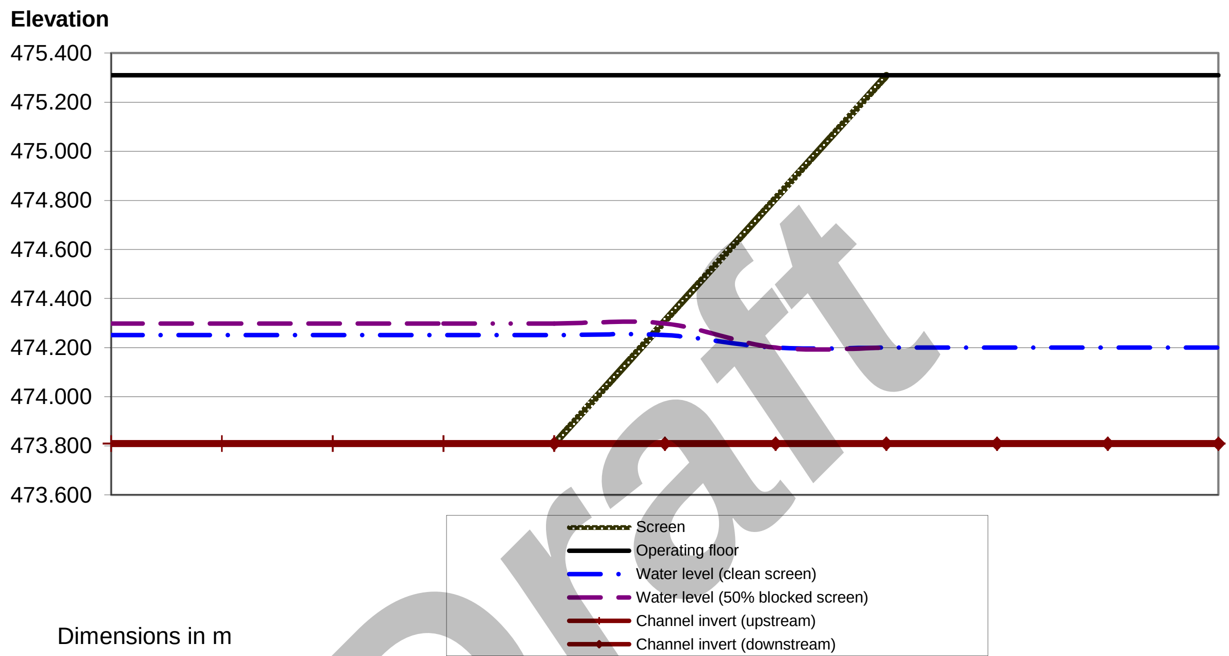
Draft

APPENDIX A
EXPECTED HEADLOSS ACROSS MECHANICAL SCREEN

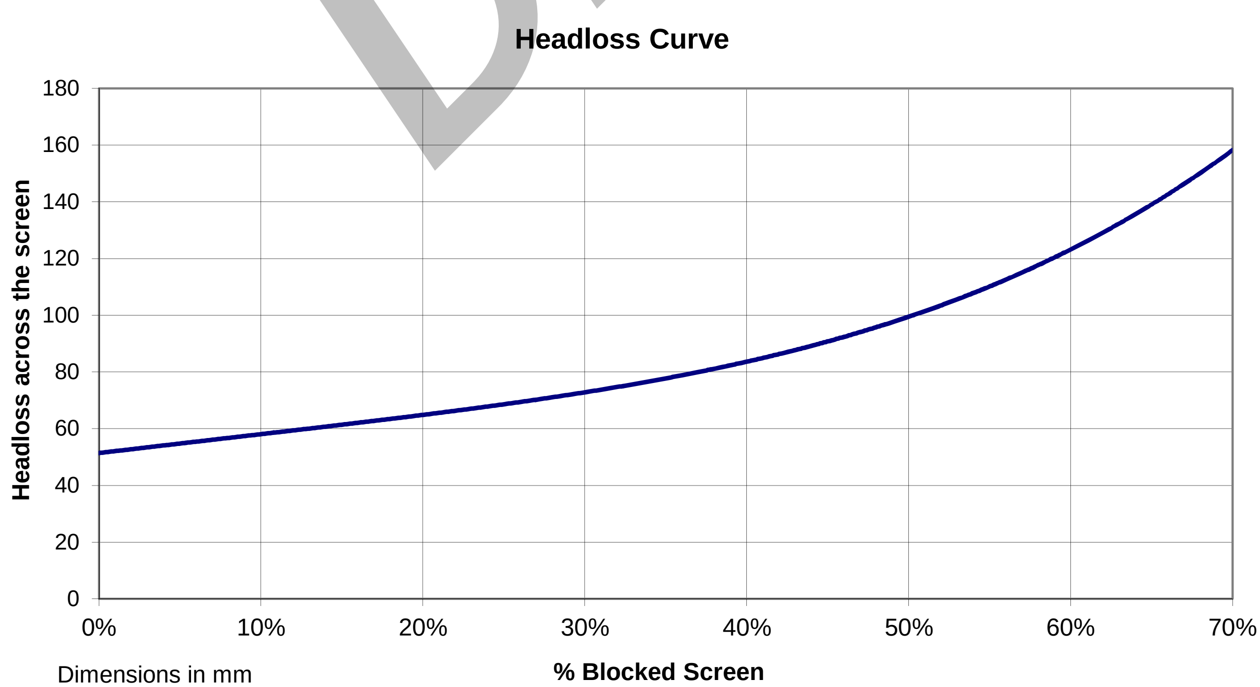
Draft

Project name	**Grand Valley, ON**
Project ref.	**AD04 Rev 0**
Model reference	**ESH6-24XA**

Peak flow per unit	**8695.00** m³/d	**Downstream hydraulic condition**	**Client profile**
Screen openings	**6.00** mm	**Downstream water depth**	**390.00** mm
Channel width	**800.00** mm	**Approach velocity (clean screen)**	**0.29** m/s
Side recess (total)	**0.00** mm	**Velocity through screen (clean screen)**	**0.62** m/s
Channel depth	**1500.00** mm	**Downstream velocity**	**0.32** m/s
Bottom recess	**150.00** mm	**Available freeboard upstream at 0%**	**1058.70** mm
Installation angle	**60** °	**Available freeboard upstream at 50%**	**1011.27** mm



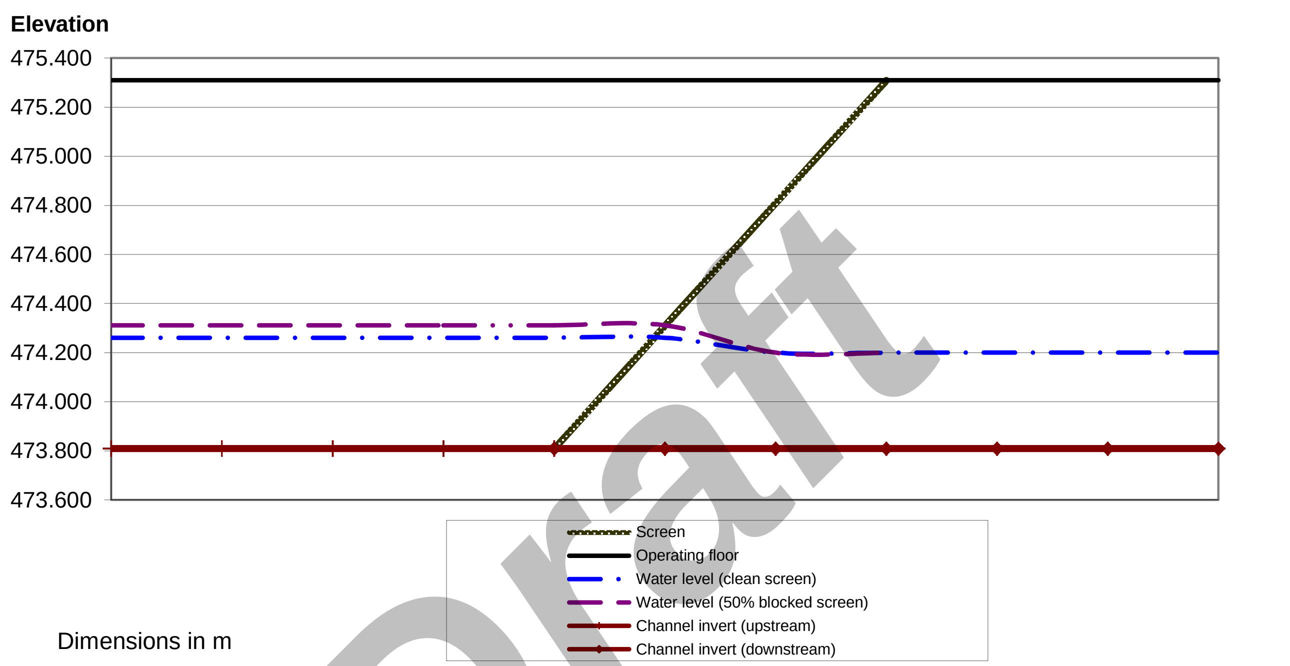
Dimensions in m

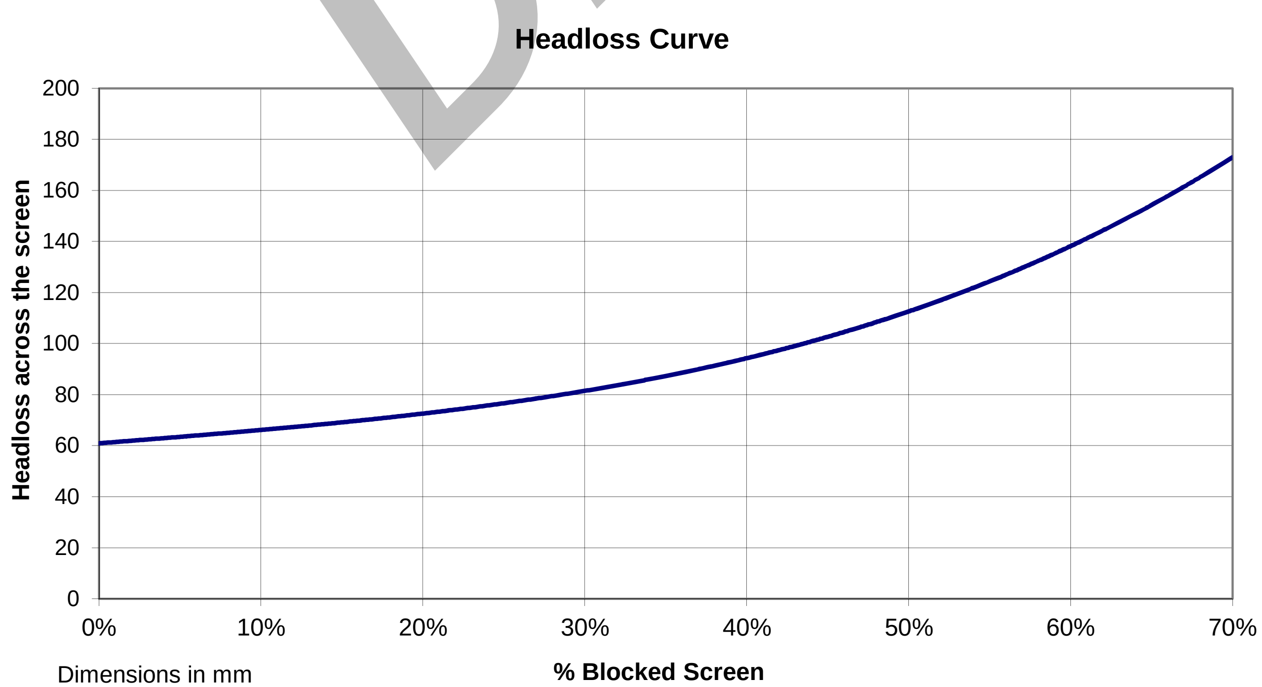


Dimensions in mm

Project name	**Grand Valley, ON**
Project ref.	**AD04 Rev 0**
Model reference	**ESH6-24XA**

Peak flow per unit	**9650.00**	m³/d	**Downstream hydraulic condition**	**Client profile**	
Screen openings	**6.00**	mm	**Downstream water depth**	**390.00**	mm
Channel width	**800.00**	mm	**Approach velocity (clean screen)**	**0.31**	m/s
Side recess (total)	**0.00**	mm	**Velocity through screen (clean screen)**	**0.68**	m/s
Channel depth	**1500.00**	mm	**Downstream velocity**	**0.36**	m/s
Bottom recess	**150.00**	mm	**Available freeboard upstream at 0%**	**1049.00**	mm
Installation angle	**60**	°	**Available freeboard upstream at 50%**	**998.26**	mm





Draft

Draft

APPENDIX D

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY
SUMMARY OF BIOWIN™ MODELLING

XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

XCG File No.: 3-252-57-01
December 6, 2016

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY
SUMMARY OF BIOWIN™ MODELLING

Draft

Prepared for:

TOWN OF GRAND VALLEY
5 Main Street, North
Grand Valley, Ontario
L9W 5S6

Attention: Jane Wilson

Prepared by:

XCG CONSULTING LIMITED
Suite 300, 2620 Bristol Circle
Oakville, Ontario
L6H 6Z7

TABLE OF CONTENTS

TABLES

FIGURES

APPENDIX

Draft

1. INTRODUCTION

1.1 Background

The Grand Valley Water Pollution Control Plant (WPCP) provides treatment for wastewater generated in the community of Grand Valley within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of Environment and Climate Change (MOECC) Certificate of Approval (C of A) No. 9706-7KWQ57, issued on February 2, 2009. The quality and quantity of effluent currently discharged by the existing WPCP is regulated by the C of A. The Grand Valley WPCP has a rated average day flow (ADF) capacity of 1,244 m^3/d.

The Town has initiated an investigation to analyze the potential to re-rate the existing Grand Valley WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. The Town has retained XCG Consulting Limited (XCG) to undertake a capacity assessment of the Grand Valley WPCP to evaluate the potential to re-rate the plant.

As part of this assessment, XCG evaluated the biological treatment capacity of the Grand Valley WPCP using historical plant data, results from an intensive sampling program conducted from October 20 – 29, 2015, and BioWin™ modelling software.

1.2 Objectives

The specific objectives of this technical memorandum are to:

- Present details of model construction and configuration;
- Present results of model calibration and validation; and
- Use future projected flows and loads to the Grand Valley WPCP to estimate the biological treatment capacity.

Draft

2. BIOWIN™ MODEL SETUP, CALIBRATION AND VALIDATION

2.1 Model Setup

A BioWin™ model of the Grand Valley WPCP was configured as shown in Figure 2.1. The model was calibrated using data obtained during the Intensive Sampling Program, conducted in October 2015. Detailed results of the Intensive Sampling Program are included in Appendix A. Specifically, the model calibration used raw wastewater quality results, final effluent quality results, and plant operating conditions recorded over the Intensive Sampling Program.

Ideal clarifiers and point clarifiers were used to model secondary clarifiers and tertiary filters, respectively, using a defined solids removal percentage estimated based on plant data. RAS and WAS were modelled as per historic plant operation, with RAS flows returned to the aeration tanks. Alum addition was added to the combined aeration tank effluent stream, ahead of the secondary clarifiers.

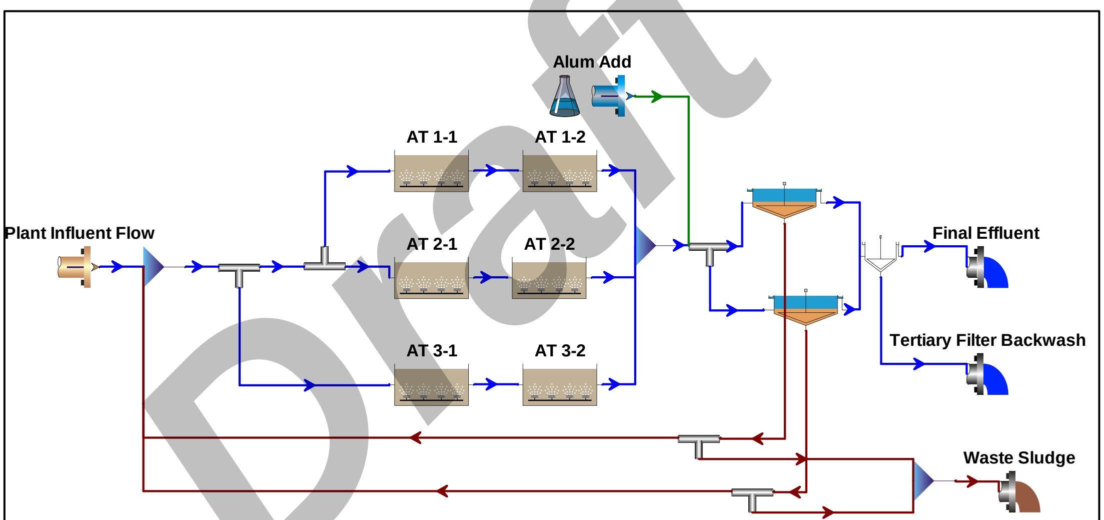


Figure 2.1 Schematic of the BioWin™ Calibration Model for the Grand Valley WPCP

Draft

2.2 Model Calibration

The model for the Grand Valley WPCP was calibrated under steady state conditions according to the procedure for model calibration detailed in Methods for Wastewater Characterization in Activated Sludge Modelling (WEF, 2003).

Influent wastewater characteristics were estimated based on results from the Intensive Sampling Program, conducted in October 2015, and using an influent specifier tool included in the BioWin™ software package. Raw influent samples were collected at the raw wastewater flow splitter box and thus contain contributions from the following three sources:

- Collection system via the Emma St. SPS;
- Septage from the onsite septage receiving station; and

- Plant recycle flow (i.e. digester supernatant and tertiary filter backwash) from the onsite pumping station.

As such, contributions from the onsite pumping station and septage receiving station were not modelled as separate inputs to the Grand Valley WPCP during calibration of the plant model.

It is important to note that, during the intensive sampling program, measured influent and effluent flow at the Grand Valley WPCP was significantly different. This difference may be, in part, related to malfunctioning solenoid valves in the plant headworks which contribute additional flow to the treatment plant. Additional details are included in Appendix A.

For purposes of model calibration and validation, modeled plant flow must represent the total estimated flow through the aeration bioreactors, secondary clarifiers and tertiary filters. Plant influent flow was estimated from the measured final effluent flow (which includes contributions from the Emma St. SPS, from the septage receiving station, and from the malfunctioning solenoid valves) and recycled flow from the onsite pumping station.

A summary of raw influent characteristics measured during the intensive sampling program and modelled raw influent characteristics is shown in Table 2.1.

Table 2.1 Raw Influent Characteristics

Parameter	Model Value	Intensive Sampling Results
Plant Influent Flow (m^3/d) (1)	781	781
Raw Wastewater Quality		
cBOD (mg/L)	100	100
COD (mg/L)	184	139
TSS (mg/L)	110	110
VSS (mg/L)	88	102
TKN (mg/L)	19.7	19.7
TP (mg/L)	2.18	2.18
Temperature (°C)	13.0	13.0

Notes:
1. Estimated from final effluent flow measurements (696 m^3/d) and the onsite pumping station (85 m^3/d).

It is important to note that the raw influent COD:BOD ratio observed during the intensive sampling program was significantly less than typically measured for residential raw wastewater. However, the observed BOD:TSS was acceptable, suggesting raw influent COD measurements were inconsistent with other measurements taken. Reasons for inconsistent COD measurements is unclear. For purposes of modelling, influent COD concentrations were adjusted as suggested by the BioWin™ influent specifier tool.

The raw wastewater fractions used in the model are presented in Table 2.2.

Draft

Table 2.2 Influent Specifier Raw Wastewater Fractions

Parameter	Modelled Plant Influent	BioWin™ Default
Fbs (g COD / g total COD)	0.304	0.160
Fac (g COD / g readily biodegradable COD)	0.151	0.150
Fxsp (g COD / g slowly biodegradable COD)	0.464	0.750
Fus (g COD / g total COD)	0.053	0.05
Fup (g COD / g total COD)	0.140	0.130
Fna (g NH_3-N / g TKN)	0.780	0.660
Fnox (g N / g Organic N)	0.500	0.500
Fnus (g N / g TKN)	0.020	0.020
FupN (g N / g COD)	0.035	0.035
Fpo4 (g PO_4-P / g TP)	0.541	0.500
FupP (g P / g COD)	0.011	0.011
Particulate Substrate COD:VSS ratio (mg COD / mg VSS)	0.75	1.60
Particulate Inert COD:VSS ratio (mg COD / mg VSS)	0.75	1.60

Notes:
Fbs - readily biodegradable COD fraction
Fac - acetate fraction of readily biodegradable COD
Fxsp - non-colloidal fraction of slowly biodegradable COD
Fus - unbiodegradable soluble COD fraction
Fup - unbiodegradable particulate COD fraction
Fna - ammonia fraction of TKN
Fnox - particulate organic nitrogen
Fnus - soluble unbiodegradable TKN
FupN - N:COD ratio for unbiodegradable particulate COD
Fpo4 - phosphate fraction of TP
FupP - P:COD ratio for unbiodegradable particulate COD
All other influent wastewater fractions, kinetic, and stoichiometric parameters were assumed to be the BioWin™ default values.

Draft

Not all treatment processes at the Grand Valley WPCP were online during the intensive sampling program. Specifically, due to low raw influent flows, the plant operated with two aerated bioreactors and one secondary clarifier online. For purposes of model calibration, there was no flow directed to Aeration Tank 3 or Secondary Clarifier 2 as shown in Figure 2.1. Flow was assumed evenly split between Aeration Tank 1 and Aeration Tank 2. Alum dosages were estimated based on operational records, and based on effluent TP concentrations.

The results of the steady state model calibration, as compared to measured plant performance during the October 2015 intensive sampling program, are presented in Table 2.3. The primary goal of the BioWin™ model is to assess the biological performance at future flows and loads. Therefore, particular attention was paid to biological process indicators, specifically effluent total ammonia nitrogen (TAN) and biochemical oxygen demand ($cBOD_5$) concentrations, during the calibration stage.

Table 2.3 BioWin™ Model Calibration Results

Parameter	Model Value	Intensive Sampling Results
Bioreactor MLSS (mg/L)		
Aeration Tank 1	6,373	6,550
Aeration Tank 2	6,373	6,480
Bioreactor MLVSS (mg/L)		
Aeration Tank 1	4,116	4,556
Aeration Tank 2	4,116	4,350
MLVSS:MLSS		
Aeration Tank 1	0.65	0.70
Aeration Tank 2	0.65	0.67
RAS Flow (m^3/d)	340	343
WAS Flow (m^3/d)	2.94	2.93
Final Effluent Quality		
COD (mg/L)	11.87	10.0
cBOD (mg/L)	0.74	< 4.0 (1)
TSS (mg/L)	1.53	< 4.0 (1)
TAN (mg/L)	0.11	< 0.10 (1)
TP (mg/L)	0.09	0.08
pH	7.07	7.5

Notes:
1. All samples from the intensive sampling program measured below the detection limit.

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

- During the intensive sampling program, final effluent concentrations of $cBOD_5$ and TAN consistently measured below the laboratory reported Method Detection Limit (MDL) concentrations (4.0 mg/L and 0.1 mg/L, respectively).
- With respect to $cBOD_5$ and TAN, the calibrated model predicted effluent concentrations consistent with those found during the intensive sampling program. With respect to effluent TAN concentrations, the calibrated model conservatively predicts slightly greater effluent concentrations than observed in plant records.
- The modelled mixed liquor volatile suspended solids (MLVSS) and mixed liquor suspended solids (MLSS) concentrations were slightly less than those recorded at the plant. The modelled MLVSS:MLSS ratio (0.65) is slightly less than the ratio measured in Aeration Tank 1 (0.70) and in Aeration Tank 2 (0.67).

Overall, calibration results indicate the BioWin™ model is capable of providing a reasonable estimate of the biological treatment capacity of the Grand Valley WPCP.

2.3 Model Validation

The BioWin™ model for the Grand Valley WPCP was validated based on effluent characteristics (particularly effluent TAN) by conducting simulations using historical plant influent flow and raw influent quality characteristics. Similar to above, plant influent flow was modelled as the sum of measured flow at the onsite pumping station and from the final effluent v-notch weir. Specifically, the following three periods, which cover a range of operating temperatures, were used for model validation:

Draft

- January to March, 2012
- April to June, 2013
- July to September, 2014

Key results from model validation are summarized in Table 2.4. With respect to plant effluent $cBOD_5$ concentrations, 100% of plant measurements were recorded to be at or below the MDL (2.0 mg/L). Similarly, 92.5% of all effluent TAN measurements were at or below the MDL (0.1 mg/L). For purposes of Table 2.4, measurements at or below the detection limit were assumed to be equal to the detection limit.

Table 2.4 BioWin™ Model Validation Results

Parameter	January to March, 2012		April to June, 2013		July to Sept. 2014	
	Model Value	Plant Measured	Model Value	Plant Measured	Model Value	Plant Measured
MLSS (1)	2,847	2,737	4,296	4,620	7,925	7,869
RAS	818	816	1,347	1,339	323	318
WAS	15.6	13.8	11.7	14.7	1.8	1.9
Effluent Characteristics (2)						
$cBOD_5$	1.02	2.0	1.10	2.0	0.83	2.0
TAN	0.18	0.11	0.10	0.10	0.06	0.12

Notes:
1. Reported MLSS concentrations are averaged between aeration tanks.
2. 100% of plant effluent $cBOD_5$ measurements and 92.5% of plant effluent TAN measurements were measured at or below the minimum detection limit. Average concentrations reported in the table have assumed concentrations equal to the minimum detection limit, where required.

In general, the BioWin™ model predicted effluent concentrations of $cBOD_5$ and TAN were comparable to final effluent samples collected at the plant. Therefore, it appears the BioWin™ model is an accurate representation of the Grand Valley WPCP and can be used to evaluate the biological treatment capacity of the plant.

Draft

3. BIOWIN™ MODELLING TO PREDICT PLANT CAPACITY

The biological treatment capacity of the Grand Valley WPCP was estimated by applying the validated BioWin™ model at projections of future flows and loads. The following assumptions were made regarding future operation of the treatment plant:

- At the biological treatment capacity, all secondary treatment processes (i.e. three aeration tanks and two secondary clarifiers) will be online, and flow will be equally split between all treatment processes;
- Typical DO concentrations of 2.0 mg/L will be maintained in all aeration tanks;
- RAS flow is approximately 100% of the raw influent flow; and,
- Future recycle stream flow is approximately 11% of the projected raw influent flow, as estimated from historical plant records.

3.1 Determining Design SRT

The approach used to determine the capacity of the Grand Valley WPCP was to first determine the minimum SRT required to achieve effluent C of A limits at projected flows and loads. Previous investigation has established a design basis for the Grand Valley WPCP at three future design scenarios. It was assumed that total effluent loading would not increase at future flows. As such, effluent objective and limit concentrations must decrease proportionally with the increase in treated flow. Design Scenario III has the greatest average day flow (1,555 m^3/d) and therefore also has the most stringent effluent quality requirements. A summary of the current C of A objectives (at an ADF of 1,244 m^3/d) and the predicted effluent requirements under Scenario III is given in Table 3.1.

Draft

Table 3.1 C of A Objective and Non-compliance Limit Concentrations

Parameter	Current C of A Effluent Requirements (ADF = 1,244 m^3/d)		Projected Effluent Requirements (ADF = 1,555 m^3/d)	
	Objective (1)	Limit (1)	Objective (1)	Limit (1)
$cBOD_5$	8.0 mg/L	10.0 mg/L	6.4 mg/L	8.0 mg/L
TSS	8.0 mg/L	10.0 mg/L	6.4 mg/L	8.0 mg/L
TP	0.13 mg/L	0.15 mg/L	0.10 mg/L	0.12 kg/d
TAN (2)				
Winter	3.0 mg/L	4.0 mg/L	2.4 mg/L	3.2 mg/L
Spring	0.8 mg/L	1.0 mg/L	0.64 mg/L	0.80 mg/L
Summer	0.6 mg/L	0.7 mg/L	0.48 mg/L	0.56 mg/L
Fall	0.8 mg/L	1.0 mg/L	0.64 mg/L	0.80 mg/L
E. coli (3)	100 CUFs/100 mL	-	100 CUFs/100 mL	-

Notes:
1. Expressed as an average monthly concentration.
2. TAN concentrations are regulated for each season: Winter (December 1 to March 31), Spring (April 1 to May 31), Summer (June 1 to September 30), and Fall (October 1 to November 30).
3. Monthly geometric mean density.

As previously noted, the purpose of developing this plant model is to estimate the biological treatment capacity of the Grand Valley WPCP through evaluation of effluent concentrations of $cBOD_5$ and TAN. However, it is important to note that objective and limit effluent concentrations of TP may decrease to 0.10 mg/L and 0.12 mg/L, respectively, at an ADF of 1,555 m^3/d. Tertiary effluent filtration can be designed to reduce effluent TP concentrations to a minimum of 0.10 mg/L (MOE, 2008). However, the existing tertiary filters have been designed for an effluent performance quality of 0.15 mg/L (R.J. Burnside, 2015). As such, Scenario III likely approaches the limit of phosphorus treatment capacity given the existing treatment processes at the Grand Valley WPCP. This TM addresses only the biological treatment capacity of the Grand Valley WPCP (i.e. its ability to meet effluent $cBOD_5$ and TAN requirements).

At the concentrations presented in Table 3.1, it is anticipated that the minimum required SRT will be limited by meeting effluent TAN requirements rather than $cBOD_5$ requirements. As noted in Table 3.1, effluent objectives for TAN vary by season. Modelling at varying mixed liquor concentrations was carried out in order to determine the minimum SRT to achieve effluent TAN limit concentrations under:

- Summer conditions (minimum temperature = 14°C);
- Winter conditions (minimum temperature = 9°C); and,
- Spring/Fall conditions (minimum temperature = 12°C).

Figure 3.1, Figure 3.2, and Figure 3.3 present the relationship between effluent TAN and SRT for winter, summer, and spring/fall conditions, respectively.

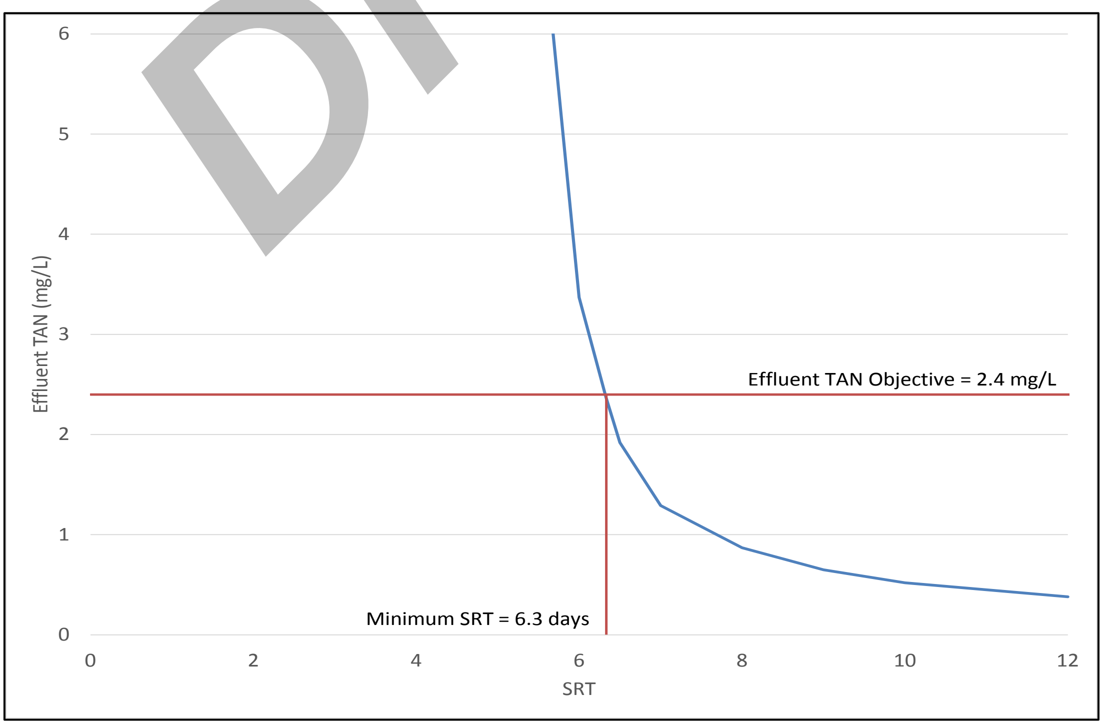


Figure 3.1 Effluent TAN Concentration v. SRT – Winter Conditions (9°C)

Draft

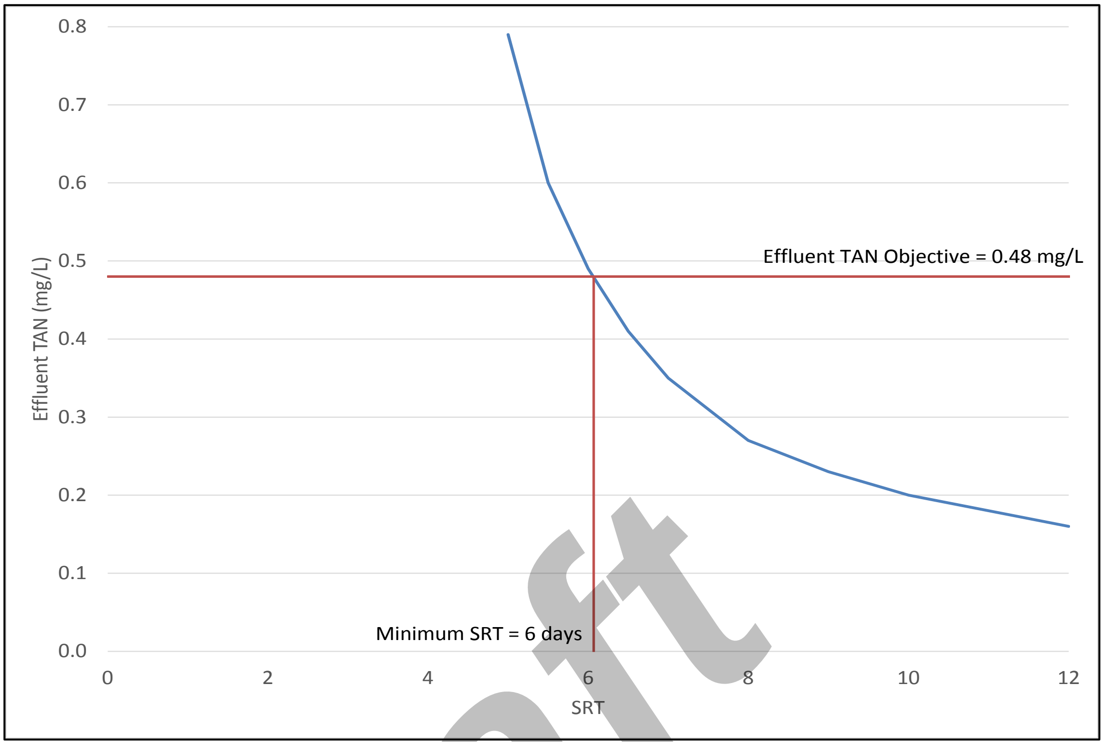


Figure 3.2 Effluent TAN Concentration v. SRT – Summer Conditions (14°C)

Draft

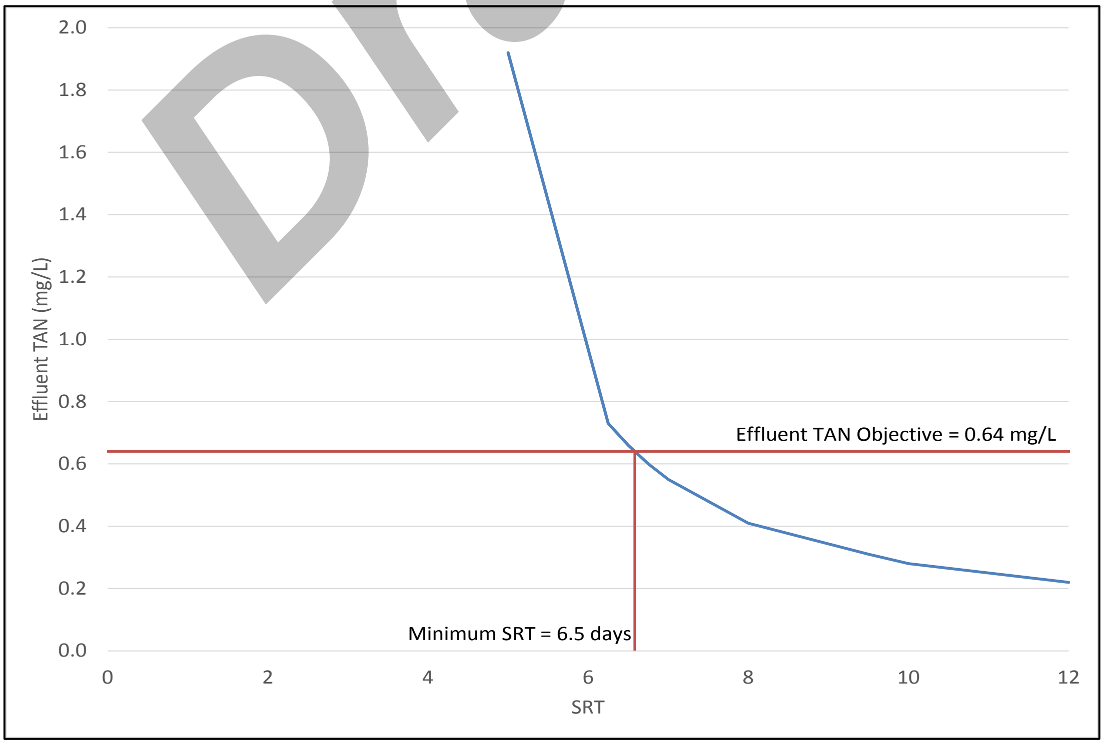


Figure 3.3 Effluent TAN Concentration v. SRT – Spring/Fall Conditions (12°C)

Based on results presented in the figures above, the most stringent minimum required SRT is 6.5 days based on spring/fall conditions.

For purposes of defining the minimum required design SRT, a safety factor of 2.3 was applied to the minimum required spring/fall SRT of 6.5 days to ensure effluent TAN requirements can be met even with fluctuations in influent flows and loadings, as well as operating conditions in the liquid treatment train. Therefore, a design SRT of 15 days was carried forward for subsequent analyses.

3.2 Biological Treatment Capacity Assessment

The objective of this section is to estimate the biological treatment capacity of the Grand Valley WPCP given the estimated design SRT of 15 days. To facilitate the capacity evaluation, the following assumptions were made:

- Design yield of 0.96 kg TSS/kg BOD_5;
- Target operating MLSS concentration of 3,000 mg/L;
- A bioreactor operating volume of 1,200 m^3; and,
- A future influent BOD_5 concentration of 158 mg/L, as per projected Scenario III design basis.

The design operating volume assumes all three bioreactors (400 m^3 each) will be online at future flows. The design yield was selected based on results of BioWin™ modelling of the Grand Valley WPCP. The recommended operating mixed liquor concentration for an extended aeration treatment process is approximately 3,000 mg/L to 5,000 mg/L (Metcalf & Eddy, 2003). In order to maximize the equivalent ADF capacity of the secondary clarifiers, a target operating MLSS concentration of 3,000 mg/L was assumed for purposes of this investigation. This is consistent with previous investigations which evaluated the equivalent ADF treatment capacity of the secondary clarifiers at the Grand Valley WPCP (XCG, 2016). MLSS concentrations greater than 3,000 mg/L will increase the biological treatment capacity, but may limit the equivalent treatment capacity of the secondary clarifiers.

Based on the assumptions above, the estimated biological treatment capacity of the Grand Valley WPCP is approximately 1,582 m^3/d, which is comparable to the projected Scenario III ADF (1,555 m^3/d).

Using the validated model of the Grand Valley WPCP, two simulations were conducted to evaluate the performance of the treatment plant at the projected Scenario III ADF under average day and maximum month loading conditions. Maximum month factors (MMFs) from historical plant operating data were found to range from 1.9 to 2.2. This is greater than typical MMFs, which range from 1.4 to 1.6. Large MMFs observed at the Grand Valley WPCP may be due to the type of raw influent sample collected at the plant (one grab sample collected per month). To be conservative, historical MMFs from plant operating data were assumed.

As noted in Section 3.1, performance of the Grand Valley WPCP was limited by operation under spring/fall conditions. Table 3.2 presents a summary of the projected

Draft

plant performance at average day and maximum month loadings under spring/fall operating conditions.

Table 3.2 Summary of Plant Performance at ADF = 1,555 m³/d Under Spring/Fall Conditions

Parameter	Average Day	Maximum Month	Typical Design Guideline
Liquid Treatment Train Influent			
Flow (m^3/d)	1,555	-	-
BOD_5 (kg/d)	245	466	-
TSS (kg/d)	322	613	-
TKN (kg/d)	59.1	112	-
TP (kg/d)	7.48	16.5	-
Aeration Tank			
MLVSS (mg/L)	1,833	1,948	-
MLSS (mg/L)	2,962	3,035	3,000 - 5,000 (1)
Organic Loading Rate (kg $BOD_5/m^3 \cdot d$)	0.20	0.39	0.17 - 0.24 (1)
F/Mv (kg BOD_5/kg MLVSS·d)	0.11	0.20	0.05 - 0.15 (1)
SRT (days)	15	7.3	>15 (1)
Secondary Clarifier			
RAS Flow (m^3/d)	1,517	1,475	-
RAS Flow %	98	95	50 - 200% of ADF (1)
RAS SS (mg/L) (1)	6,217	6,294	-
WAS Solids (kg/d)	238	507	-
Final Effluent			**Projected Effluent Objectives**
$cBOD_5$ (mg/L)	0.89	0.99	6.4
TAN (mg/L)	0.14	0.29	0.64 (Spring/Fall)
Temperature (°C)	12	12	-

Notes:
1. 2008 MOE Design Guidelines for Sewage Works for an extended aeration process.

Draft

Based on the model results presented in Table 3.2, the Grand Valley WPCP has the capacity to handle projected Scenario III average day and maximum month wastewater loads at the target MLSS concentration of 3,000 mg/L while meeting the projected ECA objectives for $cBOD_5$ and TAN.

4. SUMMARY AND CONCLUSIONS

Results of the BioWin™ modelling indicate the Grand Valley WPCP is capable of meeting all projected effluent ECA limits at the projected Scenario III ADF flow (1,555 m^3/d), BOD_5 load (245 kg/d), and TKN load (59.1 kg/d) while operating at an MLSS concentration of approximately 3,000 mg/L.

In addition, the following key points should also be highlighted:

- Results presented in this report depend on the accuracy of future projections of BOD_5 and TKN to the plant.
- The capacity of downstream treatment processes (i.e. secondary clarifiers, tertiary filters, UV disinfection) will be impacted by operation of the biological treatment train. Specifically, the biological treatment capacity will increase with increasing MLSS concentrations. However, the secondary clarifier treatment capacity, based on the SLR, will decrease with increasing MLSS concentrations. The specific relationship between the operating MLSS concentration and secondary clarifier treatment capacity was not explored as part of this evaluation.
- Future effluent requirements were estimated by assuming that current final effluent loads would not change at future flows. By this method, it was observed future effluent TP requirements at the Scenario III ADF may be approaching the phosphorus removal limit of the existing tertiary filtration technology installed at the plant.

Draft

APPENDIX A
INTENSIVE SAMPLING PROGRAM RESULTS

Draft

XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

XCG File No.: 3-252-57-01
December 6, 2016

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY INTENSIVE SAMPLING PROGRAM RESULTS

Draft

Prepared for:

TOWN OF GRAND VALLEY
5 Main Street, North
Grand Valley, Ontario
L9W 5S6

Attention: Jane Wilson

Prepared by:

XCG CONSULTING LIMITED
Suite 300, 2620 Bristol Circle
Oakville, Ontario
L6H 6Z7

TABLE OF CONTENTS

TABLES

FIGURE

APPENDICES

Draft

1. INTRODUCTION

The Grand Valley Water Pollution Control Plant (WPCP) provides treatment for wastewater generated in the community of Grand Valley within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of Environment and Climate Change (MOECC) Certificate of Approval (C of A) No. 9706-7KWQ57, issued on February 2, 2009. The quality and quantity of effluent currently discharged by the existing WPCP is regulated by the C of A. The Grand Valley WPCP has a rated average day flow (ADF) capacity of 1,244 m^3/d.

The Town has initiated an investigation to analyze the potential to re-rate the existing Grand Valley WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. The Town has retained XCG Consulting Limited (XCG) to undertake a capacity assessment of the Grand Valley WPCP to evaluate the potential to re-rate the plant.

To assist with the evaluation of the biological treatment capacity, an intensive sampling program was conducted to better characterize wastewater in the plant, and to assess the performance of individual unit processes. The purpose of this technical memorandum is to present results of the intensive sampling program.

Draft

2. SAMPLING PROGRAM OVERVIEW

The intensive sampling program was completed over seven business days from October 20 – 29, 2015. The objective of the intensive sampling program was to evaluate the performance of individual unit processes and to characterize the wastewater throughout the plant. Results of the intensive sampling program were also used for purposes of biological modelling and to review the biological treatment capacity of the Grand Valley WPCP.

In total, seven process streams were sampled during the intensive sampling program. Plant operators did not supernate the aerobic digester during the intensive sampling program. In addition, there was no septage received at the septage receiving station. As such, samples from both these process streams could not be collected.

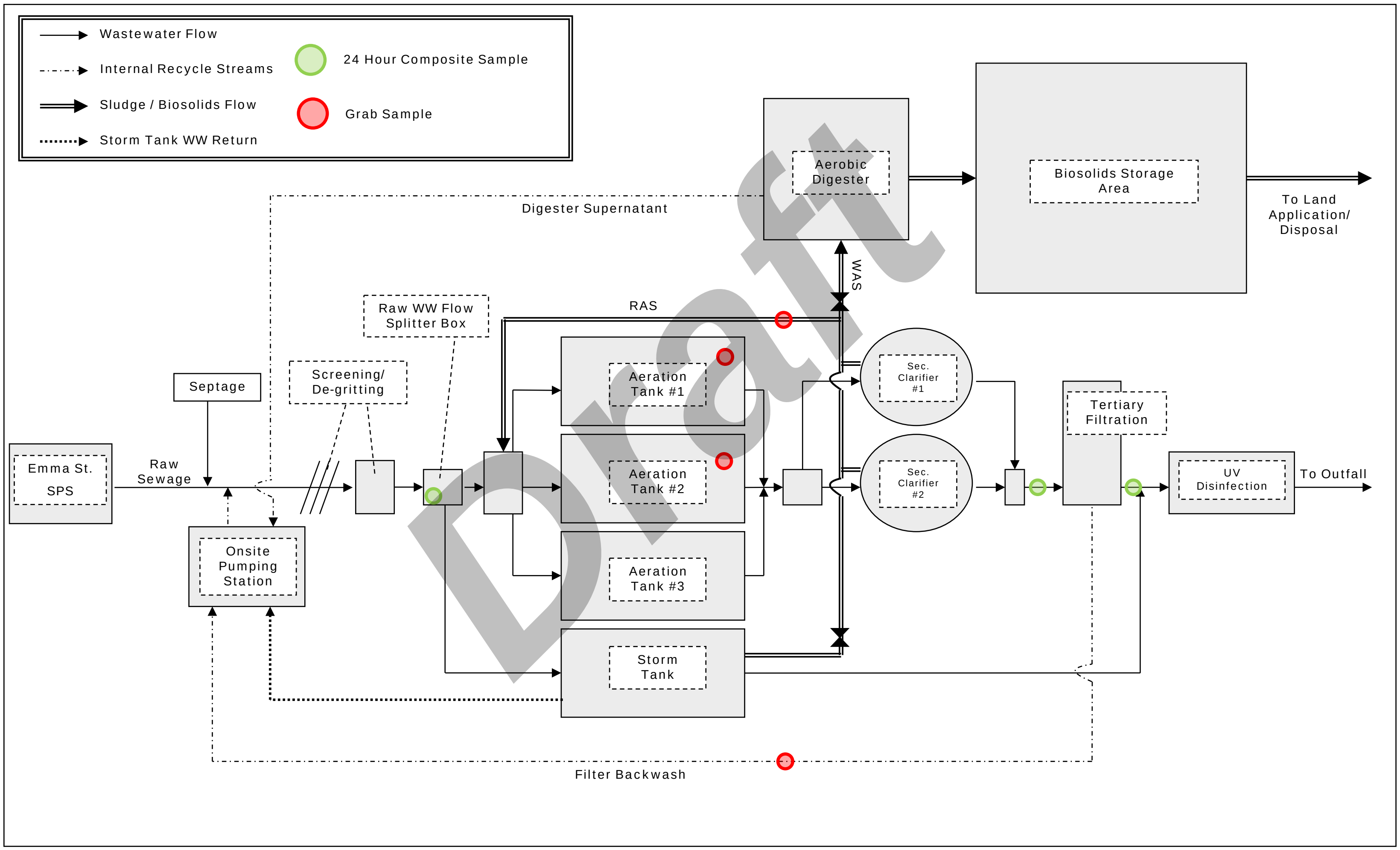
Figure 2.1 presents an overview of the sampling locations at the plant and identifies the type of sample collected at each location (i.e. 24-hour composite or grab). Analyzed parameters varied between samples, but included the following:

- Total COD (COD), filtered COD (COD-f), and flocculated and filtered COD (COD-ff)
- Total BOD5 (BOD5), carbonaceous BOD5 (cBOD5), and filtered cBOD5 (cBOD5-f)
- Total phosphorus (TP) and dissolved reactive phosphorus
- Total Kjeldahl nitrogen (TKN), total ammonia nitrogen (TAN), Nitrate + Nitrite Nitrogen
- Alkalinity (CaCO3 equivalent)
- Total suspended solids (TSS) and volatile suspended solids (VSS)
- pH

A copy of the intensive sampling program protocol, which includes details regarding sampling locations, frequencies, handling and required analyses, is included as Appendix A.

Draft

Figure 2.1 Summary of Sampling Locations at the Grand Valley WPCP



3. RESULTS

The purpose of this section is to present results from the Grand Valley WPCP Intensive Sampling Program. A full copy of all results from the accredited laboratory is included in Appendix B.

3.1 Plant Flows

For the duration of the intensive sampling program, daily measured flows were monitored at the following locations within the Grand Valley WPCP:

- Raw wastewater from the collection system as measured at the Emma St. SPS;
- Plant recycle flow as measured at the onsite pumping station;
- Measured flow from the onsite septage pumping station;
- RAS;
- WAS; and
- Final effluent flow as measured at the Grand Valley WPCP downstream of the UV disinfection system.

Table 3.1 summarizes the measured flows over the intensive sampling program.

Table 3.1 Summary of Monitored Plant Flows during the Intensive Sampling Program (m^3/d)

Date	Emma St. SPS	Onsite PS	Septage	RAS	WAS	Final Effluent
October 20	338	73	4.6	384	2.8	703
October 21	332	101	4.6	341	2.8	707
October 22	341	97	4.8	310	2.8	679
October 26	326	87	4.9	385	3.3	651
October 27	342	88	5.1	325	2.8	664
October 28	439	84	4.8	315	3.0	763
October 29	379	68	5.2	337	3.0	708
Average	**357**	**85**	**4.9**	**343**	**2.9**	**696**

Results indicate that measured flows from each monitored source were relatively stable over the entire monitoring period.

Flow continuity within the Grand Valley WPCP can be evaluated by analyzing the total influent flow (Emma St. SPS + Septage) relative to the Final Effluent flow. Considering average data collected over the entire sampling program, the total influent flow (362 m^3/d) is significantly less than the final effluent flow (696 m^3/d).

Exact rationale for the noted discrepancy is not known. However, the difference may be, in part, related to malfunctioning solenoid valves in the plant headworks and the accuracy of flow meters at the plant. In 2015, plant operators noted malfunctioning solenoid valves resulted in a larger volume of potable flushing water being added to the WPCP downstream of the influent flow meters. Malfunctioning solenoid valves

Draft

were replaced at the plant in January 2016. The final effluent flow meter was also recalibrated in January 2016, approximately two weeks after the solenoid valves were replaced. Details of the calibration process and its impact on measured effluent flow from the Grand Valley WPCP are not clear.

3.2 Plant Influent Raw Wastewater

Over the duration of the intensive sampling program, seven (7) 24-hour composite samples were collected at the raw wastewater flow splitter box, located immediately upstream of the aeration tanks and downstream of the plant headworks. As such, collected samples include contributions from the Emma St. SPS, the septage receiving station, and the onsite pumping station.

A summary of raw wastewater characterization during the intensive sampling program is given as Table 3.2. The characterization of the raw wastewater stream included several parameters which are not historically monitored to allow development of modelling parameters for BioWin^{TM}. This included approximation of the readily biodegradable chemical oxygen demand fraction (rbCOD) using a filtration-flocculation method (COD-ff). Further, the fraction of soluble carbonaceous biochemical oxygen demand ($cBOD_5$) was approximated by filtering the sample ($cBOD_5$-f).

In general, the chemical oxygen demand (COD) is a measure of the organic material in the wastewater sample which can be chemically oxidized. Biochemical oxygen demand (BOD) is a similar measurement that estimates the oxygen used by microorganisms in the oxidation of organic material. The total BOD is the sum of the carbonaceous BOD (cBOD) and nitrogenous BOD (nBOD). The cBOD measures oxygen consumption from the degradation of carbon sources, while nBOD considers the consumption of oxygen by nitrifying bacteria to oxidize ammonia into nitrate.

BOD tests are typically carried out over five days (BOD_5). $cBOD_5$ tests are commonly chemically inhibited to prevent oxygen consumption by nitrifying bacteria over the duration of the test. As such, the $cBOD_5$ is a measurement of a fraction of the total BOD_5. However, results from the sampling program show that measured concentrations of $cBOD_5$ were, on occasion, greater than the measured BOD_5 concentration. Previous discussion with staff from an accredited laboratory has indicated that such results may be a result of uncertainty within the BOD test (e.g. slight variations in the test water, the use of nitrification suppressant chemicals, etc.). For purposes of this work, influent concentrations of $cBOD_5$ which exceeded BOD_5 measurements were assumed equal to BOD_5 measurements.

As well, the measured COD concentration is expected to be greater than the BOD_5 concentration of a given wastewater sample because:

- Some complex organics present within the sample are difficult to biologically oxidize;
- Some substances within the sample can be chemically but not biologically oxidized; and
- The BOD_5 test is limited to five days.

Draft

Results from the intensive sampling program indicate one instance where the measured COD concentration was less than the BOD_5 concentration. This sample was assumed to be an outlier and removed from consideration.

Draft

Table 3.2 Summary of Raw Wastewater Characterization Results

	BOD_5	$cBOD_5$	$cBOD_5$-f	COD	COD-f	COD-ff	TP	Ortho-P	TKN	TAN	Alkalinity	TSS	VSS	pH
	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	
October 20	81	81 (1)	28	150	62	58	1.95	0.97	16.2	15.1	300	83	78	7.72
October 21	86	85	17	150	50	40	2.17	0.99	19.8	15.2	288	85	80	-
October 22	145	99	26	- (2)	62	51	2.17	1.51	21.4	15	286	134	122	7.88
October 26	154	154 (1)	36	160	60	60	2.27	0.97	20.8	15.6	289	109	98	-
October 27	134	97	25	136	70	51	2.29	1.30	19	16	287	115	105	-
October 28	125	117	25	146	63	48	2.4	1.55	20.3	16.4	297	150	142	-
October 29	84	66	21	94	51	40	1.98	0.96	19.8	13.8	275	94	89	-
Average	**116**	**100**	**25**	**139**	**60**	**50**	**2.18**	**1.18**	**19.7**	**15.3**	**289**	**110**	**102**	**7.80**

Notes:
1. Influent $cBOD_5$ concentration assumed equal to influent BOD_5 concentration.
2. Sample result assumed an outlier and removed.

Draft

3.3 Tertiary Filter Backwash

The Grand Valley WPCP uses continuous backwash tertiary filters to treat secondary effluent flow prior to disinfection and discharge. Backwash is directed to the onsite pumping station, and returned to the head of the plant. Over the duration of the intensive sampling program, seven (7) grab samples were collected of the tertiary filter backwash stream and were analyzed for BOD_5, COD, TP, orthophosphate, TSS, and VSS. Results are summarized in Table 3.3.

Results indicate the quality of backwash flow was relatively stable over the sampling period.

Table 3.3 Summary of Tertiary Filter Backwash Quality

	BOD_5	COD	TP	Ortho-P	TSS	VSS
	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L
October 20	10	9	0.74	0.07	32	22
October 21	13	35	0.65	0.07	38	27
October 22	12	12	0.85	0.11	45	32
October 26	19	20	0.8	0.07	46	35
October 27	12	14	0.7	0.12	37	26
October 28	13	40	0.71	0.08	40	28
October 29	10	13	0.83	0.08	38	28
Average	**13**	**20**	**0.75**	**0.09**	**39**	**28**

Draft

3.4 Mixed Liquor Characteristics

The mixed liquor suspended solids (MLSS) and mixed liquor volatile suspended solids (MLVSS) concentrations within each aeration tank was measured daily during the intensive sampling program. In addition, the RAS stream was sampled daily. It is important to note that RAS and WAS is pumped from the same location in the secondary clarifiers at the Grand Valley WPCP. As such, this sample is expected to be representative of both the RAS and WAS streams.

Samples were analyzed for TSS and VSS. As well, the dissolved oxygen (DO) from each aeration tank was measured daily. A summary of sample results is given in Table 3.4.

Measured MLSS concentrations in each aeration tank were relatively stable over the sampling period with two notable exceptions:

- Sample collected from Aeration Tank 1 on October 29, 2015 (MLSS concentration of 10,200 mg/L); and,

- Sample collected from Aeration Tank 2 on October 28, 2015 (MLSS concentration of 4,350 mg/L, MLVSS concentration of 3,080 mg/L).

Both samples were assumed to be outliers and removed from consideration. Between aeration tanks, MLSS and MLVSS concentrations were comparable. In general, MLSS concentrations ranged between 6,080 mg/L and 7,260 mg/L. This exceeds the typical MLSS concentration of an extended aeration process (3,000 mg/L to 5,000 mg/L). MLVSS concentrations during the sampling program ranged from 4,100 mg/L to 4,940 mg/L.

Similarly, measured solids concentrations in the RAS/WAS stream were relatively stable over the sampling period.

The pH of one grab sample from each stream was also measured during the sampling period. The pH of each sample was found to be 7.05, 7.09, and 7.05 for samples collected from Aeration Tank 1, Aeration Tank 2, and the RAS/WAS stream, respectively.

Table 3.4 Summary of Mixed Liquor Quality

	Aeration Tank 1			Aeration Tank 2			RAS/WAS	
	MLSS	MLVSS	DO	MLSS	MLVSS	DO	TSS	VSS
	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L
October 20	6,460	4,420	5.0	6,390	4,320	4.5	23,600	15,300
October 21	6,700	4,480	5.0	6,700	4,250	4.4	20,400	13,500
October 22	6,670	4,850	5.1	6,200	4,100	4.5	24,900	16,907
October 26	6,810	4,650	5.0	7,260	4,940	4.3	18,700	13,000
October 27	6,500	4,410	5.0	6,080	4,100	5.0	20,700	14,000
October 28	6,160	4,260	5.0	- (1)	- (1)	4.6	24,800	17,400
October 29	- (1)	4,820	5.0	6,250	4,380	4.4	20,600	14,000
Average	**6,550**	**4,556**	**5.0**	**6,480**	**4,350**	**4.5**	**21,957**	**14,873**

Notes:
1. Sample considered outlier and removed.

3.5 Secondary Clarifier Effluent

Over the duration of the intensive sampling program, seven (7) 24-hour composite samples were collected from the tertiary filter influent channel, and are representative of the secondary clarifier effluent stream. Due to low influent flows, only one secondary clarifier was operated for the duration of the sampling program. A summary of sampling results is located in Table 3.5.

The concentration of several measured parameters was below the minimum detection limit (MDL) established by the accredited laboratory. Samples measuring below the

Draft

MDL were assumed to be at the MDL for purposes of calculating the average concentration over the sampling program.

Over the sampling program, the TAN concentration of all samples was below the MDL, indicating complete nitrification in the aeration tanks. Further, TSS and TP concentrations were quite low, indicating the biological solids were readily settleable in the secondary clarifier.

In addition to the above results, the pH of the sample collected October 22, 2015 was measured to be 7.28.

Table 3.5 Summary of Secondary Clarifier Effluent Quality

	BOD_5	$cBOD_5$	COD	TP	Ortho-P	TKN	TAN	Nitrate	Alkalinity	TSS	VSS
	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L
October 20	< 4	< 4	< 8	0.14	0.03	< 0.5	< 0.1	11.8	195	6	4
October 21	< 4	< 4	12	0.14	0.04	1.4	< 0.1	12.2	174	4	3
October 22	< 4	< 4	8	0.16	0.05	0.9	< 0.1	12.8	170	5	5
October 26	5	< 4	10	0.16	<0.03	< 0.5	< 0.1	13.3	165	6	5
October 27	< 4	< 4	< 8	0.12	0.08	< 0.5	< 0.1	13.1	169	4	4
October 28	< 4	< 4	10	0.13	0.04	< 0.5	< 0.1	13.4	171	5	5
October 29	6	< 4	< 8	0.16	0.04	1.0	< 0.1	13.0	165	4	4
Average	**4.4**	**4.0**	**9.1**	**0.14**	**0.04**	**0.76**	**0.1**	**12.8**	**173**	**4.8**	**4.3**

3.6 Final Effluent

Over the duration of the intensive sampling program, seven (7) 24-hour composite samples were collected from the channel immediately downstream of the UV disinfection process, and are representative of the final effluent stream. A summary of sampling results is located in Table 3.6. The table also presents the final effluent objective and limit concentrations, where applicable.

Similar to above, the concentration of several measured parameters was below the minimum detection limit (MDL) established by the accredited laboratory. Samples measuring below the MDL were assumed to be at the MDL for purposes of calculating the average concentration over the sampling program.

Results show final effluent remained at a high quality over the duration of the intensive sampling program.

Draft

Table 3.6 Summary of Final Effluent Quality

	BOD_5	$cBOD_5$	COD	TP	Ortho-P	TAN	Nitrate	Alkalinity	TSS	VSS	pH
	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	
October 20	< 4	< 4	< 8	0.09	0.06	< 0.1	11.5	177	2	2	7.42
October 21	< 4	< 4	< 8	0.08	0.04	< 0.1	12.2	173	< 2	< 2	-
October 22	< 4	< 4	8	0.09	0.06	< 0.1	12.7	171	< 2	2	7.49
October 26	< 4	< 4	9	0.06	<0.03	< 0.1	13.3	164	< 2	< 2	-
October 27	< 4	< 4	17	0.07	0.04	< 0.1	13.1	157	< 2	< 2	-
October 28	< 4	< 4	12	0.06	0.03	< 0.1	13.3	170	< 2	2	-
October 29	< 4	< 4	8	0.11	0.04	< 0.1	13.0	176	< 2	2	-
Average	**4.0**	**4.0**	**10**	**0.08**	**0.04**	**0.1**	**12.7**	**170**	**2**	**2**	**7.46**
Eff. Obj.		**8.0**		**0.13**		**0.8** [(1)]			**8.0**		
Eff. Lim.		**10.0**		**0.15**		**1.0** [(1)]			**10.0**		

Notes:
1. Final effluent TAN objective and limit for the fall period (October 1 to November 30).

Draft

APPENDIX A
INTENSIVE SAMPLING PROGRAM PROTOCOL

Draft

XCG Environmental Engineers & Scientists
XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

October 14, 2015 **XCG File No.:3-252-57-01**

To: Jane Wilson, Town of Grand Valley

cc: Glenn Sterret, Town of Grand Valley
Jeff Bunn, Town of Grand Valley
Scott Craggs, OCWA

From: Graham Seggewiss and Melody Johnson, XCG Consulting Limited

Re: Grand Valley Water Pollution Control Plant Capacity Evaluation Re-rating Study - Intensive Sampling Program Protocol

Draft

The Grand Valley Water Pollution Control Plant (WPCP) provides treatment for wastewater generated in the community of Grand Valley, within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of Environment and Climate Change (MOECC) Certificate of Approval (CofA) N. 9706-7KWQ57, issued February 2, 2009. The quality and quantity of effluent currently discharged by the existing WPCP is regulated by the CofA. The Grand Valley WPCP has a rated average capacity of 1,244 m^3/d.

XCG Consulting Limited (XCG) recently completed an update to the Assimilative Capacity Study to propose effluent limits associated with an increase in the rated capacity to 2,547 m^3/d. The proposed effluent limit associated with total phosphorus (TP) for this increased capacity was very low at 0.073 mg/L. Consistently achieving such low TP requirements requires enhanced tertiary treatment, such as dual-stage tertiary filtration or membrane ultrafiltration. Upgrading the Grand Valley WPCP to provide this level of treatment would require a significant capital expenditure.

As such, the Town has retained XCG to conduct a capacity evaluation and re-rating study at the Grand Valley WPCP to potentially defer the next required plant update. An intensive sampling program was proposed as part of the capacity evaluation in order to characterize the wastewater at the plant for the purposes of subsequent BioWin™ modelling, to assess the performance of individual unit processes, and to review the ability of the current plant to maintain its required level of performance at the plant's rated capacity.

The objective of this document is to present the proposed sampling protocol developed to obtain wastewater characterization data.

1. SAMPLING PROGRAM OVERVIEW

The sampling program will consist of the collection of 24-hour composite samples at the following locations:

- Plant influent raw wastewater (including raw wastewater, septage, and recycle flow from the onsite pumping station);
- Secondary clarifier effluent; and,
- Tertiary filter effluent.

The sampling program will take place over a seven day period. As such, seven 24-hour composite samples will be collected over the duration of the sampling program at each of the locations identified above.

The sampling program will also include collection of grab samples of the following streams:

- Septage influent;
- Aeration Tank 1;
- Aeration Tank 2;
- Return activated sludge (RAS)/waste activated sludge (WAS); and,
- Tertiary filter backwash.

Seven discrete grab samples will be collected from each of the locations identified above over the duration of the sampling program, or one sample per day per stream.

With respect to the proposed sampling locations, it is important to note the following:

- Samples of raw wastewater from the collection system will not be collected. Plant operators have indicated there is no suitable location to install a composite sampler upstream of the headworks building at the Grand Valley WPCP. Raw wastewater strength will be characterised by the plant influent raw wastewater sample; and,
- Samples of the digester supernatant will not be collected. Plant operators have indicated all solids from the biosolids holding tank and the digesters were recently hauled from the plant. As such, the digesters will not be supernated over the sampling program.

A process flow diagram of the Grand Valley Wastewater Treatment Plant with identified sampling locations is presented in Figure 1. A matrix summarizing the sampling parameters and sampling locations is provided in Table 1.

Table 2 summarizes the tests which have been requested as part of this intensive sampling program. The table also indicates whether analysis will be carried out onsite or by an accredited laboratory, as well as sampling handling requirements, which are described in greater detail in Section 2.

Draft

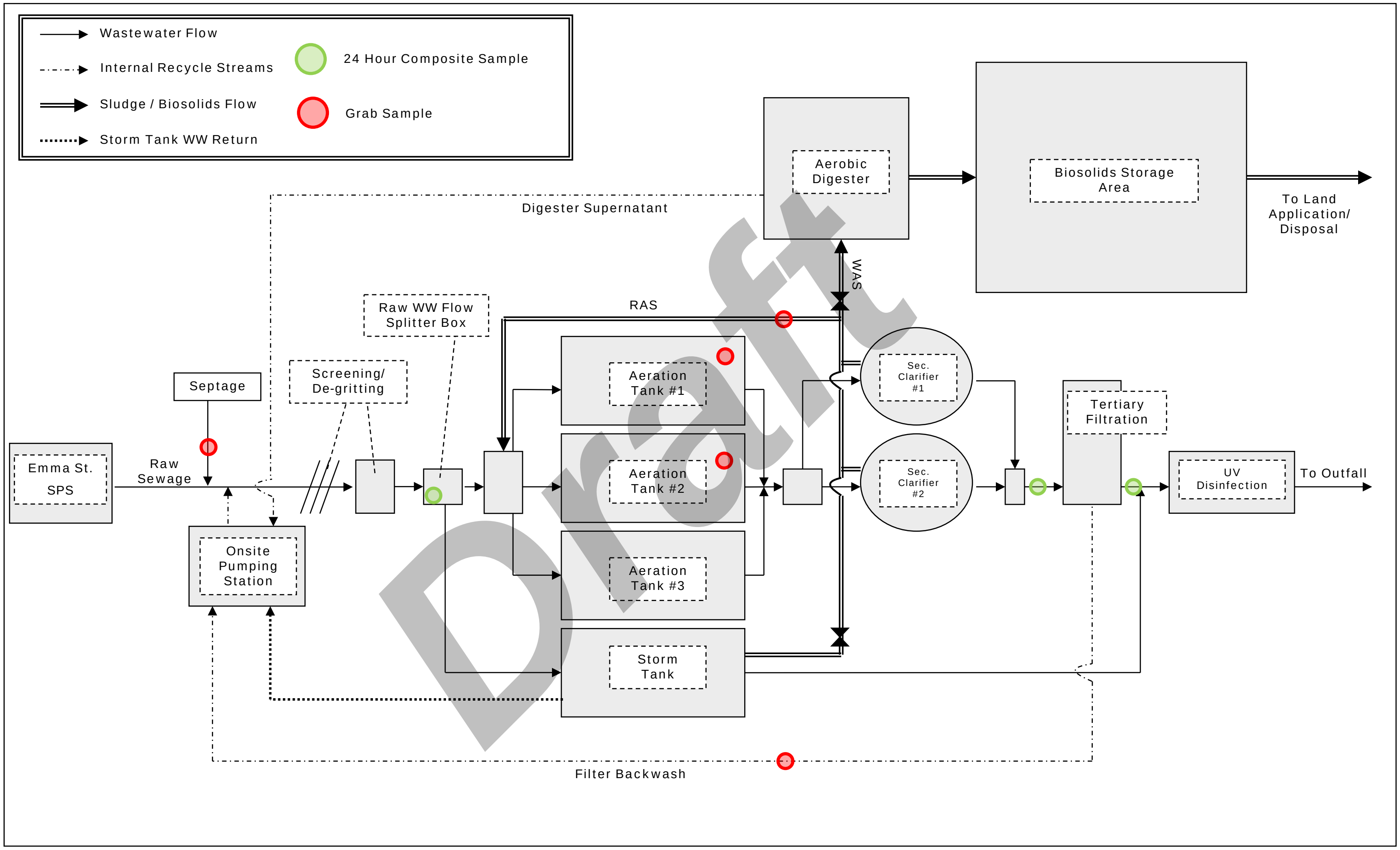


Figure 1 Process Flow Diagram of the Grand Valley WPCP

Table 1 Summary Matrix of Intensive Sampling Program

	Sample Type	BOD_5	$cBOD_5$	$fcBOD_5$	COD	fCOD	ffCOD	Total Phosphorus	Orthophosphate [1]	TKN	Total Ammonia - N	Nitrite	Nitrate	Alkalinity	TSS	VSS	pH	DO
Plant Influent Raw Wastewater	24-hr Comp.	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	
Secondary Effluent	24-hr Comp.	x	x		x			x	x	x	x	x	x	x	x	x	x	
Tertiary Effluent	24-hr Comp.	x	x		x			x	x		x	x	x	x	x	x	x	
Aeration Tank 1	Grab														x	x	x	x
Aeration Tank 2	Grab														x	x	x	x
RAS/WAS	Grab														x	x	x	
Tertiary Filter Backwash	Grab	x			x			x	x						x	x		
Septage	Grab	x	x		x			x	x		x	x	x	x	x	x	x	

Notes:
1. Orthophosphate concentration represented by measurements of soluble reactive phosphorus (SRP)

Draft

Table 2 Wastewater Characterization - Parameters for Analysis

Parameters Required	Sample Analysis	Sample Handling Requirements Prior to Bottling Sample
Total COD (tCOD)	Accredited Laboratory	None
Filtered COD (fCOD) (1)	Accredited Laboratory	On-site filtration
Flocculated and Filtered COD (ffCOD) (1)	Accredited Laboratory	On-site flocculation and filtration
$tBOD_5$	Accredited Laboratory	None
$cBOD_5$	Accredited Laboratory	None
Filtered $cBOD_5$ ($fcBOD_5$) (1)	Accredited Laboratory	On-site filtration
Total Phosphorus (TP)	Accredited Laboratory	None
Soluble Reactive Phosphorus (SRP)	Accredited Laboratory	On-site filtration
Total Kjeldahl Nitrogen (TKN)	Accredited Laboratory	None
Total Ammonia Nitrogen (TAN)	Accredited Laboratory	None
Nitrate + Nitrite Nitrogen	Accredited Laboratory	None
Alkalinity ($CaCO_3$ equivalent)	Accredited Laboratory	None
Total Suspended Solids (TSS)	Accredited Laboratory	None
Volatile Suspended Solids (VSS)	Accredited Laboratory	None
pH	Onsite	None
Dissolved Oxygen (DO) (2)	Onsite	None

Notes:
1. To be completed on the plant influent raw wastewater only.
2. As measured in the aeration tanks.

Draft

2. SAMPLING HANDLING AND ANALYSIS REQUIREMENTS

2.1 Sample Analysis

Sample containers will be obtained from the accredited laboratory pre-cleaned and will not be rinsed prior to sample collection. Preservatives, if required, will be added by the laboratory to the containers prior to shipment of the containers to the site.

All samples will be collected into the correct sample container and kept in an insulated container (i.e., cooler) packed with ice, until delivered to the laboratory.

The following procedure will be followed when filling sample bottles:

- Fill bottles to the shoulder only (do not overfill or overflow containers),
- Do not rinse out bottles or preservatives, and,
- Keep samples on ice in a cooler after collection.

A list of the analytical method for each analyte of interest is provided in Appendix A. The table also lists the type of container and sample quantity needed, preservatives and holding times for each analytical method.

2.2 Special Sample Handling Protocols

Special sample handling protocols are required for the analysis of the following parameters:

- Filtered COD;
- Filtered $cBOD_5$;
- Soluble reactive phosphorus (SRP); and,
- Flocculated / filtered COD.

The sampling handling requirements are outlined in detail below.

2.2.1 On-site Sample Filtration - for fCOD, $fcBOD_5$ and Soluble Reactive Phosphorus

Filtered COD (fCOD) and filtered $cBOD_5$ ($fcBOD_5$) analyses of wastewater samples will require on-site filtration of the samples collected prior to placement in the applicable sample bottles and subsequent submission to the laboratory for analysis. It is also recommended, but not required, that dissolved reactive phosphorus analyses be conducted on filtered samples.

Sample filtration can be accomplished by utilizing glass filters, such as those commonly used for mixed liquor suspended solids (MLSS) determinations. All filter apparatus / glassware should be thoroughly cleaned prior to filtering the samples.

The filtered samples can then be submitted for standard COD, $cBOD_5$ and dissolved reactive phosphorus analyses at the laboratory while ensuring that the filtered samples are appropriately labelled.

Draft

2.2.2 On-site Sample Flocculation and Filtration - for ffCOD

The flocculated and filtered COD (ffCOD) analysis requires the on-site flocculation and filtration of the samples prior to placement in the applicable sample bottles and subsequent submission to the laboratory for analysis.

The flocculation and filtration protocol is presented below:

FFCOD Analysis Procedure

Materials/Equipment List:

- Zinc sulfate ($ZnSO4.7H_2O$);
- 6 M sodium hydroxide;
- Distilled/deionized water;
- 500 mL beaker;
- pH analyser;
- Stir plate;
- Glass fiber filters (preferred size of 0.45 µm);

- Filtration apparatus; and,
- 0 to 10 mL pipette.

Stock Solution Procedure:

Make up a stock solution of zinc sulfate as follows:

- Dissolve 20 g of zinc sulfate (ZnSO4.7H2O) into 200 mL of distilled/deionized water.

ffCOD Procedure:

The ffCOD procedure is as follows:

- Pipette 2 mL of the 100 g/L zinc sulfate stock solution into a 200 mL sample (or 1 mL 100 g/L zinc sulfate stock solution into a 100 mL sample) of filtered wastewater (if you are doing filtered CODs, it is convenient to save some additional filtered sample for the ffCOD procedure);
- Mix the sample vigorously for approximately one minute (i.e. use a stir plate);
- Turn the mixer to low, set up a pH probe in the sample and add 6 M sodium hydroxide solution drop-wise until the pH is adjusted to approximately 10.5;
- You should see flocs start to form in the sample;
- Gently mix the sample for several minutes (e.g. 10-15);
- Turn off the mixer and allow the sample to settle. A fairly clear supernatant should be evident; and,
- Withdraw 40-50 mL of the supernatant with a pipette (trying not to pull up any of the settled solids) and filter the sample.

As with the on-site filtration procedure, the filtration steps can be accomplished by utilizing glass filters, such as those commonly used for MLSS determinations.

The flocculated and filtered samples can then be submitted for standard COD analyses at the Laboratory.

Draft

3. ROLES AND RESPONSIBILITY

XCG will coordinate set-up of the intensive sampling program with assistance from plant personnel and Town Staff. Plant personnel and Town Staff will be responsible for sample collection, chain-of-custody preparation, and sample submission.

A summary of the responsibilities of the Consultant Team and plant personnel is provided in the following Sections.

3.1 Consultant Staff Roles and Responsibilities

XCG staff will be responsible for the following:

- Provision and temporary installation of three auto-samplers installed to collect samples of plant influent raw wastewater, secondary clarifier effluent, and tertiary filter effluent;
- Program the installed auto-sampler(s) to collect composite samples as required by the testing protocol;

- Provision of pre-mixed zinc sulfate and sodium hydroxide solutions at concentrations specified in Section 2.2.2;
- Provide training to OCWA staff with respect to the operation of the auto-samplers, as well as conducting the specialized sample handling procedures for the "filtered" and "flocculated and filtered" samples as per Section 2.2; and
- Provide input to plant personnel throughout the duration of the intensive sampling program, as required. XCG's main point of contact for questions or concerns during the sampling program will be Graham Seggewiss. If there are any questions in advance or during the testing period, he can be reached at 905-829-8880 x 4224 or graham.seggewiss@xcg.com.

3.2 Plant Personnel Roles and Responsibilities

Plant personnel will be responsible for the following:

- Operation, monitoring and control of plant process and equipment to maintain plant performance during the intensive sampling program;
- Providing guidance to XCG staff with respect to appropriate installation locations for the field testing equipment. This will include providing access to 120V power outlets to power the equipment;
- Ordering the required number of sample bottles from an accredited laboratory, and co-ordinating their delivery to and pick up from the Grand Valley WPCP;
- Collecting samples from the temporary auto-samplers, placing sample aliquots in the proper sample bottles, and filling in the chain of custody forms to obtain the required analyses;
- Collecting grab samples from locations identified in Section 1, placing sample aliquots in the proper sample bottles and filling in the chain of custody forms to obtain the required analyses;
- Conduct onsite flocculation and filtration procedures for samples as identified in Section 2.2, completed onsite pH measurements as required, and measure DO at locations identified in Section 1; and,
- Provision of plant flows, pH measurements and DO concentrations during the intensive sampling period.

Draft

ATTACHMENT A
ANALYTICAL METHODS

Draft

Table A.1 Analytical Methods

Parameter	Analytical Method	Minimum Required Sample Volume (mL) [(1)]	Sample Bottle Type	Preservation Requirements	Maximum Holding Time (d)	
					External Lab	MOE
COD	APHA 5220 D	50	Plastic or glass	Chill to < 4°C	28	30
BOD_5	SM 5210 B	300	Plastic	Chill to < 4°C	4	4
$cBOD_5$	SM 5210B	300	Plastic	Chill to < 4°C	4	4
TSS	SM 2540 B,D,E	500	Plastic	Chill to < 4°C	7	7
TAN	MOE STKNP-E3199A.I	300	Plastic or glass	Chill to < 4°C	3	10
TP	MOE STKNP-E3199A.I	100	Plastic or glass	Chill to < 4°C	28	30
TKN	MOE STKNP-E3199A.I	100	Plastic or glass	pH < 2, H_2SO_4 Chill to < 4°C	28	NA
SRP	MOE STKNP-E3199A.I	- [(2)]	Plastic	Chill to < 4°C	48hr	NA
Nitrate + Nitrite	APHA 5220D	50	Plastic	Chill to < 4°C	7	7
Nitrate	APHA 4110C	50	Plastic	Chill to < 4°C	7	7
Nitrite	APHA 4110C	50	Plastic	Chill to < 4°C	7	7
Alkalinity	SM 2320B	50	Plastic	Chill to < 4°C	7	7
VSS	SM 2540 B,D,E					

Notes:
NA not applicable
1. All sample volumes should be confirmed with selected accredited laboratory.
2. Required volume as indicated by selected accredited laboratory.

Draft

APPENDIX B

COPY OF SAMPLING RESULTS

Draft

SGS

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - K0L 2H0
Phone: 705-652-2000 FAX: 705-652-6365

Works #: 110000301
Project : PO#017844

04-November-2015

OCWA-Grand Valley-XCG (WPCP)
Attn : Lisa Benoit

78 Centennial Road, Unit 6
Orangeville, ON
L9W 1P9, Canada

Phone: 519-941-1938
Fax:519-941-1794 pdf

Date Rec. : 21 October 2015
LR Report: CA13574-OCT15

Copy: #1

CERTIFICATE OF ANALYSIS
Final Report

Draft

Sample ID	Sample Date & Time	Temperature Upon Receipt °C	Field pH no unit	Field Temperature celcius	Biochemical Oxygen Demand (BOD5) mg/L	Carbonaceous Biochemical Oxygen Demand (CBOD5) mg/L	Total Suspended Solids mg/L	Volatile Suspended Solids mg/L
1: Analysis Start Date		---	---	---	21-Oct-15	21-Oct-15	22-Oct-15	22-Oct-15
2: Analysis Start Time		---	---	---	17:50	19:51	11:10	11:10
3: Analysis Approval Date		---	---	---	27-Oct-15	28-Oct-15	23-Oct-15	26-Oct-15
4: Analysis Approval Time		---	---	---	12:58	08:53	14:28	11:27
6: Raw-1	20-Oct-15 10:50	12.0	7.72	12.9	81	142	83	78
7: RawF-1	20-Oct-15 10:50	12.0	---	---	---	28	---	---
8: RawFF-1	20-Oct-15 12:00	12.0	---	---	---	---	---	---
9: SE-1	20-Oct-15 10:30	12.0	---	---	< 4	< 4	6	4
10: TE-1	20-Oct-15 10:40	12.0	7.41	16.0	< 4	< 4	2	2
11: AT1-1	20-Oct-15 09:05	12.0	7.05	12.3	---	---	6480	4420
12: AT2-1	20-Oct-15 09:05	12.0	7.09	12.4	---	---	6390	4320
13: RAS-1	20-Oct-15 09:10	12.0	7.05	12.9	---	---	23600	15300
14: TBW-1	20-Oct-15 09:20	12.0	7.24	14.1	10	---	32	22

Sample ID	Alkalinity mg/L as CaCO3	Chemical Oxygen Demand mg/L	Phosphorus (total reactive) mg/L	Phosphorus (total) mg/L	Total Kjeldahl Nitrogen as N mg/L	Ammonia+Ammonium (N) mg/L	Nitrite (as N) mg/L	Nitrate (as N) mg/L	Nitrate + Nitrite (as N) mg/L
1: Analysis Start Date	21-Oct-15	27-Oct-15	22-Oct-15	22-Oct-15	23-Oct-15	22-Oct-15	24-Oct-15	24-Oct-15	24-Oct-15
2: Analysis Start Time	15:52	09:20	08:08	10:00	07:30	07:30	08:22	08:22	08:22
3: Analysis Approval Date	22-Oct-15	04-Nov-15	23-Oct-15	24-Oct-15	23-Oct-15	26-Oct-15	27-Oct-15	27-Oct-15	27-Oct-15
4: Analysis Approval Time	14:57	09:48	09:25	21:05	15:32	11:50	15:21	15:21	15:21
6: Raw-1	300	150	0.97	1.95	16.2	15.1	< 0.03	< 0.06	< 0.06
7: RawF-1	---	62	---	---	---	---	---	---	---
8: RawFF-1	---	58	---	---	---	---	---	---	---
9: SE-1	195	< 8	0.03	0.14	< 0.5	< 0.1	< 0.03	11.8	11.8
10: TE-1	177	< 8	0.06	0.09	---	< 0.1	< 0.03	11.5	11.5
11: AT1-1	---	---	---	---	---	---	---	---	---
12: AT2-1	---	---	---	---	---	---	---	---	---
13: RAS-1	---	---	---	---	---	---	---	---	---
14: TBW-1	---	9	0.07	0.74	---	---	---	---	---

OnLine LIMS

0000543820

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - K0L 2H0
Phone: 705-652-2000 FAX: 705-652-6365

Works #: 110000301
Project : PO#017844
LR Report : CA13574-OCT15

*CBOD values should not exceed BOD values. These differences in results on the "Raw-1" sample may indicate differences in sample portions used for analysis (non-homogenous).

Carrie Greenlaw
Carrie Greenlaw
Project Specialist
Environmental Services, Analytical

Draft

OnLine LIMS

0000543820

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

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

OCWA-Grand Valley-XCG (WPCP)
Attn : Lisa Benoit

78 Centennial Road, Unit 6
Orangeville, ON
L9W 1P9, Canada

Phone: 519-941-1938
Fax:519-941-1794 pdf

Works #: 110000301
Project : PO#017844

29-October-2015

Date Rec. : 22 October 2015
LR Report: CA12483-OCT15

Copy: #1

CERTIFICATE OF ANALYSIS
Final Report

Sample ID	Sample Date & Time	Temperature Upon Receipt °C	Biochemical Oxygen Demand (BOD5) mg/L	Carbonaceous Biochemical Oxygen Demand (CBOD5) mg/L	Total Suspended Solids mg/L	Volatile Suspended Solids mg/L	Alkalinity mg/L as CaCO3	Chemical Oxygen Demand mg/L	Phosphorus (total reactive) mg/L	Phosphorus (total) mg/L	Total Kjeldahl Nitrogen as N mg/L	Ammonia+Ammonium (N) mg/L	Nitrite (as N) mg/L	Nitrate (as N) mg/L	Nitrate + Nitrite (as N) mg/L
1: Analysis Start Date		—	22-Oct-15	22-Oct-15	23-Oct-15	23-Oct-15	23-Oct-15	28-Oct-15	23-Oct-15	22-Oct-15	22-Oct-15	22-Oct-15	26-Oct-15	26-Oct-15	26-Oct-15
2: Analysis Start Time		—	17:33	19:20	09:03	09:03	11:09	11:21	06:08	22:00	20:00	22:00	08:40	08:40	08:40
3: Analysis Approval Date		—	28-Oct-15	28-Oct-15	29-Oct-15	29-Oct-15	26-Oct-15	29-Oct-15	24-Oct-15	25-Oct-15	29-Oct-15	24-Oct-15	28-Oct-15	28-Oct-15	28-Oct-15
4: Analysis Approval Time		—	16:12	21:55	09:33	13:05	11:48	11:19	14:39	15:41	13:05	15:29	12:56	12:56	12:56
5: Client Limits		—		10	10					0.15		0.7	—	—	—
6: Raw-1	21-Oct-15 10:50	14.0	86	85	85	80	288	150	0.99	2.17	19.8	16.2	< 0.03	< 0.06	< 0.06
7: RawF-1	21-Oct-15 10:50	14.0	—	17	—	—	—	50	—	—	—	—	—	—	—
8: RawFF-1	21-Oct-15 11:30	14.0	—	—	—	—	—	40	—	—	—	—	—	—	—
9: SE-1	21-Oct-15 10:30	14.0	< 4	4	4	3	174	12	0.04	0.14	1.4	< 0.1	< 0.03	12.2	12.2
10: TE-1	21-Oct-15 10:40	14.0	< 4	4	< 2	< 2	173	< 8	0.04	0.08	—	< 0.1	< 0.03	12.2	12.2
11: AT1-1	21-Oct-15 07:55	14.0	—	—	6700	4480	—	—	—	—	—	—	—	—	—
12: AT2-1	21-Oct-15 07:55	14.0	—	—	6700	4260	—	—	—	—	—	—	—	—	—
13: RAS-1	21-Oct-15 07:55	14.0	—	—	20400	13500	—	—	—	—	—	—	—	—	—
14: TBW-1	21-Oct-15 08:10	14.0	13	—	38	27	—	35	0.07	0.65	—	—	—	—	—

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

Online LIMS

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

Works #: 110000301
Project : PO#017844
LR Report : CA12483-OCT15

Carrie Greenlaw
Carrie Greenlaw
Project Specialist
Environmental Services, Analytical

Draft

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

OnLine LIMS

SGS

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

Works #: 110000301-NR
Project : PO#017844

04-November-2015

OCWA-Grand Valley-XCG (WPCP)
Attn : Lisa Benoit

78 Centennial Road, Unit 6
Orangeville, ON
L9W 1P9, Canada

Phone: 519-941-1938
Fax:519-941-1794 pdf

Date Rec. : 23 October 2015
LR Report: CA13692-OCT15

Copy: #1

CERTIFICATE OF ANALYSIS
Final Report

Draft

Sample ID	Sample Date & Time	Temperature Upon Receipt °C	Field pH no unit	Field Temperature celcius	Biochemical Oxygens Demand (BOD5) mg/L	Carbonaceou Biochemical Oxygen Demand (CBOD5) mg/L	Total Suspended Solids mg/L	Volatile Suspended Solids mg/L
1: Analysis Start Date		---	---	---	23-Oct-15	23-Oct-15	27-Oct-15	23-Oct-15
2: Analysis Start Time		---	---	---	17:24	19:29	16:11	10:55
3: Analysis Approval Date		---	---	---	28-Oct-15	29-Oct-15	29-Oct-15	30-Oct-15
4: Analysis Approval Time		---	---	---	22:29	09:09	13:00	09:12
6: Raw-1	22-Oct-15 11:00	12.0	7.68	13.3	145	99	134	122
7: RawF-1	22-Oct-15 11:00	12.0	---	---	---	26	---	---
8: RawFF-1	22-Oct-15 11:00	12.0	---	---	---	---	---	---
9: SE-1	22-Oct-15 10:35	12.0	7.28	16.4	< 4	< 4	5	5
10: TE-1	22-Oct-15 10:50	12.0	7.49	15.8	< 4	< 4	< 2	2
11: AT1-1	22-Oct-15	12.0	7.22	13.6	---	---	6870	4850
12: AT2-1	22-Oct-15	12.0	7.21	13.5	---	---	6200	4100
13: RAS-1	22-Oct-15	12.0	7.11	13.7	---	---	24900	16917
14: TBW-1	22-Oct-15 08:00	12.0	7.30	14.6	12	---	45	32

Sample ID	Alkalinity mg/L as CaCO3	Chemical Oxygen Demand mg/L	Phosphorus (total reactive) mg/L	Phosphorus (total) mg/L	Total Kjeldahl Nitrogen as N mg/L	Ammonia+Ammonium (N) mg/L	Nitrite (as N) mg/L	Nitrate (as N) mg/L	Nitrate + Nitrite (as N) mg/L
1: Analysis Start Date	26-Oct-15	29-Oct-15	23-Oct-15	23-Oct-15	23-Oct-15	27-Oct-15	27-Oct-15	27-Oct-15	27-Oct-15
2: Analysis Start Time	08:54	11:04	15:30	22:00	21:00	21:57	21:10	21:10	21:10
3: Analysis Approval Date	27-Oct-15	04-Nov-15	28-Oct-15	26-Oct-15	27-Oct-15	27-Oct-15	30-Oct-15	30-Oct-15	30-Oct-15
4: Analysis Approval Time	16:34	09:49	15:50	09:52	13:36	14:39	12:30	12:30	12:30
6: Raw-1	286	99	1.51	2.17	21.4	15.0	< 0.03	< 0.06	< 0.06
7: RawF-1	---	62	---	---	---	---	---	---	---
8: RawFF-1	---	51	---	---	---	---	---	---	---
9: SE-1	170	8	0.05	0.16	0.9	< 0.1	< 0.03	12.8	12.8
10: TE-1	171	8	0.06	0.09	---	< 0.1	< 0.03	12.7	12.7
11: AT1-1	---	---	---	---	---	---	---	---	---
12: AT2-1	---	---	---	---	---	---	---	---	---
13: RAS-1	---	---	---	---	---	---	---	---	---
14: TBW-1	---	12	0.11	0.85	---	---	---	---	---

OnLine LIMS

0005435850

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

SGS

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

Works #: 110000301-NR
Project : PO#017844
LR Report : CA13692-OCT15

*BOD values should not exceed COD values. These differences in results on the "Raw-1" sample may indicate differences in sample portions used for analysis (non-homogenous).

Carrie Greenlaw
Carrie Greenlaw
Project Specialist
Environmental Services, Analytical

Draft

OnLine LIMS

0000543850

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

SGS

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

Works #: 110000301
Project : PO#017844

OCWA-Grand Valley-XCG (WPCP)
Attn : Lisa Benoit

78 Centennial Road, Unit 6
Orangeville, ON
L9W 1P9, Canada

Phone: 519-941-1938
Fax:519-941-1794 pdf

04-November-2015

Date Rec. : 27 October 2015
LR Report: CA12566-OCT15

Copy: #1

CERTIFICATE OF ANALYSIS
Final Report

Draft

Analysis	1: Analysis Start Date	2: Analysis Start Time	3: Analysis Approval Date	4: Analysis Approval Time	6: Raw-1	7: RawF-1	8: RawFF-1	9: SE-1	10: TE-1	11: AT1-1	12: AT2-1	13: RAS-1	14: TBW-1
Sample Date & Time					26-Oct-15 09:00	26-Oct-15	26-Oct-15	26-Oct-15 08:45	26-Oct-15 08:50	26-Oct-15 08:20	26-Oct-15 08:20	26-Oct-15 08:20	26-Oct-15 08:35
Temperature Upon Receipt [°C]	—	—	—	—	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0
Biochemical Oxygen Demand (BOD5) [mg/L]	27-Oct-15	16:35	02-Nov-15	20:19	154	—	—	6	< 4	—	—	—	19
Carbonaceous Biochemical Oxygen Demand [(CBOD5) m	27-Oct-15	18:26	02-Nov-15	19:58	160	36	—	< 4	< 4	—	—	—	—
Total Suspended Solids [mg/L]	28-Oct-15	13:47	03-Nov-15	09:55	109	—	—	6	< 2	6810	7280	18700	46
Volatile Suspended Solids [mg/L]	28-Oct-15	13:47	03-Nov-15	14:46	98	—	—	5	< 2	4650	4940	13000	35
Alkalinity [mg/L as CaCO3]	28-Oct-15	06:31	28-Oct-15	20:43	289	—	—	165	164	—	—	—	—
Chemical Oxygen Demand [mg/L]	03-Nov-15	10:18	04-Nov-15	15.07	160	60	65	10	9	—	—	—	20
Phosphorus (total reactive) [mg/L]	27-Oct-15	16:50	28-Oct-15	15:32	0.97	—	—	< 0.03	< 0.03	—	—	—	0.07
Phosphorus (total) [mg/L]	27-Oct-15	21:49	28-Oct-15	14:26	2.27	—	—	0.16	0.06	—	—	—	0.80
Total Kjeldahl Nitrogen [as N mg/L]	28-Oct-15	06:20	29-Oct-15	13:01	20.8	—	—	< 0.5	—	—	—	—	—
Ammonia+Ammonium (N) [mg/L]	28-Oct-15	21:25	28-Oct-15	12:17	15.6	—	—	< 0.1	< 0.1	—	—	—	—
Nitrite (as N) [mg/L]	28-Oct-15	23:51	29-Oct-15	09.54	< 0.03	—	—	< 0.03	< 0.03	—	—	—	—
Nitrate (as N) [mg/L]	28-Oct-15	23:51	29-Oct-15	09:54	< 0.06	—	—	13.3	13.3	—	—	—	—
Nitrate + Nitrite (as N) [mg/L]	28-Oct-15	23:51	29-Oct-15	09:54	< 0.06	—	—	13.3	13.3	—	—	—	—

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

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

Works #: 110000301
Project : PO#017844
LR Report : CA12566-OCT15

Carrie Greenlaw
Carrie Greenlaw
Project Specialist
Environmental Services, Analytical

Draft

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

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

Works #: 110000301
Project : PO#017844

OCWA-Grand Valley-XCG (WPCP)
Attn : Lisa Benoit

04-November-2015

78 Centennial Road, Unit 6
Orangeville, ON
L9W 1P9, Canada

Date Rec. : 28 October 2015
LR Report: CA12617-OCT15

Copy: #1

Phone: 519-941-1938
Fax:519-941-1794 pdf

CERTIFICATE OF ANALYSIS
Final Report

Draft

Sample ID	Sample Date & Time	Temperature Upon Receipt °C	Biochemical Oxygen Demand (BOD5) mg/L	Carbonaceous Biochemical Oxygen Demand (CBOD5) mg/L	Total Suspended Solids mg/L	Volatile Suspended Solids mg/L	Alkalinity mg/L as CaCO3	Chemical Oxygen Demand mg/L	Phosphorus (total reactive) mg/L	Phosphorus (total) mg/L	Total Kjeldahl Nitrogen as N mg/L	Ammonia+Ammonium (N) mg/L	Nitrite (as N) mg/L	Nitrate (as N) mg/L	Nitrate + Nitrite (as N) mg/L
1: Analysis Start Date		—	28-Oct-15	28-Oct-15	29-Oct-15	29-Oct-15	29-Oct-15	03-Nov-15	30-Oct-15	28-Oct-15	28-Oct-15	29-Oct-15	30-Oct-15	30-Oct-15	30-Oct-15
2: Analysis Start Time		—	17:45	19:17	10:38	10:38	06:46	12:37	09:30	21:15	21:59	21:57	20:37	20:37	20:37
3: Analysis Approval Date		—	03-Nov-15	03-Nov-15	03-Nov-15	03-Nov-15	30-Oct-15	04-Nov-15	02-Nov-15	30-Oct-15	29-Oct-15	29-Oct-15	03-Nov-15	03-Nov-15	03-Nov-15
4: Analysis Approval Time		—	13:54	14:57	10:29	14:46	10:07	15:05	14:14	08:28	13:05	15:34	15:27	15:27	15:27
6: Raw-1	27-Oct-15 09:00	9.0	134	97	115	105	287	136	1.30	2.29	19.3	16.3	< 0.03	< 0.06	< 0.06
7: RawF-1	27-Oct-15 09:00	9.0	—	28	—	—	—	70	—	—	—	—	—	—	—
8: RawFF-1	27-Oct-15 09:00	9.0	—	—	—	—	—	51	—	—	—	—	—	—	—
9: SE-1	27-Oct-15 08:10	9.0	< 4	< 4	4	4	169	< 8	0.08	0.12	< 0.5	< 0.1	< 0.03	13.1	13.1
10: TE-1	27-Oct-15 08:15	9.0	< 4	< 4	< 2	< 2	157	17	0.04	0.07	—	< 0.1	< 0.03	13.1	13.1
11: AT1-1	27-Oct-15 08:30	9.0	—	—	6500	4410	—	—	—	—	—	—	—	—	—
12: AT2-1	27-Oct-15 08:30	9.0	—	—	6080	4110	—	—	—	—	—	—	—	—	—
13: RAS-1	27-Oct-15 08:30	9.0	—	—	20700	14000	—	—	—	—	—	—	—	—	—
14: TBW-1	27-Oct-15 08:05	9.0	12	—	37	26	—	14	0.12	0.70	—	—	—	—	—

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

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

Works #: 110000301
Project : PO#017844
LR Report : CA12617-OCT15

Carrie Greenlaw
Carrie Greenlaw
Project Specialist
Environmental Services, Analytical

Draft

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

SGS

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

Works #: 110000301
Project : PO#017844

05-November-2015

OCWA-Grand Valley-XCG (WPCP)
Attn : Lisa Benoit

78 Centennial Road, Unit 6
Orangeville, ON
L9W 1P9, Canada

Phone: 519-941-1938
Fax:519-941-1794 pdf

Date Rec. : 29 October 2015
LR Report: CA13855-OCT15

Copy: #1

CERTIFICATE OF ANALYSIS
Final Report

Sample ID	Sample Date & Time	Temperature Upon Receipt °C	Field pH no unit	Field Temperature celcius	Biochemical Oxygen Demand (BOD5) mg/L	Carbonaceous Biochemical Oxygen Demand (CBOD5) mg/L	Total Suspended Solids mg/L	Volatile Suspended Solids mg/L
1: Analysis Start Date		---			29-Oct-15	29-Oct-15	30-Oct-15	30-Oct-15
2: Analysis Start Time		---			16:55	18:29	16:16	16:16
3: Analysis Approval Date		---			03-Nov-15	03-Nov-15	04-Nov-15	04-Nov-15
4: Analysis Approval Time		---			15:28	16:02	15:53	16:20
6: Raw-1	28-Oct-15 09:00	14.0	7.64	10.6	125	117	150	142
7: RawF-1	28-Oct-15 09:00	14.0	---	---	---	25	---	---
8: RawFF-1	28-Oct-15 09:00	14.0	---	---	---	---	---	---
9: SE-1	28-Oct-15 08:30	14.0	7.38	14.6	< 4	< 4	5	5
10: TE-1	28-Oct-15 08:35	14.0	7.49	14.7	< 4	< 4	< 2	2
11: AT1-1	28-Oct-15	14.0	7.04	12.9	---	---	6160	4350
12: AT2-1	28-Oct-15	14.0	7.12	12.3	---	---	4260	3060
13: RAS-1	28-Oct-15	14.0	7.06	12.3	---	---	24800	17400
14: TBW-1	28-Oct-15 08:20	14.0	7.23	13.7	13	---	40	28

Sample ID	Alkalinity mg/L as CaCO3	Chemical Oxygen Demand mg/L	Phosphorus (total reactive) mg/L	Phosphorus (total) mg/L	Total Kjeldahl Nitrogen as N mg/L	Ammonia+Ammonium (N) mg/L	Nitrite (as N) mg/L	Nitrate (as N) mg/L	Nitrate + Nitrite (as N) mg/L
1: Analysis Start Date	30-Oct-15	03-Nov-15	30-Oct-15	29-Oct-15	30-Oct-15	29-Oct-15	31-Oct-15	31-Oct-15	31-Oct-15
2: Analysis Start Time	06:39	14:04	09:30	21:30	21:55	21:55	12:02	12:02	12:02
3: Analysis Approval Date	30-Oct-15	04-Nov-15	02-Nov-15	30-Oct-15	02-Nov-15	03-Nov-15	04-Nov-15	04-Nov-15	04-Nov-15
4: Analysis Approval Time	15:23	09:38	11:28	11:33	11:51	12:33	16:37	16:37	16:37
6: Raw-1	297	146	1.55	2.40	20.3	16.4	< 0.03	< 0.06	< 0.06
7: RawF-1	---	63	---	---	---	---	---	---	---
8: RawFF-1	---	48	---	---	---	---	---	---	---
9: SE-1	171	10	0.04	0.13	< 0.5	< 0.1	< 0.03	13.4	13.4
10: TE-1	170	12	0.03	0.06	---	< 0.1	< 0.03	13.3	13.4
11: AT1-1	---	---	---	---	---	---	---	---	---
12: AT2-1	---	---	---	---	---	---	---	---	---
13: RAS-1	---	---	---	---	---	---	---	---	---
14: TBW-1	---	40	0.06	0.71	---	---	---	---	---

Draft

OnLine LIMS

0000544778

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

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

Works #: 110000301
Project : PO#017844
LR Report : CA13855-OCT15

Carrie Greenlaw
Carrie Greenlaw
Project Specialist
Environmental Services, Analytical

Draft

OnLine LIMS

0000544778

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

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

OCWA-Grand Valley-XCG (WPCP)
Attn : Lisa Benoit

78 Centennial Road, Unit 6
Orangeville, ON
L9W 1P9, Canada

Phone: 519-941-1938
Fax:519-941-1794 pdf

Works #: 110000301
Project : PO#017844

05-November-2015

Date Rec. : 30 October 2015
LR Report: CA13923-OCT15

Copy: #1

CERTIFICATE OF ANALYSIS
Final Report

Draft

Sample ID	Sample Date & Time	Temperature Upon Receipt °C	Biochemical Oxygen Demand (BOD5) mg/L	Carbonaceous Biochemical Oxygen Demand (CBOD5) mg/L	Total Suspended Solids mg/L	Volatile Suspended Solids mg/L	Alkalinity mg/L as CaCO3	Chemical Oxygen Demand mg/L	Phosphorus (total reactive) mg/L	Phosphorus (total) mg/L	Total Kjeldahl Nitrogen as N mg/L	Ammonia+Ammonium (N) mg/L	Nitrite (as N) mg/L	Nitrate (as N) mg/L	Nitrate + Nitrite (as N) mg/L
1: Analysis Start Date		---	30-Oct-15	30-Oct-15	02-Nov-15	02-Nov-15	30-Oct-15	04-Nov-15	30-Oct-15	02-Nov-15	30-Oct-15	30-Oct-15	31-Oct-15	31-Oct-15	31-Oct-15
2: Analysis Start Time		---	16:25	17:53	15:11	15:11	20:34	10:48	16:06	09:30	18:15	19:56	00:01	00:01	00:01
3: Analysis Approval Date		---	04-Nov-15	04-Nov-15	05-Nov-15	04-Nov-15	03-Nov-15	04-Nov-15	03-Nov-15	03-Nov-15	02-Nov-15	02-Nov-15	04-Nov-15	04-Nov-15	04-Nov-15
4: Analysis Approval Time		---	13:30	14:48	07:57	16:20	09:43	15:07	13:13	16:12	12:02	12:26	17:06	17:06	17:06
6: Raw-1	29-Oct-15 09:00	11.0	84	66	94	89	276	94	0.96	1.98	19.8	13.8	< 0.03	< 0.06	< 0.06
7: RawF-1	29-Oct-15 09:00	11.0	---	21	---	---	---	51	---	---	---	---	---	---	---
8: RawFF-1	29-Oct-15 09:00	11.0	---	---	---	---	---	40	---	---	---	---	---	---	---
9: SE-1	29-Oct-15 08:30	11.0	6	< 4	4	4	165	< 8	0.04	0.16	1.0	< 0.1	< 0.03	13.0	13.0
10: TE-1	29-Oct-15 08:35	11.0	< 4	< 4	< 2	2	176	8	0.04	0.11	---	< 0.1	< 0.03	13.0	13.0
11: AT1-1	29-Oct-15 08:05	11.0	---	---	10200	4820	---	---	---	---	---	---	---	---	---
12: AT2-1	29-Oct-15 08:05	11.0	---	---	6250	4380	---	---	---	---	---	---	---	---	---
13: RAS-1	29-Oct-15 08:05	11.0	---	---	20600	14000	---	---	---	---	---	---	---	---	---
14: TBW-1	29-Oct-15 08:20	11.0	10	---	38	28	---	13	0.08	0.83	---	---	---	---	---

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

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

Works #: 110000301
Project : PO#017844
LR Report : CA13923-OCT15

Carrie Greenlaw
Carrie Greenlaw
Project Specialist
Environmental Services, Analytical

Draft

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.)
Test method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

OnLine LIMS

APPENDIX E

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY SECONDARY CLARIFIER, TERTIARY FILTER, AND DISINFECTION STRESS TEST RESULTS

Draft

XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

XCG File No.: 3-252-57-02
September 6, 2016

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY SECONDARY CLARIFIER, TERTIARY FILTER, AND DISINFECTION STRESS TEST RESULTS

Draft

Prepared for:

TOWN OF GRAND VALLEY
5 Main Street, North
Grand Valley, Ontario
L9W 5S6

Attention: Jane Wilson

Prepared by:

XCG CONSULTING LIMITED
Suite 300, 2620 Bristol Circle
Oakville, Ontario
L6H 6Z7

TABLE OF CONTENTS

Draft

TABLES

FIGURES

APPENDICES

Draft

1. INTRODUCTION

The Grand Valley WPCP provides treatment for wastewater generated in the community of Grand Valley within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of Environment and Climate Change (MOECC) Certificate of Approval (C of A) No. 9706-7KWQ57, issued on February 2, 2009. The quality and quantity of effluent currently discharged by the existing Water Pollution Control Plant (WPCP) is regulated by the C of A. The Grand Valley WPCP has a rated average day flow (ADF) capacity of 1,244 m^3/d.

The Town has initiated an investigation to analyze the potential to re-rate the existing Grand Valley WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. The Town has retained XCG Consulting Limited (XCG) to undertake a capacity assessment of the Grand Valley WPCP to evaluate the potential to re-rate the plant. Stress testing of the secondary clarifiers, tertiary filters, and ultraviolet (UV) disinfection system was carried out from July 12 - 18, 2016. The purpose of this technical memorandum (TM) is to present the results and conclusions from the stress testing program.

Draft

2. STRESS TESTING METHODOLOGY

2.1 Background

The Grand Valley WPCP is equipped with two identical circular secondary clarifiers, four identical continuous-backwash tertiary filters, and a UV disinfection system. A summary of these processes is presented in Table 2.1.

Table 2.1 Grand Valley WPCP Process Design Information

Unit Process	Design Parameter [(1)]
Secondary Clarifiers	
Number	2
Surface Area	75.4 m^2 (each) 150.8 m^2 (total)
Filters	
Type	Continuous up-flow, deep bed, granular media
Backwash	Continuous
Number	4
Filtration Area	4.65 m^2 (each) 18.6 m^2 (total)
Design Peak Flow Capacity	5,300 m^3/d
Disinfection	
Type	UV Disinfection
Design Peak Flow Capacity	7,680 m^3/d

Notes:
1. Based on Amended Certificate of Approval Number 9706-7KWQ57, issued February 2, 2009, and the Grand Valley Wastewater Treatment Plant Operations Manual (R.J. Burnside, 2015).

Draft

Previous analysis has developed a future design basis in terms of raw wastewater flows and loadings for the Grand Valley WPCP under three future scenarios:

- Scenario I: Full completion of planned residential developments to an ADF of 1,279 m^3/d;
- Scenario II: A 15% increase above the current C of A rated average day flow (ADF) (1,430 m^3/d); and,
- Scenario III: A 25% increase above the current C of A rated ADF (1,555 m^3/d).

Stress testing was carried out on the secondary clarifiers and tertiary filters to simulate projected peak hour and maximum day flows conditions anticipated when the plant is operated under Scenario III flows and loads. These conditions are presented in Section 2.2.1.

2.2 Detailed Description of Testing Methodology

As previously noted, the two secondary clarifiers at the Grand Valley WPCP have identical dimensions and therefore it is assumed they have equal treatment capacities. Stress testing was conducted on only one secondary clarifier, which was assumed to be representative of the performance of both secondary clarifiers.

Similarly, since the existing tertiary filters have identical dimensions and configurations, it is assumed that the capacity of each filter is equal. As such, stress testing focused on evaluating the performance of two tertiary filters.

Operation of the UV disinfection system was not modified during the stress testing program. Instead, samples of secondary clarifier and tertiary filter effluent were collected over the duration of each testing day. The performance of the UV disinfection system was evaluated by taking UVT measurements of secondary clarifier and tertiary filter effluent samples during the stress test and, comparing the observed UVT to the design UVT.

Field work was carried out over three days in July, 2016. A summary of field activities is presented in Table 2.2.

Table 2.2 Summary of Field Activities

Date	Testing Day	Processes Tested	Testing Conditions
July 12	Day 1	Set up, preparation, and baseline testing	
July 13	Day 2	Secondary Clarifiers, Tertiary Filters and UV Disinfection	Peak Hour Flow
July 18	Day 3	Secondary Clarifiers, Tertiary Filters and UV Disinfection	Maximum Day Flow

Detailed descriptions of how target flows were achieved, and the sampling and monitoring program carried out during the performance testing was included in the Secondary Clarifier and Tertiary Filter Stress Testing Protocol (XCG, 2016). A copy of the protocol is included in Appendix A. Brief details of the target flows and sampling program are included in subsequent subsections.

2.2.1 Target Operating Conditions

For purposes of this test, target peak hour and maximum day flow rates were estimated using the following assumptions:

- Proposed Scenario III future flows (XCG, 2015);
- Future storm tank overflow operation to provide sufficient volume to equalize two days of peak flows; and,
- Peak flow event characteristics similar to a historical peak flow event available from plant records.

Based on the above assumptions, the future projected maximum day flow (MDF) and peak hour flow (PHF) to secondary treatment are approximately 6,250 m^3/d and 6,500 m^3/d, respectively. As only half of the plant capacity was tested, the target MDF and PHF for purposes of this Stress Test were 3,125 m^3/d and 3,250 m^3/d, respectively. A summary of test target conditions, including surface overflow rates (SOR), solids loading rates (SLR), and filtration rates is given in Table 2.3. UVT measurements of secondary clarifier and tertiary filter effluent samples were taken for the duration of the stress testing period to evaluate the capacity of the UV system.

Draft

Table 2.3 Summary of Target Test Conditions

Test Condition	Surface Overflow Rate ($m^3/m^2·d$)	Solids Loading Rate ($kg/m^2·d$)	Filtration Rate ($L/m^2·s$)
Test Target	43 (1)	210 (2)	4.0 (1)
Typical Design (3)	37	170	3.3

Notes:
1. Based on target peak hour flows.
2. Based on target maximum day flows
3. From Design Guidelines for Sewage Works (MOE, 2008). For an extended aeration activated sludge process with nitrification and chemical phosphorus removal.

Adequate flow from the Emma St. SPS was not available to achieve the target MDF and PHF for the Stress Test. As such, prior to Day 2 and Day 3 of the Stress Test, the offline aeration tank and storm tank were filled with sufficient supplementary volume for purposes of testing that day. Plant operators were responsible for filling the offline aeration tank was with raw wastewater and the storm tank with potable water. Supplemental volume was returned to the flow split chamber immediately upstream of the aeration tanks using temporary pumps and hoses.

2.2.2 Process Monitoring and Sampling

A brief description of the monitoring program during the Stress Test is as follows:

- An automatic sampler was configured to collect effluent samples from the test clarifier and test filters. On Day 1 and Day 3, samples were collected every 15 minutes and combined to form 1 hour composite samples. On Day 2, samples were collected every 15 minutes and combined to form 30 minute composite samples. Each sample was analyzed for total suspended solids (TSS), total phosphorus (TP), orthophosphate, turbidity, and UVT.
- Mixed liquor was collected once (Day 1) and once per hour (Days 2 and 3) and analyzed for mixed liquor suspended solids (MLSS). A 30 minute settling test on the mixed liquor was conducted once on Day 2 and Day 3. Results from the settling test were used to calculate the sludge volume index (SVI).
- Sludge blanket height in the secondary clarifier was monitored using a sludge judge at three measurement points along the radius of the test secondary clarifier (i.e. exterior, middle, interior). Approximate locations for the three measurement points are shown in Figure 2.1.
- All processes were monitored continually for hydraulic limitations.

Additional details regarding the sampling and monitoring program are included in Appendix A.

Draft

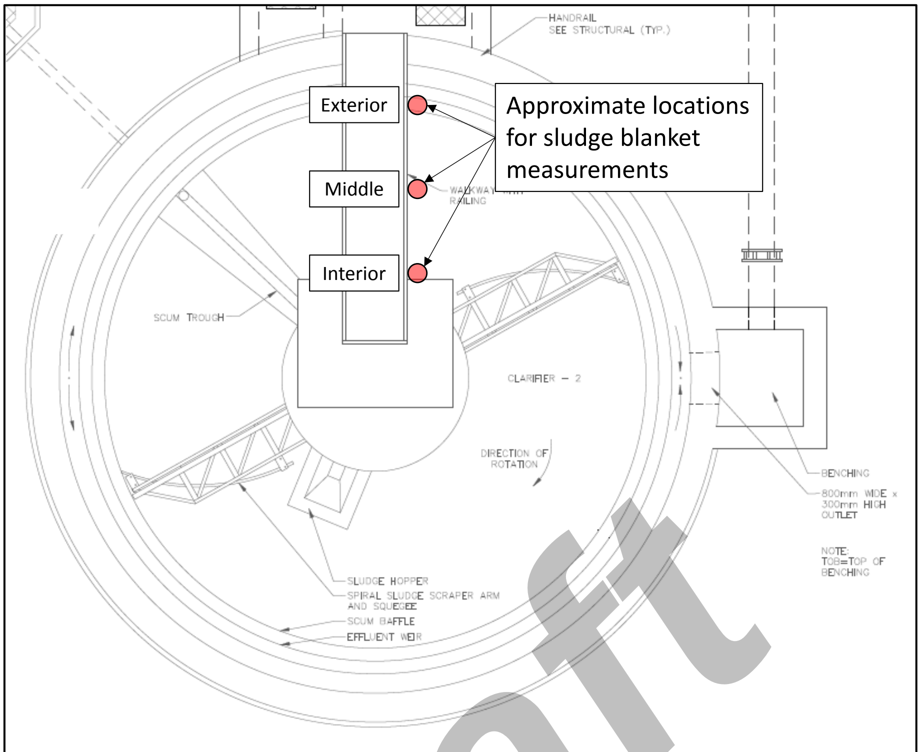


Figure 2.1 Locations for Sludge Blanket Measurements

2.2.3 Target Alum Dosage

The Grand Valley WPCP doses alum at the aeration tank effluent to precipitate phosphorus and control effluent phosphorus concentrations. The historical target alum dosage was 70 mg/L, which is less than the typical dosage rate of 110 mg/L to 225 mg/L as alum (MOE, 2008). The target alum dosage for purposes of this test was equal to the average historical alum dosage (70 mg/L). Plant operators were responsible for adjusting the alum dosage pumps based on the measured effluent of the plant.

During the testing period, it was discovered that only one alum pump could be used to deliver alum at the dosage location (aeration tank effluent), and that duty and standby pumps could not be used simultaneously. As per the plant C of A, the capacity of the alum dosing pump is approximately 12.0 L/hr which restricts the maximum alum dose to approximately 173 kg/d. As such, operational restrictions at the plant limited the alum dose to approximately 55 mg/L at target conditions.

2.2.4 Return Activated Sludge

There are three return activated sludge (RAS) pumps at the Grand Valley WPCP (two duty and one standby). The capacity of each pump is 1,244 m^3/d, giving a total RAS capacity of 200% of the existing C of A rated ADF. For the duration of the testing period, RAS pumps were set to approximately 90% of the target ADF (700 m^3/d).

Draft

3. RESULTS

3.1 Day 1 - Baseline Testing

The purpose of baseline testing was to evaluate the secondary clarifier and tertiary filter effluent quality immediately prior to the stress test at current average day flows. One secondary clarifier and two tertiary filters were online during the baseline sampling period. As previously discussed, the baseline sample consisted of four (4) discrete samples collected at 15 minute intervals and combined into one single composite sample. A summary of sample results is presented in Table 3.1. For comparison, the historical average from available plant data (2012 to May 2016) is also presented.

Table 3.1 Summary of Baseline Sample Results

Parameter	Secondary Clarifier Effluent	Tertiary Filter Effluent	Historical Final Effluent	C of A Effluent Requirements	
				Objectives	Limits
Total Suspended Solids (1)	7.0 mg/L	2.0 mg/L	3.4 mg/L	8.0 mg/L	10.0 mg/L
Total Phosphorus (1)	0.18 mg/L	0.085 mg/L	0.076 mg/L	0.13 mg/L	0.15 mg/L
Orthophosphate (2)	0.2 mg/L	0.12 mg/L	-	-	-
Turbidity (2)	2.8	1.3	-	-	-
UVT (2)	85.6	88.2	-	-	-

Notes
1. As measured by an accredited laboratory.
2. As measured onsite by XCG.

The following observations can be made from results presented in the above table:

- Tertiary filters improved the effluent quality as measured by all considered parameters.
- Both tertiary effluent TSS and TP concentrations measured during the baseline testing are comparable to the final effluent TSS and TP concentrations observed over the historical period.
- Baseline UVT measurements are significantly greater than the design minimum UVT (55%).
- Onsite orthophosphate concentrations were greater than TP concentrations measured at the accredited laboratory, in spite of the fact that orthophosphate concentrations should always be less than or equal to TP concentrations for a given sample. Given the low measured concentrations of both TP and orthophosphate, it is likely this is due to anticipated variability as concentrations approach the method detection limit (MDL) of the test methods. For the purposes of this study, it was assumed that reported TP concentrations are accurate and that almost all remaining phosphorus is soluble.

Draft

3.2 Day 2 - Peak Hour Flow Testing

The purpose during Day 2 of testing was to incrementally increase flow over one hour periods to evaluate the hydraulic capacity of the secondary clarifier. Testing took place on July 13, 2016 from approximately 9:00 am to 12:45 pm.

During testing, mixed liquor suspended solids (MLSS) concentrations decreased from approximately 5,300 mg/L to 4,400 mg/L in Aeration Tank 1 and from approximately 5,000 mg/L to 4,300 mg/L in Aeration Tank 2, indicating that mixed liquor was being transferred to the test clarifier during the stress testing.

To evaluate sludge settleability, a 30 minute settling test was conducted once during the peak hour flow test and results were used to calculate the sludge volume index (SVI). Mixed liquor concentrations were adjusted as required for purposes of calculating the SVI. One settling test was conducted for each aeration tank, and the calculated sludge settleability was assumed to be representative for the duration of the peak hour testing period. Results are summarized in Table 3.2.

Table 3.2 Summary of Settleability Tests

	Aeration Tank 1	Aeration Tank 2
Settled Volume (mL)	270	270
Estimated SVI (mL/g)	54	58

As presented, estimated SVIs for Aeration Tank 1 and Aeration Tank 2 are 54 mL/g and 58 mL/g, respectively. SVIs less than 100 mL/g are desired, and indicate a sludge with good settleability (Metcalf & Eddy, 2003). The RAS flow rate was maintained at approximately 700 m^3/d for the duration of the test period.

3.2.1 Measured Flows and Loading Rates

Surface overflow rates (SOR) from the test secondary clarifier were recorded by a velocity-area (VA) flow meter, installed by XCG on July 12, 2016. The solids loading rate (SLR) to the test secondary clarifier was estimated from the measured overflow rate, RAS flow rate, and the measured MLSS concentration. SLR calculations account for observed changes in MLSS concentrations over the test period. Filtration rates were estimated using the measured clarifier overflow rate given the tertiary filter surface area.

Figure 3.1 and Figure 3.2 show the calculated secondary clarifier SOR and SLR, respectively, for the duration of Day 2 of testing. Figure 3.3 and Figure 3.4 shows the estimated tertiary filter filtration rate and solids loading rate, respectively. Target rates are also shown on all figures where applicable.

For the duration of the testing period, secondary clarifier effluent and tertiary filter effluent channels were continuously visually monitored for hydraulic limitations and poor effluent quality (turbid).

At approximately 11:30 am, a third tertiary filter was brought online as a result of visual observations of solids in the tertiary effluent stream. The additional tertiary filter had an impact on the filtration rates (sudden decrease at approximately 11:30

Draft

am, as shown in Figure 3.3), however SOR and SLR values of the test secondary clarifier were not affected.

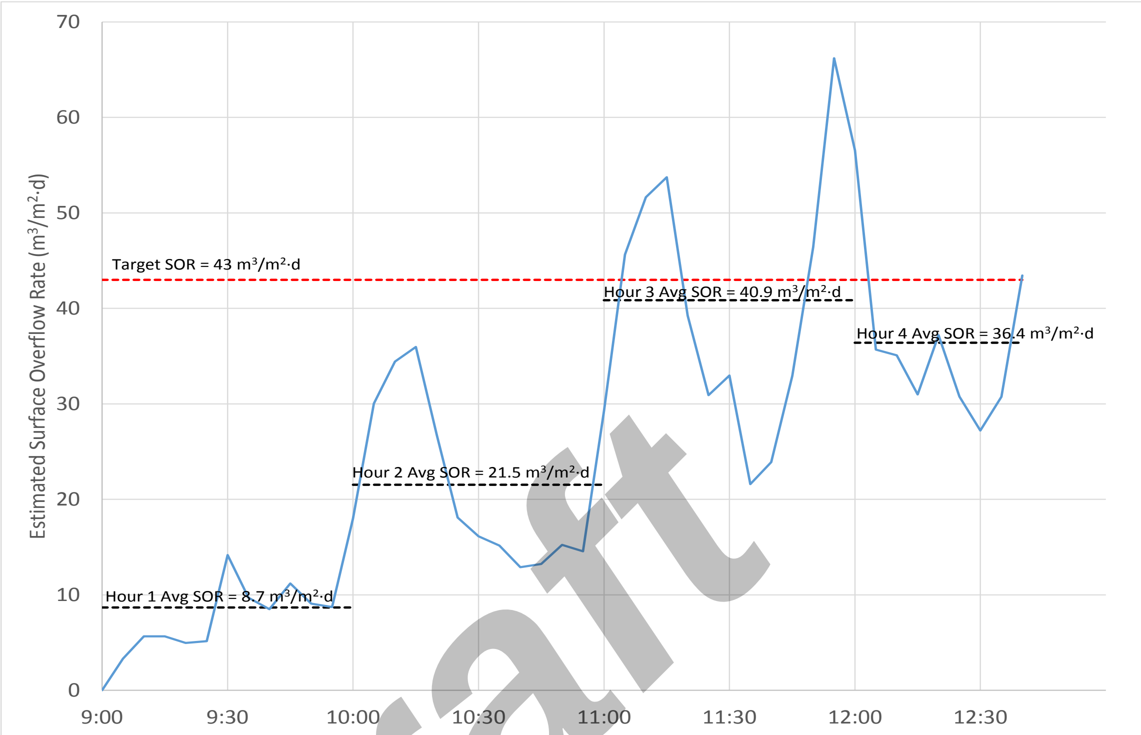


Figure 3.1 Calculated SOR for Test Secondary Clarifier (Day 2)

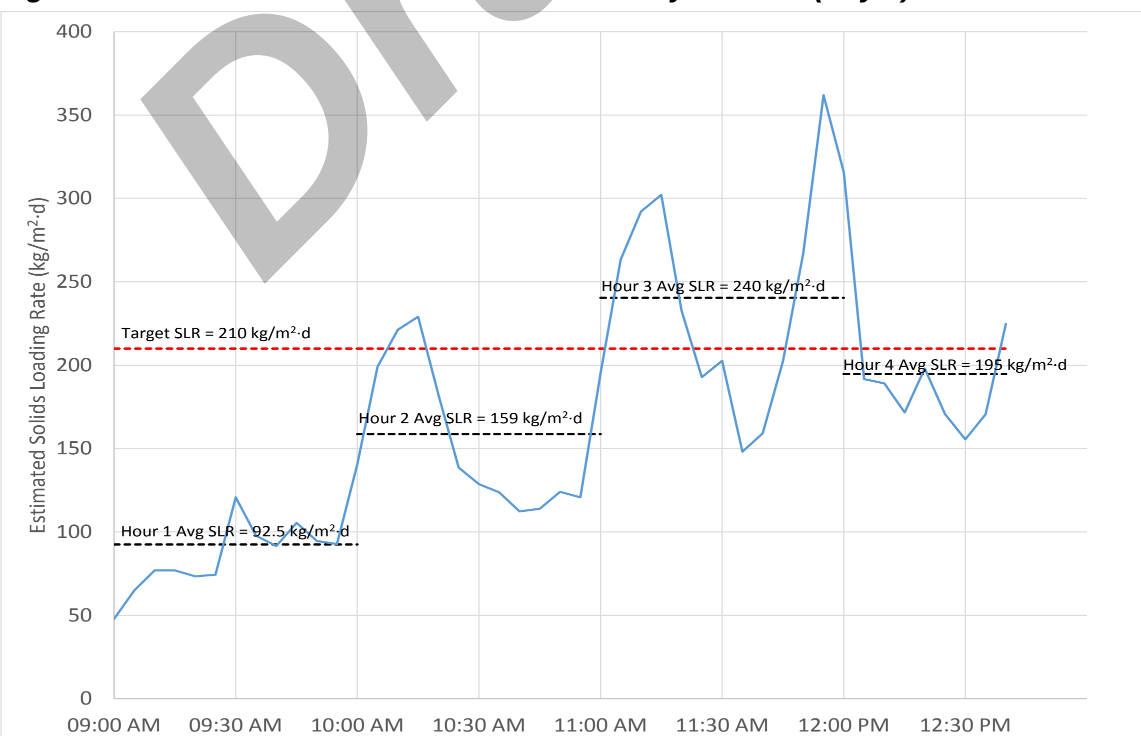


Figure 3.2 Calculated SLR for Test Secondary Clarifier (Day 2)

Draft

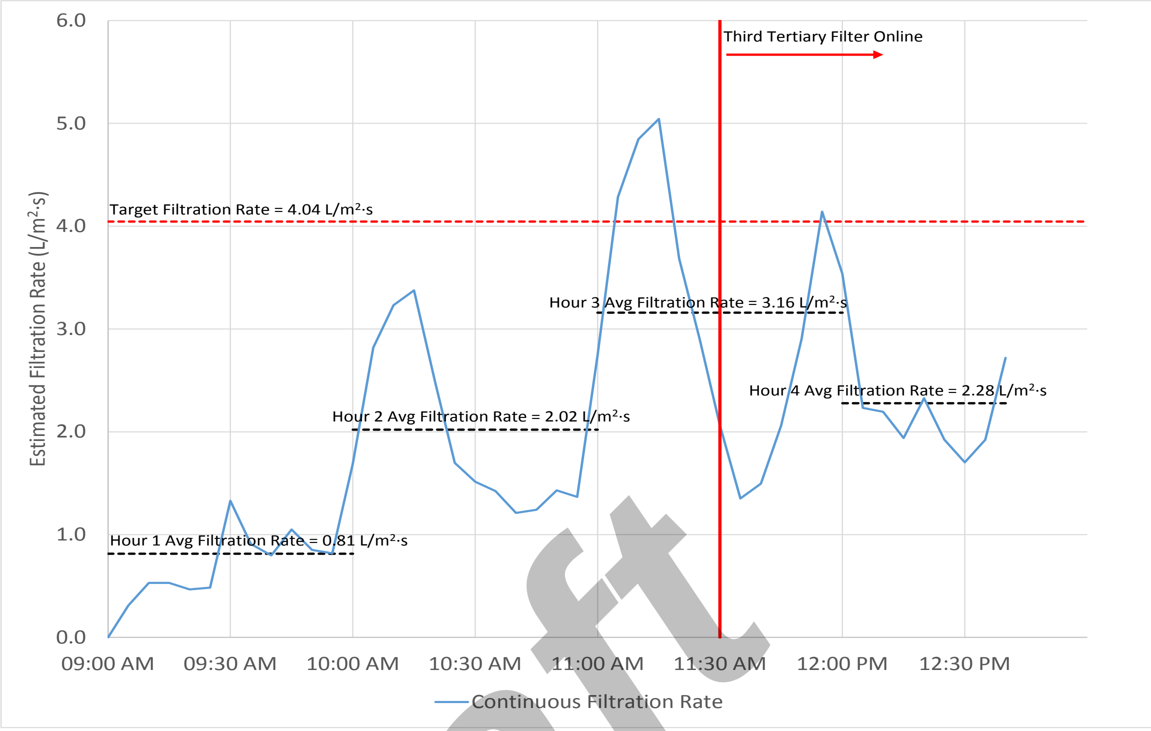


Figure 3.3 Calculated Filtration Rate for Test Tertiary Filters (Day 2)

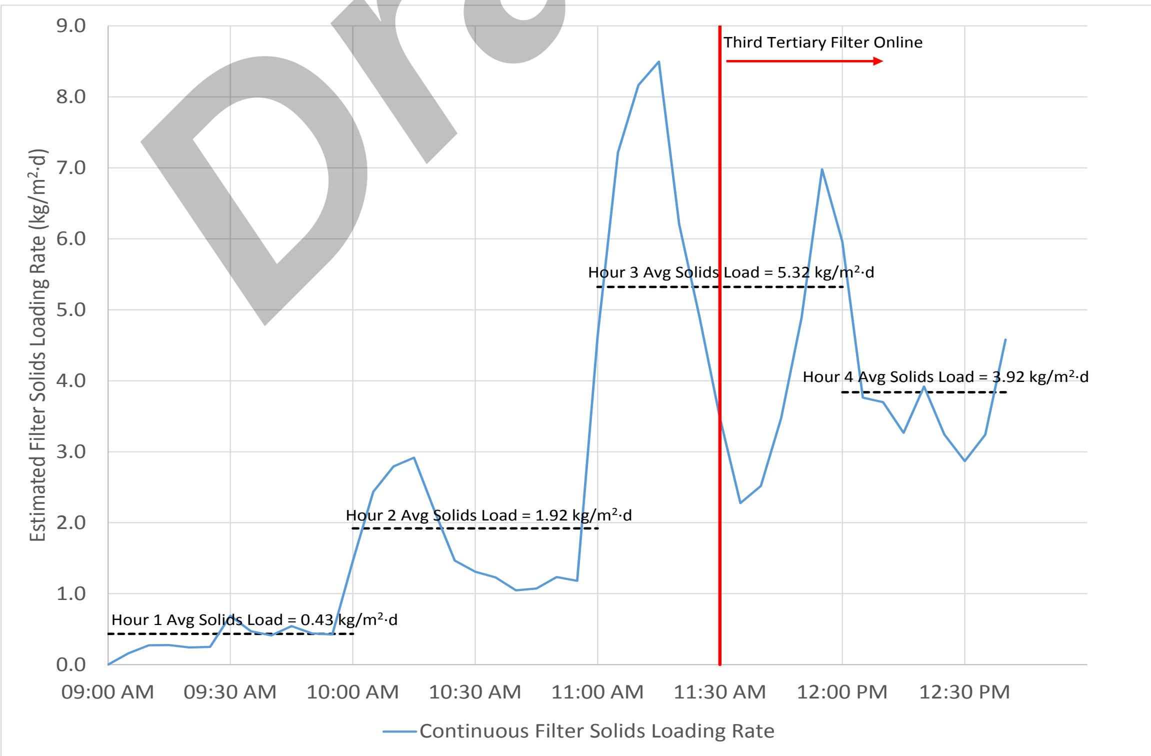


Figure 3.4 Calculated Filter Solids Loading Rate for Test Tertiary Filters (Day 2)

Draft

The test reached peak flows between 11:00 am and 12:00 pm. 1-hour average SOR, SLR, and filtration rates achieved during this period are summarized in Table 3.3.

Table 3.3 Summary Day 2 Peak Hour Operating Conditions

Test Unit	Value	Target
Secondary Clarifier		
SOR ($m^3/m^2 \cdot d$)	40.9	43
SLR ($kg/m^2 \cdot d$)	240	210
Tertiary Filter		
Filtration Rate ($L/m^2 \cdot s$)	3.16 (1)	4.04
Solids Loading Rate ($kg/m^2 \cdot d$)	5.32 (1)	-

Notes:
1. Estimated filtration rate average between 11:00 am and 12:00 pm. Average includes impact of third filter, which was brought online at 11:30 am.

The following observations can be made from results presented in Figure 3.1, Figure 3.2, Figure 3.3, Figure 3.4, and Table 3.3:

- With respect to the test secondary clarifier, the SOR and SLR reached during peak hour flow was comparable to targets established for this test.
- With respect to the filtration rate and filter solids loading rate during testing, increased solids concentrations in the tertiary effluent stream were visually observed. As a result, an additional tertiary filter was brought online prior to reaching sustained peak hour flows. As such, achieved filtration rates were below target filtration rates.

3.2.2 Measured Clarifier and Filter Performance

As previously discussed, samples of secondary clarifier and tertiary filter effluent were collected for the duration of peak hour testing. To evaluate the performance of the secondary clarifiers and tertiary filters, each sample was sent to an accredited laboratory for TSS and TP measurements. In addition, samples were processed onsite for orthophosphate, turbidity, and UVT measurements.

Figure 3.5 shows the measured TSS concentrations over the duration of Day 2. Similarly, Figure 3.6 shows the measured TP and orthophosphate concentrations. C of A final effluent objective and limit concentrations are also shown on each figure. It is important to note that current C of A effluent limits are enforced on a monthly average basis, and effluent samples are composited over a 24-hour period. As such, objectives and limits have been included for reference only, and results from samples collected during this test do not indicate compliance or exceedance with the existing C of A. Figure 3.7 and Figure 3.8 show secondary effluent and tertiary effluent measurements for turbidity and UVT, respectively, over the duration of Day 2 of testing.

For reference, the approximate time when the third tertiary filter was brought online is indicated in all figures.

Draft

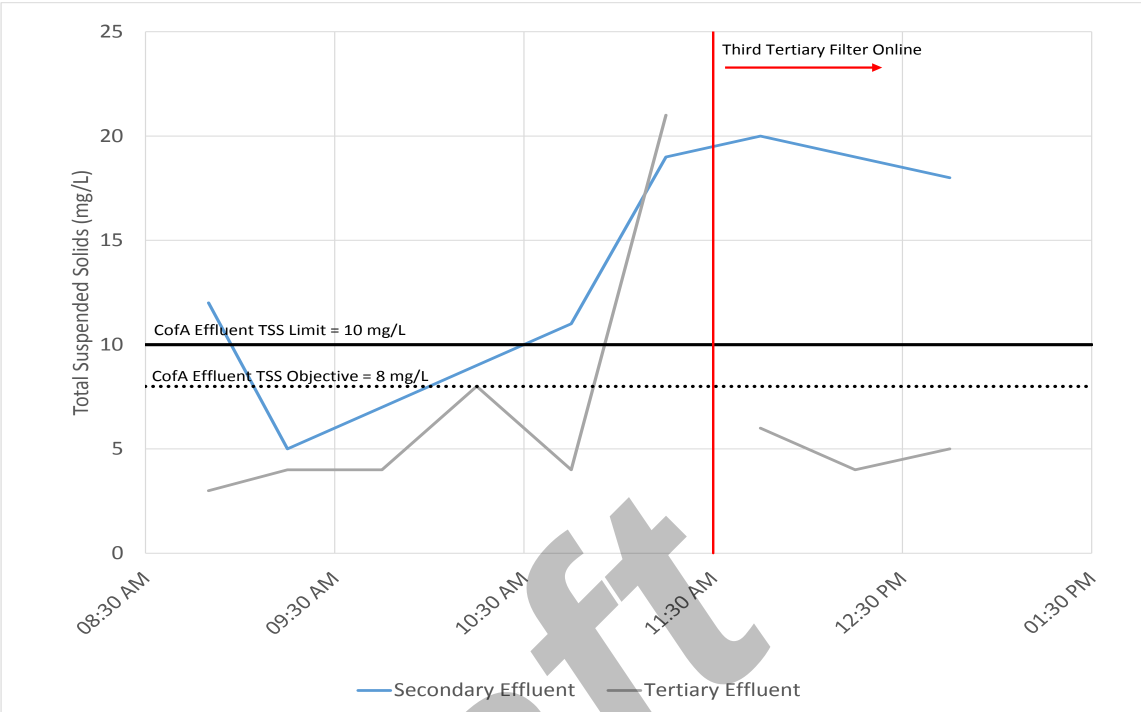


Figure 3.5 Measured Secondary Clarifier and Tertiary Filter Effluent TSS Concentrations (Day 2)

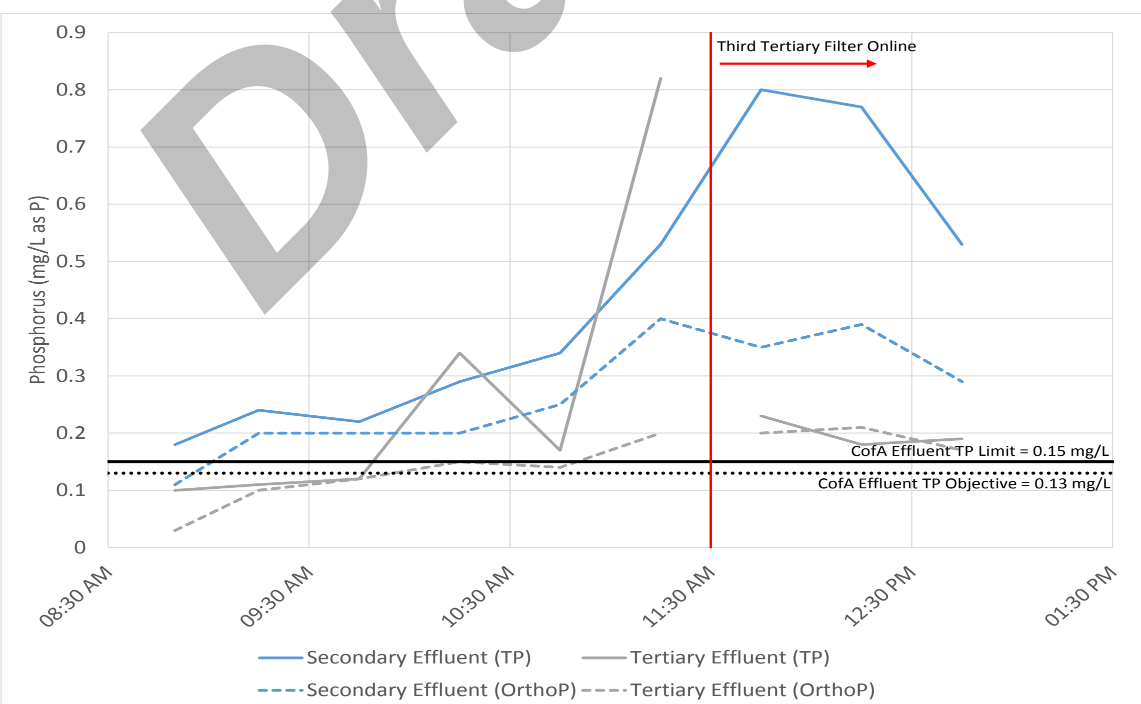


Figure 3.6 Measured Secondary Clarifier and Tertiary Filter Effluent Total Phosphorus and Orthophosphate Concentrations (Day 2)

Draft

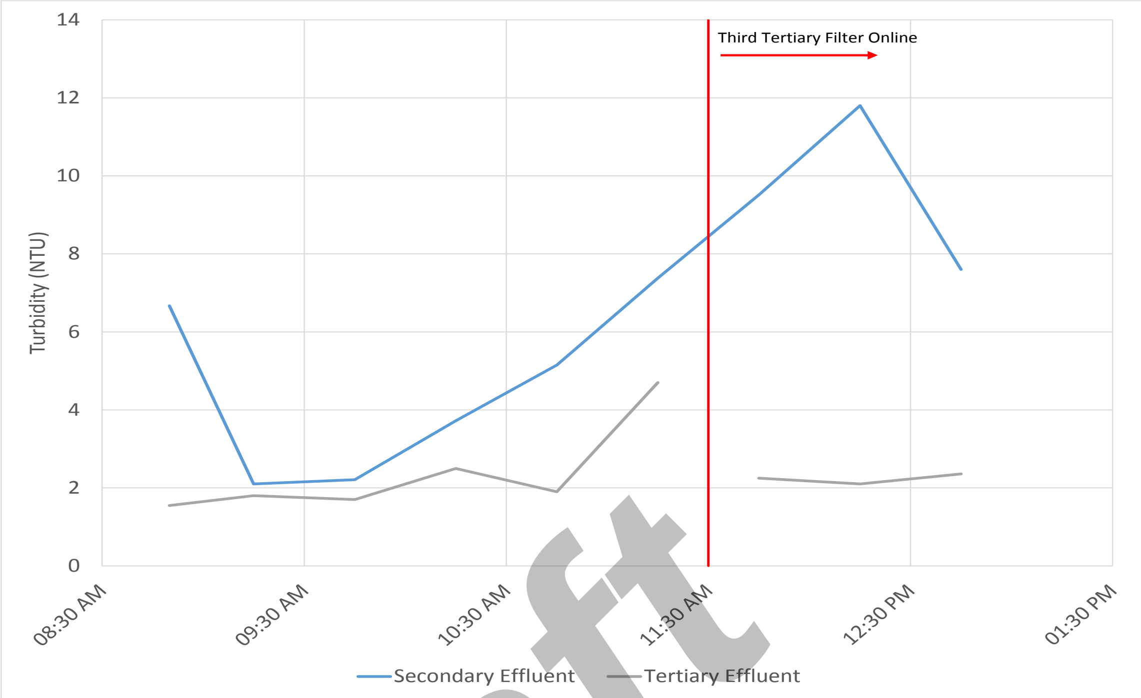


Figure 3.7 Measured Secondary Clarifier and Tertiary Filter Effluent Turbidity (Day 2)

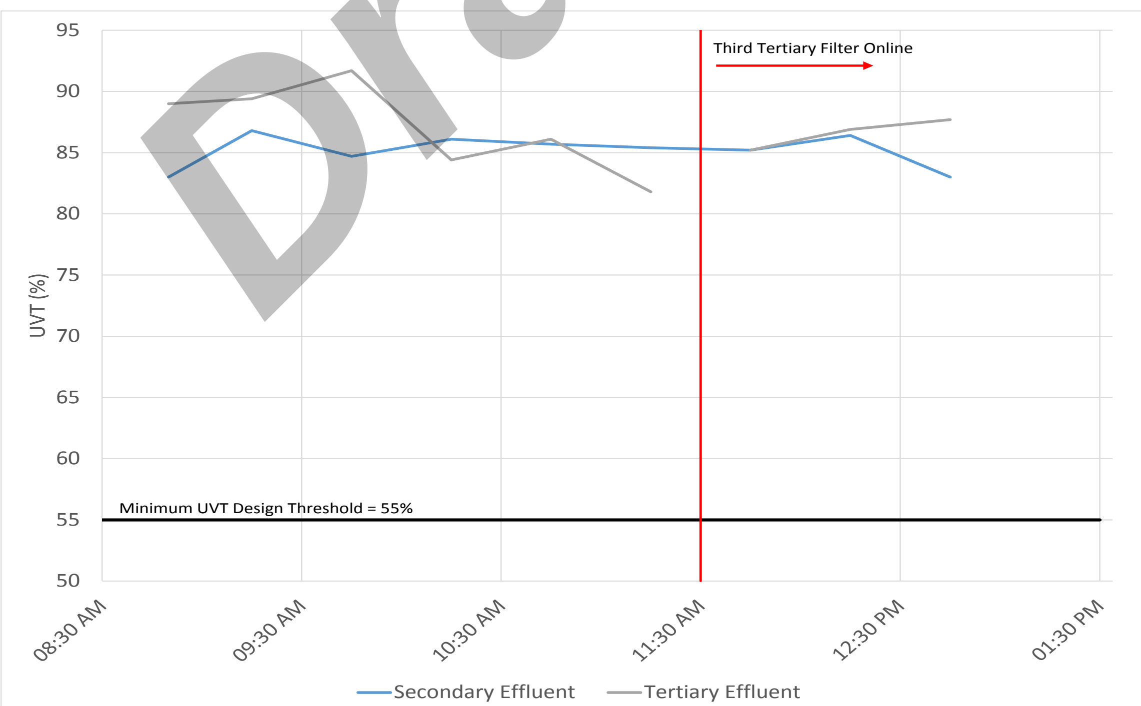


Figure 3.8 Measured Secondary Clarifier and Tertiary Filter Effluent UVT (Day 2)

Draft

Based on results presented in Figure 3.5, Figure 3.6, Figure 3.7, and Figure 3.8, the following conclusions can be drawn about the peak hour flow testing at the Grand Valley WPCP.

- Secondary effluent TSS concentrations generally rose over the duration of the test. Effluent TSS concentrations at peak flows were stable and consistently less than 20 mg/L. This is comparable to the expected secondary clarifier effluent TSS concentration from an extended aeration plant with phosphorus removal (15 mg/L) (MOE, 2008).
- Secondary effluent TP, orthophosphate, and turbidity measurements generally rose over the duration of the test. During peak flows, secondary effluent TP concentrations peaked at approximately 0.8 mg/L. Secondary effluent TP concentrations from an extended aeration plant with phosphorus removal are typically less than 1.0 mg/L (MOE, 2008).
- During peak flows, secondary effluent orthophosphate concentrations represented approximately 50% of TP concentration measurements.
- Tertiary effluent TSS and TP concentrations generally rose between 9:00 am and 11:30 am, at which point the third tertiary filter was brought online. The peak TSS concentration of 21 mg/L was measured from samples collected between 11:00 am and 11:30 am and was comparable to secondary effluent TSS concentrations over the same period, indicating the tertiary filter was likely overloaded with respect to the filtration rate or solids loading rate. The average filtration rate during this period (11:00 am to 11:30 am) was 3.65 $L/m^2 \cdot s$, and the average filter solids loading rate was 6.15 $kg/m^2 \cdot d$.
- Upon bringing the third tertiary filter online, tertiary effluent TSS concentrations fell and stabilized below the C of A objective concentration. Tertiary effluent TP concentrations also fell, and stabilized at approximately 0.2 mg/L. The estimated filtration and filter solids loading rates during this period of stable operation were 2.39 $L/m^2 \cdot s$ and 4.03 $g/m^2 \cdot d$, respectively.
- Orthophosphate concentrations in the tertiary effluent generally rose over the duration of the testing period. Tertiary effluent samples collected during the period of three filter operation showed comparable concentrations of TP and orthophosphate, indicating filters had removed almost all particulate phosphorus. Elevated concentrations of orthophosphate are likely related to alum dosing restrictions at the plant. Further TP removal may be possible by optimizing the alum dose.
- Secondary effluent and tertiary effluent UVT measurements were relatively stable over the duration of the test and consistently exceed 80%.

3.2.3 Secondary Clarifier Solids Blanket

Sludge height measurements were taken regularly over the duration of the test period. Measurements were taken at three locations along the walkway of the test clarifier to measure blanket height at the exterior, middle, and interior of the clarifier. Approximate locations for sludge blanket measurements is previously shown in Figure 2.1 Sludge blanket height measurements over the duration of the testing period is

Draft

shown in Figure 3.9. The clarifier side water depth is 4.2 m, and is represented by the top of Figure 3.9.

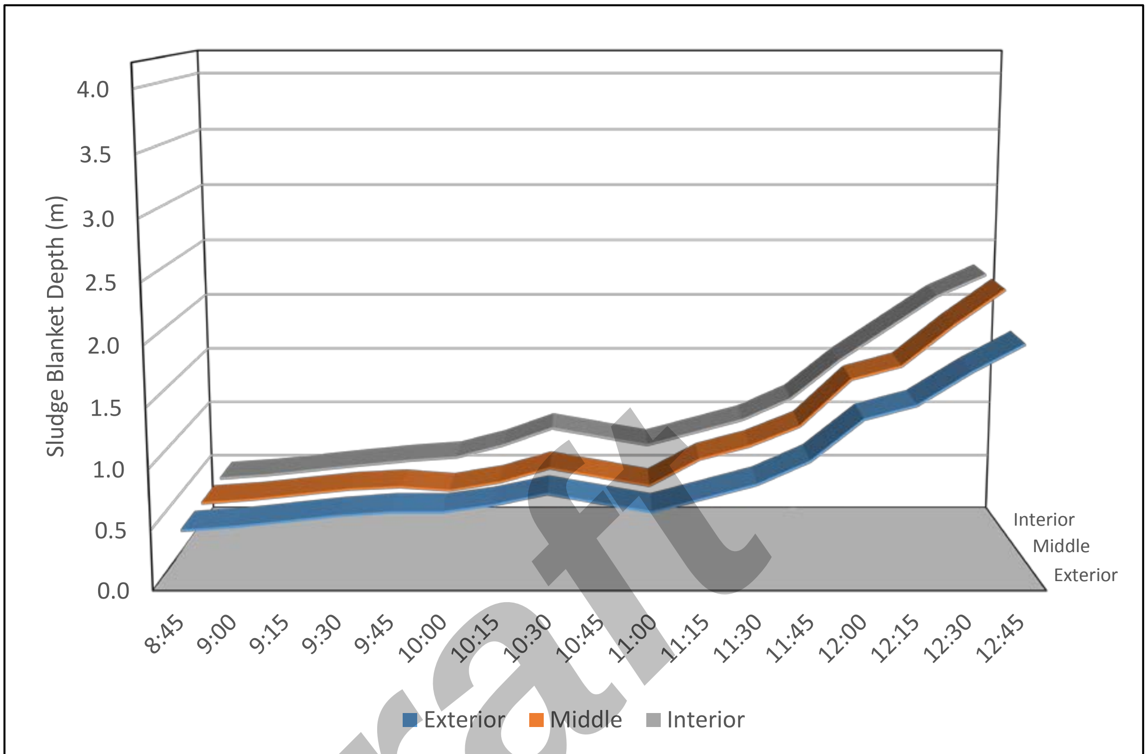


Figure 3.9 Secondary Clarifier Sludge Blanket Profile (Day 2)

Draft

The blanket depth ranged from approximately 450 mm (1.5 feet) at first measurement to approximately 2,300 mm (7.5 feet) at the middle and interior measurement points at the end of the test. From the first measurement until approximately 11:00 am, the measured sludge blanket height was relatively stable, as only minor increases to the blanket height were observed. Between 11:00 am and 12:45 pm, the measured sludge blanket height increased steadily. Day 2 of the stress test was stopped at 12:45 pm at sludge blanket heights of approximately 6.5 feet (2.0 m), 7.5 feet (2.3 m), and 7.5 feet (2.3 m) at the exterior, middle, and interior measurement points, respectively. Although blanket washout did not appear imminent, the test was stopped due to operator concerns regarding the integrity of the secondary clarifier mechanical equipment at the elevated sludge blanket height.

3.2.4 Evaluation of Secondary Clarifier Performance - Day 2

Based on results presented in Figure 3.5 and Figure 3.6, secondary clarifier effluent concentrations of TSS and TP rose significantly during the testing period. During the peak hour flow period from 11:00 am to 12:00 pm (SOR = 40.9 $m^3/m^2\cdot d$) effluent concentrations remained stable and comparable to typical secondary clarifier effluent quality of an extended aeration treatment process (MOE, 2008). However, the secondary clarifier sludge blanket was observed to rise significantly during this period, indicating steady state operation was not achieved. The SLR during the peak flow period was calculated to be approximately 240 $kg/m^2\cdot d$, which was significantly greater than both the target SLR (210 kg/m^2·d) and a typical design SLR (170 kg/m^2·d). The peak estimated SLR is due, in part, to relatively high operating MLSS concentrations in the bioreactors. Despite rising sludge blanket levels, washout of the sludge blanket did not appear imminent. Results from Day 2 of testing suggest the peak hour capacity of the secondary clarifier is less than the peak hour SOR and SLR achieved.

Conversely, sample results collected between 10:00 am and 11:00 am indicate relatively stable sludge blanket levels and increasing but low concentrations of TSS and TP in the secondary effluent. The calculated SOR and SLR achieved during this period were 21.5 m^3/m^2·d and 159 kg/m^2·d, respectively. Results from Day 2 of testing suggest the peak SOR and SLR capacity of the secondary clarifier is greater than the rates achieved between 10:00 and 11:00 am.

3.2.5 Evaluation of Tertiary Filter Performance - Day 2

As presented in Figure 3.5 and Figure 3.6, TSS and TP concentrations in the tertiary effluent rose to peak concentrations of 21 mg/L and 0.82 mg/L, respectively, between 11:00 am and 11:30 am (the beginning of the peak hour flow period). The filtration rate and filter solids loading rate between 11:00 am and 11:30 am, estimated to be 3.65 L/m^2·s and 6.15 kg/m^2·d, respectively, represent overload conditions for the tertiary filter.

Conversely, during the period of three filter operation, tertiary effluent concentrations of TSS, TP, and orthophosphate were found to be stable. The estimated filtration rate and solids loading rate during this period of stable operation was estimated to be 2.40 L/m^2·s and 4.03 kg/m^2·d, respectively. Results from Day 2 suggest the hydraulic and solids loading capacity of the tertiary filters is greater than those estimated during the period of three filter operation.

3.2.6 Evaluation of Disinfection Performance - Day 2

The UV disinfection system at the Grand Valley WPCP was designed for a peak flow of 7,680 m^3/d at a UVT of 55%. Overflow from the existing storm equalization tank will flow directly to the UV disinfection system, thereby bypassing secondary treatment. Further, a tertiary filter bypass exists for peak flows in excess of tertiary filter capacity. As a result of these bypass streams, final plant effluent flow may be of lower quality relative to the tertiary effluent stream during peak flow events. In addition, because the UV disinfection system would be subject to the design peak flow through the filters as well contributions from these bypass streams, the design peak flow capacity of the UV disinfection system exceeds the design capacity of the tertiary filters.

Results presented in Figure 3.8 indicate that the measured secondary clarifier and tertiary filter UVT remained stable and consistently above the design UVT for the entirety of the testing period, even when both of these process were pushed beyond their treatment capacities.

In the fall of 2015, samples of the raw influent and tertiary effluent streams were collected from the Grand Valley WPCP. Samples were combined in different volumetric ratios, and the UVT of these combined samples was measured to determine

Draft

the potential impact from storm tank bypass flows on the UVT of the final effluent. Samples consisting of 100% tertiary effluent had a UVT of approximately 88%, comparable to results from baseline testing conducted for the stress test. Combined samples consisting of 40% raw influent or less (by volume) consistently measured a UVT greater than 55%. However, during a peak flow event, the storm tank bypass would make up significantly less than 40% of the effluent; in addition, when stressed, the secondary clarifiers and tertiary filters continue to produce a tertiary effluent with UVT > 80%.

Overall, these results indicate that even during wet weather event, the WPCP effluent would have a UVT > 55% and, therefore, this suggests that the capacity of the existing UV disinfection system is greater than its design peak flow capacity of 7,680 m^3/d.

3.3 Day 3 - Maximum Day Flow Testing

The purpose during Day 3 of testing was to maintain a target flow rate to simulate a maximum day flow event and evaluate the performance of the secondary clarifiers and tertiary filters. Testing took place on July 18, 2016 from approximately 8:30 am to 12:30 pm. During testing, mixed liquor suspended solids (MLSS) concentrations decreased from approximately 4,500 mg/L to 2,700 mg/L in Aeration Tank 1 and from approximately 4,300 mg/L to approximately 3,700 mg/L in Aeration Tank 2.

To evaluate sludge settleability, a 30 minute settling test was conducted once during the maximum day flow test and results were used to calculate the sludge volume index (SVI). Mixed liquor concentrations were adjusted as required for purposes of calculating the SVI. One settling test was conducted for each aeration tank, and the sludge settleability was assumed unchanged for the duration of the peak hour testing period. Results are summarized in Table 3.4.

Table 3.4 Summary of Settleability Tests (Day 3)

	Aeration Tank 1	Aeration Tank 2
Settled Volume (mL)	275	265
Estimated SVI (mL/g)	81	67

As presented, estimated SVIs for Aeration Tank 1 and Aeration Tank 2 are 81 mL/g and 67 mL/g, respectively. SVIs less than 100 mL/g are desired, and indicate a sludge with good settleability (Metcalf & Eddy, 2003). The return activated sludge (RAS) flow rate was maintained at approximately 700 m^3/d for the duration of the test period.

3.3.1 Measured Flows and Loading Rates

Surface overflow rates (SOR) from the test secondary clarifier were recorded by a velocity-area (VA) flow meter, installed by XCG on July 12, 2016. The solids loading rate (SLR) to the test secondary clarifier was estimated from the measured overflow rate, RAS flow rate, and the measured MLSS concentration. SLR calculations account for observed changes in MLSS concentrations over the test period. Filtration rates were estimated using the measured clarifier overflow rate given the tertiary filter surface area.

Draft

Figure 3.10 and Figure 3.11 show the calculated secondary clarifier SOR and SLR, respectively, for the secondary clarifier for the duration of Day 3 of testing. Figure 3.12 and Figure 3.13 shows the estimated tertiary filter filtration rate and tertiary filter solids loading rate, respectively. Target rates are also shown on all figures where applicable.

For the duration of the testing period, secondary clarifier effluent and tertiary filter effluent channels were continuously visually monitored for hydraulic limitations and for solids concentrations.

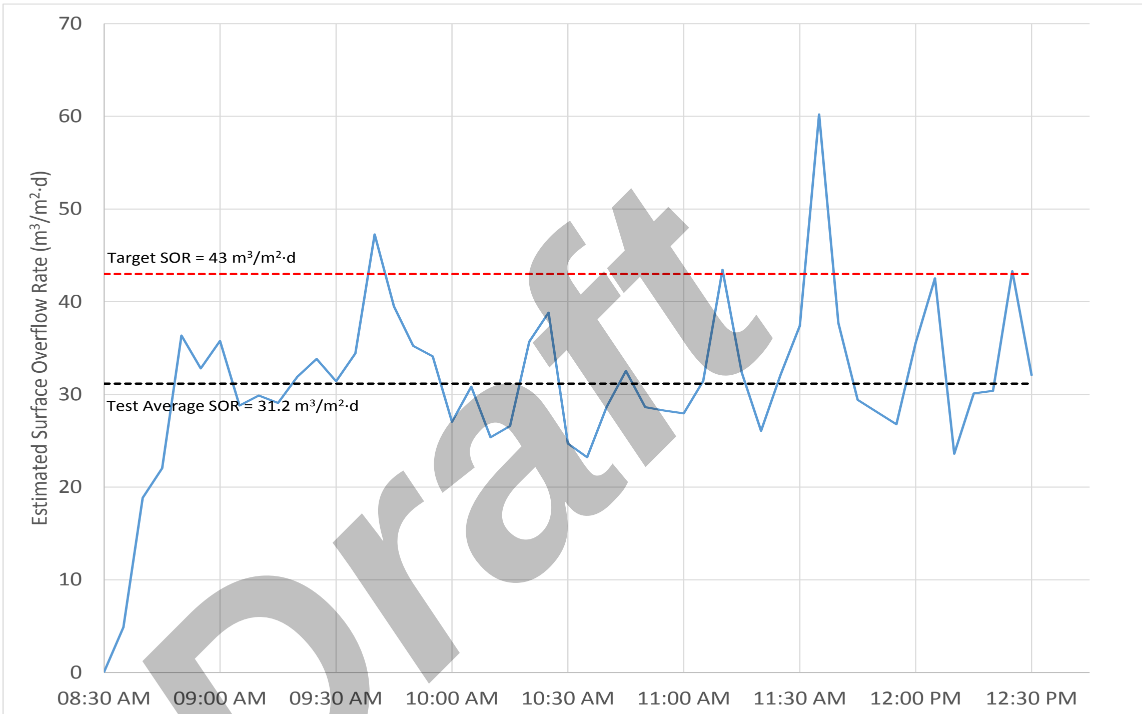


Figure 3.10 Calculated SOR for Test Secondary Clarifier (Day 3)

Draft

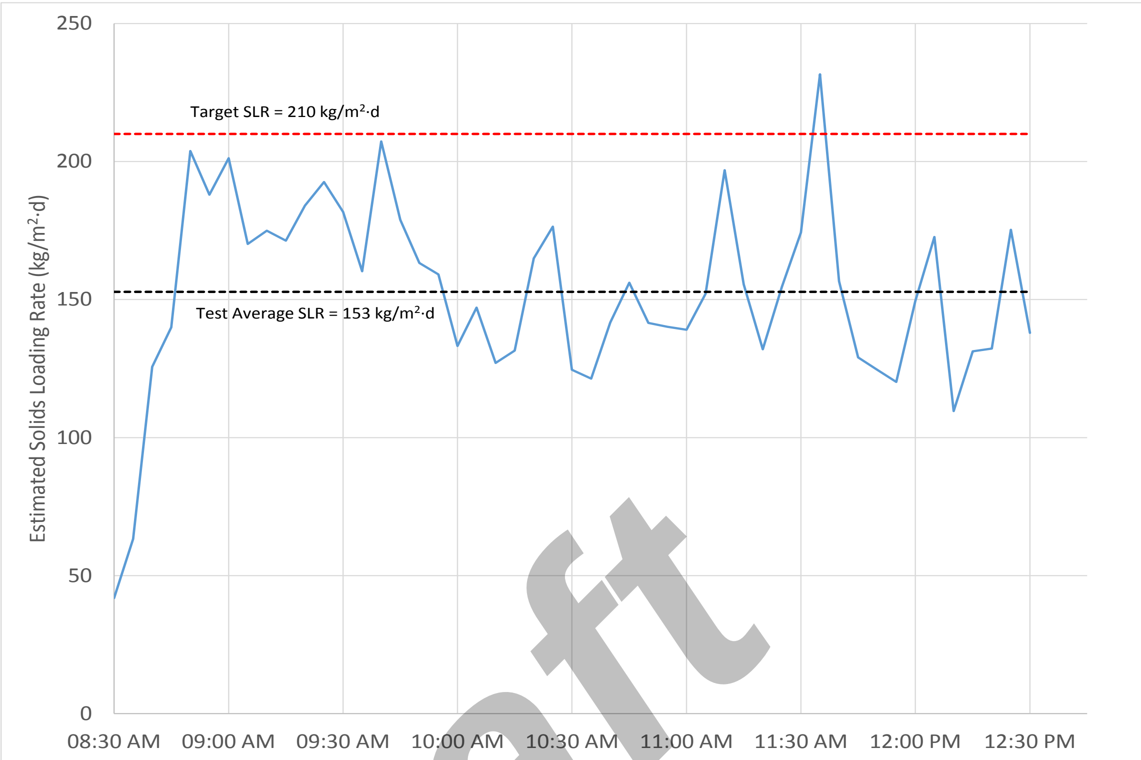


Figure 3.11 Calculated SLR for Test Secondary Clarifier (Day 3)

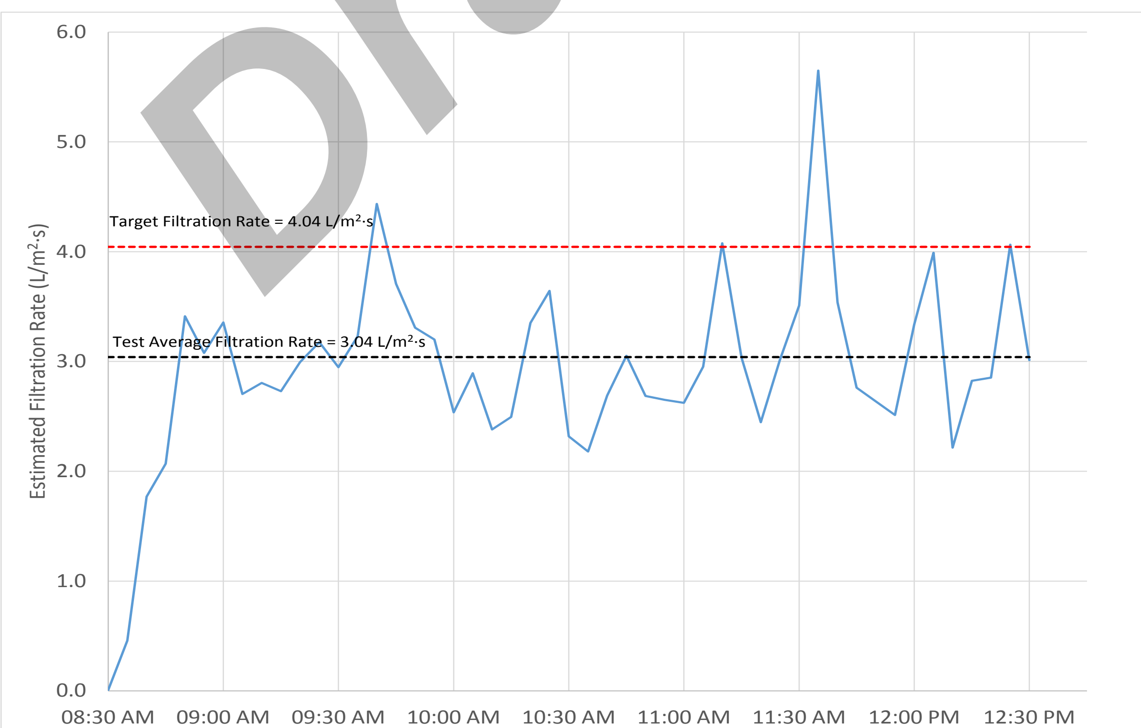


Figure 3.12 Calculated Filtration Rate for Test Tertiary Filter (Day 3)

Draft

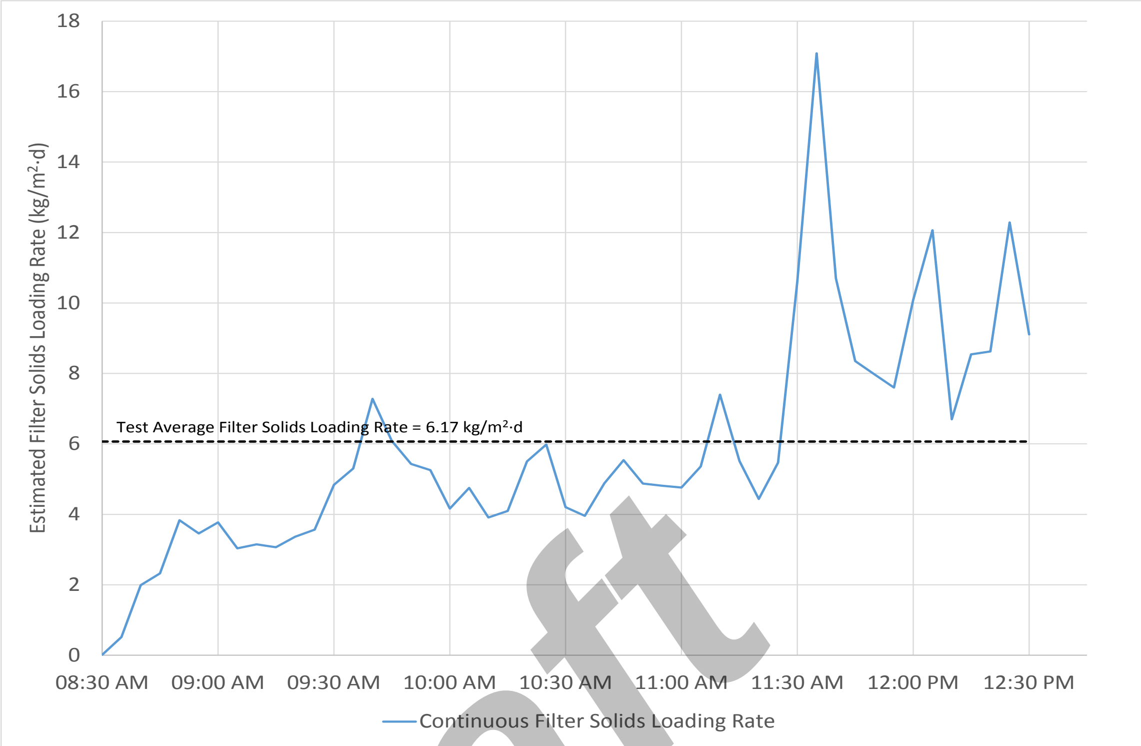


Figure 3.13 Calculated Filter Solids Loading Rate for Test Tertiary Filter (Day 3)

Test average SOR, SLR, and filtration rates achieved during this period are summarized in Table 3.5. Further, unlike Day 2 of testing, Day 3 required only two tertiary filters for the duration of the test. As such, peak hour filtration rates achieved during Day 3 exceed peak hour tertiary filtration rates achieved during Day 2 of testing. Peak filtration rates achieved during Day 3 are also presented in Table 3.5.

Table 3.5 Summary Day 3 Operating Conditions

Test Unit	Value	Target
Secondary Clarifier		
SOR ($m^3/m^2{\cdot}d$)	31.2	43
SLR ($kg/m^2{\cdot}d$)	153	210
Tertiary Filtration Rate ($L/m^2{\cdot}s$)		
Test Average	3.03	4.04
Peak Hour	3.30 (1)	
Tertiary Filter Solids Loading Rate ($kg/m^2{\cdot}d$)		
Test Average	6.17	-
Peak Hour	9.98	-

Notes:
1. Estimated filtration rate during peak hour flows from 11:30 am to 12:30 pm.

The following observations can be made from results presented in Figure 3.10, Figure 3.11, Figure 3.12, Figure 3.13, and Table 3.5:

- With respect to the test secondary clarifier, the average SOR and SLR achieved for the test duration were less than targets established for this test. This is, in part, due to variability in the influent flow from the Emma St. SPS and decreasing MLSS concentrations in the bioreactors over the duration of the test.
- With respect to the tertiary filters, average filtration rates achieved for the duration of the test were less than targets established for the test. This is, in part, due to the variability in influent flow from the Emma St. SPS. The peak filtration rate was estimated to be 3.30 $L/m^2{\cdot}s$, identical to both the C of A rated peak flow capacity and typical design peak flow rates for deep bed filters (MOE, 2008). The estimated tertiary filter solids loading rate was relatively consistent until approximately 11:30 am when a significant increase in the solids loading rate was observed due to an increase in the secondary clarifier effluent solids concentration.

3.3.2 Measured Clarifier and Filter Performance

As previously discussed, samples of secondary clarifier and tertiary filter effluent were collected for the duration of peak hour testing. To evaluate the performance of the secondary clarifiers and tertiary filters, each sample was sent to an accredited laboratory for TSS and TP measurements. In addition, samples were processed onsite for orthophosphate, turbidity, and UVT measurements.

Figure 3.14 shows the measured TSS concentrations over the duration of Day 3. Similarly, Figure 3.15 shows the measured TP and orthophosphate concentrations. C of A final effluent objective and limit concentrations are also shown on each figure. It is important to note that current C of A effluent limits are enforced on a monthly average basis, and effluent samples are composited over a 24-hour period. As such, objectives and limits have been included for reference only, and results from samples collected during this test do not indicate compliance or exceedance with the existing C of A. Figure 3.16 and Figure 3.17 show secondary effluent and tertiary effluent measurements for turbidity and UVT, respectively, over the duration of Day 3 of testing.

During regular plant operation, plant staff have observed periodic accumulation of solids in the tertiary effluent channel. Staff indicated that the channel is regularly cleaned to remove the solids, however they were not able to clean the channel prior to the stress test. Beginning at approximately 9:30 am, plant staff initiated a cleaning of the tertiary effluent channel. As a result, samples collected between approximately 9:30 am and 10:00 am reported elevated concentrations of TSS and TP. These samples were not representative of the testing conditions and were therefore excluded from this analysis.

Draft

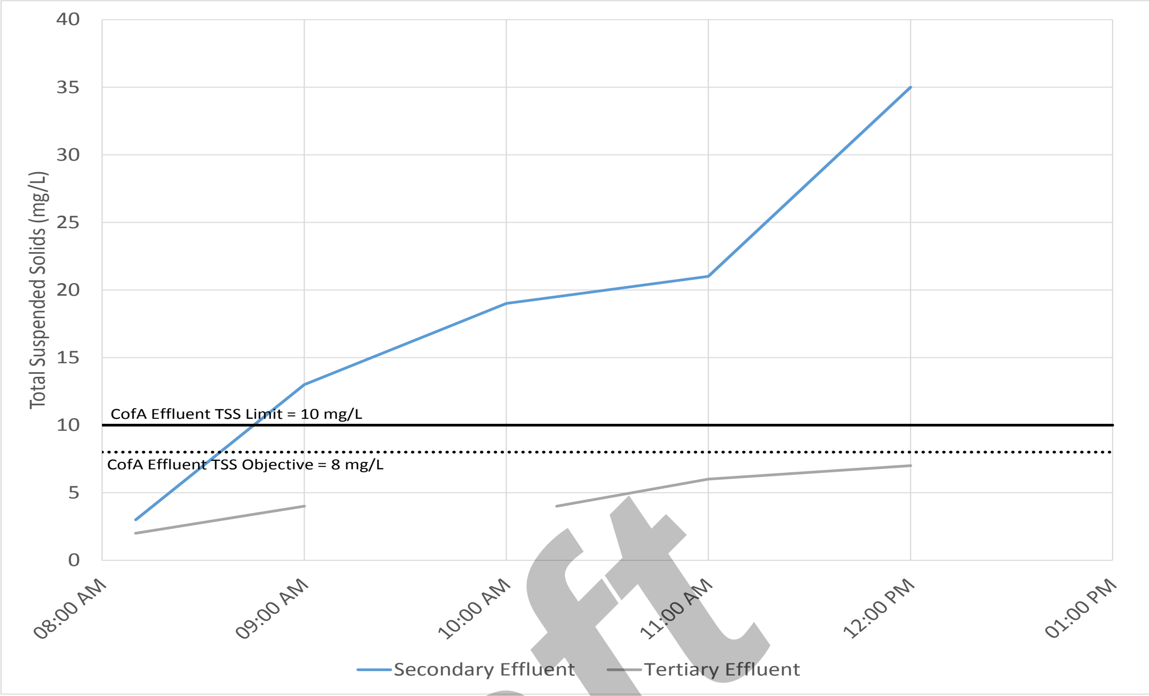


Figure 3.14 Measured Secondary Clarifier and Tertiary Filter Effluent TSS Concentrations (Day 3)

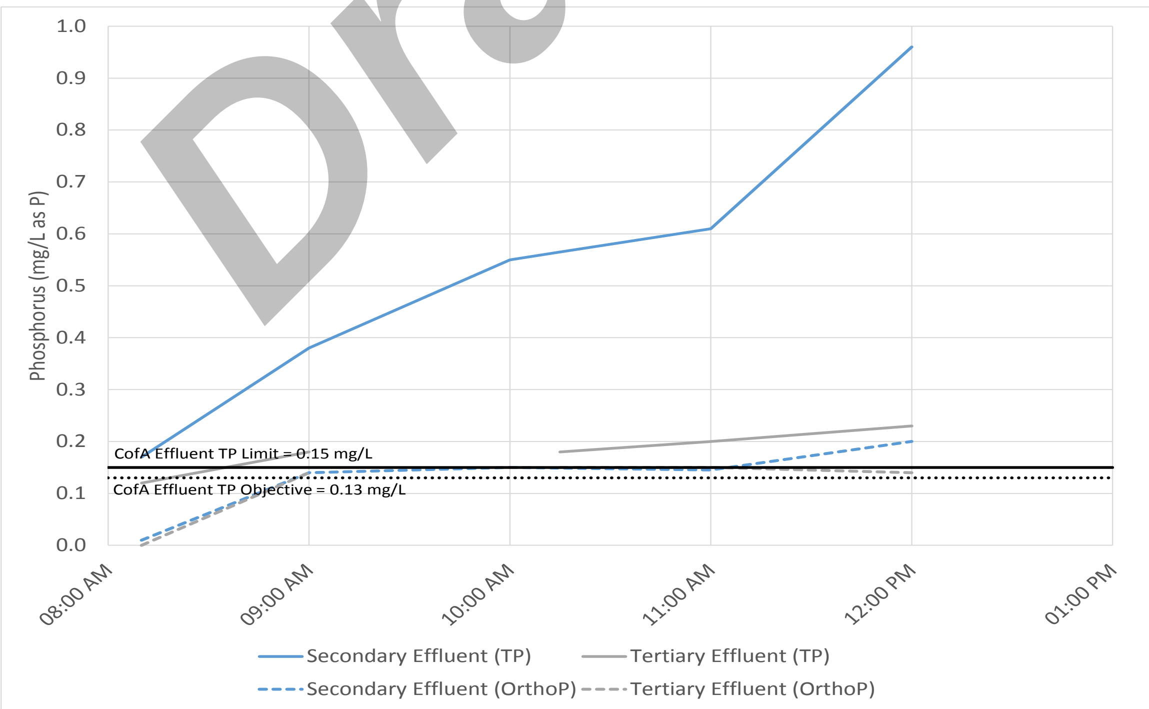


Figure 3.15 Measured Secondary Clarifier and Tertiary Filter Effluent Total Phosphorus and Orthophosphate Concentrations (Day 3)

Draft

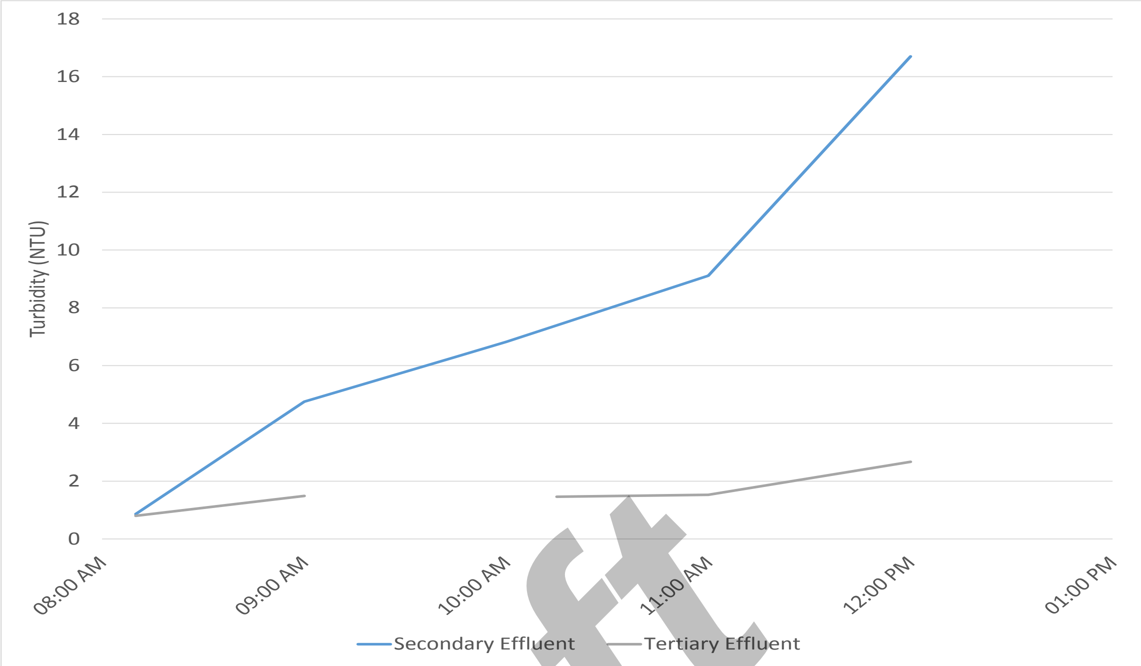


Figure 3.16 Measured Secondary Clarifier and Tertiary Filter Effluent Turbidity (Day 3)

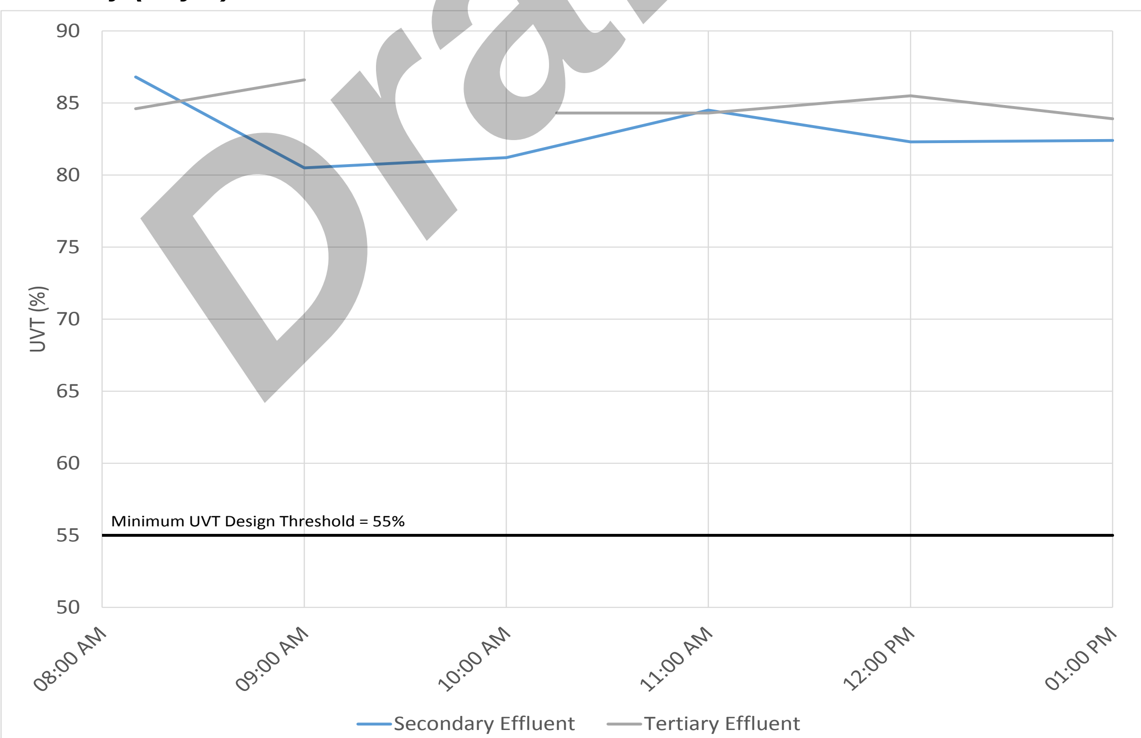


Figure 3.17 Measured Secondary Clarifier and Tertiary Filter Effluent UVT (Day 3)

Based on results presented in Figure 3.14, Figure 3.15, Figure 3.16, and Figure 3.17, the following conclusions can be drawn about the maximum day flow testing at the Grand Valley WPCP.

- Secondary effluent TSS and TP concentrations and turbidity measurements rose steadily over the duration of the test. Secondary effluent TSS concentrations peaked during the last hour of testing at approximately 35 mg/L, which is greater than expected from an extended aeration plant with phosphorus removal (15 mg/L) (MOE, 2008). Secondary effluent TP concentrations peaked at approximately 0.96 mg/L, which is consistent with expected secondary effluent TP concentrations from an extended aeration plant with phosphorus removal (less than 1.0 mg/L) (MOE, 2008).
- Tertiary effluent TSS concentrations rose steadily during the test, however all concentrations remained below the C of A effluent TSS objective concentration of 8 mg/L.
- Tertiary effluent TP and orthophosphate concentrations rose slightly over the duration of the test. Peak concentrations were measured at 0.23 mg/L and 0.15 mg/L, respectively. TP concentrations were slightly above C of A effluent limits (0.15 mg/L), but less than typical effluent TP concentrations for an extended aeration plant with chemical phosphorus removal and tertiary filtration (0.3 mg/L) (MOE, 2008). Elevated concentrations of orthophosphate (and therefore TP) are likely related to alum dosing restrictions at the plant. Further TP removal may be possible by optimizing the alum dose.
- Tertiary effluent turbidity measurements rose slightly over the duration of testing.
- Secondary and tertiary effluent UVT measurements remained relatively stable. All UVT measurements were in excess of 80%, well above the design UVT of 55%.

3.3.3 Secondary Clarifier Solids Blanket

Sludge height measurements were taken regularly over the duration of the test period. Measurement were taken at three locations along the walkway of the test clarifier to measure blanket height at the exterior, middle, and interior of the clarifier. Approximate locations for sludge blanket measurements were previously shown in Figure 2.1. Sludge blanket height measurements over the duration of the Day 3 testing period is shown in Figure 3.18. The clarifier side water depth is 4.2 m, and is represented by the top of Figure 3.18.

Draft

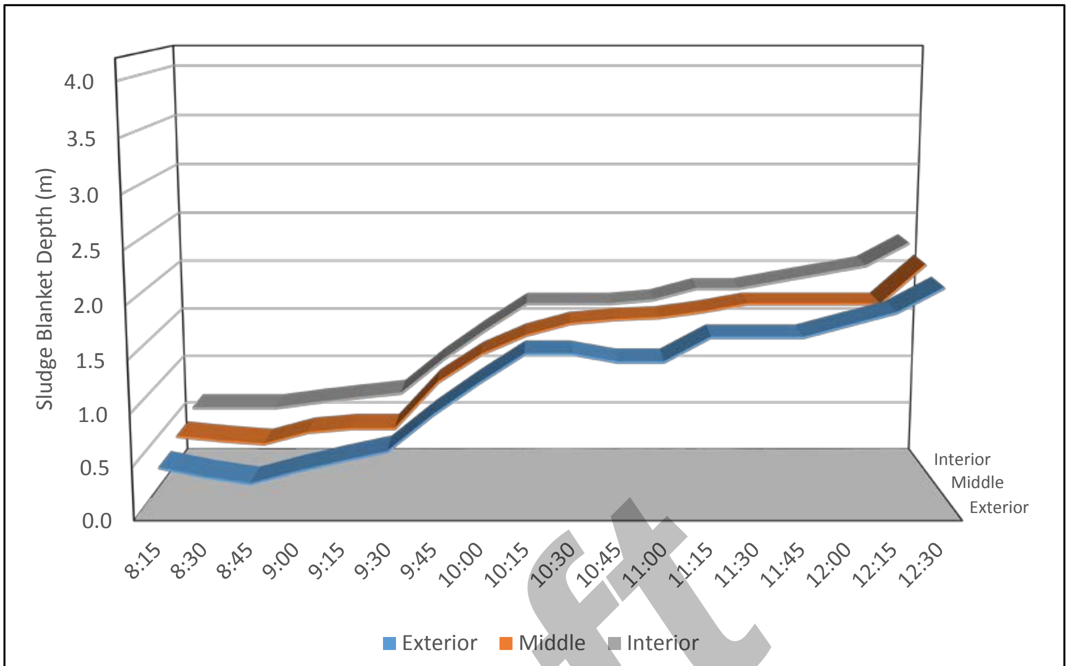


Figure 3.18 Secondary Clarifier Sludge Blanket Profile (Day 3)

The blanket depth ranged from approximately 450 mm (1.5 feet) at first measurement to approximately 2.1 m - 2.3 m (7.0 - 7.5 feet) at the end of the test. From approximately 8:30 am to 10:30 am, the sludge blanket depth rose rapidly in the secondary clarifier. For the remaining portion of the test, the sludge blanket appeared relatively stable, and sludge blanket height rose slowly. Day 3 of the stress test was stopped at 12:30 pm at sludge blanket heights of approximately 7.0 feet (2.1 m), 7.25 feet (2.2 m), and 7.5 feet (2.3 m) at the exterior, middle, and interior measurement points, respectively. Although blanket washout did not appear imminent, the test was stopped due to operator concerns regarding the integrity of the secondary clarifier mechanical equipment at the elevated sludge blanket height.

3.3.4 Evaluation of Secondary Clarifier Performance - Day 3

Average SOR and SLR values achieved during Day 3 of testing were 31.2 $m^3/m^2\cdot d$ and 153 $kg/m^2\cdot d$, respectively. Based on results presented in Figure 3.14 and Figure 3.15, average secondary clarifier effluent concentrations of TSS and TP from all samples collected over the duration of the testing period remained comparable to typical secondary effluent quality of an extended aeration treatment process (MOE, 2008).

However, secondary clarifier effluent concentrations of TSS and TP consistently rose during the testing period. Further, sludge blanket levels also rose consistently, indicating that steady state was not achieved during Day 3 of testing. Results from Day 3 of testing suggest the maximum day SLR and SOR capacities of the secondary clarifiers are less than approximately 153 $kg/m^2\cdot d$ and 31.2 $m^3/m^2\cdot d$, respectively.

Draft

3.3.5 Evaluation of Tertiary Filter Performance - Day 3

Average and peak hour filtration rates achieved during Day 3 of testing were 3.03 L/m^2·s, and 3.30 L/m^2·s, respectively. Similarly, the average and peak solids loading rates to the tertiary filter was 6.17 kg/m^2·d and 9.98 kg/m^2·d, respectively. It is important to note that the solids loading rates achieved during Day 3 of testing significantly exceed the maximum estimated solids load observed during stable filter operation on Day 2 (4.03 kg/m^2·d). As such, tertiary filter capacity at the Grand Valley WPCP appears to be limited by the filtration rate.

Based on results presented in Figure 3.14, Figure 3.15, Figure 3.16, and Figure 3.17, tertiary filter effluent quality remained high for the duration of the Day 3 testing period. As such, results from Day 3 of testing confirm the peak hour capacity of the tertiary filters to be 3.30 L/m^2·s, equal to the C of A rated peak capacity and typical design peak filtration rates (MOE, 2008).

3.3.6 Evaluation of Disinfection Performance - Day 3

All UVT measurements of secondary clarifier and tertiary filter effluent taken during Day 3 of testing measured > 80% and were consistent with results from Day 2 of testing. Therefore, results from Day 3 support previous conclusions which suggest the capacity of the UV disinfection system is greater than the peak rated capacity of 7,680 m^3/d.

Draft

4. ESTIMATED UNIT PROCESS CAPACITIES

4.1 Secondary Clarifiers

The estimated capacity of a secondary clarifier is typically evaluated at both peak hour and maximum day flows and expressed using the calculated peak hour SOR and maximum day SLR. However, as previously discussed, operation of the storm equalization tank at the Grand Valley WPCP is expected to attenuate peak flows through the treatment plant resulting in comparable maximum day and peak hour flows. Therefore, evaluation of secondary clarifier capacity at the Grand Valley WPCP should simultaneously consider both SOR and SLR under 'peak day' conditions.

For the purposes of developing clarifier capacities, the following future operating conditions were assumed:

- Both secondary clarifiers in operation (each with a surface area of 75.4 m^2);
- Operating MLSS concentration of 3,000 mg/L in the aeration tanks; and,
- An ADF of 1,244 m^3/d and a RAS:ADF ratio of 200%.

Based on results from Day 2 presented in Section 3.2, the estimated SOR and SLR capacity of the secondary clarifier was greater than 21.5 $m^3/m^2 \cdot d$ (equivalent peak day flow capacity of 3,242 m^3/d) and 159 $kg/m^2 \cdot d$ (equivalent peak day flow capacity of 5,504 m^3/d), respectively, but less than 40.9 $m^3/m^2 \cdot d$ and 240 $kg/m^2 \cdot d$.

During Day 3, the average SOR sustained for the duration of the testing period was 31.2 $m^3/m^2 \cdot d$ (equivalent daily flow of 4,705 m^3/d). However, the sustained SLR was relatively unchanged from Day 2 (i.e. within 5% of the measured SLR during stable operation on Day 2) and represented an equivalent peak daily flow of approximately 5,203 m^3/d. Stable operation of the test secondary clarifier was not observed during Day 3, therefore the capacity of the secondary clarifier appears to be limited by the SOR.

Together, results from Day 2 and Day 3 suggest that the capacity of the secondary clarifier is greater than 21.5 $m^3/m^2 \cdot d$ (3,242 m^3/d) based on stable operation observed during Day 2, but less than 31.2 $m^3/m^2 \cdot d$ (4,705 m^3/d) based on unstable operation observed during Day 3.

As previously discussed, flow through the treatment plant during the testing period was controlled using several pumps from several flow sources thereby making it difficult to maintain consistent flow through the plant. This limited ability to control plant flows also made it difficult to develop specific estimates of secondary clarifier capacity. However, periods of relatively stable flows during Day 3 of the testing period can be used to develop a more accurate estimate of clarifier capacity. Specifically, consider the period from 10:00 am to 11:00 am on Day 3. Measured secondary clarifier effluent concentrations of TSS and TP (shown as Figure 3.14 and Figure 3.15, respectively) appear relatively stable and comparable to typical secondary effluent quality of an extended aeration treatment process (MOE, 2008). As shown in Figure 3.18, sludge blanket height measurements during this period also remained relatively stable. As such, it appears steady operation of the secondary clarifier was achieved. The estimated SOR during this period of stable operation between 10:00 am

Draft

and 11:00 am on Day 3 was 29.1 $m^3/m^2 \cdot d$ (4,388 m^3/d) and represents the estimated capacity of the secondary clarifiers at the Grand Valley WPCP.

4.2 Tertiary Filters

Performance of the tertiary filters was evaluated using tertiary effluent measurements of TSS and TP. The capacity was expressed in terms of both a filtration rate per surface area ($L/m^2 \cdot s$) and solids loading rate ($kg/m^2 \cdot d$). Based on results from Day 2 of testing, the filtration capacity was found to be greater than 2.40 $L/m^2 \cdot s$, but less than 3.65 $L/m^2 \cdot s$. The design peak filtration rate is 3.30 $L/m^2 \cdot s$. Similarly, the solids loading capacity was found to be greater than 4.03 $kg/m^2 \cdot d$, but less than 6.15 $kg/m^2 \cdot d$.

During Day 3 of testing, stable filter operation was observed over the duration of the testing period. Peak hour filter flow and solids loading conditions achieved during Day 3 were 3.30 $L/m^2 \cdot s$ and 9.98 $kg/m^2 \cdot d$, respectively. Therefore, relative to Day 2, stable filter operation was achieved at significantly higher filter solids loading rates during Day 3.

Overall, results suggest filter capacity is limited by the filtration rate. Further, from the testing results, the estimated capacity of the tertiary filters is 3.30 $L/m^2 \cdot s$, equal to the design peak flow capacity.

4.3 UV Disinfection System

As previously discussed, the capacity of the UV disinfection system was evaluated using secondary clarifier and tertiary filter UVT measurements from samples collected over the duration of the testing period. Samples collected from both locations over both days of testing consistently had UVTs which measured greater than 80%, well in excess of the design UVT of 55%.

However, as a result of possible bypass flows, the quality of flow through the UV disinfection system could be of lower quality relative to the tertiary effluent stream during peak flow events. In the fall of 2015, samples of the raw influent and tertiary effluent streams were collected from the Grand Valley WPCP. Samples were combined in different volumetric ratios, and the UVT of these combined samples was measured to determine the potential impact from storm tank bypass flows on the UVT of the final effluent. Samples consisting of 100% tertiary effluent had a UVT of approximately 88%, comparable to results from baseline testing conducted for the stress test. Combined samples consisting of 40% raw influent or less (by volume) consistently measured a UVT greater than 55%. During a peak flow event, the storm tank bypass would make up significantly less than 40% of the effluent; in addition, when stressed, the secondary clarifiers and tertiary filters continue to produce a tertiary effluent with UVT > 80%.

Overall, these results suggest the capacity of the UV disinfection system is greater than the design peak flow capacity of 7,680 m^3/d.

Draft

5. CONCLUSIONS

5.1 Summary of Stress Testing Conducted

Peak hour performance testing was carried out on the secondary clarifiers and tertiary filters at the Grand Valley WPCP on July 12 (Day 1), July 13 (Day 2), and July 18 (Day 3).

During Day 2 of testing, flows were increased incrementally over 1 hour periods to try and reach the hydraulic capacity of the secondary clarifiers and tertiary filters. The test began with one secondary clarifier and two tertiary filters. As a result of increased solids concentrations in the tertiary effluent stream, an additional tertiary filter was brought online approximately halfway through the test. Testing was continued, and results were used to estimate the peak hour hydraulic capacity of the secondary clarifiers. Peak hour operating conditions achieved during the test are summarized in Table 5.1.

Table 5.1 Summary - Day 2 Peak Hour Operating Conditions Achieved During Testing

Test Unit	Value	Target
Secondary Clarifier		
SOR ($m^3/m^2 \cdot d$)	40.9	43
SLR ($kg/m^2 \cdot d$)	240	210
Tertiary Filter		
Filtration Rate ($L/m^2 \cdot s$)	3.16 (1)	4.04
Solids Loading Rate ($kg/m^2 \cdot d$)	5.32 (1)	-

Notes:
1. Estimated filtration rate average between 11:00 am and 12:00 pm. Average includes impact of third filter, which was brought online at 11:30 am.

During Day 3 of testing, flows were held constant over a 4 hour period to evaluate the maximum day capacity of the secondary clarifiers and tertiary filters. The test was conducted with one secondary clarifier and two tertiary filters. Average operating conditions over the Day 3 testing period are summarized in Table 5.2. Since only two filters were kept online for the duration of the testing period, the peak hour filtration rate achieved during Day 3 of testing was greater than the peak hour filtration rate achieved during Day 2. The peak hour filtration rate is also shown in Table 5.2.

Table 5.2 Summary - Day 3 Operating Conditions Achieved During Testing

Test Unit	Value	Target
Secondary Clarifier		
SOR ($m^3/m^2 \cdot d$)	31.2	43
SLR ($kg/m^2 \cdot d$)	153	210
Tertiary Filtration Rate ($L/m^2 \cdot s$)		
Test Average	3.03	4.04
Peak Hour	3.30 (1)	
Tertiary Solids Loading Rate ($kg/m^2 \cdot d$)		-
Test Average	6.17	
Peak Hour	9.98	

Notes:
1. Estimated filtration rate during peak hour flows from 11:30 am to 12:30 pm.

Draft

5.2 Estimated Treatment Capacities

Capacity evaluations of the secondary clarifier typically consist of a peak hour capacity (determined by the SOR) and a maximum day capacity (determined by the SLR). However, as a result of attenuation by the storm tank, peak hour and max day flows at the Grand Valley WPCP are expected to be similar. As such, a 'peak day' capacity of the secondary clarifier based on both SOR and SLR was made using measurements of secondary clarifier effluent TSS and TP concentrations, and on the height and stability of sludge blanket level measurements.

Using results from both Day 2 and Day 3, capacity of the secondary clarifier was found to be limited by the SOR. Detailed analysis of results from Day 3 of testing identified a period of stable clarifier operation between 10:00 am and 11:00 am, and was characterized by stable secondary clarifier effluent concentrations of TSS and TP, and stable measurements of sludge height. The SOR capacity, estimated from this period of stable operation, is approximately 29.1 $m^3/m^2 \cdot d$.

Capacity evaluations of tertiary filters were based on tertiary effluent TSS and TP concentrations. Capacity was found to be limited by the filtration rate, and was estimated to be 3.30 $L/m^2 \cdot s$.

Capacity evaluations of the UV disinfection system were based on secondary clarifier and tertiary filter effluent UVT measurements taken during this test, and on previous work which measured the UVT of final effluent and raw influent samples combined in different volumetric ratios. Capacity of the UV disinfection system was estimated to be in excess of the design peak capacity of 7,680 m^3/d.

Based on the results of the stress testing, Table 5.3 summarizes the estimated capacities of the selected treatment units.

Table 5.3 Recommended Operating Capacity from Stress Test Results

Treatment Process	Limiting Factor	Estimated Capacity
Secondary Clarification		
Peak Hour	SOR (29.1 $m^3/m^2 \cdot d$)	4,388 m^3/d
Maximum Day	SLR (153 kg/ $m^2 \cdot d$)	5,203 m^3/d (1)
Tertiary Filtration		
Peak Hour	Filtration Rate (3.30 L/ $m^2 \cdot s$)	5,300 m^3/d
Disinfection		
Peak Hour	UVT (>55%)	>7,680 m^3/d

Notes:
1. Assuming future MLSS concentration of 3,500 mg/L, an ADF of 1,244 m^3/d, and a RAS:ADF of 2:1.

It is important to note that the clarifier capacity calculated based on the measured SLR assumed an operating MLSS concentration of 3,000 mg/L. This target was established as part of the capacity assessment of the biological treatment system. Historically, the plant has operated at MLSS concentrations from approximately 2,500 mg/L to greater than 8,000 mg/L. As flows increase, operating at high MLSS concentrations in the future may result in the clarifier being limited by the SLR to a peak capacity less than 4,388 m^3/d.

Secondary clarifiers at the Grand Valley WPCP are typically covered to prevent growth of algae. For purposes of this test, select panels were removed to allow for installation of a flow meter and for sludge height readings. However, several panels were left during the testing period. Therefore, it was not possible to visually observe the entire overflow weir for localized areas of solids carryover resulting from short-circuiting within the clarifier. Future testing could include tracer testing to evaluate the hydraulics within the clarifier.

Finally, results from the stress test also found that alum dosing restrictions at the Grand Valley WPCP had a negative impact on final effluent concentrations of orthophosphate and TP. Future removal of orthophosphate can be optimized by increasing the alum dosing capacity to achieve historical (70 mg/L) or typical (110 to 225 mg/L) dosage rates (MOE, 2008) at design peak flows.

Draft

6. REFERENCES

1. Ministry of the Environment. Design Guidelines for Sewage Works. 2008.
2. Metcalf & Eddy. Wastewater Engineering: Treatment and Resource Recovery. Fourth Edition. Toronto. 2003.
3. R.J. Burnside & Associates Limited. Township of East Luther Grand Valley - Grand Valley Wastewater Treatment Plant Operations Manual. July 2015.
4. XCG. Grand Valley WPCP Re-rating Feasibility Study - Proposed Design Flows and Loads. November 2015.

Draft

APPENDIX A
COPY OF SECONDARY CLARIFIER AND TERTIARY FILTER STRESS TESTING PROTOCOL

Draft

XCG Environmental Engineers & Scientists
XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

Date:	June 16, 2016	**XCG File No.: 3-252-57-01**
To:	Jane Wilson, Town of Grand Valley (Town) Scott Craggs, Ontario Clean Water Agency	
cc:	Jeff Bunn and Glenn Sterret, Town	
From:	Graham Seggewiss, Melody Johnson and Linda Perry, XCG Consulting Limited (XCG)	
Re:	Grand Valley WPCP Rerating Study - Secondary Clarifier and Tertiary Filter Stress Testing Protocol	

Draft

1. INTRODUCTION

The Grand Valley Water Pollution Control Plant (WPCP) provides treatment for wastewater generated in the community of Grand Valley within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of the Environment and Climate Change (MOECC) Certificate of Approval (C of A) No. 9706-7KWQ57, issued on February 2, 2009. The quality and quantity of effluent currently discharged by the existing WPCP is regulated by the C of A. The Grand Valley WPCP has a rated average day flow (ADF) capacity of 1,244 m^3/d.

The Town has initiated an investigation to analyze the potential to re-rate the existing Grand Valley WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. The Town has retained XCG Consulting Limited (XCG) to undertake a capacity assessment of the Grand Valley WPCP to evaluate the potential to re-rate the plant. Stress testing of the secondary clarifiers, tertiary filters, and ultraviolet (UV) disinfection system was proposed to confirm the actual peak hydraulic and solids loading capacities of these unit processes.

The objective of this document is to present the proposed protocol for stress testing of the secondary clarifiers, tertiary filters, and UV disinfection processes at the Grand Valley WPCP.

2. SECONDARY CLARIFIER AND TERTIARY FILTER STRESS TESTING

2.1 Overview of Test Procedures

The Grand Valley WPCP is equipped with two circular secondary clarifiers, four continuous-backwash tertiary filters, and a UV disinfection system. A summary of these processes is included as Table 2.1.

Table 2.1 Grand Valley WPCP Process Design Information

Unit Process	Design Parameter [1]
Secondary Clarifiers	
Number	2
Surface Area	75.4 m^2 (each) 150.8 m^2 (total)
Filters	
Type	Continuous up-flow, deep bed, granular media
Backwash	Continuous
Number	4
Filtration Area	4.65 m^2 (each) 18.6 m^2 (total)
Peak Flow Capacity	5,300 m^3/d
Disinfection	
Type	UV Disinfection
Peak Flow Capacity	7,680 m^3/d

Notes:
1. Based on Amended Certificate of Approval Number 9706-7KWQ57, issued February 2, 2009, and the Grand Valley Wastewater Treatment Plant Operations Manual (R.J. Burnside, 2015).

Draft

The purpose of the stress testing is to assess the treatment capacity of the existing secondary clarifiers, tertiary filters, and UV disinfection system while meeting the effluent total suspended solids (TSS), total phosphorus (TP) and *E. coli* objectives for the plant.

The Stress Test will consist of three days of testing onsite at the Grand Valley WPCP, and will evaluate the peak hour and maximum day treatment capacities of the secondary clarifiers, tertiary filters, and UV disinfection system. During the stress tests, flow through the plant and the number of unit processes online at any given time will be controlled by operations staff to achieve the stress testing target flows.

As detailed in Table 2.1, secondary clarification at the Grand Valley WWTP consists of two identical secondary clarifiers. As the secondary clarifiers have identical dimensions, it is assumed that they have equal potential treatment capacities. Therefore, the performance of only one secondary clarifier will be tested during this program, and is assumed to be representative of the performance of both secondary clarifiers.

Similarly, since the existing tertiary filters have identical dimensions and configurations, it is assumed that the capacity of each filter is equal. For purposes of this test, the performance of two tertiary filters will be evaluated. The remaining two tertiary filters will be used as required to provide additional filtration capacity should the capacity of the two test filters be exceeded during the stress test. Additional details regarding contingency plans during the stress test are included in Section 4.

The existing UV disinfection system has been designed with a minimum UV Transmittance (UVT) of 55%. The treatment capacity of the UV system will be evaluated by collecting tertiary effluent samples throughout and recording the UVT of each sample.

2.2 Proposed Testing Schedule

Stress testing will be conducted on the secondary clarifiers, tertiary filters, and UV disinfection system at the Grand Valley WPCP by XCG, with assistance from plant personnel. Stress testing will be completed over three days, consisting of:

- Day 1 - Setup, Preparation, and Baseline Testing of the clarifiers and filters.
- Day 2 - Peak Hourly Flow Testing.
- Day 3 - Maximum Day Flow Testing.

Plant operators will be required to ensure adequate supplementary volume is available at the plant prior to testing. As such, testing may not occur on concurrent days. Additional details regarding test set up and the provision of supplemental volume is included in Section 3.

Prior to conducting the Stress Test, plant operators will be asked to adjust sludge wasting as required to achieve target MLSS concentrations in the aeration tanks. For purposes of the Stress Test, the target MLSS concentration is approximately 4,000 to 4,500 mg/L.

Day 1 – Setup, Preparation and Baseline Testing

1. Confirm sampling locations. Install and calibrate autosamplers, flow meters, and temporary pumps.
2. Collect pre-test samples of mixed liquor, secondary clarifier effluent, and tertiary filter effluent (See Section 3.2 for general sampling procedure).
3. Record the radial profile of the sludge blanket of the secondary clarifier. A sludge judge will be used to measure the sludge blanket level along the radius of the secondary clarifier and the results recorded.
4. Ensure that sludge blanket level in the secondary clarifiers is within typical range and, if higher, increase return activated sludge (RAS) pumping rate to lower the sludge blanket level in advance of the testing.
5. Record the observed headloss across the tertiary filters.

Day 2 – Peak Hourly Flow Testing

Day 2 will consist of peak hour flow (PHF) testing of the test secondary clarifier and the two test tertiary filters. The following steps will be performed on the testing day:

1. Collect pre-test samples (See Section 3.2 for general sampling procedure).
2. Gradually ramp up flows until the initial target peak hour flow is achieved (See Section 3.1 for general flow adjustment procedures).
3. Flows will be held constant for one hour periods to allow test clarifier and filters to stabilize. During each hour period, monitor flow rates, secondary and tertiary effluent quality, sludge blanket levels, and filter headloss levels (See Section 3.2 for general sampling and monitoring procedures). Continuously monitor secondary effluent will for solids carry-over throughout stress test.

Draft

4. Provided the clarifier and filters are still operating well, the supplemental flow rate will be increased incrementally at the end of each one hour period. The flow increments will be determined by XCG and OCWA staff at the time of the testing based on specific site conditions and the ultimate target PHF. See Section 3.1 for a description of the methodology to be used to increase flows to the test clarifier and filters.
5. Collect required samples during each flow increment (See Section 3.2 for general sampling procedures).
6. Record the radial profile of the sludge blanket of the secondary clarifier. A sludge judge will be used to measure the sludge blanket level along the radius of the clarifier and the results recorded.
7. Observe any flow patterns in the clarifier or along the weirs such as areas of low flow, high flow, or solids upflow. Observe channels, troughs, and weirs for any indication of hydraulic limitations.
8. Repeat steps 1 to 5 until an imminent failure of the clarifier and/or filter is observed and/or hydraulic capacity of the channels is reached and/or the target peak flow rate is met or exceeded. An imminent failure of the secondary clarifier is indicated by a significant increase in sludge blanket depth and/or deterioration in effluent quality as measured by a significant increase in the TSS concentration or turbidity. An imminent failure of the tertiary filter is indicated by increasing/unstable measured headloss, and/or a deterioration in the effluent quality as measured by effluent TSS concentrations, turbidity or UVT.
9. When PHF stress test is complete, collect post-test samples (See Section 3.2 for general sampling procedure).
10. Return plant to normal operating conditions by shutting off all supplemental flows. Coordinate with plant operations to fill supplemental flow volumes in preparation of Day 3 of testing (See Section 3.1 for general tank filling procedure).

Day 3 – Maximum Day Flow Testing

Day 3 of testing will consist of maximum day flow testing of the test secondary clarifier and two test tertiary filters. The following steps will be performed on the testing day.

1. Collect pre-test samples (See Section 3.2 for general sampling procedure).
2. Gradually ramp up flows until the target flow is achieved (See Section 3.1 for general flow adjustment procedure). The target flow will be selected based on projections and the results of the peak hourly flow testing (Day 2).
3. Flows will be held constant for up to a five hour period, representative of a high flow event controlled by the storm tank.
4. Collect required samples during test event (See Section 3.2 for general sampling procedure).
5. Record the radial profile of the sludge blanket of the secondary clarifier. A sludge judge will be used to measure the sludge blanket level along the radius of the secondary clarifier and the results recorded.

Draft

6. Continuously monitor secondary effluent for solids carry-over and tertiary effluent for a deterioration in quality. Monitor the stability of the measured filter headloss throughout the stress test.
7. Observe flow patterns in the clarifier or the effluent weirs such as areas of low flow, high flow, or solids upflow. Observe channels, troughs, and weirs for any indication of hydraulic limitations.
8. When the stress test is complete, collect post-test samples (See Section 3.2 for general sampling procedures). Return plant to normal operating conditions, and empty supplemental volume reservoirs.

3. GENERAL PROCEDURES FOR FIELD TESTING

3.1 Supplemental Flow

Test flows through the secondary clarifier and tertiary filters will be monitored over the duration of the testing period. This will be accomplished using existing flow meters measuring plant influent flow and return activated sludge flows, and a temporary flow meter to monitor test secondary clarifier effluent flow. Secondary clarifier effluent flow will be monitored through installation of a velocity-area (VA) flow meter in the effluent trough of the test secondary clarifier. Secondary clarifiers at the Grand Valley plant are typically covered to prevent algae growth. Installation of the VA flow meter will require the removal of selected covering panels by plant personnel. The procedure to achieve the target flow will depend on the influent flows to the plant during the stress test. The assistance of plant personnel will be required for flow split control and adjustment.

It is expected that sufficient, steady flow from the Emma St. SPS will not be available to achieve target flows for the duration of the proposed testing period. As such, the raw influent flow will be supplemented with flow from the offline aeration tank and the storm equalization tank. This section will review how supplemental volumes will be filled and drained for purposes of testing.

Draft

3.1.1 Tank Filling Procedure

Prior to each day of testing (i.e. Day 2 and Day 3), operations staff will ensure that the offline aeration tank and storm tank are storing sufficient supplementary volume. The offline aeration tank will be filled with raw wastewater. Air will be turned on in the offline aeration tank to prevent septic conditions prior to the test. The storm tank will be filled with potable water by plant operators using available hosing and an onsite potable water connection.

3.1.2 Target Peak Flows

For purposes of this test, target peak hour and maximum day flow rates were estimated using the following assumptions:

- Proposed Scenario III future flows (XCG, 2015);
- Future storm tank overflow operation to provide sufficient volume to equalize two days of peak flows; and

- Peak flow event characteristics similar to a historic peak flow event available from plant records.

Please note that, during the Stress Test, plant flows will be increased only as permitted by acceptable plant performance. Based on the above assumptions, the future projected MDF and PHF to the plant are approximately 6,250 m^3/d and 6,500 m^3/d, respectively. As only half of the plant capacity will be tested, the target MDF and PHF for purposes of this Stress Test are, at a minimum, 3,125 m^3/d and 3,250 m^3/d, respectively.

3.1.3 Supplemental Flows and Volume

Required supplemental flow and volume was estimated assuming an average raw influent plant flow of 500 m^3/d (approximately 5.8 L/s), estimated from historic plant records for this time of year. A summary of the available supplemental volume and pumping capacity is given in Table 3.1.

Table 3.1 Supplemental Flow Details

Supplemental Flow Source	Volume (m^3)	Return Method	Pumping Capacity (m^3/d)
Offline Aeration Tank	400	Temporary Pump	1,625 (1)
Storm Equalization Tank	400	Temporary Pump	1,625 (1)
Total	800	-	3,250 (2)
Estimated Requirements for Stress Testing			
MDF testing PHF testing	605 (3) 461 (4)	-	2,625 (5) 2,750 (5)

Notes:
1. Estimated approximate capacity of temporary pumps required to achieve total target flow (3,250 m^3/d). Temporary pump capacity to be confirmed with equipment supplier prior to testing.
2. Proposed target pumping capacity to ensure sufficient pumping capacity is available for testing purposes.
3. Assumed target flow (3,125 m^3/d) less raw influent flow (500 m^3/d) sustained for five hours and including a 10% buffer volume.
4. Assumed target starting flow (1,500 m^3/d) sustained for one hour and increased by approximately 500 m^3/d each hour for five hours or until imminent failure is observed. Assumed raw influent flow of 500 m^3/d. Assumed 10% buffer on required supplemental volume. Actual supplemental volume requirements will depend on the return pump capacity.
5. Estimated from the projected target MDF (3,125 m^3/d) or PHF (3,250 m^3/d) less the raw influent plant flow (500 m^3/d).

Draft

Actual supplemental volume requirements may differ from above and will depend on the sustained raw influent flow during the Stress Test, and the variable supplemental flows achieved during the PHF testing. To accommodate for this uncertainty, a 10% buffer has been added to the estimated required supplemental volumes in Table 3.1.

3.1.4 Flow Adjustment Procedure

Procedures to achieve required supplemental flow rates may vary depending on the influent flow to the Grand Valley WPCP during testing. Supplemental flow will be added to the head of the aeration tanks via the flow split chamber using temporary pumps and hoses. Flow from all sources of supplemental volume should be variable and measurable to provide flexibility to achieve target flow rates. Flow control on the temporary pumping system can be accomplished by providing valving on the discharge header of the temporary pumps; flow metering can be provided by meters installed on the temporary piping and/or recording liquid levels in the offline aeration tank and storm tank and/or by monitoring secondary effluent flow using the temporary area-velocity flow meter. Exact set-up of the supplemental flow system will be confirmed by XCG with a supplier prior to the Stress Test.

3.2 Process Monitoring and Sampling

An automatic sampler will be configured to collect composite samples of effluent from the test clarifier and test filters. XCG will provide and install the required autosamplers. Autosampler operation and sample collection will vary from day to day as described below.

- On Day 1: Each sample will consist of four 15 minute "sub-samples" to obtain a 1 hour composite sample.
- On Day 2: Each sample will consist of two 15 minute "sub-samples" to obtain a 30 minute composite sample for the duration of the stress test period, plus one sample before and after stress testing has been completed.
- On Day 3: Each sample will consist of four 15 minute "sub-samples" to obtain a 1 hour composite sample for the duration of the stress test period, plus one sample before and after stress testing has been completed.

Each sample will be submitted to an accredited laboratory for TSS and TP analysis. Analysis of orthophosphate, turbidity, and UVT will be conducted on-site by XCG staff.

Mixed Liquor will be collected once per hour to determine the mixed liquor suspended solids (MLSS) concentration. Each sample will be submitted to an accredited laboratory for TSS analysis. One sample of mixed liquor per day will also be analyzed for 30-minute settling sludge volume index (SVI).

A summary of the proposed sampling is in Table 3.2.

A velocity-area flow meter will be installed in the secondary clarifier effluent trough to monitor secondary clarifier effluent flow. The test secondary clarifier will be monitored for sludge blanket depth and solids carryover. If deterioration in tertiary effluent UVT below the design UVT is observed during testing, grab samples of tertiary effluent will be collected and submitted to an external laboratory for collimated beam testing to determine the potential impact on downstream UV disinfection unit performance and capacity.

Draft

Table 3.2 Proposed Sampling Details

Sample Location	Sample Type	Sample Frequency	Monitored Parameters
Mixed Liquor	Grab	Day 1: Once Day 2/3: Hourly	TSS, VSS, SVI (1)
Secondary Clarifier Effluent	Composite	Day 1: Once Day 2: Semi-hourly Day 3: Hourly	TSS, TP, Orthophosphate, turbidity, UVT
Tertiary Filter Effluent	Composite	Day 1: Once Day 2: Semi-hourly Day 3: Hourly	TSS, TP, Orthophosphate, turbidity, UVT

Notes:
1. Analyzed once per day.

Draft

4. PERFORMANCE AND CONTINGENCY PLANS

The performance of the secondary clarifier, tertiary filters, final effluent quality, and plant water levels will be carefully monitored throughout the testing. Plant tankage, channels, weirs and other control structures will be observed for any indication of hydraulic limitations as identified by submergence of weirs or imminent process bypass.

In the event of a clarifier failure, as indicated by excessive solids carry-over or sudden rise in sludge blanket depth, test flows will be gradually decreased and the secondary clarifier performance testing will be terminated. Testing will also be terminated in the event of a filter failure, as indicated by increasing headloss levels and/or a deterioration in effluent quality. In the event of tertiary filter failure before secondary clarifier failure, additional tertiary filters will be brought online and the test will be continued.

5. ROLES AND RESPONSIBILITY

XCG will coordinate the stress test with assistance from OCWA personnel and Town Staff to set-up for the stress test, operation of required equipment and instrumentation, as well as process monitoring, sample collection, and chain-of-custody preparation.

A summary of the responsibilities of XCG and plant personnel is provided in the following Sections.

5.1 XCG Staff Roles and Responsibilities

XCG staff will be responsible for the following:

- Obtaining quotes from suppliers for the installation of required equipment to transfer supplemental flow from the offline aeration tank and storm tank during the test.
- Provision and temporary installation of equipment required for the duration of the testing, including:
  - Two auto-samplers installed to collect samples of secondary and tertiary effluent from test units.
  - Secondary clarifier effluent flow monitoring equipment.

  - One sludge judge for sludge blanket depth measurement.
- Provide input to plant personnel for flow adjustment during testing.
- Program the installed auto-samplers to collect composite samples as required by the testing protocol.
- Collecting samples from the temporary auto-samplers and placing sample aliquots in the proper sample bottles and filling in the chain of custody forms to obtain the required analyses.
- Collecting grab samples of mixed liquor, settling as required, and placing sample aliquots in the proper sample bottles and filling in the chain of custody forms to obtain the required analyses.
- Provide input to plant personnel throughout the duration of the testing program, as required. XCG's main point of contact for questions or concerns during the sampling program will be Graham Seggewiss. If there are any questions in advance of the testing, he can be reached at 905-829-8880 or graham.seggewiss@xcg.com. He can also be reached on his cell phone at 519-536-3788 during the testing.

5.2 OCWA Staff Roles and Responsibilities

Plant personnel will be responsible for the following:

- Removal of selected secondary clarifier cover panels to allow for installation of the temporary VA meter in the secondary clarifier effluent trough.
- Operation, monitoring, and control of plant processes and equipment, maintain plant performance during stress testing and to achieve target flow rates.
- Coordinating the installation of the temporary pumps to transfer supplemental flow with the equipment supplier.
- Operation of temporary pumps to transfer supplemental flow from the offline aeration tank and storm tank during the test.
- Fill offline tankage (offline aeration tank with raw wastewater; equalization storm tank with potable water) to provide supplemental flow volumes prior to each day of testing.
- Adjusting the operation of the Emma St. SPS during testing as required. It is anticipated this will involve modifying the liquid level / VFD set points to operate with the jockey pump at its lowest discharge setting to reduce the frequency of pump on/off cycles.
- Providing key flow data (Emma St. SPS flow, RAS flow, Onsite Pumping Station Flow, Septage Pumping Station Flow, Final effluent flow) over the course of the stress testing in 2-5 minute intervals.
- Providing guidance to XCG staff with respect to appropriate installation locations for the field testing equipment. This will include providing access to 120V power outlets to power the equipment.

Draft

APPENDIX F

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY IMPACT OF ADDITIONAL EQUALIZATION VOLUME

Draft

XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

XCG File No.: 3-252-57-01
December 6, 2016

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY IMPACT OF ADDITIONAL EQUALIZATION VOLUME

Draft

Prepared for:

TOWN OF GRAND VALLEY
5 Main Street, North
Grand Valley, Ontario
L9W 5S6
Attention: Jane Wilson

Prepared by:

XCG CONSULTING LIMITED
Suite 300, 2620 Bristol Circle
Oakville, Ontario
L6H 6Z7

TABLE OF CONTENTS

TABLES

FIGURES

Draft

1. INTRODUCTION

The Grand Valley Water Pollution Control Plant (WPCP) provides treatment for wastewater generated in the community of Grand Valley within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of Environment and Climate Change (MOECC) Certificate of Approval (C of A) No. 9706-7KWQ57, issued on February 2, 2009. The quality and quantity of effluent currently discharged by the existing WPCP is regulated by the C of A. The Grand Valley WPCP has a rated average capacity of 1,244 m^3/d.

The Town has initiated an investigation to analyze the potential to re-rate the existing Grand Valley WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. The Town has retained XCG Consulting Limited (XCG) to undertake a capacity assessment of the Grand Valley WPCP to evaluate the potential to re-rate the plant.

Preliminary results of the assessment indicate the plant treatment capacity may be limited by peak flows capacity. As such, XCG conducted an analysis to evaluate the impact that additional equalization volume may have on the overall capacity of the Grand Valley WPCP. The purpose of this technical memorandum is to present results of that analysis.

Draft

2. GRAND VALLEY WPCP BACKGROUND INFORMATION

2.1 Existing Treatment Process

Raw sewage flows from the collection system are conveyed to the Grand Valley WPCP from the Emma St. sewage pumping station (SPS) via a forcemain. The Emma St. SPS is equipped with the following equipment:

- Two variable frequency drive (VFD) pumps (one duty and one standby), each with a rated capacity of 88.9 L/s (7,680 m^3/d).
- One VFD jockey pump with a rated capacity of 29.5 L/s (2,550 m^3/d).
- One wet will, with approximate volume of 125 m^3.

The jockey pump will not operate at peak flows. As such, the firm capacity of the Emma St. SPS is approximately 7,680 m^3/d. Over the review period (January 2012 – May 2016) there are no records of raw sewage bypassing at the Emma St. SPS or at the Grand Valley WPCP.

The Grand Valley WPCP receives septage at the septage receiving station. The septage receiving station removes solids from the raw septage using a combination of grinding, washing, and dewatering. The septage is then discharged to the plant headworks, upstream of the plant screens.

Plant influent raw wastewater flow consists of wastewater from the following sources:

- Raw wastewater from the Emma St. SPS;
- Septage from the on-site receiving station;
- Tertiary filter backwash; and,
- Digester supernatant.

Tertiary filter backwash and digester supernatant are transferred back to the head of the plant via an on-site pumping station. All flows are combined at the head of the plant, upstream of the plant headworks.

Headworks at the Grand Valley WPCP consists of a mechanical bar screen and two vortex grit separators. A manual screen also exists in parallel to the mechanical screen, and can be used as needed during peak flows or to isolate the mechanical screen. Flow to the manual screen is controlled using gates. High water levels in the screening channel can overflow the control gate, thereby initiating an emergency bypass of the mechanical screens.

Headworks effluent flow is discharged to a splitter box, where flow is directed to the aeration tanks, or to a bypass channel. Sustained peak flows in excess of 64 L/s (5,530 m^3/d) for greater than 10 minutes are directed to the bypass channel and into the 400 m^3 equalization tank (storm tank). From the equalization tank, flow can be returned to the head of the plant through the on-site pumping station. Bypass flows in excess of the equalization tank capacity are disinfected and discharged. There have been no recorded plant bypasses at the Grand Valley WPCP.

Draft

Secondary treatment at the Grand Valley WPCP consists of three aeration tanks and two secondary clarifiers. Oxygen is provided to each aeration tank through fine bubble diffusers. Alum is added immediately upstream of the secondary clarifiers for chemical phosphorus removal. Activated sludge is separated from the treated stream in the secondary clarifiers. Return activated sludge (RAS) is returned to the raw wastewater upstream of the aeration tanks. Waste activated sludge (WAS) is pumped to the aerobic digester located on-site. RAS and WAS are pumped from the same location in the secondary clarifier. Overflow from the secondary clarifiers is passed through one of four tertiary filters at the plant. Filter effluent is disinfected using ultraviolet (UV) radiation, then discharged to the Grand River. Waste activated sludge is digested and thickened on-site in the aerobic digester. Thickened sludge is pumped to the on-site biosolids storage tank, then trucked offsite for disposal.

Wastewater flow is measured at several locations at the plant. Raw wastewater from the collection system is metered at the Emma St. SPS. Wastewater flows from septage and the on-site pumping station are separately metered. Collectively, they represent the plant influent flow. Effluent flow from the Grand Valley WPCP is measured by a V-notch weir, downstream of the UV disinfection.

A process flow diagram of the Grand Valley WPCP is presented in Figure 2.1.

Draft

2.2 Plant Design Basis

For purposes of this evaluation, flows and loads to the Grand Valley WPCP were developed for three distinct scenarios. Details of each scenario are presented briefly below:

- Scenario I: Full completion of planned residential developments;
- Scenario II: A 15% increase above the current C of A rated ADF (1,430 m^3/d); and,
- Scenario III: A 25% increase above the current C of A rated ADF (1,555 m^3/d).

A summary of the updated flow design basis is given in Table 2.1 (XCG, 2016). For simplicity, the previous design basis (XCG, 2015) has not been presented in the table. This table represents raw the projected raw influent flow from the collection system to the Grand Valley WPCP, and does not include any recycle flow from the on-site pumping station. It is important to note the projected peak flows for all three scenarios exceed the existing rated capacity of the Emma St. SPS (7,680 m^3/d). Therefore, the Emma St. SPS may require upgrades at future flows provided that existing peak flows are not abated by any I/I reduction strategies. An extensive review of the Emma St. SPS capacity was not conducted as part of this review. Further, it is assumed that future peak flows to the Grand Valley WPCP will not be inhibited by the pumping capacity of the Emma St. SPS.

Table 2.1 Summary of Raw Influent Flow from the Collection System (XCG, 2016)

Parameter	Scenario I	Scenario II	Scenario III
Population	2,919	3,252	3,527
ADF	1,279 m^3/d	1,430 m^3/d	1,555 m^3/d
MDF	5,839 m^3/d	6,169 m^3/d	6,442 m^3/d
MDF Factor	4.6	4.3	4.1
PIF	7,811 m^3/d	8,291 m^3/d	8,684 m^3/d
PIF Factor	6.1	5.8	5.6

For purposes of this analysis, evaluation of the required equalization volume will be based on the projected maximum day flow through the treatment plant. It is important to note that backwash flow from the tertiary filters and supernatant from the on-site digester is discharged to the on-site pumping station where it is pumped to the head of the plant upstream of the plant headworks. As such, maximum day and peak instantaneous flows through the treatment plant are greater than those given in Table 2.1.

The maximum design backwash flow rate from the existing tertiary filters is 390 m^3/d (R.J. Burnside, 2015). For purposes of this analysis, it is assumed the digester is not supernated during a peak flow event. Table 2.2 summarizes the projected maximum day flow through the plant considering contributions from the Emma St. SPS (i.e. raw influent from the collection system) and from the on-site pumping station (i.e. tertiary filter backwash flow).

Table 2.2 Summary of Peak Flow through the Grand Valley WPCP Headworks

Maximum Day Flow	Scenario I	Scenario II	Scenario III
Emma St. SPS (Collection System)	5,839 m^3/d	6,169 m^3/d	6,442 m^3/d
On-site Pumping Station (Filter Backwash)		390 m^3/d	
Total Projected Maximum Day Flow	6,229 m^3/d	6,559 m^3/d	6,832 m^3/d

Draft

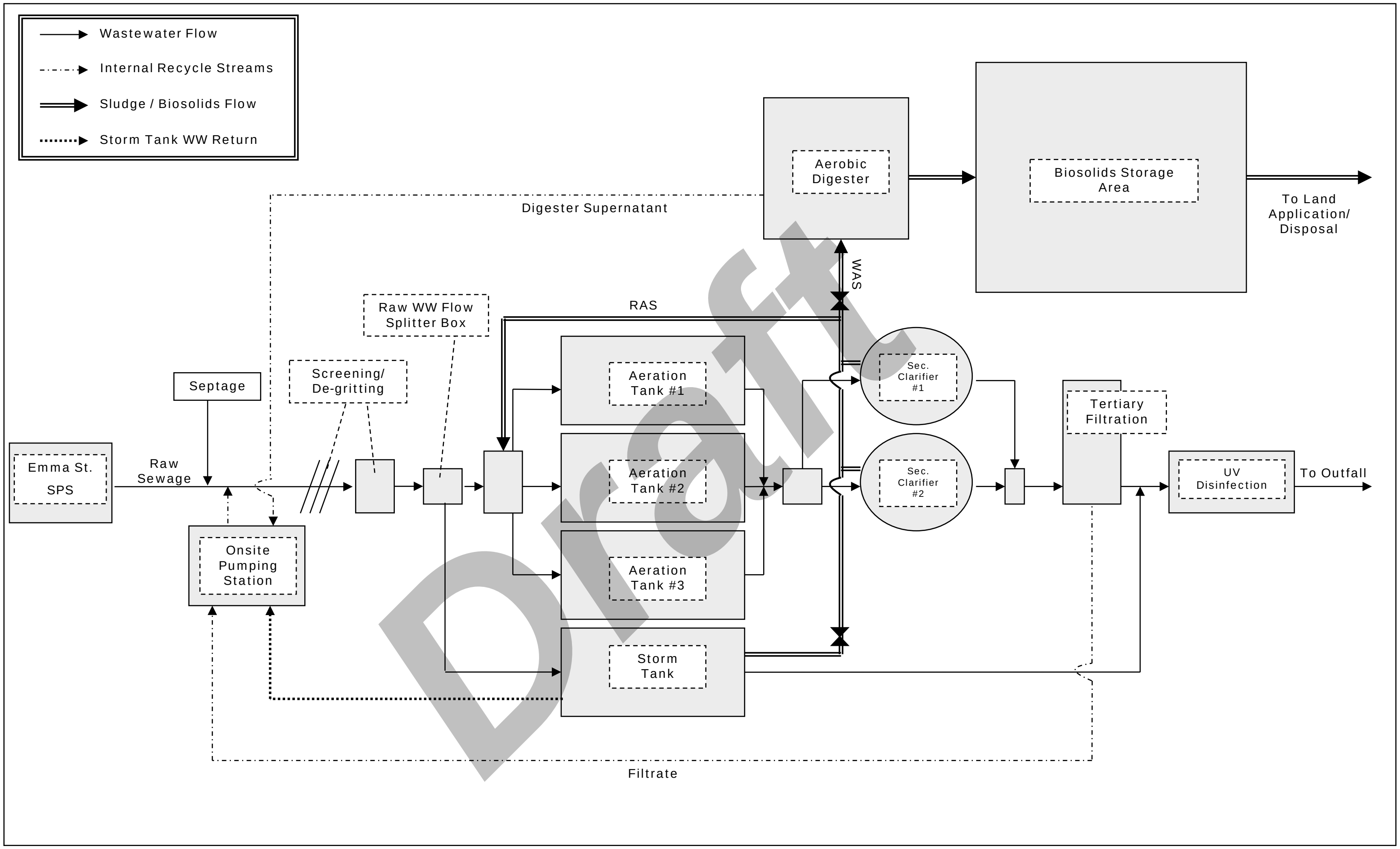


Figure 2.1 Process Flow Schematic – Grand Valley WPCP

3. DETAILS OF ADDITIONAL EQUALIZATION FOR THE GRAND VALLEY WPCP

Currently, equalization for the Grand Valley WPCP is provided by a 400 m^3 storm tank located on-site at the Grand Valley WPCP. It is assumed this storm tank would continue to be used in the future.

For purposes of this investigation, two equalization options were developed and evaluated. Details of each equalization option is included in Table 3.1.

Table 3.1 Summary of Equalization Options

Option	Details
Option 1	• Provide sufficient equalization volume to facilitate re-rating of the Grand Valley WPCP to the Scenario I flows and loads.
Option 2	• Provide sufficient equalization volume to facilitate re-rating of the Grand Valley WPCP to the Scenario III flows and loads.

The purpose of this section is to present considerations for the construction of additional equalization volume in the Town of Grand Valley.

3.1 Impact of Equalization Location

There are two potential locations where additional equalization could be constructed in Grand Valley, Ontario: at the Emma St. SPS and/or at the Grand Valley WPCP. Although space is available on-site at the Grand Valley WPCP, construction of additional equalization volume may limit the land available for future expansion of the plant. For purposes of this study, it is assumed that additional equalization installed at the Grand Valley WPCP would divert flow from the same location as the existing equalization tank. As such, projected peak flows through the plant headworks and from the Emma St. SPS would not be reduced via the installation of additional equalization volume at the WPCP site.

Conversely, the Emma St. SPS is located at the site of the old wastewater treatment plant. The majority of infrastructure has been removed from the site and minimal expansion of the existing infrastructure is expected to be required to meet future flows. As such, there is significant land available for the construction of additional equalization as required. By constructing equalization volume at the Emma St SPS, peak flows requiring conveyance through the SPS and, by extension, influent peak flows to the WPCP would be reduced.

An analysis of the hydraulic treatment capacity of the existing plant headworks (i.e. screening and grit removal) has also been completed (XCG, 2016). The results indicate that the hydraulic capacity of the existing headworks exceeds the projected Scenario III peak flows without the installation of any additional equalization volume.

As noted in Table 2.1, projected peak flows from the collection system exceed the current rated pumping capacity of the Emma St. SPS. Installation of equalization volume at Emma St. would reduce peak flows below the existing rated capacity of the raw influent pumps. Conversely, if additional equalization volume is installed at the

Draft

Grand Valley WPCP, peak flows from the Emma St. SPS would not be reduced. As such, installation of equalization volume at the Grand Valley WPCP would need to be accompanied by a detailed investigation of the pumping capacity of the Emma St. SPS and hydraulics of the forcemain between the plant and pumping station.

Therefore, to avoid the potential of additional required upgrades to the Emma St. SPS and/or the forcemain, this analysis has assumed additional equalization volume would be installed at the Emma St. SPS. Ultimate selection of the location and volume of additional equalization would be finalized during the detailed design.

3.2 Analysis of Projected Peak Flows and Estimate of Required Equalization Volumes

The following assumptions were made to develop an estimate of the required equalization volume for each equalization option:

- Sufficient volume is required to provide 24-hours of equalization at a simulated future peak flow event.
- Detailed flow characteristics of the historical peak flow event (recorded on April 14, 2014) are representative of future peak flow events.

The peak treatment capacity of the Grand Valley WPCP was evaluated through stress testing of the secondary clarifiers, tertiary filters, and UV disinfection system. Results were previously presented in the Secondary Clarifier, Tertiary Filter, and Disinfection Stress Test Results Technical Memorandum (XCG, 2016). Based on the results, the estimated peak treatment capacity of the plant including flow from the tertiary filter backwash is approximately 4,400 m^3/d and is limited by the secondary clarifiers.

Table 3.1 summarizes the estimated required equalization volume for each equalization option that maintains the projected peak flow through secondary treatment at the WPCP to less than 4,400 m^3/d.

Draft

Table 3.2 Summary of Estimated Required Equalization Volume

	Option 1 (Sufficient Capacity for Scenario I Flows)	Option 2 (Sufficient Capacity for Scenario III Flows)
Projected MDF	6,229 m^3/d	6,832 m^3/d
Total Estimated Equalization Volume Required	1,900 m^3	2,500 m^3
Existing Equalization Volume (1)	400 m^3	
Additional Equalization Volume Required at Emma St SPS	1,500 m^3	2,100 m^3
Estimated Equalized Peak Flow (2)	4,327 m^3	4,330 m^3

Notes:
1. Volume of existing storm tank at the Grand Valley WPCP.
2. Due to size of the proposed equalization volume for each option, the projected equalized maximum day and peak hour flows for each option are equal.

3.3 Installation Considerations and Capital Cost Estimations

As previously discussed, it has been assumed that additional equalization volume would be constructed at the Emma St. SPS located upstream of the Grand Valley WPCP.

Installation of additional equalization volume can be carried out as a Schedule B activity under the Municipal Class Environmental Assessment Process as per the following text:

"Establish sewage flow equalization tankage in existing sewer system or at existing sewage treatment plants, or at existing pumping stations for influent and/or effluent control"

As a Schedule B project, Phase 1 and Phase 2 of the Class EA process must be completed prior to implementation of the project (i.e. construction). Brief requirements of each Phase are given below.

Phase 1

During this phase, the problem or opportunity must be identified and described. Projects which are expected to generate significant public interest can also begin the public consultant process.

Phase 2

During this phase, potential alternative solutions will be identified and evaluated. Solutions will consider the size (volume) and location of additional equalization. This Phase will also include mandatory consultation with relevant review agencies and stakeholders (e.g. MOECC, GRCA, First Nations, etc.) and the public.

At the completion of Phase 2, the entire planning process (i.e. Phase 1 and Phase 2 activities) will be summarized and placed on file for a period of thirty (30) days. A notice of completion will be issued to review agencies and to the public.

Assuming no request for an Order is received during the review period, the Town may proceed with the design and construction of the equalization tank. Detailed design of the equalization tank would need to consider the integration of the equalization tank into the existing infrastructure in the Town of Grand Valley. Specifically, detailed design would establish the following:

- Type and location of the tank (e.g. glass fused steel storage tank located primarily above ground, rectangular cement tank located above ground or below ground, etc.);
- Additional treatment processes required upstream of the equalization tank (e.g. communitor, etc.);
- Regular maintenance required of the equalization tank (e.g. washing, etc.) and provisions to allow for required maintenance;
- Integration into the existing infrastructure, including the reuse of existing pumps and piping where possible; and

Draft

- Evaluation of existing utilities and standby power on the site.

For purposes of this conceptual level design, it is assumed a circular glass fused steel storage tank would be installed at the Emma St. SPS. A conceptual level site layout of equalization at the Emma St. SPS is included as Figure 3.2 and indicates that the site has sufficient space for construction of the equalization tank. Exact dimensions of the equalization tank and the optimal location on the site would be finalized during the detailed design.

Figure 3.1 Overview of Conceptual Level Layout for Equalization at the Emma St. SPS

Conceptual level capital costs were estimated for the installation of additional equalization volume at the Emma St. SPS. Conceptual level capital costs include installation the equalization tank, as well as allowances for excavation, piping, installation of a tank cleaning mechanism, and electrical works. These additional considerations are critical for the integration of the equalization tank into the existing infrastructure and SCADA system.

Conceptual level costs are generally considered to be accurate to -25% to +40%. Actual costs will depend on site specific factors, such as soil and groundwater conditions, the engineering design applied, construction conditions at the time of tendering, and the extent of additional upgrades to the works that may be included in the final design. Capital costs include a 30% allowance for contingency and a 12%

allowance for engineering and approvals. A summary of conceptual level capital costs for each equalization option is summarized in Table 3.2.

Table 3.3 Summary of Conceptual Level Capital Cost Estimates for Equalization at the Emma St. SPS

Item	Option 1 (Sufficient Capacity for Scenario I Flows)	Option 2 (Sufficient Capacity for Scenario III Flows)
General/Miscellaneous	$130,000	$155,000
Equalization Tank	$1,302,000	$1,545,000
Sub Total	$1,432,000	$1,700,000
Contingency (30%)	$429,000	$510,000
Engineering (12%)	$172,000	$204,000
Estimated Equalization Capital Costs [1]	**$2,033,000**	**$2,414,000**

Notes:

1. All costs are conceptual level opinions of probable costs and are considered to be accurate to within -25 to +40 percent and are exclusive of HST.

Draft

4. SUMMARY AND CONCLUSIONS

Based on the capacity assessment of the Grand Valley WPCP, and on projections of future flows and loadings, the capacity of the overall facility is limited by the peak flow treatment capacity. Through installation of additional equalization at the Emma St. SPS, peak flows to the plant may be reduced, thereby making it feasible to pursue a plant rerating to increasing the rated capacity, potentially up to an ADF capacity of 1,555 m^3/d.

There appears to be sufficient space at the existing Emma St. SPS to construct additional equalization. Estimated costs for equalization will depend on several factors, including the type of equalization tank selected and additional equipment required to integrate the equalization tank into existing infrastructure.

For purposes of this analysis, two equalization options were evaluated:

- Option 1: Sufficient equalization volume to facilitate plant rerating to Scenario I flows and loads (ADF of 1,279 m^3/d).
- Option 2: Sufficient equalization volume to facilitate plant rerating to Scenario III flows and loads (ADF of 1,555 m^3/d).

The estimated costs for equalization ranged from approximately $2.03 million (Option 1) to $2.41 million (Option 2). Construction of additional equalization volume would be carried out as a Schedule B activity under the Municipal Class EA process, therefore requiring an evaluation of alternative solutions and consultation with the public and with relevant review agencies.

Draft

5. REFERENCES

1. R.J. Burnside & Associates Limited. Grand Valley Wastewater Treatment Plant Operations Manual. 2015.
2. XCG Consulting Limited. Grand Valley WPCP Re-rating Feasibility Study Proposed Design Flows and Loads. 2015.
3. XCG Consulting Limited. Grand Valley WPCP Re-rating Feasibility Study Updated Design Basis. 2016.
4. XCG Consulting Limited. Grand Valley WPCP Headworks Hydraulics Analysis. 2016.
5. XCG Consulting Limited. Grand Valley WPCP Re-rating Feasibility Study. Secondary Clarifier, Tertiary Filter, and Disinfection Stress Test Results. 2016.

Draft

APPENDIX B

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY IMPACT OF ADDITIONAL EQUALIZATION VOLUME

Draft

XCG CONSULTING LIMITED
T 905 829 8880 F 905 829 8890 | toronto@xcg.com
2620 Bristol Circle, Suite 300, Oakville, Ontario, Canada L6H 6Z7

XCG File No.: 3-252-57-01
December 6, 2016

GRAND VALLEY WPCP RE-RATING FEASIBILITY STUDY IMPACT OF ADDITIONAL EQUALIZATION VOLUME

Draft

Prepared for:

TOWN OF GRAND VALLEY
5 Main Street, North
Grand Valley, Ontario
L9W 5S6
Attention: Jane Wilson

Prepared by:

XCG CONSULTING LIMITED
Suite 300, 2620 Bristol Circle
Oakville, Ontario
L6H 6Z7

TABLE OF CONTENTS

TABLES

FIGURES

Draft

1. INTRODUCTION

The Grand Valley Water Pollution Control Plant (WPCP) provides treatment for wastewater generated in the community of Grand Valley within the Town of Grand Valley (Town). The plant is currently operated by the Ontario Clean Water Agency (OCWA) under the Ministry of Environment and Climate Change (MOECC) Certificate of Approval (C of A) No. 9706-7KWQ57, issued on February 2, 2009. The quality and quantity of effluent currently discharged by the existing WPCP is regulated by the C of A. The Grand Valley WPCP has a rated average capacity of 1,244 m^3/d.

The Town has initiated an investigation to analyze the potential to re-rate the existing Grand Valley WPCP to provide additional treatment capacity and to defer the facility's next upgrade and expansion. The Town has retained XCG Consulting Limited (XCG) to undertake a capacity assessment of the Grand Valley WPCP to evaluate the potential to re-rate the plant.

Preliminary results of the assessment indicate the plant treatment capacity may be limited by peak flows capacity. As such, XCG conducted an analysis to evaluate the impact that additional equalization volume may have on the overall capacity of the Grand Valley WPCP. The purpose of this technical memorandum is to present results of that analysis.

Draft

2. GRAND VALLEY WPCP BACKGROUND INFORMATION

2.1 Existing Treatment Process

Raw sewage flows from the collection system are conveyed to the Grand Valley WPCP from the Emma St. sewage pumping station (SPS) via a forcemain. The Emma St. SPS is equipped with the following equipment:

- Two variable frequency drive (VFD) pumps (one duty and one standby), each with a rated capacity of 88.9 L/s (7,680 m^3/d).
- One VFD jockey pump with a rated capacity of 29.5 L/s (2,550 m^3/d).
- One wet will, with approximate volume of 125 m^3.

The jockey pump will not operate at peak flows. As such, the firm capacity of the Emma St. SPS is approximately 7,680 m^3/d. Over the review period (January 2012 – May 2016) there are no records of raw sewage bypassing at the Emma St. SPS or at the Grand Valley WPCP.

The Grand Valley WPCP receives septage at the septage receiving station. The septage receiving station removes solids from the raw septage using a combination of grinding, washing, and dewatering. The septage is then discharged to the plant headworks, upstream of the plant screens.

Plant influent raw wastewater flow consists of wastewater from the following sources:

- Raw wastewater from the Emma St. SPS;
- Septage from the on-site receiving station;
- Tertiary filter backwash; and,
- Digester supernatant.

Tertiary filter backwash and digester supernatant are transferred back to the head of the plant via an on-site pumping station. All flows are combined at the head of the plant, upstream of the plant headworks.

Headworks at the Grand Valley WPCP consists of a mechanical bar screen and two vortex grit separators. A manual screen also exists in parallel to the mechanical screen, and can be used as needed during peak flows or to isolate the mechanical screen. Flow to the manual screen is controlled using gates. High water levels in the screening channel can overflow the control gate, thereby initiating an emergency bypass of the mechanical screens.

Headworks effluent flow is discharged to a splitter box, where flow is directed to the aeration tanks, or to a bypass channel. Sustained peak flows in excess of 64 L/s (5,530 m^3/d) for greater than 10 minutes are directed to the bypass channel and into the 400 m^3 equalization tank (storm tank). From the equalization tank, flow can be returned to the head of the plant through the on-site pumping station. Bypass flows in excess of the equalization tank capacity are disinfected and discharged. There have been no recorded plant bypasses at the Grand Valley WPCP.

Draft

Secondary treatment at the Grand Valley WPCP consists of three aeration tanks and two secondary clarifiers. Oxygen is provided to each aeration tank through fine bubble diffusers. Alum is added immediately upstream of the secondary clarifiers for chemical phosphorus removal. Activated sludge is separated from the treated stream in the secondary clarifiers. Return activated sludge (RAS) is returned to the raw wastewater upstream of the aeration tanks. Waste activated sludge (WAS) is pumped to the aerobic digester located on-site. RAS and WAS are pumped from the same location in the secondary clarifier. Overflow from the secondary clarifiers is passed through one of four tertiary filters at the plant. Filter effluent is disinfected using ultraviolet (UV) radiation, then discharged to the Grand River. Waste activated sludge is digested and thickened on-site in the aerobic digester. Thickened sludge is pumped to the on-site biosolids storage tank, then trucked offsite for disposal.

Wastewater flow is measured at several locations at the plant. Raw wastewater from the collection system is metered at the Emma St. SPS. Wastewater flows from septage and the on-site pumping station are separately metered. Collectively, they represent the plant influent flow. Effluent flow from the Grand Valley WPCP is measured by a V-notch weir, downstream of the UV disinfection.

A process flow diagram of the Grand Valley WPCP is presented in Figure 2.1.

2.2 Plant Design Basis

For purposes of this evaluation, flows and loads to the Grand Valley WPCP were developed for three distinct scenarios. Details of each scenario are presented briefly below:

- Scenario I: Full completion of planned residential developments;
- Scenario II: A 15% increase above the current C of A rated ADF (1,430 m^3/d); and,
- Scenario III: A 25% increase above the current C of A rated ADF (1,555 m^3/d).

A summary of the updated flow design basis is given in Table 2.1 (XCG, 2016). For simplicity, the previous design basis (XCG, 2015) has not been presented in the table. This table represents raw the projected raw influent flow from the collection system to the Grand Valley WPCP, and does not include any recycle flow from the on-site pumping station. It is important to note the projected peak flows for all three scenarios exceed the existing rated capacity of the Emma St. SPS (7,680 m^3/d). Therefore, the Emma St. SPS may require upgrades at future flows provided that existing peak flows are not abated by any I/I reduction strategies. An extensive review of the Emma St. SPS capacity was not conducted as part of this review. Further, it is assumed that future peak flows to the Grand Valley WPCP will not be inhibited by the pumping capacity of the Emma St. SPS.

Draft

Table 2.1 Summary of Raw Influent Flow from the Collection System (XCG, 2016)

Parameter	Scenario I	Scenario II	Scenario III
Population	2,919	3,252	3,527
ADF	1,279 m^3/d	1,430 m^3/d	1,555 m^3/d
MDF	5,839 m^3/d	6,169 m^3/d	6,442 m^3/d
MDF Factor	4.6	4.3	4.1
PIF	7,811 m^3/d	8,291 m^3/d	8,684 m^3/d
PIF Factor	6.1	5.8	5.6

For purposes of this analysis, evaluation of the required equalization volume will be based on the projected maximum day flow through the treatment plant. It is important to note that backwash flow from the tertiary filters and supernatant from the on-site digester is discharged to the on-site pumping station where it is pumped to the head of the plant upstream of the plant headworks. As such, maximum day and peak instantaneous flows through the treatment plant are greater than those given in Table 2.1.

The maximum design backwash flow rate from the existing tertiary filters is 390 m^3/d (R.J. Burnside, 2015). For purposes of this analysis, it is assumed the digester is not supernated during a peak flow event. Table 2.2 summarizes the projected maximum day flow through the plant considering contributions from the Emma St. SPS (i.e. raw influent from the collection system) and from the on-site pumping station (i.e. tertiary filter backwash flow).

Draft

Table 2.2 Summary of Peak Flow through the Grand Valley WPCP Headworks

Maximum Day Flow	Scenario I	Scenario II	Scenario III
Emma St. SPS (Collection System)	5,839 m^3/d	6,169 m^3/d	6,442 m^3/d
On-site Pumping Station (Filter Backwash)		390 m^3/d	
Total Projected Maximum Day Flow	6,229 m^3/d	6,559 m^3/d	6,832 m^3/d

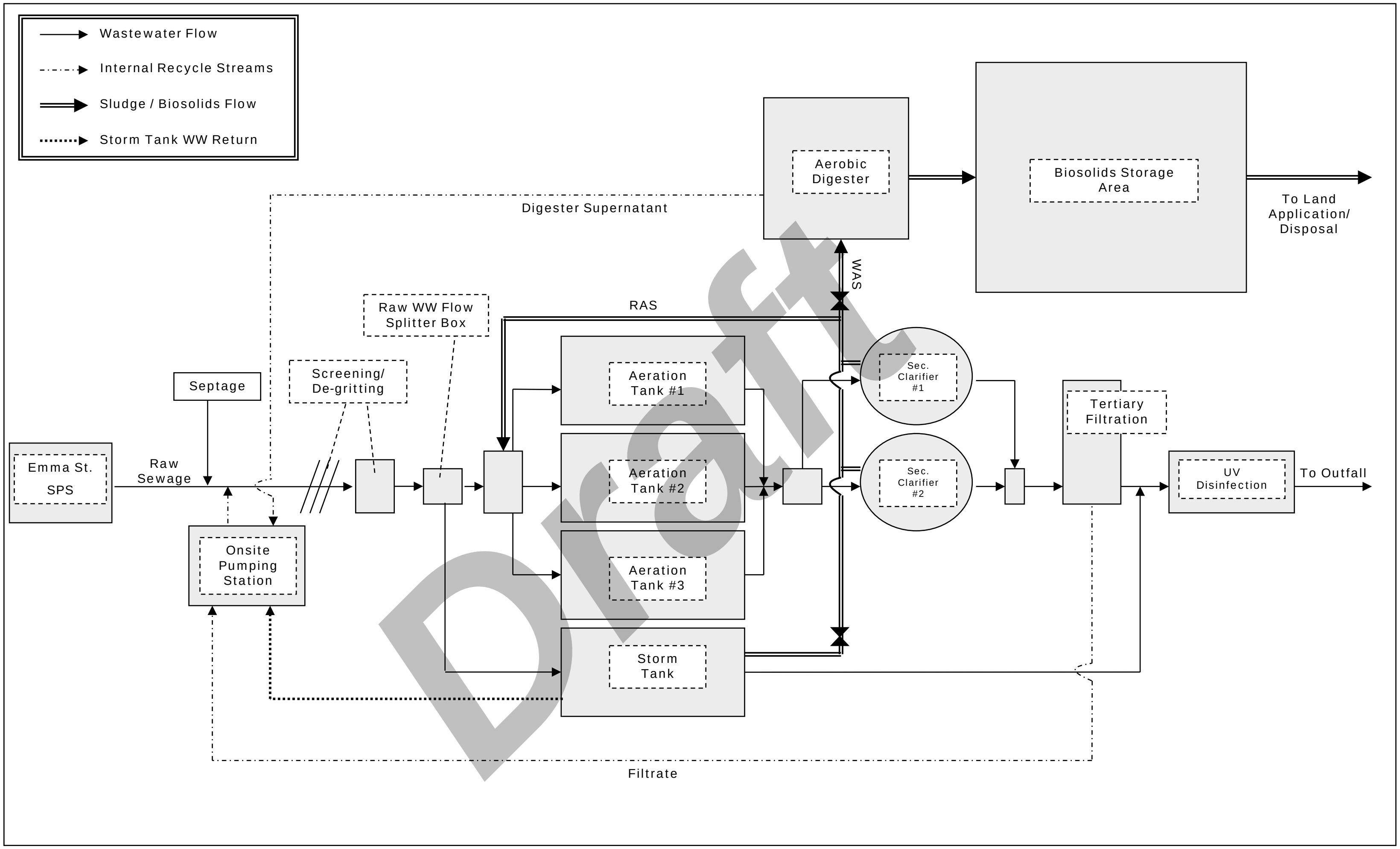


Figure 2.1 Process Flow Schematic – Grand Valley WPCP

3. DETAILS OF ADDITIONAL EQUALIZATION FOR THE GRAND VALLEY WPCP

Currently, equalization for the Grand Valley WPCP is provided by a 400 m^3 storm tank located on-site at the Grand Valley WPCP. It is assumed this storm tank would continue to be used in the future.

For purposes of this investigation, two equalization options were developed and evaluated. Details of each equalization option is included in Table 3.1.

Table 3.1 Summary of Equalization Options

Option	Details
Option 1	• Provide sufficient equalization volume to facilitate re-rating of the Grand Valley WPCP to the Scenario I flows and loads.
Option 2	• Provide sufficient equalization volume to facilitate re-rating of the Grand Valley WPCP to the Scenario III flows and loads.

The purpose of this section is to present considerations for the construction of additional equalization volume in the Town of Grand Valley.

3.1 Impact of Equalization Location

There are two potential locations where additional equalization could be constructed in Grand Valley, Ontario: at the Emma St. SPS and/or at the Grand Valley WPCP. Although space is available on-site at the Grand Valley WPCP, construction of additional equalization volume may limit the land available for future expansion of the plant. For purposes of this study, it is assumed that additional equalization installed at the Grand Valley WPCP would divert flow from the same location as the existing equalization tank. As such, projected peak flows through the plant headworks and from the Emma St. SPS would not be reduced via the installation of additional equalization volume at the WPCP site.

Conversely, the Emma St. SPS is located at the site of the old wastewater treatment plant. The majority of infrastructure has been removed from the site and minimal expansion of the existing infrastructure is expected to be required to meet future flows. As such, there is significant land available for the construction of additional equalization as required. By constructing equalization volume at the Emma St SPS, peak flows requiring conveyance through the SPS and, by extension, influent peak flows to the WPCP would be reduced.

An analysis of the hydraulic treatment capacity of the existing plant headworks (i.e. screening and grit removal) has also been completed (XCG, 2016). The results indicate that the hydraulic capacity of the existing headworks exceeds the projected Scenario III peak flows without the installation of any additional equalization volume.

As noted in Table 2.1, projected peak flows from the collection system exceed the current rated pumping capacity of the Emma St. SPS. Installation of equalization volume at Emma St. would reduce peak flows below the existing rated capacity of the raw influent pumps. Conversely, if additional equalization volume is installed at the

Grand Valley WPCP, peak flows from the Emma St. SPS would not be reduced. As such, installation of equalization volume at the Grand Valley WPCP would need to be accompanied by a detailed investigation of the pumping capacity of the Emma St. SPS and hydraulics of the forcemain between the plant and pumping station.

Therefore, to avoid the potential of additional required upgrades to the Emma St. SPS and/or the forcemain, this analysis has assumed additional equalization volume would be installed at the Emma St. SPS. Ultimate selection of the location and volume of additional equalization would be finalized during the detailed design.

3.2 Analysis of Projected Peak Flows and Estimate of Required Equalization Volumes

The following assumptions were made to develop an estimate of the required equalization volume for each equalization option:

- Sufficient volume is required to provide 24-hours of equalization at a simulated future peak flow event.
- Detailed flow characteristics of the historical peak flow event (recorded on April 14, 2014) are representative of future peak flow events.

The peak treatment capacity of the Grand Valley WPCP was evaluated through stress testing of the secondary clarifiers, tertiary filters, and UV disinfection system. Results were previously presented in the Secondary Clarifier, Tertiary Filter, and Disinfection Stress Test Results Technical Memorandum (XCG, 2016). Based on the results, the estimated peak treatment capacity of the plant including flow from the tertiary filter backwash is approximately 4,400 m^3/d and is limited by the secondary clarifiers.

Table 3.1 summarizes the estimated required equalization volume for each equalization option that maintains the projected peak flow through secondary treatment at the WPCP to less than 4,400 m^3/d.

Table 3.2 Summary of Estimated Required Equalization Volume

	Option 1 (Sufficient Capacity for Scenario I Flows)	Option 2 (Sufficient Capacity for Scenario III Flows)
Projected MDF	6,229 m^3/d	6,832 m^3/d
Total Estimated Equalization Volume Required	1,900 m^3	2,500 m^3
Existing Equalization Volume (1)	400 m^3	
Additional Equalization Volume Required at Emma St SPS	1,500 m^3	2,100 m^3
Estimated Equalized Peak Flow (2)	4,327 m^3	4,330 m^3

Notes:
1. Volume of existing storm tank at the Grand Valley WPCP.
2. Due to size of the proposed equalization volume for each option, the projected equalized maximum day and peak hour flows for each option are equal.

Draft

3.3 Installation Considerations and Capital Cost Estimations

As previously discussed, it has been assumed that additional equalization volume would be constructed at the Emma St. SPS located upstream of the Grand Valley WPCP.

Installation of additional equalization volume can be carried out as a Schedule B activity under the Municipal Class Environmental Assessment Process as per the following text:

"Establish sewage flow equalization tankage in existing sewer system or at existing sewage treatment plants, or at existing pumping stations for influent and/or effluent control"

As a Schedule B project, Phase 1 and Phase 2 of the Class EA process must be completed prior to implementation of the project (i.e. construction). Brief requirements of each Phase are given below.

Phase 1

During this phase, the problem or opportunity must be identified and described. Projects which are expected to generate significant public interest can also begin the public consultant process.

Phase 2

During this phase, potential alternative solutions will be identified and evaluated. Solutions will consider the size (volume) and location of additional equalization. This Phase will also include mandatory consultation with relevant review agencies and stakeholders (e.g. MOECC, GRCA, First Nations, etc.) and the public.

At the completion of Phase 2, the entire planning process (i.e. Phase 1 and Phase 2 activities) will be summarized and placed on file for a period of thirty (30) days. A notice of completion will be issued to review agencies and to the public.

Assuming no request for an Order is received during the review period, the Town may proceed with the design and construction of the equalization tank. Detailed design of the equalization tank would need to consider the integration of the equalization tank into the existing infrastructure in the Town of Grand Valley. Specifically, detailed design would establish the following:

- Type and location of the tank (e.g. glass fused steel storage tank located primarily above ground, rectangular cement tank located above ground or below ground, etc.);
- Additional treatment processes required upstream of the equalization tank (e.g. communitor, etc.);
- Regular maintenance required of the equalization tank (e.g. washing, etc.) and provisions to allow for required maintenance;
- Integration into the existing infrastructure, including the reuse of existing pumps and piping where possible; and

Draft

- Evaluation of existing utilities and standby power on the site.

For purposes of this conceptual level design, it is assumed a circular glass fused steel storage tank would be installed at the Emma St. SPS. A conceptual level site layout of equalization at the Emma St. SPS is included as Figure 3.2 and indicates that the site has sufficient space for construction of the equalization tank. Exact dimensions of the equalization tank and the optimal location on the site would be finalized during the detailed design.

Figure 3.1 Overview of Conceptual Level Layout for Equalization at the Emma St. SPS

Conceptual level capital costs were estimated for the installation of additional equalization volume at the Emma St. SPS. Conceptual level capital costs include installation the equalization tank, as well as allowances for excavation, piping, installation of a tank cleaning mechanism, and electrical works. These additional considerations are critical for the integration of the equalization tank into the existing infrastructure and SCADA system.

Conceptual level costs are generally considered to be accurate to -25% to +40%. Actual costs will depend on site specific factors, such as soil and groundwater conditions, the engineering design applied, construction conditions at the time of tendering, and the extent of additional upgrades to the works that may be included in the final design. Capital costs include a 30% allowance for contingency and a 12%

allowance for engineering and approvals. A summary of conceptual level capital costs for each equalization option is summarized in Table 3.2.

Table 3.3 Summary of Conceptual Level Capital Cost Estimates for Equalization at the Emma St. SPS

Item	Option 1 (Sufficient Capacity for Scenario I Flows)	Option 2 (Sufficient Capacity for Scenario III Flows)
General/Miscellaneous	$130,000	$155,000
Equalization Tank	$1,302,000	$1,545,000
Sub Total	$1,432,000	$1,700,000
Contingency (30%)	$429,000	$510,000
Engineering (12%)	$172,000	$204,000
Estimated Equalization Capital Costs (1)	**$2,033,000**	**$2,414,000**

Notes:
1. All costs are conceptual level opinions of probable costs and are considered to be accurate to within -25 to +40 percent and are exclusive of HST.

Draft

4. SUMMARY AND CONCLUSIONS

Based on the capacity assessment of the Grand Valley WPCP, and on projections of future flows and loadings, the capacity of the overall facility is limited by the peak flow treatment capacity. Through installation of additional equalization at the Emma St. SPS, peak flows to the plant may be reduced, thereby making it feasible to pursue a plant rerating to increasing the rated capacity, potentially up to an ADF capacity of 1,555 m^3/d.

There appears to be sufficient space at the existing Emma St. SPS to construct additional equalization. Estimated costs for equalization will depend on several factors, including the type of equalization tank selected and additional equipment required to integrate the equalization tank into existing infrastructure.

For purposes of this analysis, two equalization options were evaluated:

- Option 1: Sufficient equalization volume to facilitate plant rerating to Scenario I flows and loads (ADF of 1,279 m^3/d).
- Option 2: Sufficient equalization volume to facilitate plant rerating to Scenario III flows and loads (ADF of 1,555 m^3/d).

The estimated costs for equalization ranged from approximately $2.03 million (Option 1) to $2.41 million (Option 2). Construction of additional equalization volume would be carried out as a Schedule B activity under the Municipal Class EA process, therefore requiring an evaluation of alternative solutions and consultation with the public and with relevant review agencies.

Draft

5. REFERENCES

1. R.J. Burnside & Associates Limited. Grand Valley Wastewater Treatment Plant Operations Manual. 2015.
2. XCG Consulting Limited. Grand Valley WPCP Re-rating Feasibility Study Proposed Design Flows and Loads. 2015.
3. XCG Consulting Limited. Grand Valley WPCP Re-rating Feasibility Study Updated Design Basis. 2016.
4. XCG Consulting Limited. Grand Valley WPCP Headworks Hydraulics Analysis. 2016.
5. XCG Consulting Limited. Grand Valley WPCP Re-rating Feasibility Study. Secondary Clarifier, Tertiary Filter, and Disinfection Stress Test Results. 2016.

Draft