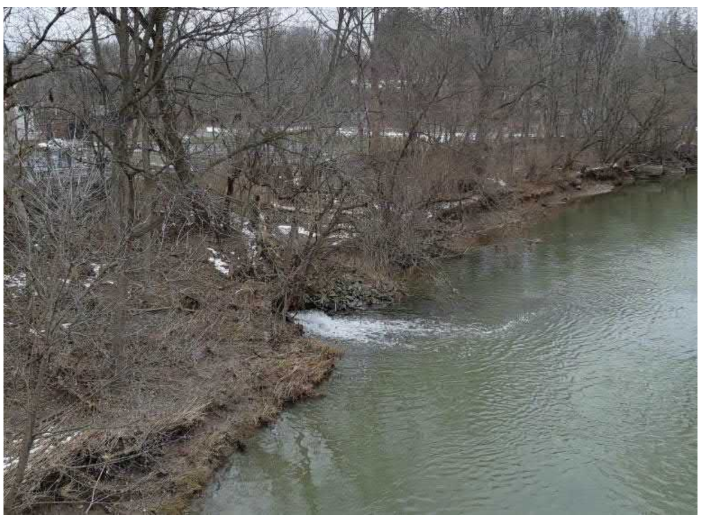

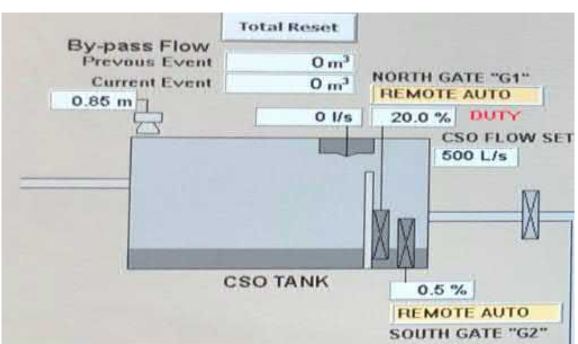

City of St. Thomas Pollution Prevention Control Plan Study

Final Report

January 28, 2022

Prepared for:

ST. THOMAS
THE RAILWAY CITY

arva

CANADA'S BEST MANAGED COMPANIES Platinum member

St. Thomas Pollution Prevention Control Plan
Final Report

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1.0 INTRODUCTION

The City of St. Thomas (City) has retained R.V. Anderson Associates Limited (RVA) for the preparation of a Pollution Prevention Control Plan (PPCP) as part of the City's ongoing efforts to improve the performance of their sanitary and storm sewer infrastructure. In addition, it would provide the City with a road map for implementation of infrastructure and operational improvements that will mitigate the impacts of wet weather sewer system overflows on the environment. Key deliverables include:

- Undertaking the PPCP using Master Plan Approach # 1 per the MEA Municipal Class EA document (October 2000, as amended in 2007, 2011 & 2015);
- Developing a hydraulic model of the City's sanitary sewer system to characterize issues and develop solutions;
- Prepare a PPCP Report Document; and
- Present findings of the PPCP to Senior City staff and Council.

This PPCP Report compiles the public feedback, analysis undertaken in the Technical Memoranda to develop an implementation plan involving ongoing study to develop a better understanding of the collection system, operational changes and recommended capital works refurbishment and improvement projects.

2.0 PLANNING FRAMEWORK

2.1 Overview

This PPCP is being undertaken in accordance with the Master Planning requirements of the Municipal Class Environmental Assessment (MCEA) October 2000, as amended in 2007 2011, and 2015 (Class EA Document). A Class EA is a planning document which sets out the process that a proponent must follow to meet the requirements of the Ontario Environmental Assessment Act for a class or category of infrastructure projects. Projects are divided into schedules based on the type of projects and activities. Schedules are categorized as A, A+, B and C with reference to the magnitude of their anticipated environmental impact. These are described briefly in the following paragraphs.

Schedule A projects have minimal adverse environmental effects and are pre-approved and therefore many proceed to implementation without the full planning process. Projects include municipal maintenance and operational activities.

Schedule A+ projects require some type of public notification to occur for pre-approved projects. Although the public is to be notified, no formal public consultation process is required.

Schedule B projects are those which have a potential for adverse environmental effects. A screening process must be undertaken which includes consultation with directly affected public and relevant review agencies. Projects generally include improvements and minor expansions to existing facilities. The project process must be filed, and all documentation prepared for public and agency review.

Schedule C projects have the potential for significant environmental effects and must follow the full planning and documentation procedures specified in the Class EA document. An Environmental Study Report (ESR) must be prepared and filed for review by public and review agencies. Projects generally include the construction of new facilities and major expansions to existing facilities.

2.2 Planning Process

There are five key elements in the Class EA planning process. These include:

1. Phase 1 – Identification of problem (deficiency) or opportunity;
2. Phase 2 – Identification of alternative solutions to address the problem or opportunity. Public and review agency contact is mandatory during this phase and input received along with information on the existing environment is used to establish the preferred solution. It is at this point that the appropriate Schedule (B or C) is chosen for the undertaking. If Schedule B is chosen, the process and decisions are then documented in a Project File. Schedule C projects proceed through the following Phases;
3. Phase 3 – Examination of alternative methods of implementing the preferred solution established in Phase 2. This decision is based on the existing environment, public and review agency input, anticipated environmental effects and methods of minimizing negative effects and maximizing positive effects;
4. Phase 4 – Preparation of an Environmental Study Report summarizing the rationale, planning, design, and consultation process of the project through Phases 1-3. The ESR is then to be made available to agencies and the public for review; and

5. Phase 5 – Completion of contract drawings and documents. Construction and operation to proceed. Construction to be monitored for adherence to environmental provisions and commitments. Monitoring during operation may be necessary if there are special conditions.

The overall process is shown in Figure 2.1.

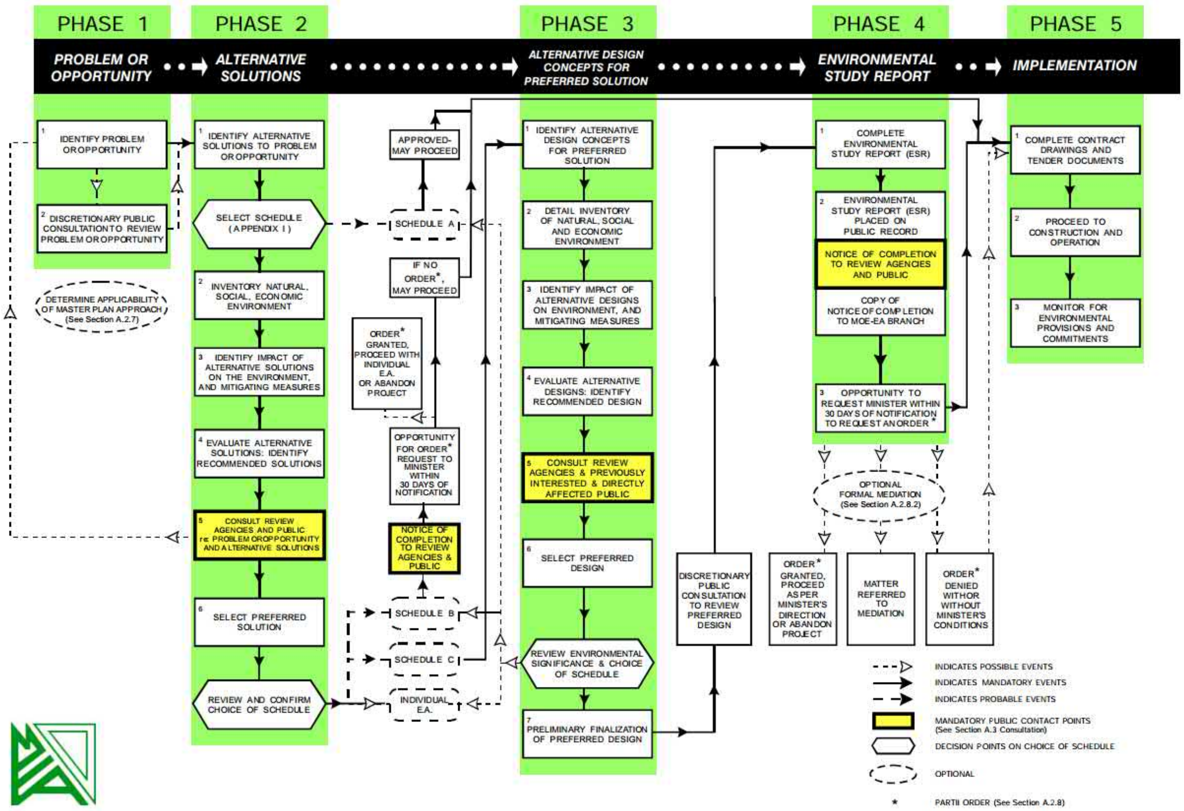


Figure 2.1 – Municipal Class EA Planning and Design Process

2.3 Master Planning

Master Plans are not subject to requests from the public or agencies for a Minister's Order (Part II Order). However, individual projects identified within an EA process can be subject to a Part II Order. As such, the Master Plan can be implemented following Council approval. The MEA offers four approaches for undertaking a master plan and based on our review, Municipal Class EA Approach #1 appears to be the most appropriate. This approach involves the preparation of a Master Plan document at the conclusion of Phases 1 and 2 of the Class EA process. Per the Class EA Document, Approach #1 allows for:

> "The preparation of a Master Plan document at the conclusion of Phases 1 and 2 of the Municipal Class EA process. The Master Plan document would be made available for public comment prior to being approved by the municipality.

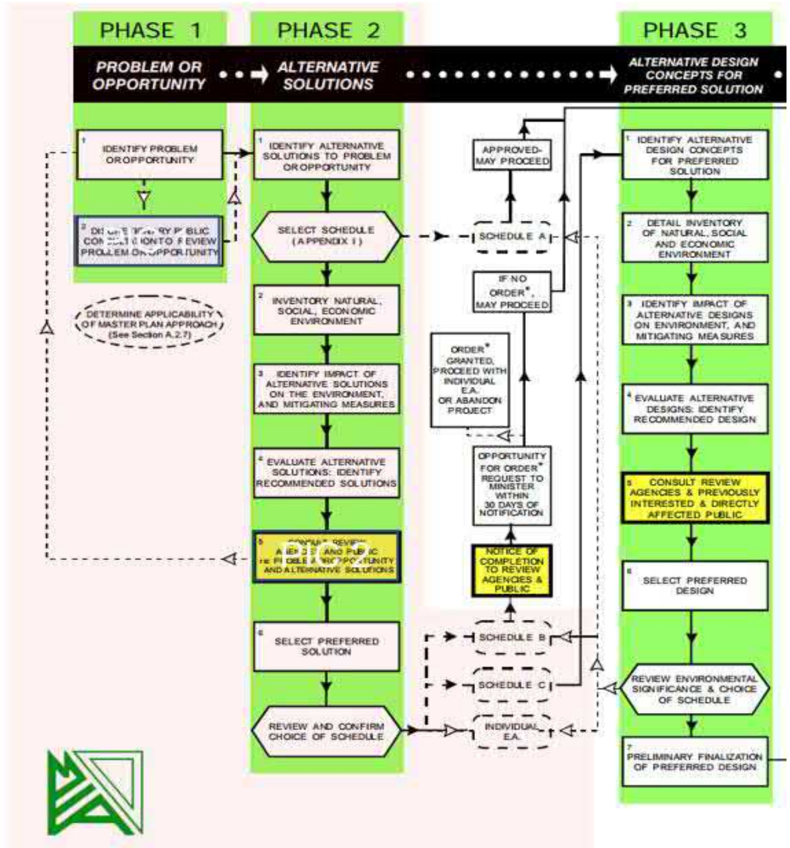


Figure 2.2 –Municipal Class EA Planning and Design Process (highlighted) followed by the PPCP

Typically, the Master Plan would be done at a broad level of assessment thereby requiring more detailed investigations at the project-specific level to fulfill the Municipal Class EA documentation requirements for the specific Schedule B and C projects identified within the Master Plan.

The Master Plan would therefore become the basis for and be used in support of future investigations for the specific Schedule B and C projects identified within it. Schedule B projects would require the filing of the Project file for public review while Schedule C projects would have to fulfill Phases 3 and 4 prior to filing an Environmental Study Report (ESR) for public review."
Figure 2.2 shows the scope of the Class EA process to be undertaken by the PPCP.

2.4 Project Problem/Opportunity Statement

In letters sent to agencies, stakeholders and the public, the following was included which defines the project problem/opportunity statement:

> "The PPCP will be a part of the City's ongoing efforts to improve the performance of our sanitary and storm sewer infrastructure.
>
> The PPCP is aimed at reducing sewer system overflows (SSO's) and bypasses of pumping stations and the water pollution control plant during extreme weather events.
>
> The PPCP will act as a master planning level tool that provides St. Thomas with guidance for capital planning and project implementation for the next 20 years and beyond."

The consultation process is an integral component of the Municipal Class EA process. Effective communication with Aboriginal communities, agencies, stakeholders, and the public can reduce or avoid controversy that can ultimately lead to project delays and general discontent of project stakeholders. RVA, in consultation with City staff identified stakeholders, agencies and Aboriginal communities that may have an interest in the study, the methods of contact, and the timing of contact for this project. This section details the consultation process followed by the Master Plan. **Appendix 1.1** contains the public notices that have been filed as part of this process.

2.5 Stakeholder Consultation

Potential stakeholders included but were not limited to:

- Public – This includes individual members of the public including property owners who may be affected by the project, individual citizens who may have a general interest in the project, special interest groups, community representatives, and developers;
- Review agencies – This includes government agencies who represent the policy positions of their respective departments, ministries, authorities, or agencies; and
- City of St. Thomas internal departments.

Members of the public were notified of project commencement and invited to attend Public Information Centres (PICs) by way of notices published in a local area newspaper.

A list of relevant agencies and the appropriate contact person was developed at the onset of the project. Throughout the process, these contacts were sent letters notifying them of the project progress. **Appendix 1.2** contains the contact list developed for this project. **Appendix 1.3** contains responses from both the public and agencies.

2.6 Aboriginal Consultation

Based on discussions and recommendations provided by the Ministry of Environment and Conservation of Parks (MECP) regional office, the City contacted Aboriginal Affairs and Northern Development Canada (AANDC) and the Ministry of Aboriginal Affairs (MAA) separately from the general notifications sent to review agencies. The purpose of the contact was to request which, if any, Aboriginal communities may be affected by the project alternatives. The additional information may result from existing claims not readily available to the public. Information provided ensures the appropriate communities have been included in the contact lists for the duration of the Class EA project. These government agencies were not included in general notifications. The Aboriginal agency contact letters are in **Appendix 1.2**.

2.7 Public Information Meetings

Public Information Centres are a method to communicate with the public, interested parties and review agencies. For this project two PICs were held.

PIC 1 – Was held as a virtual meeting due to COVID-19 restrictions from 5 PM to 7 PM on October 21, 2020. The presentation handout and comment sheet given to attendees are attached as are any comments received in **Appendix 1.4**; and

PIC 2 – Was held as a virtual meeting due to COVID-19 restrictions from 4 PM to 5 PM on December 1, 2021. The presentation handout and comment sheet given to attendees are attached as are any comments received in **Appendix 1.5**.

2.8 Notices

The Notice of Study Commencement and PIC 1 as well as the notice of PIC 2 were published on the City of St. Thomas' Notice to Residents website (Notices to Public). Letters were emailed to all identified project contacts.

The Notice of Completion was sent out to agencies and interested parties informing them that the PPCP had been completed via email as well as being published on the City of St. Thomas' Notice to Residents website (Notices to Public). Copies of the notices are included in **Appendix 1.1**.

3.0 PRINCIPLES FOR PPCP

To review the issues and opportunities in St. Thomas with regards to pollution prevention over the 20-year planning period, guiding principles were established at the outset of the project by the City/RVA team. The intent of these Guiding Principles is to:

1. Allow City to translate Strategic Plan and Risk Management policies into more specific PPCP priorities;
2. Allow City to identify other servicing priorities;
3. Allow for each proposed solution to be tested based on whether they meet the City's priorities;
4. Allow for comparison and ranking of proposed solutions (# of principles met, to what degree, etc.); and
5. Allow City to have a consistent framework to evaluate PPCP issues.

The Guiding Principles are as follows:

1. The PPCP should achieve pollution control in a systematic and sustainable manner;
2. PPCP solutions should fit into the City's Risk Management plan;
3. The PPCP should support the City's Corporate Strategic Plan;
4. The PPCP solution should integrate the collection system, pumping stations and the WPCP to achieve the F-5-5 requirements;
5. Attempts should be made to separate all combined sewers within City boundaries;
6. PPCP solutions should be fully funded through adequate planning, budgeting and identified revenue streams;
7. The PPCP should support the City's obligations under Source Water Protection Regulations and other mandated requirements;
8. Growth impacts including infill need to pay for its fair portion of the PPCP;
9. Utilize existing infrastructure efficiently as part of the PPCP solution;
10. The PPCP should favour solutions that minimize and reduce complexity;
11. Work cooperatively with MECP, Aboriginal Communities, other stakeholders, and the public to develop the PPCP solution; and
12. Look for opportunities to share costs with other municipalities or higher levels of government.

4.0 BACKGROUND INVESTIGATIONS AND STUDIES

4.1 Information Sources

The following information sources were identified and reviewed as part of this study:

- City of St. Thomas Edgeware Line Employment Lands– Servicing Study, July 2020;
- City of St. Thomas Positioned for Growth – Planning Justification Report, February 2020;
- St. Thomas Orchard Park Area Sanitary Sewer Flow Monitoring & Analysis, TQI, April 2020;
- Lands – Servicing Study City of St. Thomas Sewer Flow Monitoring Study, Flowmetrix Technical Services Inc, February 2018;
- City of St. Thomas Aldborough/Leger and Woodworth Wastewater Sanitary Catchments Inflow and Infiltration Study, Cole Engineering, December 2014;
- St. Thomas Water Pollution Control Plant – Annual Performance Reports, 2008-2018;
- City of Proposed Urban Expansion Areas, Infrastructure Master Plan Sanitary Sewer Servicing, November 2008;
- City of St. Thomas WWTP historic flow records from Annual Reports;
- City of St. Thomas pumping station historic runtime/overflow volume records; and
- WWTP historic flow records sewers (diameter, material, length, inverts) linked to the City's GIS data base.

As the first step of the development of the PPCP – Technical Memorandum (TM) #1: Existing Document Review and Summary was prepared with the objective of characterization of the existing wastewater system regarding its flow capacity, inflow, and infiltration (I&I) issues, overflows, and their impact on the environment. This document provides detailed descriptions of the St. Thomas Water Pollution Control Plan (WPCP), the Combined Sewer Overflow Storage Facility (CSO) and the pumping stations that direct flow to the CSO and WPCP.

4.2 Existing WPCP and CSO Facility

4.2.1 WPCP

St. Thomas WPCP services the City of St. Thomas and portions of the Municipalities of Southwold and Central Elgin. It is in St. Thomas at 40359 Bush Line and is bordered by Sunset Drive to the North-East, Bush Line to the North-West, and Kettle Creek to the South-West. The plant is owned and operated by the City of St. Thomas. It has a rated capacity of 27,300 m^3/d (316 L/S) and peak flow capacity of 54,600 m^3/d (632 L/s) (per Amended Environmental Compliance Approval Number 5385 AHYLTD Issue Date: February 16, 2017).

St. Thomas WPCP is a conventional activated sludge facility with three (3) separate treatment trains (Plant 2, Plant 3, and Plant 4). The original plant (Plant 1) was constructed in 1925 and is no longer in service. There have been several upgrades since that time. Plants 2 and 3 were constructed in 1960s, while Plant 4 was completed in two phases between 1980 to 2003. Each of the existing plants (2, 3 and 4) includes primary clarification, aeration, and secondary clarification processes. There is a common headworks facility and a common ultraviolet (UV) disinfection process. Standby power is provided for the facility. Treated effluent is discharged from the St. Thomas WPCP to Kettle Creek, located to the South-West of the facility. Effluent pumping is available for effluent discharge during periods of high creek levels.

4.2.2 CSO Facility

A CSO facility was constructed in 2001 to mitigate wet weather peaks experienced at the WPCP and reduce overflows in the collection system. The facility is located northeast of Sunset Drive and Bush Line in the Mill Creek Valley, immediately upstream of the WPCP on the main sewer leading from the City's sewershed. The inline CSO facility is 290 m long with a storage capacity of 4,000 m^3 and includes inlet, outlet, and overflow control structures.

The purpose of this tank is to control and mitigate peak flows to the WPCP, biological process upsets and prevent plant overflow events. The design allows the normal dry weather flow to pass unimpeded at a velocity that is adequate to maintain self-cleansing conditions. In the event of an overflow, the discharge enters Mill Creek upstream of the WPCP. The actuated gates to the outlet of this CSO Tank are set to limit the peak flow to the WPCP at 500 L/s. This limit was selected as the WPCP's grit chamber overflows at flows exceeding 500 L/s, creating hazardous conditions and safety issues at the WPCP. As the instantaneous flow starts exceeding this limit, the actuated gates adjust the openings to limit the outflow to the set point. This makes the excess flow volume accumulate in the CSO leading to a rise in the liquid level in the same. In cases of sustained peak flows exceeding 500 L/s, the liquid level rises to the overflow elevation of the CSO causing it to overflow to Mill Creek through a bypass line.

Figure 4.1 – CSO Facility Under Construction (RVA Photo)

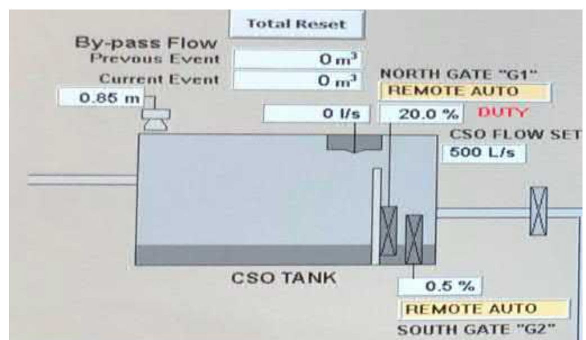


Figure 4.2 – CSO Facility Operation (City SCADA Screen)

4.3 Characterization of the Existing Wastewater System

The key components covered for characterizing the existing system in this memo include:

1. The current state of the collection system with regards to influence of extraneous flows via inflow and infiltration (I&I):
2. The ability of the collection system to convey normal and wet weather flows;
3. Quantity and quality of system overflows and by-passes;
4. The natural environment;
5. The impact of existing system deficiencies on the natural environment; and

6. Recommendations for short-term remedial measures and further investigations for a long-term PPCP.

4.3.1 Collection System

Currently, the City has approximately 2.4 km of combined sewers in its inventory and the *10 Year Capital Plan – 2020 to 2029* shows that most of these sewers will be separated in the next 10 years. In addition, the collection system has 16 sewage pumping stations. Table 4.1 gives a summary of each of the pumping stations with regards to its age, equipment details (make, model, and capacity), and operational configuration. See Figure 4.1 for City's sewerage system map.

Table 4.1 – St. Thomas Pumping Stations Data

Pumping Station	Construction Date	Make and Model of the Pumps	Duty/ Stand by	Firm capacity (L/s)	TDH (m)
Axford	1997	Gorman-Rupp ECM	1/1	56.6	8.9
Burwell Rd	1993	ITT Flygt 3170.180	1/1	44	30
Confederation Dr	1968	Smith & Loveless	1/1	67	NA
Crescent Ave.	1988	Hydromatic Pentair	1/1	16	9.54
Elm St.	2018	Flygt 3153	1/1	44.35	13.1
Harper Rd	1973	Gorman-Rupp	1/1	21	9.1
Karen St.	2011	Flygt 3153	1/1	43.2	NA
Lynhurst	1996	Flygt 3102	1/1	23	NA
Parkside Dr.	1970	Flygt CP3127	1/1	NA	NA
Shaw valley	2005	Flygt 3153	1/1	62.7	17
St. George St.	1966	Gorman-Rupp	1/1	94.6	37.2
Sunset Drive	1973	Barnes	1/1	23	8.5
Talbot Line	2014	Xylem NP-3153	1/1	25	34
Hughes St.	1993	ITT Flygt 3127	1/1	19.7	NA
Woodland	1988	Hydromatic Pentair	1/1	7	33.8
Woodworth Ave.	1972	Smart Turner Hayward	2/1	101	13.7

4.3.2 Current Wastewater Flows

Table 4.2 summarizes the average day, maximum day, and minimum day flows over the period from 2015 to 2020 to the St. Thomas WPCP as taken from the annual WPCP reports. As can be seen in this table average daily flows have been increasing over this period.

Table 4.2 – WPCP Flows (2015 – 2020)

Flows	2015	2016	2017	2018	2019	2020
Annual Average Day Flow (m^3/d)	14,341	15,436	15,929	17,406	18,400	17,870
Minimum Day Flow (m^3/d)	7,838	8,328	8,144	8,628	10,117	10,322
Maximum Day Flow (m^3/d)	39,159	41,364	42,242	40,922	41,097	41,616

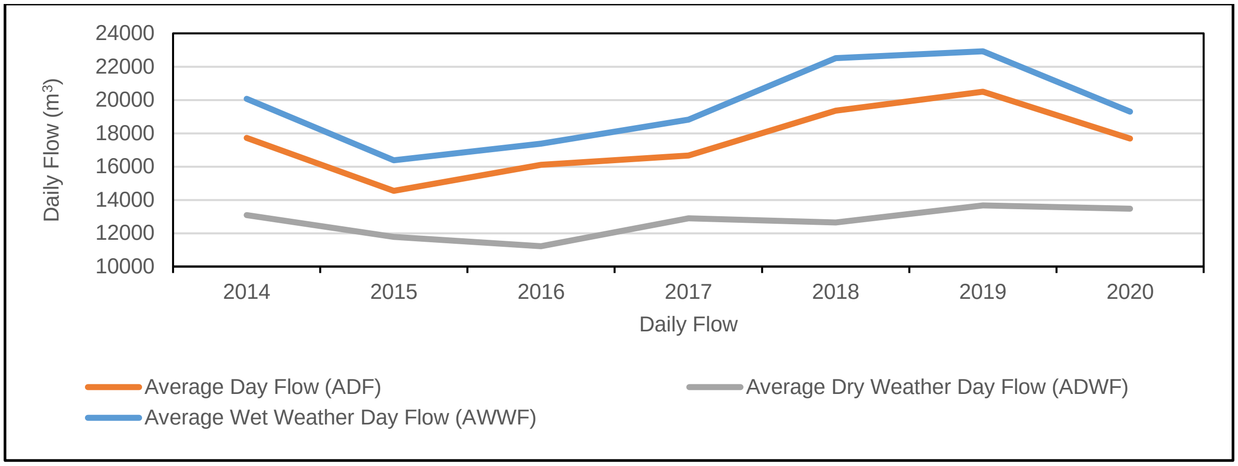


Figure 4.3 – UV System and Effluent Discharge – Normal Conditions

Based upon a review of daily flow data over this period and correlating it to rain events, Figure 4.3 shows the difference between average day wet weather and average day dry weather flows at the WPCP.

4.4 Natural Environment

As part of the PPCP, a Natural Environment Review (NER) was prepared to identify and characterize the significance and sensitivity of the natural water features in the study area. It is included as a section in TM#1 in **Appendix 2**. The City of St. Thomas covers a land area of approximately 35.5 km^2. For the purposes of the PPCP, the Study Area included in this NER includes the whole of the city limits.

4.4.1 Physiography

The City of St. Thomas is situated in three physiographic regions. The majority of the Study Area is in the Ekfrid Clay Plain. The St. Thomas Moraine enters the City boundary from the east and west but does not connect through the Study Area. Lastly, a small area of the Norfolk Sand Plain enters the Study Area from the south (Chapman & Putnam, 1984).

4.4.2 Watersheds and Water Quality

The City of St. Thomas is located almost exclusively within the Kettle Creek watershed with a small part of the collection area situated within the Catfish Creek watershed boundary. According to the 2018 KCCA watershed Report Card, surface water quality within the Kettle Creek watershed was reported as ‘D’ grade, or poor. The low grade is due primarily to phosphorus concentrations that regularly exceed (97% of all samples) the Provincial Water Quality Objective (PWQO) of 0.02mg/L as well as poor benthic invertebrate Family Biotic Index results. E. coli concentrations throughout the watershed were found to be fair, or C grade.

Kettle Creek’s water quality directly affects the water quality of Lake Erie and is a potential point source of contamination to the Elgin Area drinking water supply. Raw water for the Elgin Area Primary Water Supply System is taken from Lake Erie into which Kettle Creek drains. Studies have found that littoral drift within the lake carries sediment from the mouth of Kettle Creek to the intake pipe.

The upper main branch of Catfish Creek is reported as being the area where water quality is the most impaired, with improvement as the creek flows downstream. The

Nineteen Creek sub-watershed, which incudes the small eastern area of the Study Area, was reported as 'C' grade, or fair. The grade, like Kettle Creek, was a result of nutrient levels (phosphorus), intrinsic geology and topography as all being factors affecting water quality within the watershed.

4.5 Current Overflow Issues in the Wastewater System

4.5.1 CSO Facility Overflows

All overflow events between years 2015 to 2020 were reviewed and analyzed. The overflow events are classified as CSO facility overflows and Sanitary Pumping Station (SPS) overflows are shown in Table 4.3. Table 4.4 summarizes the total overflow volumes, the overflow volumes exceeding the WPCP peak flow capacity, and the comparison of overflow volumes to the annual flow volumes treated at WPCP from 2015 to 2019.

Table 4.3 – CSO Tank Overflows Summary (2015 – 2019)

Plant Data	2015	2016	2017	2018	2019	2020
No. of Overflow Events	6	12	13	22	24	3
Total amount of overflow (m^3/d)	34,131	126,299	124,044	355,385	388,373	115,475
Annual overflow volumes as % of flow treated at WPCP	0.6%	2.1%	2.1%	6.0%	6.5%	1.76%
Overflow volume at flows under WPCP peak day capacity (m^3/d)	34,131	120,056	64,553	221,151	254,108	115,475
% Overflow volumes at flows under WPCP peak day capacity	100%	95%	52%	62%	65%	100%

Table 4.4 – Pumping Station Overflows Summary (2015 – 2020)

Plant Data	2015	2016	2017	2018	2019	2020
No. of Overflow Events	4	5	8	18	12	8
CSO overflow volume (m^3/d)	34,131	126,299	124,044	355,385	388,373	115,475
PS overflow volume (m^3/d)	140	943	16,530	6,763	2,298	6,061
PS overflow volume as % of CSO overflow volume	0.40%	0.70%	13.30%	1.90%	0.60%	5.25%

4.5.2 Quality and Characteristics of Overflows

All overflows at the CSO facility are sampled and monitored for quality. The overflow characteristics data from 2015 to 2020 was reviewed regarding overall annual loading to the Creek and as percentage of annual effluent loadings at the WPCP. Table 4.5 summarizes this information.

Table 4.5 – CSO Facility Overflow Loadings

Contributor	Loading (kg/annum)		
	$cBOD_5$	TSS	TP
WPCP effluent[1]	30,204	41,701	3,011

Contributor	Loading (kg/annum)		
	$cBOD_5$	TSS	TP
2015 overflows	1,049	1,485	35
2016 overflows	2,274	3,633	116
2017 overflows	3,206	4,803	135
2018 overflows	8,974	13,934	380
2019 overflows	12,025	23,156	1,165
2020 overflows	3,233	3,695	231
Historic annual average effluent loading (2015 – 2020)			

4.5.3 WPCP By-Passes

Apart from the overflows at the CSO facility and the Pumping Stations, by-pass events have also been reported at the WPCP. However, all such events at the plant are due to mechanical issues and/or to power outages, which cause UV and/or blowers to go off-line for short intervals until back up generation if at that facility come online. None of these events are due to high flows as the wet weather peaks are shaved to the plant's hydraulic capacity by the CSO facility upstream of the plant. As such, while the partially treated effluent is still passing through the temporarily un-operational unit process during such events, it is technical considered a by-pass as the unit process is unable to provide treatment during that period. Table 4.6 shows the combined annual by-pass volumes from 2015 to 2020.

Table 4.6 – WPCP By-Passes

Year	2015	2016	2017	2018	2019	2020
Overflow volume (m^3)	0	626	0	14,869	35,644	1,373

4.6 Conformance to MECP Policy F-5-5

Sections 6.1 and 6,2 of F-5-5 Determination of Treatment Requirements for Municipal and Private Combined and Partially Separated Sewers, “Minimum combined sewer overflow (CSO) controls” state:

“ 1. Eliminate CSOs during dry-weather periods except under emergency conditions.

2. Each municipality shall demonstrate that the combined sewer system, including the regulators, and associated treatment facilities are adequate for the transmission and treatment of all peak dry weather flows from the service area. An emergency condition would exist when e.g., basement flooding, damage to equipment at treatment works or pumping stations, or treatment process washout was occurring or was imminent.”

Based upon a detailed review of overflow events as reported in the St. Thomas Water Pollution Control Plant Annual Performance Reports from 2015 to 2020, all overflow events are associated with wet weather events or equipment malfunctions (which were addressed). The St. Thomas collection system can adequately address dry weather flows and meets sections 6.1 and 6.2 of Policy F-5-5.

Section 6.9 of F-5-5 states:

> " 9. "During a seven-month period commencing within 15 days of April 1, capture and treat for an average year all the dry weather flow plus 90% of the volume resulting from wet weather flow that is above the dry weather flow. The volumetric control criterion is applied to the flows collected by the sewer system immediately above each overflow location unless it can be shown through modelling and on-going monitoring that the criterion is being achieved on a system-wide basis. No increases in CSO volumes above existing levels at each outfall will be allowed except where the increase is due to the elimination of upstream CSO outfalls. During the remainder of the year, at least the same storage and treatment capacity should be maintained for treating wet weather flow."

Our review of the St. Thomas sanitary collection system is set up to convey flows to the WCPC via pumping stations and gravity trunk sewers to the WPCP with the CSO Facility immediately upstream of the WPCP being used as the control point for wet weather events. Table 4.7 summarizes the overflow volumes noted from 2015 to 2020 over the seven-month period noted in MECP policy F-5-5. As can be seen, the overall collection system meets Section 6.9 of policy F-5-5.

Table 4.7 – St Thomas Conformance to Section 6.9 of MECP Policy F-5-5

Year	Sewage Flow (m^3)					% Overflow of Total Flow
	Total flow	CSO Overflow	PS Overflow	WPCP Overflow	Total Overflow	
2015	3,025,304	26,270	5	228	26,503	0.88%
2016	3,041,667	48,093	4	216	48,313	1.59%
2017	3,157,631	58,155	45	45	58,246	1.84%
2018	3,438,888	85,159	1,422	13,823	100,404	2.92%
2019	3,957,463	235,920	146	1,697	237,762	6.01%
2020	3,159,629	0	767	0	767	0.02%
Average	**3,296,764**	**75,599**	**398**	**2,668**	**78,666**	**2.39%**

The remaining paragraphs of Section 6 of F-5-5 refer to implementation of maintenance measures which are part of the City's ongoing operations, as well as the utilization of system capacity, removal of treatment bottlenecks and other improvements which are the subject of this PPCP.

4.7 Volume of Overflow Events

Figure 4.4 shows a graph of the 85 recorded overflow events between 2015 and 2020. This figure shows the total flow occurring on the day of the event including the recorded overflow at the WPCP, SPS or CSO facility as well as the flow that passed through the WPCP. As can be seen some of these events were for daily flows of less than the WPCP capacity. Some of these can be attributed to equipment failure or operational issues but others can be from a very brief peak event that could have overcome the system capacity for a short duration. Twenty-one of these events had total flows which exceeded the WPCP capacity.

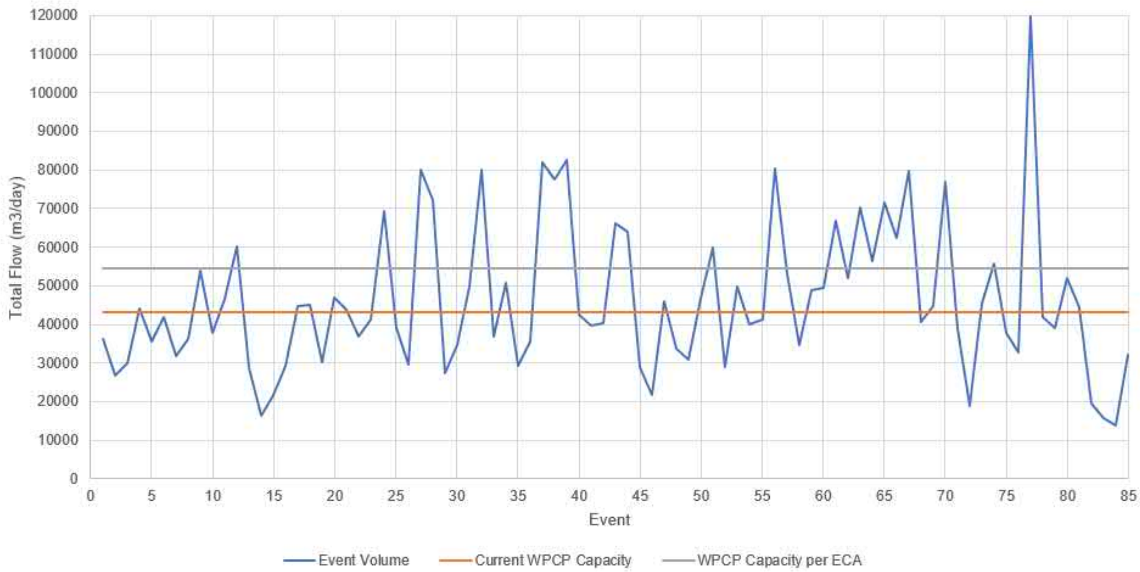


Figure 4.4 – Overflow Events between 2015 to 2020

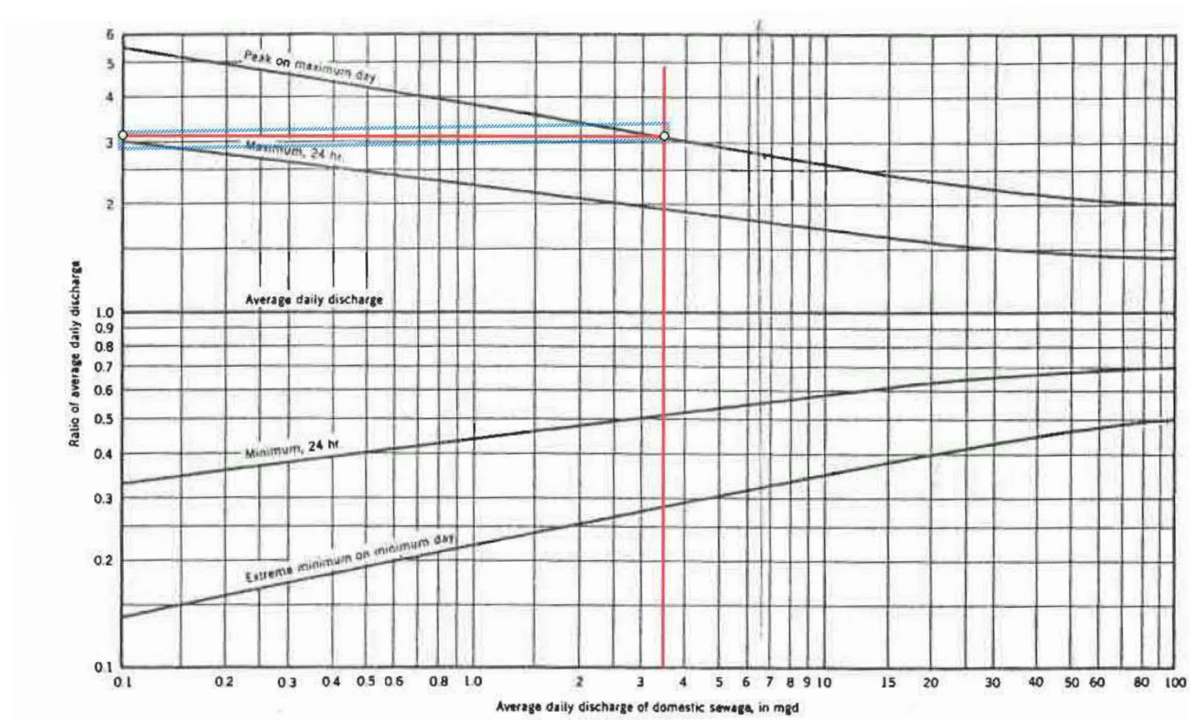


Figure 4.5 – Typical Range for Peaking Factor at a Wastewater Plant

Figure 4.5 shows that for an average day flow of 16,500 m^3/day (equivalent to 3.63 US million gallons per day), the peak day flow based on the Wastewater Treatment Plant Design (WPCF Manual of Practice No. 8, ASCE Manual on Engineering Practice No. 36, 1982) has typically a peaking factor of approximately 3.2. The St. Thomas WPCP historic data shows an average peaking factor of 4.4 for events for which the WPCP capacity was exceeded the (with an extreme event of 7.5). This is significantly higher than the typical peaking factor for a plant of its size.

4.8 Assessment Summary of Wastewater System and Overflows

Based upon our review of exiting conditions, we have made the following conclusions:

1. For the historic average flow value of approximately 16,500 m^3/d at the WPCP, the peak day flow (PDF) in the collection system (including treated flows at the WPCP and the overflows) can be as high as 80,000 m^3/d. This translates into a PDF factor of approximately 5.0, which, in comparison to a typical PDF factor of 3.2 for the current average flow (WEF guidelines), indicates excessive I&I issues in the collection system.
2. The high wet weather flows cause significant overflow issues in the collection system with an annual average overflow volume of 2.9%, and a maximum of 6.0%, of the annual flow volumes treated at the WPCP.
3. The historic average annual $cBOD_5$ and TSS loadings from these overflows to Mill Creek were approximately 20% of the WPCP effluent loadings, and as high as 40% in 2018 and 2019. Similarly, average annual TP loading by the overflows was 12% with a maximum of 55% in 2019. In addition to that, the high E-Coli loadings from the overflows make them a significant source of pollution to the Creek.
4. While the overflows in the collection system occur both at the CSO facility as well as the pumping stations, the CSO facility is the major source of overflows with over 97% of the overflow volume contributed by the same. Further, given the high peaks and I&I in the system, and low frequency and intensity of overflows in the collection system compared to the CSO facility, indicates that the sewers, pumping stations and forcemains are sized adequately to handle the current high peaks for most part, with potential minor exceptions.
5. Out of the 16 pumping stations, overflows have been observed only at 5 stations including – Sunset, Woodworth. St George, and Confederation PSs and the Oak St. Ravine overflow. Out of these 5, majority of the events (over 80%) occur at the Sunset and Woodworth pumping stations. The overflows at the other 3 pumping stations are significantly less frequent and intense in comparison and mostly caused by mechanical issues.
6. Given the remoteness of the Woodworth PS from the CSO facility, the overflows at the Woodworth PS are unlikely to be connected to the CSO facility overflows and potentially caused by high I&I in its sewershed and/or inadequate pumping capacity.
7. Approximately 50-70% of the overflows at the CSO facility occur at peak day flows lower than the WPCP's PDF capacity of 54,400 m^3/d or 632 L/s. The key reason for these overflows is the current operation of the CSO facility which restricts the maximum flow to the WPCP at 500 L/s due to hydraulic bottlenecks at the plant.
8. The 500 L/s restriction causes the CSO facility to surcharge and overflow during longer wet weather events (lasting more than 3-4 hours). As such there is a significant potential to mitigate these overflows by removing the bottlenecks at the plant and increasing the peak flow setting to WPCP's PDF capacity of 632 L/s.

4.9 WPCP Wet Weather Optimization Study

4.9.1 Background

RVA completed a Wet Weather Optimization Study for the St. Thomas WPCP in 2020. The objective of the study was to conduct an engineering assessment of the wet weather flow capacity at the WPCP to identify hydraulic issues at the plant and select an optimization strategy to address these to restore the full peak capacity of the WWTP. As such the key objectives of the study included:

- Identification of the hydraulic inefficiencies and bottle necks;
- Maximum flowrates through various WPCP processes during wet weather flows;
- Assessment of UV Channel flow and capacity during wet weather flows; and
- Assessment of Effluent Pumping system during wet weather flows.

This report is attached as **Appendix 3**.

4.9.2 Hydraulic Modelling of the WPCP

The Wet Weather Optimization Study included hydraulic modelling of the Plant 2, Plant 3 and Plant 4 was done on Visual Hydraulics (Version 4.1). For initial calibration, the model was set up at a total flow rate of 500 L/s and the current flow distribution ratio set up by operations per Table 4.8. The model was validated with the existing hydraulic profiles from the As-built drawings for WPCP upgrades in 2003. The objective of this model was to verify the observations in the field at flows exceeding 500 L/s, and to identify the hydraulic bottlenecks and issues at this flow value.

Table 4.8 – Current Flow Distribution at WWTP

Plant	Flow setting (L/s)	Flow setting (%)	Process capacity share (%)
Plant 2	95	19%	20%
Plant 3	185	37%	38%
Plant 4	220	44%	42%

The key hydraulic bottlenecks identified in the model include the raw sewage influent piping to Plant 3, WPCP outfall, and effluent pumping system. These are discussed in detail in the following sections.

4.9.3 Hydraulic Bottlenecks in Influent Piping

Plant 3 influent feed pipe is a 500 mm diameter pipe which starts from the flow distribution chamber following the grit chamber and has an approximately 60 m length to Plant 3 primary clarifiers. Exiting from the flow split chamber, it has a 3.0 m long 300 mm diameter section fitted with a flow meter and a flow control plug-valve. The pipe diameter at the start of this section reduces from 500 mm to 300 mm for the installed flow meter, followed by increase

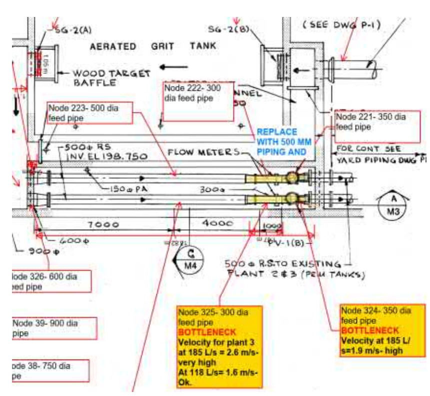


Figure 4.6 – UV System and Effluent Discharge

from 300 to 350 mm for the installed plug valve, and finally expansion to 500 mm via a 350 x 500 mm expander. These fittings within a span of approximately 3.0 m of pipe length create a total headloss of 585 mm at a flow of 185 L/s. In addition, the remaining stretch of the pipe creates an additional headloss of 405 mm due to pipe friction and fittings, leading to a total headloss of 990 mm making the grit tank liquid level elevation approach overflow conditions as observed in the field. This is shown in Figure 4.6.

4.9.4 Hydraulic Bottlenecks in WPCP Outfall and Effluent Pumping System

Under normal conditions, the disinfected final effluent overflows the level control weir, flows to the flood control chamber by via a 600 mm pipe, and is finally discharged via a 600 mm outfall to the creek. See Figure 4.7 for details. Both the weir and the effluent channel are significantly oversized for the peak capacity requirement of 632 L/s and therefore are not the bottlenecks under peak flows. However, the existing 600 mm gravity outfall is a hydraulic bottleneck, which can make the water level in the effluent wet well rise at 500 L/s and flood the UV system. In addition, other hydraulic issues with the outfall system that limit its discharge capacity are discussed below.

Under normal conditions, the effluent overflows the level control weir to the flood control chamber and discharged via a 600 mm outfall pipe to the Creek as shown in Figure 4.7. However, during wet weather as the Creek water level rises to the high-water level in the effluent channel, the effluent is no longer able to flow to the Creek due to lack of driving head and needs another arrangement to be discharged to the Creek.

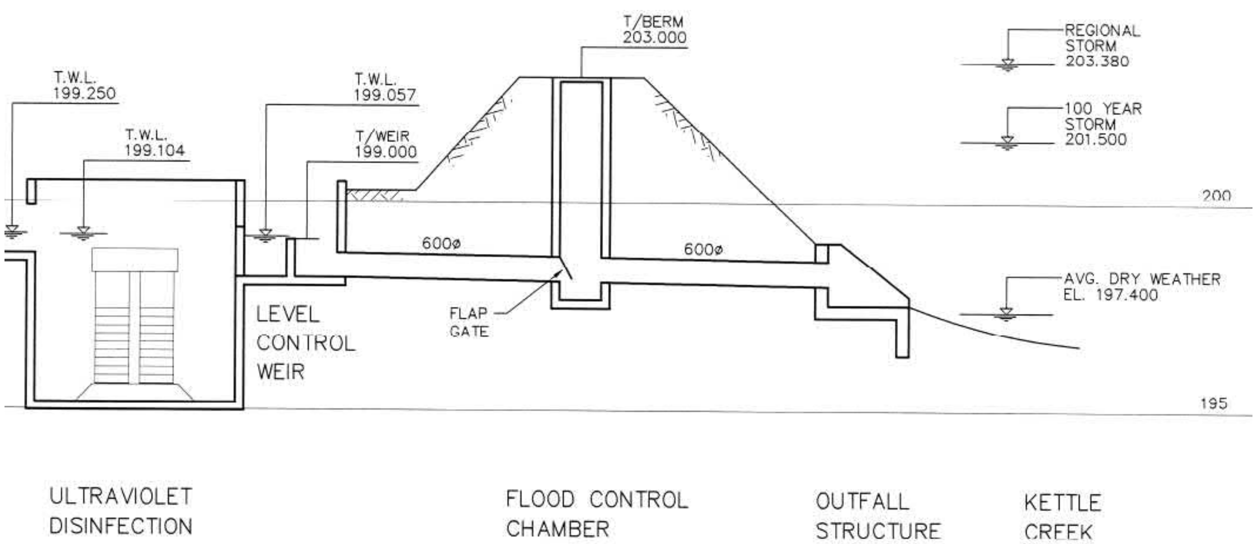


Figure 4.7 – UV System and Effluent Discharge – Normal Conditions

As illustrated in Figure 4.8, effluent pumping system comprising three pumps (2 duty/ 1 Standby) gets triggered at certain set water level to discharge the effluent to elevated effluent well from where it gravity flows to the flood control chamber, followed by its discharge to the creek via the 600 mm outfall. The additional head provided by the elevated effluent well in these conditions allows the effluent flow through the 600 mm outfall pipe. The existing effluent pumps have enough capacity to discharge the peak flows and as such the pumping system itself is not a hydraulic bottleneck for the peak flow conditions. However, given that the driving head in these conditions can vary based on the Creek level, the effluent flow capacity can be significantly reduced below 632 L/s at high Creek levels.

In addition, while the effluent pumping system is primarily provided to address the high creek levels (limiting the gravity flow of effluent), it also serves the purpose of controlling surges in effluent wet well level during high flows. In both cases reduction in effluent discharge capacity due to either of these hydraulic limitations to less than 500 L/s may cause an overflow in the upstream UV channel.

Furthermore, the effluent pump configuration is 2 duty one stand-by. All three pumps have separate discharge headers, so the flow with multiple pumps increase in direct proportion to the pump capacity and number of pumps operation. Each duty pump is rated for 330 L/s so 2 duty pumps can do 660 L/s. However, during surge conditions, the current control settings and/or the small volume of the wet well do not allow the lag pump to kick off fast enough to catch up with the surge leading to UV flooding.

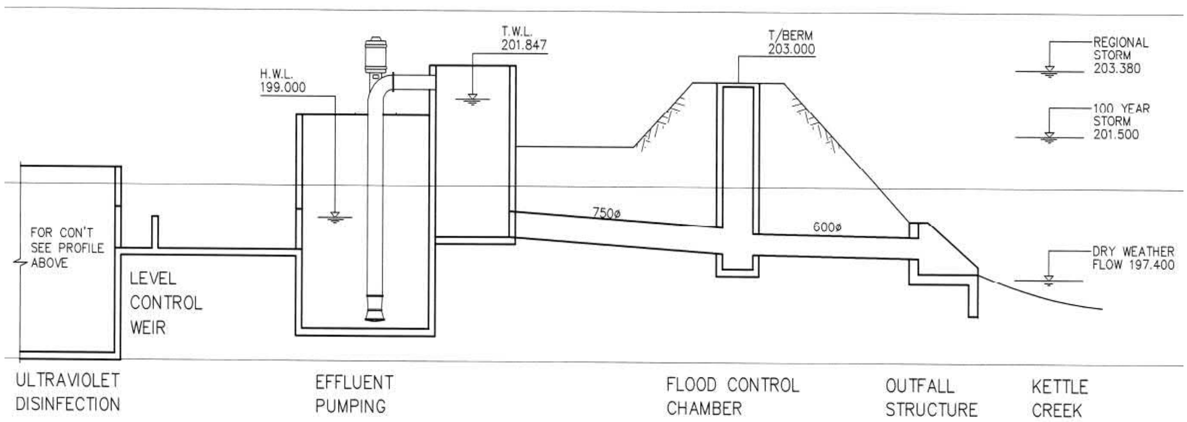


Figure 4.8 – Effluent Discharge Under Wet-Weather Conditions

4.10 Current City Programs Related to PPCP

4.10.1 Asset Management Plan

The City undertakes regular updates of its Asset Management Plan. The 2020 update as shown in Figure 4.9 shows that 16% of the sanitary sewer system is rated as Fair to Poor which is approximately 31 km of sewer.

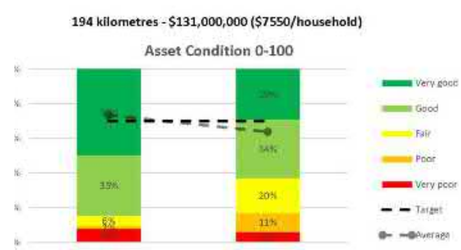


Figure 4.9 – Sanitary Sewer Condition (City December 2020 Asset Mgt Plan)

4.10.2 Annual Sewer Camera Program

The 2019 City budget included a line item of $50,000 for camera work of which almost 80% of it was not used according to the 2020 City budget.

4.10.3 Low Impact Development Measures

The City encourages the use of Low Impact Development Measures (LID) to limit impacts from impervious areas on run off. Section 8.0 Stormwater Management Design

Criteria of the St. Thomas Design Guidelines Manual, 2019 Edition, Subsection 8.0.2. Low Impact Development (LID) Measures states:

> "The City of St. Thomas promotes the use of Low Impact Development (LID) SWM measures. LID measures, such as infiltration galleries, shall be distributed around the site rather than at a single "end of pipe" location.
>
> All LID facilities shall have a design capacity that exceeds the existing conditions recharge volume by 15 percent as a factor of safety to account for aging, compaction, and potential clogging. LID must also demonstrate reasonable drawdown time. Based on Environment Canada data, seven rainfall events occur in a typical month that are greater than 5 mm. A maximum drawdown time of 4 days shall therefore be required.
>
> During construction all LID measures shall be bypassed to prevent accelerated clogging. "

This measure is of limited effectiveness given the predominance of clay soils in the City.

4.10.4 Basement Flooding and Rodding Grant Program

The City has a basement flooding grant program for property owners to provide financial grants to disconnect foundation drains from the sanitary collection system and to install backwater valves and sump pumps for disposal of foundation drain water to a suitable outlet. Between October 2017 and August 2021, the City has provided 229 grants worth a total of approximately $450,000.

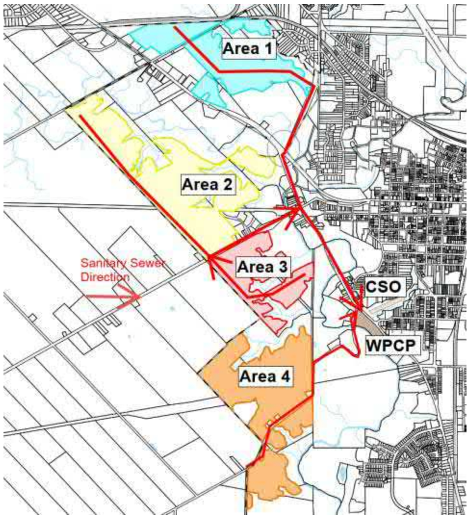
From September 2020 to August 2021, the City has provided 40 rebates to property owners for plumbing fees to rod their drain connections because of blockages on the City portion of the sanitary sewer connection.

4.11 Expected Impact of Future Growth on Pollution Prevention

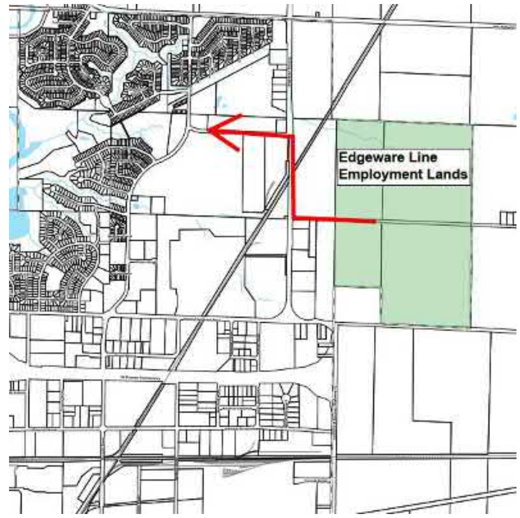
The City's "Positioned for Growth – Planning Justification Report", (February 2020) is intended to determine growth out to the year 2041. After factoring in available land within the City limits, the 2018 Population Update established the need for an additional 76 hectares of residential land to 2041. As per Figure 4.10, this study identified Area 1 as having the available land to accommodate this population. The study also noted other lands available for future residential development (Areas 2, 3, and 4). Areas 1, 2 and 3 could ultimately house an approximate population of 9600 with a build out time of between 30 to 40 years. Area 4, further in the future could house an additional 4300 residents.

Areas 1 through 4 will ultimately connect to the WPCP from a new connection south running along Sunset Drive or north along Bush Line. These sanitary connections may connect into the existing CSO facility. Areas 1 to 4 are not routed through the existing sanitary collection system within the City.

In addition to these areas, the City has recently designated 113 ha of Employment Lands on either side of Edgeware Line east of Highbury Ave. Per the "Edgeware Line Employment Lands– Servicing Study", these lands will have a sanitary connection via the collection system along Burwell Road.



Residential Growth Areas (southwest)



Employment Lands Growth Area (northeast)

Figure 4.10 – Future City Growth Areas

4.12 Flow Data Requirements

In addition to the data reviewed in TM # 1, TM # 2/3 Flow Monitoring and Hydraulic Modelling Data Gap Analysis reviewed and confirmed the requirement for additional data. TM # 2/3 is attached as **Appendix 4**. Based on this review; the following additional flow monitoring locations were recommended:

- Northwest (upstream of the St. George Street pumping station);
- Northeast (upstream of the Burwell Road and Confederation Drive pumping stations); and
- East (Wellington and Elm Streets) sections of the collection system.

The City retained SCG Flowmetrix to undertake flow monitoring at seven locations:

- Site 1 - St. George Street Pumping Station (MH1378);
- Site 2 - Burwell Road Pumping Station (MH1989);
- Site 3a - Confederation Drive Pumping Station (MH1521);
- Site 3b - Confederation Drive Pumping Station (MH1519);
- Site 4a- Mary Street East and Wellington Street (MH570);
- Site 4b - Mary Street East and Wellington Street (MH551); and
- Site 5 - Elm Street at Park Ave (MH1920).

These flow monitoring stations were installed on October 7, 2020 and maintained until they were decommissioned on February 27, 2021. Data was uploaded via telemetry, or manually to the online Flowmetrix software. Flowmetrix data analysts reviewed the raw data collected by the meter and identified any inconsistencies in the data collected by the monitor and flagged it for further investigation. This data was then prepared and provided to the City and RVA via Flowmetrix' "Flowworks" website. RVA's modelers were able to download it and use it to develop the current model. **Appendix 5** provides details on the flow monitoring that was undertaken.

5.0 HYDRAULIC MODELLING OF THE COLLECTION SYSTEM

Refer to RVA's Technical Memorandum# 4/5 – Sanitary Model Build and Hydraulic Results Review and Proposed Solutions which is in **Appendix 6**.

5.1 Model Build-Up

Technical Memo #2/3 provided the available collection system data and past studies and acquired flow data from the City to help build-up the model. Additional flow monitoring location as proposed for the northwest, the northeast and east sections of the collection system were completed per the requirement identified in TM#2/3. Also, missing inverts for required sections of the sewer system were collected from the field. Historic and recent flow monitoring, historic I&I studies (Aldborough/Leger and Woodworth areas), pumping station records, and the historic flow and overflow records for the WPCP and CSO facility were used in calibration of the hydraulic model.

5.1.1 Asset Data Input

RVA received from the City of St Thomas the GIS shapefiles which were imported in InfoWorks to build the model. Since the City's sanitary sewer system includes some areas outside the City's boundaries, the City also provided the GIS shapefiles of the relevant assets from the Municipality of Central Elgin. The GIS Shapefiles which were used to build the model included sewer and maintenance hole (MH) data, elevations of the assets and pumping station information. There were 16 sanitary pumping stations considered in this study and their operational data was imported as a background in InfoWorks to accurately locate the nodes to represent their pumps. Further information was obtained from the Operation and Manual files of the stations to determine the wet well sizes, pump types and to derive the pump performance curves, the switch-on and switch-off levels and the emergency overflow elevation where available.

5.1.2 Catchment Delineation

There were two methods of delineation which were used in this analysis—the area based, and the parcel based sub-catchment delineation. The delineation process to represent the catchments are described below. Figure 5.1 shows an example of the Infiltration and Inflow strip that was generated around each sewer for wet weather flow calibration purposes.

To represent the inflow and infiltration (I&I) in the sanitary sewer system, area-based delineation around the sewer lines were established. The area-based catchment was based on a 45 m strip drawn from each sewer line as illustrated in Figure 5.1. Each MH was setup to have its own I&I strip which was bounded at equal distances between the manholes. This process was completed in GIS environment using a series of ArcGIS Tools.

To represent the wastewater inflow to the sewers, the catchments were based on and aligned to the connected property parcels. The property parcels included areas from the City of St. Thomas and from the Municipality of Central Elgin. The parcels were carefully analyzed to include only the parcels which were required in the sanitary sewer system. This process included removing parcels outside the boundary of the sewer system, parcels representing paths, and empty lots. Address points were created for reference to represent the property parcels at their centroids. The property parcels (residential and non-residential) were then grouped according to the nearest manhole and were combined to comprise the parcel-based catchments—each with corresponding nearest manhole. Figure 5-2 illustrates parcel-based catchments in this study.

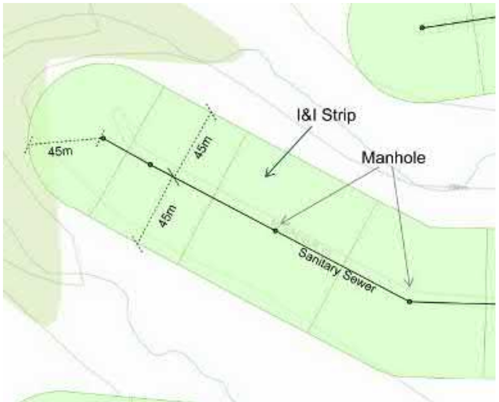


Figure 5.1 - Area Based Catchment Delineation Example

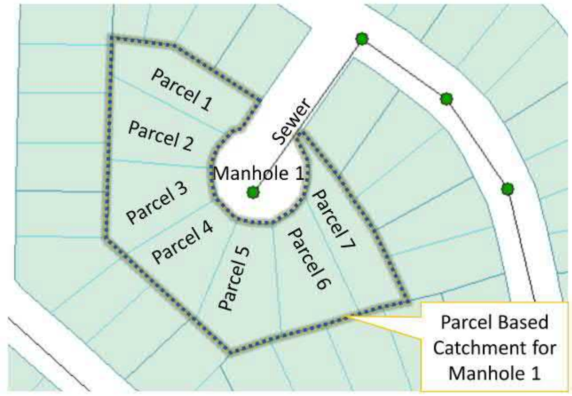


Figure 5-2 - Parcel-Based Catchment Delineation Example

5.1.3 Design Rainfall

Design rainfall profiles were developed from Environment Canada Intensity Depth Frequency curves (IDF) that were derived from monitoring station at St Thomas WPCP (Station ID 6137362). Historical data cover a very long period of 90 years from 1926 to 2016. From the extracted IDF coefficients the rainfall intensities for 4-, 6- and 12-hour duration storms with return periods of 2-years, 5-years, 10-years, 25-years, 50-years, and 100-years were developed. The design rainfall pattern was compiled according to the Chicago type storm distribution and a time to peak ratio of 40% (r=0.4) was used. The rainfall profiles were generated for 5-minute timesteps to match the flow survey timesteps.

5.1.4 Climate Change Considerations

To assess potential impacts of climate change to the sewer network, the University of Western Ontario's (UWO) IDF CC online tool was used (https://www.idf-cc-uwo.ca/). This website links Environment Canada's IDF weather station locations and produces comparable IDF parameter tables between historic data and future climate change IDF parameters. RVA selected a future time for the years 2050 to 2100 and reviewed the three main climate change projections RCP2.6, RCP4.5 and RCP8.5. It was determined that for the location of St. Thomas, the RCP8.5 scenario (unmitigated growth) would produce the highest increase in precipitation and this scenario was then used for comparison with the historic IDF curves. As a result of this comparison, it can be concluded that under the worst-case climate change scenario, precipitation is expected to increase between 22% to 27% starting from a 2-year return period storm towards a 100-year return period storm respectively.

5.1.5 Rainfall Series (Typical Year)

To test for longer term network performance and allow the quantification of CSO spills and their analysis, a real measured rainfall series was chosen to be used as the baseline performance criteria. Typically, a long-term serial simulation should cover a period of 10 years or longer to be able to statistically evaluate impacts. However, since historical data are not available in the required timestep resolution, a different approach was chosen. The City of Toronto has a large permanent rain gauge network installed with over 16 rain gauges distributed over the City. According to their evaluation, the year 1991 represented a typical rainfall year that shows a good rainfall distribution with a variety of storms that test sewer network performance. Rainfall events starting on April 1st until October 27th of that year were recorded and have been added into a serial simulation.

Table 5.1 – IDF conversion factor analysis, City of Toronto to St. Thomas

Multiplying Factor Calculation											
	Station ID	Toronto Old Weston Road (ID: 6158764)	Toronto City (ID: 6158355)	Toronto Island (ID: 6158665)	Toronto Booth (ID: 6158406)	Average	St. Thomas WPCP (ID: 6137362)	Change (mm)	Change Fraction	Change Factor
	Event	2-Year Precipitation (mm)					2-Year Precipitation (mm)			
Duration	5 min	8.52	8.65	8.07	8.61	**8.46**	8.21	-0.253	-0.031	1.031
	10 min	12.11	12.28	12.14	12.02	**12.14**	12.29	0.153	0.012	0.988
	15 min	14.8	14.63	15.12	14.36	**14.73**	14.85	0.123	0.008	0.992
	30 min	19.74	18.98	19.62	18.17	**19.13**	19.96	0.833	0.042	0.958
	1 h	24.03	23.73	24.4	20.96	**23.28**	25.85	2.570	0.099	0.901
	2 h	28.2	27.61	28.74	25.65	**27.55**	30.01	2.460	0.082	0.918
	6 h	35.49	33.62	35.74	33.72	**34.64**	38.1	3.458	0.091	0.909
	12 h	38.42	40.22	39.7	38.52	**39.22**	44.19	4.975	0.113	0.887
	24 h	41.84	45.63	44.19	44.26	**43.98**	49.7	5.720	0.115	0.885
									Average Multiplying Factor:	**0.941**

RVA agreed with the City of St. Thomas to use this data as a basis and modify the data to suit the geographic location of St. Thomas. For this purpose, four (4) historic Toronto IDF stations were analyzed and compared against the St. Thomas WPCP station for 2-year return period storms from 5-minute duration to 24 hours duration storms. An average adjustment factor was established that was then applied to the original 1991 Toronto rainfall year series to adjust rainfall intensities for St. Thomas. The adjustment factor was found to be 0.941 when compared to the City of Toronto. Table 5-1 Summarizes the conversion factor analysis undertaken.

5.2 Flow Monitoring Data

5.2.1 Introduction

For model adjustment and calibration purposes, historic flow monitoring data (2017) and recent (2020) additional flow monitoring data as identified in TM#2/3 from the sewer network were analyzed and used to define wastewater diurnal profiles in the model. There are eight (8) flow monitoring locations with 2017 data results available and seven (7) flow monitoring locations that were measured in 2020. Figure 5.3 below shows the locations of the flow monitors.

2017 flow monitoring data were available for a period of three (3) months from August 16/17, 2017 to November 23, 2017. No rain gauge information was available with the flow monitoring data. The Environment Canada rainfall information that is available for the St. Thomas WPCP can be reviewed in daily or monthly timesteps but would require a finer resolution to be useful for wet weather flow calibration. For this reason, the 2017 flow data was only used for dry weather flow calibration. The 2020 flow monitoring data were available for a period of under three (3) months from October 7, 2020, to December 29, 2020. A rain gauge was installed with the flow meters and rainfall data is available in 5-minute timesteps.

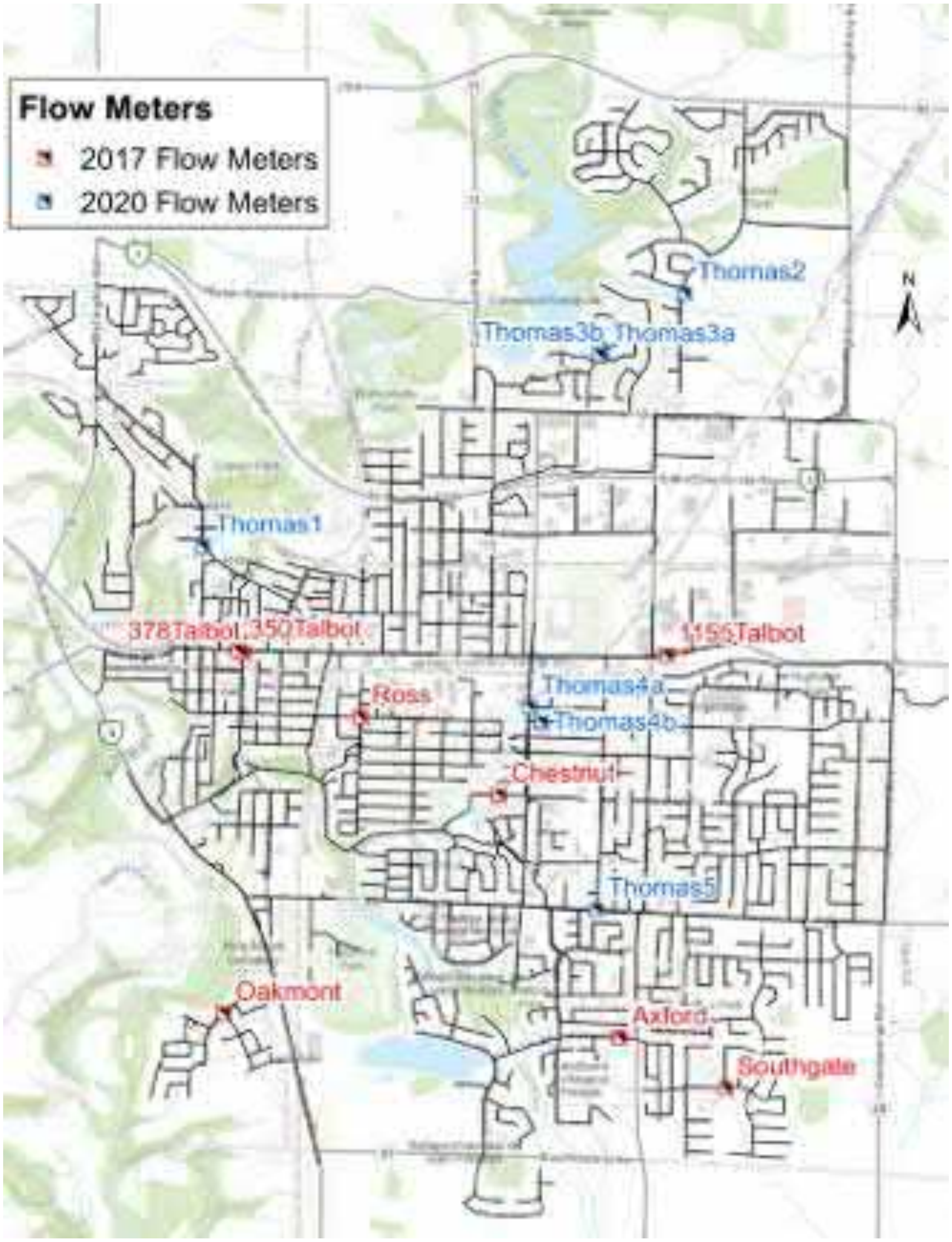


Figure 5.3 – Flow Monitoring Locations

5.2.2 Dry Weather Flow Patterns

RVA analyzed the diurnal profiles for weekday patterns and weekend patterns while considering the upstream flow monitor residential population and calculated/ estimated trade flows that were based on metered water consumption records. The groundwater infiltration (GWI) was estimated based on minimum observed nighttime flows where 80%-85% of that minimum flow in the observed weekend profile was attributed to GWI. Weekend profiles were assumed to have no trade flows occurring, while weekday trade profiles were adjusted to show a block discharge pattern from 7:00 AM to typically 6:00 PM.

Because of the addition of trade flows during weekdays, the per capita flow rate in the weekday diurnal profiles is slightly lower than for the weekend profiles. The hydraulic model uses only one per capita flow number, and the weekday per capita flow number was used in the model. For that reason, the diurnal weekend flow pattern was adjusted to the lower per capita and day flow numbers from weekday profiles to be input into the model. Residential wastewater flows for the weekend profiles are slightly higher than during weekdays.

5.2.3 Wet Weather Flow

During the approximate three (3) month monitoring period, 26 rainfall events of varying intensity and duration were recorded. Rainfall data was analyzed to select the best representative rainfall events that would display intensity and duration as well as best possible prior dry days. Three rainfall events were selected that matched the criteria best.

The first event (Event 1) lasted from 2020-10-23 5:00PM to 2020-10-24 12:00AM, a duration of 19hrs and had a total precipitation of 9.0 mm. It showed one prior dry day. The second event (Event 2) lasted from 2020-11-15 12:00AM to 2020-11-15 2:00PM, a duration of 14 hrs and had a total precipitation of 19.5 mm. It showed three prior dry days and over 10 prior days without significant rainfall. Finally, the third event (Event 3) lasted from 2020-12-12 8:00AM to 2020-12-13 3:00AM, a duration of 19 hrs and had a total precipitation of 14.2 mm. It showed one prior dry day, but over seven prior days without significant rainfall.

The three described events were used to analyze the observed wet weather flows for the selected flow monitor locations and comparing them against simulated model flows for calibrating the hydrologic runoff parameters as described in the following sections.

5.2.4 Rainfall-Derived Infiltration

Rainfall derived infiltration and inflow (RDII) is stormwater and groundwater that enters sanitary sewers. To model RDII to the system, the Sanitary Sewer Overflow Analysis and Planning (SSOAP) Toolbox developed by the U.S. Environmental Protection Agency (EPA) was used. The SSOAP toolbox uses the rainfall data to identify dry-weather flow days that can be used to determine the base wastewater flow and groundwater infiltration components of the total sewer flow, discriminating between weekdays, weekends, and holidays.

6.0 MODEL RESULTS

Refer to RVA's Technical Memorandum# 4/5 – Sanitary Model Build and Hydraulic Results Review and Proposed Solutions which is in **Appendix 6**.

6.1 Rainfall-Derived Infiltration

From flow monitoring data evaluation and past I&I study reports certain catchment areas could be quantified with typical Groundwater Inflow (GWI) rates during dry weather conditions and with higher Rainfall Derived Infiltration and Inflow (RDII) rates. Table 6.1 and Figure 6.1 show the derived infiltration rates and the corresponding areas from this modeling exercise.

Table 6.1 – Derived Infiltration Rates for Areas in St. Thomas

Area #	RDII(L/s/ha)	Area #	RDII(L/s/ha)
1	0.436	10	0.577
2	0.326	11	0.706
3	0.146	12	0.561
4	0.436	13	0.151
5	0.524	14	0.595
6	0.093	15	1.623
7	0.096	16	3.793
8	0.417	17	2.700
9	0.706	18	3.769

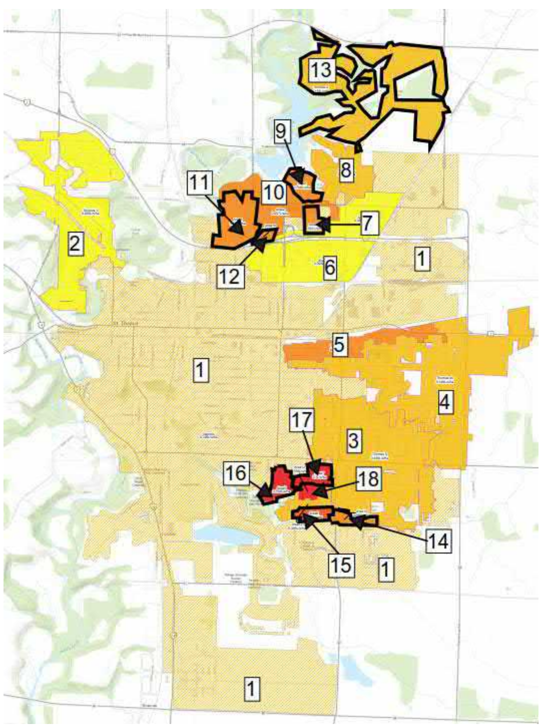


Figure 6.1 – Catchment Areas for RDII Rates

Area 1 represents a generic I&I rate that RVA used for areas that were not monitored. Based upon our judgement, it was given a relatively high but not an extreme rate based on the areas monitored.

6.2 Network Performance

The sewer network performance has been mapped out for design storm events and the maps presented in Appendix 7 of Technical Memorandum# 4/5 (**Appendix 6**) show the hydraulic bottlenecks for sewer surcharge and for hydraulic grade lines in maintenance holes (MH) for 2-year to 100-year design storm events. For the MH a hydraulic grade line criterion of 1.8 m freeboard (compared to road centreline) was used. This would typically match the level of basements and any sewer surcharge above would trigger flooded basements if foundation drains or any sanitary basement installations are made. MH were color coded with green color showing freeboards of 1.8 m or more, yellow showing less than 1.8 m freeboard and red color showing 0 freeboard or flooding above ground.

Color coding for sewers was used in the same three colors for green sewer showing no surcharge, yellow color showing surcharge conditions by depth but not representing necessarily a bottleneck and red color showing a capacity restriction based on limited pipe carrying capacity. The model shows localized sewer bottlenecks for the lower return period storms and a larger portion of the network shows capacity problems for the 100-year return period storms. Pockets of flood clusters can be recognized, and these typically match the areas that have been identified with high I&I rates, since this data has been entered into the model hydrology.

Figure 6.2 – Sample Section from a Sewer Performance Map (5-year event)

6.3 Storage Tank Performance and Control

6.3.1 Sluice Gate– Current Operation

The main flow control feature to limit flows towards the WPCP is a real time-controlled twin sluice gate. The sluice Gates are each 0.6 m wide and have a maximum opening of 0.9 m with actuated penstocks that can close the gate opening when tank influent flows and the hydraulic head on the gate rises and increases pass forward flows. A flow monitor or level gauge downstream of the Sluice gates measures the flows and sends a SCADA signal to the sluice control to limit flows to 500 L/s maximum.

For the future scenario where treatment plant bottlenecks are removed, an increased flow rate can be accepted that will reach the current ECA consent of 632L/s.

There are two additional sources of flow arriving at the main sewer downstream of the sluice gates that cannot be controlled or throttled. One source is the flows from Sunset Drive SPS that currently operates at 21 L/s capacity but is scheduled to be upgraded to 59 L/s capacity. The second source of uncontrolled inflow comes from a 200 mm diameter sanitary sewer connection that has a capacity of approximately 20 L/s. This serves a limited area along Sunset Drive close to the WPCP. The peak wet weather flow from this 200 mm sewer area should pass before the peak flow is received from the main collection system.

The current operational mode of the sluice gates allows maximum opening of the gates pre-set at 0.18 m, thereby allowing the head build-up via wet weather flow storage to push higher flows (up to 500 L/s) through the gate opening. While protective of the downstream processes, this approach is clearly over-conservative in flow control due to high level of CSO overflows, and their occurrence at peak flows lower than the PDF capacity of the plant.

6.3.2 Storage Tank Performance with Optimized Sluice Gate Operation

For CSO spill analysis, the hydraulic model was set up for the existing scenario to limit flows to 500 L/s and for the future scenario to limit flows to 589 L/s at the slice gates. Real time control rules were applied to the model and the speed of increment or decrement of the sluice gate openings was set to 5 cm/s as a best guess estimate.

The sluice gate operation logic was set-up to have the gate fully open (0.9 m) at flows below 500 L/s, reduce the opening to 0.15 when flow exceeds 500 L/s, and open incrementally as the flow decreases to 500 L/s or lower. Due to the rule setting for the sluice gate Real Time Control (RTC), the setup currently produces some minor operational inefficiencies that are caused by the sluice gate closing move that will cause an initial period of over-controlling flows before opening again. The time difference between an efficient RTC operation and an oscillating RTC operation could be up to 40 minutes (or approximately 15%) of storage time for example in the 800L/s steady inflow event.

The CSO storage tank performance was initially tested against constant inflow events to evaluate potential storage times when inflows exceed the controlled outlet flows. A comparison was made between the current 500 L/s flow control and the proposed increase to 589 L/s. See Table 6-2 for details.

Table 6-2 - Tank Storage Performance for Test Inflows

Scenario	Tank Gates controlled to 500 L/s	Tank Gates controlled to 589 L/s	Storage Time Increase [hrs]
	Storage Time [hrs]	Storage Time [hrs]	
800 l/s steady inflow	4:15	5:10	**0:55**
1000 l/s steady inflow	3:20	4:00	**0:40**
1100 l/s steady inflow	1:50	2:15	**0:25**

Scenario	Tank Gates controlled to 500 L/s	Tank Gates controlled to 589 L/s	Storage Time Increase [hrs]
	Storage Time [hrs]	Storage Time [hrs]	
1200 l/s steady inflow	1:29	1:50	**0:21**
1400 l/s steady inflow	1:05	1:15	**0:15**

Note that the storage time shown by the model even at the current flow limit of 500 L/s is likely much higher than what is currently available with the existing overly restrictive control philosophy of the gates. As such modification of the existing control philosophy (in line with the one used in the model) alone is likely to lead to major reduction in spills before additional benefit is achieved by increasing the flow limit after removal of hydraulic bottlenecks at the WPCP.

6.4 CSO Spill Analysis

A typical year rainfall series were developed that are based on real measured events in Toronto with their intensity slightly decreased to adjust for the location of St. Thomas according to their IDF curves. In addition, alternative higher intensity scenarios were run including a possible worst-case scenario were created to analyse the spill response at the CSO tank.

However, none of the scenarios was observed to yield any spills which did not align with the actual conditions where several spills are observed in a typical year. The key reason for this was inferred to be the current operational mode of the sluice gates in which the maximum opening of the gates is pre-set at 0.18 m, in contrast to 0.9 m (initial full opening of the gate) used in the spill analysis model.

In the baseline scenario (pass-forward flow limit of 500 L/s) the peak flows and volumes created by the sewer network always exceed the CSO tank storage capacity.

In addition, the model scenario with optimized sluice-gate operation also showed that during a typical year, the worst occurring storms would create peak flows of 694 L/s into the CSO tank and the tank level would fill up seven (7) times during the year with the top level reaching 204.6 m, which is below the 205 m spill weir elevation.

It is to be considered that the design storm is a 12-hour duration storm that has a considerable impact on the catchment area in terms of saturation and wet weather flow response. Whilst the proposed upgrades at the WPCP can only slightly reduce the peak flows (-0.88% to -0.19%), they would reduce the total spill volume between approximately 4% and 9%. Therefore, initial benefits will be seen in the number and volume of typical spills. However, for the climate change scenario during years 2050 to 2100, it can be expected that this initial reduction trend will reverse and could show an increase in peak spills (10% to 15%) with increase in volume between approximately 23.6% to 26.5%. Climate change predictions come with an uncertainty and will depend on future economic activities and mitigations. For this climate change impact assessment, the highest predicted changes and worst possible outcomes were used to showcase the maximum potential for flow and volume increases.

Table 6-3 CSO Tank Spills for Design Storm Scenarios (Peak Flow and Volume)

	Flow (m^3/s)				
Storm Event	Existing Scenario	Proposed Scenario	%Change vs Existing	Proposed-Climate Change Scenario	%Change vs Existing
2yr	1.25	1.24	-0.88	1.41	12.56
5yr	1.52	1.51	-0.59	1.75	15.13
10yr	1.71	1.70	-0.64	1.93	12.69
25yr	1.93	1.92	-0.73	2.13	10.47
50yr	2.05	2.04	-0.29	2.26	10.44
100yr	2.16	2.16	-0.19	2.38	10.28
	Volume (m^3)				
Storm Event	Existing Scenario	Proposed Scenario	%Change vs Existing	Proposed-Climate Change Scenario	%Change vs Existing
2yr	29,708	26,964	-9.24	37,359	25.75
5yr	46,910	43,649	-6.95	59,382	26.59
10yr	59,756	56,216	-5.92	73,893	23.66
25yr	77,412	73,111	-5.56	96,481	24.63
50yr	90,901	86,390	-4.96	114,031	25.45
100yr	105,026	100,703	-4.12	132,336	26.00

6.5 Sewage Pumping Station Related Spills

6.5.1 Overview

Of the 16 existing sewage pumping stations in the model, seven (7) show several spills through the emergency overflows at the pumping station itself or at a nearby high-level overflow. One extra overflow link was monitored on Sunset Drive south of the CSO tank and is shown as link SAMH891 in the table. The table below shows details of the pumping stations.

Table 6-4 SPS Spills for Design Storm Scenarios (Return Period)

	Spill Occurrence					
Sewage Pumping Station	2-yr	5-yr	25-yr	50-yr	100-yr	Number of spills – Typical Year
#1 Axford	x	x	x	x	x	0
#2 Burwell				x	x	0
#3 Confederation			x	x	x	0
SAMH891.1 (SSO Sunset Dr., south of CSO tank)			x	x	x	0
#11 St. George					x	0
#12 Sunset	x	x	x	x	x	37

	Spill Occurrence					
Sewage Pumping Station	**2-yr**	**5-yr**	**25-yr**	**50-yr**	**100-yr**	**Number of spills – Typical Year**
#14 Wolfe	x	x	x	x	x	0
#16 Woodworth	x	x	x	x	x	0

St. Thomas Water Pollution Control Plant Annual Performance Reports from 2015 to 2020 were reviewed for reported overflows from SPS. The table below summarizes the 6 years of reporting.

Table 6-5 – Number of SPS Overflows Reported in WPCP Annual Reports (2015-2020)

Sewage Pumping Station	0<OF <10m^3	10<OF <100m^3	100<OF <1,000m^3	1,000<OF <10,000m^3	OF> 10,000 m^3
#3 Confederation	1		1		
#11 St. George			2	1	
#12 Sunset	8	13	1		
#16 Woodworth	4	5	8	1	1

Whilst many of the above sewage pumping station show spills for the design storm events, only #12 Sunset Drive SPS shows a frequent spill activity for the typical year storm series. This pumping station is already being proposed for an upgrade to approximately double its pumping capacity. The design of the replacement SPS should address the current frequent small overflows from the existing SPS.

Another pumping station that shows spills from a 2-year design storm is Woodworth Avenue SPS. The annual reports overflow records show that this pumping station has spilled every year between 2015 and 2020.

6.5.2 Woodworth Ave SPS and Collection System

The Woodworth Ave SPS has a setup of three pumps in a duty/ lag/ standby arrangement. The design capacity is 101 L/s at 13.7 m TDH for each pump. The pumps discharge into a 400 mm diameter forcemain. As for all pumping stations, dynamic head discharge curves were added for each pump. Since actual pump performance curves were not available, pump curves from the pump manufacturer’s website were looked up and adjusted to represent a best estimation. With the current pump setup, the model predicts a maximum pumping station performance of 228 L/s that generates approximately 1.8 m/s velocity in the forcemain. However, this is not sufficient to pump the total inflows to the pumping station. The model predicts total inflows to be 303L/s, 365 L/s and 400 L/s for the 2-year, 5-year and 10-year design storms respectively. This leads to spill events at the high-level overflow.

An upgrade of the Woodworth SPS to operate 3 pumps with the above capacity would combine to a total pump performance of 297 L/s at approximately 2.36 m/s velocity in the forcemain. Whilst such an upgrade will increase the pumping station’s spill protection to approximately a 2-year design storm event and will eliminate the currently experienced annual spills, it would hydraulically overload portions of the downstream sewer from the discharge point. The existing sewers are 675 mm and 750 mm in diameter whilst the first sewer section is steeper with a 450 mm diameter. The approximate sewer capacity of this section is in the range of 350 – 380 L/s. Since there are other sewer inflow apart from Woodworth SPS, this sewer shows surcharge in the 2-year storm event with 230 mm freeboard at a low point. Therefore, this sewer section would require an upgrade for additional capacity of approximately 100 L/s. The downstream sewer section to the above section is 1050 mm diameter and has varying capacity with a minimum capacity of 1000 L/s.

6.5.3 Sunset Drive SPS and Collection System

At present, the capacity upgrade of the sewage pumping station at Sunset Drive from 21 L/s to 59 L/s to accommodate the Area 1 development (and future Areas 2 and 3) will require an increase in forcemain capacity. This pumping station capacity increase should also look to address the current overflows noted from the PPCP spill analysis. The existing forcemain has 567 m length and discharges at a maintenance hole on Sunset Drive. From there a 200 mm diameter gravity sewer with an approximate capacity around 20-23 L/s runs for 407 m until it connects to the trunk sewer downstream of the CSO tank and into the WPCP. Both assets, the forcemain and the gravity sewer would require upgrades to accommodate the increased flows. It is recommended that the forcemain be extended over the 407 m gravity line and to be connected to the CSO tank. This will increase the amount of controlled flow from this pumping station to the WPCP to nearly 100%, whilst at the same time the sluice gate operation could be adjusted to an increased rate that matches the 632L/s WPCP wet weather capacity. This would further increase storage time in the CSO tank enabling it to pass flows to the plant more efficiently.

6.5.4 Burwell Rd SPS and Collection System

The Burwell Rd SPS has a setup of 2 pumps in a duty/ lag arrangement. The design capacity is 44 L/s at 30 m TDH for each pump. The pumps discharge into a 200 mm diameter forcemain. Under 10-year wet weather conditions, with the additional future flows from the Edgeware Line Employment Lands, there will be a requirement to increase the PS capacity, upsize the forcemain and approximately 1200 m of sewers.

7.0 CONCLUSIONS

The following are the conclusions from our background investigation review and modeling analysis.

7.1 Assessment of Current Flows

Based upon our review of exiting conditions, we have made the following conclusions:

1. For the historic average flow value of approximately 16,000 m^3/d at the WPCP, the peak day flow (PDF) in the collection system (can be as high as 80,000 m^3/d. This translates into a PDF factor of 5.0, which indicates excessive I&I issues in the collection system;
2. Review of Annual WWTP reports indicated the following
    a. The annual average overflow volume of 3.5%, and a maximum of 6.5%, of the annual flow volumes treated at the WPCP,
    b. Average annual $cBOD_5$ and TSS loadings from these overflows to Mill Creek were approximately 20% of the WPCP effluent loadings, and as high as 40% in 2018 and 2019,
    c. Average annual TP loading by the overflows was 12% with a maximum of 55% of the effluent loadings in 2019. In addition to that, the high E-Coli loadings from the overflows make them a significant source of pollution to the Creek;
3. The CSO facility is the major source of overflows with over 97% of the overflow volume contributed by the same;
4. Out of the 16 pumping stations, overflows have been observed only at 4 stations (Sunset, Woodworth. St George, Confederation) and the Oak St. Ravine overflow. Out of these 5 sites, most of the events (over 80%) occur at the Sunset and Woodworth pumping stations;
5. Overflows at the Woodworth Ave SPS are potentially caused by high I&I in its sewershed and/or inadequate pumping capacity;
6. 50-70% of the overflows at the CSO facility occur at peak day flows lower than the WPCP's PDF capacity of 54,400 m^3/d or 632 L/s;
7. The 500 L/s restriction and the current operating mode of the flow control sluice-gates at the CSO facility, cause it to surcharge and overflow frequently during wet weather events.

7.2 Sewer Camera Work

At present the City has a $50,000/year budget of which almost 80% of it was not used in 2019 according to the 2020 City budget. This budget should be fully utilized each year to help confirm locations of excessive I&I in addition to its other uses.

7.3 Need for Better Rainfall Data

To best improve sewer network performance a real measured rainfall is the best means of developing rainfall events for analysis. This report used Toronto based rainfall data that was transposed to St. Thomas. It would be in the City's best interest to install a permanent rain gauge station at the WPCP should the City wish to conduct future flow monitoring assignments and have concurrent rain data information in 5-minute interval resolution to match standard flow monitoring timesteps. A further benefit of such an installation would be the ability to measure extreme storm events and replicate known flood events for further hydraulic model calibration.

7.4 Need to Better Define Infiltration and Inflow Rates

The age and parcel fabric over the St. Thomas catchment varies considerably and in connection with that, a high deviation in I&I rates was observed for monitored sewersheds and reviewed from previous Infiltration and Inflow studies such as the 2015 Study for the Aldborough/ Leger and Woodworth Avenue SPS study areas. I&I inflow rates between certain areas were observed with differences with a factor up to eight (8) times as high as the lowest I&I rates. Whilst the hydraulic model was calibrated for wet weather flows with available 2020 flow monitoring data, a large area (996 ha), 63% of the total sewershed was not covered by this calibration and had to be estimated, based on surrounding I&I rates. It is recommended to carry out further flow monitoring in future to refine the model calibration and peak flow response in combination to finding areas with extreme high I&I rates and exploring the source of the infiltration.

7.5 Long Term Infiltration and Inflow Rate Reduction

A future objective should be to mitigate I&I where practicable and cost efficient. We foresee considerable scope of wet weather flow reduction from targeted improvement assignments once the overall system is better understood. The 2020 flow monitoring and 2015 I&I study show a portion of the I&I problems.

7.6 Sealing of MH Upstream of CSO

It is observed that the CSO tank’s maximum surcharge level impacts connected sanitary sewers and causes surcharge. Some nearby maintenance holes that are connected to the tank show a lower ground elevation than the tank level itself. It is therefore advisable to periodically check if nearby low-lying maintenance holes are properly sealed or appropriately raised to avoid spills to the environment and the nearby watercourse.

7.7 CSO Tank Performance

Performance testing of the CSO tank has shown that modifying the operational mode of the flow control sluice gates and the capacity increase in flow to the treatment plant from 500 L/s to 589 L/s will lead to an increased storage time in the tank accommodate peak flow events before a spill over the emergency overflow weir occurs. These upgrades will provide storage time of 5.2 hrs for an inflow of 800L/s and 1.25 hrs for 1,400 L/s.

7.8 Flows Downstream of the CSO Tank

The 200 mm diameter sanitary sewer from Farley Place that currently connects to a mainline sewer downstream of the CSO tank sluice gates and could be connected to the CSO tank to be able to further control and regulate the flow to the treatment plant. The capacity upgrade of the Sunset Drive SPS (near Sunset Drive) from 21 L/s to 59 L/s will likely require an increase in forcemain capacity. The forcemain has 567 m length and discharges at a maintenance hole in Sunset Drive. From there a 200 mm diameter gravity sewer with an approximate capacity of 20-23 L/s runs for 407 m until it connects to the mainline sewer downstream of the CSO tank. Both assets, the forcemain and the gravity sewer would require upgrades to accommodate the increased flows. It is recommended that the forcemain be extended over the 407 m gravity line and connected to the CSO tank. This will increase the amount of flow that is controlled to the WPCP to nearly 100%, whilst at the same time the sluice gate operation could be adjusted to an increased rate that matches the WPCP’s 632L/s maximum flow limit.

8.0 POLLUTION PREVENTION MEASURES

8.1 Collection System Upgrades

The required improvements focus on three SPSs and their related sewers.

8.1.1 #12 Sunset Drive SPS and Collection System

The Sunset Drive SPS is already being proposed for an upgrade to approximately double its pumping capacity. The design of the proposed upgrade should address the current minor overflows from the existing SPS. The forcemain from the new SPS should be extended to the CSO tank.

8.1.2 Improvements to the Woodworth Ave SPS and Collection System

Improvements to the Woodworth Ave SPS will require detailed study to balance the impacts of periodic sewage overflows versus ensuring operational efficiency for dry-weather operation (which is most of the time). This will require a separate engineering planning and detailed design assignment. The Woodworth SPS was upgraded last in 2011 to provide for a third sewage pump (each pump rated at 101.8 L/s) which provides the station with a firm capacity of 203.6 L/s according to the current ECA. Our modeling indicates that each of these pumps would have to be upsized to a capacity of approximately 150 L/s to manage flows up to the 2-year return period. At present, the SPS is not equipped with variable frequency drives on the pumps which indicates that the current pumps cannot operate optimally at lower flows.

Our modeling has identified that capacity in the collection system will have to be increased approximately 1760 m downstream of the SPS at a minimum. These changes will involve:

1. Upsizing the current 1007 m of 400 mm forcemain (including a crossing of the multiple rail tracks on First Ave);
2. Replacing 250 m of 450 mm sanitary sewer from forcemain outlet to Talbot Street on First Ave; and
3. Replacing 523 m of 600/750 mm sanitary sewer from First Ave to south of Wellington St.

At detailed design, the specific sizes of the upgraded pipes will be confirmed.

8.1.3 Improvements to the Burwell Rd SPS and Collection System

Under 10-year wet weather conditions, with the additional future flows from the Edgeware Line Employment Lands, there will be a requirement to increase the capacity of the Burwell Ave SPS. The Burwell Rd SPS would require to be upgraded to a capacity of 219L/s. The current 200 mm diameter forcemain would require twinning and forcemain and approximately 1200 m of sewers are required to be upsized. This requirement for expansion is in general agreement with the ultimate expansion of the Burwell SPS as stated in the *March 2017 Development Engineering St. Thomas Sports & Recreation Complex — Servicing & SWM Design Report* which stated:

> "Based on the 1993 Dillon Dalewood Meadows Subdivision Sanitary Pumping Station Report the station was originally sized for a capacity of 59 L/s with an ultimate capacity of 240 L/s."

The City should confirm the consolidated requirement for capacity for both wet weather flows and future service demand as part of the planning and design of the Burwell Rd SPS, forcemain and gravity sewer upgrades.

8.1.4 Annual Sanitary Sewer Lining Program

Approximately 31 km of the sanitary sewers were rated to be in “Fair to Poor” condition in the City’s December 2020 Asset Management Plan. Based upon the investigation work to be undertaken, it is assumed that the City will look to line a minimum of 750 m per year and assuming this corresponds generally to the quantity of “Fair to Poor” condition sewer, this will take 41 years to complete.

8.2 CSO Operation Optimization

The CSO operation is currently based on a single flow limit value of 500 L/s. This is flow limit is based on the existing hydraulic bottleneck in the Plant 3 influent, along with those in the effluent outfall and pumping system, all of which limit the maximum flow to this value. Removal of these bottlenecks via recommended upgrades would not only restore the full PDF capacity of 632 L/s but also the associated PHF and PIF capacities, as predicted and discussed in section 8.4 and TM # 4/5.

This work should be undertaken following upgrades to WPCP to remove bottlenecks. It is not anticipated to require any additional capital cost but will require a trial-and-error approach to programming the gates, reviewing the results, and adjusting based upon some months of operation. The MECP should be advised of this approach so that there is no miscommunication while the gate operation is being optimized.

8.3 Removal of Capacity Constraints at the WPCP

8.3.1 Upsizing Plant 3 Influent Flow Meter and Plug Valve

The intent of this upgrade is to remove the bottleneck in Plant 3 influent pipe which causes the upstream grit tank to overflow at flows over 500 L/s. The upgrade would entail replacing the 300/350 mm pipe section with a 450 mm section in the Plant 3 influent pipe thereby restoring its peak flow capacity of 214 L/s. In addition, this would help restore the WPCP’s rated peak day flow capacity of 632 L/s.

8.3.2 UV System Upgrade

The objective of this upgrade would be to provide redundancy, improved efficiency, and resilience against extreme wet weather events to the UV system. This upgrade would include the following:

- Twinning of the existing channel and have two parallel channels; and
- Replacement of the UV banks in the existing channel to new state of the art UV banks

Implementation of these upgrades would not only provide operational redundancy and improved efficiency to the system but also allow disinfection of wet weather peaks exceeding 632 L/s for short durations on peak flow days through the plant. Further, the added capacity of the stand-by channel can be used in case of extreme wet weather events if needed, thereby reducing the necessity to throttle the influent flow at the CSO and mitigating the associated overflows at the same.

8.3.3 Effluent Pumping System Upgrades

The objective of this upgrade will be to remove the bottleneck caused by 600 mm outfall pipe as well as the effluent pumping system capacity and operation. The Effluent Pumping Station upgrades will include the following items:

- Replacement of the existing 600 mm outfall pipe to a 750- or 900-mm diameter pipe, as required.
- Raising the walls of the elevated effluent well by 1.0 m, along with associated modification to the effluent pump discharge header.
- Replacement of two of the existing effluent pumps with capacity of 660 L/s each, to mitigate the wet weather flow surges in the wet well.
- Modify the control narrative of the wet well pump operation to improve the pump response in case of high flows.

Implementing these upgrades would remove the existing bottleneck caused by the 600 mm outfall pipe. In addition, raising of the elevated wet well will add further flow capacity to the upgraded pipe at high creek level during wet weather. Furthermore, provision of two large (660 L/s) pumps in the Effluent Pumping Station would facilitate a quicker response to wet weather flow surges in the effluent wet well thereby protecting the upstream UV system from flooding.

8.3.4 Inter-Plant Flow Distribution Optimization

Table 8.1 summarizes interplant flow distribution based on the process capacities of the individual plants. Currently the hydraulic capacity of Plant 3 is limited to 150 L/s due to a bottleneck in its influent piping, removal of this bottleneck will restore its peak capacity of 214 L/s. Further, while Plant 2 and 3 hydraulic capacities are limited to the indicated values, Plant 4 has significantly higher hydraulic capacity that can be utilized during wet weather flows when the peak instantaneous and peak hourly flows exceed 632 L/s for short durations during a peak day.

Table 8.1 – Interplant Flow Distribution up to PDF Capacity

Parameter	Unit	Plant 2	Plant 3	Plant 4	Total
Flow distribution up to WPCP PDF capacity	%	17%	34%	49%	100%
Flow distribution up to WPCP PDF capacity	L/s	106	214	312	632

In the absence of availability of historic peak hourly flow (PHF) and peak instantaneous flow (PIF) data, anticipated design values of these flows have been determined based on WEF guidelines as indicated in Table 8.2.

Table 8.2 – Guideline Values for Peak Hourly Flow (PHF) and Peak Instantaneous Flow

Parameter	Unit	Value	Remarks
ADF	L/s	312	ECA
PDF	L/s	632	ECA
PIF	L/s	961	Based on PIF to PDF ratio for a rated capacity of 312 L/s per WEF guidelines
PHF	L/s	869	Average of PIF and PDF

The existing large hydraulic capacity of Plant 4, and the additional hydraulic capacity to be gained in the UV system and the outfall via the recommended upgrades can be used to pass flows exceeding 632 L/s for short durations during peak flow days. This can be achieved by increasing the flow proportion to Plant 4 beyond 632 L/s, while holding the maximum flows to Plant 2 and 3 constant as shown in Figure 8.1. The limits and duration of higher intermittent peaks through the plant can be assessed once the upgrades to Plant 3, UV, outfall, and effluent pumping systems are completed.

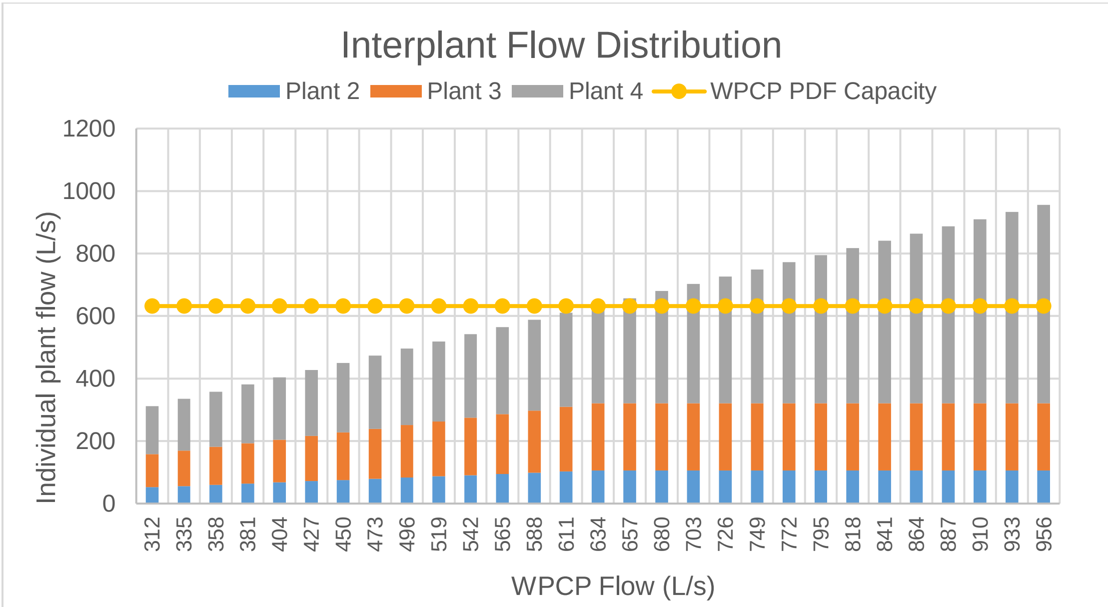


Figure 8.1 – Interplant Flow Distribution Optimization

8.4 Long Term I&I Mitigation Measures

8.4.1 Introduction

The age and parcel fabric over the St. Thomas catchment varies considerably and in connection with that, a high deviation in I&I rates were observed for monitored sewersheds both in the current PPCP and in the 2015 Study for the Aldborough/ Leger and Woodworth Avenue SPS study areas. To determine a holistic solution that best balances the cost effectiveness of I&I reduction measures, collection system improvements and wet weather capacity improvements to the WPCP, an ongoing program to improve the City's understanding of the collection system is recommended. The hydraulic model requires further fine-tuning through flow monitoring data in previously unmonitored areas. This effort should be carried out in conjunction with I&I analysis.

8.4.2 Installation of a Permanent Rain Gauge

The City of St. Thomas should install a permanent rain gauge station at the WPCP to be able to conduct future flow monitoring assignments and have concurrent rain data information in 5-minute interval resolution to match standard flow monitoring timesteps. A further benefit of such an installation would be the ability to measure extreme storm events and replicate known flood events for further hydraulic model calibration.

8.4.3 Annual Flow Monitoring Program

We would recommend that the City install three to four flow meters per year to better understand the inflow within the collection system over the next 5 years. Each flow meter would be in place for a 9-month period. Using the data from the rain gauge information and the flow data collected, the current model can be updated to better reflect the conditions in the system and to address any issues that the City wishes to review. After year 5 of the flow monitoring, the City can decide if they wish this annual program to continue.

8.4.4 Annual Updating of the Hydraulic Model

Based upon the collected rainfall data and the annual flow monitoring program, it is recommended to carry out further flow analysis in future to refine the model calibration and peak flow response in combination to finding areas with extreme I&I rates and exploring the sources of inflow. A future objective should become to eliminate portions of I&I where practicable and cost efficient. We foresee considerable scope of wet weather flow reduction from targeted improvement assignments once the overall system is better understood.

8.5 PPCP Recommendations and MECP Policy F-5-5

The implementation of the recommendations of this PPCP will allow the City to further meet Section 6, Minimum Combined Sewer Overflow (CSO) Controls, of MECP F-5-5 Determination of Treatment Requirements for Municipal and Private Combined and Partially Separated Sewers. This is summarized in Table 8.3.

Table 8.3 – Policy F-5-5 Section 6.0 Compliance

#	Description	Current PPCP Plan
1.	Eliminate CSOs during dry-weather periods except under emergency conditions.	Presently met but improvements to pumping stations, WPCP bottlenecks, and CSO overflow limit will improve this situation.
2.	Each municipality shall demonstrate that the combined sewer system, including the regulators, and associated treatment facilities are adequate for the transmission and treatment of all peak dry weather flows from the service area. An emergency condition would exist when e.g., basement flooding, damage to equipment at treatment works or pumping stations, or treatment process washout was occurring or was imminent.	City has capital plan program to reduce combined sewers. This is presently met but improvements to WPCP bottlenecks and CSO overflow limit will reduce sewage escape to the environment. PPCP plan includes additional flow monitoring and modeling to better define overflow potentials and sewer lining program will reduce high flows to CSO.
3.	Establish and implement Pollution Prevention programs that focus on pollutant reduction activities at source e.g. reduced use of potential pollutants like fertilizer and pesticides in parks; public education programs on e.g. anti-littering and illegal dumping of used motor oil and other materials into catchbasins; water conservation to reduce dry weather sanitary flow and hence CSOs; street cleaning to reduce CSO floatables; roof-leader disconnection and installing rain barrels to reduce flows into the sewer system; education/assistance for industries to minimize the use/discharge of pollutants; and enforcement of municipal by-laws or regulations.	City has capital plan program to reduce combined sewers. Improvements to WPCP capacity and CSO overflow limit will reduce sewage escape to the environment. City has standard operating procedures in place to address this.

#	Description	Current PPCP Plan
4.	Establish and implement proper operation and regular inspection and maintenance programs for the combined sewer system in order to ensure continued proper system operation.	PPCP plan includes additional flow monitoring and modeling to better define overflow potentials and sewer lining program will reduce high flows to CSO.
5.	Establish and implement a floatables control program to control coarse solids and floatable materials e.g., by reducing the amount of street litter that enters the catchbasins and the CSS; by removing debris from CSOs at the outfalls using measures such as trash racks and screens; and by removing floatables from the surface of the receiving water after a CSO occurs.	City has standard operating procedures in place to address this.
6.	Maximize the use of the collection system for the storage of wet weather flows which are conveyed to the Sewage Treatment Plant for treatment when capacity is available e.g., by adjusting regulator settings.	Improvements to WPCP capacity and CSO overflow limit will meet this requirement.
7.	Maximize the flow to the Sewage Treatment Plant for the treatment of wet weather flows e.g., by removing obstructions to flow.	Improvements to WPCP capacity and CSO overflow limit will meet this requirement.
8.	The secondary treatment capacity should be utilized as much as possible for treating wet weather flows with the balance of flows being subject to primary treatment. Measures to increase the wet weather hydraulic capacity at the Sewage Treatment Plant (e.g., Step Feed operation) should be investigated.	Improvements to WPCP capacity and CSO overflow limit will meet this requirement.
9.	During a seven-month period commencing within 15 days of April 1, capture and treat for an average year all the dry weather flow plus 90% of the volume resulting from wet weather flow that is above the dry weather flow. The volumetric control criterion is applied to the flows collected by the sewer system immediately above each overflow location unless it can be shown through modelling and on-going monitoring that the criterion is being achieved on a system-wide basis. No increases in CSO volumes above existing levels at each outfall will be allowed except where the increase is due to the elimination of upstream CSO outfalls. During the remainder of the year, at least the same storage and treatment capacity should be maintained for treating wet weather flow.	The overall collection system currently meets this requirement.

9.0 PPCP IMPLEMENTATION PLAN

9.1 Levels of Cost Estimation

ASTM E 2516-11 (Standard Classification for Cost Estimate Classification System) provides a five-level classification system based on several characteristics, with the primary characteristic being the level of project definition (i.e., percentage of design completion). The ASTM standard, shown in Table 9.1, illustrates the typical accuracy ranges that may be associated with the general building industries.

Table 9.1 –ASTM E2516-11 Accuracy Range of Cost Opinions for General Building Industries

Cost Estimate Class	Expressed as % of Design Completion	Anticipated Accuracy Range as % of Actual Cost
5	0-2	-30 to +50
4	1-15	-20 to +30
3	10-40	-15 to +20
2	30-70	-10 to +15
1	50-100	-5 to +10

The cost estimates developed in this report would be best described as a Class 5 Cost Estimate which is typically used for high level study projects.

9.2 Cost Estimate Assumptions

The total cost estimate was prepared based on;

1. Assumed prices from RVA experience for SPS upgrades, rain gauge, flow monitoring and modeling;
2. For sewers and forcemains, costing data from Table 3-3 of the City of London 2014 Wastewater Servicing DC Update Unit Rate Costs for Watermains (Revised March 2014) adjusted for inflation;
3. Assessment that CSO improvements will involve programing changes only; and
4. For construction 20% of the construction cost was estimated for planning and engineering costs.

9.3 Design Cost Estimate

A Class 5 cost estimate prepared by RVA are detailed as follow:

- Table 9.2 itemizes the PPCP recommended program costs by project;
- Table 9.3 provides a summary of the expected cashflow for the PPCP; and
- Table 9.4 provides a summary of the cumulative costs over the 5,10, 20 and 40year periods.

Additional details on the development of the cost opinions are provided in **Appendix 7**.

Table 9.2 – Conceptual Cost Opinion Per Recommended Item

Component	Cost Estimate Per Activity			Timeframe/ Comment
	Capital	Planning and Engineering	Total	
Recommended Collection System Upgrades				
Walnut (Sunset) SPS Improvements to coordinate with PPCP	$0	$25,000	$25,000	Assume that this may be only a design change in the new PS and not impact the construction cost.
Additional cost to reroute the new Walnut SPS forcemain to the CSO	$100,000	$20,000	$120,000	Undertake following upgrades to WPCP to remove bottlenecks when now Walnut St SPS is being built.
Woodworth Ave SPS Upgrades	$2,500,000	$500,000	$3,000,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Woodworth Ave SPS Collection System	$3,849,283	$577,392	$4,426,675	When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Upgrades	$2,000,000	$400,000	$2,400,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Downstream Collection System	$1,500,000	$225,000	$1,725,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Annual Sewer Lining (500 m/year)	$650,000	$65,000	$715,000	It will take 62 years to undertake the lining of the current total of 31 km of fair to poor sanitary sewers in the system.
CSO Operation Optimization				
Improvements to CSO Tank Operation	$0	$100,000	$100,000	2023 - undertake following upgrades to WPCP to remove bottlenecks. Assume that this involves changes in controls only. Does not include costs for removing bottlenecks in WPCP.
Removal of Capacity Constraints at the WPCP				

Component	Cost Estimate Per Activity			Timeframe/ Comment
	Capital	Planning and Engineering	Total	
Remove WPCP Bottlenecks	$2,727,000	$273,000	$3,000,000	2022-23 -Modify plant flow distribution, remove pipe bottlenecks, twin UV channel, and add a new parallel unit, upgrade outfall pipe.
Long Term I & I Mitigation Measures				
Permanent Rain Gauge Installation	$15,000	$4,000	$19,000	Early 2022 installation.
Annual Camera Work in Collection System	$250,000	$0	$250,000	Yearly work ($50,000) over a 5-year period.
Flow Monitor Installation, Maintenance, Removal	$176,000	$0	$176,000	Yearly work ($35,200) over a 5-year period.
Building on the Current Hydraulic Model	$0	$79,000	$79,000	Yearly work (15,800) over a 5-year period.

Table 9.3 – PPCP Cashflow

Component	Cashflow (Years)			Timeframe/ Comment
	1 to 5	6-10	11 to 20	
Recommended Collection System Upgrades				
Walnut (Sunset) SPS Improvements to coordinate with PPCP	$25,000			Assume that this may be only a design change in the new PS and not impact the construction cost.
Additional cost to reroute the new Walnut SPS forcemain to the CSO	$20,000	$120,000		Undertake following upgrades to WPCP to remove bottlenecks when now Walnut St SPS is being built.
Woodworth Ave SPS Upgrades	$4,800,000			When City deems necessary to do/ High level estimate/ City may look at other options.

Component	Cashflow (Years)			Timeframe/ Comment
	1 to 5	6-10	11 to 20	
Woodworth Ave SPS Collection System	$4,426,675			When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Upgrades		$2,400,000		When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Downstream Collection System		$1,725,000		When City deems necessary to do/ High level estimate/ City may look at other options.
Annual Sewer Lining (500 m/year)	3,575,000	$3,575,000	$7,150,000	Start sewer lining in year 3 after 2 years of additional modeling and data city will take 41 years to undertake the lining of the current total of 31 km of fair to poor sanitary sewers in the system.
CSO Operation Optimization				
Improvements to CSO Tank Operation	$100,000			2023 - undertake following upgrades to WPCP to remove bottlenecks. Assume that this involves changes in controls only. Does not include costs for removing bottlenecks in WPCP.
Removal of Capacity Constraints at the WPCP				
Remove WPCP Bottlenecks	$3,000,000			2022-23 -Modify plant flow distribution, remove pipe bottlenecks, twin UV channel, and add a new parallel unit, upgrade outfall pipe.
Long Term I & I Mitigation Measures				
Permanent Rain Gauge Installation	$19,000			Early 2022 installation.
Annual Camera Work in Collection System	$250,000	$250,000		Yearly work ($50,000) over a 5-year period. Stop at year 10 when a new MP should be undertaken.
Flow Monitor Installation,	$176,000	$176,000		Yearly work ($35,200) over a 5-year period. Stop at

Component	Cashflow (Years)			Timeframe/ Comment
	1 to 5	6-10	11 to 20	
Maintenance, Removal				year 10 when a new MP should be undertaken.
Building on the Current Hydraulic Model	$79,000	$79,000		Yearly work ($15,800) over a 5-year period. Stop at year 10 when a new MP should be undertaken.

Table 9.4 – PPCP Cumulative Program Cost

	To Year 5	To Year 10	To Year 20
Estimated Cost	$16,470,675	$24,795,675	$31,945,675
Low (-30%)	$11,529,473	$17,356,973	$22,361,973
High (+50%)	$24,706,013	$37,193,513	$47,918,513

APPENDIX 1

Public Consultation

APPENDIX 1-1

Public Notices

ENVIRONMENTAL SERVICES DEPARTMENT
545 Talbot St., P.O. Box 520, City Hall
St. Thomas, Ontario, N5P 3V7
t. (519) 631.1680

Notice of Study Commencement
St. Thomas Pollution Prevention Control Plan

The City of St. Thomas is preparing a Pollution Prevention Control Plan (PCPP) as part of ongoing efforts being undertaken to improve the performance of the City's sanitary and storm sewer infrastructure. The PPCP will provide the City with a road map for implementation of infrastructure and operational improvements that will mitigate the impacts of wet weather sewer system overflows on the environment. This is in alignment with the City's commitment to environmental stewardship and the provision of sustainable municipal services.

The study is being undertaken in accordance with the requirements of the Municipal Class Environmental Assessment (EA) process for Master Plans (Municipal Engineer's Association Class EA document October 2000, as amended in 2007, 2011 & 2015).

How Do I Get Involved?

There will be opportunities to participate throughout the study. Two public engagement events will be held during the study to provide opportunities to review project information and provide feedback to the study team. Project updates will be available on the City's website www.stthomas.ca/P_P_C_P.

For more information, or to be added to the study's distribution list to receive updates, please contact a member of the study team below:

Nathan Bokma, P. Eng.
Manager of Development and Compliance
Environmental Services Dept.
City of St. Thomas
Tel: 519-631-1680 ext. 4151
nbokma@stthomas.ca
545 Talbot St., PO Box 520
St. Thomas, ON N5P 3V7

John Tyrrell, M.Sc. (Eng.), P. Eng.
Senior Project Manager, Municipal
R.V. Anderson Associates Limited
Tel: 519-681-9916 ext. 5038
jtyrrell@rvanderson.com
557 Southdale Road East, Suite 200
London, ON N6E 1A2

This notice was first distributed on June 25, 2020

ENVIRONMENTAL SERVICES DEPARTMENT
545 Talbot St., P.O. Box 520, City Hall
St. Thomas, Ontario, N5P 3V7
t. (519) 631.1680

Notice of Virtual Public Consultation Meeting
St. Thomas Transportation Master Plan Update

The City of St. Thomas is updating its Transportation Master Plan (TMP) to serve as a long-range strategic plan for the City. The TMP will address existing transportation challenges and opportunities, support growth, and recommend policies to promote an efficient, multi-modal transportation network which fosters vehicular, bicycle, pedestrian, and transit mobility. The study will provide an assessment of the City's transportation improvement needs and provide recommendations to improve operational, design, and transportation policies which St. Thomas uses to manage its transportation infrastructure.

How Do I Participate?

We are hosting the first round of Virtual Public Consultation Meetings to provide more information on the Master Plan Update and to provide you with the opportunity to share information with the project team relating to the existing or future transportation network in St. Thomas. The Public Consultation Meeting will be hosted online using the Zoom platform and will include a presentation by the project team followed by a live Question and Answer Session.

When? Wednesday, September 9, Afternoon Session: 2:00-3:30pm; Evening Session: 6:00pm-7:30pm

How? **Afternoon Session (2:00-3:30pm):** To join the meeting through your computer, tablet, or smartphone, click on the following link: https://zoom.us/j/96852881669

If you prefer to join in and listen live via telephone, dial 1 647 558 0588 (long distance charges may apply). When prompted, dial the Webinar ID: 968 5288 1669

Evening Session (6:00-7:30pm): https://zoom.us/j/98056546236

If you prefer to join in and listen live via telephone, dial: 1 647 558 0588 (long distance charges may apply), and when prompted, dial the Webinar ID: 980 5654 6236

The presentation, along with a summary of the Question and Answers will be posted on the project webpage following the meeting: www.stthomas.ca/TMP. More information on participating in a Zoom meeting is provided on the following page.

For more information, or to be added to the study's distribution list to receive updates, please contact a member of the study team below:

Nathan Bokma, P. Eng.
Manager of Development and Compliance
City of St. Thomas
Phone: 519-631-1680 ext. 4151
nbokma@stthomas.ca

Brandon Orr, BES, MCIP, RPP
Project Manager
Stantec Consulting Ltd.
Direct: 437-221-5339
Brandon.Orr@stantec.com
100-401 Wellington Street West

This notice was first distributed on August 26, 2020

ST. THOMAS
THE RAILWAY CITY

ENVIRONMENTAL SERVICES DEPARTMENT
545 Talbot St., P.O. Box 520, City Hall
St. Thomas, Ontario, N5P 3V7
t. (519) 631.1680

Participating in a Zoom Meeting:

As a Zoom meeting attendee, you can participate in the meeting by listening, asking a question verbally or by text using the Chat feature. Note that all participants will be muted until the presentation by the Project Team is complete.

How to Ask a Question:

Using a computer, tablet, or smartphone:

1. To use your device's audio to ask your question, when prompted, click the "raise hand" icon at the bottom of the Zoom window. (INSERT SCREEN SHOT/ICON). We will call you by name when you have been unmuted so you can ask your question to the project team.
2. If you no longer wish to ask your question, simply click the same icon, now labelled "lower hand".
3. Alternatively, you can type your question using the "Question and Answer" feature. Simply click the Q&A icon at the bottom of the Zoom window, type your question in the answer box, and click "send."

Using a Telephone:

1. When prompted, press *9 to raise your hand. We will call you by name when you have been unmuted so you can ask your question to the project team.

Virtual Meeting Tips, Tricks, and meeting Etiquette

1. Pick a spot with a strong internet connection (e.g. don't join the meeting in your backyard where your wifi connection is weak)
2. Close all other applications or web pages on your device
3. Ensure you are using the latest version of your device's web browsing software

This notice was first distributed on August 26, 2020

ENVIRONMENTAL SERVICES DEPARTMENT
545 Talbot St., P.O. Box 520, City Hall
St. Thomas, Ontario, N5P 3V7
t. (519) 631.1680

Notice of Virtual Public Consultation Meeting
St. Thomas Pollution Prevention Control Plan

The City of St. Thomas is preparing a Pollution Prevention Control Plan (PPCP), which is part of ongoing efforts being undertaken to improve the performance of the City's sanitary and storm sewer infrastructure. The PPCP will provide the City with a road map for implementation of infrastructure and operational improvements that will mitigate the impacts of wet weather sewer system overflows on the environment. This is in alignment with the City's commitment to environmental stewardship and the provision of sustainable municipal services.

The study is being undertaken in accordance with the requirements of the Municipal Class Environmental Assessment (EA) process for Master Plans (Municipal Engineer's Association Class EA document October 2000, as amended in 2007, 2011 & 2015).

How do I Participate?

We are hosting the second of two Virtual Public Consultation Meetings to review the findings and next steps of the PPCP and provide members of the public with an opportunity to provide comments. The Public Consultation Meeting will be hosted online using the Zoom platform and will include a brief presentation by the project team followed by a live Question and Answer Session.

When? Wednesday December 1, 2021 from 4:00 – 5:00 p.m.

How? To join the meeting through your computer, tablet, or smartphone, click on the following link: **https://us06web.zoom.us/j/81549233728**

If you prefer to join in and listen live via telephone, dial 1 (647) 374-4685 (long distance charges may apply). When prompted, dial the Webinar ID: 815 4923 3728

The presentation, along with a summary of Question and Answers will be posted on the project webpage following the meeting: **www.stthomas.ca/P_P_C_P**

For more information, or to be added to the study's distribution list to receive updates, please contact a member of the study team below:

Nathan Bokma, P. Eng.
Manager of Development and Compliance
Environmental Services Dept.
City of St. Thomas
Tel: 519-631-1680 ext. 4151
nbokma@stthomas.ca
545 Talbot St., PO Box 520
St. Thomas, ON N5P 3V7

John Tyrrell, M.Sc. (Eng.), P. Eng.
Senior Project Manager, Municipal
R.V. Anderson Associates Limited
Tel: 519-681-9916 ext. 5038
jtyrrell@rvanderson.com
557 Southdale Road East, Suite 200
London, ON N6E 1A2

This notice was first distributed on November 10, 2021

ST. THOMAS
THE RAILWAY CITY

ENVIRONMENTAL SERVICES DEPARTMENT
545 Talbot St., P.O. Box 520, City Hall
St. Thomas, Ontario, N5P 3V7
t. (519) 631.1680

Participating in a Zoom Meeting:

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How to Ask a Question:

Using a computer, tablet, or smartphone:

1. To use your device's audio to ask your question, when prompted, click the "raise hand" icon at the bottom of the Zoom window. To locate the "raise hand" icon, first click on the icon labelled "Participants" at the bottom centre of your screen. At the bottom of the window on the right side of the screen, click the button labeled "raise hand". Your digital hand is now raised. You will be called by name when you have been unmuted so you can ask your question to the project team.
2. If you no longer wish to ask your question, simply click the same icon, now labelled "lower hand".
3. Alternatively, you can type your question using the "Question and Answer" feature. Simply click the Q&A icon at the bottom of the Zoom window, type your question in the answer box, and click "send".

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2. Close all other applications or web pages on your device.
3. Ensure you are using the latest version of your device's web browsing software.

This notice was first distributed on November 10, 2021

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APPENDIX 1-2

Agency/Public Contact List and Notifications

St. Thomas Pollution Prevention Control Plan
Technical Agency Stakeholder Contact List

Updated: 2022-01-12

Agency	Contact	Title	Email	Address	Phone	*Notes*	Date Added to List	Removed from List	Notice of Study (date sent)	Notice of PIC1 (date sent)	Notice of PIC2 (date sent)
Provincial Ministries, Agencies and Departments											
Ministry of the Environment, Conservation and Parks (MECP)	Southwest Region		eanotification.swregion@ontario.ca	733 Exeter Road, London, ON N6E 1L3	1-800-265-7672	*Complete the **project information form** and send copy of notice + form by email* ***NOTICE OF COMMENCEMENT ONLY***					
	General (Notices)		MEA.Notices.EAAB@ontario.ca			***NOTICE OF COMPLETION ONLY***					
Ministry of Natural Resources and Forestry (MNRF)	Karina Cerniavskaja	District Planner - Aylmer	karina.cerniavskaja@ontario.ca	615 John Street N., Aylmer, ON N5H 2S8	519-773-4757						12-Nov-21
Ministry of the Environment, Conservation and Parks (MECP)	Scott Abernethy	Surface Water Evaluator/Team Leader	scott.abernethy@ontario.ca	733 Exeter Road, London, ON N6E 1L3	519-873-4779	***Notice of PIC #2 Undeliverable***			20-Jul-20		12-Nov-21
Ministry of the Environment, Conservation and Parks (MECP)	Roland Plante	Water Inspector	roland.plante@ontario.ca	733 Exeter Road, London, ON N6E 1L3	519-281-1508				20-Jul-20		12-Nov-21
Ministry of the Environment, Conservation and Parks (MECP)	Trevor Bell	Environmental Resource Planner / EA Coordinator	trevor.bell@ontario.ca	5775 Yonge Street, 8th Floor Toronto, ON	416-326-3577				20-Jul-20		12-Nov-21
Ministry of the Environment, Conservation and Parks (MECP)	Ron Griffiths	Surface Water Specialist	ron.griffiths@ontario.ca	733 Exeter Road, London, ON N6E 1L3	519-873-5015	***Notice of PIC #2 Undeliverable***			20-Jul-20	6-Oct-20	12-Nov-21
Ministry of the Environment, Conservation and Parks (MECP)	Kathryn Markham	Management Biologist	kathryn.markham@ontario.ca	615 John St. N, Aylmer, ON N5H 2S8	519-773-4711				20-Jul-20	6-Oct-20	12-Nov-21
Ministry of the Environment, Conservation and Parks (MECP)	Mark Smith	Water Compliance Supervisor	Mark.Smith@ontario.ca	733 Exeter Road, London, ON N6E 1L3	519-317-8116				20-Jul-20	6-Oct-20	12-Nov-21
Ministry of Municipal Affairs and Housing (EA Policy)	Erick Boyd	Manager (Acting)	erick.boyd@ontario.ca	659 Exeter Road, 2nd Floor, London, ON N6E 1L3	519-873-4031				20-Jul-20	6-Oct-20	12-Nov-21
Ministry of Agriculture, Food and Rural Affairs	David Marriott	Rural Planner, Western Ontario	david.marriott@ontario.ca	1 Stone Road W, 3rd Floor, Guelph, ON N1G 4Y2	519-766-5990				20-Jul-20	6-Oct-20	12-Nov-21
Ministry of Economic Development, Job Creation and Trade	David B. Meyer	Director	david.b.meyer@ontario.ca	30th Flr Suite 3001, 250 Yonge St, Toronto, ON M5B 2L7	416-212-6280				20-Jul-20	6-Oct-20	12-Nov-21
Ministry of Tourism, Culture and Sport (MTCS)	Karla Barboza	Team Lead(A), Heritage Heritage Planning Unit Programs and Services Branch	karla.barboza@ontario.ca	Suite 1700, 401 Bay Street, Toronto ON M7A 0A7	416-314 7120				20-Jul-20	6-Oct-20	12-Nov-21
Ministry of Tourism, Culture and Sport (MTCS)	Dan Minkin	Heritage Planner (Culture Services Unit)	Dan.Minkin@ontario.ca	Suite 1700, 401 Bay Street, Toronto, ON M7A 0A7	416-314-7147				20-Jul-20	6-Oct-20	12-Nov-21
Ministry of Tourism, Culture and Sport (MTCS)	Rosi Zirger	Heritage Planner (Culture Services Unit)	rosi.zirger@ontario.ca	Suite 1700, 401 Bay Street, Toronto, ON M7A 0A7	416-314-7159				20-Jul-20	6-Oct-20	12-Nov-21
Ministry of Indigenous Affairs	Lise Chabot	Manager, Ministry Partnerships Unit	lise.Chabot@ontario.ca	Suite 400, 160 Bloor St. E, Toronto, ON M7A 2E6	647-532-0761				20-Jul-20	6-Oct-20	12-Nov-21
Environmental Assessment and Permissions Branch		Director	enviropermissions@ontario.ca	135 St. Clair Avenue West, 1st Floor, Toronto ON M4V 1P5					20-Jul-20	6-Oct-20	12-Nov-21
Municipal, MPs, MPPs											
City of St. Thomas	Justin Lawrence	Director and City Engineer	jlawrence@stthomas.ca	545 Talbot Street, PO Box 520, St. Thomas, ON N5P 3V7					20-Jul-20	6-Oct-20	12-Nov-21
City of St. Thomas	Wendall Graves	City Manager	wgaves@stthomas.ca	545 Talbot Street, PO Box 520, St. Thomas, ON N5P 3V7		***Notice of PIC #2 Undeliverable***			20-Jul-20	6-Oct-20	12-Nov-21
County of Elgin	Brian Lima	General Manager, Engineering, Planning, Enterprise	blima@elgincounty.ca	450 Sunset Drive, St.Thomas, ON N5R 5V1		***Notice of PIC #2 Undeliverable***			20-Jul-20	6-Oct-20	12-Nov-21
County of Elgin	Julie Gonyou	CAO	jgonyou@elgincounty.ca	450 Sunset Drive, St.Thomas, ON N5R 5V1		***Notice of PIC #2 Undeliverable***			20-Jul-20	6-Oct-20	12-Nov-21
County of Elgin	Nancy Pasato	Manager of Planning	npasato@elgincounty.ca	450 Sunset Drive, St.Thomas, ON N5R 5V1		***Notice of PIC #2 Undeliverable***			20-Jul-20	6-Oct-20	12-Nov-21
									20-Jul-20	6-Oct-20	12-Nov-21
Municipality of Central Elgin	Geoff Brooks	Director of Infrastrucutre and Commmunity Services	gbrooks@centralelgin.org	450 Sunset Drive, St.Thomas, ON N5R 5V1					20-Jul-20	6-Oct-20	12-Nov-21
Towship of Southwold	Lisa Higgs	CAO	cao@southwold.ca	35663 Fingal Line, Fingal, ON N0L 1K0					20-Jul-20	6-Oct-20	12-Nov-21
	Peter Kavic	Director of Infrastrucutre and Development Services	development@southwold.ca	35663 Fingal Line, Fingal, ON N0L 1K0	519-769-2010				20-Jul-20	6-Oct-20	12-Nov-21
Provincial MPP	Jeff Yurek	MPP Elgin-Middlesex-London	jeff.yurekco@pc.ola.org	201 West Wing- 750 Talbot Street, St.Thomas, ON N5P 1E2	519-631-0666				20-Jul-20	6-Oct-20	12-Nov-21
Federal MP	Karen Vecchio	MP Elgin-Middlesex-London	karen.vecchio@parl.gc.ca	203-750 Talbot Street, St. Thomas, ON N5P 1E2					20-Jul-20	6-Oct-20	12-Nov-21
Elgin-St.Thomas Health Unit				1230 Talbot Street, St. Thomas, ON N5P 1G9	519-631-9900 ext. 1250				20-Jul-20	6-Oct-20	12-Nov-21
City of London	????								20-Jul-20	6-Oct-20	12-Nov-21
Conservation Authority											
Kettle Creek Conservation Authority	Joe Gordon	Assistant Manager, Supervisor of Planning & Conservaton Areas	joe@kettlecreekconservation.on.ca	44015 Ferguson Line, St. Thomas, ON N5P 3T3	519-631-1270 ext. 226				20-Jul-20	6-Oct-20	12-Nov-21

Agency	Contact	Title	Email	Address	Phone	*Notes*	Date Added to List	Removed from List	Notice of Study (date sent)	Notice of PIC1 (date sent)	Notice of PIC2 (date sent)
Catfish Creek Conservation Authority	Christopher Wilkinson	General Manager / Secretary-Treasurer	generalmanager@catfishcreek.ca	8079 Springwater Road, RR#5 Aylmer, ON N5H 2R4	519-773-9037				20-Jul-20	6-Oct-20	12-Nov-21
Indigenous Groups											
Metis Natio of Ontario	Margaret Frosh	Chief	MargaretF@metisnation.org	311-75 Sherbourne Street, Toronto, ON M5A 2P9					20-Jul-20	6-Oct-20	12-Nov-21
	Linda Norheim	Director, Lands, Resources and Consultations	lindan@metisnation.org		416-977-9881				20-Jul-20	6-Oct-20	12-Nov-21
			consultations@metisnation.org	Métis Consultation Unit Métis Nation of Ontario Head Office Suite 1100 – 66 Slater Street Ottawa, ON K1P 5H1					20-Jul-20	6-Oct-20	12-Nov-21
Aamjiwnaang First Nation	Chris Plan	Chief	chief.plain@aamjiwnaang.ca	978 Tashmoo Avenue, Sarnia, ON N7T 7H5	519-336-8410				20-Jul-20	6-Oct-20	12-Nov-21
Caldwell First Nation	Mary Frances Duckworth	Chief	chief.duckworth@caldwellfirstnation.ca	P.O. Box 388 Leamington, ON N8H 3W3					20-Jul-20	6-Oct-20	12-Nov-21
Chippewas of Kettle and Stony Point First Nation	Tom Bressette	Chief	thomas.bressette@kettlepoint.org	6247 Indian Lane, Forest, ON N0N 1J0					20-Jul-20	6-Oct-20	12-Nov-21
Chippewa of the Thames First Nation	Fallon Burch	Consultation Coordinator - Lands & Environment		320 Chippewa Road, Muncey, ON N0L 1Y0	519-289-2662 ext. 213				20-Jul-20	6-Oct-20	
	Leslee-White-Eye	Chief									
Delaware Nation (Moravian of the Thames)	Greg Peters	Chief	apeters@mnsi.net	14760 School House Line RR3 Thamesville ON N0P 2K0					20-Jul-20	6-Oct-20	12-Nov-21
	Justin Logan		loganju@xplornet.ca						20-Jul-20	6-Oct-20	12-Nov-21
Munsee-Delaware Nation	Roger Thomas	Chief	Chief.thomas@munsee-delaware.org	279 Jubilee Road, Muncey ON N0L 1Y0		***Notice of PIC #2 Undeliverable***			20-Jul-20	6-Oct-20	12-Nov-21
	Glen Forest										
Oneida of the Thames First Nation	Sheri Doxtator	Chief	sheri.doxtator@oneida.on.ca	2212 Elm Ave, Southwold, ON N0L 2G0	519-652-6161	***Notice of PIC #2 Undeliverable***			20-Jul-20	6-Oct-20	12-Nov-21
	Holly Elijah										
Bkejwanong Territory (Walpole Island)	Dan Miskokomon			Wallaceburg, ON N8A 4K9							
	Jarend Macbeth		Jared.macbeth@wifn.org						20-Jul-20	6-Oct-20	12-Nov-21

John Tyrrell

From: Tisha Doucette
Sent: July 30, 2020 4:02 PM
Cc: John Tyrrell; Bokma, Nathan
Subject: Notice of Study Commencement, Pollution Prevention Control Plan - City of St. Thomas
Attachments: PPCP_Notice of Study Commencement - FINAL.pdf

Good afternoon,

Attached please find a Notice of Study Commencement for the Pollution Prevention Control Plan being prepared by the City of St. Thomas.

Thank you for your participation in this study.

Kind regards,
Tisha Doucette

rva

RVA IS GROWING!

Our NEW Halton and Halifax offices are now open.

Tisha Doucette, B.Sc., EP
Senior Planning Ecologist

P: (519) 681-9915 ext. 5035
C: (519) 868-1658

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557 Southdale Road East, Suite 200, London, ON N6E 1A2

rvanderson.com

RVA celebrates the summer season from June 26th to September 4th. Our offices will be closed at 2 pm each Friday.

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From: Tisha Doucette
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From:	Tisha Doucette
Sent:	July 30, 2020 4:46 PM
To:	eanotification.swregion@ontario.ca
Cc:	John Tyrrell; Bokma, Nathan
Subject:	Notice of Study Commencement - St. Thomas Pollution Prevention Control Plan
Attachments:	PPCP_Notice of Study Commencement - FINAL.pdf; streamlined_ea_project_information_form_2.xlsx

Good afternoon,

Please see attached.

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Senior Planning Ecologist

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C: (519) 868-1658

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APPENDIX 1-3

Agency/Public Responses

APPENDIX 1-4

PIC 1 October 21, 2020

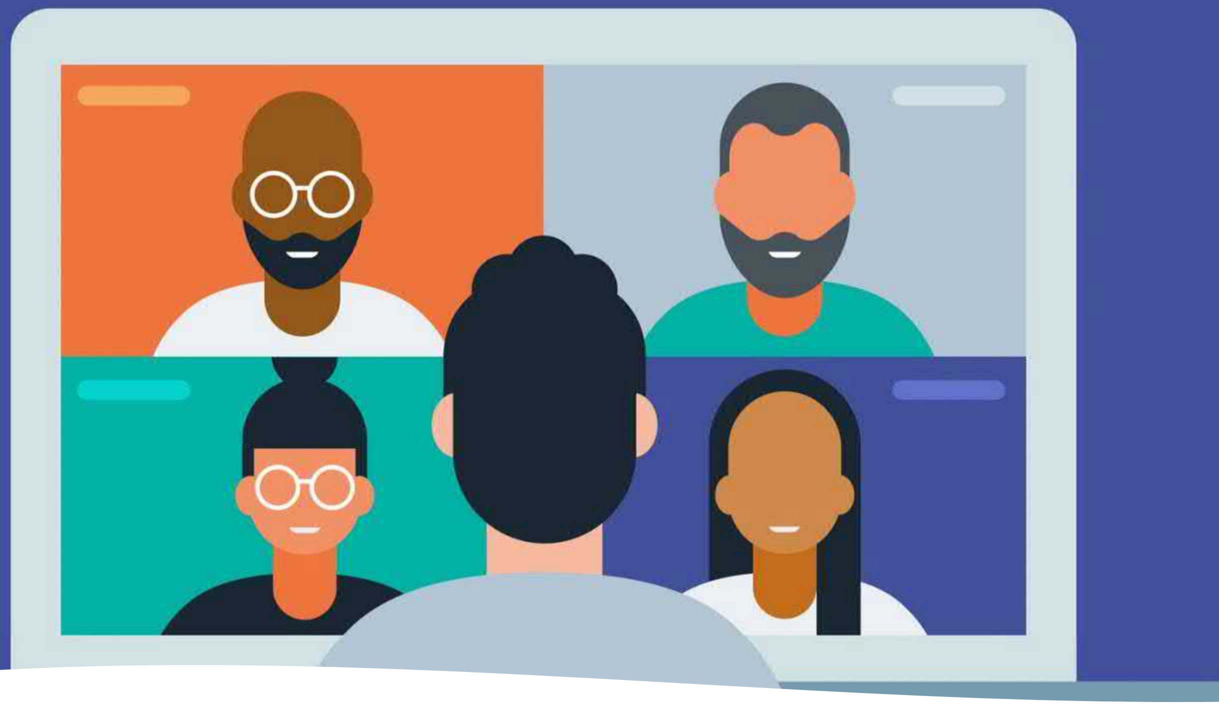
ST. THOMAS
THE RAILWAY CITY

Pollution Prevention Control Plan

Virtual Public Consultation Meeting
October 21, 2020

R.V. Anderson Associates Limited
engineering • environment • infrastructure

CANADA'S BEST MANAGED COMPANIES Platinum member



Virtual Meeting Format

- Presentation by Project Team.
- Question and Answer Period "Raise Your Hand" or Dial "9".
- Presentation, Transcript and Question and Answer Summary will be available at **www.stthomas.ca/P_P_C_P** after the meeting.
- Please provide your comments by November 6, 2020.

Purpose of Meeting

- Introduce you to the study.
- Provide an overview of the study process.
- Identify the issues and reason for this study.
- Summarize the current state of the City's water and wastewater collection system, and water quality of receiving water bodies.

We want to hear from you!

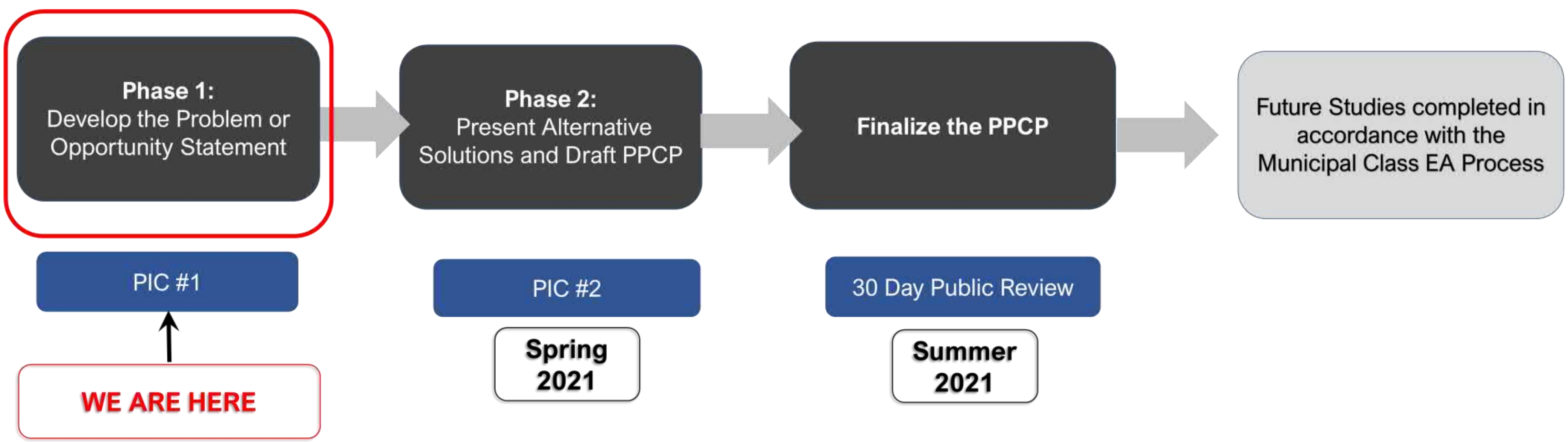
- Do you have any observations that you would like to share?
- Do you have any questions regarding the study?
- Do you have any questions regarding the Master Plan process?

Municipal Class Environmental Assessment

This study is being undertaken in accordance with the Municipal Class Environmental Assessment process for Master Plans.

Master Plans are long range plans, which integrate infrastructure requirements for existing and future land use with environmental assessment principles.

This Master Plan, the Pollution Prevention Control Plan, will address Phases 1 and 2 of the Municipal Class EA process.

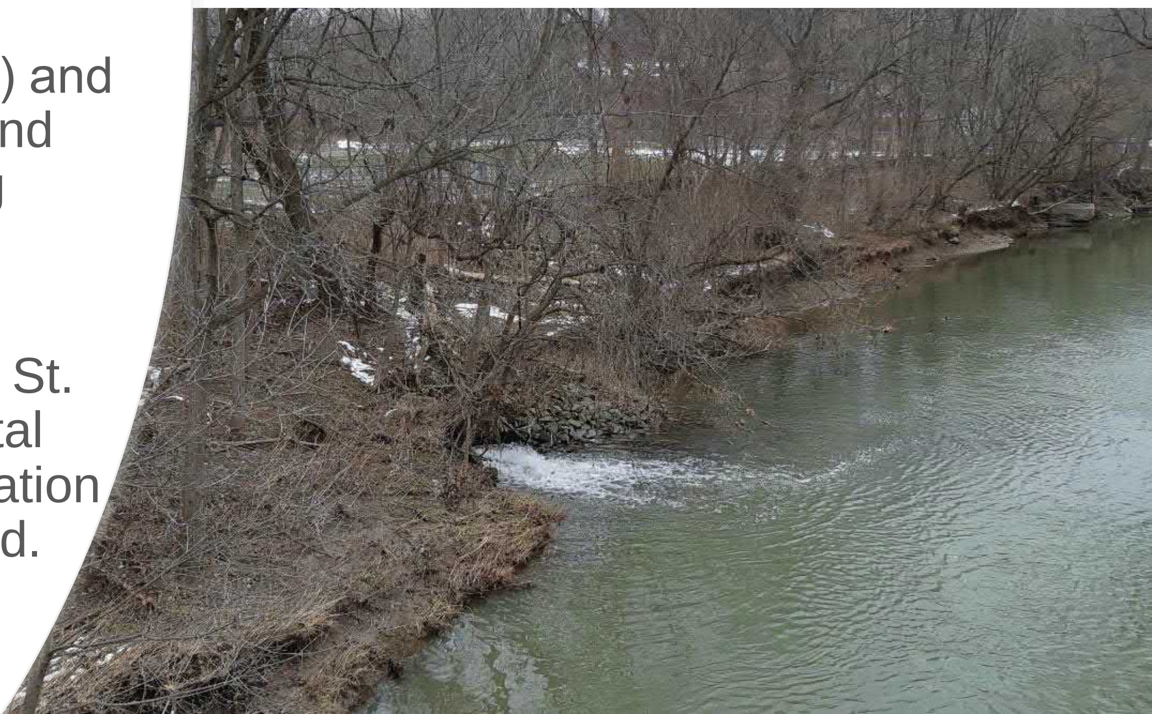


Problem or Opportunity Statement

The PPCP will be a part of the City's ongoing efforts to improve the performance of our sanitary and storm sewer infrastructure.

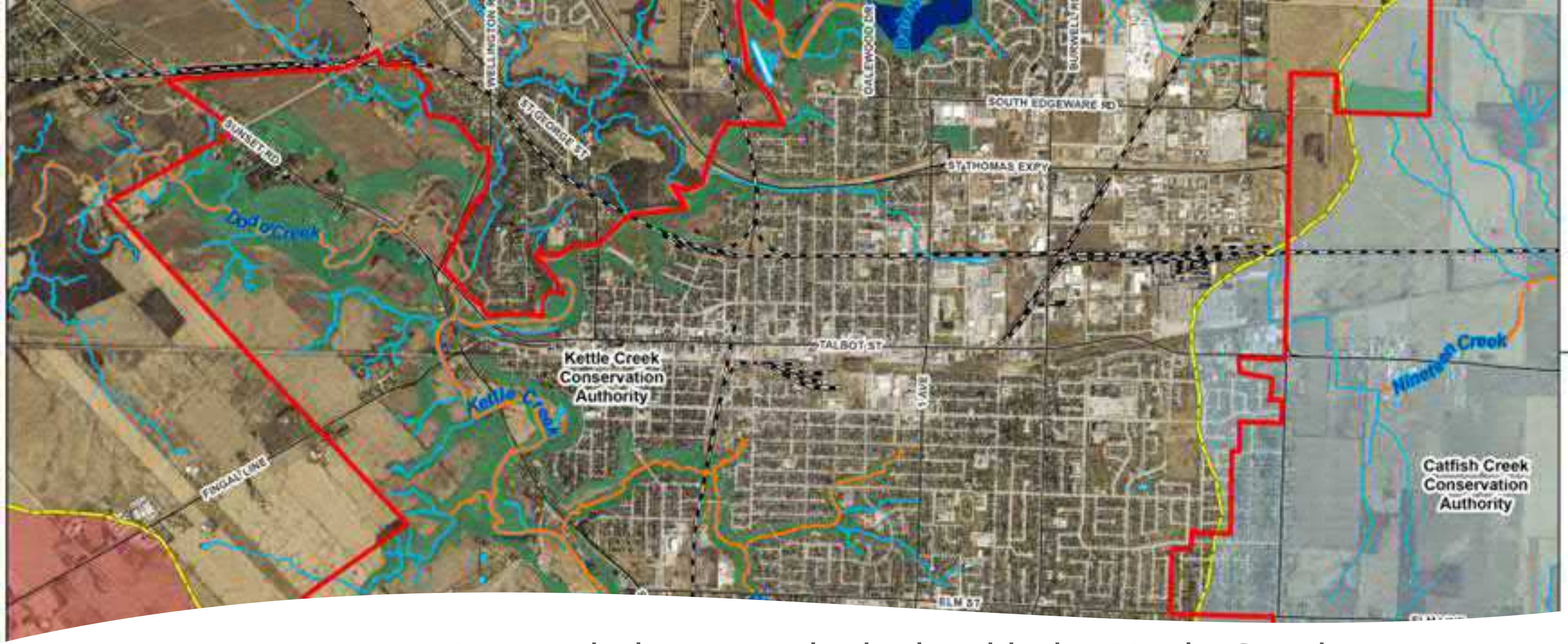
The PPCP is aimed at reducing sewer system overflows (SSO's) and bypasses of pumping stations and the pollution control plant during extreme weather events.

The PPCP will act as a master planning level tool that provides St. Thomas with guidance for capital planning and project implementation for the next 20 years and beyond.



Key Components to Prepare the PPCP

1. Review of natural water features within the City of St. Thomas and the impact on these features as a result of the existing infrastructure deficiencies.
2. Inventory and review of the current state of the collection system.
3. Asses the ability of the collection system to convey normal and wet weather flows.
4. Assess the quantity and quality of system overflows and by-passes.
5. Provide recommendations for short-term remedial measures and further investigations for a long-term PPCP.



Existing Water Features and Water Quality

Located almost exclusively with the Kettle Creek watershed with a small area within the Catfish Watershed boundary.

Poor water quality due primarily to high nutrient levels including phosphorus and nitrate.

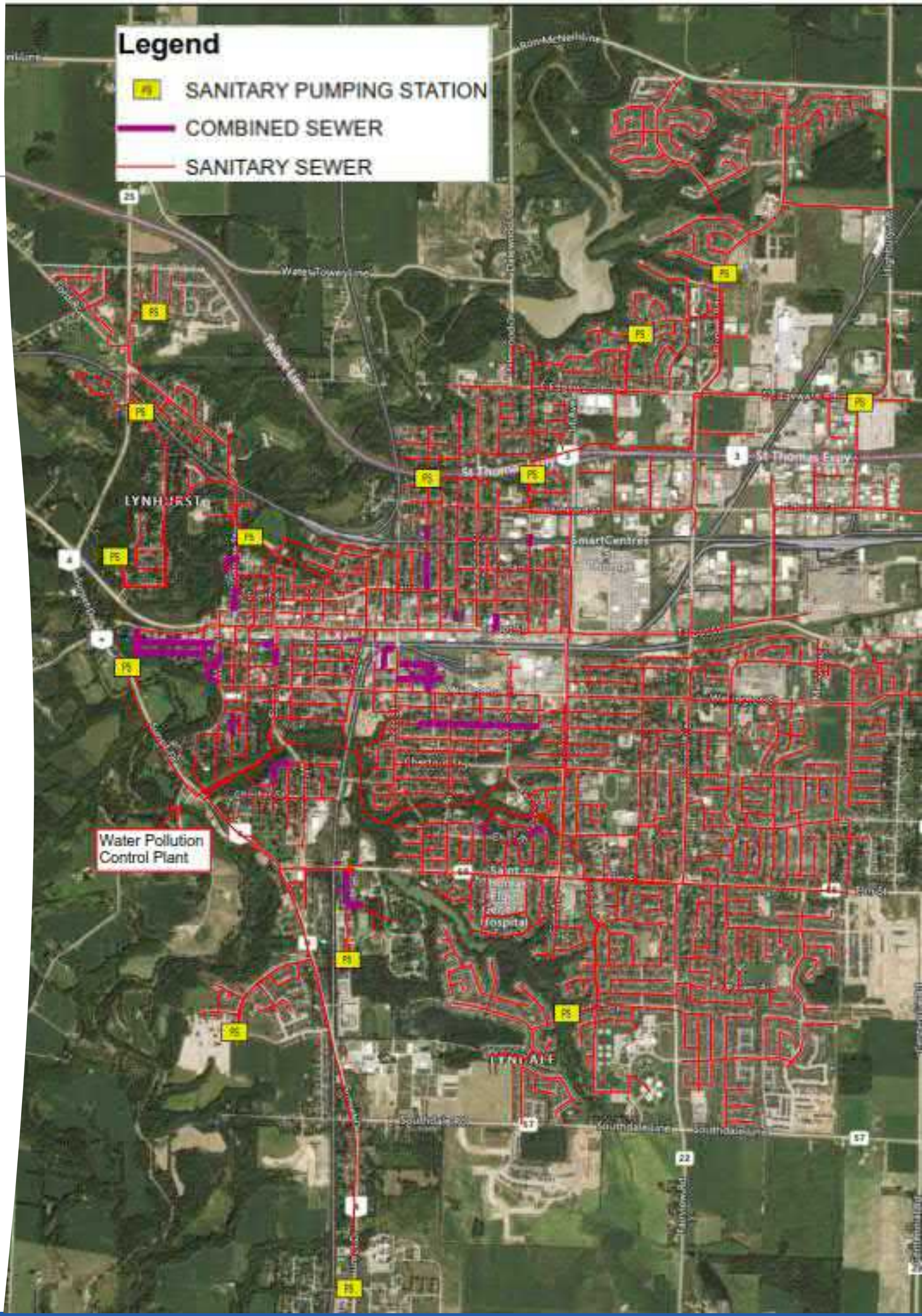
Other factors contributing to poor water quality include growing urban centres, increasing temperatures, decreasing baseflows and low levels of dissolved oxygen.

Existing Sewer Infrastructure

City of St. Thomas, covers a land area of approximately 35.5 km^2 and has a population of 43,276.

Sanitary Collection and Treatment System consists of:

- 220 km of sanitary sewers.
- 6 km of combined sewers (combining storm and sanitary sewage mostly in downtown).
- 16 Sanitary Pumping Stations.
- 4000 m^3 Combined Sewer Overflow Facility (upstream of plant).
- Water Pollution Control Plant (at Sunset and Bush Line) with a rated treatment capacity of 316 litres per second and a peak flow capacity of 632 litres per second.



Combined Sewers

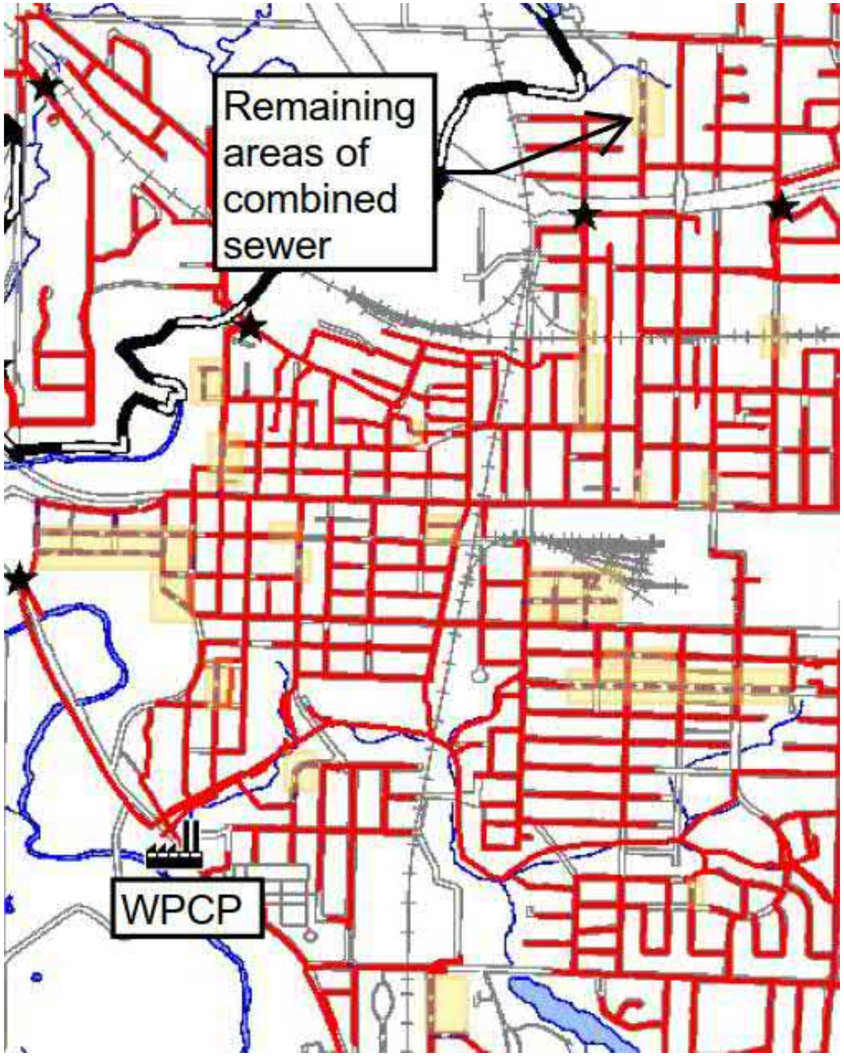
Combined sewers collect rainwater runoff, domestic sewage, and industrial wastewater in the same pipe.

These are in the some of the older sections of the City.

Currently, the City has approximately 6 km of combined sewers in its inventory.

The City's 10 Year Capital Plan – 2020 to 2029 indicates that 72% of these sewers will be separated in the next 10 years.

The current Capital Plan should minimize their impact.



Combined Sewer Overflow (CSO) Facility

A CSO facility was built in the early 2000's to store high sanitary sewer flows to minimize overflows at the WPCP.

Located on north side of Sunset Dr.

Flows in excess of the WPCP capacity overflow to drain, which is connected to Kettle Creek.

	2015	2016	2017	2018	2019
No. of Overflow Events	6	12	13	22	24
Annual overflow volumes as % of flow treated at WPCP	0.6%	2.1%	2.1%	6.0%	6.5%

Pumping Station & WPCP Facility Overflows

Pumping stations pump collected sanitary flows to the WPCP or to other sewers which flow to the WPCP. On occasion they experience flows in excess of their capacity which are directed to a local water course.

	2015	2016	2017	2018	2019
No. of Overflow Events	4	5	8	18	12
PS overflow volume as % of CSO overflow volume	0.4%	0.7%	13.3%	1.9%	0.6%

On occasion, the WPCP cannot treat all the flows that are directed to it and some of these are directed to overflow sewers to Kettle Creek prior to full treatment.

	2015	2016	2017	2018	2019
No. of Overflow Events	6	12	13	22	24
Overflow volume as % of WPCP volume	0.00%	0.01%	0.00%	0.23%	0.50%

Basement Flooding

Basement flooding complaints were reviewed for period from 2015 to 2019.

Most basement flooding was due to weeping tile connection to sanitary sewer, drain blockage, root growth of a tree, poor grading, or other similar reasons.

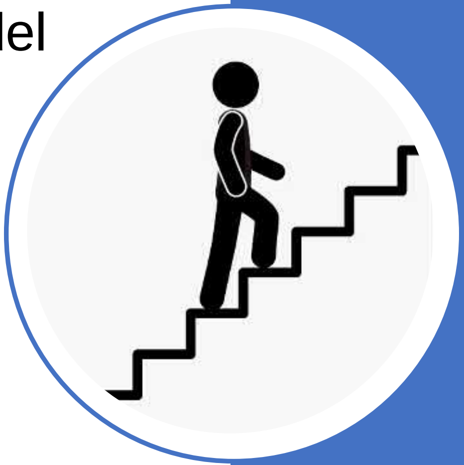
Remoteness of the affected residences from the overflowed pumping stations indicates connection between the two is unlikely.

Aldborough/Leger and Woodworth Sanitary Catchments were studied in response to flooding in 2014.

More study is needed as part of PPCP.

Next Steps

- Review and consider feedback and data received from the public and agencies.
- Collect more flow data and prepare a computer model of the City’s sanitary sewer system.
- Identify alternatives solutions to address needs.
- Present findings at PIC No. 2 – Spring 2021.
- Review input and data from agencies and public.
- Finalize PPCP – Summer 2021.



Questions?

- Please feel free to “raise your hand” to ask a question or submit your comments via email or phone to a member of the study team.
- This presentation, transcript and question and answer summary will be posted on **www.stthomas.ca/P_P_C_P**

Nathan Bokma, P. Eng.
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John Tyrrell, M.Sc. (Eng.), P. Eng.
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R.V. Anderson Associates Limited
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London, ON N6E 1A2

APPENDIX 1-5

PIC 2 December 1, 2021

ST. THOMAS
THE RAILWAY CITY

Pollution Prevention Control Plan

Virtual Public Consultation Meeting
December 1, 2021

arva | R.V. Anderson Associates Limited
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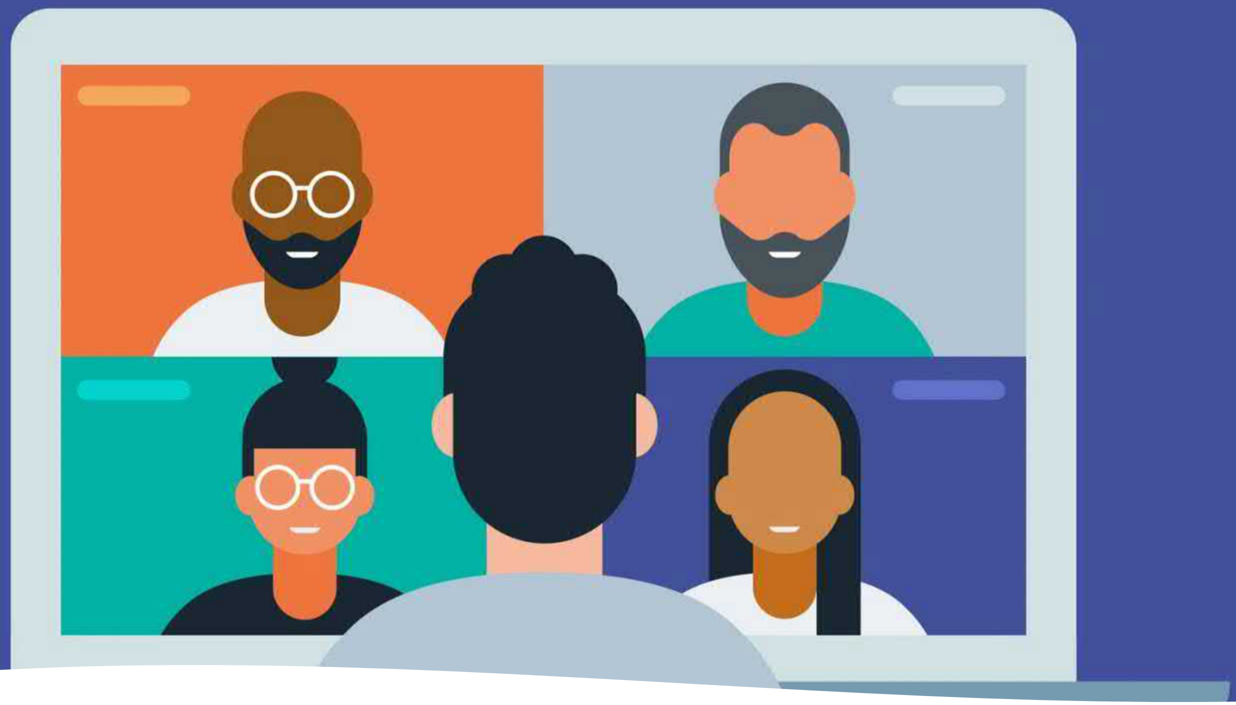
Platinum member



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- Question and Answer Period “Raise Your Hand” or Dial “9”.
- Presentation, Transcript and Question and Answer Summary will be available at **www.stthomas.ca/P_P_C_P** after the meeting.
- Please provide your comments by December 10, 2021.

Purpose of Meeting

- Update you on the study.
- Provide an overview of the study process.
- Highlight key findings.
- Report on key recommendations for future studies and projects to address noted issues.

We want to hear from you!

- Do you have any observations that you would like to share?
- Do you have any questions regarding the study?
- Do you have any questions regarding the Master Plan process?

Municipal Class Environmental Assessment

This study is being undertaken in accordance with the Municipal Class Environmental Assessment process for Master Plans.

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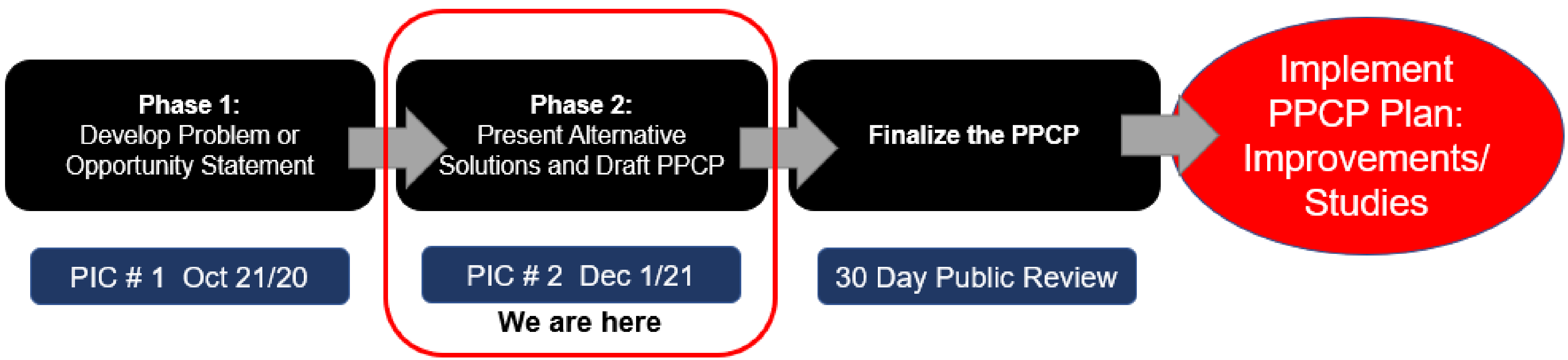

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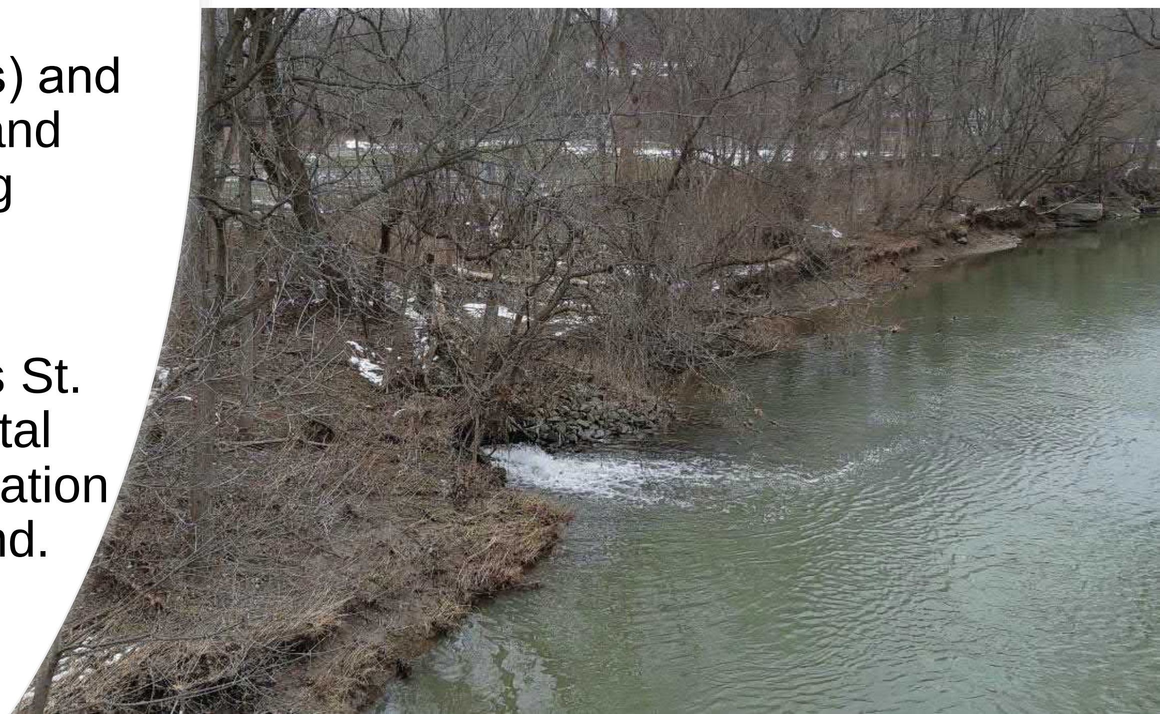

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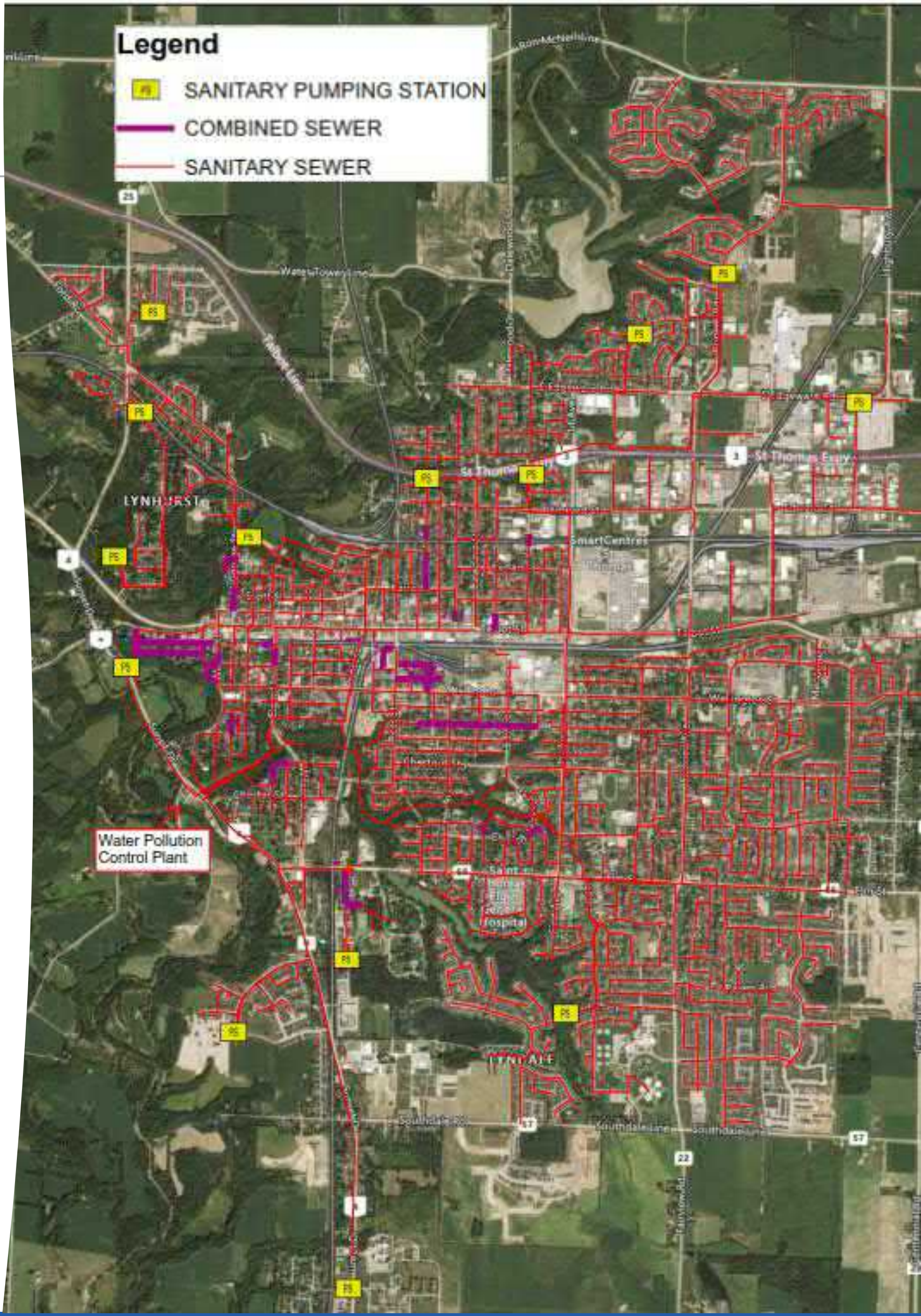

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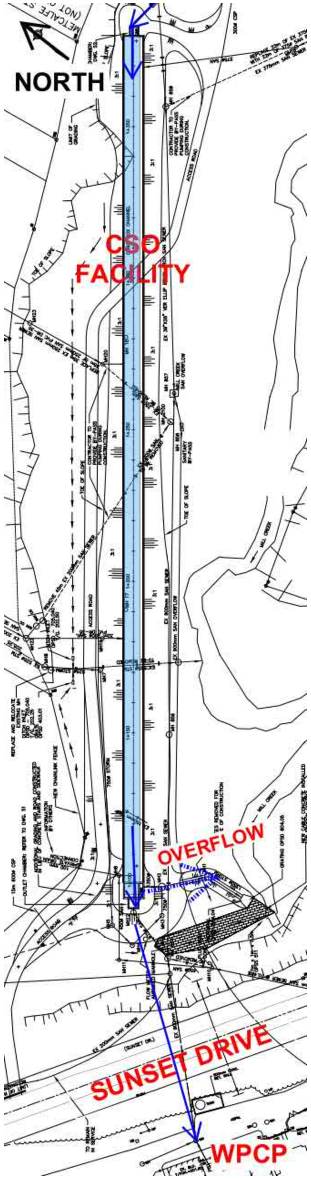
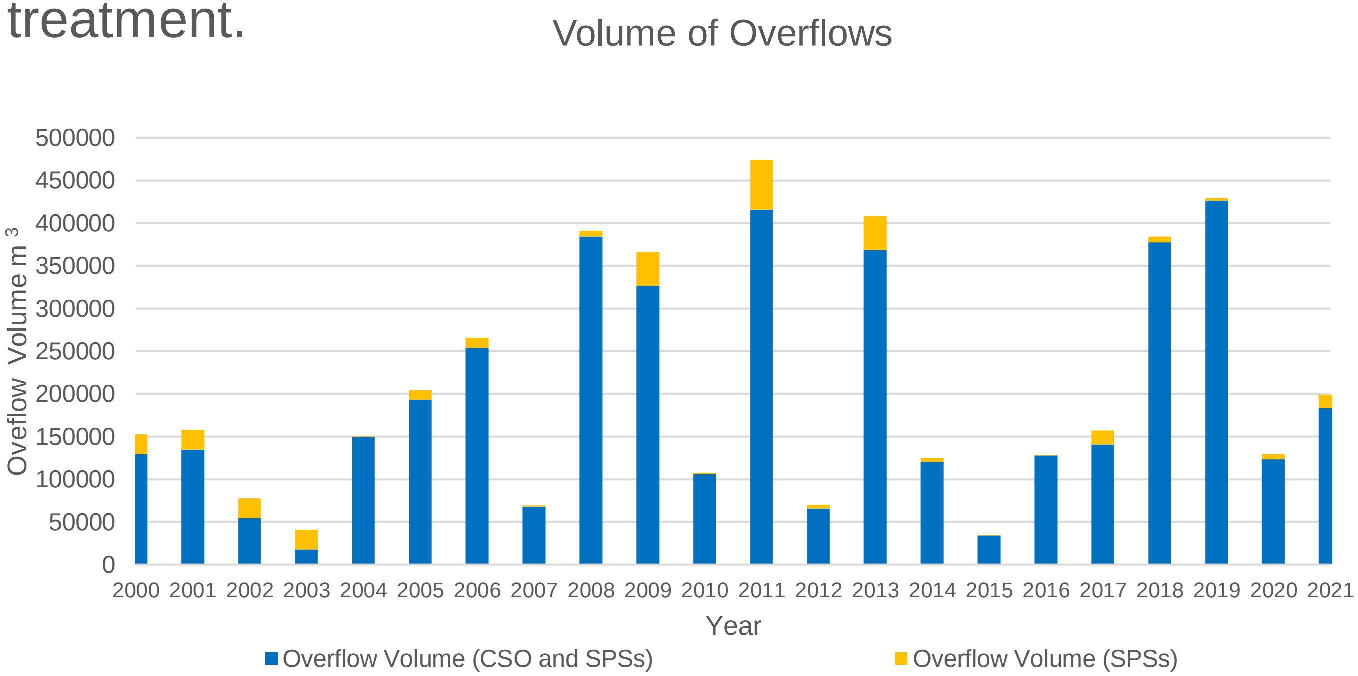


SPS, CSO, & WPCP Overflows

A Sanitary pumping station (SPS) pumps collected sanitary flows to the WPCP or to other sewers which flow to the WPCP. On occasion they experience flows in excess of their capacity which are directed to a local water course.

On occasion, the WPCP cannot treat all the flows that are directed to it and some of these are directed to overflow sewers to Kettle Creek prior to full treatment.



Key Components to Prepare the PPCP

#	Component	Activity
1	Review of natural water features within the City of St. Thomas and the impact on these features as a result of the existing infrastructure deficiencies.	• Background Review – Reported in PIC 1
2	Inventory and review of the current state of the collection system.	• Background Review – Reported in PIC 1
3	Assess the ability of the collection system to convey normal and wet weather flows.	• Review of Previous Studies • Additional Flow Monitoring (October 2020 to February 2021) • Review of CSO Operation • Develop Hydraulic Model of the Collection System
4	Assess the quantity and quality of system overflows and by-passes.	• Use model to review and confirm impacts on the collection system and CSO facility • Use model to review potential solutions
5	Provide recommendations for short-term remedial measures and further investigations for a long-term PPCP.	• Develop recommended list of solutions including infrastructure improvements, operational changes and ongoing study

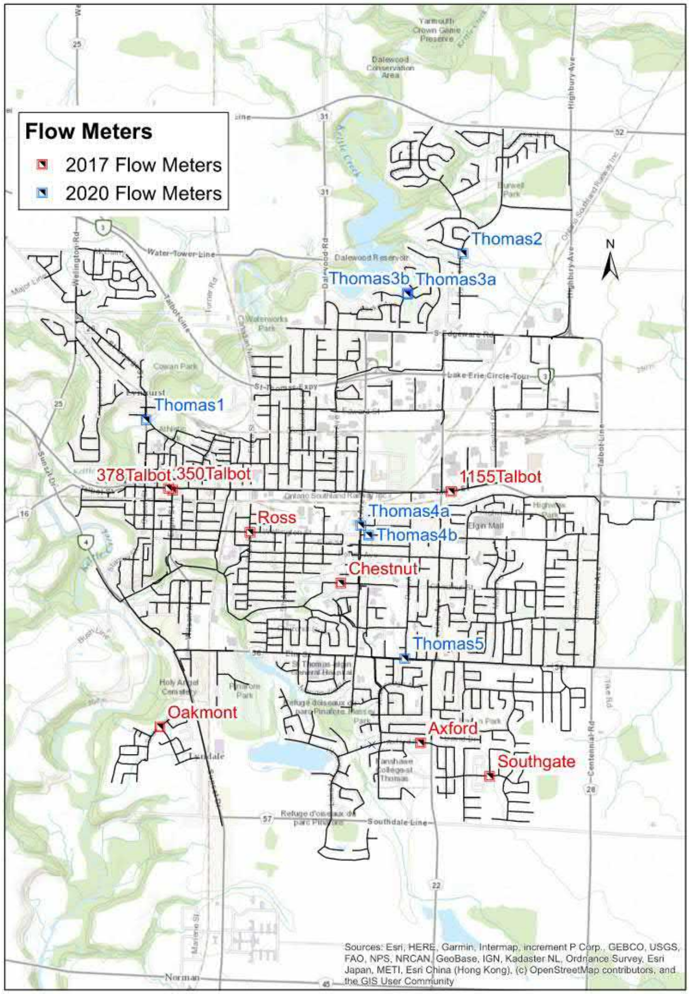
Flow Modeling

The following information was collected to build the hydraulic model of the City's collection system:

- GIS infrastructure data;
- Population and parcel data;
- Rainfall data; and
- Flow monitoring data.

The model reviewed both dry weather and wet weather conditions to determine the impact of groundwater and rainfall induced groundwater flow.

Wet weather analysis consisted of looking at more extreme weather events 1:2 year to 1:100 year.



Analysis

Wet weather analysis consisted of running rainfall events from 1:2-year to 1:100-year.

The impact on the sewage pumping stations and the CSO Facility were reviewed as well as the flows directed to the WPCP.

The level of water in the system was reviewed to confirm the potential for basement flooding.

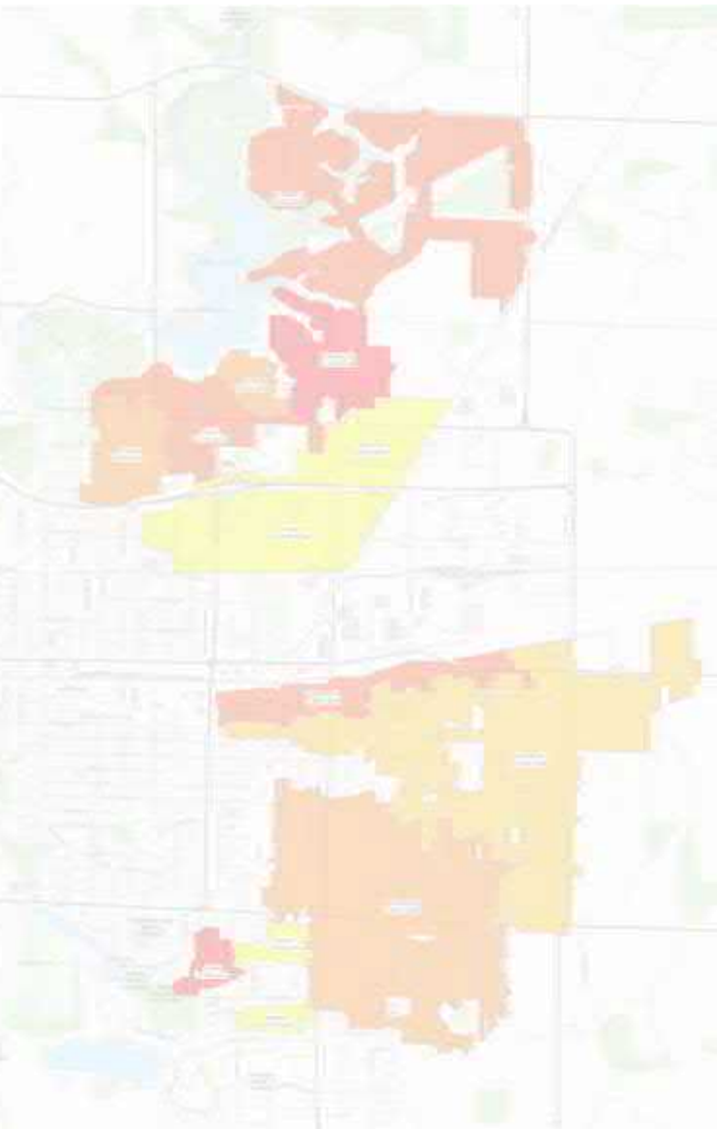
Potential overflow volumes were reviewed based on the existing system.

Overflow volumes were reviewed based on potential system improvements.

Reviewed existing data to validate required improvements.

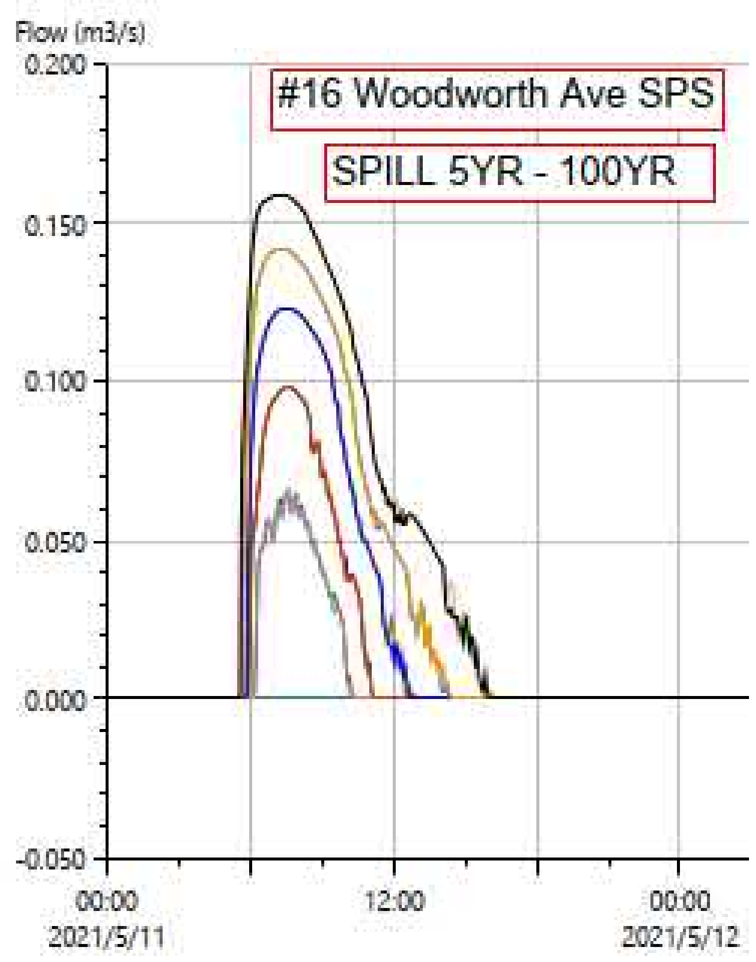
Major Conclusions

1. The peak day flow of the collection system is 4 to 5 times that of the average day flow which indicates high inflow and infiltration in the collection system.
2. The CSO facility is the major location of overflows as it is responsible for 97% of the overflow volumes.
3. 50-70% of the overflows at the CSO facility occur at flows lower than the WPCP's capacity as there are bottlenecks at the WPCP that restrict it passing its allowable peak flow.
4. Overflows have been observed only at 5 SPSs with most of the events (over 80%) occurring at the Sunset and Woodworth SPSs.



Major Conclusions

5. Sunset SPS is being reconstructed to allow for growth and this will address overflows at this station.
6. Overflows at the Woodworth Ave SPS are potentially caused by high inflow and infiltration in its sewershed and/or inadequate pumping capacity.
7. Demand growth may require the capacity expansion of the Burwell Road SPS.
8. More study may help to better characterize the inflow and infiltration issues in the collection system to confirm the long-term overflow reduction strategy.



PPCP Recommendations

1. Upgrade capacity of Sunset SPS and Collection System as part of current expansion project.
2. Undertake Improvements to the Woodworth Ave SPS and/or Collection System to address overflow issues.
3. Undertake Improvements to the Burwell Ave SPS and Collection System to address potential overflow issues.
4. Undertake an Annual Sanitary Sewer Lining Program to reduce inflow.
5. Removal of Capacity Constraints at the WPCP to allow it to pass permitted wet weather flows.
6. Optimize the CSO to control flows to the allowable limit of the WPCP.
7. Long Term I&I Mitigation Measures
    1. Installation of a Permanent Rain Gauge at the WPCP to better understand local conditions;
    2. Undertake annual flow monitoring program; and
    3. Annual Updating of the Hydraulic Model prepared for the PPCP.

PPCP Costs

The estimated costs for this proposed program are:

Recommended Projects	Planning Level Cost
Sunset SPS in addition to current expansion costs	$145,000
Woodworth Ave SPS and Collection System Upgrades	$7,427,000
Burwell Rd SPS Upgrades and Collection System Upgrade	$4,125,000
Remove Bottlenecks at the WPCP	$3,000,000
Permanent Rain Gauge Installation at WPCP	$19,000
Sewer Lining (500 m/year)	$715,000 per year
Camera Inspection of Collection System	$50,000 per year
Placement of Additional Flow Monitors	$35,200 per year
Annual Update of Collection System Model	$15,800 per year

The 10-year cost of this program is $22,876,000 (+50%/-30%).

Next Steps

- Review and consider feedback and data received from the public and agencies.
- Finalize PPCP Report.
- Present findings of PPCP to City Council.
- File PPCP with MECP.
- City to implement recommendations at their discretion.

Questions?

- Please feel free to “raise your hand” to ask a question or submit your comments via email or phone to a member of the study team.
- This presentation, transcript and question and answer summary will be posted on **www.stthomas.ca/P_P_C_P**

Nathan Bokma, P. Eng.
Manager of Development and Compliance
Environmental Services Dept.
City of St. Thomas
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Senior Project Manager/
Regional Manager
R.V. Anderson Associates Limited
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London, ON N6E 1A2

ST. THOMAS
THE RAILWAY CITY

rva

APPENDIX 2

Technical Memorandum # 1
Characterization of the Existing Wastewater System

City of St. Thomas Pollution Prevention Control Plan Study

Technical Memorandum # 1
Characterization of the
Existing Wastewater System

Final

City of St. Thomas

RVA 205153

July 31, 2020

St. Thomas Pollution Preventive Control Plan Characterization of the Existing Wastewater System

TABLE OF CONTENTS

LIST OF TABLES

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1.0 INTRODUCTION AND OBJECTIVES

The City of St. Thomas (The City) has retained RV Anderson Associates Limited (RVA) for the preparation of a Pollution Prevention Control Plan (PPCP). The PPCP is a part of the City’s ongoing efforts to improve the performance of their sanitary and storm sewer infrastructure. In addition, it would provide the City with a road map for implementation of infrastructure and operational improvements that will mitigate the impacts of wet weather sewer system overflows on the environment and help the City mitigate risk in alignment with the City's commitment to environmental stewardship and the provision of sustainable municipal services.

Pursuant to the above, this technical memorandum (Tech Memo) has been prepared with the objective of characterization of the existing wastewater system by reviewing the background information.

The key components covered for characterizing the existing system include:

1. The current state of the collection system with regards to influence of extraneous flows via inflow and infiltration (I&I):
2. The ability of the collection system to convey normal and wet weather flows;
3. Quantity and quality of system overflows and by-passes;
4. The natural environment;
5. The impact of existing system deficiencies on the natural environment; and
6. Recommendations for short-term remedial measures and further investigations for a long-term PPCP.

2.0 OVERVIEW OF THE EXISTING WASTEWATER SYSTEM

2.1 Collection System

Currently, the City has approximately 6 km of combined sewers in its inventory and the *10 Year Capital Plan – 2020 to 2029* shows that approximately 72% of these sewers will be separated in the next 10 years. This leaves 9 sections of combined sewer with a total length of approximately 1.7 km unseparated. In addition, the collection system has 16 sewage pumping stations. Table 2.1 gives a summary of each of the pumping stations with regards to its age, equipment details (make, model, and capacity), and operational configuration. See Figure 2.1 for City's sewerage system map.

Table 2.1 – St. Thomas Pumping Stations Data

Pumping Station	Construction Date	Make and Model	Duty/ Standby	Firm capacity (L/s)	TDH (m)
Axford	1997	Gorman-Rupp ECM	1/1	56.6	8.9
Burwell Rd	1993	ITT Flygt 3170.180	1/1	44	30
Confederation Dr	1968	Smith & Loveless	1/1	67	NA
Crescent Ave.	1988	Hydromatic Pentair	1/1	16	9.54
Elm St.	2018	Flygt 3153	1/1	44.35	13.1
Harper Rd	1973	Gorman-Rupp	1/1	21	9.1
Karen St.	2011	Flygt 3153	1/1	43.2	NA
Lynhurst	1996	Flygt 3102	1/1	23	NA
Parkside Dr.	1970	Flygt CP3127	1/1	NA	NA
Shaw valley	2005	Flygt 3153	1/1	62.7	17
St. George St.	1966	Gorman-Rupp	1/1	94.6	37.2
Sunset	1973	Barnes	1/1	23	8.5
Talbot Line	2014	Xylem NP-3153	1/1	25	34
Hughes St.	1993	ITT Flygt 3127	1/1	19.7	NA
Woodland	1988	Hydromatic Pentair	1/1	7	33.8
Woodworth Ave.	1972	Smart Turner Hayward	2/1	101	13.7

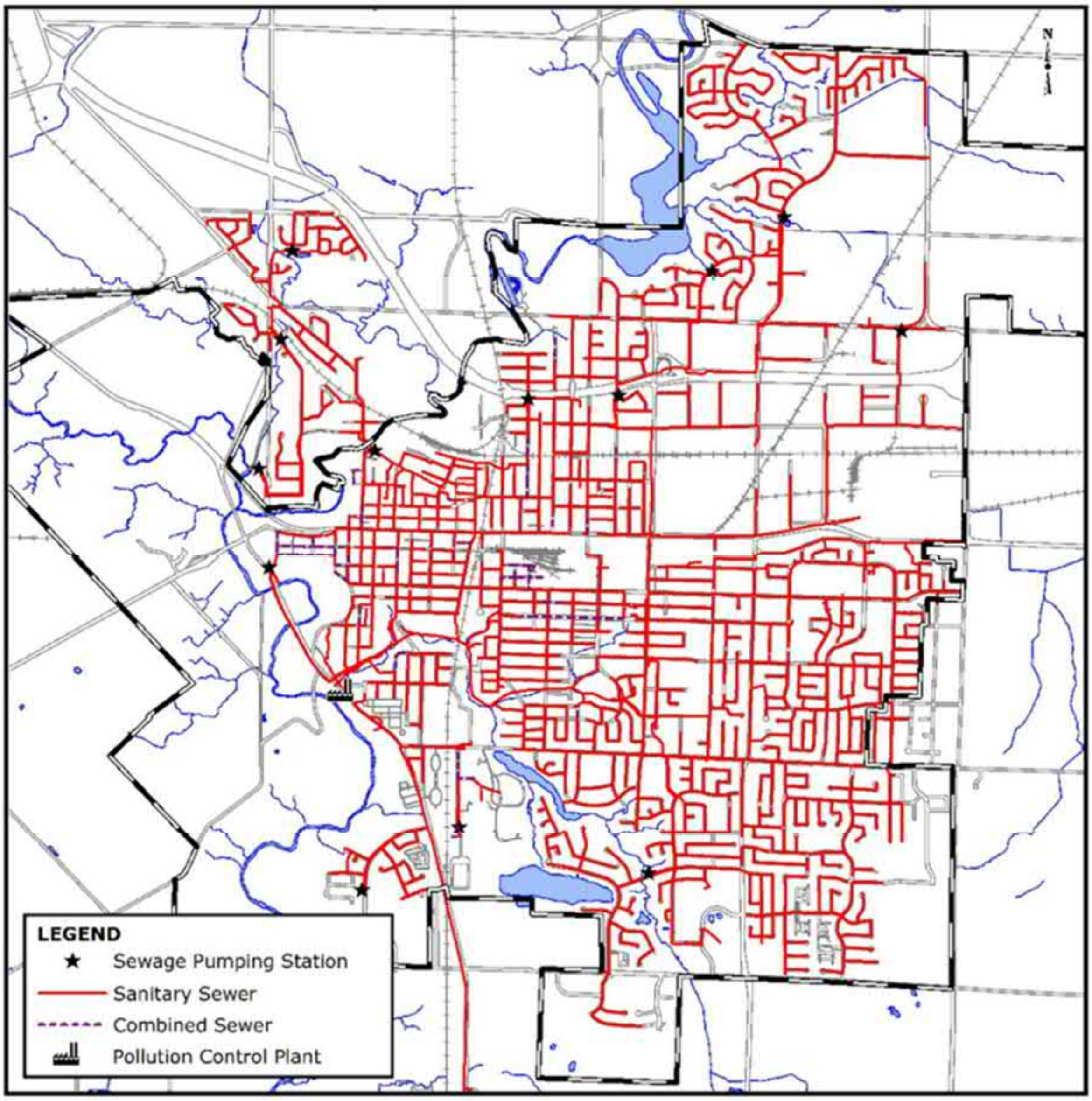


Figure 2.1 – St. Thomas Sewerage System Map

2.2 CSO Facility

A combined sewer overflow (CSO) facility was constructed in 2001 to mitigate wet weather peaks experienced at the WWTP and reduce overflows in the collection system. The facility is located north east of Sunset Drive and Bush Line in the Mill Creek Valley, immediately upstream of the WPCP on the main sewer leading from the City's sewershed. The inline CSO facility is 290 m long with a storage capacity of 4,000 m^3 and includes inlet, outlet, and overflow control structures. The storage channel comprises a cast-in-place V-channel base with a side slope of 1.5 horizontal to 1 vertical to minimize the accumulation of solids. See Figures 2.2 and 2.3 for design concept and operational details of the CSO facility.

The purpose of this tank is to control and mitigate peak flows to the WPCP, biological process upsets and prevent plant overflow events. The design allows the normal dry weather flow to pass unimpeded at a velocity that is adequate to maintain self-cleansing conditions. In the event of an overflow, the discharge enters Mill Creek upstream of the WPCP.

Based on discussion with Plant Operations, the actuated gates to the outlet of this CSO Tank are set to limit the peak flow to the WPCP at 500 L/s. This limit was selected as the plant's grit chamber overflows at flows exceeding 500 L/s, creating hazardous conditions and safety issues at the WPCP. As the instantaneous flow starts exceeding this limit, the actuated gates adjust the openings to limit the outflow to the set point. This makes the excess flow volume accumulate in the CSO leading to a rise in the liquid level in the same. In cases of sustained peak flows exceeding 500 L/s, the liquid level rises to the overflow elevation of the CSO causing it to overflow to Mill Creek through a bypass line.

Figure 2.2 – CSO Facility Design Concept

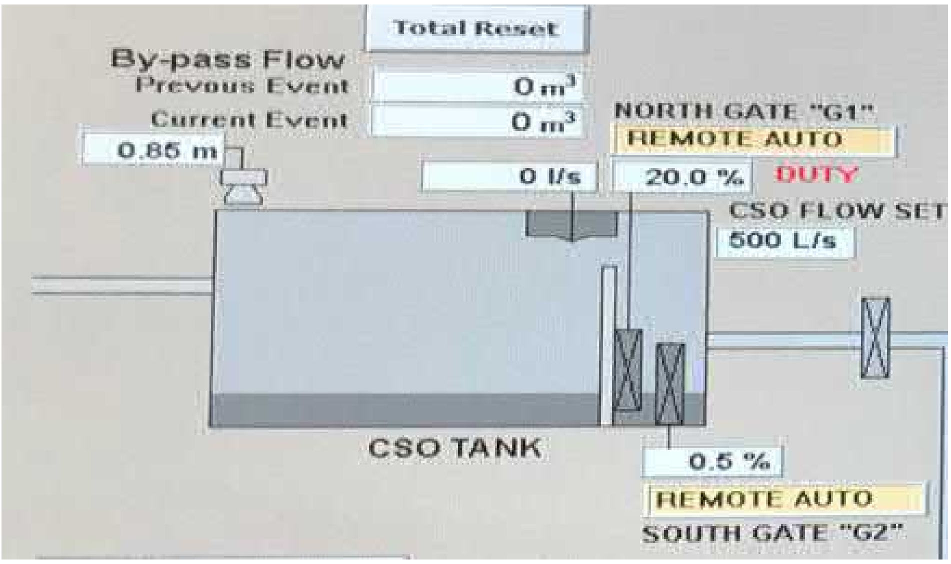


Figure 2.3 – CSO Facility Operation

2.3 St. Thomas WPCP

St. Thomas Water Pollution Control Plant (WPCP) is located at 40359 Bush Line in St Thomas. It has a rated capacity of 27,300 m^3/d (316 L/S) and peak flow capacity of 54,600 m^3/d (632 L/s). The WPCP services the City of St. Thomas and portions of the Municipalities of Southwold and Central Elgin. The plant is owned and operated by the City of St. Thomas.

St. Thomas WPCP is a conventional activated sludge facility with three (3) separate treatment trains (Plant 2, Plant 3, and Plant 4), each includes primary clarification, aeration, and secondary clarification processes. There is a common headworks facility and a common ultraviolet (UV) disinfection process. Effluent pumping is available during periods of high creek levels. Standby power is provided for the facility. Treated water is discharged from the St. Thomas WPCP to Kettle Creek, located to the South-West of the facility.

The original plant (Plant 1) was constructed in 1925 and is no longer in service. There have been several upgrades since that time. Plants 2 and 3 were constructed in 1960s, while Plant 4 was completed in two phases between 1980 to 2003.

3.0 NATURAL ENVIRONMENT

3.1 Natural Environment Review

As part of the Master Plan Municipal Class Environmental Assessment (EA) for the St. Thomas Pollution Prevention Control Plan (PPCP), a Natural Environment Review (NER) was prepared to identify and characterize the significance and sensitivity of the natural water features in the study area. This NER was prepared through a desktop review of available federal and provincial databases and is intended to provide a general framework for future water/wastewater pollution control projects. This NER technical memorandum documents the methodology and results of the preliminary background review of the existing conditions of the natural environment in the Study Area focused on water features.

3.2 Study Area and Methodology

The City of St. Thomas, located in Elgin County, covers a land area of approximately 35.5 km^2. For the purposes of the EA, the Study Area included in this NER includes the whole of the city limits.

3.2.1 Methodology – Data Collection

The following sources were reviewed for information related to natural water features and components, associated policy, and physiology within the Study Area:

- Natural Heritage Information Centre (NHIC) Make A Map Application;
- Land Information Ontario (LIO) Mapping – Aquatic Resource Areas (ARA);
- Fisheries and Oceans Canada (DFO) Aquatic SAR Mapping;
- Ontario Nature Reptile and Amphibian Atlas (ORAA);
- Ministry of Agriculture, Food and Rural Affairs (MAFRA) – AgMaps;
- Kettle Creek Conservation Authority (KCCA) Watershed Report Cards;
- Catfish Creek Conservation Authority (CCCA) Watershed Report Cards;
- Elgin County Natural Heritage Systems Study (2019);
- Geology Ontario;
- Physiography of Southern Ontario; Ontario Geological Survey – Chapman and Putnam (1984); and
- Kettle Creek Watershed Characterization Report (V. 2.0, January 2008).

3.2.2 Methodology – Field Investigations

Fieldwork was not a component of this existing natural environment characterization. Prior to any future works, a site-specific field investigation program should be planned and implemented, subject to the extent of work proposed, through discussions with the City of St. Thomas and relevant agency staff.

3.3 Environmental Planning and Policy Review

The following planning and policy documents are applicable to the natural aquatic environment in the Study Area.

3.3.1 Provincial Policy Statement (2020)

The wise use and management of the natural environment is understood to be vital for Ontario's long-term prosperity, environmental health, and social well-being. Accordingly, Section 2.1 (Natural Heritage) of the Provincial Policy Statement (PPS) provides direction for the long-term protection, rehabilitation, and improvement of the diversity and connectivity of natural features and the ecological function and biodiversity of natural systems. In the PPS, natural heritage features include significant wetlands, significant woodlands, significant valley-lands, significant wildlife habitat, significant areas of natural and scientific interest, and coastal wetlands. Additionally, Section 2.2 (Water) of the PPS describes the requirement to protect, improve and restore the quality and quantity of water at a watershed scale.

3.3.2 The Official Plan of the City of St. Thomas (2018)

Schedule “E” of the St. Thomas Official Plan shows Natural Heritage areas, primarily associated with the watercourses. It also identifies the designated Open Space and Conservation areas, which are smaller, disjunct, and generally located within the natural heritage system.

3.3.3 Species at Risk Act and Endangered Species Act

These are federal and provincial legislations which protect Species at Risk (SAR) and their habitats. There are currently no aquatic SAR identified within the Study Area, and therefore the direction in these acts is not applicable at this time. Prior to any future works, updated SAR information should be sought from the MECP to confirm potential impacts, permitting and approval requirements.

3.3.4 Conservation Authority Act

Under the Conservation Authority Act, the Kettle Creek and Catfish Creek Conservation Authorities are responsible for conservation, restoration, development, and management of natural resources in their respective watersheds. They must approve the development or site alteration within hazardous areas adjacent to shorelines, watercourses, and wetlands, as detailed in the Ontario Regulation 157/06: Regulation of Development, Interference with Wetlands and Alterations to Shorelines and Watercourses. The regulation limits of these two conservation authorities are found in the Study Area.

3.4 Existing Conditions

An overview of the natural environment features, conservation authority boundaries and regulation limit, found in the Study Area is presented in Map 1 – Appendix A.

3.4.1 Physiography

The City of St. Thomas is situated in three physiographic regions. The majority of the Study Area is in the Ekfrid Clay Plain. The St. Thomas Moraine enters into the City boundary from the east and west but does not connect through the Study Area. Lastly, a small area of the Norfolk Sand Plain enters the Study Area from the south (Chapman & Putnam, 1984).

3.4.2 Watersheds

The City of St. Thomas is located almost exclusively within the Kettle Creek watershed with a small part of the collection area situated within the Catfish Creek watershed boundary.

3.4.2.1 Kettle Creek Watershed

The Kettle Creek watershed drains approximately 520 km^2 of land from the southern end of London, through to Port Stanley. Kettle Creek originates at Lake Whittaker, a kettle lake, in the northeastern portion of the watershed. The upper portion of Kettle Creek flows southwesterly to the City of St. Thomas where it is joined by a major tributary, Dodd Creek. Kettle Creek then flows predominately southward towards Lake Erie at Port Stanley. There is a significant drop in elevation as Kettle Creek approaches Lake Erie, approximately 1.75 m per kilometer (141 m total). This significant decrease in elevation can result in flash flooding which in turn leads to intense erosion along the banks of Kettle Creek. The Kettle Creek watershed has been reported as having the most rapidly eroding shoreline in the Great Lakes basin. The overall erosion rate in the watershed is compounded by the fact that 83 percent of the watershed lands are in agricultural use. The watershed is a relatively small in area, the population of the entire watershed was 44,406 in 2001 and is now reported to have an approximate watershed population of 70,000. The City of St. Thomas is the largest population centre within the Kettle Creek watershed. Kettle Creek provides habitat for communities of aquatic organisms, recreational opportunities as well as livestock watering and agricultural irrigation. However, the tributaries of the Kettle Creek watershed are primarily used for waste assimilation from industrial and/or sewage treatment plant discharge and as habitat for aquatic life.

3.4.2.2 Catfish Creek Watershed

Catfish Creek watershed includes Catfish Creek and its tributaries, which drains approximately 490 km^2 in Elgin and Oxford counties and enters Lake Erie at Port Bruce. A small area of the City of St. Thomas (1,662 people within the area) are within the Catfish Creek watershed boundary.

3.4.3 Surface Water Features and Aquatic Species

3.4.3.1 Kettle Creek

Kettle Creek is described above in Section 4.2.1. The watercourse provides warm water habitats for a diverse fish community including catfish, darters, carps and minnows, bass, gar, sunfish, suckers, pike, chub, and perch.

3.4.3.2 Dodd Creek

Dodd Creek has a drainage area of approximately 130 km^2, making it Kettle Creek's largest tributary. It flows from the headwaters in the northwest corner of the watershed, easterly into the City of St. Thomas where it converges with Kettle Creek. Dodd Creek flows primarily through agricultural lands over the relatively flat clay plain which results in high runoff, little groundwater recharge and little continuous baseflow.

Dodd Creek is a very murky, warm water stream with midsummer temperatures ranging from 22 to 27 °C. Deep water pools in Dodd Creek do not exceed 1.2 m, and substrates range from gravel to muck. Historically, there are sections of the creek that do not flow in July, August, and September, except during major precipitation events.

Despite its minimal baseflow and warm temperatures, Dodd Creek provides habitat for a diverse fish community. Recorded species include catfish, darters, carps and minnows, bass, gar, sunfish, suckers, pike, chub, and perch.

3.4.3.3 Lake Margaret

Lake Margaret is a retired gravel pit that is filled with natural groundwater and provides recharge to Mill Creek, a tributary to Kettle Creek. Since the surrounding soils are predominately gravel, the lake contains waters that are clearer than most other systems in the Study Area, which are typically murky due to a high clay content.

The lake provides habitat for a warm water fishery which includes a significant bass population. According to the Kettle Creek Watershed Characterization Report (2008), past monitoring showed that benthic invertebrates in the lake consisted primarily of aquatic worms, which indicates low oxygen conditions. This may be a result of groundwater influence or due to the biomass in the lake consuming oxygen.

3.4.3.4 Pinafore Creek, Lake and Wetland

Pinafore Creek is a clear, warm water stream with a depth ranging from 0.05 m to 0.5 m, and substrates ranging from gravel to clay and muck. Fish species recorded in the creek consisted of darters, minnows, suckers, rock bass, and chub.

Pinafore Lake is located south of Elm Street in the City of St. Thomas and is associated with a historic nature park. At the southern end of the lake is a small wetland that covers approximately 2 ha. The wetland is predominately swamp in nature with some marsh and open lake areas.

3.4.3.5 Dalewood Reservoir and Wetlands

Dalewood Reservoir was constructed as a water source for the City of St. Thomas. It has since been taken over by KCCA and is managed as a flood control structure and Provincially Significant Wetlands (PSWs). Since the 1980s it has been documented that due to intensive upstream agriculture and tile drainage, the reservoir had begun rapidly filling with silt. Over a 25-year period, the reservoir surface area reduced by almost 30%, going from 51 ha to 35 ha. The increase in sediment reduced the water quality in the reservoir but it also created a growth of wetlands surrounding the Dalewood Reservoir. It appeared that the Dalewood Reservoir had reached equilibrium in the mid-2000s and became a net source of sediment to downstream portions of the watershed. The wetlands surrounding the reservoir contain diverse vegetation communities and support a wide variety of fish species, and the reservoir itself acts a summer refuge. Wetlands, in general, are protected by the Conservation Authority. Provincially, wetlands are ranked to determine those areas identified by the province as being the most valuable and should receive special protection as “provincially significant”. Significance is determined by the Ontario Wetland Evaluation System (OWES). The Dalewood Wetlands (also known as the Kettle Creek Woods) consists of twelve individual wetlands, altogether protected as a PSW.

3.4.4 Water Quality

3.4.4.1 Kettle Creek Watershed Quality

According to the 2018 KCCA watershed Report Card, surface water quality within the Kettle Creek watershed was reported as ‘D’ grade, or poor. The low grade is due primarily to phosphorus concentrations that regularly exceed (97% of all samples) the

Provincial Water Quality Objective (PWQO) of 0.02mg/L as well as poor benthic invertebrate Family Biotic Index results. E. coli concentrations throughout the watershed were found to be fair, or C grade.

A 2006 report prepared by KCCA and Grand River Conservation Authority (GRCA) summarized the water quality conditions within the Kettle Creek watershed from 1991-1995. The purpose of the report was to identify key water quality issues within the watershed. Like the 2018 findings, nutrient levels, primarily phosphorus and nitrate, were high throughout the watershed. Nitrate was found to be significantly higher in the Lower Kettle Creek than the rest of the watershed. Phosphorus concentrations were also found to be highest in Lower Kettle Creek but were also found to be consistently high throughout the watershed and exceeded the PWQO.

Kettle Creek's water quality directly affects the water quality of Lake Erie and is a potential point source of contamination to the Elgin Area drinking water supply. Raw water for the Elgin Area Primary Water Supply System is taken from Lake Erie into which Kettle Creek drains. Studies have found that littoral drift within the lake carries sediment from the mouth of Kettle Creek to the intake pipe.

In general, surface water quality within the watershed has been reported as being negatively affected by increasing summer temperatures, decreasing baseflows, and potentially low levels of DO and extensive nutrient and sediment loading. According to the Watershed Characterization Report (2008), most of the tributaries within the Kettle Creek watershed are thermally stressed – this had become a primary water quality concern. High water temperatures can impact dissolved oxygen saturations and can limit the diversity of aquatic species present.

As of 2018, Ontario beaches are to follow recreational water guidelines and protocols where samples at beaches are taken weekly during the summer months. Water samples were found to fail guidelines 10% of the time in 2018 and 13% of the time in 2019.

The pressures on the watershed, with growing urban centres, increasing temperatures, decreasing base flows, low levels of dissolved oxygen, and excessive nutrient and sediment concentrations could lead to increasing negative impacts on the water quality within the watershed if management measures are not implemented.

3.4.4.2 Catfish Creek Watershed Quality

The upper main branch of Catfish Creek is reported as being the area where water quality is the most impaired, with improvement as the creek flows downstream. The Nineteen Creek sub-watershed, which incudes the small eastern area of the Study Area, was reported as 'C' grade, or fair. The grade, similar to Kettle Creek, was a result of nutrient levels (phosphorus), intrinsic geology and topography as all being factors affecting water quality within the watershed.

3.5 Discussion

This NER was prepared through a desktop review of available sources, intended to provide a general framework for future water/wastewater pollution control projects. In support of this NER, information requests have been made to the KCCA, CCCA, and MECP to provide any additional or updated information related to the existing aquatic habitat features or water quality concerns in or as may be affected by the Study Area. If any additional information is received it will be incorporated into this document and appended as correspondence.

4.0 CURRENT OVERFLOW ISSUES

4.1 CSO Facility Overflows

All overflow events between years 2015 to 2019 were reviewed and analyzed. The overflow events are classified as CSO facility overflows and Pumping Station (PS) overflows. The following sections provide description and review of these overflows over the last 5 years.

4.1.1 2015 Overflow Events

Table 4.1 and Figure 4.1 give details of the overflow events at the CSO facility in 2015. There were 5 overflow events from CSO facility due to rainfall and snow melt. One of the 5 overflow events happened continuously over two days and therefore has been shown as two events on consecutive days, which makes it a total of 6 days of the overflow events in 2015. As indicated, these 6 events during the year resulted in a total combined sewage overflow volume of 34,131 m^3 to the creek during the year. In all 6 events, the total day flow inclusive of the by-passes was less than the Plant's Peak Day Flow Capacity of 54,000 m^3/d. A potential reason for such by-passes is prolonged hourly peaks exceeding of the 500 L/s limit despite overall day flow being less than 500 L/s.

Table 4.1 – CSO Facility Overflow Details - 2015

Date	Plant flow (m^3/d)	Overflow volume (m^3)	Total flow m^3
31-May	33598	2531	36129
14-Jun	24574	2196	26770
27-Jun	25999	3895	29894
28-Jun	39159	4817	43976
28-Oct	32398	12831	35550
29-Dec	36029	7861	41767
	Total	**34,131**	

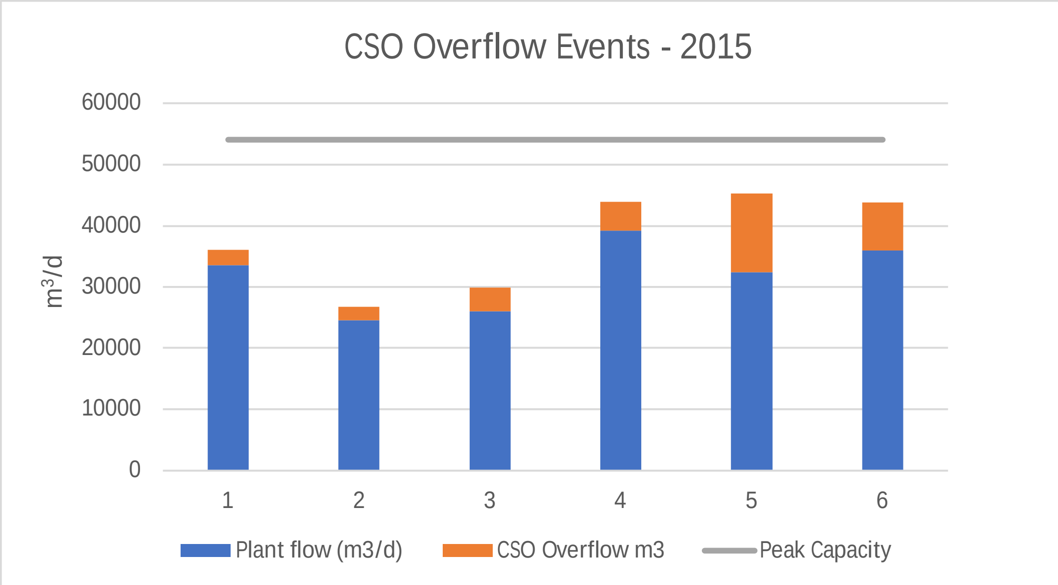


Figure 4.1 – CSO Facility Overflow Events - 2015

4.1.2 2016 Overflow Events

Table 4.2 and Figure 4.2 give details of the overflow events at the CSO facility in 2016. There were 9 overflow events from CSO facility due to rainfall and snow melt. Some of the overflow events happened continuously over 2-3 days and therefore has been shown as separate events on consecutive days, which makes it a total of 12 days of the overflow events in 2016. These 12 events resulted in a total combined sewage overflow volume of 126,299 m^3 to the Creek during the year. In 11 out of the 12 events, the total day flow inclusive of the by-passes was less than the Plant's Peak Day Flow Capacity of 54,000 m^3/d, while exceeding the latter by 6,243 m^3/d during one event.

Table 4.2 – CSO Facility Overflow Details - 2016

Date	Plant flow (m^3/d)	Overflow volume (m^3)	Total flow m^3
24-Feb	28758	3102	31860
25-Feb	34290	1902	36192
28-Mar	38071	15926	53997
29-Mar	37658	267	37925
31-Mar	35671	17547	46682
01-Apr	41213	30362	60243
07-May	14242	11974	28603
11-May	12967	3462	16429
25-Jul	20449	522	21611

Date	Plant flow (m^3/d)	Overflow volume (m^3)	Total flow m^3
25-Aug	25909	2857	29178
28-Dec	25492	19265	44757
29-Dec	25825	19113	44938
	Total	**126,299**	

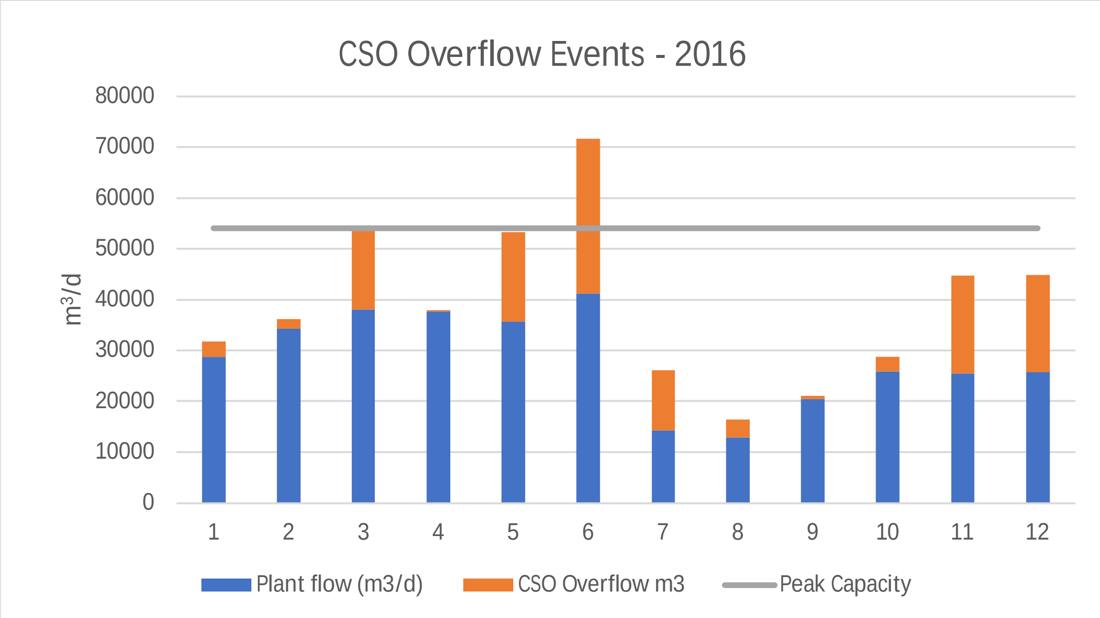


Figure 4.2 – CSO Facility Overflow Events - 2016

4.1.3 2017 Overflow Events

Table 4.3 and Figure 4.3 give details of the overflow events at the CSO facility in 2017. There were 7 overflow events from CSO facility due to rainfall and snow melt. Some of the overflow events happened continuously over 2-3 days and therefore has been shown as separate events on consecutive days, which makes it a total of 13 days of the overflow events in 2017. These 13 events resulted in a total combined sewage overflow volume of 124,044 m^3 to the Creek during the year. In 10 out of the 13 events, the total day flow inclusive of the by-passes was less than the Plant's Peak Day Flow Capacity of 54,000 m^3/d, while exceeding the latter during 3 events.

Table 4.3 – CSO Facility Overflow Details - 2017

Date	Plant flow (m^3/d)	Overflow volume (m^3)	Total flow m^3
10-Jan	17108	3249	20357
07-Feb	23622	6677	30299
08-Feb	39195	7656	46851
01-Mar	39409	4537	43946
07-Mar	35169	1675	36852
30-Mar	29471	11651	41122
31-Mar	42242	27068	69310
01-Apr	38705	282	38987
04-May	24727	4992	29719
05-May	41683	30561	80083
06-May	40837	22602	72098
18-Nov	25541	1964	27505
19-Nov	33523	1130	34653
	Total	**124,044**	

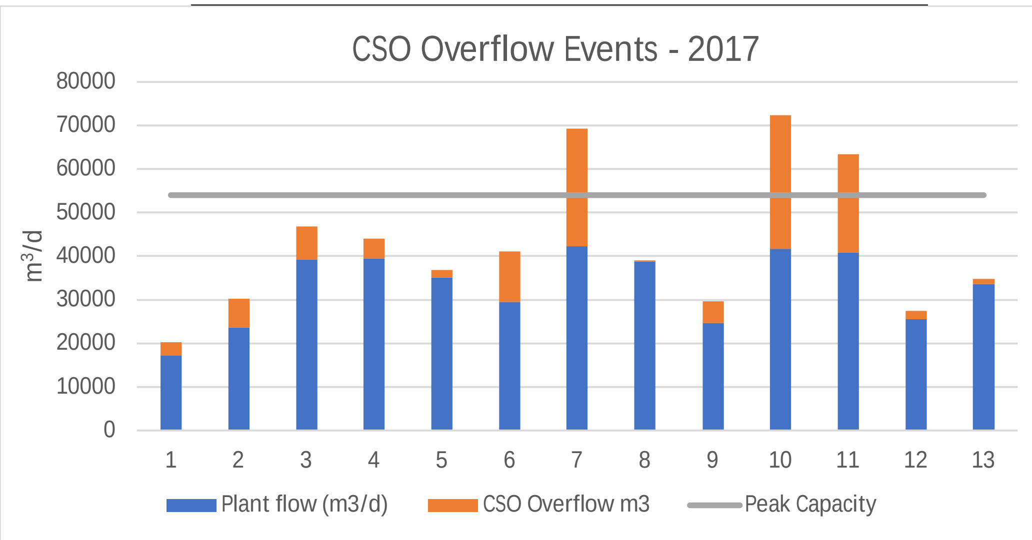


Figure 4.3 – CSO Facility Overflow Events - 2017

4.1.4 2018 Overflow Events

Table 4.4 and Figure 4.4 give details of the overflow events at the CSO facility in 2018. There were 13 overflow events from CSO facility due to rainfall and snow melt. Some of the overflow events happened continuously over 2-3 days and therefore has been shown as separate events on consecutive days, which makes it a total of 22 days of the overflow events in 2018. These 22 events resulted in a total combined sewage overflow volume of 355,385 m^3 to the Creek during the year. In 15 out of the 22 events, the total day flow inclusive of the by-passes was less than the Plant's Peak Day Flow Capacity of 54,000 m^3/d, while exceeding the latter during 7 events.

Table 4.4 – CSO Facility Overflow Details - 2018

Date	Plant flow (m^3/d)	Overflow volume (m^3)	Total flow m^3
11-Jan	35827	13913	49766
12-Jan	39322	40819	80216
13-Jan	34458	2432	36894
23-Jan	40695	10092	50787
24-Jan	28976	368	29344
19-Feb	24416	10945	35490
20-Feb	27646	49378	82015
21-Feb	28018	49378	77563
22-Feb	33252	49378	82630
23-Feb	36610	5987	42597
04-Apr	39156	488	39644
15-Apr	30096	10154	40250
16-Apr	38153	27852	66105
17-Apr	38427	25322	63849
24-Jul	25364	3320	28914
06-Aug	18504	2709	21597
08-Aug	32187	13488	46054
06-Oct	32833	965	33818
31-Oct	30927	861	30958
01-Nov	32205	14379	46602
02-Nov	40992	18832	59856
31-Dec	24650	4325	28975
	Total	**355,385**	

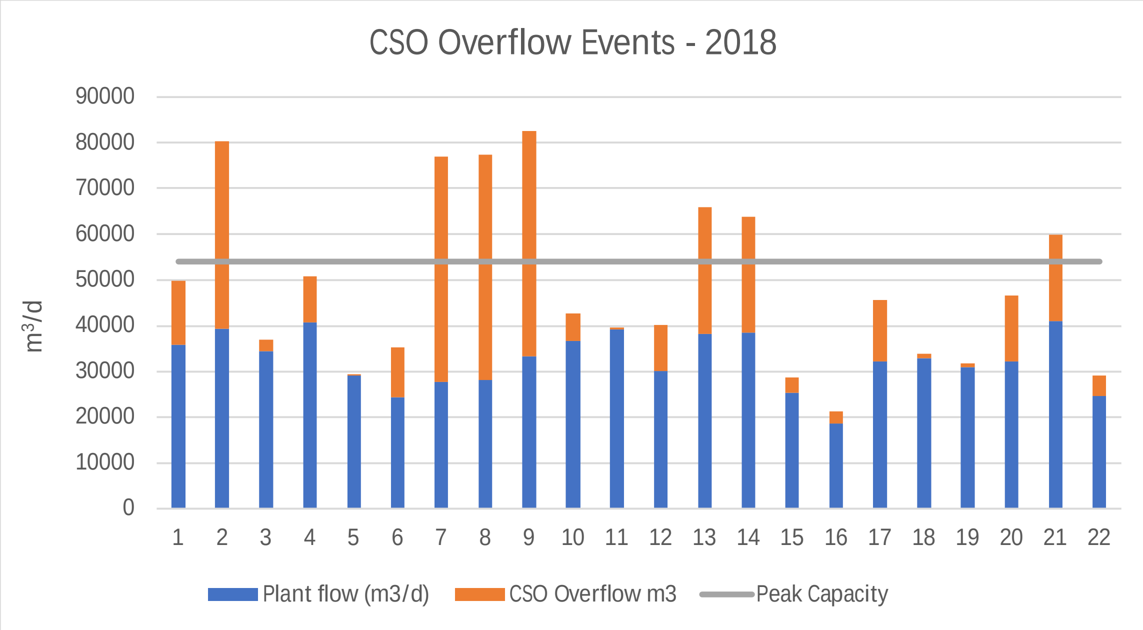


Figure 4.4 – CSO Facility Overflow Events - 2018

4.1.5 2019 Overflow Events

Table 4.5 and Figure 4.5 give details of the overflow events at the CSO facility in 2019. There were 15 overflow events from CSO facility due to rainfall and snow melt. Some of the overflow events happened continuously over 2-3 days and therefore has been shown as separate events on consecutive days, which makes it a total of 24 days of the overflow events in 2019. These 24 events resulted in a total combined sewage overflow volume of 388,373 m^3 to the Creek during the year. In 15 out of the 24 events, the total day flow inclusive of the by-passes was less than the Plant's Peak Day Flow Capacity of 54,000 m^3/d, while exceeding the latter during 9 events.

Table 4.5 – CSO Facility Overflow Details - 2019

Date	Plant flow (m^3/d)	Overflow volume (m^3)	Total flow m^3
23-Jan	32045	17677	49722
24-Jan	35763	4339	40102
04-Feb	33352	7762	41185
05-Feb	38133	42325	80458
06-Feb	37465	15741	53206
12-Feb	26176	8372	34548
24-Feb	36901	12099	49000
30-Mar	32277	17172	49449

Date	Plant flow (m^3/d)	Overflow volume (m^3)	Total flow m^3
31-Mar	40935	25861	66796
19-Apr	35335	16783	52118
20-Apr	38634	31542	70176
21-Apr	36661	19871	56532
01-May	39740	31755	71495
02-May	40712	21651	62363
03-May	41097	38512	79609
04-May	39663	942	40605
07-May	36372	8481	44853
10-May	40576	36359	76935
11-May	39018	38	39056
21-Aug	16898	1837	18735
27-Oct	36335	9057	45392
31-Oct	38776	17125	55901
01-Nov	35715	1966	37681
27-Nov	31534	1106	32640
	Total	**388,373**	

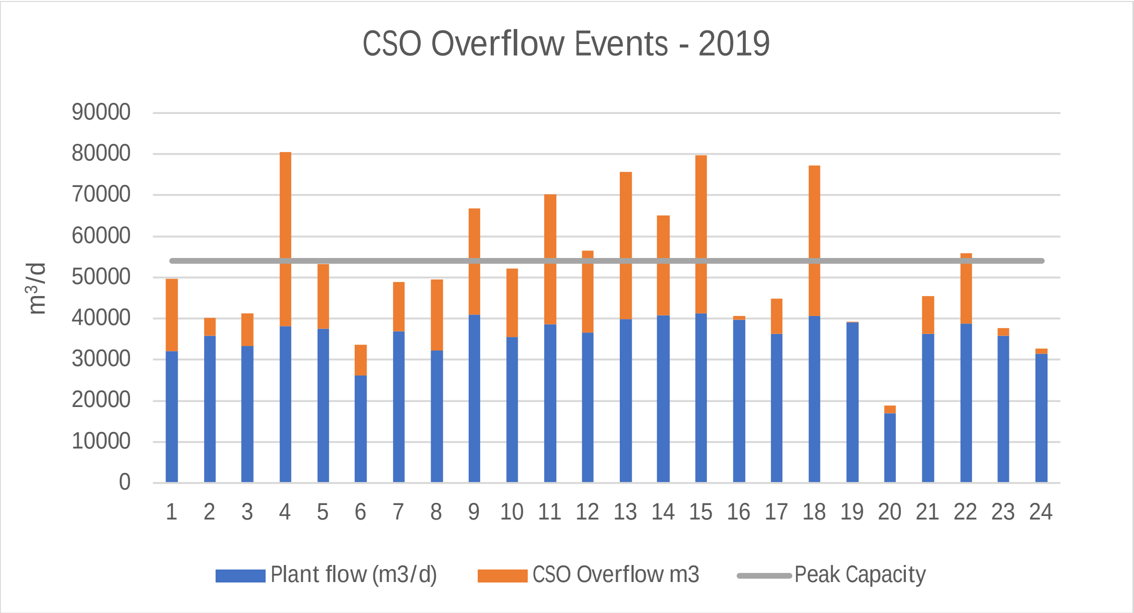


Figure 4.5 – CSO Facility Overflow Events - 2019

4.1.6 CSO Overflows Summary and Key Observations (2015 – 2019)

Table 4.6 summarizes the total overflow volumes, the overflow volumes exceeding the WPCP peak flow capacity, and the comparison of overflow volumes to the annual flow volumes treated at WPCP from 2015 to 2019.

Table 4.6 – CSO Tank Overflows Summary (2015 – 2019)

Plant Data	2015	2016	2017	2018	2019
No. of Overflow Events	6	12	13	22	24
Total amount of overflow (m^3/d)	34,131	126,299	124,044	355,385	388,373
Annual overflow volumes as % of flow treated at WPCP	0.6%	2.1%	2.1%	6.0%	6.5%
Overflow volume at flows under WPCP peak day capacity (m^3/d)	34,131	120,056	64,553	221,151	254,108
% Overflow volumes at flows under WPCP peak day capacity	100%	95%	52%	62%	65%

Given below are the key observations on the CSO facility overflows based on the historic data review.

- For the historic average flow value of approximately 16,000 m^3/d at the WPCP, the peak day flow (PDF) in the collection system (including treated flows at the WPCP and the overflows) can be as high as 80,000 m^3/d. This translates into a PDF factor of 5.0, which, in comparison to a typical PDF factor of 2.0 for a plant capacity of St Thomas WPCP (WEF guidelines), indicates excessive I&I issues in the collection system.

- The annual overflow volumes have shown an increasing trend over the last five years particularly 2017 and 2018 in which it was up to three times more than the previous two years. With no operational changes in the CSO facility during this period, this increase was likely caused by relatively higher precipitation during the 2018 and 2019.

- The total overflow volumes at the CSO facility during the last 5 years averaged at 3.5% of the treated flow volumes at the WPCP. While this average ranged between 1-2% from 2015 to 2017, it was 6% or above in 2018 and 2019.

- In 2015 and 2016 nearly all overflows occurred at peak day flows lower than the WPCP's peak day flow (PDF) capacity of 54,400 m^3/d or 632 L/s, which means almost 100% of the overflow volume during these years was contributed by flows lower than PDF capacity. While the overflow contribution by lower than PDF flows was relatively lower at 50-65% during the last three years, it is still very significant.

- The key reason for these overflows was the current operation of the CSO facility which restricts the maximum flow to the WPCP at 500 L/s due to hydraulic bottleneck at the plant. This restriction causes the CSO facility to fill up and overflow during longer (lasting more than 3-4 hours) wet weather events. As such there is a significant potential to mitigate these overflows by removing the bottlenecks at the plant and increasing the peak flow setting to WPCP's PDF capacity of 632 L/s.
- MECP's Procedure F-5-5 allows overflow volumes not exceeding 10% of the wet weather flow volumes during the seven-month period of concern in a combined sewer systems. In a partially separated sewer system like that in St. Thomas, Procedure F-5-5 applies only to the flows from the area served by the combined sewer systems. Given that a relatively small portion of the existing collection system has combined sewers, the allowable limits for the overflows would be significantly lower than 10% of the treated flows. As such it is highly likely that the current system is in non-compliance with F-5-5.

4.2 Pumping Stations Overflow Events

In addition to the overflow events at CSO facility, overflow events at the pumping systems were reviewed as well to identify potential causes and particularly to see any relation to the CSO facility overflow events. The sections below summarize discuss the overflow events at Pumping Stations (PS) from 2015 to 2019.

4.2.1 2015 Overflow Events

Table 4.7 gives a summary of PS overflow events in 2015. Woodworth PS had 2 overflow events, one due to rainfall and the other due to a mechanical issues. Oak St. Ravine PS had one overflow event over 2 consecutive days due to a mechanical issues. One of the four overflow events coincided with the CSO facility overflow, however the overflow volume less than 0.1% of that at the CSO facility.

Table 4.7 – Pumping Stations Overflow Details for 2015

Date	Plant flow (m^3/d)	PS Facility	Overflow due to rainfall (m^3)	Overflow due to mechanical or power Issues (m^3)	CSO overflow (m^3)
03-Jul	32398	Woodworth	-	33	
28-Oct	32398	Woodworth	7.1	-	12831
18-Nov	10951	Oak St Ravine	-	50	
19-Nov	13462	Oak St Ravine	-	50	
		Total	**7.1**	**133**	

4.2.2 2016 Overflow Events

Table 4.8 gives a summary of PS overflow events in 2016. Sunset PS had 4 overflow events, Woodworth PS had 3 and St. George PS 1 event. Out of total 8 overflow events, 5 coincided with the CSO facility overflows, with a combined overflow volume of 0.1% of that at the CSO facility.

Table 4.8 – Pumping Stations Overflow Details for 2016

Date	Plant flow (m^3/d)	PS Facility	Overflow due to rainfall (m^3)	Overflow due to mechanical or power Issues (m^3)	CSO overflow (m^3)
25-Jul	20449	Sunset	14.4	-	522
17-Aug	14610	Sunset	6.6	-	0
25-Aug	25909	Sunset	14	-	2857
31-Aug	20702	Sunset	15.5	-	0
31-Mar	35671	Woodworth	13.2	-	17547
25-Aug	25909	Woodworth	398	-	2857
26-Aug	14242	Woodworth	51	-	0
26-Dec	22944	St. George	430	-	19265
		Total	**943**	**0**	

4.2.3 2017 Overflow Events

Table 4.9 gives a summary of PS overflow events in 2017. Sunset PS had 2 overflow events, and Woodworth PS had 3 events. Out of the total 5 overflow events, 4 occurred due to wet weather and one due to mechanical issues. Also 3 out of the 4 wet-weather related overflows coincided with the CSO facility overflows. The combined overflow volume at the pumping stations was of 13% of that at the CSO facility. This relatively high percentage was mainly due to a single large overflow event contributing over 99% of the total overflow volume in 2017. The overflow event was caused by an extreme wet weather event with total day flow of over 80,000 m^3/d, that also caused a large overflow at the CSO facility as indicated in Table 4.9.

Table 4.9 – Pumping Stations Overflow Details for 2017

Date	Plant flow (m^3/d)	PS Facility	Overflow due to rainfall (m^3)	Overflow due to mechanical or power Issues (m^3)	CSO overflow (m^3)
07-Mar	35169	Sunset	7.9	-	1675

Date	Plant flow (m^3/d)	PS Facility	Overflow due to rainfall (m^3)	Overflow due to mechanical or power Issues (m^3)	CSO overflow (m^3)
23-Jun	24671	Sunset	12	-	-
05-May	41683	Woodworth	16498	-	30561
06-May	40837	Woodworth	0.3	-	22602
07-Nov	17690	Woodworth		11.7	
		Total	**16518**	**11.7**	

4.2.4 2018 Overflow Events

Table 4.10 gives a summary of PS overflow events in 2018. Sunset PS had 9 overflow events, Woodworth PS had 7 events, while Confederation PS and St George PS had 1 overflow event each. Out of the total 18 overflow events, 16 occurred due to wet weather and 2 due to mechanical issues. Also 12 out of the 16 wet-weather related overflows coincided with the CSO facility overflows. The combined overflow volume at the pumping stations was less than 2% of that at the CSO facility.

Table 4.10 – Pumping Stations Overflow Details for 2018

Date	Plant flow (m^3/d)	PS Facility	Overflow due to rainfall (m^3)	Overflow due to mechanical or power Issues (m^3)	CSO overflow (m^3)
20-Feb	27646	Sunset	861.8	-	49378
27-May	28018	Sunset	6.1	-	-
23-Jul	17882	Sunset	10.1	-	3320
30-Jul	11909	Sunset	0.5	-	-
06-Aug	18504	Sunset	27.5	-	2708
08-Aug	32187	Sunset	17.4	-	13488
27-Aug	22100	Sunset	13.5	-	-
03-Sep	14058	Sunset	12.4	-	-
06-Oct	32833	Sunset	20	-	965
11-Jan	24416	Woodworth	3.4	-	13913
20-Feb	27646	Woodworth	757	-	49378
24-Jul	25364	Woodworth	230.1	-	3320
06-Aug	18504	Woodworth	356.7	-	2708
08-Aug	32187	Woodworth	362.2	-	13488
12-Sep	12132	Woodworth		364	-
01-Nov	32205	Woodworth	44.7	-	14379
20-Feb	27646	St. George	3668	-	49378

Date	Plant flow (m^3/d)	PS Facility	Overflow due to rainfall (m^3)	Overflow due to mechanical or power Issues (m^3)	CSO overflow (m^3)
23-Mar	13955	Confederation		8.3	-
		Total	**6391**	**372**	

4.2.5 2019 Overflow Events

Table 4.11 gives a summary of PS overflow events in 2019. Sunset PS had 4 overflow events, Woodworth PS had 7 events, and Confederation had 1 overflow event. Out of the total 12 overflow events, 7 occurred due to wet weather and 5 due to mechanical issues. Also 4 out of the 7 wet-weather related overflows coincided with the CSO facility overflows. The combined overflow volume at the pumping stations was less than 0.2% of that at the CSO facility.

Table 4.11 – Pumping Stations Overflow Details for 2019

Date	Plant flow (m^3/d)	PS Facility	Overflow due to rainfall (m^3)	Overflow due to mechanical or power Issues (m^3)	CSO overflow (m^3)
06-Jul	18844	Sunset	7.3	-	-
04-Aug	16687	Sunset	22.9	-	-
21-Aug	16898	Sunset	11.4	-	1837
02-Oct	16769	Sunset	1.4	-	-
04-Feb	33352	Woodworth	70.8	-	7762
01-May	39740	Woodworth	45.1	-	31755
19-Jul	13219	Woodworth	-	9	-
21-Aug	16898	Woodworth	496	-	1837
01-Sep	11802	Woodworth	-	107	-
02-Sep	12961	Woodworth	-	792	-
11-Oct	11031	Woodworth	-	204	-
20-Dec	13911	Confederation	-	531	-
		Total	**655**	**1643**	

4.2.6 Pumping Station Overflows Summary and Key Observations (2015 – 2019)

Table 4.12 indicates the total overflow volumes at the pumping stations, and the comparison of these overflow volumes to those at the CSO facility from 2015 to 2019.

Table 4.12 – Pumping Station Overflows Summary (2015 – 2019)

Plant Data	2015	2016	2017	2018	2019
No. of Overflow Events	4	5	8	18	12
CSO overflow volume (m^3/d)	34,131	126,299	124,044	355,385	388,373
PS overflow volume (m^3/d)	140	943	16530	6763	2298
PS overflow volume as % of CSO overflow volume	0.4%	0.7%	13.3%	1.9%	0.6%

Given below are the key observations on the Pumping Station overflows based on the historic data review.

- The average annual overflow volume at the pumping stations is 2.6 % of that the CSO facility, and 0.1% of the flows treated at the WPCP.
- Out of 47 overflow events during the five-year period, Woodworth and Sunset St pumping stations accounted for 39 of them, while only 6 occurred at the other stations which means many pumping stations to not experience overflows. Out of the 39 events at the above two stations, 22 occurred at the Woodworth PS, and 17 at Sunset PS. See Figure 4.6 for collection system map and location of these stations.
- There were 17 overflow events at the Sunset PS over the last 5 years, which could indicated high I&I in its sewershed and inadequate pumping capacity could be potential contributors to the overflows.
- Out of the 22 events at the Woodworth PS over the last 5 years, 10 coincided with the overflow events at the CSO facility. However, given the remoteness of the Woodworth PS from the CSO facility, the latter is unlikely to have any relation to the overflows at the PS. As such the overflows at the Woodworth are likely caused by high I&I in its sewershed and/or inadequate pumping capacity.
- Out of the 6 overflow events at 3 pumping stations other than Sunset and Woodworth PS, 4 were caused by mechanical issues and only two due to wet weather. Further, there were no overflows reported at 11 out of the 16 pumping stations during this period.

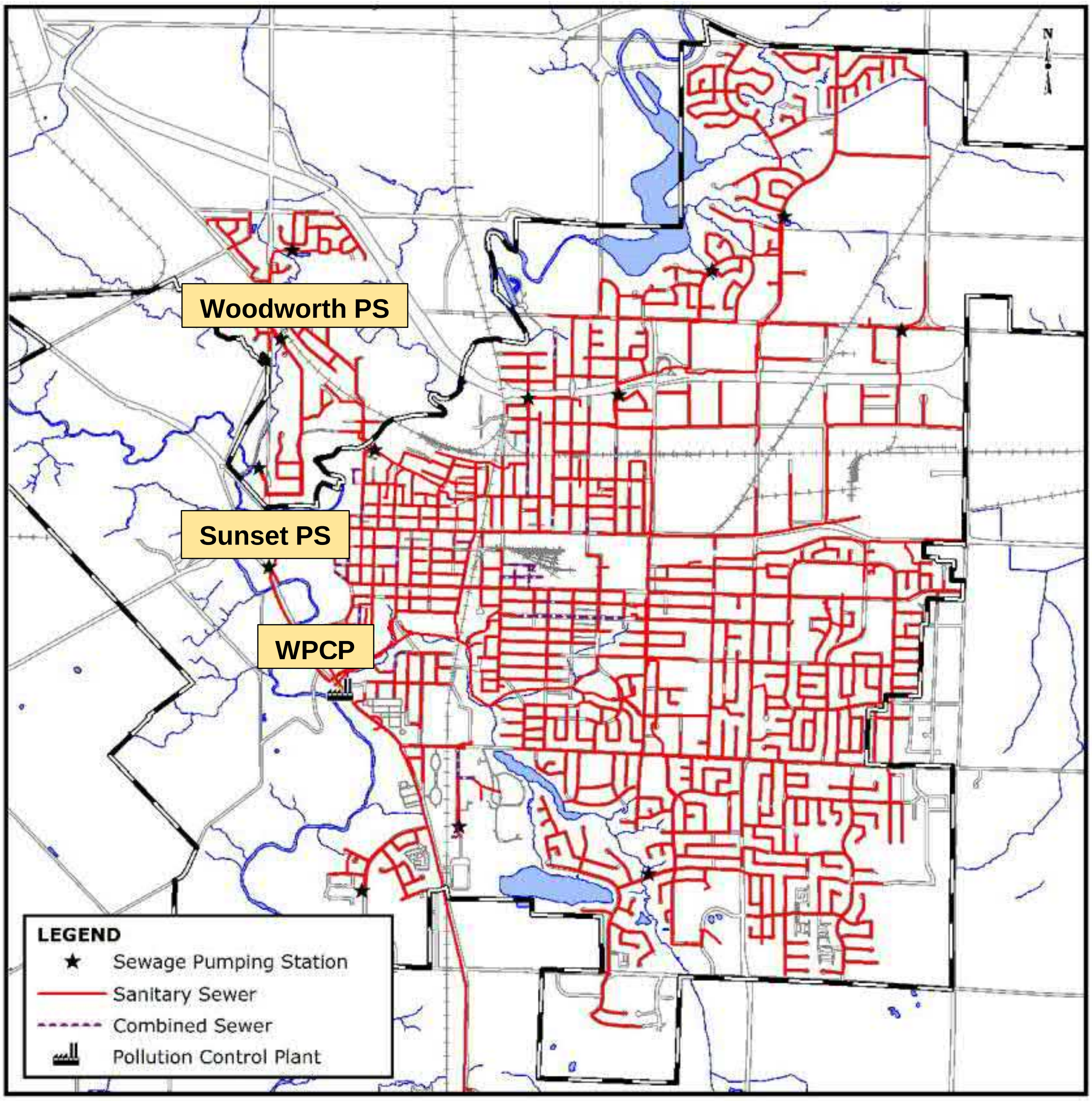


Figure 4.6 – Sunset and Woodworth PS Locations

4.2.7 Quality and Characteristics of Overflows

All overflows at the CSO facility are sampled and monitored for quality. The overflow characteristics data from 2015 to 2019 was reviewed regarding overall annual loading to the Creek and as percentage of annual effluent loadings at the WPCP. Table 4.12 and Figure 4.7 summarize this information.

Table 4.13 – CSO Facility Overflow Loadings

Contributor	Loading (kg/annum)		
	$cBOD_5$	TSS	TP
WPCP effluent[1]	30204	41701	3011
2015 overflows	1049	1485	35
2016 overflows	2274	3633	116
2017 overflows	3206	4803	135
2018 overflows	8974	13934	380
2019 overflows	12025	23156	1165

1. Historic annual effluent loading (2015 – 2019)

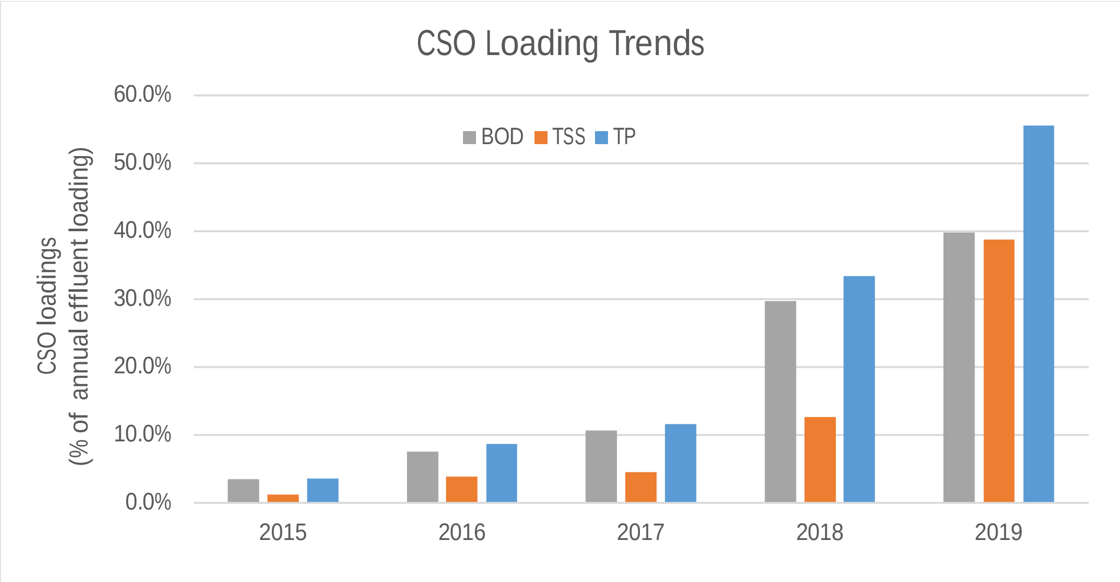


Figure 4.7 – CSO Facility Overflow Loadings (2015 – 2019)

Given below are the key observations on the quality of overflows.

- $cBOD_5$ loadings from the overflows ranged from 3% to as high as 40% over the last five years. While averaging between 3-10% from 2015 to 2017, the $cBOD_5$ loadings increased to 30-40% of the WPCP loadings in 2018 and 2019 due to significantly higher overflow volumes during these years.

- TSS loadings from the overflows ranged from 1% to as high as 40% of the WPCP effluent loadings over the last five years. While averaging between 1-5% from 2015 to 2017, the TSS loadings increased to 10% of the WPCP loadings in 2018, and 40% in 2019 due to significantly higher overflow volumes during these years.

- TP loadings showed similar trends to BOD and TSS. With an overall range of 3-55% of effluent loadings during this period, these increased from 3-10% range up to 2017, to 30-55% in the last two years due to higher overflow volumes.
- In addition, the E-Coli loading to the Creek by overflows were 30-300 times higher than the annual E-Coli loads by the WPCP effluent.

4.3 WPCP By-passes

Apart from the overflows at the CSO facility and the Pumping Stations, by-pass events have also been reported at the WPCP. However, all such events at the plant are due to mechanical issues and/or to power outage, which cause UV and/or blowers to go off-line for short intervals. None of these events are due to high flows as the wet weather peaks are shaved to the plant's hydraulic capacity by the CSO facility upstream of the plant. As such, while the partially treated effluent is still passing through the temporarily un-operational unit process during such events, it is technical considered a by-pass as the unit process is unable to provide treatment during that period. Table 4.12 shows the combined annual by-pass volumes from 2015 to 2019.

Table 4.14 – WPCP By-Passes

Year	2015	2016	2017	2018	2019
Overflow volume (m^3)	0	626	0	14,869	35,644

4.4 Basement Flooding

Basement flooding complaints report was reviewed for the period between years 2015 and 2019. The report is primarily a record of the residents' complaints without any details/comments on the potential causes of the basement flooding. However, from the report, no direct relationship could be found between the cause of the basement flooding and the rainfall / snowmelt events. Most of the basement flooding were reported due to drain blockage, root growth of a tree, poor grading, and others.

In complaints coinciding with the rainfall events and overflows at pumping stations, the remoteness of the affected residences from the overflowed pumping stations suggested an unlikely connection between the two. However, more data is required to state any actual linkage between basement flooding with the wet-weather overflow events.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the above collection system review, given below are the key conclusions related to the current overflow issues and the high I&I in the collection system.

- For the historic average flow value of approximately 16,000 m^3/d at the WPCP, the peak day flow (PDF) in the collection system (including treated flows at the WPCP and the overflows) can be as high as 80,000 m^3/d. This translates into a PDF factor of 5.0, which, in comparison to a typical PDF factor of 2.0 for a plant capacity of St Thomas WPCP (WEF guidelines), indicates excessive I&I issues in the collection system.

- The high wet weather flows cause significant overflow issues in the collection system with an annual average overflow volume of 3.5%, and a maximum of 6.5%, of the annual flow volumes treated at the WPCP. The historic average annual $cBOD_5$ and TSS loadings from these overflows to Mill Creek were approximately 20% of the WPCP effluent loadings, and as high as 40% in 2018 and 2019. Similarly, average annual TP loading by the overflows was 12% with a maximum of 55% in 2019. In addition to that, the high E-Coli loadings from the overflows make them a significant source of pollution to the Creek.

- MECP’s Procedure F-5-5 allows overflow volumes not exceeding 10% of the wet weather flow volumes during the seven-month period of concern in combined sewer systems. In a partially separated sewer system like that in St. Thomas, Procedure F-5-5 applies only to the flows from the area served by the combined sewer systems. Given that a relatively small portion of the existing collection system has combined sewers, the allowable limits for the overflows would be significantly lower than 10% of the treated flows. As such it is highly likely that the current system is in non-compliance with Procedure F-5-5.

- While the overflows in the collection system occur both at the CSO facility as well as the pumping stations, the CSO facility is the major source of overflows with over 97% of the overflow volume contributed by the same. Further, given the excessive peaks and I&I in the system, and low frequency and intensity of overflows in the collection system compared to the CSO facility, indicates that the sewers, pumping stations and forcemains are sized adequately to handle the current high peaks for most part, with potential minor exceptions.

- Out of the 16 pumping stations, overflows have been observed only at 5 stations including – Sunset, Woodworth. St George, Confederation and Oak St. Ravine PS. Out of these 5, majority of the events (over 80%) occur at the Sunset and Woodworth pumping stations. The overflows at the other 3 pumping stations are significantly less frequent and intense in comparison and mostly caused by mechanical issues. Overflows at the Sunset PS appear to be connected to the surcharging of the CSO facility given the proximity of the former to the latter in the collection system. On the other hand, given the remoteness of the Woodworth PS from the CSO facility, the overflows at the Woodworth PS are unlikely to be connected to the CSO facility overflows and potentially caused by high I&I in its sewershed and/or inadequate pumping capacity.

- Approximately 50-70% of the overflows at the CSO facility occur at peak day flows lower than the WPCP's PDF capacity of 54,400 m^3/d or 632 L/s. The key reason for these overflows is the current operation of the CSO facility which restricts the maximum flow to the WPCP at 500 L/s due to hydraulic bottlenecks at the plant. This restriction causes the CSO facility to surcharge and overflow during longer wet weather events (lasting more than 3-4 hours). As such there is a significant potential to mitigate these overflows by removing the bottlenecks at the plant and increasing the peak flow setting to WPCP's PDF capacity of 632 L/s.

Based on the above conclusions, given below are the short-term and long-term recommendations to address the current overflow issues and the high I&I in the system.

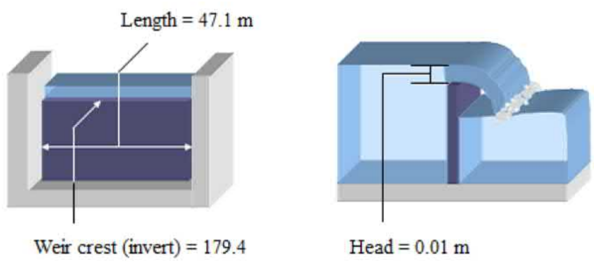
- A wet weather flow treatment investigation at the WPCP recently completed by RVA has identified the hydraulic bottlenecks at the plant and recommended remedial measures to address these. These remedial measures should be implemented as soon as possible to recover the full peak capacity of the WPCP. This would allow the WPCP to be operated at its rated peak capacity and would mitigate a significant portion of the overflows.

- Given the high frequency of overflows caused by mechanical issues at the Woodworth and other pumping stations, it is recommended to do a proper investigation of the stations to implement effective long-term solutions to address these issues.

- While a significant mitigation would be achieved by implementing the above measures, the high I&I in the collection system would persist and need a long-term solution to address the issue. The City's current program of separating storm water collection from the sewage sewers should be continued and completed soon. In addition, however, long-term flow monitoring of the key sewershed areas should be carried out to identify and address the areas with high I&I.

APPENDIX 3

St. Thomas WPCP Wet Weather Flow Optimization Study, June 2020



St. Thomas WPCP Wet Weather Flow Optimization Study

Study Report

Final

June 30, 2020

Prepared for:

R.V. Anderson Associates Limited
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June 30, 2020

RVA **195099**

City of St. Thomas
P.O. Box 520, City Hall
545 Talbot Street
St. Thomas, ON N5P3V7

Attention: John Mansell, CET
Manager of Pollution Control

Dear John:

Re: St. Thomas WPCP Wet Weather Flow Optimization Study

We are pleased to provide the enclosed final the Study Report for the St. Thomas WPCP Wet Weather Flow Optimization Study.

We appreciate the opportunity to work with the City of St. Thomas on this project and look forward to continuing our work together.

Should you have any questions or require additional information, please contact the undersigned.

Yours very truly,

R.V. ANDERSON ASSOCIATES LIMITED

John Tyrrell, M.Sc. (Eng.), P.Eng.
Project Manager

Harpreet Rai, PhD, P.Eng.
Project Engineer

Encl: Study Report (final)

St. Thomas WPCP Wet Weather Optimization Study Report

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1.0 INTRODUCTION

St. Thomas Water Pollution Control Plant (WPCP) is located at 115 Sunset Drive in St Thomas. It is a conventional activated sludge treatment plant with a rated capacity of 27,300 m^3/d (316 L/S) and peak flow capacity of 54,600 m^3/d (632 L/s). R.V. Anderson Associates Limited (RVA) has been retained by the City of St. Thomas to conduct a wet weather flow optimization study. The WPCP experiences overflow events and the City wants to reduce the number of wet weather overflows and maximize the amount of sewage that is treated.

1.1 Project Background and Objectives

St. Thomas WPCP services the City of St. Thomas and portions of the Municipalities of Southwold and Central Elgin. It is located in St. Thomas at 40359 Bush Line and is bordered by Sunset Drive to the North-East, Bush Line to the North-West, and Kettle Creek to the South-West. The plant is owned and operated by the City of St. Thomas.

St. Thomas WPCP is a conventional activated sludge facility with three (3) separate treatment trains (Plant 2, Plant 3, and Plant 4), each includes primary clarification, aeration and secondary clarification processes. There is a common headworks facility and a common ultraviolet (UV) disinfection process. Effluent pumping is available during periods of high creek levels. Standby power is provided for the facility. Treated water is discharged from the St. Thomas WPCP to Kettle Creek, located to the South-West of the facility. The plant was constructed in 1982 and has undergone several upgrades since that time by adding treatment trains.

Plants 2 and 3 were constructed in 1960s, while Plant 4 was completed in two phases between 1980 to 2003. In addition, a combined sewer overflow (CSO) was constructed in 2000 to mitigate wet weather peaks experienced at the WWTP and reduce overflows in the collection system. While the CSO facility has been successful in mitigating the wet weather peaks to a large extent, potential hydraulic issues within the WWTP prevent utilization of its theoretical peak flow capacity. As a result of that, the hydraulic peaks are controlled and restricted by the CSO to the WPCP's current hydraulic capacity, which causes overflows at the CSO and upstream locations in the collection system.

In light of the above, the project objective is to conduct an engineering assessment of the wet weather flow capacity at the WPCP to identify the hydraulic issues at the plant and select an optimization strategy to address these in order to restore the full peak capacity of the WWTP. As such the key objectives of the study include:

- Identifying hydraulic inefficiencies and bottle necks;

- Analysis of maximum flowrates through various WPCP processes during wet weather flows;
- Assessment of UV Channel flow and capacity during wet weather flows; and
- Assessment of Effluent Pumping system during wet weather flows.

2.0 BACKGROUND INFORMATION REVIEW

The following reference documents/drawings were reviewed during the study:

- As-built drawings (Contract I, II & III);
- As-built drawings (2003);
- Amended ECA of WPCP (# 9081-B7BQ9C dated Jan 14, 2019); and
- Operation and Maintenance Manual of WPCP (Rev-2, June 2018).

2.1 CSO Facility Operation

The combined sewer overflow (CSO) tank, commissioned in 2001, was constructed upstream of the WPCP on the main sewer leading from the St. Thomas sewershed. It is located north east of Sunset Drive and Bush Line in the Mill Creek Valley. The inline CSO facility is 290 m long with a storage capacity of 4,000 m^3 and includes inlet, outlet, and overflow control structures. The storage channel comprises a cast-in-place V-channel base with a side slope of 1.5 horizontal to 1 vertical to minimize the accumulation of solids. See Figures 2.1 and 2.2 for design concept and operational details of the CSO facility.

The purpose of this tank is to control and mitigate peak flows to the WPCP, biological process upsets and prevent plant overflow events. The design allows the normal dry weather flow to pass unimpeded at a velocity that is adequate to maintain self-cleansing conditions. In the event of an overflow, the discharge enters Mill Creek upstream of the WPCP.

Based on discussion with Plant Operations, the actuated gates to the outlet of this CSO Tank are set to limit the peak flow to the WPCP at 500 L/s. This limit was selected as the plant's grit chamber overflows at flows exceeding 500 L/s, creating hazardous conditions and safety issues at the WPCP. As the instantaneous flow starts exceeding this limit, the actuated gates adjust the openings to limit the outflow to the set point. This makes the excess flow volume accumulate in the CSO leading to a rise in the liquid level in the same. In cases of sustained peak flows exceeding 500 L/s, the liquid level rises to the overflow elevation of the CSO causing it to overflow to Mill Creek through a bypass line.

Figure 2.1 – CSO Facility Design Concept

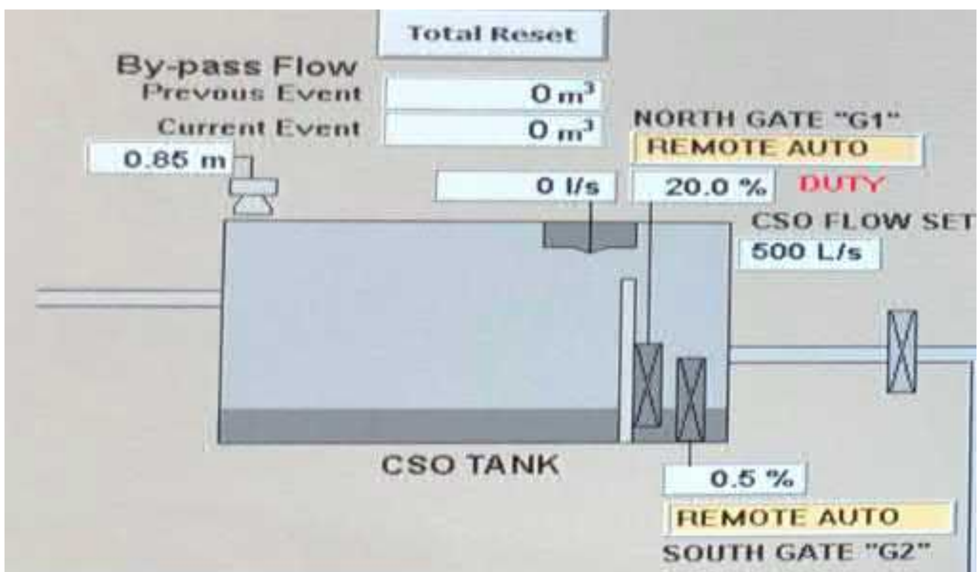


Figure 2.2 – CSO Facility Operation

2.2 Plant Operation

Operations has set up the flow distribution by adjusting the plug valves on the individual feed pipes from the Grit Tank outlet to Plant 2, Plant 3 & Plant 4. The plug valves are actuated control valves with flowmeters upstream. The current flow distribution is:

- 20% of total flow to Plant 2;

- 38% of total flow to Plant 3; and
- 42% of total flow to Plant 4. See Figure 2 of SCADA Screen showing the flow distribution.

See Figure 2.3 for current operational set-up and WPCP details. The City informed RVA that during the wet weather flow events, when the total flow rate is more than 500 L/s, the Grit Tank starts to overflow.

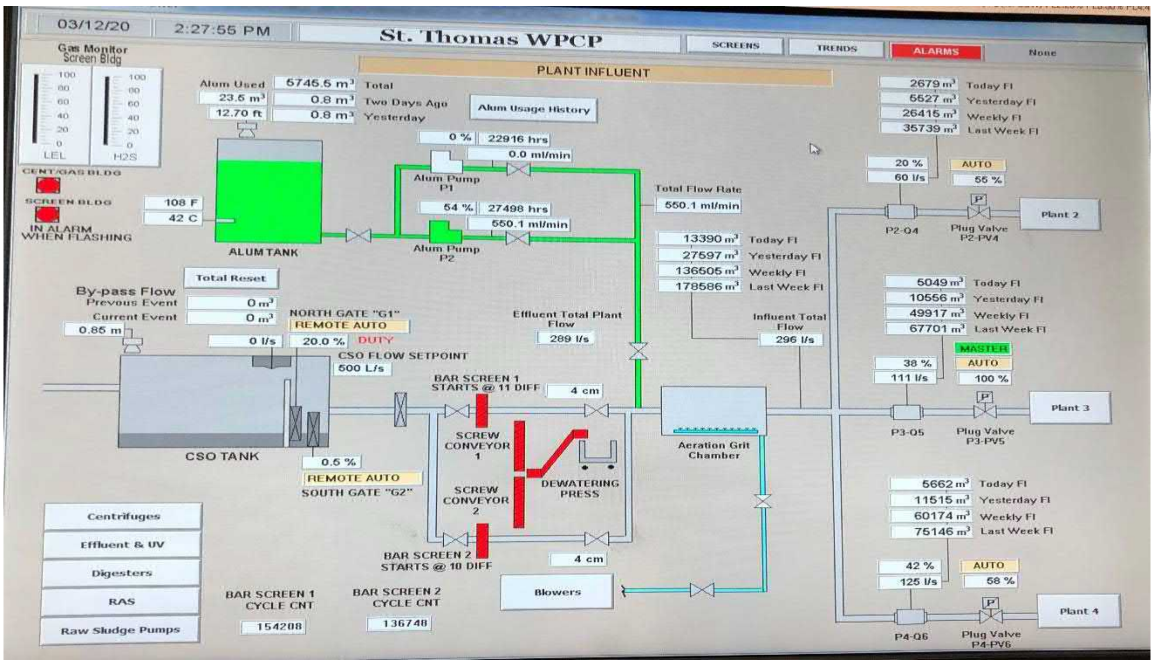


Figure 2.3 – St. Thomas WPCP SCADA Screen

3.0 PLANT PROCESS AND HYDRAULICS REVIEW

3.1 Process Capacity Review

Peak process capacities of the individual unit processes including – primary clarifiers, aeration tanks and secondary clarifiers were determined based on the MECP guidelines. As indicated the overall capacity of the WPCP is limited by the secondary clarifiers' peak flow capacity of 632 L/s.

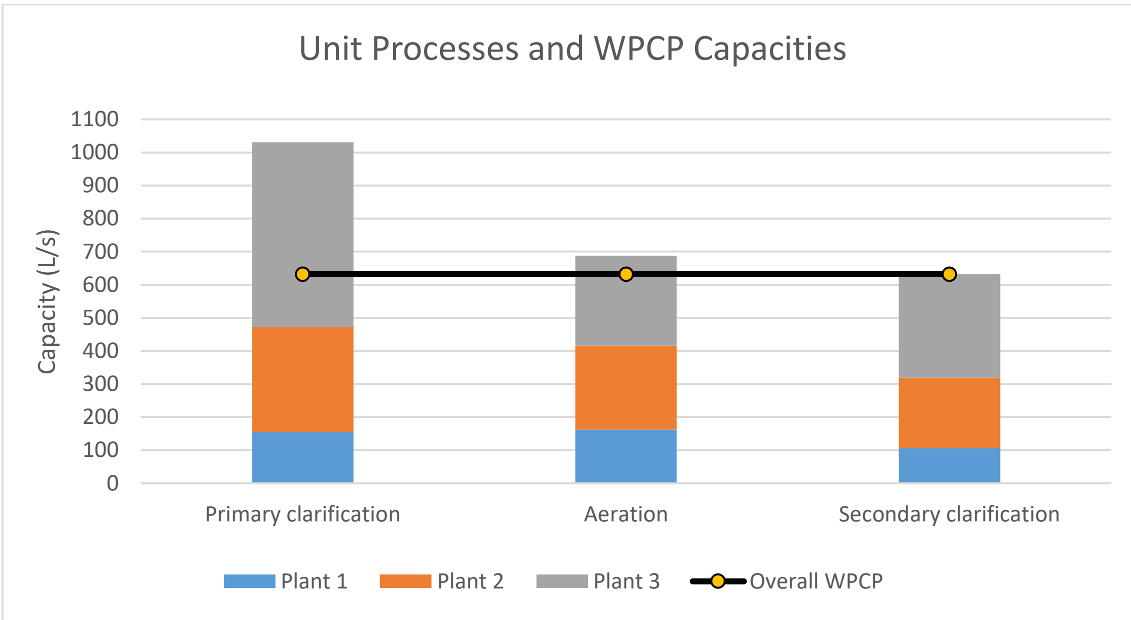


Figure 3.1 – Unit Processes and WWTP Capacities

3.2 Hydraulic Capacity Assessment

3.2.1 Hydraulic Modelling

The hydraulic modelling of the Plant 2, Plant 3 and Plant 4 was done on Visual Hydraulics (Version 4.1). For the purpose of initial calibration, the model was set up at a total flow rate of 500 L/s and the current flow distribution ratio set up by operations, and was validated with the existing hydraulic profiles (from As-built drawings - 2003) of the WPCP in Appendix 1. Note that the current flow distribution approximates the process capacity distribution of the three plants indicated in Section 2.2. See Table 3.1 for details. The objective of this model was to verify the observations in the field at flows exceeding 500 L/s, and to identify the hydraulic bottlenecks and issues at this flow value.

Table 3.1 – Current Flow Distribution at WWTP

Plant	Flow setting (L/s)	Flow setting (%)	Process capacity share (%)
Plant 2	95	19%	20%
Plant 3	185	37%	38%
Plant 4	220	44%	42%

3.2.2 Key Observations and Hydraulic Bottlenecks

The hydraulic model at 500 L/s flow rate indicated that while there are no hydraulic issues in Plants 2 and 4 at their respective flow values of 95 L/s and 220 L/s respectively, the following major bottlenecks in Plant 3 (at a flow of 185 L/s) cause the flow to back-up in the grit chamber leading to the overflows:

1. Plant 3 feed pipe in the basement of the Grit Tank reduces from 500 mm to 300 mm for installation of the flow meter, followed by increase from 300 to 350 mm for the plug valve installation, and finally expansion to 500 mm via a 350x500mm expander. These fittings within a span of approximately 3.0 m of pipe length create a total headloss of 585 mm at a flow of 185 L/s. See Figure 3.1 for the pipe fitting details.

2. In addition, the other parts of the Plant 3 influent piping create an additional headloss of 405 mm due to pipe friction and fittings. This leads to a total headloss of 990 mm, which raises the liquid level elevation in the Grit Tank to 201.01 m. This leaves a freeboard of only 90 mm with the Grit Tank top of concrete elevation of 201.10 m, and therefore approaches the overflow conditions as observed in the field. This model prediction validates the WPCP's current peak limit of 500 L/s under the current operating conditions.

3. As such the Plant 3 influent pipe must receive lower flow than 185 L/s to reduce the head loss to a degree that provides an adequate buffer against an overflow in the Grit Tank.

4. Incremental flow reduction to Plant 3 in the model revealed an optimal peak flow value of 150 L/s which reduces the headloss to 660 mm from the 990 mm observed at 185 L/s and provides a free board of 330mm in the Grit tank, and therefore an adequate protection against overflow.

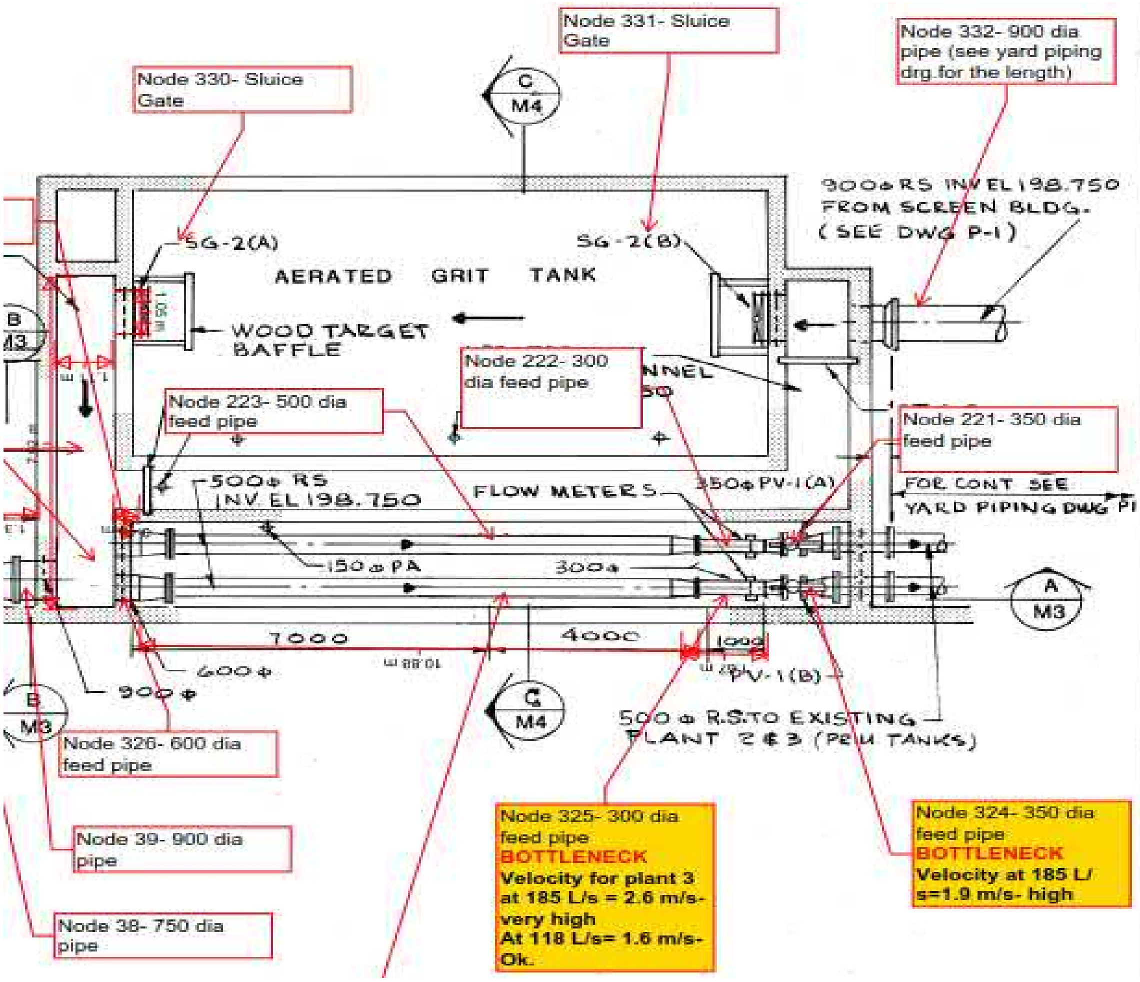


Figure 3.2 – Plant 3 Flowmeter and Piping

5. Reduction in peak flow to Plant 3 however means that higher flow has to be pushed through either Plant 2 or 4 or both. Plant 2 is limited by its process capacity of 106 L/s, as such the flow through it should not exceed this value. The process peak capacity of Plant 4 is 312 L/s and therefore the peak flow to it can be increased to this value if allowed by hydraulics. In summary flows of 106 L/s, 150 L/s and 312 L/s through Plants 2, 3 and 4 should give a peak capacity of 568 L/s.

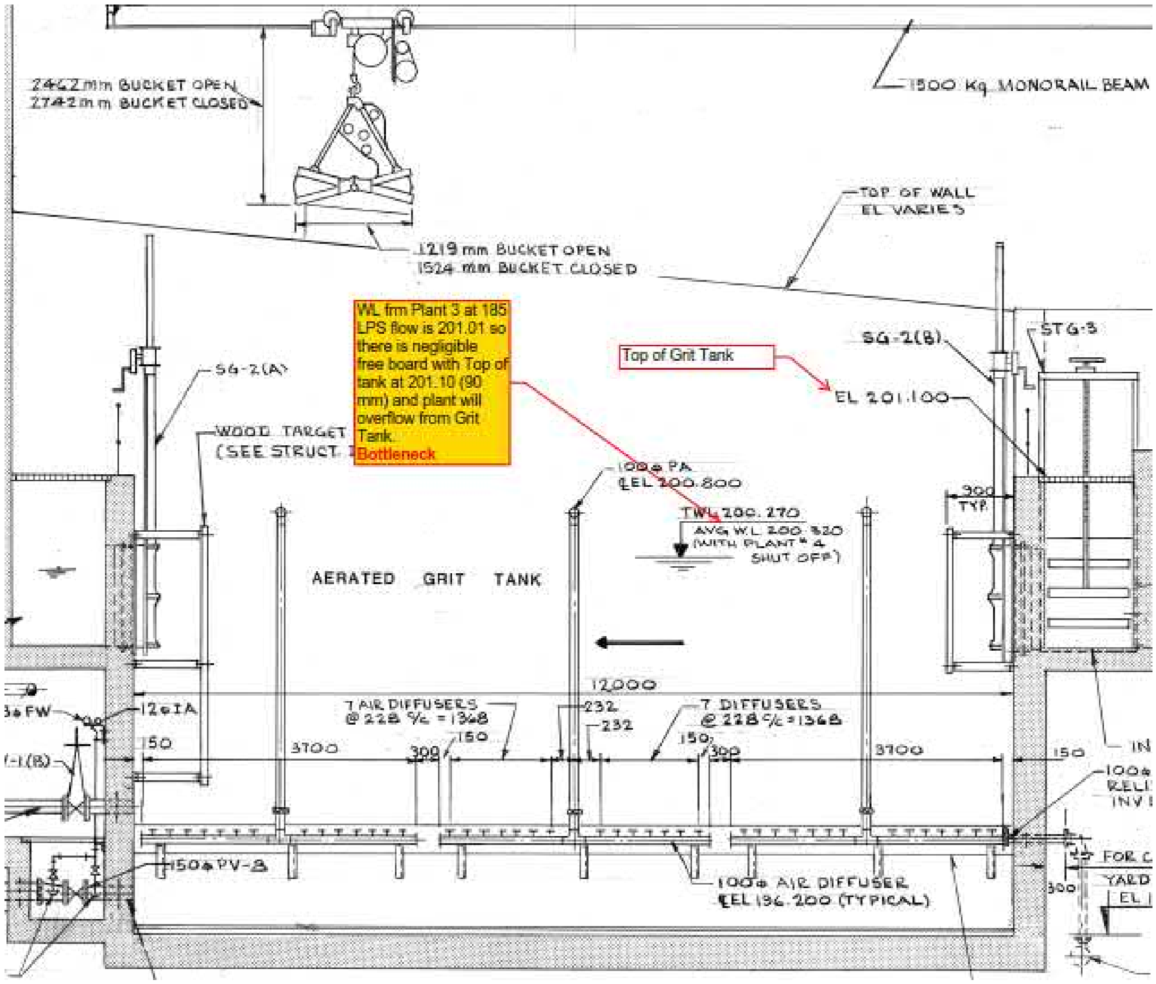


Figure 3.3 – Grit Tank Elevations

3.2.3 UV and Effluent Discharge Systems

Under normal conditions, the disinfected final effluent overflows the level control weir, flows to the flood control chamber by via a 600 mm pipe, and is finally discharged via a 600 mm outfall to the creek. See Figure 3.4 for details. Both the weir and the effluent channel are significantly oversized for the peak capacity requirement of 632 L/s and therefore are not the bottlenecks under peak flows.

During wet weather, as the creek level rises up to the high water level in the effluent channel, gravity flow via this route is no longer possible due to lack of driving head. These conditions trigger the effluent pumping system comprising three pumps (2 duty, 1 stand-by) each rated for 330 L/s at 49 m TDH. With each pump provided with a separate discharge header, the effluent pumping station has a firm capacity of is 660 L/s, and a total capacity of 990 L/s. As such the pumping system itself is not a hydraulic bottleneck for the peak flow conditions.

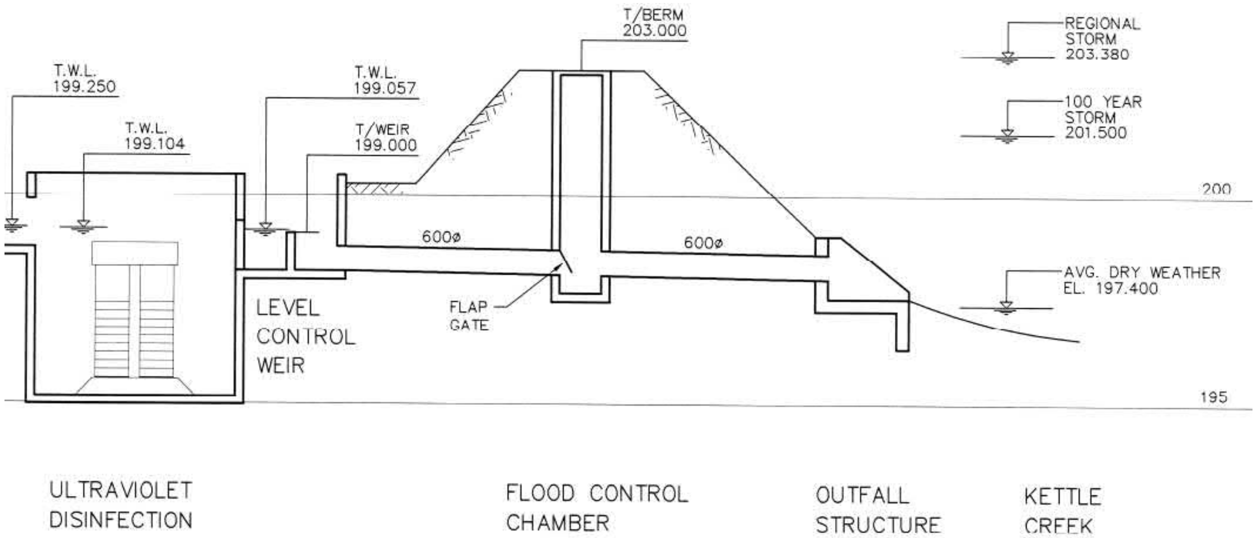


Figure 3.4 – UV System and Effluent Discharge – Normal Conditions

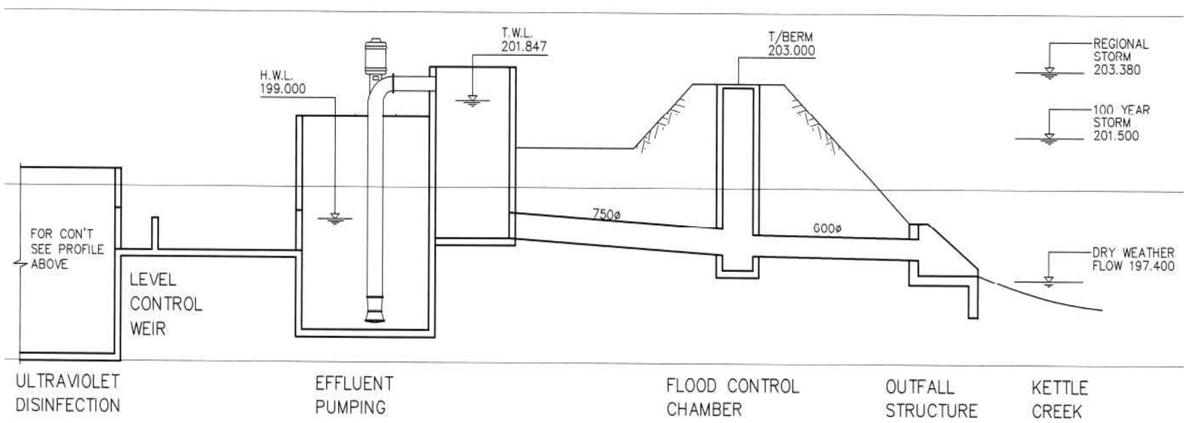


Figure 3.5 – Effluent Discharge Under Wet-Weather Conditions

The effluent pumps discharge the effluent to the elevated effluent well which is connected to the flood control chamber via a 750 mm pipe. The additional head provided by the elevated chamber now drives the flow in the 600 mm outfall. Figure 3.6 shows the correlation between available head and the flow through the pipe (flowing full) based on Manning's equation.

An increase in effluent flow or the creek level raises the water level in the flood control chamber and maintains the differential head and the required flow as long as the liquid level in the elevated well is below the TWL, and the available head is more than 430 mm. In other words, the peak flow capacity of the outfall system is variable, with significantly higher flow potential above the Plant's peak capacity of 632 L/s at heads higher than 430 mm, and lower than the peak capacity once the head drops below this critical value. Note that the critical head value is significantly higher than the available head of 347 mm at 100-year storm, which means that the peak capacity could be reduced below 632 L/s even during a lesser intensity storm event.

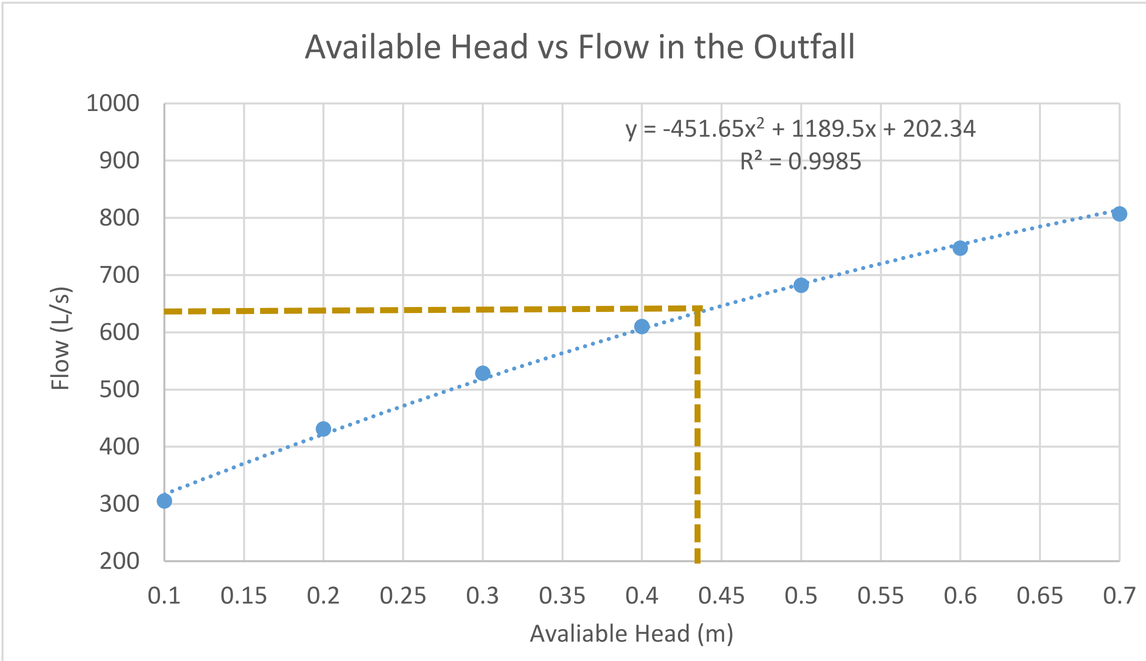


Figure 3.6 – Available Head Vs Flow Correlation in the Outfall

3.3 Measures to Address the Hydraulic Bottlenecks

To determine the potential remedial measures, the hydraulic model was run at 632 L/S with changes in the flow distribution as well as the identified bottleneck elements. Four (4) measures were identified which would incrementally restore the full hydraulic capacity of the WPCP. The following sections describe the recommended remedial measures to address the hydraulic bottlenecks.

3.3.1 Measure #1 – Adjustment of Inter-Plant Flow Distribution

As indicated in Section 3.2.2, reducing the Plant 3 Peak flow to 150 L/s and increasing that of Plant 2 and 4 to 106 L/s and 312 L/s respectively would increase the WPCP's peak capacity to 568 L/s. The flow distribution in this case is summarized in Table 3.2. The peak flow values in all three plants are equal to or less than the peak process capacities of each indicating no process issues with this flow distribution.

Table 3.2 – Adjusted Flow Distribution at WWTP – Measure 1

Plant	Flow distribution setting (%)	Flow distribution (L/s)		Peak process capacity (L/s)
		Average	Peak	
Plant 2	19%	60	106	106
Plant 3	26%	82	150	214
Plant 4	55%	174	312	312
Total	100%	316	568	632

While the operating staff have process related concerns in reducing the flows to Plant 3 (due to low F/M conditions at the current loads, this can be resolved by reducing the current operating range of MLSS to achieve target SRTs of 5-8 days as opposed to the current target of 8-12 days.

3.3.2 Measure #2 – Upsizing Plant 3 Influent Flow Meter and Plug Valve

As discussed in the previous section, the reduced 300/350 mm pipe section with flow meter and plug valve causes a major head loss in the Plant 3 influent pipe. The model predicts that replacing this section (including the flow meter and plug valve), with a 450mm section would reduce the headloss in the Plant 3 influent pipe from the current 990 mm at a peak flow of 185 L/s to 820 mm at its rated peak flow capacity of 214 L/s, giving a freeboard of 170 mm at the grit chamber. Also, this would not impact the flow meter accuracy as the flow velocity will remain within the recommended velocity range of 0.3 to 3 m/s. As such, this change will increase the WPCP capacity to its full peak capacity of 632 L/s.

The flow distribution in this case is summarized in Table 3.3. As indicated, the peak flow values in all three plants are equal to or less than the peak process capacities of each indicating no process issues with this flow distribution.

Table 3.3 – Adjusted Flow Distribution at WWTP – Measure 2

Plant	Flow distribution setting (%)	Flow distribution (L/s)		Peak process capacity (L/s)
		Average	Peak	
Plant 2	18%	57	106	106
Plant 3	30%	95	214	214
Plant 4	52%	164	312	312
Total	100%	316	632	632

It is important to note that the available freeboard of 170 mm at the grit chamber with this upgrade is lower than the target minimum freeboard of 300 mm. Based on that, upsizing the influent pipe of Plant 3 from 500 mm to 600 mm was considered. While this upsizing would decrease the head loss and provide the free board close to 300 mm at peak flow, the corresponding velocity in 600 mm pipe would be 0.65 m/s which approaches a non-scouring velocity and would likely cause silting in the pipe under average flows.

On the other hand, given that the current flow settings provide less than 10 mm freeboard at the current peak flow of 185 L/s to Plant 3, 170 mm freeboard at 214 L/s, although not ideal, would be a significant improvement over the current peak flow operation. As such, implementation of this measure would retrieve the rated peak capacity of 632 L/s of the WPCP, while providing an adequate hydraulic buffer at peak flow.

3.3.3 Measure #3 – Raising the Elevated Effluent Well Walls

As indicated, the capacity of the outfall varies with the outfall level and is limited by a critical level in the creek, at which its peak capacity drops below 632 L/s due to lack of available head to drive the required flow. As such despite implementing the recommended upstream measures in the WPCP, the full peak capacity may be compromised by the high creek levels during intense wet weather events.

This can be addressed by raising the walls the elevated effluent well by one meter which would increase the operating TWL in the well and therefore the available head at high creek levels. Since raising the TWL by more than 0.5m would bring it above the discharge elevation of the current discharge headers, the discharge headers' elevations would have to be raised as well.

3.3.4 Summary of Remedial Measures, Costs, and Capacities

Table 3.5 summarizes the remedial measures, incremental capacities achieved with each, and the associated cost estimates.

Table 3.4 – Capacities Summary with Remedial Measures

Scenario	Peak Flow Capacity (L/s)				Cost Opinion	Remarks
	Plant 2	Plant 3	Plant 4	Total		
Base	95	185	220	500	NA	Negligible free board at GC due to Plant 3 flow
Remedial measure #1 – Adjustment of Flow Distribution between the Plants	106	150	312	568	$25k	Plant 3 flow reduced to maintain minimum 300 mm freeboard at GC
Remedial measure #2 – Upsizing Plant 3 influent flow meter and plug valve	106	185	312	632	$100k	632 L/s achievable but with 170 mm free board available at GC
Measure #3 – Raising the Elevated Effluent Well Walls	106	214	312	632	$150k	All bottlenecks removed and full capacity restored

3.3.5 CSO/Plant Operational Changes

The Wet Weather investigation has determined that the capacity issues are related to hydraulic bottlenecks which can be addressed via optimal flow balancing between the plants and physical removal of the bottlenecks. The proposed operational changes for re-balancing the flows between the plants can be completed with the current control system and will only require changes to the plant flow values. In addition, with the restored incremental capacities achieved with removal of each of the bottlenecks, the peak flow limit at the CSO will need to be increased in accordance with the corresponding peak capacity achieved.

4.0 CONCLUSIONS

Based on this study and evaluation, the removal of hydraulic bottlenecks would allow the City to restore the full peak capacity of the WPCP, allowing higher wastewater flows to be treated during wet weather, and lessen the overflows at the CSO Facility.

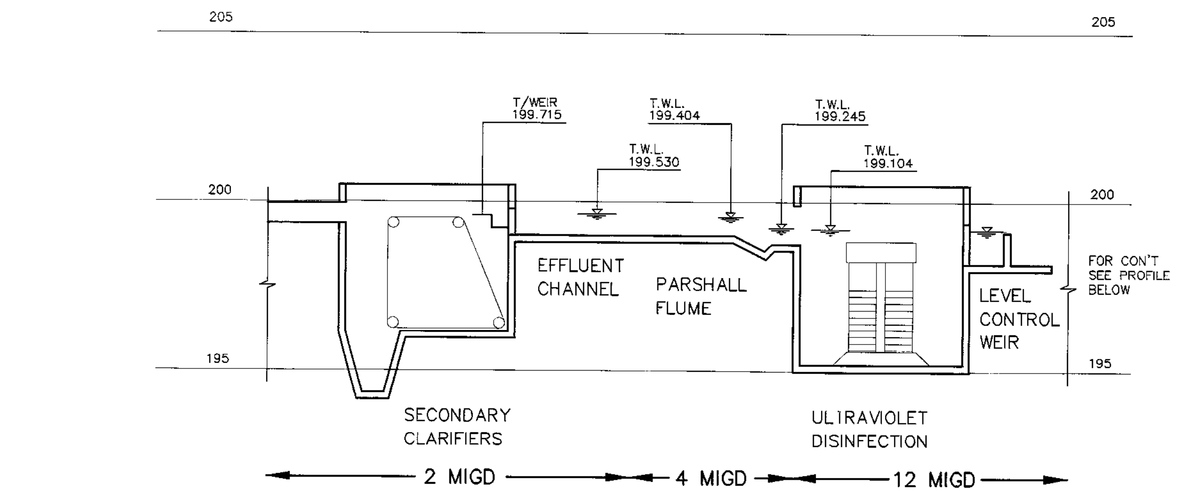
Measures 1 and 2, being easy to implement, cost-effective, and with the ability to restore a major portion of the currently unavailable capacity, should be given priority.

While implementing the above measures would restore the peak capacity for most wet wet-weather events, the full capacity would not be available for more intense/extreme events that raise the creek level beyond the critical elevation. As such measure 3 would be required to meet the capacity during these events.

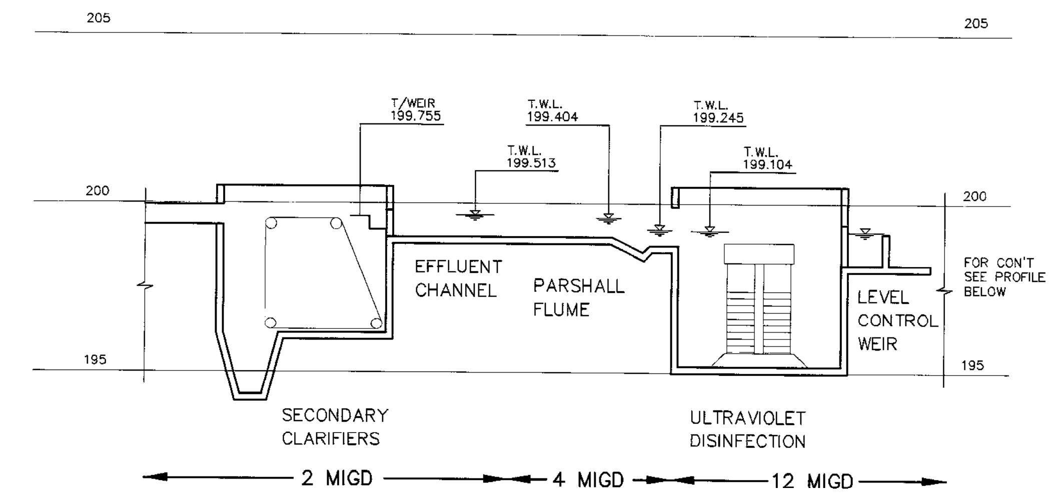
It is also important to note that the WPCP, as designed, has no protection against the Regional Flood as the Regional Flood elevation is significantly higher than that of the berm. As such, raising the berm and the elevated effluent well should be considered by the City before implementing measure 3 as a part of the ongoing Pollution Prevention Control Plan (PPCP) Project. Further, additional studies for enhancing the WPCP hydraulic capacity like, on-site-equalization, chemically enhanced primary treatment, and Plant 2 reconfiguration or upgrades should be considered as a part of the PPCP.

APPENDIX 1

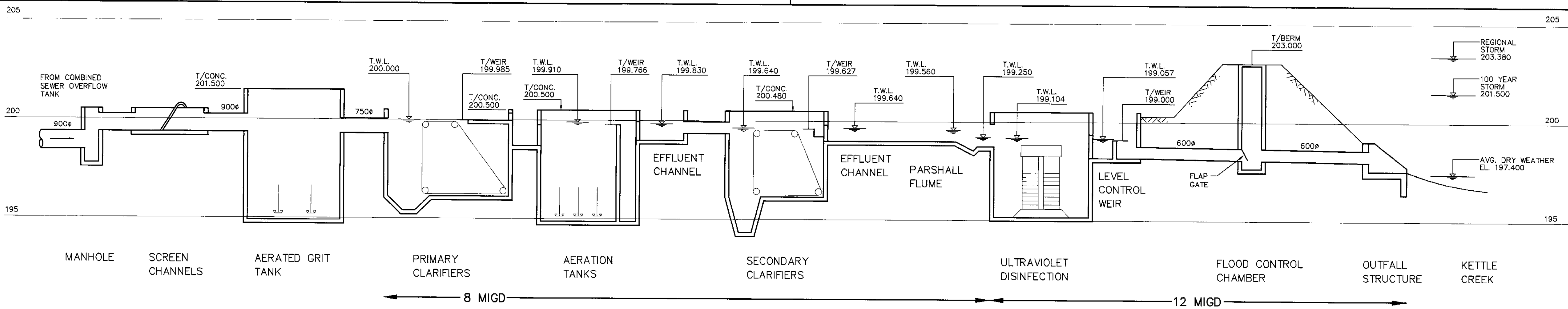
AS-BUILT HYDRAULIC PROFILE



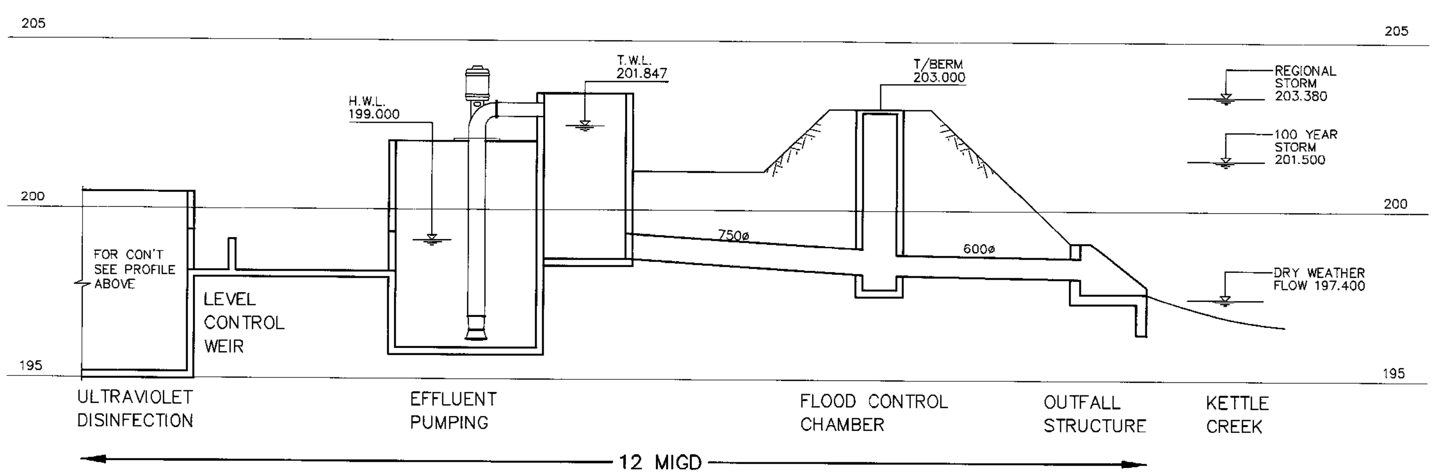
PLANT No. 2 HYDRAULIC PROFILE



PLANT No. 3 HYDRAULIC PROFILE



PLANT No. 4 HYDRAULIC PROFILE



PLANT No. 4 HYDRAULIC PROFILE w/EFFLUENT PUMPING

Hydromantis, Inc.
Consulting Engineers

HAMILTON (905) 522 0012 | CAMBRIDGE (519) 624 7223 | ONTARIO

NOTES:

NO.	REVISION	DATE	INITIAL
1	RECORD COPY	MAR./03	J.M.
0	ISSUED FOR TENDER	OCT.9/01	J.M.

APPROVED

ORIGINAL DRAWING SEALED & SIGNED BY J. F. MYATT P. ENG. DATED OCT. 4, 2001

GENERAL

HYDRAULIC PROFILES

CITY OF ST. THOMAS

WPCP UPGRADE

EARTH TECH

Earth Tech (Canada) Inc. London, Ontario 519.673.0510

DRAWN BY: A.B. CHECKED BY: J.M.	SCALE: N.T.S.	DATE: SEPT. 01	SHEET NO. 8
CADD SYSTEM AutoCAD	DWG NO. A1-00406-C104	REV. 1	CONTRACT

7-0344

APPENDIX 2

HYDRAULIC MODEL AND CALCULATION SUMMARY FOR 500 L/s

Removed for Document Brevity in PPCP Appendix

APPENDIX 4

St. Thomas WPCP Wet Weather Flow Optimization Study, June 2020

City of St. Thomas
Pollution Prevention Control Plan Study

Technical Memorandum # 2/3
Flow Monitoring and Hydraulic Modelling Data Gap Analysis

City of St. Thomas

RVA 205153

September 30, 2020

St. Thomas Pollution Preventive Control Plan
Flow Monitoring and Hydraulic Modelling Data Gap Analysis

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1.0 INTRODUCTION

1.1 Objectives

The City of St. Thomas (City) has retained R.V. Anderson Associates Limited (RVA) for the preparation of a Pollution Prevention Control Plan (PPCP). The PPCP is a part of the City's ongoing efforts to improve the performance of their sanitary and storm sewer infrastructure. In addition, it would provide the City with a road map for implementation of infrastructure and operational improvements that will mitigate the impacts of wet weather sewer system overflows on the environment and help the City mitigate risk in alignment with the City's commitment to environmental stewardship and the provision of sustainable municipal services.

Pursuant to the above, it was originally intended to produce two separate technical memoranda the first with the objective of identifying gaps in the flow monitoring and configuration data (Tech Memo #2) and the second to identify a flow monitoring program required to perform a more complete hydraulic analysis of the collection system, pumping stations, overflows and to assess overall wet weather performance (Tech Memo #3). The City and RVA decided that that given the significant overlap in these two reports, a single technical memorandum would be prepared (Tech Memo #2/3)

Tech Memo #2/3 undertakes the following activities:

1. Review of the combined sewer, sanitary sewer, manhole and pumping station data provided by the City to determine the level of detail and completeness for use in developing a hydraulic model of the collection system;
2. Review of past inflow and infiltration (I&I) studies for the Aldborough-Leger and Woodworth areas to understand the performance observations and recommendations, the flow monitoring data (and it's applicability to this project) and the system configuration data used for hydraulic modelling;
3. Review of other relevant studies (i.e. 2017 flow monitoring, 2010 wastewater master plan, etc.) to develop and understanding of the collection system performance that could aid in the development of the hydraulic model; and
4. Provide recommendations for further data collection/investigations to ensure sufficient detail for the development of a hydraulic model and analysis of the wastewater collection system performance.

1.2 Hydraulic Modelling Requirements

The development of a hydraulic model of the wastewater collection system will be completed using the PCSWMM software package. PCSWMM is based on the USEPA SWMM engine and is a widely used platform for analyzing the dynamic performance of sanitary and combined collection systems including pump cycles, overflows, surcharging and backwater effects. The primary objectives of developing the model are to analyze the following:

- The collection system inflow and infiltration response to precipitation;
- The collection system hydraulic performance including pipe capacities, bottleneck identification, surcharge dynamics and wet weather flow capture;
- The pumping station capacities to convey peak wet weather flow;
- The overflow volumes at the pumping stations and CSO facility;
- The hydraulic impacts of proposed and future developments; and
- The effectiveness of proposed collection system upgrades.

The data required to construct a hydraulic model of the St. Thomas sanitary and combined sewer systems will include the following:

- Manholes (locations, inverts, and ground elevations);
- Sewers (diameter, material, length, inverts);
- Overflows and outfalls (diameter, location, flap gate, tailwater elevation if applicable)
- Pumping stations (pump curves/capacities, start/stop elevations, overflow details, wet well size/configuration);
- Wet weather flow storage (CSO facility details, location, volume, configuration);
- Dry weather flow distribution (to be estimated based on development and flow monitoring data); and
- Wet weather flow parameters to model inflow/infiltration responses (to be estimated/calibrated using flow monitoring data).

The calibrated collection system model will be used during the development of the pollution prevention control plan but can also be a valuable planning tool for the City to check the impacts of subsequent developments and system upgrades.

2.0 DATA REVIEW

2.1 Collection System Configuration

2.1.1 Sewers and Manholes

The primary data source for the combined and sanitary sewers and manholes is the GIS database provided by the City. This database is MAPINFO-based and contains most of the required data for modelling the sewers and manholes. The gravity pipe sections of the collection system are well defined in the database as shown in the sample data presented below.

Table 2.1 – Sample of the GIS Database for the Combined and Sanitary Sewers

ASSET_ID	FROM_STREET	ON_STREET	TO_STREET	DIAMETER	LENGTH	MATERIAL	SLOPE	UP_INVERT	DOWN_INVERT	CONTRUCTION	AVG_DEPTH
SAS5	DAVID DRIVE	PENHALE AVENUE	SOUTH END	200	36	Asbestos Cement	0	240.19	240.12	1980	3
SAS6	RAVEN AVENUE	PENHALE AVENUE	DAVID DRIVE	200	30.8	Asbestos Cement	0.57	240.11	239.93	1980	3.1
SAS7	RAVEN AVENUE	DAVID DRIVE	PENHALE AVENUE	200	76.8	Asbestos Cement	2.82	239.87	237.71	1980	3.2
SAS8	RAVEN AVENUE	DAVID DRIVE	PENHALE AVENUE	200	85.6	Asbestos Cement	0.79	237.71	237.03	1980	3.1
SAS9	RAVEN AVENUE	DAVID DRIVE	PENHALE AVENUE	200	75.1	Asbestos Cement	0	237.03	236.71	1980	2.9
SAS10	RAVEN AVENUE	DAVID DRIVE	PENHALE AVENUE	200	77.9	Asbestos Cement	0.52	236.71	236.3	1980	2.8
SAS11	RAVEN AVENUE	DAVID DRIVE	PENHALE AVENUE	200	81.1	Asbestos Cement	0.59	236.3	235.82	1980	2.8
SAS12	RAVEN AVENUE	DAVID DRIVE	PENHALE AVENUE	200	53.6	Asbestos Cement	0.8	235.78	235.36	1980	3.1
SAS13	RAVEN AVENUE	DAVID DRIVE	PENHALE AVENUE	200	54.9	Asbestos Cement	0.73	235.36	234.96	1980	3.7
SAS14	PENHALE AVENUE	RAVEN AVENUE	NOBLE LANE	200	92	Asbestos Cement	0.61	239.61	239.05	1980	3.8
SAS15	RAVEN AVENUE	PORTER PLACE	NORTH END	200	65.5	Asbestos Cement	0.82	240.19	239.65	1980	3.4
SAS16	RAVEN AVENUE	PORTER PLACE	NORTH END	200	66.8	Asbestos Cement	0.96	239.65	239.01	1980	3.2
SAS17	LAWRENCE AVENUE	RAVEN AVENUE	PORTER PLACE	200	92	Asbestos Cement	0.71	238.99	238.33	1980	3.2
SAS18	DYER STREET	RAVEN AVENUE	LAWRENCE AVENUE	200	86	Asbestos Cement	0	238.33	237.92	1980	3
SAS20	DAVID DRIVE	RAVEN AVENUE	DYER STREET	200	59.6	Asbestos Cement	2.4	237.14	235.71	1980	3.6
SAS21	CALDWELL STREET	DYER STREET	RAVEN AVENUE	200	54.6	Asbestos Cement	0	237.9	237.63	1980	3
SAS22	CALDWELL STREET	DYER STREET	RAVEN AVENUE	200	53.9	Asbestos Cement	0	237.63	237.39	1980	3.5
SAS23	CALDWELL STREET	DYER STREET	RAVEN AVENUE	200	69.2	Asbestos Cement	0.52	237.36	237.01	1980	3.7
SAS24	CALDWELL STREET	DYER STREET	RAVEN AVENUE	200	70.3	Asbestos Cement	0	237.01	236.66	1980	3.2
SAS25	CALDWELL STREET	DYER STREET	RAVEN AVENUE	200	84.9	Asbestos Cement	0.52	236.66	236.22	1980	3.1
SAS26	KER STREET	VANIER PLACE	EAST END	200	80.8	Asbestos Cement	1.9	233.94	232.41	1975	2.6
SAS27	KER STREET	VANIER PLACE	EAST END	200	91.4	Asbestos Cement	1.9	232.41	230.67	1975	2.6
SAS28	KER STREET	VANIER PLACE	LEGER AVENUE	200	91.4	Asbestos Cement	0.5	230.67	230.21	1975	2.7
SAS29	KER STREET	VANIER PLACE	LEGER AVENUE	250	91.4	Asbestos Cement	0	230.16	229.85	1975	2.6
SAS30	KER STREET	VANIER PLACE	LEGER AVENUE	250	82.6	Asbestos Cement	0	229.85	229.57	1975	2.5

The database, however, is not complete and was found to be lacking pipe invert elevations for several sanitary and combined sewer sections. The sewers with missing elevation data are presented in Figures 2.1 and 2.2 on the following pages.

The locations of the sanitary and combined sewer manholes are included in the GIS database but the top elevations (ground surface) and inverts are not directly attributed to these manholes. The pipe elements (as noted in Table 2.1 above) include the invert elevations and an estimate of the average depths of burial and can be used to define the manhole elevations. We will also use the City's digital elevation model of the terrain to assign approximate ground elevations to the manhole covers to refine these estimates.

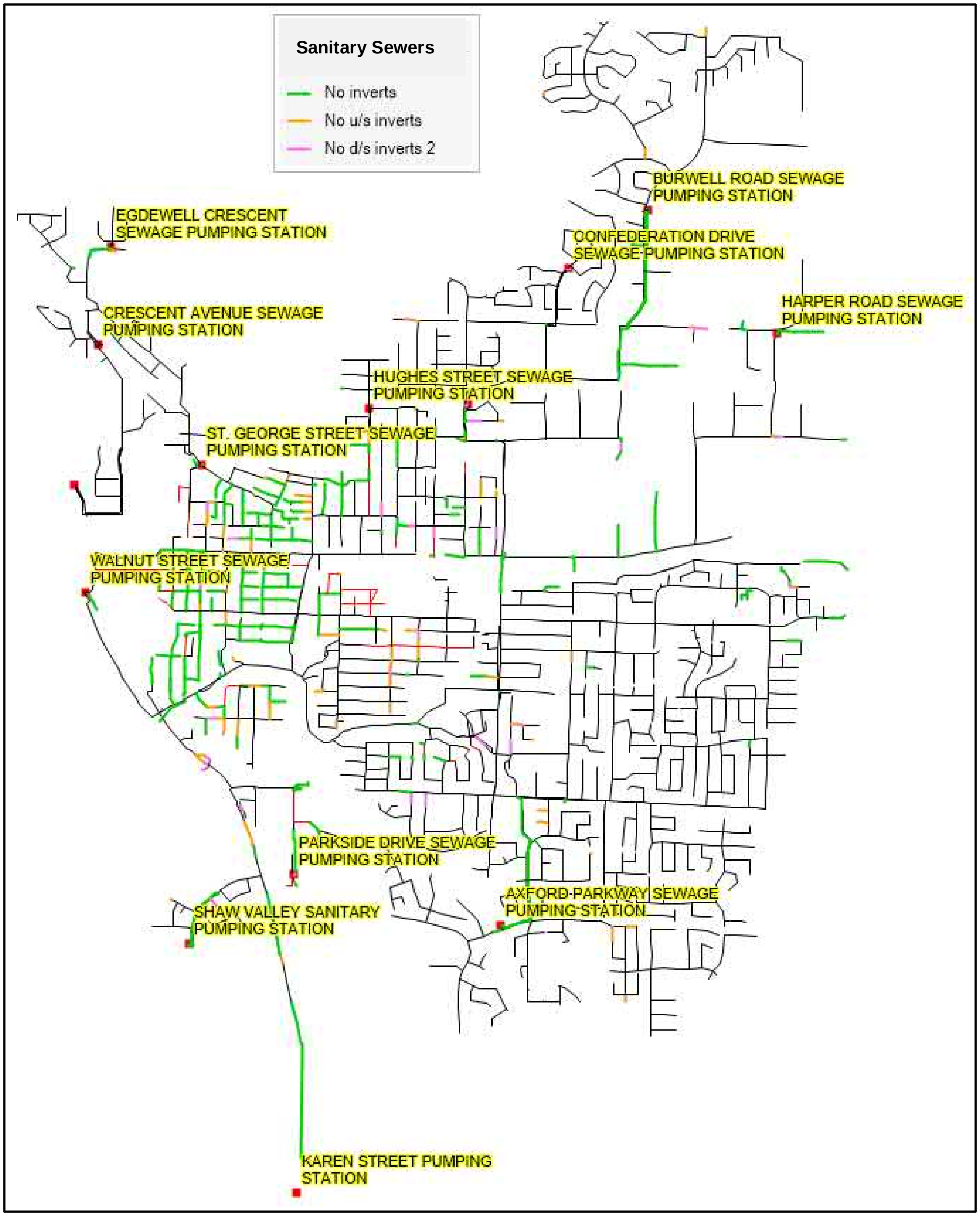

Figure 2.1 – Sanitary Sewer Missing Invert Data Overview

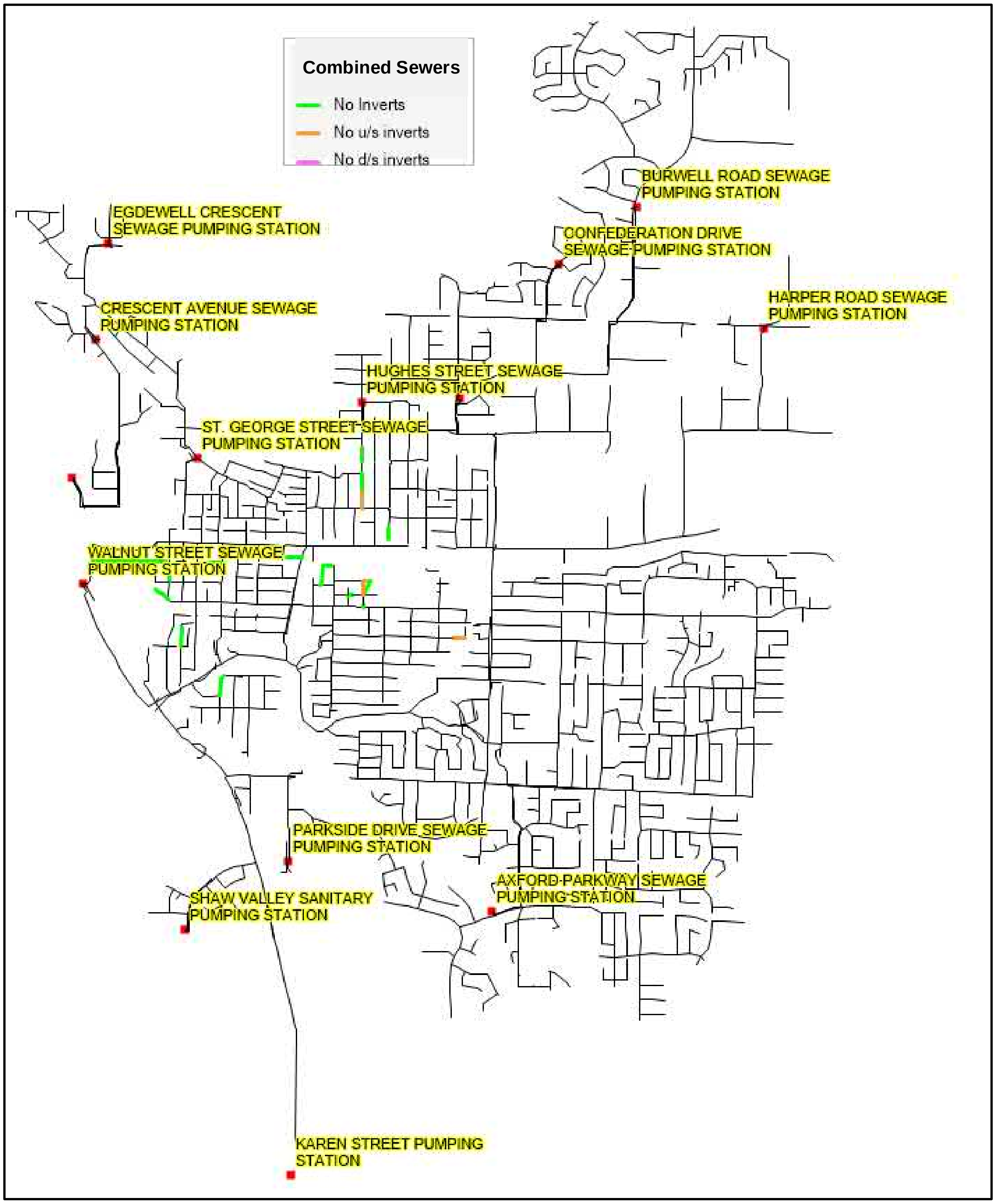

Figure 2.2 – Combined Sewer Missing Invert Data Overview

2.1.2 Pumping Stations

Accurate representation of the pumping stations is an important component of a comprehensive collection system model. The cycling of pumps during dry and wet weather simulations allows the model to replicate field conditions and facilitates calibration. The model will include the City's 16 sewage pumping stations which are identified in Figure 2.3 and summarized in Table 2.2.

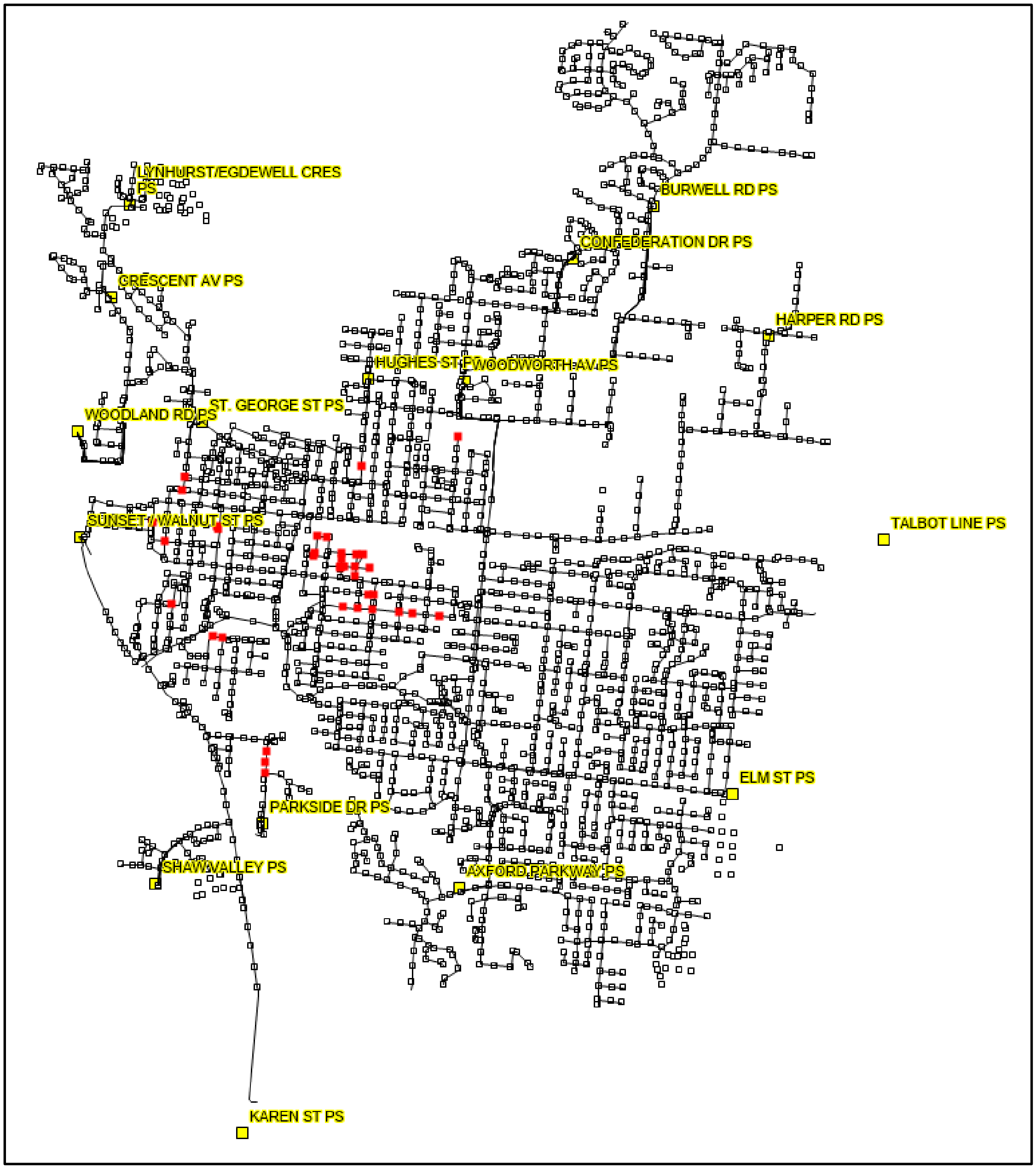

Figure 2.3 – St. Thomas Pumping Station Overview

Table 2.2 – St. Thomas Pumping Stations

Pumping Station	Construction Date	Make and Model	Duty/ Standby	Firm Capacity (L/s)	O&M Manual
Axford Parkway	1997	Gorman-Rupp ECM	1/1	56.6	Yes
Burwell Rd	1993	ITT Flygt 3170.180	1/1	44	Yes
Confederation Dr	1968	Smith & Loveless	1/1	67	Yes
Crescent Ave	1988	Hydromatic Pentair	1/1	16	Yes
Elm St	2018	Flygt 3153	1/1	44.35	Yes
Harper Rd	1973	Gorman-Rupp	1/1	21	Yes
Karen St	2011	Flygt 3153	1/1	43.2	Yes
Lynhurst	1996	Flygt 3102	1/1	23	Yes
Parkside Dr	1970	Flygt CP3127	1/1	NA	Yes
Shaw Valley	2005	Flygt 3153	1/1	62.7	Yes
St. George St	1966	Gorman-Rupp	1/1	94.6	Yes
Sunset	1973	Barnes	1/1	23	Yes
Talbot Line	2014	Xylem NP-3153	1/1	25	Yes
Hughes St	1993	ITT Flygt 3127	1/1	19.7	Yes
Woodland	1988	Hydromatic Pentair	1/1	7	Yes
Woodworth Ave	1972	Smart Turner Hayward	2/1	101	Yes

The pumping station data required for the PCSWMM model will include the following:

- Type of pumping station;
- Number of pumps;
- Pump curves/capacities;
- Start/stop elevations;
- Wet well size/configuration;
- Overflow details; and
- Historic run time and observed overflow records.

The City has very good records for its pumping stations as each station has an Operations and Maintenance Manual. This document contains all the required modelling parameters and defines how each station operates. An example of the details provided within the manuals is provided below in Figure 2.4. This illustrates the wet well configuration, pump control elevations as well as the overflow elevation.

Although run time records and overflow data will be useful information for model calibration, it is not expected that additional configuration details will be required for the pumping stations.

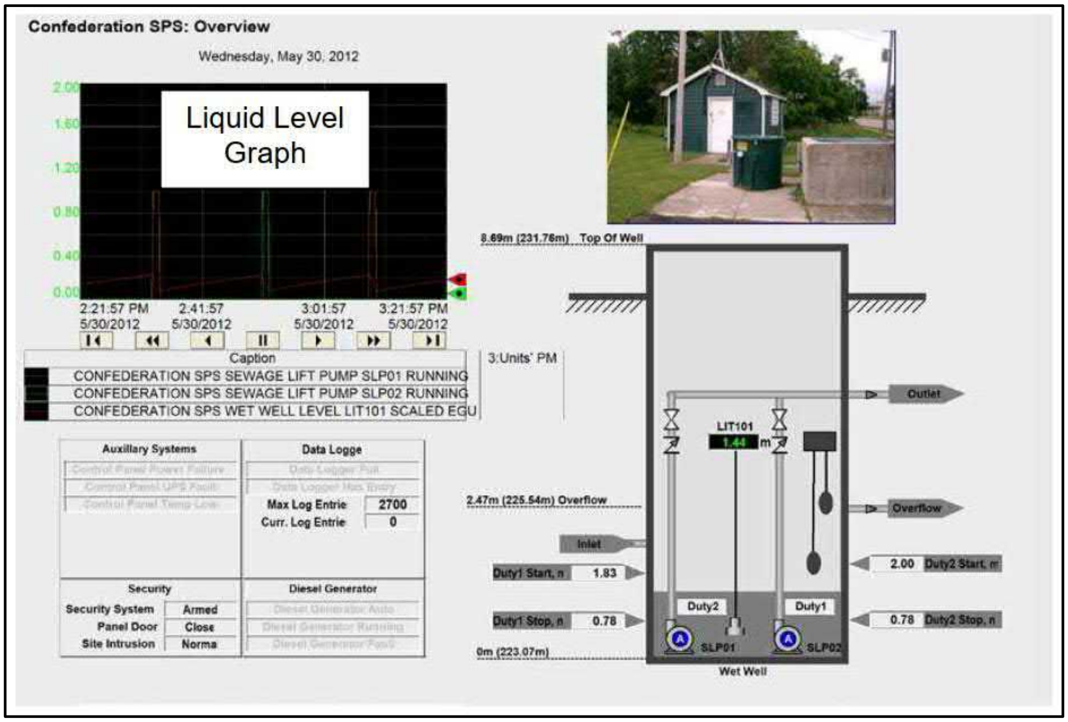

Figure 2.4 – St. Thomas Sewerage System Map

2.2 Flow Data

2.2.1 Data Requirements for the Model

Dry weather and wet weather flow data will be required for the model. Dry weather flows will be estimated based on flow monitoring data and the types and level of development for each sanitary catchment.

Theoretical dry weather flows can be derived based on serviced populations, water usage, development records, numbers of homes and businesses, and will be calibrated using flow monitoring data to reflect actual sewage generation rates. These flows tend to exhibit dual peaks (morning and evening) and will serve as the baseline flows within the model.

Wet weather flows will be estimated for both the combined sewer (predominantly surface water inflow) and the sanitary sewer (inflow and groundwater infiltration) sections of the collection system. Flow monitoring data and influent records for the wastewater treatment plant will be the most important sources of wet weather flow data and will be used to calibrate the model. Inflow and infiltration responses will be modelled using past rainfall records and calibrated to match observed flow responses. Once the model is calibrated to past observed events, the system can be analyzed in terms of typical year performance (i.e. annual wet weather flow capture) and design storm performance (i.e. 1 in 5-year rainfall).

2.2.2 Past Studies

The following past studies were reviewed and identified as useful sources of flow data for modelling:

- City of St. Thomas Sewer Flow Monitoring Study, Flowmetrix Technical Services Inc, February 2018;
- Aldborough/Leger and Woodworth Wastewater Sanitary Catchments Inflow and Infiltration Study, Cole Engineering, December 2014;
- St. Thomas Orchard Park Area Sanitary Sewer Flow Monitoring & Analysis, TQI, April 2020;
- St. Thomas Water Pollution Control Plant – Annual Performance Reports, 2008-2018
- City of Proposed Urban Expansion Areas, Infrastructure Master Plan Sanitary Sewer Servicing, November 2008;
- City of St. Thomas WWTP historic flow records;
- City of St. Thomas pumping station historic runtime/overflow volume records; and
- WWTP historic flow records sewers (diameter, material, length, inverts).

While all sources of historic data can be useful, the most important data for model calibration comes from flow monitoring. Flow monitoring data includes not only the measured flows, but also the depths and velocities. These two parameters are critical in understating depths and hydraulic grade lines within the sewer and the velocity deceases that occur during wet weather backups near capacity bottlenecks. The City has good quality/recent flow monitoring data from the Flowmetrix study and the Cole inflow and infiltration studies that cover much of the collection system. The locations of these monitoring sites are presented on Figure 2.5.

The coverage of the data is sufficient for basic construction and calibration of the hydraulic model; however, the City may wish to supplement this data with additional flow monitoring to increase the resolution and accuracy for upstream sewer sections. Additional monitoring in the northwest (Lynhurst), northeast (Burwell) and east sections of the City would fill data gaps and better match the monitoring resolution of the recent inflow and infiltration studies. Preliminary monitoring location suggestions are presented in the bullets below and on Figure 2.5.

- Northwest area of the sewer system: A flow meter is recommended upstream of the St. George Street pumping station. This will provide insight into flows draining to the Lynhurst/Edgewell Crescent pumping station, the Crescent Avenue pumping station, and the Woodlawn Road pumping station, which have relatively small catchments on their own.

- Northeast area of the sewer system: Flow meters (2) are recommended upstream of the Burwell Road and Confederation Drive pumping stations.
- East area of the sewer system: Flow meters (2) are recommended Elm and Wellington Streets.

These potential monitoring locations represent a baseline to collect representative flows for the areas of the collection system. These will be discussed with City staff in greater detail to refine the locations/numbers of meters, identify potential areas of concern and the timing impacts on this project.

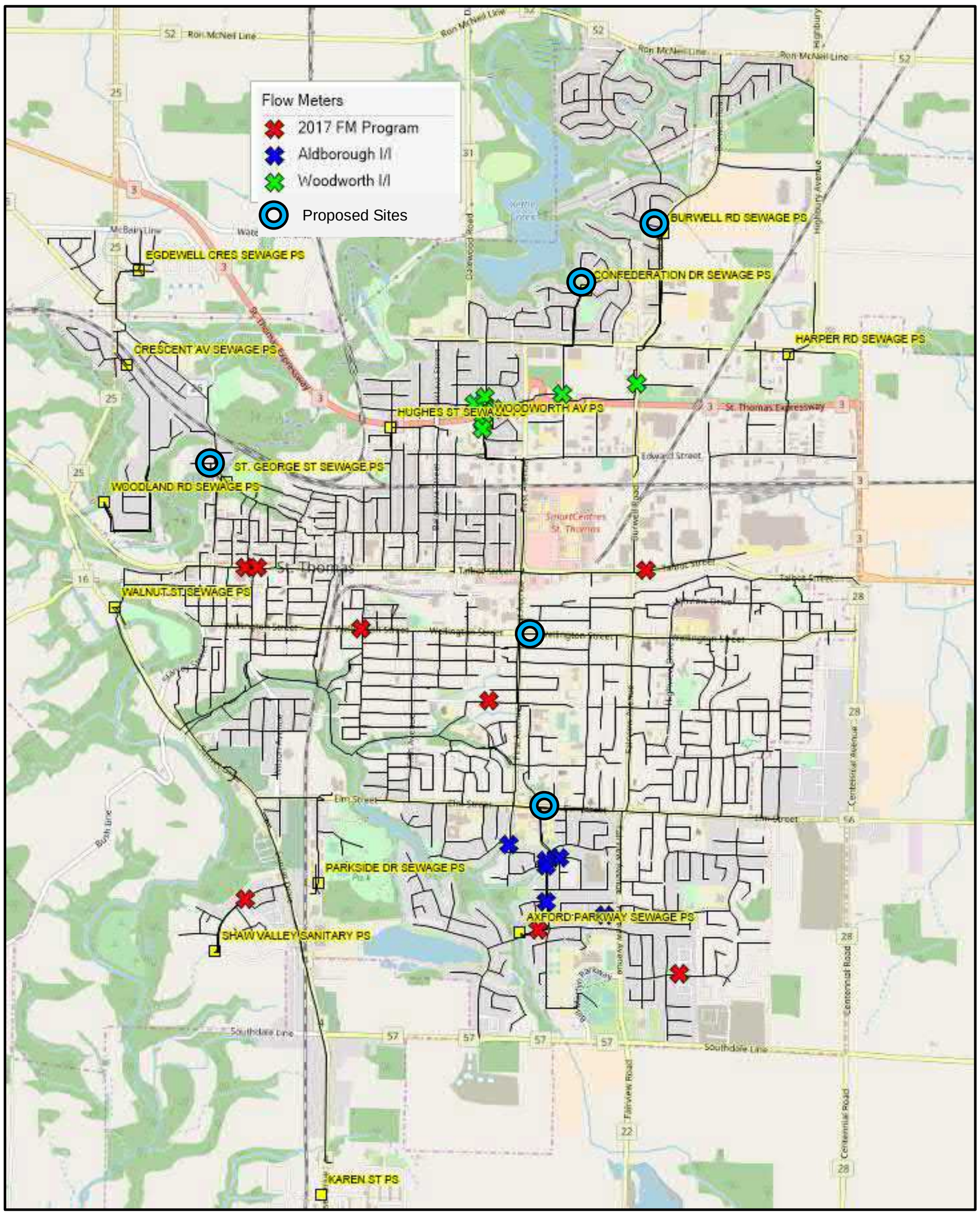

Figure 2.5 – Flow Monitoring Sites

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our review of the available collection system data and past studies, we believe the City has most of the configuration and flow data required to construct and calibrate a dynamic model of the combined and sanitary collection system. Overall, the City has very good records of the collection system and there are only a few areas where additional data is required as noted in the summary below:

- The manhole and sewer system data records are approximately 80% complete. Pipe material, diameter and length and manhole locations are complete; however, inverts are required for sections of sewer. This data was not in the City's GIS database but may be available on record drawings. If the drawings are not available, some elevations may need to be collected in the field.
- The pumping station records are comprehensive, and it is not expected that additional configuration data will be required.
- The historic flow and overflow records for the WWTP, CSO facility and pumping stations are useful and will be used to verify the calibrated model performance.
- The quality of the flow monitoring data from the 2017 Flow Monitoring Study and the Aldborough/Leger and Woodworth Inflow and Infiltration Studies is good and can be used to calibrate much of the wet weather model component.
- Additional flow monitoring locations are proposed for the northwest (upstream of the St. George Street pumping station), the northeast (upstream of the Burwell Road and Confederation Drive pumping stations) and east (Wellington and Elm Streets) sections of the collection system. These locations recommendations are preliminary and are intended to increase the resolution of the hydraulic model in areas where past monitoring and inflow and infiltration work has not been performed. We will discuss these sites with City staff to identify potential areas of concerns that should be added/addressed and to acknowledge the impacts on the project schedule.

APPENDIX 5

Flowmetrix Real Time Flow Monitoring Services Monthly Report, December 2020

SCG

FLOWMETRIX | METCON | PROCESS

REAL TIME FLOW MONITORING SERVICES MONTHLY REPORT

PREPARED FOR:

CITY OF ST. THOMAS

DECEMBER 2020

December 2, 2020

Nathan Bokma, P.Eng.
Manager of Development and Compliance
The City of St. Thomas
545 Talbot St, P.O. Box 520
St. Thomas, ON N5P 3V7
T: 519-631-1680 ext. 4151
E: nbokma@stthomas.ca

RE: City of St. Thomas
Real-Time Sewer Flow Monitoring Services – Monthly Report #1

Dear Mr. Bokma,

Please accept the submission of the first monthly report for this sewer flow monitoring project. The following report includes a description of the current status of the flow monitoring program and an update on the data quality of every site.

I trust that all requirements are met in the attached report. If you have any questions or concerns regarding the attached documents, please let me know at your earliest convenience.

Sincerely,

NCarlone

Natalie Carlone, EIT
Junior Project Manager
T: (226) 213-7274
ncarlone@flowmetrix.ca

1. Flow Monitoring

1.1 Site Location

The seven flow monitoring site locations and related site information are presented below in Table 1.

Table 1 Flow Monitoring Site Information

Site Name	Manhole	Pipe Size (mm)	Material	Install	Location
Thomas-1	MH1378	250	PVC	Inlet	St. George Street Pumping Station
Thomas-2	MH1989	600	Concrete	Inlet	Burwell Road Pumping Station
Thomas-3a	MH1521	200	Clay	Inlet	Confederation Drive Pumping Station
Thomas-3b	MH1519	350	Concrete	Inlet	Confederation Drive Pumping Station
Thomas-4a	MH570	380	Clay	Inlet	Mary Street East and Wellington Street
Thomas-4b	MH551	375	Clay	Inlet	Mary Street East and Wellington Street
Thomas-5	MH1920	450	Clay	Inlet	179 University Ave E

2.2 Site Assessment and Installation

The City of St Thomas provided 7 site locations for assessment by Flowmetrix prior to the installation of the flow monitoring equipment. The sites were assessed based on hydraulic suitability and the condition of the infrastructure. Flowmetrix has successfully installed monitoring equipment in 7 site locations and 1 rain gauge on October 7th, 2020. Refer to Appendix A for Installation Reports.

Thomas 1: Target manhole, M1689, had a drop pipe, so equipment could not be installed here. The meter was then installed one manhole upstream, at MH 1378.

Thomas 2: The outlet of the target manhole, MH 1987, was calcified and turbulent. The meter was installed one manhole downstream at MH1989 in the parking lot.

Thomas 3a: The outlet of the target manhole, MH1521, is turbulent as the flow drops in from the inlet behind the sensor, so the meter was installed at the inlet

Thomas 3b: Installed at the target manhole, MH1519.

Thomas 4a: Target manhole, MH560, was almost in surcharge conditions, so it was not possible to perform an installation. The meter was installed one manhole downstream at MH 570.

Thomas 4b: Installed in target manhole, MH551.

Thomas 5: Target manhole, MH1857, has very poor flow conditions, so the meter was installed one manhole upstream at MH1920 on Elm St. East of the target as the target flow conditions are very bad. Possible manual winch as rungs are badly deteriorated.

St Thomas Rain Gauge: The rain gauge was installed on the roof of the main office building of the treatment plant, at the south east corner.

2. Data Analysis

Data is uploaded via telemetry, or manually to the online RDA software. Data analysts could view the raw data collected by the meter and examine its integrity. Flowmetrix analysts review both site verification records and comments provided during each visit. This technique would allow the analysts to identify any inconsistencies in the data collected by the monitor, and flag it for further investigation.

2.1 Data QA/QC

Flowmetrix data analysts reviewed the data daily and were responsible for issuing work orders if the site required service, or to schedule regular maintenance. The site was flagged for observations such as:

- No Telemetry Communication (i.e. no new data)
- Low Battery Voltage
- Depth Sensor Comparison
- Velocity Sensor Functionality
- Change in Typical Trend
- Response to Rain Events

Refer to Appendix B for Site Confirmation Reports.

2.2 Data Quality

The sites experienced good data quality since installation.

Table 2 Overall Data Quality

Site ID	Overall Condition	Comments
Thomas-1	Good	Pump influenced
Thomas-2	Good	
Thomas-3a	Good	Low flow
Thomas-3b	Good	
Thomas-4a	Good	
Thomas-4b	Good	
Thomas-5	Good	

APPENDIX A

Installation Reports

Installation Report for Site:
Thomas-1

Site Basic Information

Status	Installed	Location	St. George Street Pumping Station	Asset ID	MH 1378
Installation Target	Inlet			Atmospheric Hazard	None
Date/Time	2020-10-07 10:11AM	Access Detail	On road besdie bridge	MH Chamber Condition	Good
Weather	Sunny / Warm				
		Traffic Plan	TL-19	Rim To Invert	3320

Site Vicinity Photo

MH Top View Photo

Site Assessment Information

Pipe Height	250.00	Pipe Shape	Circular	Range (AirDOF)	
Pipe Width	250.00	Flow Condition	Laminar	Velocity	1.84
Pipe Material	PVC	Flow Depth (Wet)	66.00	Silt Depth	0

Outlet Photo

Inlet Photo

Additional Photo 1

Site Installation Information

Meter Type	Triton +	Sensor Type 1|2	Peak		Comparison	Manual	Meter	+/-
Meter S/N	20293	Sensor S/N 1|2	57169		Depth of Flow	67.00	63.25	3.75
SIM IP Address	*49194 / 10.250.2.244	Physical Offset 1|2	0		Peak Velocity	1.77	1.84	0.07

Flow Meter Installation Photo

Sensor Installation Photo

Additional Photo 2

Installation Report for Site:
Thomas-1

Additional Information / Comment(s)	
Date	**Comment**
07-Oct-20	Site: 40.78.66.187 Username: ftpflowmetrix_profi Password: 311AdnLy# flow makes a slight curve between the inlet and outlet making inlet more suitable.

Installation Report for Site:
Thomas-2

Site Basic Information					
Status	Installed	Location	Burwell Road Pumping Station	Asset ID	MH 1989
Installation Target	Inlet			Atmospheric Hazard	None
Date/Time	2020-10-07 11:07AM	Access Detail	In parking lot	MH Chamber Condition	Good
Weather	Warm / Sunny				
		Traffic Plan	Pedestrian controls	Rim To Invert	6850

Site Vicinity Photo

MH Top View Photo

Site Assessment Information					
Pipe Height	600.00	Pipe Shape	Circular	Range (AirDOF)	
Pipe Width	600.00	Flow Condition	Laminar	Velocity	
Pipe Material	Concrete	Flow Depth (Wet)	68.00	Silt Depth	0

Outlet Photo

Inlet Photo

Additional Photo 1

Site Installation Information								
Meter Type	Triton +	Sensor Type 1|2	Peak		Comparison	Manual	Meter	+/-
Meter S/N	44183	Sensor S/N 1|2	57259		Depth of Flow	78.67	78.67	0.00
SIM IP Address	*47114 / 10.250.2.188	Physical Offset 1|2	0		Peak Velocity	0.80	0.98	0.18

Flow Meter Installation Photo

Sensor Installation Photo

Additional Photo 2

Installation Report for Site:
Thomas-2

Additional Information / Comment(s)	
Date	**Comment**
07-Oct-20	

Installation Report for Site:
Thomas-3a

Site Basic Information

Status	Installed	Location	Confederation Drive Pumping Station	Asset ID	MH 1519
Installation Target	Inlet			Atmospheric Hazard	None
Date/Time	2020-10-07 12:30PM	Access Detail	On road infront of pump station	MH Chamber Condition	Good
Weather	Warm / Sunny				
		Traffic Plan	Tl-19	Rim To Invert	4000

Site Vicinity Photo

MH Top View Photo

Site Assessment Information

Pipe Height	200.00	Pipe Shape	Circular	Range (AirDOF)	
Pipe Width	200.00	Flow Condition	Laminar	Velocity	
Pipe Material	Clay	Flow Depth (Wet)	15.00	Silt Depth	0

Outlet Photo

Inlet Photo

Additional Photo 1

Site Installation Information

Meter Type	Triton +	Sensor Type 1|2	Peak		Comparison	Manual	Meter	+/-
Meter S/N	40856	Sensor S/N 1|2	58954		Depth of Flow	15.33	36.00	20.67
SIM IP Address	*06989 /	Physical Offset 1|2	0		Peak Velocity	0.00	0.00	0.00

Flow Meter Installation Photo

Sensor Installation Photo

Additional Photo 2

Installation Report for Site:
Thomas-3a

Additional Information / Comment(s)	
Date	**Comment**
07-Oct-20	

Installation Report for Site:
Thomas-3b

Site Basic Information					
Status	Installed	Location	Confederation Drive Pumping Station	Asset ID	MH 1521
Installation Target	Inlet			Atmospheric Hazard	None
Date/Time	2020-10-07 1:22PM	Access Detail	On Road in front of pump station	MH Chamber Condition	Good
Weather	Warm / Sunny				
		Traffic Plan	Tl-19	Rim To Invert	5240

Site Vicinity Photo

MH Top View Photo

Site Assessment Information					
Pipe Height	350.00	Pipe Shape	Circular	Range (AirDOF)	
Pipe Width	350.00	Flow Condition	Laminar	Velocity	0.65
Pipe Material	Concrete	Flow Depth (Wet)	50.00	Silt Depth	0

Outlet Photo

Inlet Photo

Additional Photo 1

Site Installation Information								
Meter Type	Triton +	Sensor Type	1|2	Peak	Comparison	Manual	Meter	+/-
Meter S/N	50308	Sensor S/N	1|2	23355	Depth of Flow	69.00	75.00	6.00
SIM IP Address	*49434 / 10.250.3.70	Physical Offset	1|2	0	Peak Velocity	0.55	0.66	0.10

Flow Meter Installation Photo

Sensor Installation Photo

Additional Photo 2

Installation Report for Site:
Thomas-3b

Additional Information / Comment(s)	
Date	**Comment**
07-Oct-20	

Installation Report for Site:
Thomas-4a

Site Basic Information					
Status	Installed	Location	Mary Street East and Wellington Street	Asset ID	MH 570
Installation Target	Inlet			Atmospheric Hazard	None
Date/Time	2020-10-08 8:29AM	Access Detail	On road in turning lane	MH Chamber Condition	Good but Lid is cracked - be
Weather	Warm / Sunny				
		Traffic Plan		Rim To Invert	3660

Site Vicinity Photo

MH Top View Photo

Site Assessment Information					
Pipe Height	380.00	Pipe Shape	Circular	Range (AirDOF)	
Pipe Width	380.00	Flow Condition	Laminar	Velocity	
Pipe Material	Clay	Flow Depth (Wet)	60.00	Silt Depth	0

Outlet Photo

Inlet Photo

Additional Photo 1

Site Installation Information								
Meter Type	Triton +	Sensor Type 1|2	Peak		Comparison	Manual	Meter	+/-
Meter S/N	51834	Sensor S/N 1|2	24767		Depth of Flow	59.00	59.67	0.67
SIM IP Address	*49731 / 10.250.3.15	Physical Offset 1|2	0		Peak Velocity	0.00	0.00	0.00

Flow Meter Installation Photo

Sensor Installation Photo

Additional Photo 2

Installation Report for Site:
Thomas-4a

Additional Information / Comment(s)	
Date	**Comment**
08-Oct-20	City has wrong lid installed, it is a storm lid that could produce high levels of infiltration and it is also cracked trhough presenting a safety risk.

Installation Report for Site:
Thomas-4b

Site Basic Information					
Status	Installed	Location	Mary Street East and Wellington Street	Asset ID	MH 551
Installation Target	Inlet			Atmospheric Hazard	None
Date/Time	2020-10-07 2:39PM	Access Detail	On Road in front of fire station	MH Chamber Condition	Good
Weather	Warm / Sunny				
		Traffic Plan		Rim To Invert	3405

Site Vicinity Photo

MH Top View Photo

Site Assessment Information					
Pipe Height	375.00	Pipe Shape	Circular	Range (AirDOF)	
Pipe Width	375.00	Flow Condition	Laminar	Velocity	
Pipe Material	Clay	Flow Depth (Wet)	59.00	Silt Depth	0

Outlet Photo

Inlet Photo

Additional Photo 1

Site Installation Information								
Meter Type	Triton +	Sensor Type 1|2	Peak		Comparison	Manual	Meter	+/-
Meter S/N	52606	Sensor S/N 1|2	57439		Depth of Flow	75.33	78.00	2.67
SIM IP Address	*49392 / 10.250.2.155	Physical Offset 1|2	0		Peak Velocity	1.40	1.26	0.14

Flow Meter Installation Photo

Sensor Installation Photo

Additional Photo 2

Installation Report for Site:
Thomas-4b

Additional Information / Comment(s)	
Date	**Comment**
07-Oct-20	

Installation Report for Site:
Thomas-5

Site Basic Information

Status	Installed	Location	Elm Street	Asset ID	MH 1920
Installation Target	Outlet			Atmospheric Hazard	None
Date/Time	2020-10-07 3:48PM	Access Detail	On Road in "T" Intersection	MH Chamber Condition	Good few bad rungs
Weather	Hot / Sunny				
		Traffic Plan		Rim To Invert	

Site Vicinity Photo

MH Top View Photo

Site Assessment Information

Pipe Height	450.00	Pipe Shape	Circular	Range (AirDOF)	
Pipe Width	450.00	Flow Condition	Lamianr	Velocity	
Pipe Material	Clay	Flow Depth (Wet)		Silt Depth	0

Outlet Photo

Inlet Photo

Additional Photo 1

Site Installation Information

Meter Type	Triton +	Sensor Type	1|2	Peak	Comparison	Manual	Meter	+/-
Meter S/N	64687	Sensor S/N	1|2	43821	Depth of Flow	80.67	73.00	7.67
SIM IP Address	*17907 / 10.250.0.249	Physical Offset	1|2	0	Peak Velocity	1.14	1.36	0.22

Flow Meter Installation Photo

Sensor Installation Photo

Additional Photo 2

Installation Report for Site:
Thomas-5

Additional Information / Comment(s)	
Date	**Comment**
07-Oct-20	

APPENDIX B

Confirmation Reports

Site ID	Work Type	Date	Status	Time	Level Verifications (mm)			Velocity Verifications (m/s)		Silt Level (mm)	Comments
					Manual	PDepth	UpDepth	Manual	Peak		
Thomas-1	Installation	2020-10-07	Completed	10:11	67	65	63	1.77	1.84	0	Flow makes a slight curve between the inlet and outlet making inlet more suitable.
	Maintenance	2020-11-05	Completed	10:07	53	45	55			0	Manual measurements match meter.
	Maintenance	2020-11-19	Completed	16:49	53	48	53			0	Manual measurements match meter.
Thomas-2	Installation	2020-10-07	Completed	11:07	79	69.00	79.00	0.8	0.98	0	
	Maintenance	2020-11-05	Completed	11:01	81	77.00	78.00	0.8	0.92	0	
	Maintenance	2020-11-19	Completed	14:40	85	81.00	88.00	0.86	0.96	0	Sensor was cleaned. manual measurement match meter. Flow is wavy.
Thomas-3a	Installation	2020-10-07	Completed	12:30	15	14.00	36.00			0	
	Maintenance	2020-11-05	Completed	11:29	30	10.00	33.00			0	Flow pooling around sensor. Manual measurements match meter.
	Maintenance	2020-11-19	Completed	15:29	17	15.00	35.00			0	Due to high velocity, there is a wave over the sensor. Manual measurements match Pdepth. Also, data was collected and fixed the telemetry issues.
Thomas-3b	Installation	2020-10-07	Completed	13:22	69	71.00	75.00			0	
	Maintenance	2020-11-05	Completed	11:54	74	79.00	78.00			0	Manual measurements match meter.
	Maintenance	2020-11-19	Completed	16:04	73	75.00	75.00			0	Sensor was cleaned. manual measurement match meter. Also, data was collected and fixed the telemetry issues.
Thomas-4a	Installation	2020-10-07	Completed	8:29	59	121.00	60.00			0	It is a storm lid that could produce high levels of infiltration and it is also cracked through presenting a safety risk.
	Maintenance	2020-11-05	Completed	12:20	56	59.00	59.00			0	Manual measurements match meter. Lid is a storm lid allowing for a lot of infiltration. Site is emitting an exceptionally strong chemical smell.
	Maintenance	2020-11-19	Completed	13:53	61	63.00	62.00	0.5	0.53	0	Sensor was cleaned. Manual measurement match meter.
Thomas-4b	Installation	2020-10-07	Completed	14:39	75	78.00	78.00	1.4	1.26	0	
	Maintenance	2020-11-05	Completed	12:47	64	63.00	61.00			0	
	Maintenance	2020-11-19	Completed	13:17	62	65.00	60.00	1.2	1.19	0	
Thomas-5	Installation	2020-10-07	Completed	15:48	81	83.00	73.00	1.14	1.36	0	Flow makes a slight curve between the inlet and outlet making inlet more suitable.
	Maintenance	2020-11-05	Completed	13:24	56	48.00	57.00			0	Manual measurements match meter.
	Maintenance	2020-11-19	Completed	12:17	61	60.00	57.00	1.19	1.26	0	Manual measurements match meter.

APPENDIX 6

Technical Memorandum# 4/5 – Sanitary Model Build and Hydraulic Results Review and Proposed Solutions

R.V. Anderson Associates Limited
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St. Thomas – Pollution Prevention Control Plan

Technical Memorandum# 4/5 – Sanitary Model Build and Hydraulic Results Review and Proposed Solutions

Prepared for:
City of St Thomas

RVA 205153

January 14, 2022

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APPENDICES

1.0 INTRODUCTION

The City of St. Thomas has retained R.V. Anderson Associates Limited (RVA) for the preparation of a Pollution Prevention Control Plan (PPCP). The PPCP is a part of the City's ongoing efforts to improve the performance of their sanitary and storm sewer infrastructure. In addition, it would provide the City with a road map for implementation of infrastructure and operational improvements that will mitigate the impacts of wet weather sewer system overflows on the environment and help the City mitigate risk in alignment with the City's commitment to environmental stewardship and the provision of sustainable municipal services.

This Technical Memorandum (TM) is one of a series of technical memoranda with previously submitted TM's being:

1. TM#1 – Existing Document Review and Summary; and
2. TM#2/3 – Flow Monitoring and Hydraulic Modelling Data Gap Analysis.

This report combines what was originally intended to be two separate technical memoranda, the first detailing the sewer model build process and the assessment of the modeling results (Tech Memo #4) and the review and development of solutions to address issues found in the PPCP study to date (Tech Memo #5).

2.0 MODEL BUILD

2.1 Asset Data Input

RVA received from the City of St Thomas the GIS shapefiles which were imported in InfoWorks to build the model. Since the City's sanitary sewer system includes some areas outside the City's boundaries, the City also provided the GIS shapefiles of the relevant assets from the Municipality of Central Elgin.

The geographic and projected coordinate systems used to develop the model are listed below:

- Projected Coordinate System: NAD_1983_UTM_Zone_17N
- Geographic Coordinate System: GCS_North_American_1983

The GIS Shapefiles which were used to build the model are described below:

2.1.1 Sewers and Manholes

The sewer and manhole shapefiles were generally divided into three (3) types according to function—sanitary, combined, and storm. For this analysis, only the sanitary and combined sewer and manhole shapefiles were used to build the model. The storm assets were used as reference when needed (i.e., when overflows were involved). There were approximately 2900 sewers and 2800 manholes which comprised the model.

Before the GIS data were imported, various solutions were used to fill the data gaps. These solutions included the following:

- Removing duplicated asset IDs;
- Renaming assets with no IDs;
- Updating upstream and downstream manhole IDs of sewers;
- Adding manholes at sewers with missing manhole;
- Removing orphaned manholes; and
- Splitting pipes which need to be split at a manhole.

2.1.2 Elevation

Another important input in the model is the elevation. There were different types of elevation data which were required during model build—the ground elevation for manholes and invert elevation for sewers.

The provided lidar data was used to interpolate the ground elevation of the manholes. On the other hand, the invert elevations at the upstream and downstream ends of sewers can only be obtained from the attributes of the received GIS sewer data. It was observed that not all sewers have upstream and downstream invert elevations. Using engineering judgment as well as the model interference function to interpolate invert elevations between known points, these elevations were estimated by observing the neighboring lines, ground profile elevation from lidar, and layout of the sewer network. Where end of sewer network branches was missing sewer invert elevations without an end reference point, the sewer inverts were set at a minimum of two (2) meter depths.

2.1.3 Pumping Stations

There were 16 sanitary pumping stations (SPS) considered in this study as listed below:

1. Axford SPS;
2. Burwell Rd. SPS;
3. Confederation Dr. SPS;
4. Crescent Ave. SPS;
5. Elm St SPS;
6. Harper Rd SPS;
7. Karen St SPS;
8. Lyndhurst SPS;
9. Parkside Drive SPS;
10. Shaw Valley Dr SPS;
11. St. George SPS;
12. Sunset Dr. SPS;
13. Talbot Line SPS;
14. Wolfe (Hughes St) SPS;
15. Woodland SPS; and
16. Woodworth SPS.

The pumping stations were also imported as a background in InfoWorks to accurately locate the nodes to represent these pumps. Further information was obtained from the Operation and Manual files of the stations to determine the wet well sizes, pump types and to derive the pump performance curves, the switch on and switch off levels and the emergency overflow elevation where available.

Appendix 1 shows the pump details listed.

2.2 Catchment Delineation

There were two methods of delineation which were used in this analysis—the area based, and the parcel based sub-catchment delineations. The delineation process to represent the catchments are described below.

The figure below shows an example of the Infiltration and Inflow strip that was generated around each sewer for Wet weather flow calibration purposes.

2.2.1 Area Based Catchment Delineation

To represent the inflow and infiltration (I&I) in the sanitary sewer system, area-based delineation around the sewer lines were established. The area-based catchment was based on a 45 m strip drawn from each sewer line as illustrated in Figure 2-1. Each manhole was setup to have its own I&I strip which was bounded at equal distances between the manholes. This process was completed in GIS environment using a series of ArcGIS Tools.

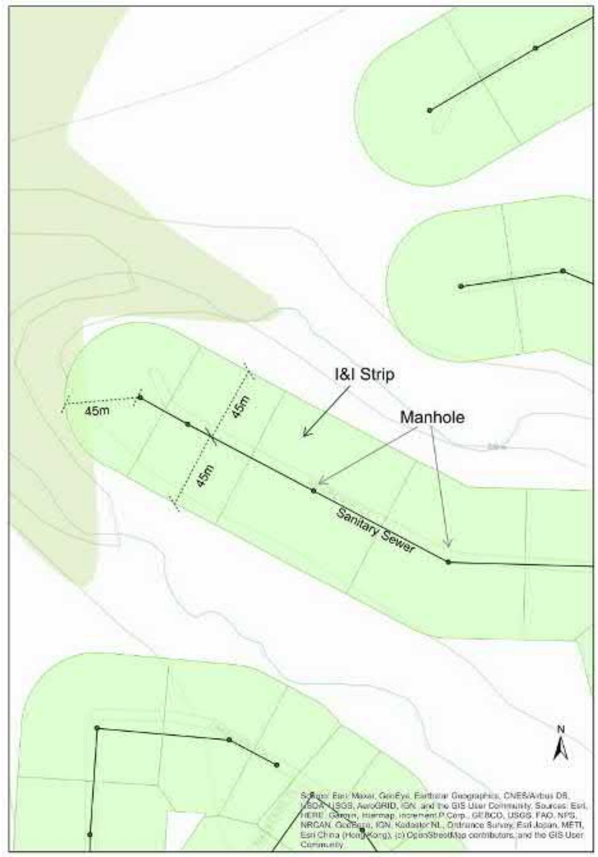


Figure 2-1 - Area Based Catchment Delineation Example

2.2.2 Parcel Based Catchment Delineation

To represent the wastewater inflow to the sewers, the catchments were based on and aligned to the connected property parcels. The property parcels included areas from the City of St. Thomas and from the Municipality of Central Elgin. The parcels were carefully analyzed to include only the parcels which were required in the sanitary sewer system. This process included removing parcels outside the boundary of the sewer system, parcels representing paths, and empty lots.

The received property parcels included an attribute called “Type_MPAC” which were used to check the land use types so that the correct population density per address unit (PPU) could be assigned. Where data was not available, research efforts were made (i.e., searching at Google Maps Imagery) to check the number of units and make best guess efforts to assign a land use type.

Population density was then assigned to each residential property parcel according to these land use types. This was based on the Ten (10)-year growth forecast from late 2019 to late 2029 (refer to St. Thomas 2019 Development Charges Study), the population density of 2.527 Persons Per Unit (PPU) was generally assigned to most residential properties; and 1.680 PPU was assigned to multiple residential units.

For non-residential property parcels, the industrial-commercial-institutional (ICI) flows were used based on the water consumption data for year 2019 and 2020.

Address points were created for reference to represent the property parcels at their centroids. The property parcels (residential and non-residential) were then grouped according to the nearest manhole and were combined to comprise the parcel-based catchments—each with corresponding nearest manhole. Figure 2-2 illustrates parcel-based catchments in this study.

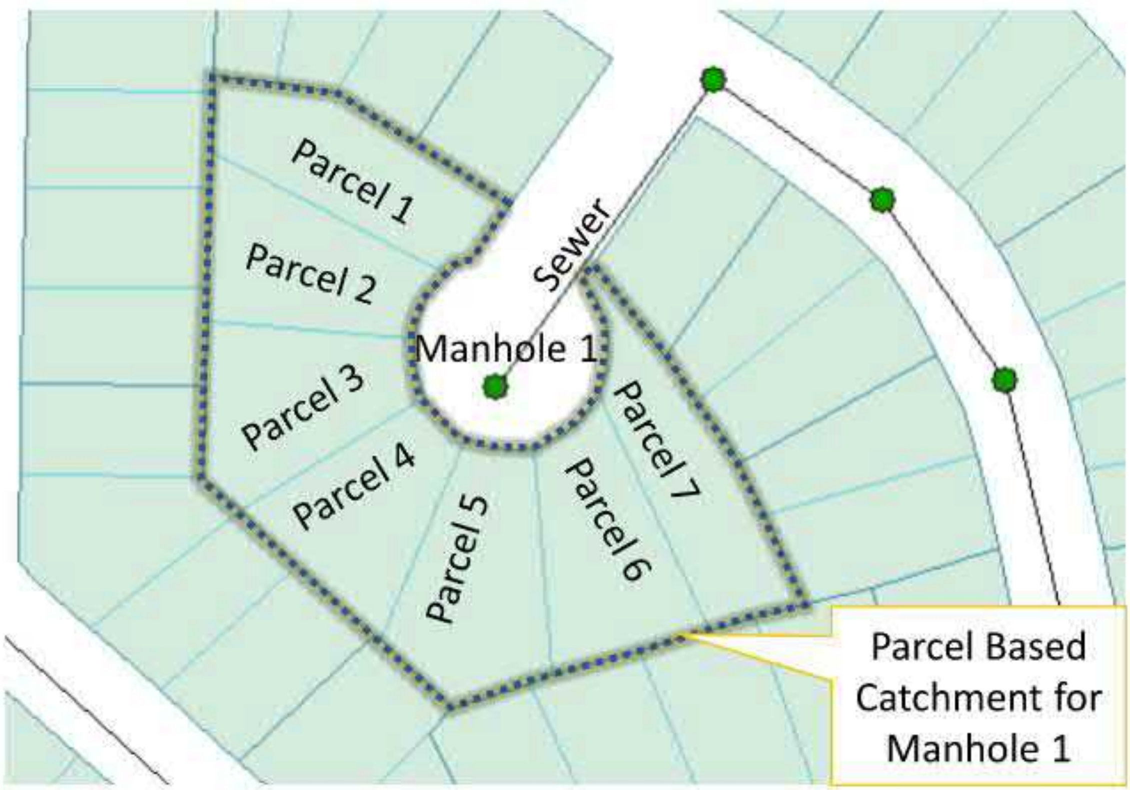


Figure 2-2 - Parcel-Based Catchment Delineation Example

2.3 Design Rainfall

Design rainfall profiles were developed from Environment Canada Intensity Depth Frequency curves (IDF) that was derived from monitoring station ST Thomas WPCP (Station ID 6137362). Historical data cover a very long period of 82 years from 1926 to 2016. From the extracted IDF coefficients the rainfall intensities for 4-, 6- and 12-hour duration storms with return periods of 2-years, 5-years, 10-years, 25-years, 50-years and 100-years have been developed. The design rainfall pattern was compiled according to the Chicago type storm distribution and a time to peak ratio of 40% (r=0.4) was used. The rainfall profiles were generated for 5-minute timesteps to match the flow survey timesteps.

The design rainfalls with different durations were tested for worst case flow response and results showed that the 12-hour duration storms would produce the highest peak flows and volumes for the sewer network and flows to the CSO tank and treatment plant. The design storm range was therefore run further with 12-hour duration storms.

Appendix 2 shows the detailed rainfall profiles and IDF station data that was collected.

2.3.1 Climate Change Considerations

To assess potential impacts of climate change to the sewer network, the University of Western Ontario’s (UWO) IDF CC online tool was used. This website links Environment Canada’s IDF weather station locations and produces comparable IDF parameter tables between historic data and future climate change IDF parameters. RVA selected a future period for the years 2050 to 2100 and reviewed the three main climate change projections RCP 2.6, RCP4.5 and RCP8.5. It was determined that for the location of St. Thomas, the RCP8.5 scenario (unmitigated growth) would produce the highest increase in precipitation and this scenario was then used for comparison with the historic IDF curves.

As a result of this comparison, it can be concluded that under the worst-case climate change scenario, precipitation is expected to increase between 22% to 27% starting from a 2-year return period storm towards a 100-year return period storm respectively.

Details for this Climate Change assessment are contained within Appendix 3.

2.4 Rainfall Series (Typical Year)

To test for longer term network performance and allow the quantification of CSO spills and their analysis, a real measured rainfall series was chosen to be used as the baseline performance criteria. Typically, a long-term serial simulation should cover a period of 10 years or longer to be able to statistically evaluate impacts. However, since historical data are not available in the required timestep resolution, a different approach was chosen.

The City of Toronto has a large permanent rain gauge network installed with over 16 rain gauges distributed over the City. According to their evaluation, the year 1991 represented a typical rainfall year that shows a good rainfall distribution with a variety of storms that test sewer network performance. Rainfall events starting on April 1st until October 27th of that year were recorded and have been added into a serial simulation.

RVA agreed with the City of St. Thomas to use this data as a basis and modify the data to suit the geographic location of St. Thomas. For this purpose, four (4) historic Toronto IDF stations were analyzed and compared against the St. Thomas WPCP station for 2-year return period storms from 5minute duration to 24 hours duration storms. An average adjustment factor was established that was then applied to the original 1991 Toronto rainfall year series to adjust rainfall intensities for St. Thomas. The adjustment factor was found to be 0.941 when compared to the City of Toronto. The table below shows the comparison summary of this assessment. Further detail can be found in Appendix 4.

Table 2-1 – IDF Conversion Factor Analysis, City of Toronto to St. Thomas

Multiplying Factor Calculation											
	Station ID	Toronto Old Weston Road (ID: 6158764)	Toronto City (ID: 6158355)	Toronto Island (ID: 6158665)	Toronto Booth (ID: 6158406)	Average	St. Thomas WPCP (ID: 6137362)	Change (mm)	Change Fraction	Change Factor
	Event	2-Year Precipitation (mm)					2-Year Precipitation (mm)			
Duration	5 min	8.52	8.65	8.07	8.61	**8.46**	8.21	-0.253	-0.031	1.031
	10 min	12.11	12.28	12.14	12.02	**12.14**	12.29	0.153	0.012	0.988
	15 min	14.8	14.63	15.12	14.36	**14.73**	14.85	0.123	0.008	0.992
	30 min	19.74	18.98	19.62	18.17	**19.13**	19.96	0.833	0.042	0.958
	1 h	24.03	23.73	24.4	20.96	**23.28**	25.85	2.570	0.099	0.901
	2 h	28.2	27.61	28.74	25.65	**27.55**	30.01	2.460	0.082	0.918
	6 h	35.49	33.62	35.74	33.72	**34.64**	38.1	3.458	0.091	0.909
	12 h	38.42	40.22	39.7	38.52	**39.22**	44.19	4.975	0.113	0.887
	24 h	41.84	45.63	44.19	44.26	**43.98**	49.7	5.720	0.115	0.885
Average Multiplying Factor:										**0.941**

2.5 Flow Monitoring Data

For model adjustment and calibration purposes, historic flow monitoring data (2017) and recent (2020) added flow monitoring data as identified in TM#2/3 from the sewer network were analyzed and used to define wastewater diurnal profiles in the model.

There are eight (8) flow monitoring locations with 2017 data results available and seven (7) flow monitoring locations that were measured in 2020. Figure x below shows the locations of the flow monitors.

2017 flow monitoring data were available for a period of three (3) months from August 16/17th, 2017 to November 23, 2017. No rain gauge information was available with the flow monitoring data. The Environment Canada rainfall information that is available for the ST. Thomas WPCP can be reviewed in daily or monthly timesteps but would require a finer resolution to be useful for wet weather flow calibration. For this reason, the f2017 flow data will only be used for dry weather flow calibration.

The 2020 flow monitoring data were available for a period of under three (3) months from October 7th, 2020, to December 29th, 2020. A rain gauge was installed with the flow meters and rainfall data is available in 5-minute timesteps.

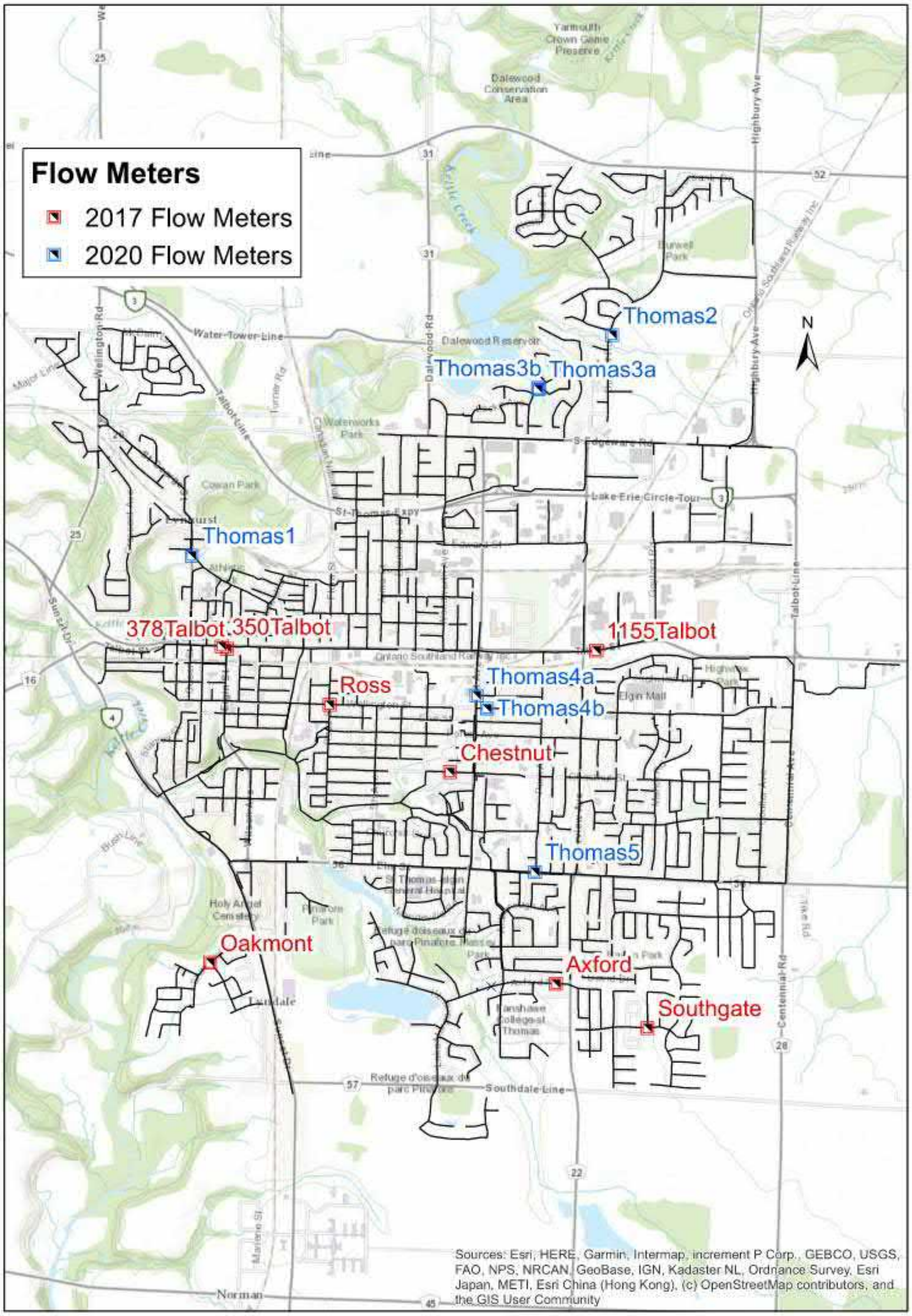


Figure 2-3 Flow Meter Locations

2.5.1 Dry Weather Flow Patterns

RVA analyzed the diurnal profiles for weekday patterns and weekend patterns whilst considering the upstream flow monitor residential population and calculated/ estimated trade flows that were based on metered water consumption records. The groundwater infiltration (GWI) was estimated based on minimum observed nighttime flows where 80%-85% of that minimum flow in the observed weekend profile was attributed to GWI. Weekend profiles were assumed to have no trade flows occurring, whilst weekday trade profiles were adjusted to show a block discharge pattern from 7:00 AM to typically 18:00 PM.

Because of the addition of trade flows during weekdays, the per capita flow rate in the weekday diurnal profiles is slightly lower than for the weekend profiles. The hydraulic model uses only one per capita flow number, and the weekday per capita flow number was used in the model. For that reason, the diurnal weekend flow pattern was adjusted to the lower per capita and day flow numbers from weekday profiles to be input into the model. Residential wastewater flows for the weekend profiles are slightly higher than during weekdays.

The following tables summarize key flow data that were extracted from the flow monitors:

Table 2-2 – In 2017 Installed Flow Monitors- Key Data Analysis

Flow Monitor Name	Measured Population	Catchment Size [ha] * Note 1	Groundwater Inflow (GWI) [l/s/ha]	Per Capita flow [L/c/d] *Note 2	Notes/ Comments
350 Talbot	1,375	14.85	0.13	183.0	
378 Talbot	2,106	31.1	0.097	262.0	Location is D/S of 350 Talbot
378 Talbot – 350 Talbot	731	16.25	0.16	352.6	
Ross	735	24.76	0.019	50.29	Flows are too small to accurately measure monitor results are outside of good error range- will not be used
Chestnut	13,625	543.03	0.048	180.4	Covers a large catchment area and will be used for calibration where the 2020 flow monitors Thomas 4a, Thomas 4b, Thomas 3a, Thomas 3B and Thomas 2 will not cover or have unreliable data.
Oakmont	338	10.17	0.021	193.7	
Axford (2)	2,830	59.6	0.024	114.15	
1155 Talbot	10	54.22	0.042	N/A	Low population and industrial use land- high trade flow and water consumption would require more investigation to establish correct flow pattern. FM discarded. Area covered by FM Chestnut.
Southgate	1,430	29.57	0.021	69.87	Flows are below expected values for the population. FM discarded for calibration. FM Axford will be used.

Note 1: Catchment size is based on a catchment of 45 m drawn to either side of the sewers to consistently measure the I&I influence zone without catchment area distortions.
Note 2: Per Capita Wastewater generation is based on weekday DWF profile. Weekend profiles are typically slightly (5-10%) higher due to missing trade flows and different usage patterns.

Table 2-3 – In 2020 Installed Flow Monitors- Key Data Analysis

Flow Monitor Name	Measured Population	Catchment Size [ha] * Note 1 (Parcel based catchment)	Groundwater Inflow (GWI) [l/s/ha]	Per Capita flow [L/c/d] *Note 2	Notes/ Comments
Thomas 1	2,388	105.35 (108)	0.019 (0.019)	267.3	
Thomas 2	3,153	101.3 (101)	0.092 (0.092)	253.45	
Thomas 3a	1,664	11.63	0.000065	32.4	Flows are too low (,1L/s) to measure accurately, FM site discarded for calibration
Thomas 3b	1,661	32.38 (33.7)	0.106 (0.102)	189.3	
Thomas 4a	839	33.67 (27.1)	0.077 (0.096)	376.4	
Thomas 4b	4,323	141.94 (169.9)	0.041 (0.034)	144.0	
Thomas 5	5,425	167.37 (139.7)	0.039 (0.047)	156.3	

Note 1: Catchment size is based on a catchment of 45 m drawn to either side of the sewers to consistently measure the I&I influence zone without catchment area distortions. (value in brackets shown is the true catchment area, based on parcel sizes)
Note 2: Per Capita Wastewater generation is based on weekday DWF profile. Weekend profiles are typically slightly (5-10%) higher due to missing trade flows and different usage patterns.

Appendix 5 shows the detailed diurnal curves that were produced for each flow monitor.

The above-described flow monitoring data was used to setup Wastewater flow patterns for residential flow in the model and to distribute the observed and calculated groundwater infiltration as baseflow. Some of the described flow monitoring results were not used for flow calibration due to problems with the data and obvious erroneous results displayed. The flow monitors that were not used are Ross, 1155Talbot, Southgate and Thomas 3a. Measured flows for most of these locations were too low to properly display meaningful patterns due to high error readings, scattered data plots and not credible usage patterns. FM 1155 Talbot monitored a mainly industrial area with close to no residential population. However, the trade flow pattern could not be properly established since the measured flow was lower than the water consumption records for the area for the years 2019 and 2020 show. Instead, the generic setup trade flow pattern was kept for that area with matching trade flows for the 2020 water consumption records.

As mentioned under section 2.2.2. for the Parcel Based Catchment Delineation, for non-residential property parcels, the industrial-commercial-institutional (ICI) flows were used based on the water consumption data for year 2019 and 2020. These flows were part of the overall monitored flows and have been discounted for.

Flow patterns for the 2017 flow monitors were input first because they covered a larger catchment area in the model and where an overlap of monitored catchments occurred from 2020 flow monitoring data, this flow pattern was corrected to match the 2020 flow monitoring data.

2.5.2 Wet Weather Flows

The raw flow monitor recordings from the 2017 period and the FlowMetrix Flow monitoring report from February 2018 did not contain rain gauge information or any rainfall data with the measured flows. Environment Canada operates a rain gauge station at the St. Thomas WPCP. However, the data available is for daily and monthly rainfall recordings only. Therefore, this flow data could only be checked for dry weather days and used for dry weather calibration.

The flow monitoring that was conducted in 2020 did have rainfall data available in the same 5-minute timestep as the flow monitor recordings.

During the slightly less than three (3) month monitoring period, approximately 26 rainfall events of varying intensity and duration were recorded. Rainfall data was analyzed to select the best representative rainfall events that would display intensity and duration as well as best possible prior dry days. Three rainfall events were selected that matched the criteria best.

The first event (Event 1) lasted from 2020-10-23 5:00PM to 2020-10-24 12:00AM, a duration of 19hrs and had a total precipitation of 9.0 mm. It showed one prior dry day. The second event (Event 2) lasted from 2020-11-15 12:00AM to 2020-11-15 2:00PM, a duration of 14 hrs and had a total precipitation of 19.5 mm. It showed three prior dry days and over 10 prior days without significant rainfall. Finally, the third event (Event 3) lasted from 2020-12-12 8:00AM to 2020-12-13 3:00AM, a duration of 19 hrs and had a total precipitation of 14.2 mm. It showed one prior dry day, but over seven prior days without significant rainfall.

The three described events were used to analyze the observed wet weather flows for the selected flow monitor locations and compare them against simulated model flows whilst calibrating the hydrologic runoff parameters as described in the following sections.

2.6 Hydraulic/ Hydrologic Parameters

2.6.1 Rainfall-Derived Infiltration and Inflow Parameter Calibration (RTK)

To model rainfall-derived infiltration and inflows (RDII) to the system, the Sanitary Sewer Overflow Analysis and Planning (SSOAP) Toolbox developed by the U.S. Environmental Protection Agency (EPA) was used.

The SSOAP toolbox uses the rainfall data to identify dry-weather flow days that can be used to determine the base wastewater flow and groundwater infiltration components of the total sewer flow, discriminating between weekdays, weekends, and holidays. Comparisons between the automatically computed dry weather hydrographs and the ones calculated analysing the observed time series in Excel showed acceptable agreement (See Figure below for Thomas 4a catchment for example).

a) b)

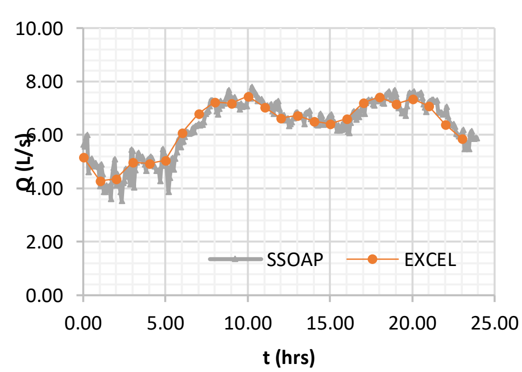


Figure 2-4 Comparison of Dry Weather Hydrographs for Catchment Thomas 4a (a. weekdays, b. weekends)

To simulate RDII flows, SSOAP uses the RTK unit hydrograph method, which uses three triangular unit hydrographs to describe the RDII flows. The R parameters represent the fraction of rainfall that enters the sewer system, T is the time to the peak for each unit hydrograph, and K is the ratio of the time of recession to the time of peak. The three hydrographs represent the system's fast response (R1, T1, K1), medium response (R2, T2, K2), and late response (R3, T3, K3), and they are added through convolution as shown in the figure below:

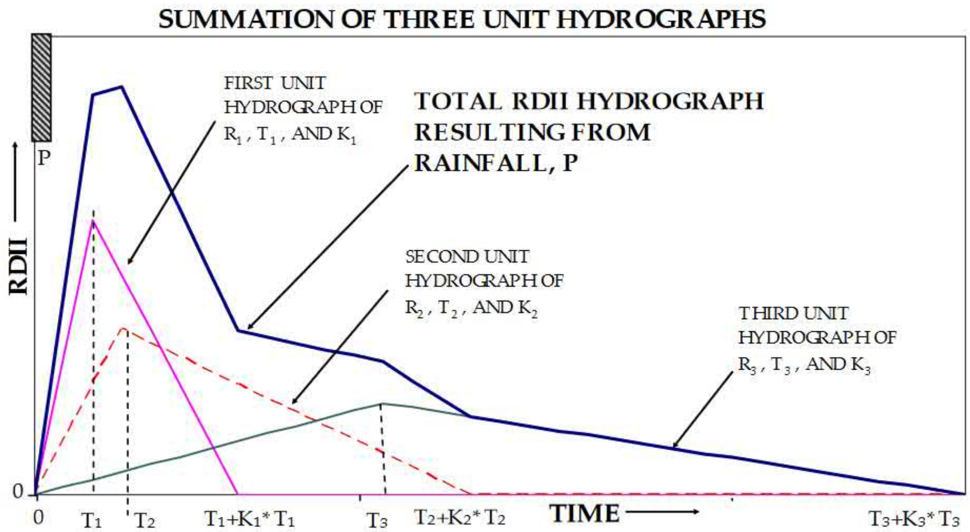


Figure 2-5 Schematic Representation of Unit Hydrograph Summation for the RTK Method

The RTK parameters were assigned through a manual calibration process for each of the metered catchments (Thomas 1, Thomas 2, Thomas 3b, Thomas 4a, Thomas 4b, and Thomas 5) for each of the three rainfall events (Event 1, Event 2, and Event 3). The SSOAP toolbox allows to view graphically the observed and simulated RDII time series, allowing the user to select the combination of parameters that best fits the two curves. An example of the observed-simulated curve adjustment is shown below:

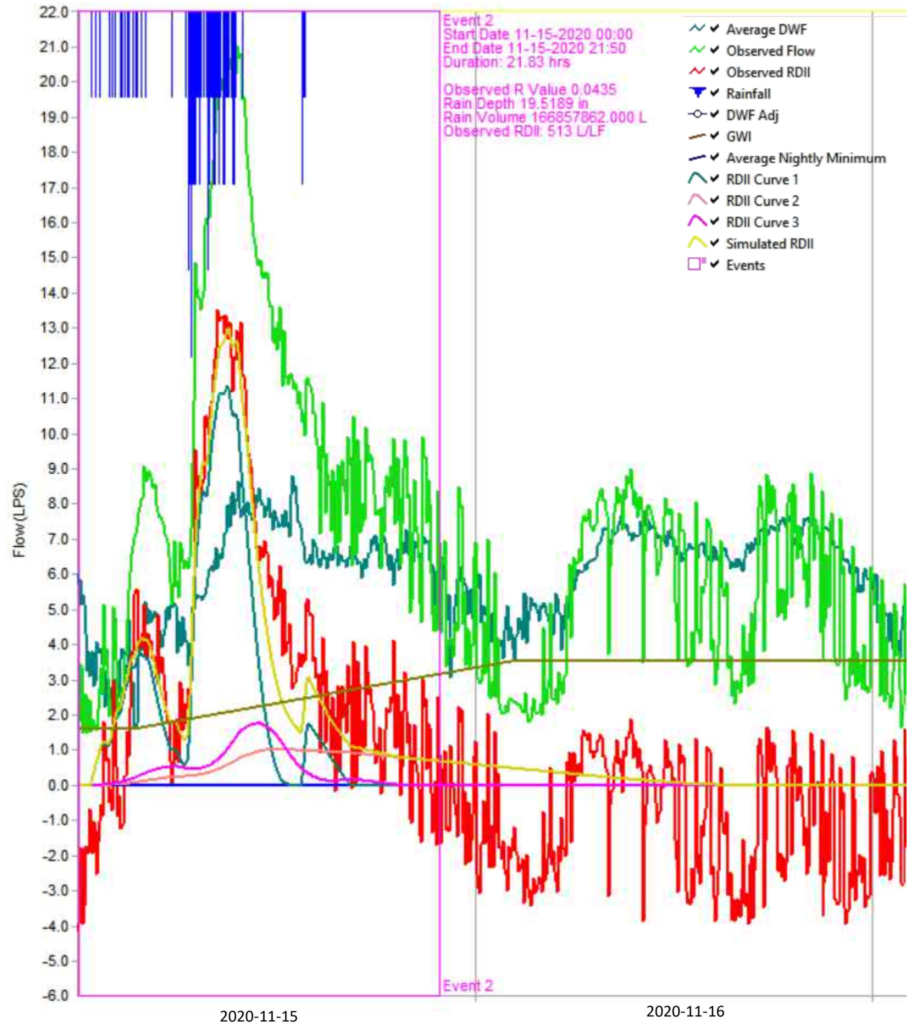


Figure 2-6 SSOAP Curve Fitting Example for Thomas 4a – Event 2

The final RTK parameters for metered catchments was computed as weighted averages of the parameters per event, to be included in the InfoWorks model. Table *2-4* shows the calibrated RTK values for the six metered catchments (Thomas 1 to 5).

In addition to the recently completed flow monitoring, Cole Engineering Ltd. completed an inflow and infiltration study for two catchments of the city in 2014. The report, named *Aldborough/Leger and Woodworth Wastewater Sanitary Catchments Inflow and Infiltration Study*, used a combination of calibrated computational models and statistical analysis to estimate the RDII flowrates for 15 subcatchments in the city. The Cole estimated RDII rates are for 1 in 20-year storm flows. RVA used these values to estimate RTK parameters for those subcatchments and downscale them to a level that would correspond to measured flows below a 1 in 2-year storm. A proportionality factor was computed for two study areas for which this present study and Cole Engineering's study overlapped. The factor related the RDII estimates with the RTK calibrated parameters, so an extrapolation for the unmetered areas covered by the Cole

Engineering's study could be done. Table *2-4* shows the estimated RTK values for the 13 subcatchments analyzed by Cole Engineering (not including the two overlapping areas).

For the remaining study area where no flow survey data exists, an estimate for RTK values had to be made. Since a correlation analysis showed no correlation between the calibrated parameters and the parcel fabric and characteristics of the catchments, a conservative approach using simple average values plus one standard deviation over metered catchments was used to assign RTK values for the other unmetered catchments (see Table 2-4). Appendix 6 shows validation results from using the computed RTK parameters in the whole system.

Table 2-4 - Calibrated RTK Parameters

	Area (ha)	Length of Sewers (m)	R1	T1	K1	R2	T2	K2	R3	T3	K3
Thomas 1	105.4	14626	0.0072	0.6	9.0	0.0061	3.0	9.0	0.0000	-	-
Thomas 2	101.3	15683	0.0099	0.6	9.0	0.0124	3.0	9.0	0.0415	12.0	2.1
Thomas 3b	32.4	39989	0.0410	0.6	7.7	0.0246	3.0	9.0	0.0100	12.0	1.0
Thomas 4a	33.7	4313	0.0466	0.6	9.0	0.0326	3.0	10.0	0.0287	5.5	5.0
Thomas 4b	141.9	19881	0.0218	0.5	9.0	0.0202	3.0	9.0	0.0000	-	-
Thomas 5	167.4	21577	0.0328	1.0	5.0	0.0206	3.0	9.0	0.0000	-	-
Area1 (Cole Eng. Study)			0.1038	0.7	8.1	0.0796	3.0	9.2	0.0389	9.8	2.7
Area2 (Cole Eng. Study)			0.0819	0.7	8.1	0.0629	3.0	9.2	0.0307	9.8	2.7
Area3 (Cole Eng. Study)			0.0137	0.7	8.1	0.0105	3.0	9.2	0.0051	9.8	2.7
Area4 (Cole Eng. Study)			0.1411	0.7	8.1	0.1083	3.0	9.2	0.0528	9.8	2.7
Area5 (Cole Eng. Study)			0.2335	0.7	8.1	0.1791	3.0	9.2	0.0874	9.8	2.7
Area6 (Cole Eng. Study)			0.2335	0.7	8.1	0.1791	3.0	9.2	0.0874	9.8	2.7
Area7 (Cole Eng. Study)			0.2335	0.7	8.1	0.1791	3.0	9.2	0.0874	9.8	2.7
Area10 (Cole Eng. Study)			0.0824	0.7	8.1	0.0632	3.0	9.2	0.0308	9.8	2.7
Area11 (Cole Eng. Study)			0.0141	0.7	8.1	0.0108	3.0	9.2	0.0053	9.8	2.7
Area12 (Cole Eng. Study)			0.0578	0.7	8.1	0.0444	3.0	9.2	0.0216	9.8	2.7
Area13 (Cole Eng. Study)			0.2335	0.7	8.1	0.1791	3.0	9.2	0.0874	9.8	2.7
Area14 (Cole Eng. Study)			0.0874	0.7	8.1	0.0671	3.0	9.2	0.0327	9.8	2.7
Area16 (Cole Eng. Study)			0.1138	0.7	8.1	0.0873	3.0	9.2	0.0426	9.8	2.7

	Area (ha)	Length of Sewers (m)	R1	T1	K1	R2	T2	K2	R3	T3	K3
Average of metered catchments			0.0265	0.7	8.1	0.0194	3.0	9.2	0.0134	9.8	2.7
Std. Dev. of metered catchments			0.0163	0.2	1.6	0.0093	0.0	0.4	0.0177	3.8	2.1
Average + Std.Dev. (used for other unmetered catchments)			0.0428	0.8	9.7	0.0287	3.0	9.6	0.0311	9.8	2.7

3.0 MODEL RESULTS

3.1 Network Performance

The sewer network performance has been mapped out for design storm events and the maps presented in Appendix 7 show the hydraulic bottlenecks for sewer surcharge and for hydraulic grade lines in maintenance holes for 2-year to 100-year design storm events. For the maintenance holes a hydraulic grade line criteria of 1.8 m freeboard (compared to road centreline) was used. This would typically match the level of basements and any sewer surcharge above would trigger flooded basements if foundation drains or any sanitary basement installations are made. Maintenance holes were color coded with green color showing freeboards of 1.8 m or more, yellow showing less than 1.8 m freeboard and red color showing 0 freeboard or flooding above ground.

Color coding for sewers was used in the same three colors for green sewer showing no surcharge, yellow color showing surcharge conditions by depth but not representing necessarily a bottleneck and red color showing a capacity restriction based on limited pipe carry capacity.

The model shows localized sewer bottlenecks for the lower return period storms and a larger portion of the network shows capacity problems for the 100-year return period storms. Pockets of flood clusters can be recognized, and these typically match the areas that have been identified with high I&I rates, since this data has been entered into the model hydrology. As mentioned above, a large portion of the sewer catchment is currently unmetered and has been assigned with a high average I&I profile and an average wastewater profile based on 250 l/cap/d. Therefore, the sewer network performance can mostly show a picture of known hotspots and for the remainder of the area flows that are based on best estimates.

3.1.1 I&I Area Analysis

From flow monitor data evaluation and past I&I study reports certain catchment areas could be quantified with typical Groundwater Inflow (GWI) rates during dry weather conditions and with higher Rainfall Derived Infiltration and Inflow (RDII) rates. It was noted that for the known catchment areas, the I&I distribution varies considerably with a large rate spread that could be as large as a factor of 8 for the example when comparing the RDII rates between sewer catchment for FM Thomas 1 with the catchment for FM Thomas 4a. Table 3.1 shows the calibrated average RDII rates for the sewer catchments where the 2020 flow monitoring survey took place and produced useable results.

Table 3-1 - I&I Rates for the Calibration Rainfall Events

Flow Monitor Name	Catchment area [ha]	Storm Event 1		Storm Event 2		Storm Event 3		RDII Average Rate [L/s/ha]
		Peak RDII Flow [L/s]	RDII Rate [L/s/ha]	Peak RDII Flow [L/s]	RDII Rate [L/s/ha]	Peak RDII Flow [L/s}	RDII Rate [L/s/ha]	
Thomas 1	108.0	2.5	0.023	7.1	0.066	11.4	0.106	**0.065**
Thomas 2	101.0	10.9	0.108	16.2	0.160	18.6	0.184	**0.151**
Thomas 3b	33.7	10.1	0.300	13.1	0.389	18.9	0.562	**0.417**
Thomas 4a	27.1	10.1	0.373	13.5	0.499	18.9	0.699	**0.524**
Thomas 4b	169.9	24.3	0.143	22.1	0.130	28.1	0.165	**0.146**
Thomas 5	139.7	38.6	0.276	56	0.401	88	0.630	**0.436**

Further data was produced for the former Cole I&I study area “*Aldborough/Leger and Woodworth”* catchments. Appendix 8 shows the sewer catchments with calculated GWI rates and RDII Rates.

GWI rates for the 2020 monitored catchments vary their inflow from 0.019 l/s/ha to 0.102 l/s/ha. The RDII rates for the 2020 monitored catchments show a range of 0.065 l/s/ha to 0.524 l/s/ha. Some of the worst performing I&I catchments that were measured by Cole in 2014/2015 showed peak RDII rates of up to 3.8 l/s/ha.

Based on the data and the wide variation, an estimate for the currently unmetered area that represents approximately 63% of the sewershed was made and that RTK parameter estimate was based on the average of the 2020 measured data plus one standard variation. Should there be further pockets of very high infiltration such as measured during the 2015 Cole study present, then this estimate is underpredicting the I&I flows for the catchment area. Given that this area represents 996 ha in size (63% of total area), this represents a considerable amount of uncertainty for the flow predictions of the remaining unmetered study area. It is recommended to further investigate sewersheds and quantify the amount of I&I for previously unknown areas. This could eventually produce a larger map of I&I target areas that need further investigation to single out the major I&I sources and develop strategies to reduce this inflow.

3.2 Storage Control and Tank Performance

3.2.1 Sluice Gate- Real Time Control

The main flow control feature to limit flows towards the Wastewater Treatment Plant is a real time-controlled twin sluice gate. The sluice Gates are each 0.6 m wide and have a maximum opening of 0.9 m with actuated penstocks that can close the gate opening when tank flows and the hydraulic head on the gate rises and increases pass forward flows. A flow monitor or level gauge downstream of the Sluice gates measures the flows and sends a SCADA signal to the sluice control to limit flows to approximately 500 l/s maximum. For the future scenario where treatment plant bottlenecks are removed, an increased flow rate can be accepted that will reach the current ECA consent of 632L/s.

There are two main additional sources of flow arriving at the main sewer downstream of the sluice gates that cannot be controlled or throttled. One source is the flows from Sunset SPS that currently operates at 21 L/s capacity but is scheduled to be upgraded to 43L/s capacity. The second source of uncontrolled inflow comes from a 200 mm diameter sanitary sewer connection that has a capacity of approximately 20L/s.

Whilst in the future scenario, it is recommended to connect the 200 mm diameter sewer to the CSO tank, the incoming sewer from Sunset SPS will remain connected downstream of the tank.

Therefore, the acceptable flow limitation in the future scenario is 632 L/s – 43 L/s = 589 L/s.

The hydraulic model was set up for the existing scenario to limit flows to 500 L/s and for the future scenario to limit flows to 589 L/s at the sluice gates. Real time control rules were applied to the model. A flow monitor control was set at the sewer downstream of the sluice gate. At the start of the simulation, the sluice gates are fully opened to 900 mm height. Should the flow at the downstream pipe increase beyond the set flows, a rule was implemented to decrease the opening height. A similar rule was set for a situation where flows recede and the monitored flows decrease below the set maximum pass flow forward rate, so that the sluice gates open towards full opening height again. The speed of incrementation or decrementation of the sluice gate openings was set to 5 cm/s as a best estimate. The gate controller was set to check flows every 60 seconds during the simulation to determine if the set rules were true.

3.2.2 Storage Tank Performance

The CSO storage tank performance was initially tested against constant inflow events to evaluate potential storage times when inflows exceed the controlled outlet flows. A comparison was made between the current 500 L/s flow control and the proposed increase to 589 L/s. See Table 3.2 for details.

Table 3-2 - Tank Storage Performance for Test Inflows

Scenario	Tank Gates controlled to 500 L/s	Tank Gates controlled to 589 L/s	Storage Time Increase [hrs]
	Storage Time [hrs]	Storage Time [hrs]	
800 l/s steady inflow	4:15	5:10	**0:55**
1000 l/s steady inflow	3:20	4:00	**0:40**
1100 l/s steady inflow	1:50	2:15	**0:25**

Scenario	Tank Gates controlled to 500 L/s	Tank Gates controlled to 589 L/s	Storage Time Increase [hrs]
	Storage Time [hrs]	Storage Time [hrs]	
1200 l/s steady inflow	1:29	1:50	**0:21**
1400 l/s steady inflow	1:05	1:15	**0:15**

The sluice gate operation logic was set-up have the gate fully open (0.9 m) at flows below 500 L/s, reduce the opening to 0.15 when flow exceeds 500 L/s, and open incrementally as the flow decreases to 500 L/s or lower. Due to the rule setting for the sluice gate Real Time Control (RTC), the setup currently produces some minor operational inefficiencies that are caused by the sluice gate closing move that will cause an initial period of over-controlling flows before opening again. This iterative process of opening and closing until the sluice gate finds the optimum flow control can be shown as an oscillation of the regulator state or sluice gate opening. Figures 3.1 and 3.2 below show on the example of a 2-year design storm graphically the regulator state (sluice gate opening depth) in combination with the passed flow forward and in the second figure in combination with the tank level. The inefficiencies slightly reduce the storage times when compared with a theoretical storage time when the maximum outlet flow would be constantly applied without considering the opening and closing process. The above table shows tank storage times for a linear sluice gate closing process without over-throttling and opening during the tank filling process. The time difference between an efficient RTC operation and an oscillating RTC operation could be up to 40 minutes (or approximately 15%) of storage time for example in the 800L/s steady inflow event.

3.3 Spill Analysis

Under section 2.4 Rainfall Series, a typical year rainfall series were developed that are based on real measured events in Toronto with their intensity slightly decreased to adjust for the location of St. Thomas according to their IDF curves. Whilst running the entire rainfall series, it was noted that the rainfall is not sufficient to trigger a spill from the CSO tank which does not align with the actual conditions where several spills are observed in a typical year. Alternative scenarios were run with modified rain gauge profiles of which four (4) were initially applied and a possible worst-case scenario was created by using a single rain gauge only that would show the same peak precipitation over the entire catchment instead of a spatial variation.

However, this approach also did not yield any spills which led to the conclusion that the tank does not get 100% utilized during the typical year rainfall series. The key reason for this seems to be the current operational mode of the sluice gates in which the maximum opening of the gates is pre-set at 0.18 m, in contrast to 0.9 m (initial full opening of the gate) used in the spill analysis model.

The current model shows that during a typical year, the worst occurring storms would create peak flows of 694 L/s into the CSO tank and the tank level would fill up notably approximately seven (7) times during the year with the top level reaching 204.6 m, which is below the 205 m spill weir elevation.

1. The current model scenarios were simulated with a real time control for the tank sluice gates. Whilst we noted some sluice gate oscillations for the flow adjustment for example when the tank level rises, the sluice gates briefly overcontrol flows before opening again. This could be further optimized through the gate closing and opening speed or by setting additional RTC rules. The possibility exists that the simulated sluice gate operation is more efficient than the current sluice gate operation. If that is that case and the sluice gates overcontrol the tank flows on site, an operational change could show a much higher spill reduction from current to the proposed scenario by adjusting the sluice gate operation.

2. Portions of the model catchment could be currently underrepresenting runoff flows in terms of wet weather flow response and I&I rates. Given that approximately 63% of the catchment were applied to an estimated RDII profile, variations can be expected but it is unclear if this would lead to the added flows to trigger a CSO tank spill.

The full series of design storms from 2-yr to 100-yr storm events were run under existing conditions and for proposed conditions with increased flow rate to the treatment plant. A set of design storms that incorporate a climate change factor were also run and the table below compares the proposed scenario and proposed plus climate change scenario against the existing scenario that was used as a baseline.

In the baseline scenario the peak flows and volumes created by the sewer network always exceed the CSO tank storage capacity. It is to be considered that the design storm is a 12-hour duration storm that has a considerable impact on the catchment area in terms of saturation and wet weather flow response. Whilst the proposed upgrades can only slightly reduce the peak flows (-0.88% to -0.19%), it does reduce the total spill volume between approximately 4% and 9%. Therefore, initial benefits will be seen in the number and volume of typical spills. However, for the climate change scenario during years 2050 to 2100, it can be expected that this initial reduction trend will reverse and could show an increase in peak spills (10% to 15%) with increase in volume between approximately 23.6% to 26.5%. Climate change predictions come with an uncertainty and will depend on future economic activities and mitigations. For this climate change impact assessment, the highest predicted changes and worst possible outcomes were used to showcase the maximum potential for flow and volume increases.

Table 3-3 - CSO Tank Spills for Design Storm Scenarios (Peak Flow and Volume)

	Flow (m^3/s)				
Storm Event	**Existing Scenario**	**Proposed Scenario**	**%Change vs Existing**	**Proposed-Climate Change Scenario**	**%Change vs Existing**
2yr	1.25	1.24	-0.88	1.41	12.56
5yr	1.52	1.51	-0.59	1.75	15.13
10yr	1.71	1.70	-0.64	1.93	12.69
25yr	1.93	1.92	-0.73	2.13	10.47
50yr	2.05	2.04	-0.29	2.26	10.44
100yr	2.16	2.16	-0.19	2.38	10.28
	Volume (m^3)				
Storm Event	**Existing Scenario**	**Proposed Scenario**	**%Change vs Existing**	**Proposed-Climate Change Scenario**	**%Change vs Existing**
2yr	29,708	26,964	-9.24	37,359	25.75
5yr	46,910	43,649	-6.95	59,382	26.59
10yr	59,756	56,216	-5.92	73,893	23.66
25yr	77,412	73,111	-5.56	96,481	24.63
50yr	90,901	86,390	-4.96	114,031	25.45
100yr	105,026	100,703	-4.12	132,336	26.00

3.3.1 Sewage Pumping Station Related Spills

Of the 16 existing sewage pumping stations in the model, seven (7) show several spills through the emergency overflows at the pumping station itself or at a nearby high-level overflow. One extra overflow link was monitored on Sunset Drive south of the CSO tank and is shown as link SAMH891 in the table. The table below shows details of the pumping stations.

Table 3-4 - SPS Spills for Design Storm Scenarios (Return Period)

	Spill Occurrence					
Sewage Pumping Station	**2-yr**	**5-yr**	**25-yr**	**50-yr**	**100-yr**	**Number of spills – Typical Year**
#1 Axford	x	x	x	x	x	0
#2 Burwell				x	x	0
#3 Confederation			x	x	x	0

	Spill Occurrence					
Sewage Pumping Station	**2-yr**	**5-yr**	**25-yr**	**50-yr**	**100-yr**	**Number of spills – Typical Year**
SAMH891.1 (SSO Sunset Dr., south of CSO tank)			x	x	x	0
#11 St. George					x	0
#12 Sunset	x	x	x	x	x	37
#14 Wolfe	x	x	x	x	x	0
#16 Woodworth	x	x	x	x	x	0

St. Thomas Water Pollution Control Plant Annual Performance Reports from 2015 to 2020 were reviewed for reported overflows from SPS. The table below summarizes the 6 years of reporting.

Table 3-5 – Number of SPS Overflows Reported in WPCP Annual Reports (2015-2020)

Sewage Pumping Station	0<OF <10m^3	10<OF <100m^3	100<OF <1,000m^3	1,000<OF <10,000m^3	OF> 10,000 m^3
#3 Confederation	1		1		
#11 St. George			2	1	
#12 Sunset	8	13	1		
#16 Woodworth	4	5	8	1	1

Whilst many of the above sewage pumping station show spills for the design storm events, only #12 Sunset SPS shows a frequent spill activity for the typical year storm series. This pumping station is already being proposed for an upgrade to approximately double its pumping capacity. The design of the replacement SPS should address the current frequent small overflows from the existing SPS.

Another pumping station that shows spills from a 2-year design storm is Woodworth Avenue SPS. The annual reports overflow records show that this pumping station has spilled every year between 2015 and 2020.

The Woodworth SPS has a setup of three pumps in a duty/ lag/ standby arrangement. The design capacity is 101 L/s at 13.7 m TDH for each pump. The pumps discharge into a 400 mm diameter forcemain. As for all pumping stations, dynamic head discharge curves were added for each pump. Since actual pump performance curves were not available, pump curves from the pump manufacturer's website were looked up and adjusted to represent a best estimation. With the current pump setup, the model predicts a maximum pumping station performance of 228 L/s that generates approximately 1.8 m/s velocity in the forcemain. However, this is not sufficient to pump the total inflows to the pumping station. The model predicts total inflows to be 303L/s, 365 L/s and 400 L/s for the 2-year, 5-year and 10-year design storms respectively. This leads to spill events at the high-level overflow.

An upgrade of the Woodworth SPS to operate 3 pumps with the above capacity would combine to a total pump performance of 297 L/s at approximately 2.36 m/s velocity in the forcemain. Whilst such an upgrade will increase the pumping station's spill protection to approximately a 2-year design storm event and will eliminate the currently experienced annual spills, it would put overload approximately 1760 m of sewer section from the discharge point. The existing sewers are 675 mm and 750 mm in diameter whilst the first sewer section is steeper with 450 mm diameter. The approximate sewer capacity of this section is in the range of 350 – 380 L/s. Since there are other sewer inflow apart from Woodworth SPS, this sewer shows surcharge in the 2-year storm event with 0.23 m freeboard at a low point. Therefore, this sewer section would require an upgrade to add approximately 100 L/s capacity to its line. The following downstream sewer section is on 1050 mm diameter and has varying capacity but a minimum of 1 m^3/s.

4.0 RECOMMENDED IMPROVEMENTS

4.1 Impact of Removing Hydraulic Constraints in the Collection System

When hydraulic constraints are removed in the collection system, more influent flow will reach the WPCP. While this may provide the basis for limiting discharges of untreated sewage to the environment, the additional wet weather and infiltration flows will put a strain on the WPCP. This would increase both average as well peak flows experienced by the plant, thereby reducing the residual capacity of the plant, and therefore limiting the growth potential within the current rated capacity of the WPCP. In particular, given the high I&I in the collection system, the peak flows will increase significantly and therefore will be the likely limiting factor for growth going forward. It would be recommended that the upcoming WPCP Master Plan look to identify how much additional wet weather flows could be treated through the plant based on potential upgrade options.

The 99 percentile PDF peaking factor based on the historic data approaches approximately 3.0, which means at the current average flow of about 18,000 m^3/d, the WPCP would already be at its peak capacity once the CSO overflows get controlled and the additional flow volumes are diverted to the WPCP. This means that the wet weather peaks corresponding to higher than current average flows, would likely start exceeding the plant capacity, thereby leading to an upward trend in CSO overflows going forward. While these future overflows are likely to be much lower than the current ones both in frequency and intensity in the first few years, these are likely to increase with City’s growth and utilization of the residual capacity. As such, while difficult to predict in terms of timing and degree, this future increase in CSO overflows may have to be addressed by pushing more flow through the WPCP and providing it partial treatment instead of letting raw sewage overflow at the CSO.

Removal of some of the hydraulic bottlenecks at WPCP via the measures recommended in this report would add to hydraulic and process capacity of some unit processes like, grit chamber and UV disinfection. In addition, hydraulic and process capacity of some of the unit processes like primary clarifiers of Plant 4, is potentially higher than their current proportion of capacity which may allow the additional wet weather flows (beyond the PDF capacity of the WPCP) being passed through Plant 4 primary clarifiers and UV for partial treatment. To this effect, upgrading the Plant 4 primary clarification to chemically enhanced primary treatment (CEPT) for wet weather flow treatment is recommended over the next five years.

When implemented, this upgrade will allow additional wet weather flows to receive enhanced treatment in Plant 4 clarifiers, secondary treatment up to Plant 4 secondary system capacity (by-passing the excess flow from the secondary process), and finally UV treatment up to the latter's capacity. As such this upgrade would significantly mitigate the impact of any future overflows at the plant.

4.2 Building on the Current Hydraulic Model

To determine a holistic solution that best balances the cost effectiveness of infiltration reduction measures, collection system improvements and wet weather capacity improvements to the WPCP so as not to reduce its rated capacity, an ongoing program to improve the City's understanding of the collection system is recommended. The hydraulic model requires further fine tuning through the acquisition of flow monitoring data in previously unmonitored areas. This effort should be joined by a simultaneous I&I analysis.

The City of St. Thomas should install a permanent rain gauge station at the wastewater treatment plant to be able to conduct future flow monitoring assignments and have concurrent rain data information in 5-minute interval resolution to match standard flow monitoring timesteps. A further benefit of such an installation would be the ability to measure extreme storm events and replicate known flood events for further hydraulic model calibration.

The age and parcel fabric over the St. Thomas catchment varies considerably and in connection with that, a high deviation in Infiltration and Inflow rates was observed for monitored sewersheds and reviewed from previous Infiltration and Inflow studies such as the 2015 Study for the Aldborough/ Leger and Woodworth Avenue SPS study areas. I&I inflow rates between certain areas were observed with differences with a factor up to eight (8) times as high as the lowest I&I rates. Whilst the hydraulic model was calibrated for wet weather flows with available 2020 flow monitoring data, a large area (996 ha), 63% of the total sewershed was not covered by this calibration and had to be estimated, based on surrounding I&I rates. Since pockets of very high infiltration were observed within the unmonitored areas, we have adjusted the generic RTK profiles towards the higher end of the observed I&I rates but not the extreme I&I rates that were observed for pockets in the 2015 I&I study. It is recommended to carry out further future flow analysis to further refine the model calibration and peak flow response in combination to finding areas with extreme high I&I rates and exploring the source of the infiltration. A future objective should become to eliminate portions of I&I where practicable and cost efficient. We foresee considerable scope of wet weather flow reduction from targeted improvement assignments once the overall system is better understood. The 2020 flow monitoring and 2015 I&I study show a portion of such problems.

We would recommend that the City install three to four flow meters per year to better understand the inflow within the collection system over the next 5 years. Each flow meter would be in place for a 9-month period. Using the data from the rain gauge information and the flow data collected, the current model can be updated to better reflect the conditions in the system and to address any issues that the City wishes to review.

After year 5 of the flow monitoring, the City can decide if they wish this annual program to continue.

4.3 Improvements to the CSO Tank Operation

The current operational mode of the sluice gates allows maximum opening of the gates pre-set at 0.18 m, thereby allowing the head build-up via wet weather flow storage to push higher flows (up to 500 L/s) through the gate opening. While protective of the downstream processes, this approach is clearly over-conservative in flow control due to high level of CSO overflows, and their occurrence at peak flows lower than the PDF capacity of the plant.

In contrast the approach used in the hydraulic model uses a gate operation logic in which the gate fully open (0.9 m) at flows below 500 L/s and reduces the opening to 0.15 when flow exceeds 500 L/s and open incrementally as the flow decreases to 500 L/s or lower. The increment/decrement rate of the sluice gate openings was set to 5 cm/s as a best estimate. The gate controller was set to check flows every 60 seconds during the simulation to determine if the set rules were true. Given that this operating strategy generated no overflows at the CSO tank up to a flow of 694 L/s indicates significant potential of mitigating overflows by upgrading the current control settings to the above approach. As such the CSO gate operation is recommended to be upgraded based on this approach once the hydraulic bottlenecks at the plant are addressed.

This work should be undertaken following upgrades to WPCP to remove bottlenecks. It is not anticipated to require any additional capital cost but will require a trail and error approach to programming the gates, reviewing the results and adjusting based upon some months of operation. The MECP should be advised of this approach so that there is not miscommunication while the gate operation is being optimized.

4.4 Improvements to SPSs to Reduce Overflows

4.4.1 #12 Sunset SPS

This Sunset SPS is already being proposed for an upgrade to approximately double its pumping capacity. The design of the replacement SPS should address the current frequent small overflows from the existing SPS.

4.4.2 Improvements to the Woodworth SPS and Downstream Collection System

4.4.2.1 SPS Upgrades

Improvements to the Woodworth SPS will require detailed study to balance the impacts of periodic sewage overflows versus ensuring operational efficiency for dry-weather operation (which is most of the time). This will require a separate engineering planning and detailed design assignment.

The Woodworth SPS was upgraded last in 2011 to provide for a third sewage pump (each with a capacity at their operating point of 101.8 L/s) which provides the station with a hard capacity of 201.6 L/s according to the current ECA. Our modeling indicates that each of these pumps would have to be upsized to a capacity of approximately 150 L/s to manage flows up to the 2-year return period. At present, the SPS is not equipped with variable frequency drives on the pumps which indicates that the current pumps can provide adequate service at lower flows. Potential pump configuration options are shown in the table below.

Table 4-1 – Woodworth SPS Upgrade Configuration Options

Option	P1	P2	P3	Comment
1	150 L/s	150 L/s	150 L/s	Hard Capacity with 2 Duty Pumps. VFD required on all pumps to ensure low flow coverage
2	300 L/s	300 L/s	jockey	Hard Capacity with one Duty Pump. VFD probably required on P1 and P2 to ensure range of flow c overage provided

4.4.2.2 Downstream Forcemain/Sewer Capacity

Increasing the capacity of the Woodworth SPS will require detailed study of the impacts to the downstream forcemain and sewers to ensure that no adverse impacts occur when the SPS' capacity is increased.

Our modeling has identified that capacity in the collection system will have to be increased approximately 1760 m downstream at a minimum. These changes will involve:

1. Upsizing the current 1007 m of 400 mm forcemain (including a crossing of the multiple rail tracks on First Ave);
2. Replacing 250 m of 450 mm sanitary sewer from forcemain outlet to Talbot Street on First Ave; and
3. Replacing 523 m of 600/750 mm sanitary sewer from First Ave to south of Wellington St.

Immediately downstream of these upgrades is a steep 450 mm sewer which crosses First Ave and connects to an 825 mm sanitary sewer which continues south.

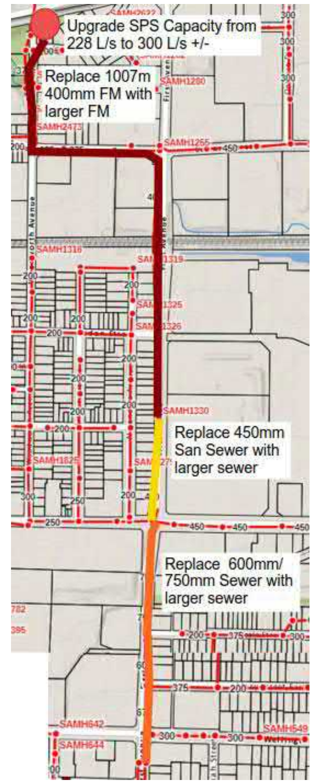


Figure 4-1 Upgrades required to Woodworth SPS and Downstream Collection System

4.4.3 Burwell Rd SPS and Collection System

The Burwell Rd SPS has a setup of 2 pumps in a duty/ lag arrangement. The design capacity is 44 L/s at 30 m TDH for each pump. The pumps discharge into a 200 mm diameter forcemain Under 10-year wet weather conditions, with the additional future flows from the Edgeware Line Employment Lands, there will be a requirement to increase the PS capacity, upsize the forcemain and approximately 1200 m of sewers.

Under 10-year wet weather conditions, with the additional future flows from the Edgeware Line Employment Lands, there will be a requirement to increase the capacity of the Burwell Ave SPS. The Burwell Rd SPS would require to be upgraded to a capacity of 219 L/s. The current 200 mm diameter forcemain would require twinning and forcemain and approximately 1200 m of sewers are required to be upsized.

5.0 PPCP SOLUTION COST OPINION

5.1 Levels of Cost Estimation

ASTM E 2516-11 (Standard Classification for Cost Estimate Classification System) provides a five-level classification system based on several characteristics, with the primary characteristic being the level of project definition (i.e., percentage of design completion). The ASTM standard, shown in Table 6.1, illustrates the typical accuracy ranges that may be associated with the general building industries.

Table 5-1 – ASTM E2516-11 Accuracy Range of Cost Opinions for General Building Industries

Cost Estimate Class	Expressed as % of Design Completion	Anticipated Accuracy Range as % of Actual Cost
5	0-2	-30 to +50
4	1-15	-20 to +30
3	10-40	-15 to +20
2	30-70	-10 to +15
1	50-100	-5 to +10

The cost estimates developed in this report would be best described as a Class 5 Cost Estimate which is an order of magnitude cost opinion, also referred to as a parameter or conceptual cost opinion. It is generally used for strategic business or capital planning, assessment of viability, or for comparative purposes to establish a base ranking of alternatives. There is usually a very low level of project definition and limited information available. A more detailed review of our cost estimation procedure is provided in the Appendix 9.

5.2 Solution Costs

A Class 5 cost estimate prepared by RVA are detailed as follow:

- Table 5.2 itemizes the PPCP recommended program costs by project;
- Table 5.3 provides a summary of the expected cashflow for the PPCP; and
- Table 5.4 provides a summary of the cumulative costs over the 5,10, 20 and 40-year periods.

Additional details on the development of the cost opinions are provided in Appendix 9.

Table 5.2 – Conceptual Cost Opinion Per Recommended Item

Component	Cost Estimate Per Activity			Timeframe/ Comment
	Capital	Planning and Engineering	Total	
Recommended Collection System Upgrades				
Sunset SPS Improvements to coordinate with PPCP	$0	$25,000	$25,000	Assume that this may be only a design change in the new PS and not impact the construction cost.
Additional cost to reroute the new Sunset SPS forcemain to the CSO	$100,000	$20,000	$120,000	Undertake following upgrades to WPCP to remove bottlenecks when now Sunset St SPS is being built.
Woodworth Ave SPS Upgrades	$2,500,000	$500,000	$3,000,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Woodworth Ave SPS Collection System	$3,849,283	$577,392	$4,426,675	When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Upgrades	$2,000,000	$400,000	$2,400,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Downstream Collection System	$1,500,000	$225,000	$1,725,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Annual Sewer Lining (500 m/year)	$650,000	$65,000	$715,000	It will take 62 years to undertake the lining of the current total of 31 km of fair to poor sanitary sewers in the system.
CSO Operation Optimization				
Improvements to CSO Tank Operation	$0	$100,000	$100,000	2023 - undertake following upgrades to WPCP to remove bottlenecks. Assume that this involves changes in controls only. Does not include costs for removing bottlenecks in WPCP.

Component	Cost Estimate Per Activity			Timeframe/ Comment
	Capital	Planning and Engineering	Total	
Removal of Capacity Constraints at the WPCP				
Remove WPCP Bottlenecks	$2,727,000	$273,000	$3,000,000	2022-23 -Modify plant flow distribution, remove pipe bottlenecks, twin UV channel, and add a new parallel unit, upgrade outfall pipe.
Long Term I & I Mitigation Measures				
Permanent Rain Gauge Installation	$15,000	$4,000	$19,000	Early 2022 installation.
Annual Camera Work in Collection System	$250,000	$0	$250,000	Yearly work ($50,000) over a 5-year period.
Flow Monitor Installation, Maintenance, Removal	$176,000	$0	$176,000	Yearly work ($35,200) over a 5-year period.
Building on the Current Hydraulic Model	$0	$79,000	$79,000	Yearly work (15,800) over a 5-year period.

Table 5.3 – PPCP Cashflow

Component	Cashflow (Years)				Timeframe/ Comment
	1 to 5	6-10	11 to 20	21-40	
Recommended Collection System Upgrades					
Sunset SPS Improvements to coordinate with PPCP	$25,000				Assume that this may be only a design change in the new PS and not impact the construction cost.
Additional cost to reroute the new Sunset SPS forcemain to the CSO	$20,000	$120,000			Undertake following upgrades to WPCP to remove bottlenecks when now Sunset SPS is being built.
Woodworth Ave SPS Upgrades	$4,800,000				When City deems necessary to do/ High level estimate/ City

Component	Cashflow (Years)				Timeframe/ Comment
	1 to 5	6-10	11 to 20	21-40	
					may look at other options.
Woodworth Ave SPS Collection System	$4,426,675				When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Upgrades		$2,400,000			When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Downstream Collection System		$1,725,000			When City deems necessary to do/ High level estimate/ City may look at other options.
Annual Sewer Lining (500 m/year)	$3,575,000	$3,575,000	$7,150,000	$14,300,000	Start sewer lining in year 3 after 2 years of additional modeling and data city will take 41 years to undertake the lining of the current total of 31 km of fair to poor sanitary sewers in the system.
CSO Operation Optimization					
Improvements to CSO Tank Operation	$100,000				2023 - undertake following upgrades to WPCP to remove bottlenecks. Assume that this involves changes in controls only. Does not include costs for removing bottlenecks in WPCP.
Removal of Capacity Constraints at the WPCP					
Remove WPCP Bottlenecks	$3,000,000				2022-23 -Modify plant flow distribution, remove pipe bottlenecks, twin UV channel, and add a new parallel unit, upgrade outfall pipe.
Long Term I & I Mitigation Measures					
Permanent Rain Gauge Installation	$19,000				Early 2022 installation.

Component	Cashflow (Years)				Timeframe/ Comment
	1 to 5	6-10	11 to 20	21-40	
Annual Camera Work in Collection System	$250,000	$250,000			Yearly work ($50,000) over a 5-year period. Stop at year 10 when a new MP should be undertaken.
Flow Monitor Installation, Maintenance, Removal	$176,000	$176,000			Yearly work ($35,200) over a 5-year period. Stop at year 10 when a new MP should be undertaken.
Building on the Current Hydraulic Model	$79,000	$79,000			Yearly work ($15,800) over a 5-year period. Stop at year 10 when a new MP should be undertaken.

Table 5.4 – PPCP Cumulative Program Cost

	To Year 5	To Year 10	To Year 20	To Year 40
Estimated Cost	$16,470,675	$24,795,675	$31,945,675	$46,245,675
Low (-30%)	$11,529,473	$17,356,973	$22,361,973	$32,371,973
High (+50%)	$24,706,013	$37,193,513	$47,918,513	$69,368,513

APPENDIX 1

Sewage Pumping Station Details

#	Pumping Station	Year Built	Address	ECA	# Pumps	Inlet Elevation (mASL)	All Pump Stop (mASL)	Lead Pump Start (mASL)	Lag Pump Start (mASL)	High Level Alarm (mASL)	Overflow Elevation (mASL)	Pump 1		Pump 2		Pump 3		Overflow Location	Backup Generator	Comment	Wet Well Cross Sectional Area (m^2)	Wet Well Bottom Elevation (mASL)
												Type	Capacity	Type	Capacity	Type	Capacity					
1	Axford SPS	1997	111 Axford Parkway	3-1194-96-006	2	222.25	221.54	222.11	222.21	223.05	225.96	Gorman-Rupp Model ECM	56.6 L/s @ 8.9 m TDH	Gorman-Rupp Model ECM	56.6 L/s @ 8.9 m TDH			Creek to NW	Yes		8.21	221.33
2	Burwell Rd. SPS	1993	315 Burwell Road	3-0584-93-006 (Station)	2	225.25	223.36	225.11	225.31	225.77	228.56	ITT Flygt 3170.180	44 L/s @ 30 m TDH	ITT Flygt 3170.180	44 L/s @ 30 m TDH			Creek to NW	Yes		6.16	222.11
3	Confederation Dr. SPS	1967	39 Confederation Dr.	3-0200-68-006	2	225.63	223.85	224.90	225.07	225.15	225.54	Smith & Loveless	50.47 L/s @ 14.87 m TDH	Smith & Loveless	50.47 L/s @ 14.87 m TDH			Creek to NW to Dalewood Res.	Yes		11.16	223.07
4	Crescent Ave. SPS	1988	99A Crescent Avenue	3-0224-87-006	2	223.40	222.72	223.22	223.42	223.52	225.39	Hydromatic Pentair Water Company	16 L/s @ 9.54 m	Hydromatic Pentair Water Company	16 L/s @ 9.54 m			Creek to NW	No		4.68	222.4
5	Elm St SPS	2020	Approx. 500 m east of Centennial	N/A	2	N/A	N/A	N/A	N/A	N/A	N/A	Xylem NP3153 MT 3 - 486	44.35 L/S @13.1 TDH	Xylem NP3153 MT 3 - 486	44.35 L/S @13.1 TDH			N/A	Yes			
6	Harper Rd SPS	1973	120 Harper Road	N/A	2	239.57	238.79	239.24	239.32	239.37	N/A	Gorman-Rupp T Series	21 L/s @ 9.1 m TDH	Gorman-Rupp T Series	21 L/s @ 9.1 m TDH			Overflow at 1st San MH u/s from PS	No		10.52	237.82
7	Karen St SPS	2013	446 Sunset Drive	6882-8A8JDW	2	223.87	221.90	223.55	223.65	223.70	232.85	Flygt 3153	43.2 L/s @ 15 m TDH	Flygt 3153	43.2 L/s @ 15 m TDH			None	No	O&M States "raw sewage will overflow through the top of the wetwell chamber and may cause surcharging into basements within the sewershed"	7.07	220.90
8	Lynhurst SPS	1997	22A Edgewell Crescent	3-0215-97-006	2	224.52	223.95	224.50	224.60	224.70	225.75	Flygt Model 3102	23 L/s @ 8 m TDH	Flygt Model 3102	23 L/s @ 8 m TDH			SE to retention pond	Yes		7.07	223.50
9	Parkside Drive SPS	1970	65 Parkside Drive	3-0518-70-006	2	228.90	228.06	228.41	228.43	228.45	229.83	ITT Flygt Model CP3102.180	14.51 L/s @ 6.4 m TDH	ITT Flygt Model CP3102.180	14.51 L/s @ 6.4 m TDH			Storm sewer	No	Overflow to storm sewer on Parkside Dr	4.67	227.70
10	Shaw Vally Dr SPS	2006	135 Shaw Valley Drive	2307-6RLK9P	2	219.30	219.00	219.25	219.25	223.25	none	Flygt 3153	62.7 L/s @ 17 m TDH	Flygt 3153	62.7 L/s @ 17 m TDH			None	No		11.82	218.10
11	St. George SPS	1967/ 1997	95 St. George Street	3-0224-87-006	2	201.05	200.31	201.40	201.45	201.70	204.19	Gorman-Rupp T Series (T-10)	94.6 L/s @ 37.2 m TDH	Gorman-Rupp T Series (T-10)	94.6 L/s @ 37.2 m TDH			Creek to W	Yes		7.3	198.55
12	Walnut St. (Sunset) SPS	1973/ 1994	68 Sunset Drive	4339-BDBJL3	2	200.25	199.58	200.25	200.33	200.41	202.21	Barnes Model #4XSE7554A	23 L/s @ 8.5 m TDH	Barnes Model #4XSE7554A	23 L/s @ 8.5 m TDH			SE to park and eventually to Kettle Creek	No	ECA has ultimate peak flow of 46 L/s	3.58	198.73
13	Talbot Line SPS	2015	43973 Talbot Line	0930-99PS6Y	2	228.60	227.15	227.95	228.25	229.45	236.15	Xylem Model NP-3153.181-1370355	25 L/s @ 34 m TDH	Xylem Model NP-3153.181-1370355	25 L/s @ 34 m TDH			Creek to E	No	Overflow is top of PS at 236.15 m	23.12	226.45
14	Wolfe (Hughes St) SPS	1981	4276 Talbot Line	3-0372-79-006	2	217.32	214.44	215.32	215.37	215.57	217.84	ITT Flygt 3127.180	23.66 L/s @ 17.68 m TDH	ITT Flygt 3127.180	23.66 L/s @ 17.68 m TDH			Creek to NE	No		7.3	213.37
15	Woodland SPS	1988	35 Woodland Road	3-0224-87-006	2	204.79	204.05	204.40	204.45	204.47	207.72	Hydromatic Pentair S4LRC	7 L/s @ 33.8 m TDH	Hydromatic Pentair S4LRC	7 L/s @ 33.8 m TDH			Kettle Creek to S of MH1657	No	Overflow is three MH U/S on Woodland Ave	2.63	203.75
16	Woodworth Ave SPS	197?/2010/ 2019	4 Joyce Street	2276-82KM9F	3	222.73	221.73	222.73	222.93	223.43	223.80	Smart Turner 6WMVS	101 L/s @ 13.7 m	Smart Turner 6WMVS	101 L/s @ 13.7 m	Smart Turner 6WMVS	101 L/s @ 13.7 m	N to ditch	Yes	Two pumps can be run in paralell	26.2	220.73

PUMP CURVE DATA

#1 (Assumed)

Head (m)	Flow (L/s)
8.9	56.60
8	61.00
6	75.00
4	88.00
2	105.00

#2 (ITT FLYGT 3170.180)

Head (m)	Flow (L/s)
30	44.00
25	58.00
20	70.00
17	78.00

#3 (Assumed)

Head (m)	Flow (L/s)
14.87	50.47
13	65.00
11	77.00
9	90.00
7	100.00
5	115.00
3	127.00

#4 (Pentair S4LRC)

Head (m)	Flow L/s)
9.54	15.999624
8	20.1888
4.572	30.2832

#5 (Xylem NP3153 MT 3 - 486)

Head (m)	Flow (L/s)
13.1	44.35
12	55.00
10	68.00
8	82.00
6	96.00
4	108.00
2	121.00

#6 (Assumed)

Head (m)	Flow (L/s)
9.1	21.00
8	33.00
6	45.00
4	58.00
2	72.00

#7 (ITT Flygt 3153)

Head (m)	Flow (L/s)
17.68	23.66
16	24.00
14	25.00
12	27.00
10	28.00
8	30.50

#8 (FLYGT 3102)

Head (m)	Flow (L/s)
8	23.00
7	27.00
6	31.50
5	36.00
4	40.00
3	44.00
2	48.00

#9 (FLYGT 3102)

Head (m)	Flow (L/s)
6.4	14.51
5	20.50
4	25.00
3	29.00
2	33.50

#10 (FLYGT 3153)

Head (m)	Flow (L/s)
17	62.70
15	75.00
13	87.00
11	97.00
9	109.00
7	120.00
5	132.00

#11 (Assumed)

Head (m)	Flow (L/s)
37.2	94.60
30	100.00
25	115.00
20	125.00
15	135.00
10	145.00

#12 (Assumed)

Head (m)	Flow (L/s)
8.5	23.00
7	33.00
5	47.00
3	60.00
1	77.00

#13 (Xylem NP-3153.181)

Head (m)	Flow (L/s)
34	25.00
28	30.00
24	34.00
20	37.00
16	42.00
12	44.00

#14 (ITT Flygt 3127.180)

Head (m)	Flow (L/s)
17.68	23.66
16	24.00
14	25.00
12	27.00
10	28.00
8	30.50

#15

Head (m)	Flow (L/s)
33.8	7.00299
28	25.55145
24	36.5922
20	45.4248
16	52.9956
12	61.8282

#16 (Assumed)

Head (m)	Flow (L/s)
13.7	101.00
12	110.00
10	120.00
8	143.00

APPENDIX 2

IDF Station Data for Design Rainfall

St. Thomas WPCP ID: 6137362 General Information

IDF for: ST THOMAS WPCP ID:6137362

Station Info | IDF historical data | IDF under climate change

Station name: **ST THOMAS WPCP**

ID: **6137362**

Latitude: **42.77**

Longitude: **-81.21**

Starting year: **1926**

Ending year: **2016**

Number of years (with data): **82**

St. Thomas IDF Curve (1926-2016)

IDF for: ST THOMAS WPCP ID:6137362

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Tables | Plots | Interpolation Equations

The table below provides coefficients for the interpolation equations fitted to the IDF curve using the GEV distribution.

T (years)	Coefficient A	Coefficient B	Coefficient t_0
2	27.7	-0.812	0.131
5	38.6	-0.823	0.149
10	46.1	-0.824	0.156
20	53.6	-0.824	0.161
25	56.0	-0.823	0.162
50	63.6	-0.821	0.165
100	71.4	-0.818	0.167

Use the coefficients provided in the table above with the following equation:

$$i\left(\frac{mm}{h}\right) = A \cdot (t + t_0)^B$$

Where:

- i is the precipitation intensity rate in $\frac{mm}{h}$
- A, B and t_0, are the coefficients for each return period (T) in years
- t, the time (duration) of the precipitation event in hours (h)

St. Thomas IDF Curve – Climate Change (2050-2100) (Scenario RCP 8.5)

IDF for: ST THOMAS WPCP ID:6137362

Station Info | IDF historical data | IDF under climate change

Climate Model Selection | Scenario RCP 2.6 | Scenario RCP 4.5 | Scenario RCP 8.5 | Comparison Graphs

Tables | Plots | Interpolation Equations | Box Plot - Uncertainty

The table below provides the coefficients for the interpolated equations fitted to the average IDF for future scenario RCP 8.5

T (years)	Coefficient A	Coefficient B	Coefficient t_0
2	33.9	-0.813	0.131
5	47.9	-0.822	0.149
10	56.6	-0.824	0.156
20	66.8	-0.826	0.162
25	70.4	-0.826	0.163
50	80.5	-0.824	0.166
100	91.5	-0.823	0.169

Use the coefficients provided in the table above with the following equation:

$$i\left(\frac{mm}{h}\right) = A \cdot (t + t_0)^B$$

Where:

- i is the precipitation intensity rate in $\frac{mm}{h}$
- A, B and t_0, are the coefficients for each return period (T) in years
- t, the time (duration) of the precipitation event in hours (h)

St. Thomas Total Precipitation (1926-2016)

IDF for: ST THOMAS WPCP ID:6137362

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Tables | Plots | Interpolation Equations

Total precipitation amounts are presented in mm and precipitation intensity rates are presented in mm/h for different return periods (T) presented in years

Total PPT (mm) Intensity rates (mm/h)

T (years)	2	5	10	20	25	50	100
5 min	8.21	10.97	13.00	15.10	15.80	18.07	20.51
10 min	12.29	16.35	19.03	21.58	22.39	24.87	27.31
15 min	14.85	19.99	23.44	26.78	27.84	31.15	34.46
30 min	19.96	27.13	32.22	37.38	39.08	44.49	50.15
1 h	25.85	35.37	41.65	47.66	49.56	55.41	61.19
2 h	30.01	41.84	50.48	59.42	62.40	72.04	82.34
6 h	38.10	51.72	62.08	73.15	76.91	89.33	103.00
12 h	44.19	59.36	70.26	81.42	85.10	96.94	109.43
24 h	49.70	67.02	79.26	91.62	95.68	108.58	122.02

St. Thomas Total Precipitation – Climate Change (2050-2100) (Scenario RCP 8.5)

IDF for: ST THOMAS WPCP ID:6137362

Station Info | IDF historical data | IDF under climate change

Climate Model Selection | Scenario RCP 2.6 | Scenario RCP 4.5 | Scenario RCP 8.5 | Comparison Graphs

Tables | Plots | Interpolation Equations | Box Plot - Uncertainty

Total precipitation amounts presented in mm and precipitation intensity rates presented in mm/h for different return periods (T) presented in years

Total PPT (mm) Intensity rates (mm/h)

T (years)	2	5	10	20	25	50	100
5 min	10.03	13.63	15.97	18.71	19.72	22.65	25.84
10 min	15.04	20.30	23.35	26.89	28.30	31.69	35.39
15 min	18.16	24.82	28.76	33.50	35.12	39.58	44.48
30 min	24.39	33.71	39.56	46.48	48.96	56.04	63.78
1 h	31.63	43.90	51.10	59.72	62.62	70.61	79.29
2 h	36.65	51.97	61.98	73.62	77.85	90.34	103.80
6 h	46.51	64.24	76.26	90.19	95.43	111.22	128.11
12 h	54.00	73.76	86.29	101.11	106.48	121.88	138.70
24 h	60.75	83.29	97.32	113.99	119.95	136.86	155.40

City of Toronto, Old Weston Road Total Precipitation

IDF for: TORONTO OLD WESTON RD ID:6158764

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Tables | Plots | Interpolation Equations

Total precipitation amounts are presented in mm and precipitation intensity rates are presented in mm/h for different return periods (T) presented in years

Total PPT (mm) Intensity rates (mm/h)

T (years)	2	5	10	20	25	50	100
5 min	8.52	11.28	12.95	14.45	14.90	16.24	17.49
10 min	12.11	15.90	18.30	20.53	21.22	23.30	25.31
15 min	14.80	19.86	23.09	26.10	27.03	29.86	32.59
30 min	19.74	26.98	31.51	35.67	36.95	40.79	43.92
1 h	24.03	31.90	35.86	38.93	39.78	42.07	43.92
2 h	28.20	38.54	44.65	50.02	51.63	56.31	60.59
6 h	35.49	46.64	53.90	60.78	62.94	69.55	76.03
12 h	38.42	50.44	59.19	68.24	71.25	80.97	91.33
24 h	41.84	54.40	63.38	72.52	75.54	85.18	95.31

City of Toronto, Toronto City Total Precipitation

IDF for: TORONTO CITY ID:6158355

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Tables | Plots | Interpolation Equations

Total precipitation amounts are presented in mm and precipitation intensity rates are presented in mm/h for different return periods (T) presented in years

Total PPT (mm) Intensity rates (mm/h)

T (years)	2	5	10	20	25	50	100
5 min	8.65	11.88	14.41	17.18	18.13	21.34	24.94
10 min	12.28	16.23	19.22	22.39	23.46	27.00	30.87
15 min	14.63	19.92	24.08	28.65	30.22	35.53	41.51
30 min	18.98	26.44	32.03	37.94	39.93	46.47	53.58
1 h	23.73	32.78	39.12	45.49	47.57	54.16	60.99
2 h	27.61	37.94	45.54	53.46	56.10	64.69	73.91
6 h	33.62	45.16	54.21	64.13	67.56	79.06	92.03
12 h	40.22	53.09	62.78	73.05	76.53	87.94	100.40
24 h	45.63	59.35	69.21	79.29	82.63	93.32	104.60

City of Toronto, Toronto Island Total Precipitation

IDF for: TORONTO ISLAND A ID:6158665

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Tables | Plots | Interpolation Equations

Total precipitation amounts are presented in mm and precipitation intensity rates are presented in mm/h for different return periods (T) presented in years

◉ Total PPT (mm) ○ Intensity rates (mm/h)

T (years)	2	5	10	20	25	50	100
5 min	8.07	11.42	13.95	16.64	17.56	20.57	23.87
10 min	12.14	16.36	19.45	22.63	23.69	27.13	30.79
15 min	15.12	20.69	24.82	29.16	30.62	35.37	40.51
30 min	19.62	27.38	33.09	39.02	41.01	47.44	54.35
1 h	24.40	34.53	41.69	48.91	51.27	58.80	66.64
2 h	28.74	38.71	45.53	52.23	54.39	61.15	68.01
6 h	35.74	47.09	54.73	62.17	64.55	71.94	79.37
12 h	39.70	51.51	59.55	67.43	69.33	74.69	79.69
24 h	44.19	54.99	61.59	67.53	69.33	74.69	79.69

City of Toronto, Toronto Booth Total Precipitation

IDF for: TORONTO BOOTH ID:6158406

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Tables | Plots | Interpolation Equations

Total precipitation amounts are presented in mm and precipitation intensity rates are presented in mm/h for different return periods (T) presented in years

◉ Total PPT (mm) ○ Intensity rates (mm/h)

T (years)	2	5	10	20	25	50	100
5 min	8.61	11.48	13.22	14.77	15.24	16.62	17.90
10 min	12.02	16.43	19.28	21.96	22.80	25.36	27.85
15 min	14.36	20.37	24.94	29.82	31.47	36.94	42.95
30 min	18.17	25.46	31.58	38.66	41.18	49.96	60.38
1 h	20.96	30.40	38.56	48.21	51.71	64.03	78.97
2 h	25.65	36.91	46.09	56.45	60.10	72.57	87.03
6 h	33.72	47.33	57.89	69.36	73.30	84.56	95.15
12 h	38.52	52.01	61.57	71.25	74.43	84.56	95.15
24 h	44.26	58.53	68.35	78.05	81.18	91.03	101.08

APPENDIX 3

Climate Change Assessment

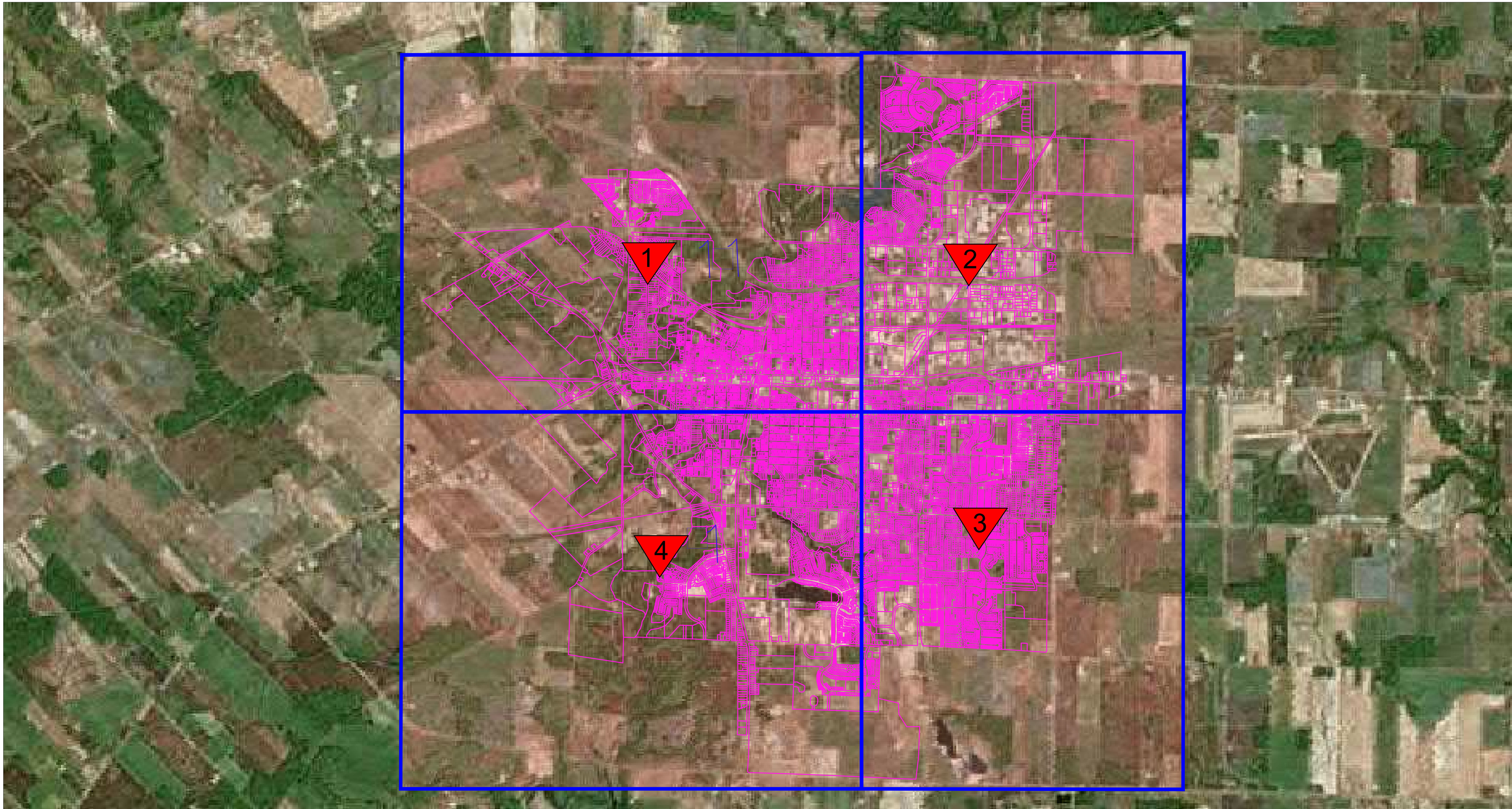
St. Thomas WPCP ID: 6137362 - Total Precipitation Comparison

	T(years)	2-Year (CC)**	2-Year***	% Change	5-Year (CC)**	5-Year***	% Change	10-Year (CC)**	10-Year***	% Change	20-Year (CC)**	20-Year***	% Change	25-Year (CC)**	25-Year***	% Change	50-Year (CC)**	50-Year***	% Change	100-Year (CC)**	100-Year***	% Change
Duration	5 min	10.03	8.21	22.17	13.63	10.97	24.25	15.97	13.00	22.85	18.71	15.10	23.91	19.72	15.80	24.81	22.65	18.07	25.35	25.84	20.51	25.99
	10 min	15.04	12.29	22.38	20.30	16.35	24.16	23.35	19.03	22.70	26.89	21.58	24.61	28.30	22.39	26.40	31.69	24.87	27.42	35.39	27.31	29.59
	15 min	18.16	14.85	22.29	24.82	19.99	24.16	28.76	23.44	22.70	33.50	26.78	25.09	35.12	27.84	26.15	39.58	31.15	27.06	44.48	34.46	29.08
	30 min	24.39	19.96	22.19	33.71	27.13	24.25	39.56	32.22	22.78	46.48	37.38	24.34	48.96	39.08	25.28	56.04	44.49	25.96	63.78	50.15	27.18
	1 h	31.63	25.85	22.36	43.90	35.37	24.12	51.10	41.65	22.69	59.72	47.66	25.30	62.62	49.56	26.35	70.61	55.41	27.43	79.29	61.19	29.58
	2 h	36.65	30.01	22.13	51.97	41.84	24.21	61.98	50.48	22.78	73.62	59.42	23.90	77.85	62.40	24.76	90.34	72.04	25.40	103.80	82.34	26.06
	6 h	46.51	38.10	22.07	64.24	51.72	24.21	76.26	62.08	22.84	90.19	73.15	23.29	95.43	76.91	24.08	111.22	89.33	24.50	128.11	103	24.38
	12 h	54.00	44.19	22.20	73.76	59.36	24.26	86.29	70.26	22.82	101.11	81.42	24.18	106.48	85.10	25.12	121.88	96.94	25.73	138.70	109.43	26.75
	24 h	60.75	49.70	22.23	83.29	67.02	24.28	97.32	79.26	22.79	113.99	91.62	24.42	119.95	95.68	25.37	136.86	108.58	26.05	155.40	122.02	27.36
Average % Change		2-Year Increase %		22%	5-Year Increase %		24%	10-Year Increase %		23%	20-Year Increase %		24%	25-Year Increase %		25%	50-Year Increase %		26%	100-Year Increase %		27%

*Total Precipitation is in mm.
**Scenario RCP 8.5 (Worst-Case Climate Change Scenario based on 2050-2100)
***Precipitation Data Based on Rainfall data during 1926-2016

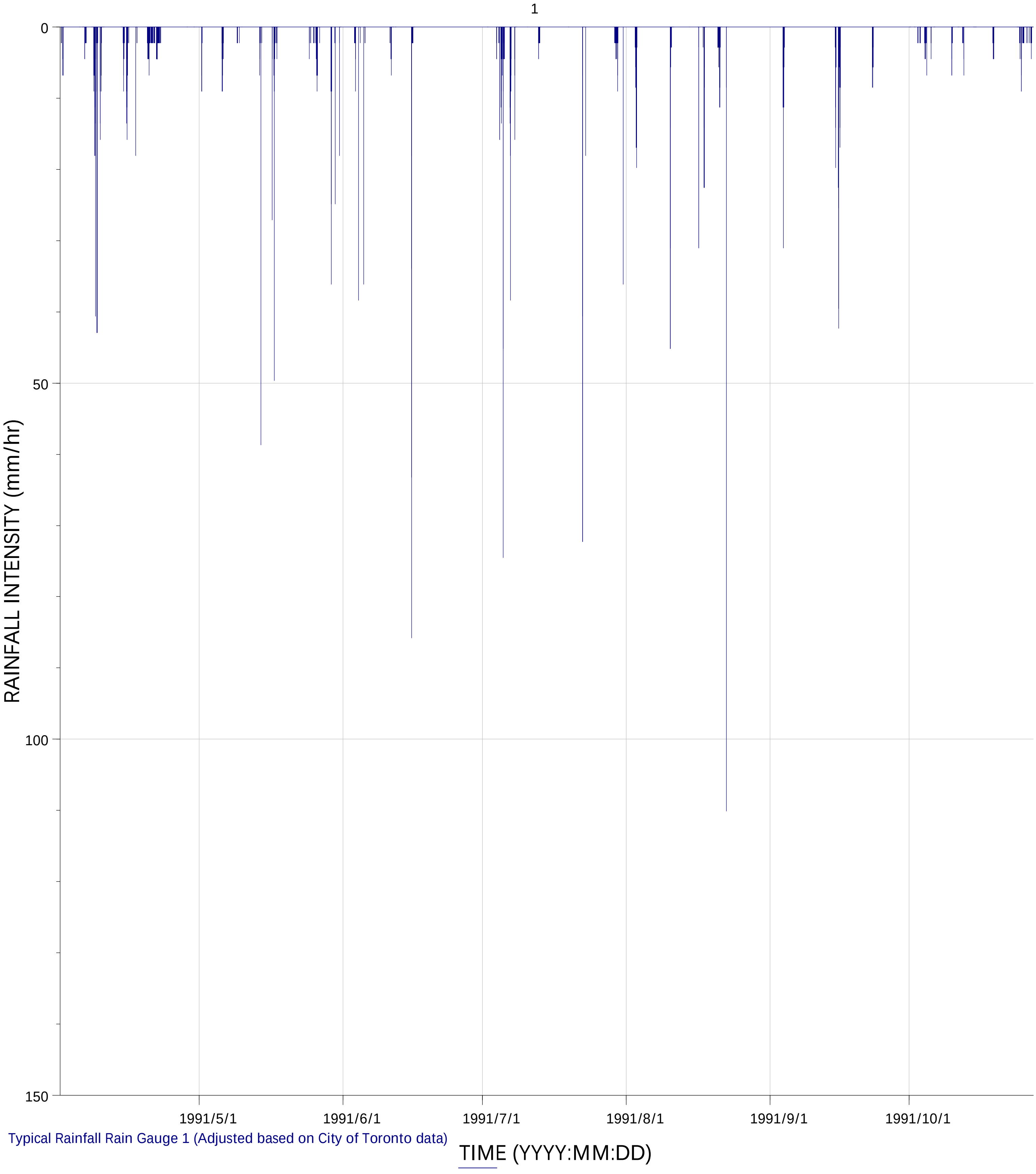
APPENDIX 4

Rainfall Series, Typical Year (incl. Conversion from Toronto)

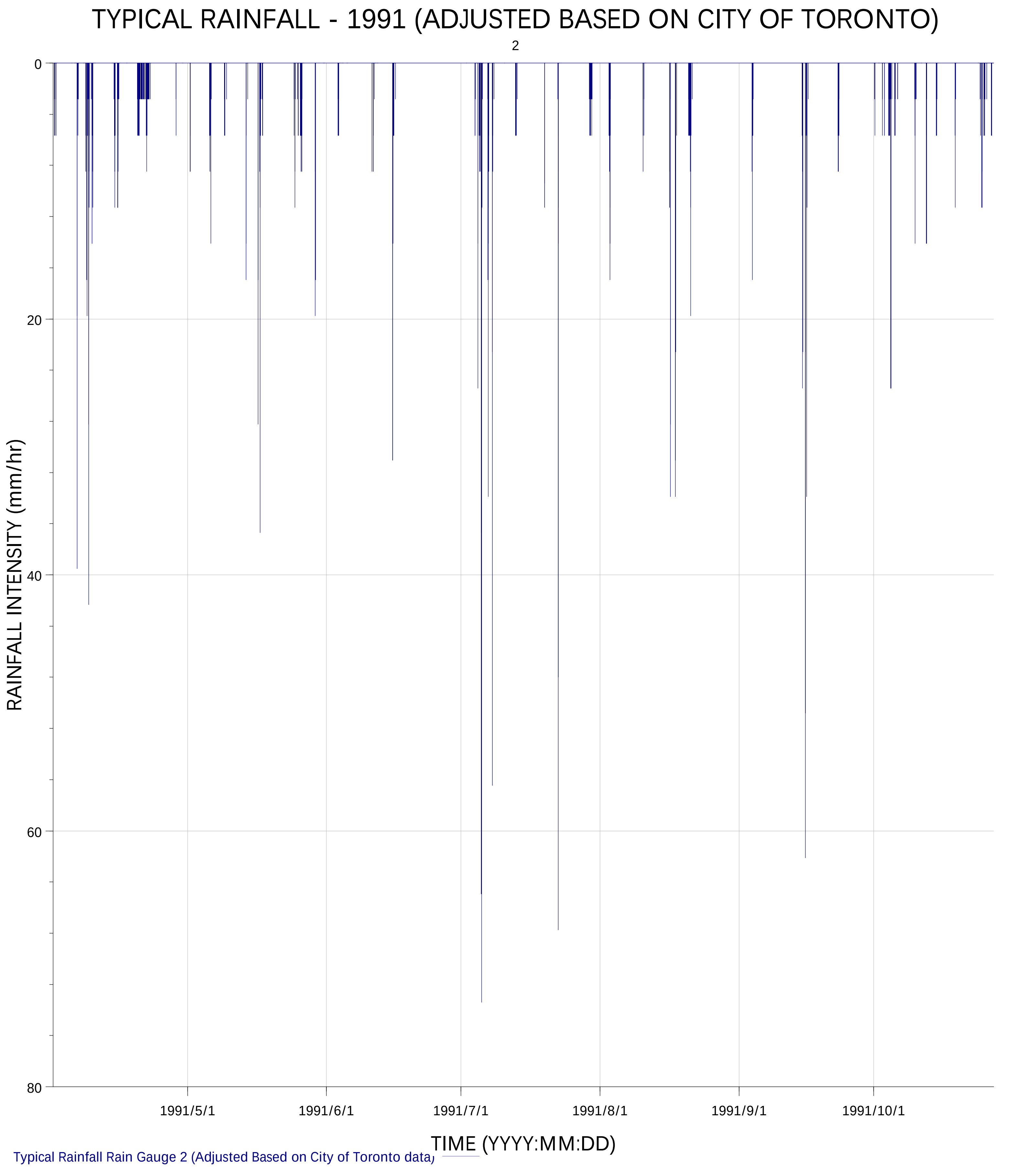


RAIN GAUGE LOCATIONS	LEGEND RAIN GAUGE ID CATCHMENT BOUNDARY UNDER RAIN GAUGE	FIGURE 1	PROJECT NO. 205153 2021-07-30	arva

TYPICAL RAINFALL - 1991 (ADJUSTED BASED ON CITY OF TORONTO)



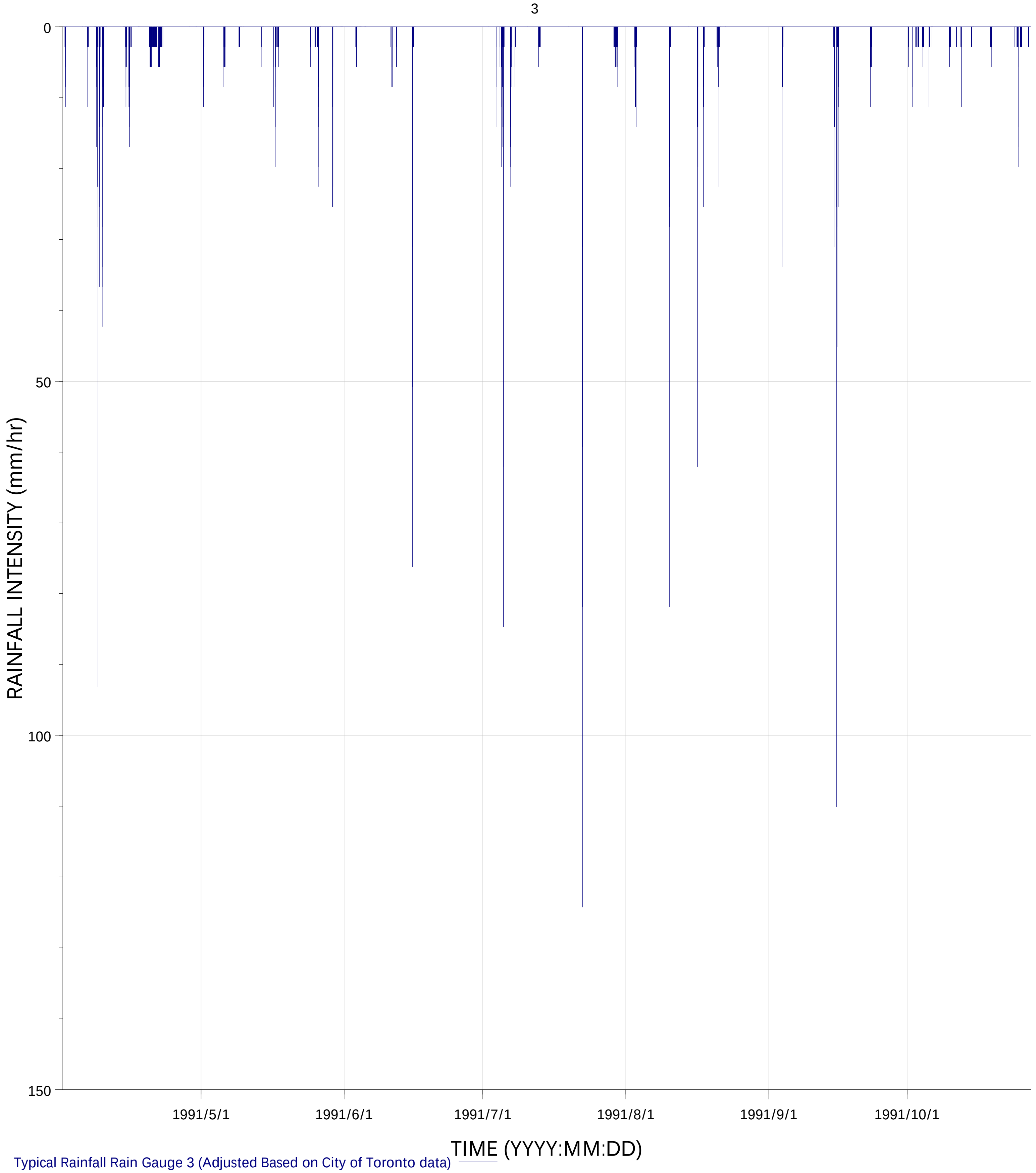
RAIN GAUGE 1



RAIN GAUGE 2

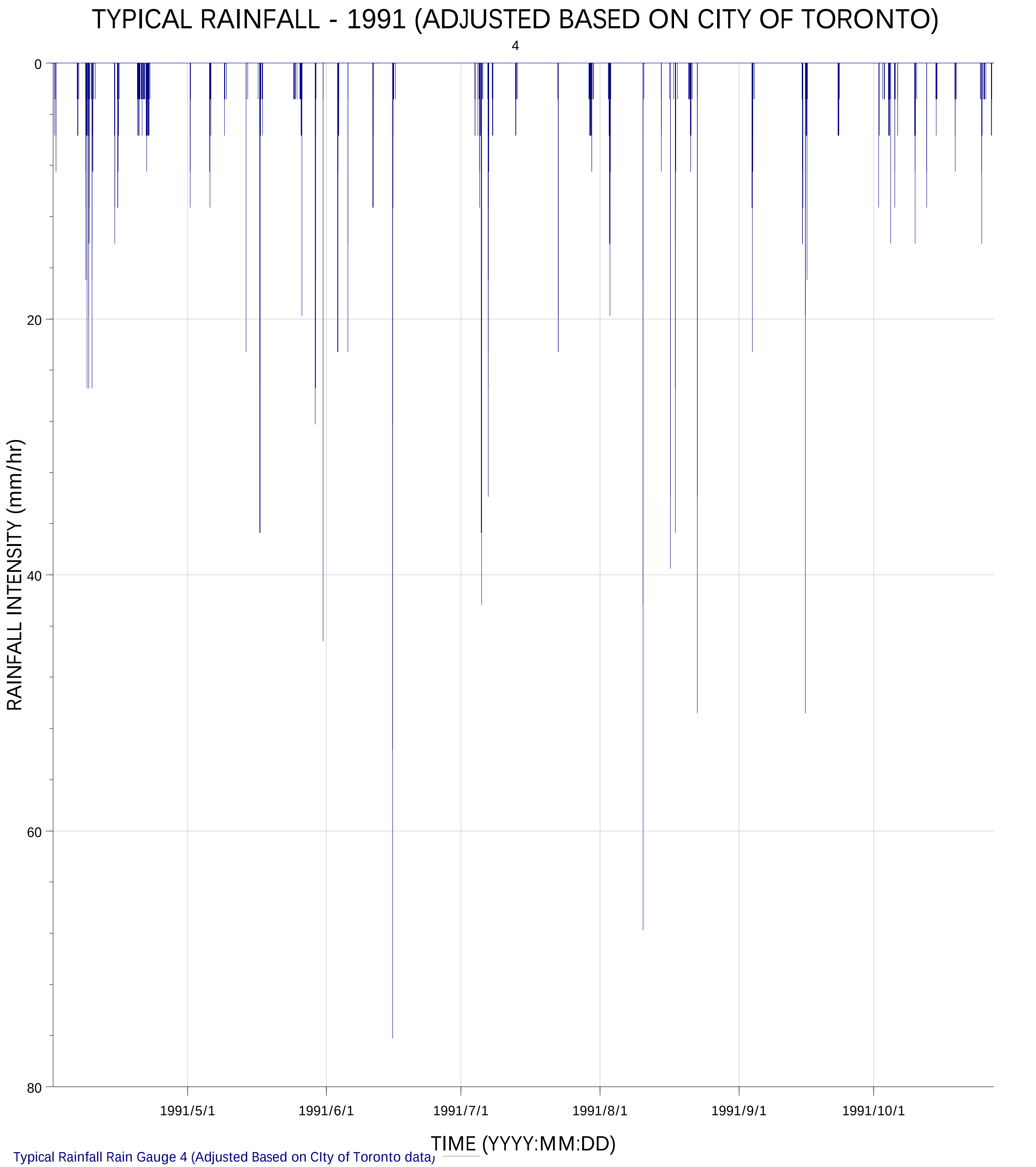
arva

TYPICAL RAINFALL - 1991 (ADJUSTED BASED ON CITY OF TORONTO)



RAIN GAUGE 3

arva



TYPICAL RAINFALL - 1991 (ADJUSTED BASED ON CITY OF TORONTO)
4
RAINFALL INTENSITY (mm/hr)
0
20
40
60
80
1991/5/1
1991/6/1
1991/7/1
1991/8/1
1991/9/1
1991/10/1
TIME (YYYY:MM:DD)
Typical Rainfall Rain Gauge 4 (Adjusted Based on CIty of Toronto data)

RAIN GAUGE 4

APPENDIX 5

Dry Weather Pattern for 2020 Flow Monitors (diurnal curves)

350 Talbot - Weekday Data (2017)

Change Data Source of Pivot table

Row Labels	Average of Flow (l/s)
12 AM	3.30
1 AM	2.60
2 AM	2.79
3 AM	2.79
4 AM	3.06
5 AM	3.74
6 AM	5.16
7 AM	6.71
8 AM	6.53
9 AM	6.41
10 AM	5.83
11 AM	5.74
12 PM	5.60
1 PM	5.16
2 PM	5.02
3 PM	5.60
4 PM	5.56
5 PM	5.90
6 PM	5.93
7 PM	6.40
8 PM	6.41
9 PM	5.79
10 PM	4.48
11 PM	4.09
Average	**5.03**

Trade Flow (l/s): 0.29

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	3.30	1.98	1.32	0	0	1.32	45%	0.45
1	2.60	1.98	0.63	0	0	0.63	21%	0.21
2	2.79	1.98	0.81	0	0	0.81	28%	0.28
3	2.79	1.98	0.81	0	0	0.81	28%	0.28
4	3.06	1.98	1.08	0	0	1.08	37%	0.37
5	3.74	1.98	1.76	0	0	1.76	60%	0.60
6	5.16	1.98	3.18	0	0	3.18	109%	1.09
7	6.71	1.98	4.73	1	0.29	4.44	152%	1.52
8	6.53	1.98	4.55	1	0.29	4.26	146%	1.46
9	6.41	1.98	4.43	1	0.29	4.14	142%	1.42
10	5.83	1.98	3.85	1	0.29	3.56	122%	1.22
11	5.74	1.98	3.76	1	0.29	3.47	119%	1.19
12	5.60	1.98	3.62	1	0.29	3.33	114%	1.14
13	5.16	1.98	3.18	1	0.29	2.89	99%	0.99
14	5.02	1.98	3.04	1	0.29	2.75	94%	0.94
15	5.60	1.98	3.62	1	0.29	3.33	114%	1.14
16	5.56	1.98	3.58	1	0.29	3.29	113%	1.13
17	5.90	1.98	3.92	1	0.29	3.63	125%	1.25
18	5.93	1.98	3.95		0	3.95	136%	1.36
19	6.40	1.98	4.42	0	0	4.42	152%	1.52
20	6.41	1.98	4.43	0	0	4.43	152%	1.52
21	5.79	1.98	3.81	0	0	3.81	131%	1.31
22	4.48	1.98	2.50	0	0	2.50	86%	0.86
23	4.09	1.98	2.11	0	0	2.11	72%	0.72
Average	5.02				0.132916667	2.91	1.00	1.00
Min	2.60							
Max.	6.71							

Weekday

Min. total flow	2.60	
Average total flow	5.02	
Daily average flow [L/d]	434,108.97	
Subtract trade flows [L/d]	11,484.00	
Residential flow [L/d]	422,624.97	
Metered Pop	1,375.00	
GWI %	76.00%	
GWI [L/s]= % x Min DWF	1.98	
GWI [L/d]	171,034.84	
Per Capita flow [L/c/d]	182.97	
GWI [L/ha/s]	0.133304372	14.85 ha

Put 80% GWI where Min Flow is lower

Adjust so that Flow-GWI is not super low in the table

Resuling GWI / (other set of data's minimum) = GWI% of that other set of data

Choose: e.g. right if you start adjusting at right side

right / left 75.46%

left/right 80.57%

350 Talbot - Weekend Data (2017)

Change Data Source of Pivot table

Row Labels	Average of Flow
12 AM	3.76
1 AM	3.21
2 AM	3.37
3 AM	2.46
4 AM	2.66
5 AM	2.90
6 AM	3.29
7 AM	4.79
8 AM	6.24
9 AM	8.14
10 AM	7.21
11 AM	7.08
12 PM	6.77
1 PM	5.57
2 PM	5.71
3 PM	5.46
4 PM	5.39
5 PM	5.67
6 PM	6.36
7 PM	5.95
8 PM	5.68
9 PM	5.15
10 PM	4.88
11 PM	3.34
Average	**5.04**

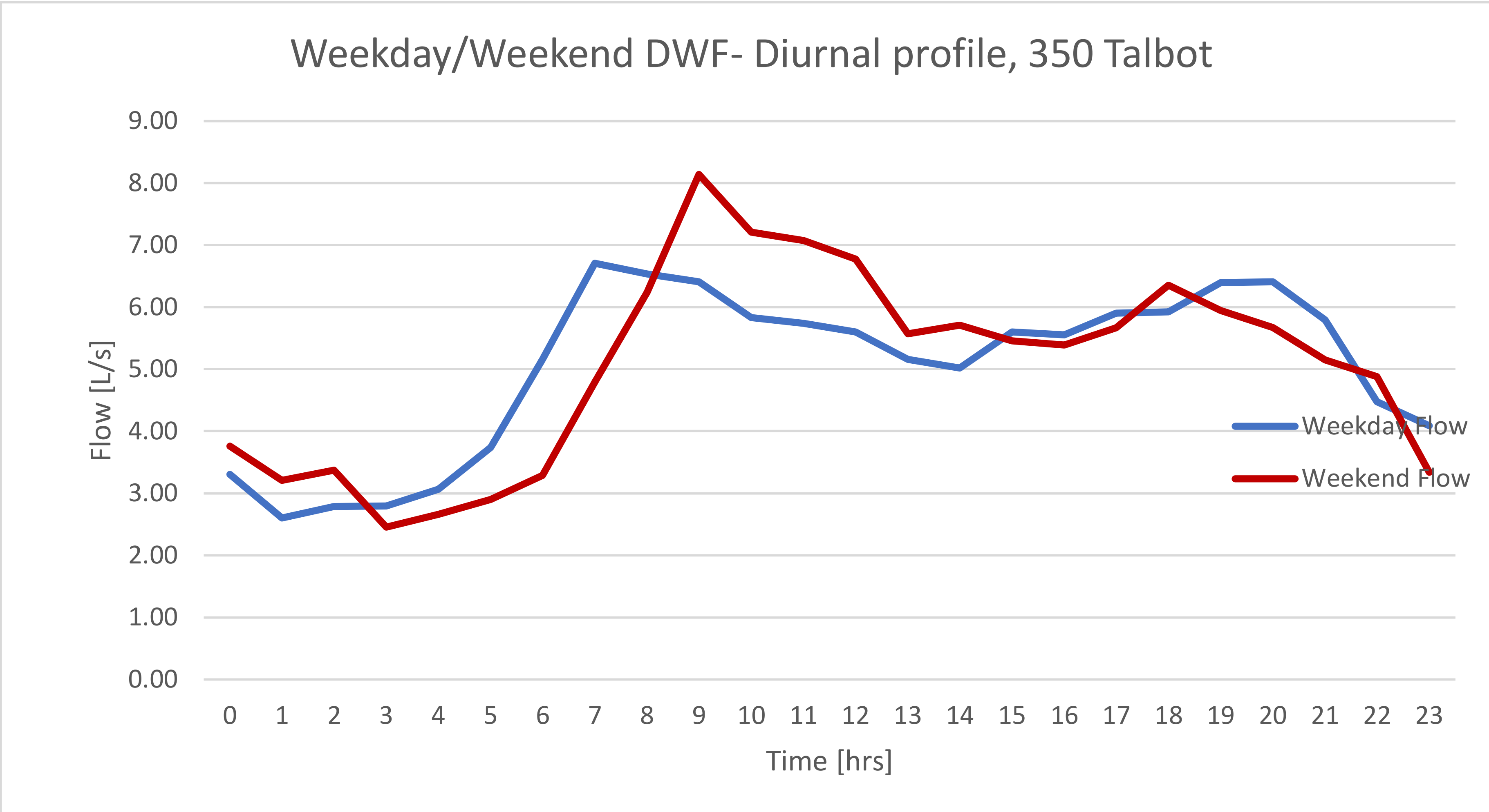
Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	3.76	1.97	1.80	58%	58%	0.62
1	3.21	1.97	1.24	40%	40%	0.43
2	3.37	1.97	1.41	46%	46%	0.48
3	2.46	1.97	0.49	16%	16%	0.17
4	2.66	1.97	0.70	23%	23%	0.24
5	2.90	1.97	0.94	30%	30%	0.32
6	3.29	1.97	1.32	43%	43%	0.45
7	4.79	1.97	2.82	92%	92%	0.97
8	6.24	1.97	4.27	139%	139%	1.47
9	8.14	1.97	6.17	201%	201%	2.12
10	7.21	1.97	5.24	170%	170%	1.80
11	7.08	1.97	5.11	166%	166%	1.76
12	6.77	1.97	4.81	156%	156%	1.65
13	5.57	1.97	3.60	117%	117%	1.24
14	5.71	1.97	3.74	122%	122%	1.29
15	5.46	1.97	3.49	114%	114%	1.20
16	5.39	1.97	3.42	111%	111%	1.18
17	5.67	1.97	3.71	120%	120%	1.27
18	6.36	1.97	4.39	143%	143%	1.51
19	5.95	1.97	3.98	129%	129%	1.37
20	5.68	1.97	3.71	121%	121%	1.27
21	5.15	1.97	3.18	103%	103%	1.09
22	4.88	1.97	2.91	95%	95%	1.00
23	3.34	1.97	1.37	45%	45%	0.47
Adjust factor for weekday PCD use in model:						1.06
Average	**5.04**		**3.08**	**1.00**	**1.00**	**1.06**
Min	**2.46**					
Max	**8.14**					

Weekend

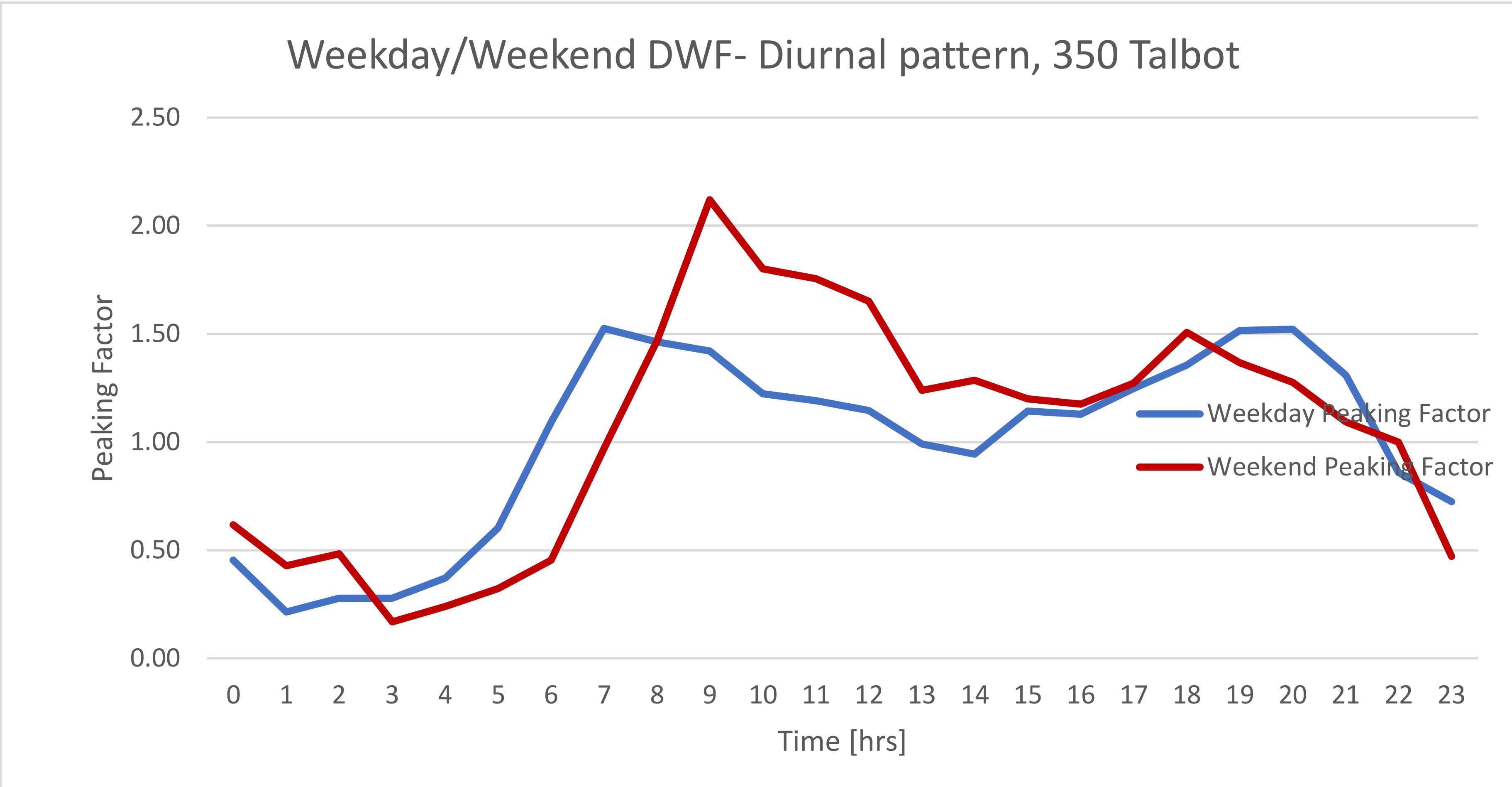
Min.	2.46	
Average	5.04	
Daily average flow [L/d]	435,712.18	
Subtract trade flows [L/d] set t	0.00	
Residential flow [L/d]	435,712.18	
Metered Pop	**1,375.00**	
GWI %	80.00%	
GWI [L/s]= % x Min DWF	1.97	
GWI [L/d]	169823.95	
Per Capita flow [L/c/d]	193.37	
GWI [L/ha/s]	0.13	14.85 ha

arva

350 Talbot - Diurnal Profile (2017)



350 Talbot - Diurnal Pattern (2017)



arva

378 Talbot - Weekday Data (2017)

Change Data Source of Pivot table

Row Labels	Average of Flow (l/s)
12 AM	7.85
1 AM	6.46
2 AM	6.17
3 AM	5.90
4 AM	6.55
5 AM	7.72
6 AM	10.22
7 AM	13.25
8 AM	13.64
9 AM	13.84
10 AM	13.77
11 AM	14.46
12 PM	13.53
1 PM	13.05
2 PM	12.92
3 PM	13.49
4 PM	13.32
5 PM	14.05
6 PM	14.07
7 PM	14.90
8 PM	14.57
9 PM	13.49
10 PM	10.96
11 PM	9.56
Average	**11.57**

Trade Flow (l/s): 2.3

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	7.85	3.03	4.82	0	0	4.82	64%	0.64
1	6.46	3.03	3.43	0	0	3.43	46%	0.46
2	6.17	3.03	3.14	0	0	3.14	42%	0.42
3	5.90	3.03	2.87	0	0	2.87	38%	0.38
4	6.55	3.03	3.52	0	0	3.52	47%	0.47
5	7.72	3.03	4.68	0	0	4.68	63%	0.63
6	10.22	3.03	7.18	0	0	7.18	96%	0.96
7	13.25	3.03	10.22	1	2.3	7.92	106%	1.06
8	13.64	3.03	10.61	1	2.3	8.31	111%	1.11
9	13.84	3.03	10.81	1	2.3	8.51	114%	1.14
10	13.77	3.03	10.73	1	2.3	8.43	113%	1.13
11	14.46	3.03	11.42	1	2.3	9.12	122%	1.22
12	13.53	3.03	10.50	1	2.3	8.20	110%	1.10
13	13.05	3.03	10.02	1	2.3	7.72	103%	1.03
14	12.92	3.03	9.89	1	2.3	7.59	101%	1.01
15	13.49	3.03	10.46	1	2.3	8.16	109%	1.09
16	13.32	3.03	10.29	1	2.3	7.99	107%	1.07
17	14.05	3.03	11.02	1	2.3	8.72	116%	1.16
18	14.07	3.03	11.04	0	0	11.04	147%	1.47
19	14.90	3.03	11.86	0	0	11.86	159%	1.59
20	14.57	3.03	11.53	0	0	11.53	154%	1.54
21	13.49	3.03	10.45	0	0	10.45	140%	1.40
22	10.96	3.03	7.93	0	0	7.93	106%	1.06
23	9.56	3.03	6.53	0	0	6.53	87%	0.87
Average	11.57				1.054166667	7.48	1.00	1.00
Min	5.90							
Max.	14.90							

Weekday

Min. total flow	5.90
Average total flow	11.57
Daily average flow [L/d]	999,831.08
Subtract trade flows [L/d]	91,080.00
Residential flow [L/d]	908,751.08
Metered Pop	**2,106.00**
GWI %	51.39%
GWI [L/s]= % x Min DWF	3.03
GWI [L/d]	262,051.96
Per Capita flow [L/c/d]	307.07
GWI [L/ha/s]	0.097524398

31.1 ha

Put 80% GWI where Min Flow is lower	Choose: e.g. right if you start adjusting at right side
Adjust so that Flow-GWI is not super low in the table	right / left 58.64%
Resuling GWI / (other set of data's minimum) = GWI% of that other set of data	left/right 52.58%

378 Talbot - Weekend Data (2017)

Change Data Source of Pivot table

Row Labels	Average of Flow
12 AM	9.41
1 AM	7.76
2 AM	7.35
3 AM	5.96
4 AM	5.77
5 AM	6.45
6 AM	7.41
7 AM	10.70
8 AM	13.19
9 AM	15.80
10 AM	15.55
11 AM	15.84
12 PM	15.98
1 PM	14.70
2 PM	15.17
3 PM	14.53
4 PM	13.53
5 PM	14.48
6 PM	16.76
7 PM	14.39
8 PM	13.54
9 PM	13.29
10 PM	12.04
11 PM	9.41
Average	**12.04**

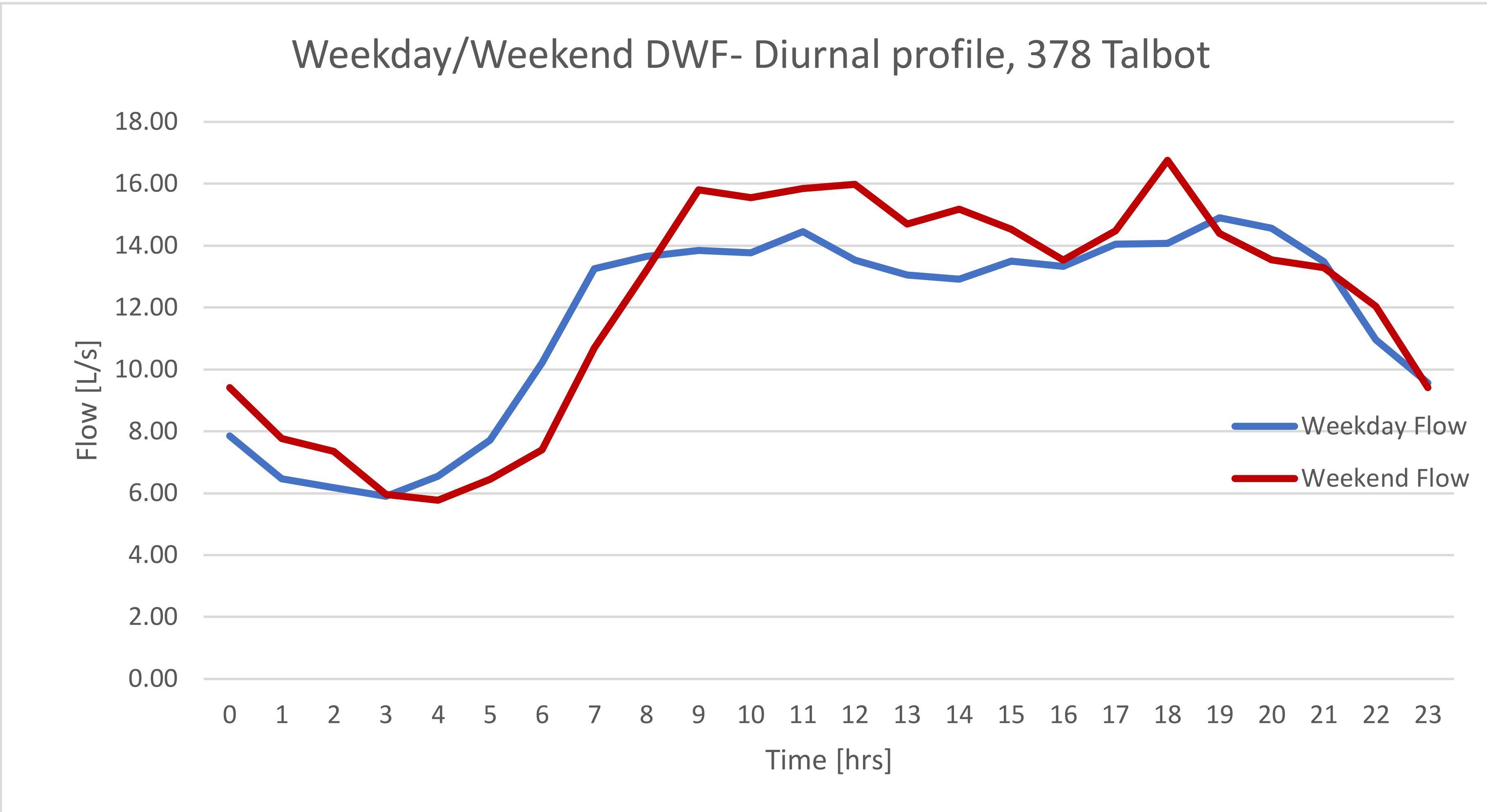
Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	9.41	3.46	5.95	69%	69%	0.79
1	7.76	3.46	4.30	50%	50%	0.57
2	7.35	3.46	3.89	45%	45%	0.52
3	5.96	3.46	2.50	29%	29%	0.33
4	5.77	3.46	2.31	27%	27%	0.31
5	6.45	3.46	2.99	35%	35%	0.40
6	7.41	3.46	3.94	46%	46%	0.53
7	10.70	3.46	7.24	84%	84%	0.97
8	13.19	3.46	9.73	113%	113%	1.30
9	15.80	3.46	12.34	144%	144%	1.65
10	15.55	3.46	12.09	141%	141%	1.62
11	15.84	3.46	12.38	144%	144%	1.65
12	15.98	3.46	12.52	146%	146%	1.67
13	14.70	3.46	11.24	131%	131%	1.50
14	15.17	3.46	11.71	136%	136%	1.56
15	14.53	3.46	11.07	129%	129%	1.48
16	13.53	3.46	10.07	117%	117%	1.35
17	14.48	3.46	11.02	128%	128%	1.47
18	16.76	3.46	13.30	155%	155%	1.78
19	14.39	3.46	10.93	127%	127%	1.46
20	13.54	3.46	10.08	117%	117%	1.35
21	13.29	3.46	9.83	114%	114%	1.31
22	12.04	3.46	8.57	100%	100%	1.15
23	9.41	3.46	5.95	69%	69%	0.79
Adjust factor for weekday PCD use in model:						1.15
Average	**12.04**		**8.58**	**1.00**	**1.00**	**1.15**
Min	**5.77**					
Max	**16.76**					

Weekend	
Min.	5.77
Average	12.04
Daily average flow [L/d]	1,040,453.79
Subtract trade flows [L/d] set t	0.00
Residential flow [L/d]	1,040,453.79
Metered Pop	**2,106.00**
GWI %	60.00%
GWI [L/s]= % x Min DWF	3.46
GWI [L/d]	299038.37
Per Capita flow [L/c/d]	352.05
GWI [L/ha/s]	0.11

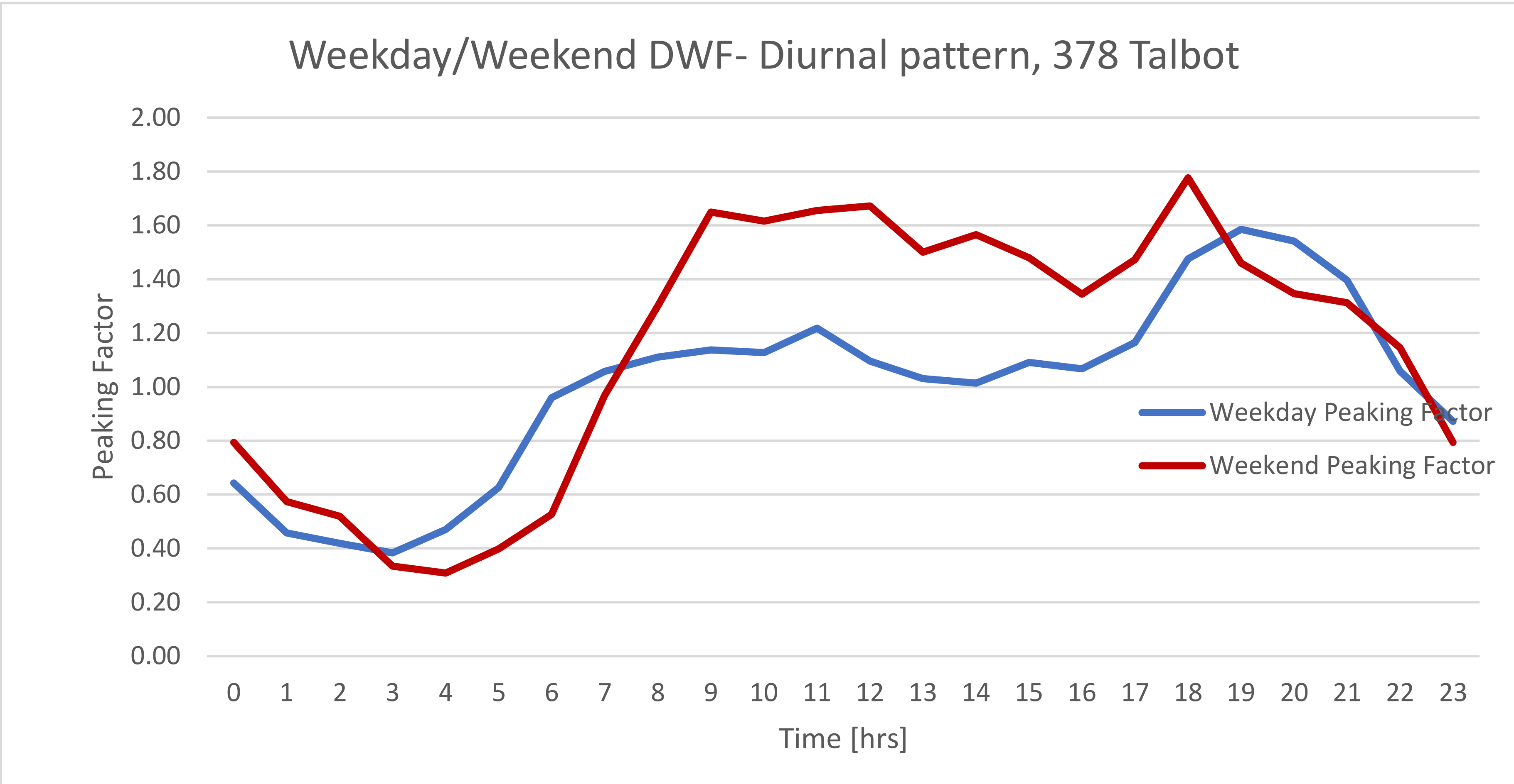
31.1 ha

arva

378 Talbot - Diurnal Profile (2017)



378 Talbot - Diurnal Pattern (2017)



378 Talbot - Weekday Data: 378 Talbot - 350 Talbot (remaining area) (2017)

Trade Flow (l/s): 2.01

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	4.55	2.64	1.90	0	0	1.90	25%	0.64
1	3.85	2.64	1.21	0	0	1.21	16%	0.41
2	3.38	2.64	0.74	0	0	0.74	10%	0.25
3	3.11	2.64	0.47	0	0	0.47	6%	0.16
4	3.49	2.64	0.85	0	0	0.85	11%	0.28
5	3.98	2.64	1.33	0	0	1.33	18%	0.45
6	5.06	2.64	2.42	0	0	2.42	32%	0.81
7	6.54	2.64	3.89	1	2.01	1.88	25%	0.63
8	7.11	2.64	4.47	1	2.01	2.46	33%	0.82
9	7.43	2.64	4.79	1	2.01	2.78	37%	0.93
10	7.94	2.64	5.29	1	2.01	3.28	44%	1.10
11	8.72	2.64	6.08	1	2.01	4.07	54%	1.36
12	7.93	2.64	5.29	1	2.01	3.28	44%	1.10
13	7.89	2.64	5.25	1	2.01	3.24	43%	1.09
14	7.90	2.64	5.26	1	2.01	3.25	43%	1.09
15	7.89	2.64	5.25	1	2.01	3.24	43%	1.09
16	7.77	2.64	5.12	1	2.01	3.11	42%	1.04
17	8.15	2.64	5.50	1	2.01	3.49	47%	1.17
18	8.15	2.64	5.50	0	0	5.50	74%	1.84
19	8.50	2.64	5.86	0	0	5.86	78%	1.96
20	8.16	2.64	5.52	0	0	5.52	74%	1.85
21	7.69	2.64	5.05	0	0	5.05	67%	1.69
22	6.48	2.64	3.84	0	0	3.84	51%	1.29
23	5.47	2.64	2.83	0	0	2.83	38%	0.95
Average	**6.55**				**0.92125**	**2.98**	**0.40**	**1.00**
Min	**3.11**							
Max.	**8.72**							

Weekday		
Min. total flow	3.11	
Average total flow	6.55	
Daily average flow [L/d]	565,722.11	
Subtract trade flows [L/d]	79,596.00	
Residential flow [L/d]	486,126.11	
Metered Pop	**731.00**	
GWI %	85.00%	
GWI [L/s]= % x Min DWF	2.64	
GWI [L/d]	228,395.53	
Per Capita flow [L/c/d]	352.57	
GWI [L/ha/s]	0.16267488	16.25 ha

378 Talbot - Weekend Data: 378 Talbot - 350 Talbot (remaining area) (2017)

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	5.65	1.50	4.15	48%	75%	1.10
1	4.55	1.50	3.05	36%	55%	0.81
2	3.98	1.50	2.48	29%	45%	0.66
3	3.51	1.50	2.01	23%	37%	0.53
4	3.10	1.50	1.61	19%	29%	0.43
5	3.54	1.50	2.05	24%	37%	0.54
6	4.12	1.50	2.62	31%	48%	0.70
7	5.91	1.50	4.41	51%	80%	1.17
8	6.96	1.50	5.46	64%	99%	1.45
9	7.66	1.50	6.17	72%	112%	1.64
10	8.35	1.50	6.85	80%	124%	1.82
11	8.77	1.50	7.27	85%	132%	1.93
12	9.21	1.50	7.71	90%	140%	2.05
13	9.13	1.50	7.63	89%	139%	2.03
14	9.46	1.50	7.97	93%	145%	2.12
15	9.07	1.50	7.58	88%	138%	2.01
16	8.14	1.50	6.65	77%	121%	1.77
17	8.81	1.50	7.32	85%	133%	1.94
18	10.40	1.50	8.91	104%	162%	2.37
19	8.44	1.50	6.95	81%	126%	1.85
20	7.86	1.50	6.37	74%	116%	1.69
21	8.14	1.50	6.64	77%	121%	1.76
22	7.16	1.50	5.66	66%	103%	1.50
23	6.07	1.50	4.57	53%	83%	1.22
Adjust factor for weekday PCD use in model:						1.46
Average	**7.00**		**5.50**	**0.64**	**1.00**	**1.46**
Min	**3.10**					
Max	**10.40**					

Weekend		
Min.	3.10	
Average	7.00	
Daily average flow [L/d]	604,741.61	
Subtract trade flows [L/d] set t	0.00	
Residential flow [L/d]	604,741.61	
Metered Pop	**731.00**	
GWI %	85.00%	
GWI [L/s]= % x Min DWF	2.64	
GWI [L/d]	228012.20	
Per Capita flow [L/c/d]	515.36	
GWI [L/ha/s]	0.16	16.25 ha

arva

Axford(2) - Weekday Data (2017)

Change Data Source of Pivot table

Row Labels	Average of Flow (l/s)
12 AM	2.67
1 AM	2.08
2 AM	1.86
3 AM	1.79
4 AM	2.15
5 AM	4.41
6 AM	7.45
7 AM	8.53
8 AM	6.81
9 AM	6.13
10 AM	5.83
11 AM	5.23
12 PM	4.91
1 PM	4.56
2 PM	4.64
3 PM	4.96
4 PM	5.58
5 PM	6.50
6 PM	7.96
7 PM	8.36
8 PM	7.60
9 PM	6.34
10 PM	5.29
11 PM	3.89
Average	**5.23**

Trade Flow (l/s): 0.09

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	2.67	1.45	1.22	0	0	1.22	33%	0.33
1	2.08	1.45	0.63	0	0	0.63	17%	0.17
2	1.86	1.45	0.41	0	0	0.41	11%	0.11
3	1.79	1.45	0.34	0	0	0.34	9%	0.09
4	2.15	1.45	0.70	0	0	0.70	19%	0.19
5	4.41	1.45	2.96	0	0	2.96	79%	0.79
6	7.45	1.45	6.00	0	0	6.00	161%	1.61
7	8.53	1.45	7.08	1	0.09	6.99	187%	1.87
8	6.81	1.45	5.36	1	0.09	5.27	141%	1.41
9	6.13	1.45	4.68	1	0.09	4.59	123%	1.23
10	5.83	1.45	4.38	1	0.09	4.29	115%	1.15
11	5.23	1.45	3.78	1	0.09	3.69	99%	0.99
12	4.91	1.45	3.46	1	0.09	3.37	90%	0.90
13	4.56	1.45	3.11	1	0.09	3.02	81%	0.81
14	4.64	1.45	3.19	1	0.09	3.10	83%	0.83
15	4.96	1.45	3.51	1	0.09	3.42	92%	0.92
16	5.58	1.45	4.13	1	0.09	4.04	108%	1.08
17	6.50	1.45	5.05	1	0.09	4.96	133%	1.33
18	7.96	1.45	6.51	0	0	6.51	174%	1.74
19	8.36	1.45	6.91	0	0	6.91	185%	1.85
20	7.60	1.45	6.15	0	0	6.15	165%	1.65
21	6.34	1.45	4.89	0	0	4.89	131%	1.31
22	5.29	1.45	3.84	0	0	3.84	103%	1.03
23	3.89	1.45	2.44	0	0	2.44	65%	0.65
Average	5.23				0.04125	3.74	1.00	1.00
Min	1.79							
Max.	8.53							

Weekday

Min. total flow	1.79
Average total flow	5.23
Daily average flow [L/d]	451,906.74
Subtract trade flows [L/d]	3,564.00
Residential flow [L/d]	448,342.74
Metered Pop	2,830.00
GWI %	81.00%
GWI [L/s]= % x Min DWF	1.45
GWI [L/d]	125,308.70
Per Capita flow [L/c/d]	114.15
GWI [L/ha/s]	0.024334432

59.6 ha

Put 80% GWI where Min Flow is lower
Adjust so that Flow-GWI is not super low in the table
Resuling GWI / (other set of data's minimum) = GWI% of that other set of data

Choose: e.g. right if you start adjusting at right side
right / left 81.01%
left/right 79.99%

arva

Axford(2) - Weekend Data (2017)

Change Data Source of Pivot table

Row Labels	Average of Flow
12 AM	3.66
1 AM	2.60
2 AM	2.06
3 AM	1.84
4 AM	1.81
5 AM	2.18
6 AM	3.70
7 AM	6.56
8 AM	9.72
9 AM	10.06
10 AM	10.08
11 AM	9.25
12 PM	8.30
1 PM	8.30
2 PM	7.00
3 PM	7.28
4 PM	7.36
5 PM	7.83
6 PM	9.39
7 PM	9.19
8 PM	8.11
9 PM	6.28
10 PM	5.03
11 PM	4.04
Average	**6.32**

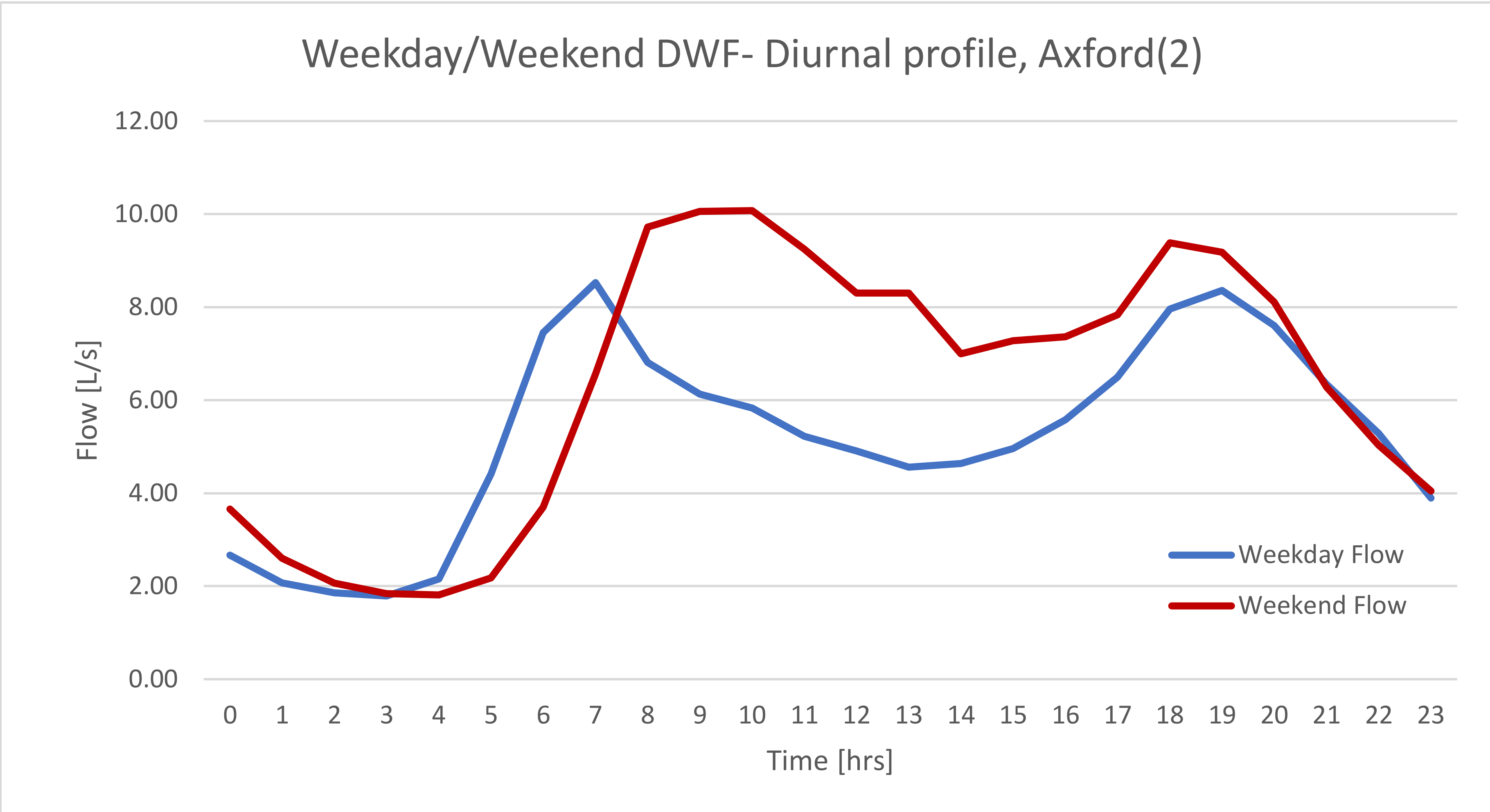
Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	3.66	1.45	2.21	45%	45%	0.59
1	2.60	1.45	1.15	24%	24%	0.31
2	2.06	1.45	0.61	13%	13%	0.16
3	1.84	1.45	0.39	8%	8%	0.10
4	1.81	1.45	0.36	7%	7%	0.10
5	2.18	1.45	0.73	15%	15%	0.20
6	3.70	1.45	2.25	46%	46%	0.60
7	6.56	1.45	5.11	105%	105%	1.37
8	9.72	1.45	8.27	170%	170%	2.21
9	10.06	1.45	8.61	177%	177%	2.30
10	10.08	1.45	8.63	177%	177%	2.31
11	9.25	1.45	7.80	160%	160%	2.09
12	8.30	1.45	6.85	141%	141%	1.83
13	8.30	1.45	6.85	141%	141%	1.83
14	7.00	1.45	5.55	114%	114%	1.48
15	7.28	1.45	5.83	120%	120%	1.56
16	7.36	1.45	5.91	121%	121%	1.58
17	7.83	1.45	6.38	131%	131%	1.71
18	9.39	1.45	7.94	163%	163%	2.12
19	9.19	1.45	7.74	159%	159%	2.07
20	8.11	1.45	6.66	137%	137%	1.78
21	6.28	1.45	4.83	99%	99%	1.29
22	5.03	1.45	3.58	74%	74%	0.96
23	4.04	1.45	2.59	53%	53%	0.69
Adjust factor for weekday PCD use in model:						1.30
Average	**6.32**		**4.87**	**1.00**	**1.00**	**1.30**
Min	**1.81**					
Max	**10.08**					

Weekend	
Min.	1.81
Average	6.32
Daily average flow [L/d]	545,952.66
Subtract trade flows [L/d] set t	0.00
Residential flow [L/d]	545,952.66
Metered Pop	**2,830.00**
GWI %	**80.00%**
GWI [L/s]= % x Min DWF	1.45
GWI [L/d]	125324.59
Per Capita flow [L/c/d]	148.63
GWI [L/ha/s]	0.02

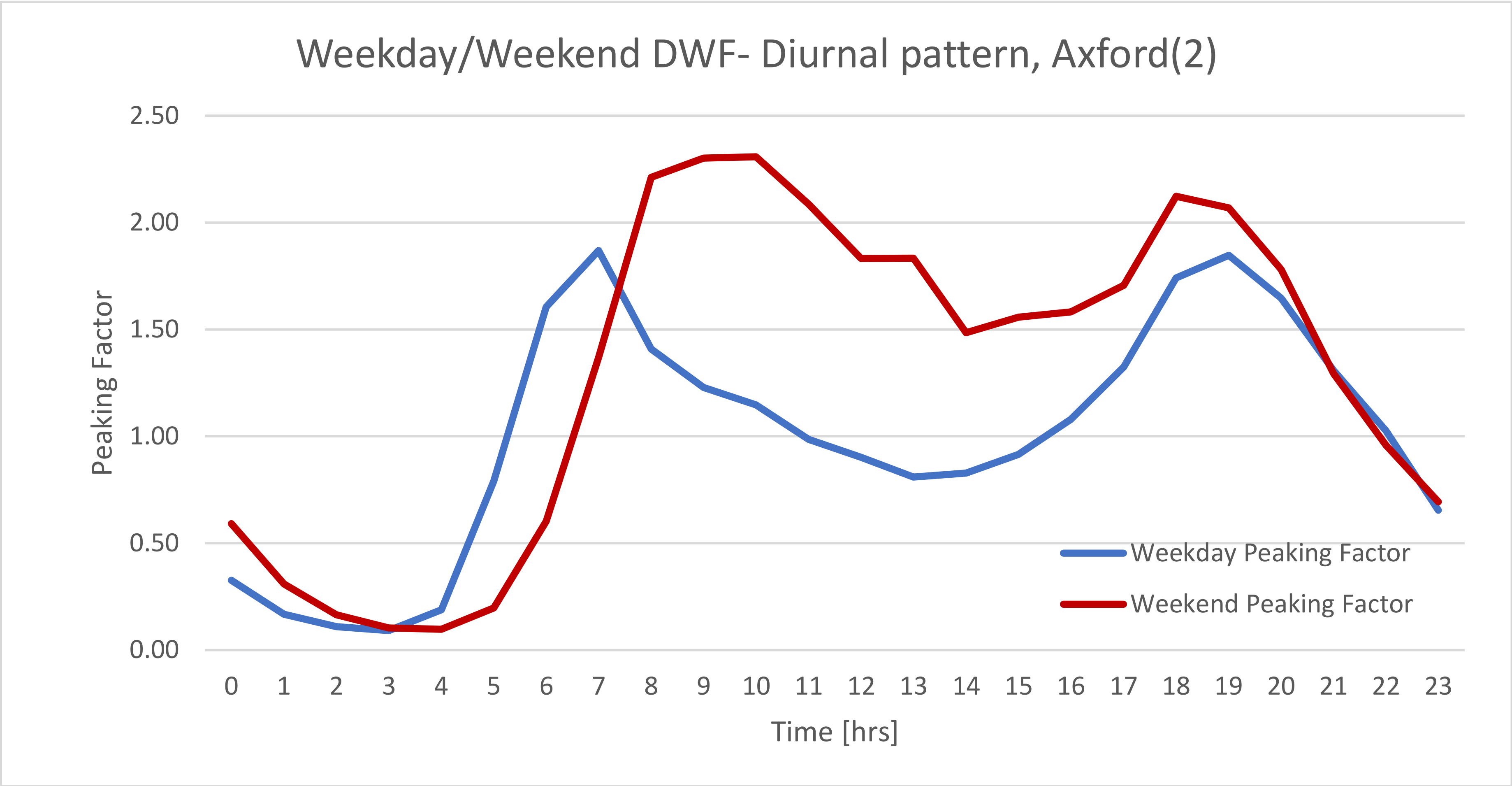
59.6 ha

arva

Axford(2) - Diurnal Profile (2017)



Axford(2) - Diurnal Pattern (2017)



Chestnut - Weekday Data (2017)

Change Data Source of Pivot table

Trade Flow (l/s): 9.12

Row Labels	Average of Flow (l/s)
12 AM	42.00
1 AM	37.33
2 AM	36.76
3 AM	35.60
4 AM	34.03
5 AM	39.33
6 AM	50.19
7 AM	70.61
8 AM	75.47
9 AM	72.88
10 AM	70.82
11 AM	72.02
12 PM	64.98
1 PM	64.22
2 PM	65.78
3 PM	62.90
4 PM	63.90
5 PM	67.06
6 PM	74.08
7 PM	70.21
8 PM	72.92
9 PM	63.96
10 PM	60.15
11 PM	50.41
Average	**59.07**

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	42.00	26.44	15.56	0	0	15.56	55%	0.55
1	37.33	26.44	10.89	0	0	10.89	38%	0.38
2	36.76	26.44	10.32	0	0	10.32	36%	0.36
3	35.60	26.44	9.16	0	0	9.16	32%	0.32
4	34.03	26.44	7.59	0	0	7.59	27%	0.27
5	39.33	26.44	12.89	0	0	12.89	45%	0.45
6	50.19	26.44	23.75	0	0	23.75	83%	0.83
7	70.61	26.44	44.17	1	9.12	35.05	123%	1.23
8	75.47	26.44	49.04	1	9.12	39.92	140%	1.40
9	72.88	26.44	46.45	1	9.12	37.33	131%	1.31
10	70.82	26.44	44.38	1	9.12	35.26	124%	1.24
11	72.02	26.44	45.58	1	9.12	36.46	128%	1.28
12	64.98	26.44	38.54	1	9.12	29.42	103%	1.03
13	64.22	26.44	37.78	1	9.12	28.66	101%	1.01
14	65.78	26.44	39.35	1	9.12	30.23	106%	1.06
15	62.90	26.44	36.46	1	9.12	27.34	96%	0.96
16	63.90	26.44	37.46	1	9.12	28.34	100%	1.00
17	67.06	26.44	40.63	1	9.12	31.51	111%	1.11
18	74.08	26.44	47.64	0	0	47.64	167%	1.67
19	70.21	26.44	43.77	0	0	43.77	154%	1.54
20	72.92	26.44	46.48	0	0	46.48	163%	1.63
21	63.96	26.44	37.52	0	0	37.52	132%	1.32
22	60.15	26.44	33.71	0	0	33.71	119%	1.19
23	50.41	26.44	23.97	0	0	23.97	84%	0.84
Average	59.07				4.18	28.45	1.00	1.00
Min	34.03							
Max.	75.47							

Weekday

Min. total flow	34.03	
Average total flow	59.07	
Daily average flow [L/d]	5,103,478.01	
Subtract trade flows [L/d]	361,152.00	
Residential flow [L/d]	4,742,326.01	
Metered Pop	13,625.00	
GWI %	77.70%	
GWI [L/s]= % x Min DWF	26.44	
GWI [L/d]	2,284,356.01	
Per Capita flow [L/c/d]	180.40	
GWI [L/ha/s]	0.048691171	543.03 ha

Put 80% GWI where Min Flow is lower	Choose: e.g. right if you start adjusting at right side	
Adjust so that Flow-GWI is not super low in the table	right / left	77.68%
Resuling GWI / (other set of data's minimum) = GWI% of that other set of data	left/right	85.02%

Chestnut - Weekend Data (2017)

Change Data Source of Pivot table

Row Labels	Average of Flow
12 AM	45.22
1 AM	37.82
2 AM	34.26
3 AM	31.28
4 AM	31.10
5 AM	34.07
6 AM	35.22
7 AM	47.42
8 AM	63.60
9 AM	76.75
10 AM	80.02
11 AM	75.98
12 PM	79.29
1 PM	70.61
2 PM	66.89
3 PM	66.98
4 PM	64.50
5 PM	63.52
6 PM	69.00
7 PM	70.39
8 PM	72.01
9 PM	58.71
10 PM	58.17
11 PM	49.27
Average	**57.59**

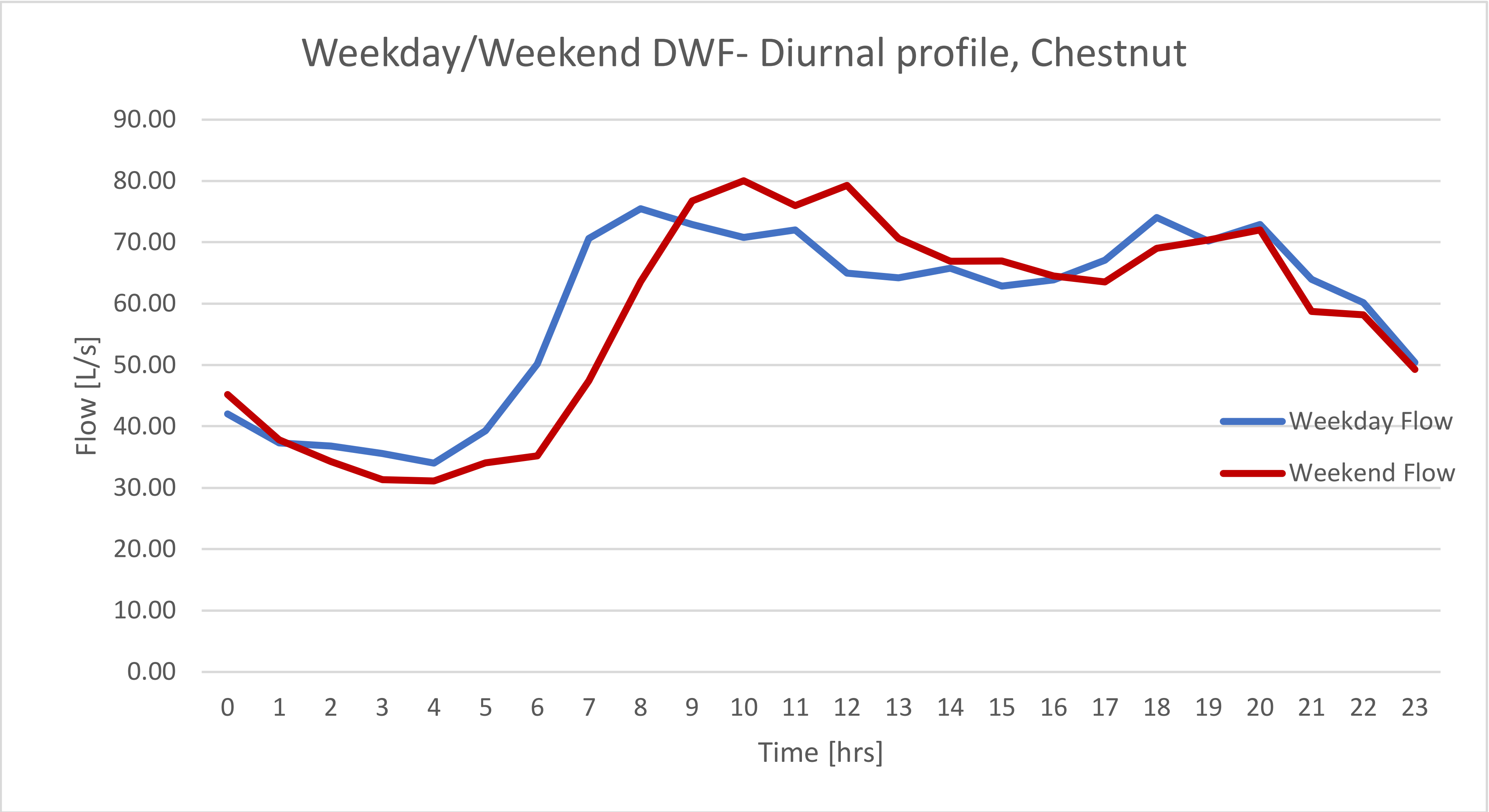
Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	45.22	26.43	18.78	60%	60%	0.66
1	37.82	26.43	11.38	37%	37%	0.40
2	34.26	26.43	7.83	25%	25%	0.28
3	31.28	26.43	4.85	16%	16%	0.17
4	31.10	26.43	4.66	15%	15%	0.16
5	34.07	26.43	7.63	25%	25%	0.27
6	35.22	26.43	8.78	28%	28%	0.31
7	47.42	26.43	20.98	67%	67%	0.74
8	63.60	26.43	37.16	119%	119%	1.31
9	76.75	26.43	50.31	162%	162%	1.77
10	80.02	26.43	53.59	172%	172%	1.88
11	75.98	26.43	49.54	159%	159%	1.74
12	79.29	26.43	52.86	170%	170%	1.86
13	70.61	26.43	44.18	142%	142%	1.55
14	66.89	26.43	40.45	130%	130%	1.42
15	66.98	26.43	40.55	130%	130%	1.43
16	64.50	26.43	38.07	122%	122%	1.34
17	63.52	26.43	37.09	119%	119%	1.30
18	69.00	26.43	42.57	137%	137%	1.50
19	70.39	26.43	43.96	141%	141%	1.55
20	72.01	26.43	45.58	146%	146%	1.60
21	58.71	26.43	32.28	104%	104%	1.13
22	58.17	26.43	31.74	102%	102%	1.12
23	49.27	26.43	22.84	73%	73%	0.80
Adjust factor for weekday PCD use in model:						1.10
Average	**57.59**		**31.15**	**1.00**	**1.00**	**1.10**
Min	**31.10**					
Max	**80.02**					

Weekend

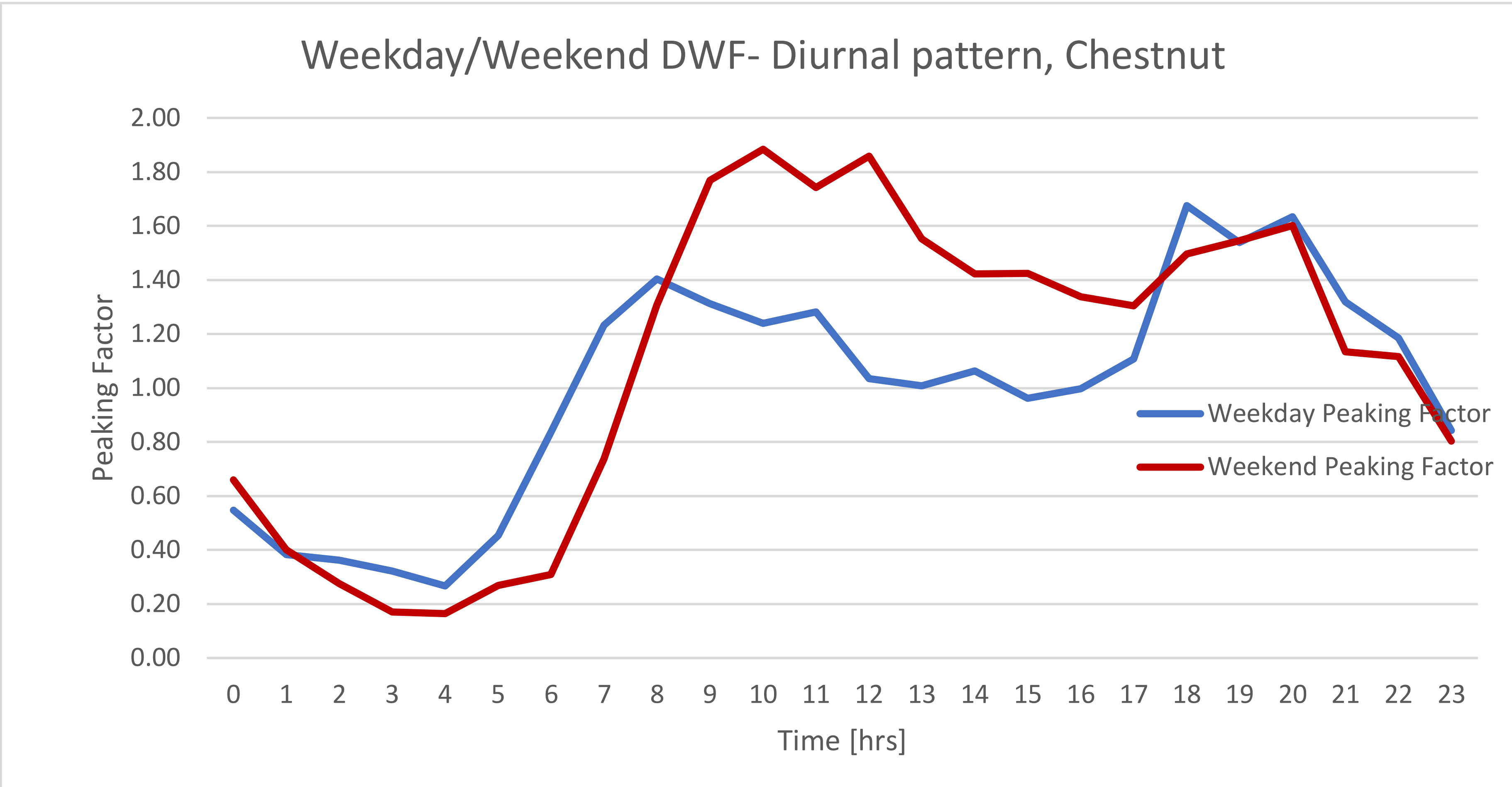
Min.	**31.10**	
Average	**57.59**	
Daily average flow [L/d]	**4,975,449.88**	
Subtract trade flows [L/d] set t	**0.00**	
Residential flow [L/d]	**4,975,449.88**	
Metered Pop	**13,625.00**	
GWI %	**85.00%**	
GWI [L/s]= % x Min DWF	**26.43**	
GWI [L/d]	**2283879.44**	
Per Capita flow [L/c/d]	197.55	
GWI [L/ha/s]	**0.05**	543 ha

arva

Chestnut - Diurnal Profile (2017)



Chestnut - Diurnal Pattern (2017)



Oakmont - Weekday Data (2017)

Change Data Source of Pivot table

Row Labels	Average of Flow (l/s)
12 AM	0.64
1 AM	0.57
2 AM	0.31
3 AM	0.45
4 AM	0.80
5 AM	0.87
6 AM	1.69
7 AM	1.27
8 AM	0.97
9 AM	0.80
10 AM	0.83
11 AM	0.77
12 PM	0.96
1 PM	1.11
2 PM	1.09
3 PM	1.06
4 PM	1.08
5 PM	1.07
6 PM	1.21
7 PM	1.24
8 PM	1.44
9 PM	1.20
10 PM	1.17
11 PM	0.72
Average	**0.97**

Trade Flow (l/s): 0

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	0.64	0.21	0.42	0	0	0.42	56%	0.56
1	0.57	0.21	0.36	0	0	0.36	47%	0.47
2	0.31	0.21	0.09	0	0	0.09	12%	0.12
3	0.45	0.21	0.24	0	0	0.24	31%	0.31
4	0.80	0.21	0.59	0	0	0.59	78%	0.78
5	0.87	0.21	0.66	0	0	0.66	87%	0.87
6	1.69	0.21	1.47	0	0	1.47	194%	1.94
7	1.27	0.21	1.06	1	0	1.06	139%	1.39
8	0.97	0.21	0.76	1	0	0.76	100%	1.00
9	0.80	0.21	0.59	1	0	0.59	77%	0.77
10	0.83	0.21	0.62	1	0	0.62	82%	0.82
11	0.77	0.21	0.56	1	0	0.56	74%	0.74
12	0.96	0.21	0.74	1	0	0.74	98%	0.98
13	1.11	0.21	0.89	1	0	0.89	118%	1.18
14	1.09	0.21	0.88	1	0	0.88	116%	1.16
15	1.06	0.21	0.85	1	0	0.85	112%	1.12
16	1.08	0.21	0.86	1	0	0.86	114%	1.14
17	1.07	0.21	0.86	1	0	0.86	114%	1.14
18	1.21	0.21	1.00	0	0	1.00	131%	1.31
19	1.24	0.21	1.02	0	0	1.02	135%	1.35
20	1.44	0.21	1.22	0	0	1.22	161%	1.61
21	1.20	0.21	0.99	0	0	0.99	130%	1.30
22	1.17	0.21	0.96	0	0	0.96	126%	1.26
23	0.72	0.21	0.51	0	0	0.51	67%	0.67
Average	0.97				0	0.76	1.00	1.00
Min	0.31							
Max.	1.69							

Weekday

Min. total flow	0.31	
Average total flow	0.97	
Daily average flow [L/d]	84,000.09	
Subtract trade flows [L/d]	0.00	
Residential flow [L/d]	84,000.09	
Metered Pop	338.00	
GWI %	70.00%	
GWI [L/s]= % x Min DWF	0.21	
GWI [L/d]	18,534.78	
Per Capita flow [L/c/d]	193.68	
GWI [L/ha/s]	0.021093695	10.17 ha

Put 80% GWI where Min Flow is lower	Choose: e.g. right if you start adjusting at right side	
Adjust so that Flow-GWI is not super low in the table	right / left	69.27%
Resuling GWI / (other set of data's minimum) = GWI% of that other set of data	left/right	80.84%

Oakmont - Weekend Data (2017)

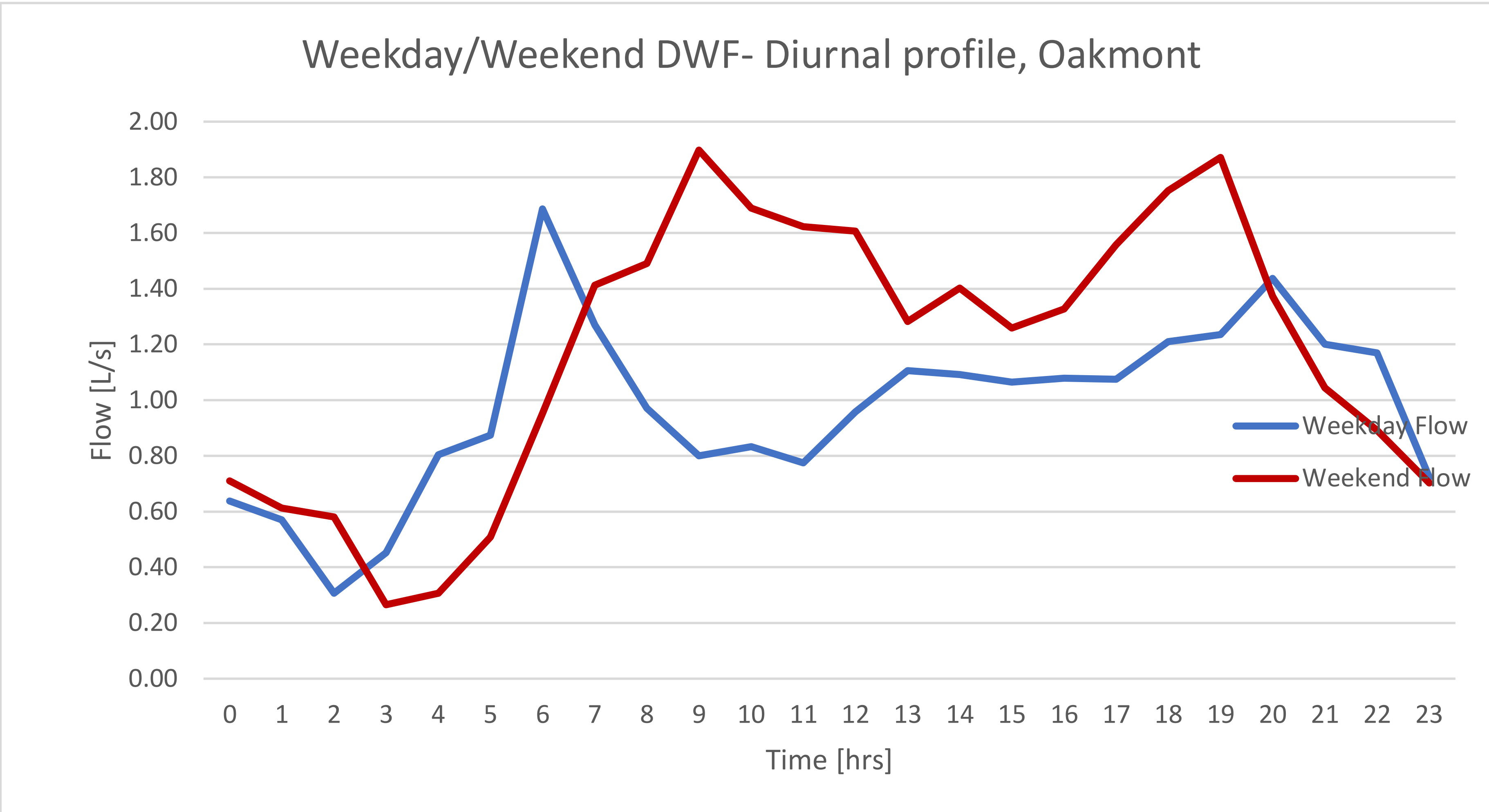
Change Data Source of Pivot table

Row Labels	Average of Flow
12 AM	0.71
1 AM	0.61
2 AM	0.58
3 AM	0.27
4 AM	0.31
5 AM	0.51
6 AM	0.95
7 AM	1.41
8 AM	1.49
9 AM	1.90
10 AM	1.69
11 AM	1.62
12 PM	1.61
1 PM	1.28
2 PM	1.40
3 PM	1.26
4 PM	1.33
5 PM	1.56
6 PM	1.75
7 PM	1.87
8 PM	1.37
9 PM	1.04
10 PM	0.89
11 PM	0.70
Average	**1.17**

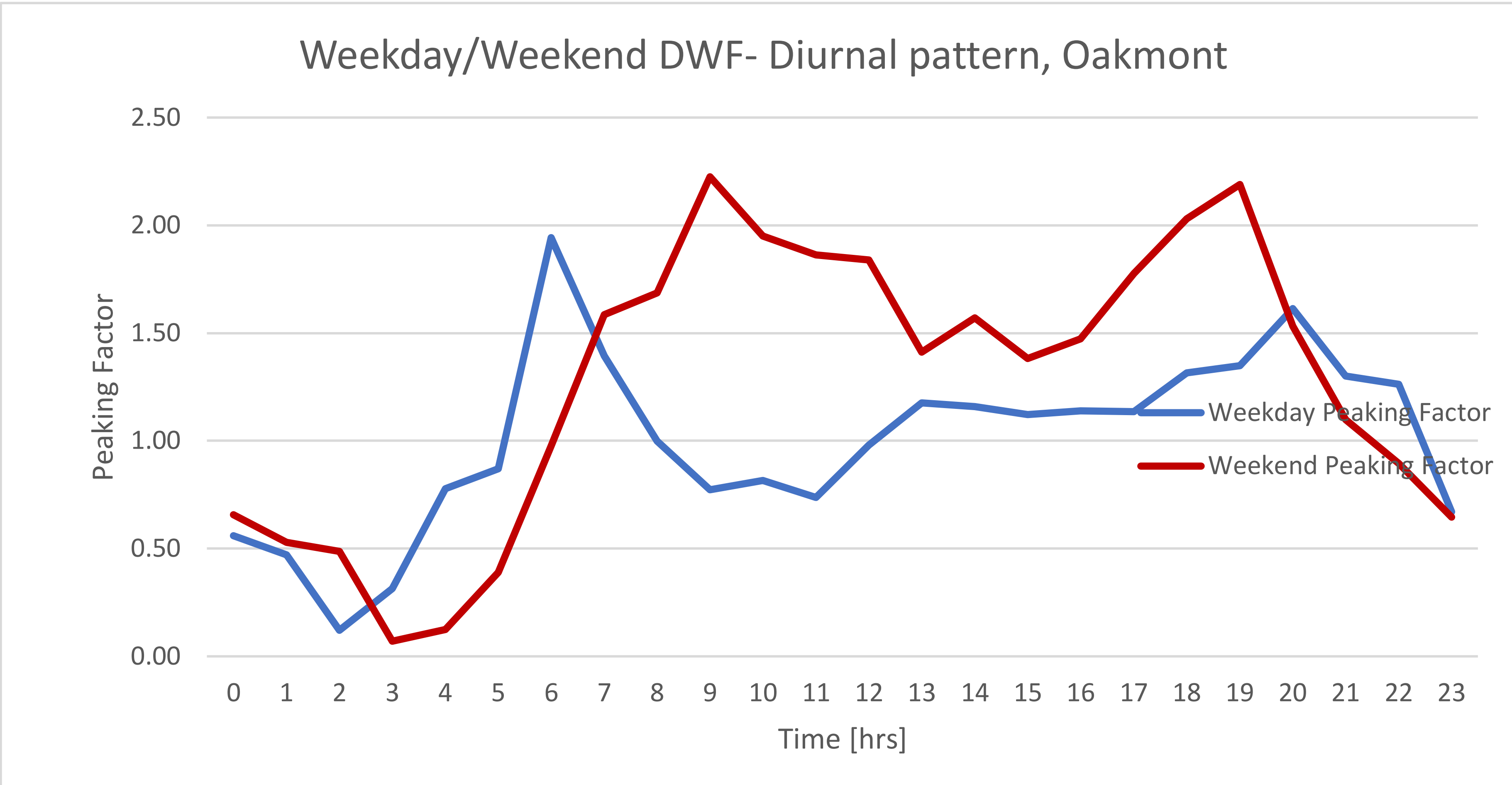
Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	0.71	0.21	0.50	52%	52%	0.66
1	0.61	0.21	0.40	42%	42%	0.53
2	0.58	0.21	0.37	38%	38%	0.49
3	0.27	0.21	0.05	6%	6%	0.07
4	0.31	0.21	0.09	10%	10%	0.13
5	0.51	0.21	0.30	31%	31%	0.39
6	0.95	0.21	0.74	77%	77%	0.98
7	1.41	0.21	1.20	125%	125%	1.58
8	1.49	0.21	1.28	133%	133%	1.69
9	1.90	0.21	1.69	176%	176%	2.23
10	1.69	0.21	1.48	154%	154%	1.95
11	1.62	0.21	1.41	147%	147%	1.86
12	1.61	0.21	1.39	145%	145%	1.84
13	1.28	0.21	1.07	111%	111%	1.41
14	1.40	0.21	1.19	124%	124%	1.57
15	1.26	0.21	1.05	109%	109%	1.38
16	1.33	0.21	1.12	116%	116%	1.47
17	1.56	0.21	1.35	140%	140%	1.78
18	1.75	0.21	1.54	160%	160%	2.03
19	1.87	0.21	1.66	173%	173%	2.19
20	1.37	0.21	1.16	121%	121%	1.53
21	1.04	0.21	0.83	87%	87%	1.10
22	0.89	0.21	0.68	71%	71%	0.90
23	0.70	0.21	0.49	51%	51%	0.65
Adjust factor for weekday PCD use in model:						1.27
Average	**1.17**		**0.96**	**1.00**	**1.00**	**1.27**
Min	**0.27**					
Max	**1.90**					

Weekend		
Min.	**0.27**	
Average	**1.17**	
Daily average flow [L/d]	**101,238.22**	
Subtract trade flows [L/d] set t	**0.00**	
Residential flow [L/d]	**101,238.22**	
Metered Pop	**338.00**	
GWI %	**80.00%**	
GWI [L/s]= % x Min DWF	**0.21**	
GWI [L/d]	**18341.25**	
Per Capita flow [L/c/d]	245.26	
GWI [L/ha/s]	**0.02**	10.17 ha

Oakmont - Diurnal Profile (2017)



Oakmont - Diurnal Pattern (2017)



Thomas 1 - Weekday Data (2020)

Change Data Source of Pivot table

Trade Flow (l/s): 0

Row Labels	Average of Flow (l/s)
12 AM	7.58
1 AM	6.40
2 AM	5.07
3 AM	4.16
4 AM	5.09
5 AM	4.53
6 AM	7.21
7 AM	11.30
8 AM	13.00
9 AM	11.67
10 AM	11.07
11 AM	10.13
12 PM	10.19
1 PM	9.98
2 PM	9.71
3 PM	9.23
4 PM	9.42
5 PM	10.21
6 PM	11.21
7 PM	12.65
8 PM	12.77
9 PM	12.08
10 PM	11.31
11 PM	9.55
Average	**9.40**

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	7.58	2.01	5.57	0	0	5.57	75%	0.75
1	6.40	2.01	4.39	0	0	4.39	59%	0.59
2	5.07	2.01	3.07	0	0	3.07	41%	0.41
3	4.16	2.01	2.15	0	0	2.15	29%	0.29
4	5.09	2.01	3.08	0	0	3.08	42%	0.42
5	4.53	2.01	2.52	0	0	2.52	34%	0.34
6	7.21	2.01	5.20	0	0	5.20	70%	0.70
7	11.30	2.01	9.29	0	0	9.29	126%	1.26
8	13.00	2.01	10.99	0	0	10.99	149%	1.49
9	11.67	2.01	9.66	0	0	9.66	131%	1.31
10	11.07	2.01	9.06	0	0	9.06	123%	1.23
11	10.13	2.01	8.12	0	0	8.12	110%	1.10
12	10.19	2.01	8.19	0	0	8.19	111%	1.11
13	9.98	2.01	7.97	0	0	7.97	108%	1.08
14	9.71	2.01	7.70	0	0	7.70	104%	1.04
15	9.23	2.01	7.22	0	0	7.22	98%	0.98
16	9.42	2.01	7.41	0	0	7.41	100%	1.00
17	10.21	2.01	8.21	0	0	8.21	111%	1.11
18	11.21	2.01	9.20	0	0	9.20	125%	1.25
19	12.65	2.01	10.64	0	0	10.64	144%	1.44
20	12.77	2.01	10.76	0	0	10.76	146%	1.46
21	12.08	2.01	10.07	0	0	10.07	136%	1.36
22	11.31	2.01	9.30	0	0	9.30	126%	1.26
23	9.55	2.01	7.54	0	0	7.54	102%	1.02
Average	9.40				0	7.39	1.00	1.00
Min	4.16							
Max.	13.00							

Weekday

Min. total flow	4.16	
Average total flow	9.40	
Daily average flow [L/d]	811,874.96	
Subtract trade flows [L/d]	0.00	0 m3/s
Residential flow [L/d]	811,874.96	
Metered Pop	2,388.00	
GWI %	48.30%	
GWI [L/s]= % x Min DWF	2.01	
GWI [L/d]	173,560.41	
Per Capita flow [L/c/d]	267.30	
GWI [L/ha/s]	0.019067879	105.35

Thomas 1 - Weekend Data (2020)

Change Data Source of Pivot table

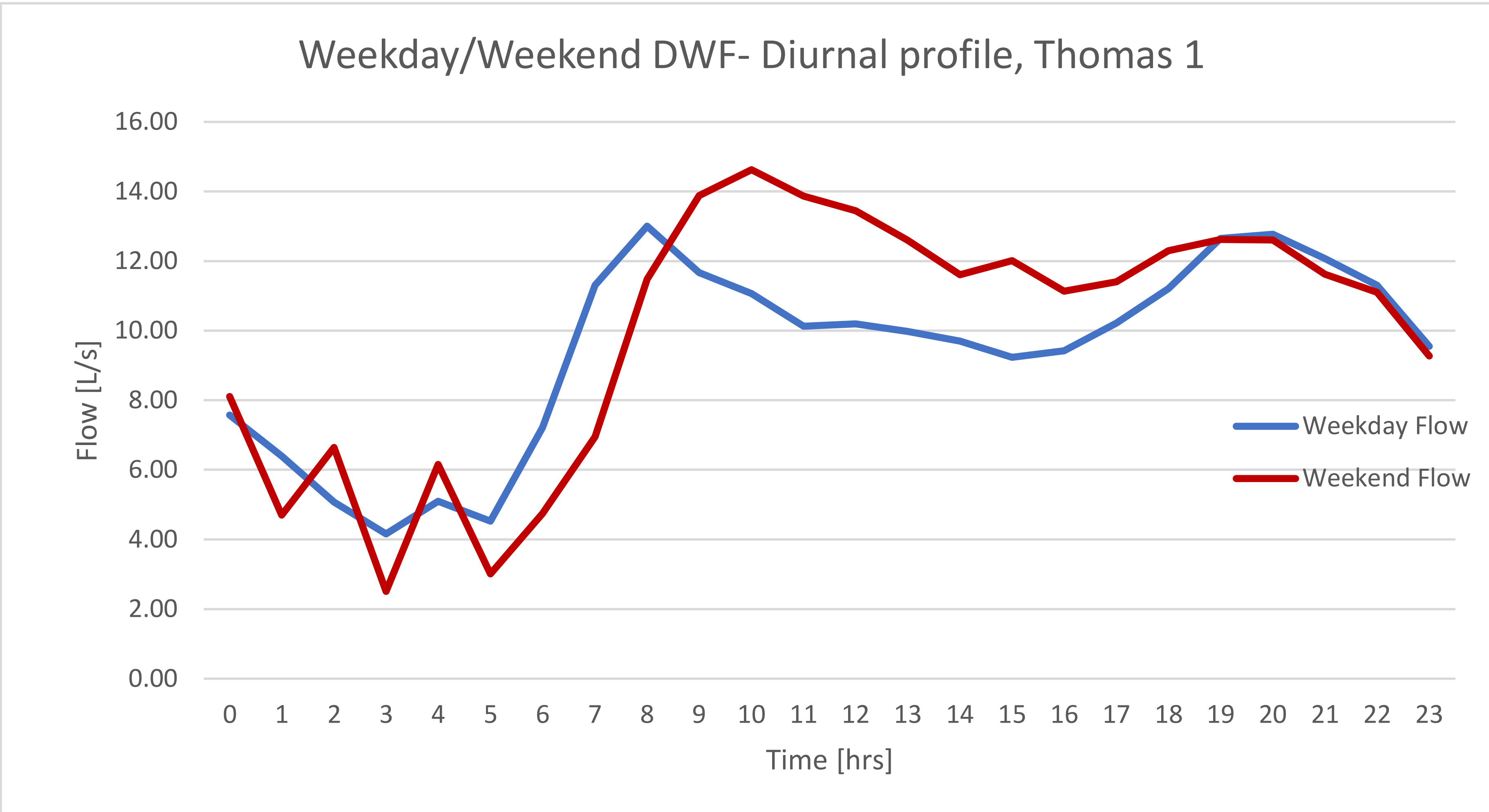
Row Labels	Average of Flow
12 AM	8.11
1 AM	4.70
2 AM	6.64
3 AM	2.51
4 AM	6.16
5 AM	3.02
6 AM	4.74
7 AM	6.95
8 AM	11.47
9 AM	13.88
10 AM	14.62
11 AM	13.87
12 PM	13.44
1 PM	12.59
2 PM	11.61
3 PM	12.01
4 PM	11.14
5 PM	11.41
6 PM	12.30
7 PM	12.61
8 PM	12.60
9 PM	11.63
10 PM	11.10
11 PM	9.27
Average	**9.93**

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	8.11	2.01	6.11	77%	77%	0.83
1	4.70	2.01	2.69	34%	34%	0.36
2	6.64	2.01	4.63	58%	58%	0.63
3	2.51	2.01	0.50	6%	6%	0.07
4	6.16	2.01	4.15	52%	52%	0.56
5	3.02	2.01	1.01	13%	13%	0.14
6	4.74	2.01	2.73	34%	34%	0.37
7	6.95	2.01	4.94	62%	62%	0.67
8	11.47	2.01	9.46	119%	119%	1.28
9	13.88	2.01	11.87	150%	150%	1.61
10	14.62	2.01	12.62	159%	159%	1.71
11	13.87	2.01	11.87	150%	150%	1.61
12	13.44	2.01	11.43	144%	144%	1.55
13	12.59	2.01	10.59	134%	134%	1.43
14	11.61	2.01	9.60	121%	121%	1.30
15	12.01	2.01	10.00	126%	126%	1.35
16	11.14	2.01	9.13	115%	115%	1.24
17	11.41	2.01	9.40	119%	119%	1.27
18	12.30	2.01	10.30	130%	130%	1.39
19	12.61	2.01	10.61	134%	134%	1.44
20	12.60	2.01	10.60	134%	134%	1.43
21	11.63	2.01	9.62	121%	121%	1.30
22	11.10	2.01	9.09	115%	115%	1.23
23	9.27	2.01	7.26	92%	92%	0.98
Adjust factor for weekday PCD use in model:						1.07
Average	**9.93**		**7.93**	**1.00**	**1.00**	**1.07**
Min	**2.51**					
Max	**14.62**					

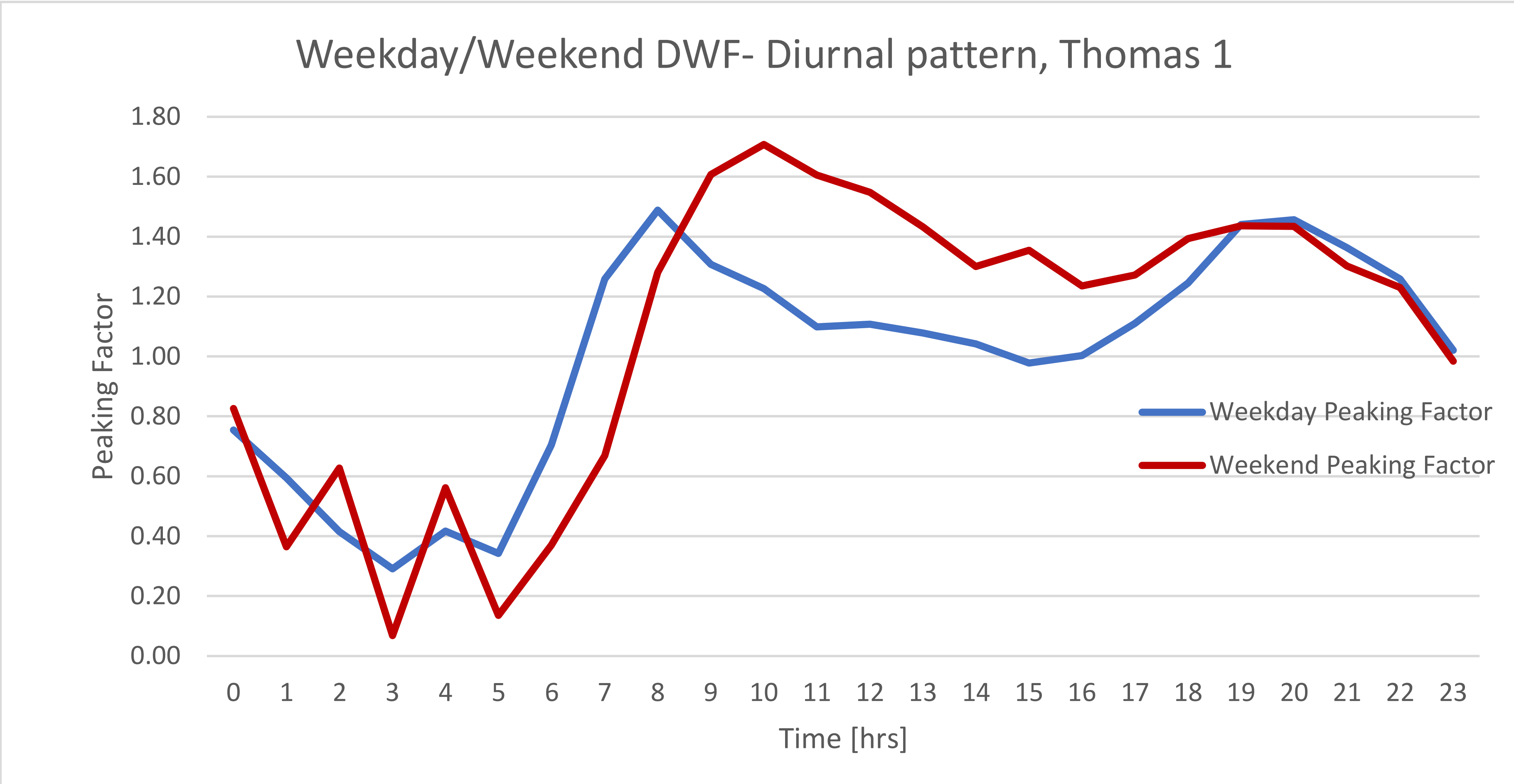
Weekend	
Min.	**2.51**
Average	**9.93**
Daily average flow [L/d]	**858,210.59**
Subtract trade flows [L/d] set t	**0.00**
Residential flow [L/d]	**858,210.59**
Metered Pop	**2,388.00**
GWI %	**80.00%**
GWI [L/s]= % x Min DWF	**2.01**
GWI [L/d]	**173451.02**
Per Capita flow [L/c/d]	286.75
GWI [L/ha/s]	**0.02**

105.35 ha

Thomas 1 - Diurnal Profile (2020)



Thomas 1 - Diurnal Pattern (2020)



Thomas 2 - Weekday Data (2020)

Change Data Source of Pivot table

Row Labels	Average of Flow (l/s)
12 AM	15.28
1 AM	13.33
2 AM	13.12
3 AM	11.88
4 AM	11.69
5 AM	12.46
6 AM	16.85
7 AM	22.38
8 AM	23.39
9 AM	22.61
10 AM	22.00
11 AM	20.63
12 PM	20.30
1 PM	20.09
2 PM	18.98
3 PM	18.46
4 PM	17.52
5 PM	19.00
6 PM	21.77
7 PM	24.51
8 PM	26.50
9 PM	23.47
10 PM	20.09
11 PM	18.79
Average	**18.96**

Trade Flow (l/s): 0.72427

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	15.28	9.38	5.90	0	0	5.90	64%	0.64
1	13.33	9.38	3.95	0	0	3.95	43%	0.43
2	13.12	9.38	3.74	0	0	3.74	40%	0.40
3	11.88	9.38	2.50	0	0	2.50	27%	0.27
4	11.69	9.38	2.31	0	0	2.31	25%	0.25
5	12.46	9.38	3.08	0	0	3.08	33%	0.33
6	16.85	9.38	7.47	0	0	7.47	81%	0.81
7	22.38	9.38	13.00	1	0.72427	12.27	133%	1.33
8	23.39	9.38	14.01	1	0.72427	13.28	144%	1.44
9	22.61	9.38	13.23	1	0.72427	12.50	135%	1.35
10	22.00	9.38	12.62	1	0.72427	11.90	129%	1.29
11	20.63	9.38	11.25	1	0.72427	10.53	114%	1.14
12	20.30	9.38	10.92	1	0.72427	10.19	110%	1.10
13	20.09	9.38	10.71	1	0.72427	9.99	108%	1.08
14	18.98	9.38	9.60	1	0.72427	8.88	96%	0.96
15	18.46	9.38	9.08	1	0.72427	8.36	90%	0.90
16	17.52	9.38	8.14	1	0.72427	7.41	80%	0.80
17	19.00	9.38	9.62	1	0.72427	8.89	96%	0.96
18	21.77	9.38	12.39	0	0	12.39	134%	1.34
19	24.51	9.38	15.13	0	0	15.13	164%	1.64
20	26.50	9.38	17.12	0	0	17.12	185%	1.85
21	23.47	9.38	14.09	0	0	14.09	152%	1.52
22	20.09	9.38	10.71	0	0	10.71	116%	1.16
23	18.79	9.38	9.41	0	0	9.41	102%	1.02
Average	**18.96**				**0.331957083**	**9.25**	**1.00**	**1.00**
Min	**11.69**							
Max.	**26.50**							

Weekday

Min. total flow	11.69	
Average total flow	18.96	
Daily average flow [L/d]	1,638,246.92	
Subtract trade flows [L/d]	28,681.09	
Residential flow [L/d]	1,609,565.83	
Metered Pop	3,153.00	
GWI %	80.22%	
GWI [L/s]= % x Min DWF	9.38	
GWI [L/d]	810,432.00	
Per Capita flow [L/c/d]	253.45	
GWI [L/ha/s]	0.092596249	101.3 ha

Thomas 2 - Weekend Data (2020)

Change Data Source of Pivot table

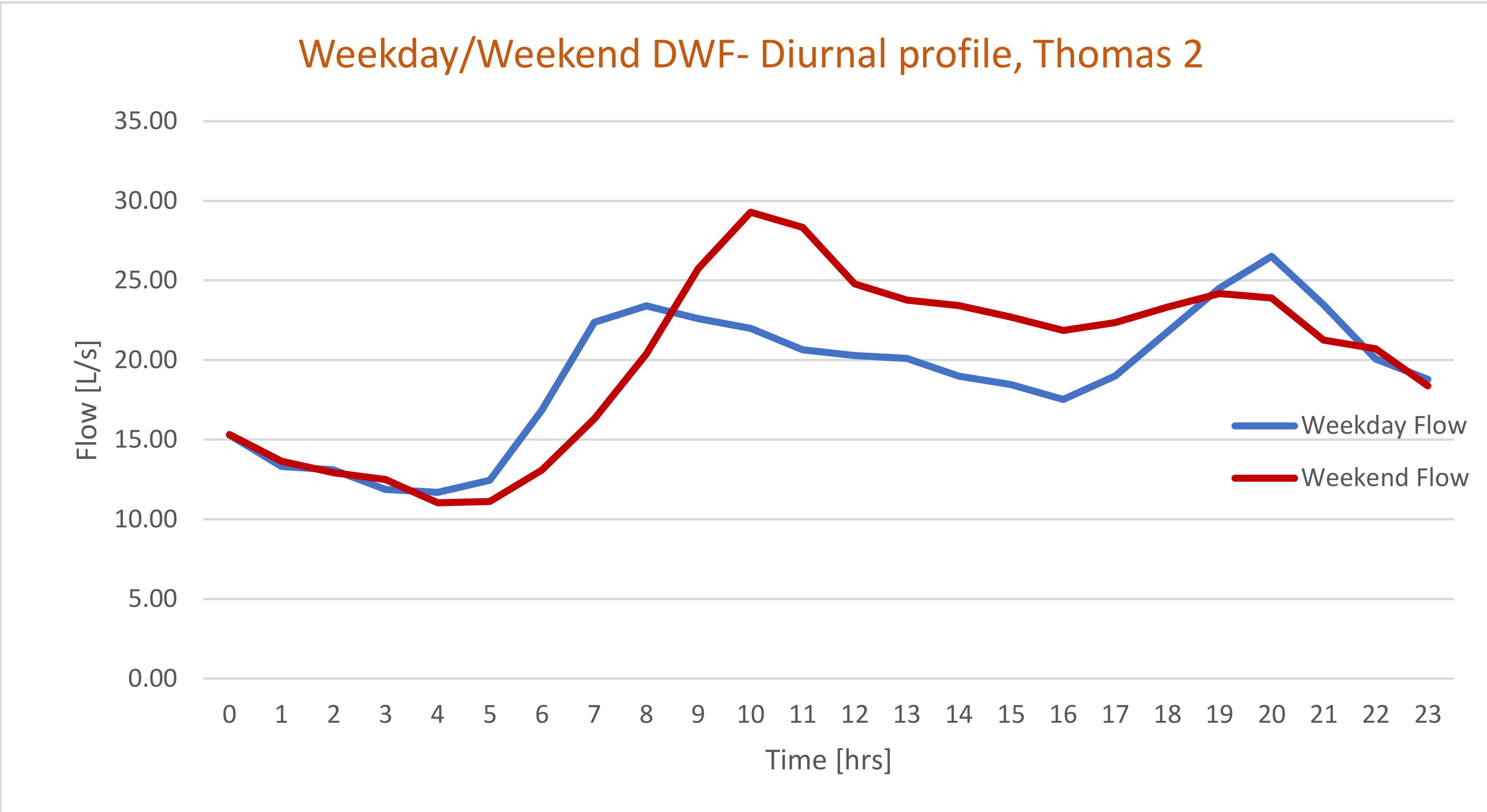
Row Labels	Average of Flow (l/s)
12 AM	15.32
1 AM	13.65
2 AM	12.93
3 AM	12.50
4 AM	11.03
5 AM	11.13
6 AM	13.08
7 AM	16.32
8 AM	20.37
9 AM	25.75
10 AM	29.28
11 AM	28.32
12 PM	24.78
1 PM	23.78
2 PM	23.43
3 PM	22.70
4 PM	21.85
5 PM	22.34
6 PM	23.33
7 PM	24.16
8 PM	23.88
9 PM	21.25
10 PM	20.71
11 PM	18.37
Average	**20.00**

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	15.32	9.38	5.95	56%	56%	0.64
1	13.65	9.38	4.27	40%	40%	0.46
2	12.93	9.38	3.56	33%	33%	0.38
3	12.50	9.38	3.12	29%	29%	0.34
4	11.03	9.38	1.65	16%	16%	0.18
5	11.13	9.38	1.75	16%	16%	0.19
6	13.08	9.38	3.71	35%	35%	0.40
7	16.32	9.38	6.95	65%	65%	0.75
8	20.37	9.38	10.99	103%	103%	1.19
9	25.75	9.38	16.37	154%	154%	1.77
10	29.28	9.38	19.90	187%	187%	2.15
11	28.32	9.38	18.94	178%	178%	2.05
12	24.78	9.38	15.41	145%	145%	1.67
13	23.78	9.38	14.40	135%	135%	1.56
14	23.43	9.38	14.05	132%	132%	1.52
15	22.70	9.38	13.33	125%	125%	1.44
16	21.85	9.38	12.47	117%	117%	1.35
17	22.34	9.38	12.96	122%	122%	1.40
18	23.33	9.38	13.95	131%	131%	1.51
19	24.16	9.38	14.78	139%	139%	1.60
20	23.88	9.38	14.50	136%	136%	1.57
21	21.25	9.38	11.88	112%	112%	1.28
22	20.71	9.38	11.33	107%	107%	1.22
23	18.37	9.38	8.99	85%	85%	0.97
Adjust factor for weekday PCD use in model:						1.15
Average	**20.01**		**10.63**	**1.00**	**1.00**	**1.15**
Min	**11.03**					
Max	**29.28**					

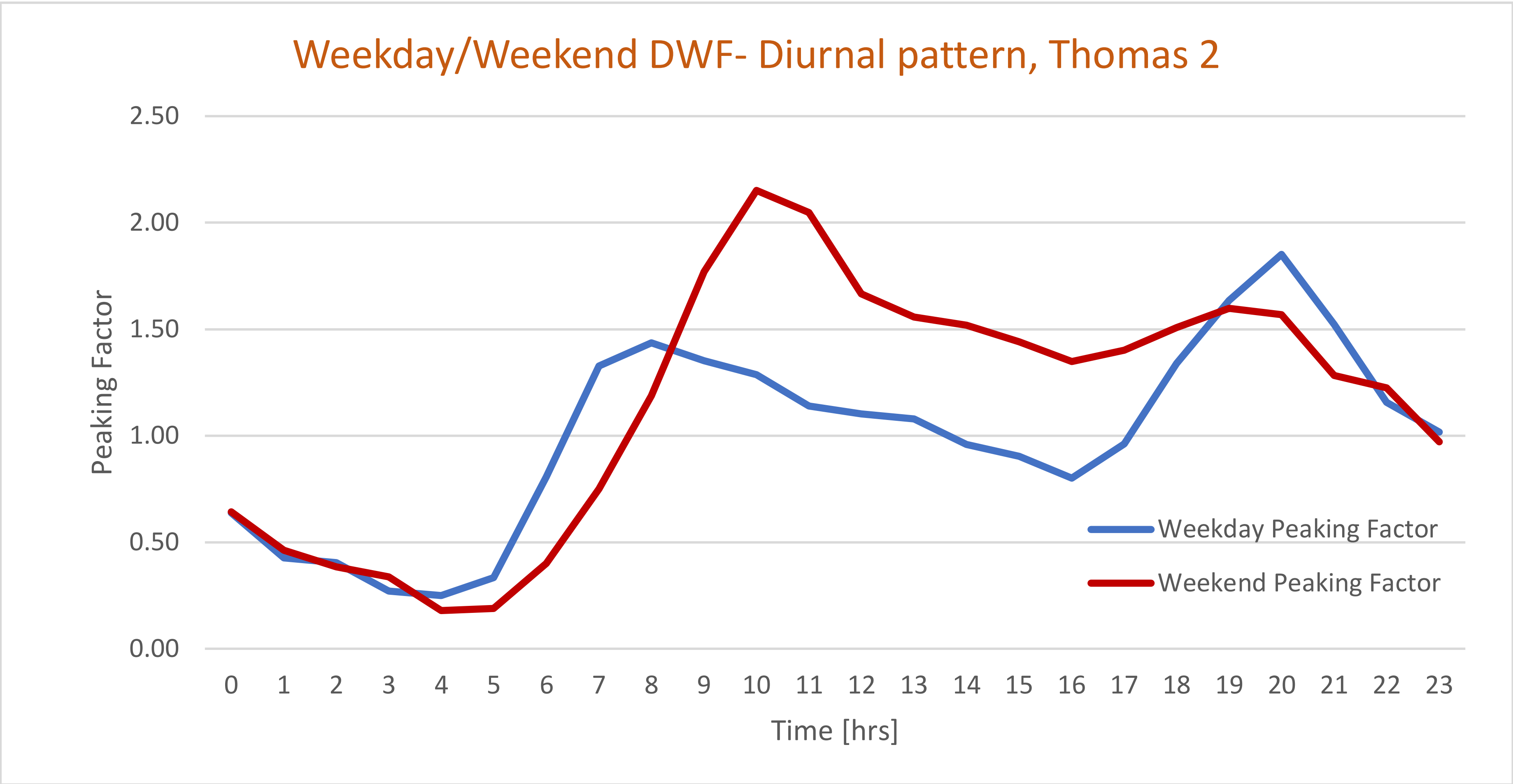
Weekend

Min.	11.03	
Average	20.01	
Daily average flow [L/d]	1,728,968.11	
Subtract trade flows [L/d] set t	0.00	
Residential flow [L/d]	1,728,968.11	
Metered Pop	3,153.00	
GWI %	85.00%	
GWI [L/s]= % x Min DWF	9.38	
GWI [L/d]	810247.79	
Per Capita flow [L/c/d]	291.38	
GWI [L/ha/s]	0.09	101.3 ha

Thomas 2 - Diurnal Profile (2020)



Thomas 2 - Diurnal Pattern (2020)



Thomas 3b - Weekday Data (2020)

Change Data Source of Pivot table

Row Labels	Average of Flow (l/s)
12 AM	5.79
1 AM	5.18
2 AM	4.32
3 AM	3.91
4 AM	3.95
5 AM	4.63
6 AM	6.08
7 AM	7.62
8 AM	9.07
9 AM	8.39
10 AM	8.28
11 AM	8.44
12 PM	8.41
1 PM	8.61
2 PM	8.47
3 PM	7.75
4 PM	7.79
5 PM	7.93
6 PM	8.59
7 PM	9.02
8 PM	8.87
9 PM	7.07
10 PM	6.72
11 PM	5.80
Average	**7.12**

Trade Flow (l/s): 0.05125

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	5.79	3.45	2.34	0	0	2.34	64%	0.64
1	5.18	3.45	1.73	0	0	1.73	48%	0.48
2	4.32	3.45	0.87	0	0	0.87	24%	0.24
3	3.91	3.45	0.46	0	0	0.46	13%	0.13
4	3.95	3.45	0.50	0	0	0.50	14%	0.14
5	4.63	3.45	1.18	0	0	1.18	32%	0.32
6	6.08	3.45	2.63	0	0	2.63	72%	0.72
7	7.62	3.45	4.17	1	0.05125	4.11	113%	1.13
8	9.07	3.45	5.62	1	0.05125	5.57	153%	1.53
9	8.39	3.45	4.94	1	0.05125	4.89	134%	1.34
10	8.28	3.45	4.83	1	0.05125	4.78	131%	1.31
11	8.44	3.45	4.99	1	0.05125	4.94	136%	1.36
12	8.41	3.45	4.96	1	0.05125	4.91	135%	1.35
13	8.61	3.45	5.16	1	0.05125	5.11	140%	1.40
14	8.47	3.45	5.02	1	0.05125	4.97	137%	1.37
15	7.75	3.45	4.30	1	0.05125	4.25	117%	1.17
16	7.79	3.45	4.34	1	0.05125	4.29	118%	1.18
17	7.93	3.45	4.48	1	0.05125	4.43	122%	1.22
18	8.59	3.45	5.14	0	0	5.14	141%	1.41
19	9.02	3.45	5.57	0	0	5.57	153%	1.53
20	8.87	3.45	5.42	0	0	5.42	149%	1.49
21	7.07	3.45	3.62	0	0	3.62	100%	1.00
22	6.72	3.45	3.27	0	0	3.27	90%	0.90
23	5.80	3.45	2.35	0	0	2.35	65%	0.65
Average	**7.11**				**0.023489583**	**3.64**	**1.00**	**1.00**
Min	**3.91**							
Max.	**9.07**							

Weekday

Min. total flow	3.91	
Average total flow	7.11	
Daily average flow [L/d]	614,569.69	
Subtract trade flows [L/d]	2,029.50	
Residential flow [L/d]	612,540.19	
Metered Pop	1,661.00	
GWI %	88.21%	
GWI [L/s]= % x Min DWF	3.45	
GWI [L/d]	298,080.00	
Per Capita flow [L/c/d]	189.32	
GWI [L/ha/s]	0.106547251	32.38 ha

Thomas 3b - Weekend Data (2020)

Change Data Source of Pivot table

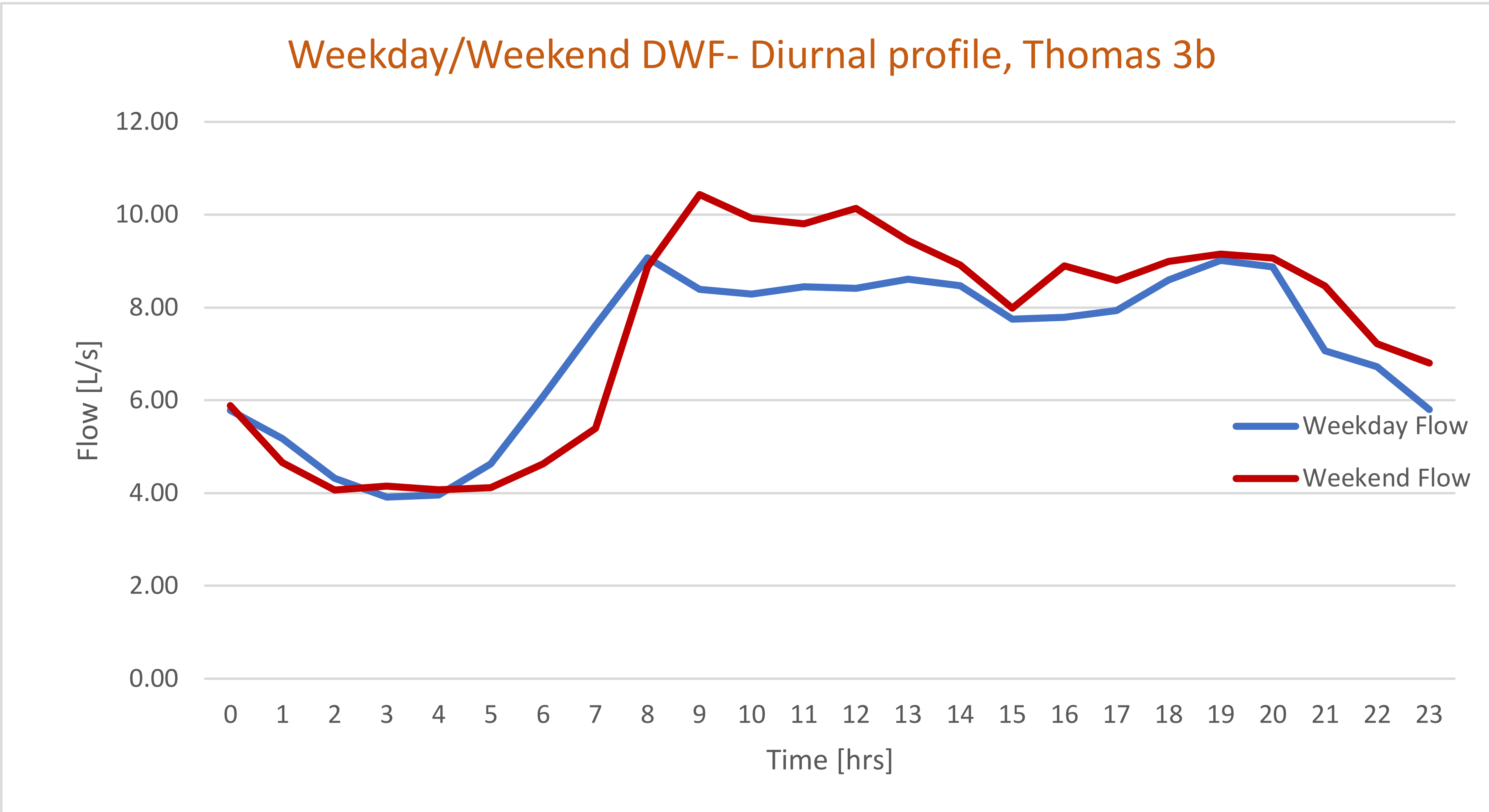
Row Labels	Average of Flow (l/s)
12 AM	5.88
1 AM	4.66
2 AM	4.06
3 AM	4.15
4 AM	4.07
5 AM	4.11
6 AM	4.63
7 AM	5.40
8 AM	8.87
9 AM	10.43
10 AM	9.92
11 AM	9.80
12 PM	10.14
1 PM	9.44
2 PM	8.91
3 PM	7.98
4 PM	8.90
5 PM	8.58
6 PM	8.99
7 PM	9.15
8 PM	9.07
9 PM	8.47
10 PM	7.22
11 PM	6.80
Average	**7.50**

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	5.88	3.45	2.42	60%	60%	0.67
1	4.66	3.45	1.20	30%	30%	0.33
2	4.06	3.45	0.61	15%	15%	0.17
3	4.15	3.45	0.69	17%	17%	0.19
4	4.07	3.45	0.62	15%	15%	0.17
5	4.11	3.45	0.66	16%	16%	0.18
6	4.63	3.45	1.18	29%	29%	0.32
7	5.40	3.45	1.94	48%	48%	0.53
8	8.87	3.45	5.41	134%	134%	1.49
9	10.43	3.45	6.98	173%	173%	1.92
10	9.92	3.45	6.46	160%	160%	1.78
11	9.80	3.45	6.35	157%	157%	1.74
12	10.14	3.45	6.68	166%	166%	1.84
13	9.44	3.45	5.99	149%	149%	1.65
14	8.91	3.45	5.45	135%	135%	1.50
15	7.98	3.45	4.53	112%	112%	1.24
16	8.90	3.45	5.44	135%	135%	1.50
17	8.58	3.45	5.13	127%	127%	1.41
18	8.99	3.45	5.54	137%	137%	1.52
19	9.15	3.45	5.70	141%	141%	1.56
20	9.07	3.45	5.61	139%	139%	1.54
21	8.47	3.45	5.01	124%	124%	1.38
22	7.22	3.45	3.77	93%	93%	1.04
23	6.80	3.45	3.35	83%	83%	0.92
Adjust factor for weekday PCD use in model:						1.11
Average	**7.48**		**4.03**	**1.00**	**1.00**	**1.11**
Min	**4.06**					
Max	**10.43**					

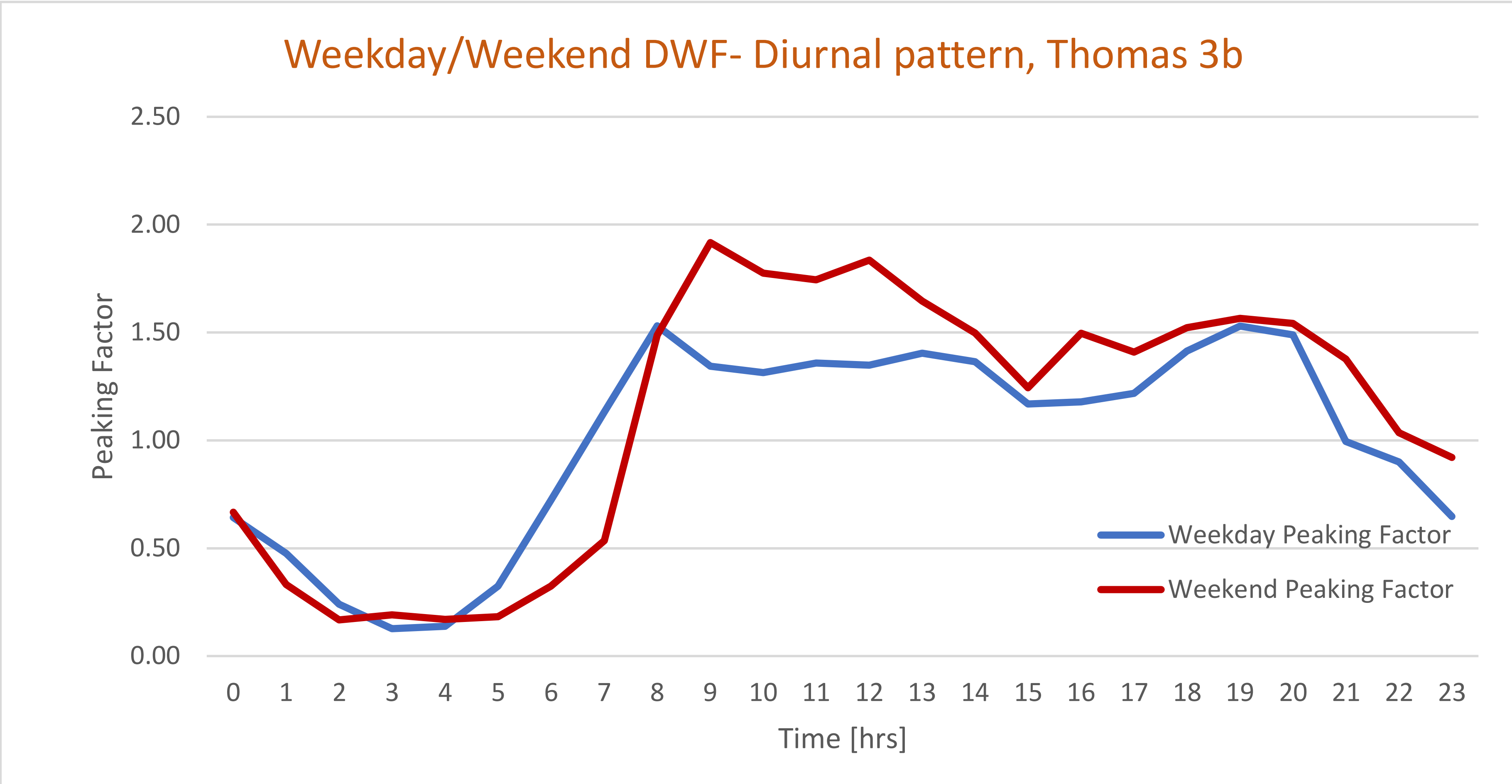
Weekend

Min.	4.06	
Average	7.48	
Daily average flow [L/d]	646,632.20	
Subtract trade flows [L/d] set t	0.00	
Residential flow [L/d]	646,632.20	
Metered Pop	1,661.00	
GWI %	85.00%	
GWI [L/s]= % x Min DWF	3.45	
GWI [L/d]	298365.61	
Per Capita flow [L/c/d]	209.67	
GWI [L/ha/s]	0.11	32.38 ha

Thomas 3b - Diurnal Profile (2020)



Thomas 3b - Diurnal Pattern (2020)



Thomas 4a - Weekday Data (2020)

Change Data Source of Pivot table

Trade Flow (l/s): 0.14

Row Labels	Average of Flow (l/s)
12 AM	5.18
1 AM	4.28
2 AM	4.37
3 AM	4.97
4 AM	4.93
5 AM	5.04
6 AM	6.09
7 AM	6.80
8 AM	7.23
9 AM	7.18
10 AM	7.46
11 AM	7.05
12 PM	6.63
1 PM	6.73
2 PM	6.51
3 PM	6.41
4 PM	6.62
5 PM	7.21
6 PM	7.42
7 PM	7.15
8 PM	7.36
9 PM	7.09
10 PM	6.39
11 PM	5.86
Average	**6.33**

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	5.18	2.61	2.57	0	0	2.57	70%	0.70
1	4.28	2.61	1.67	0	0	1.67	46%	0.46
2	4.37	2.61	1.76	0	0	1.76	48%	0.48
3	4.97	2.61	2.36	0	0	2.36	65%	0.65
4	4.93	2.61	2.32	0	0	2.32	63%	0.63
5	5.04	2.61	2.43	0	0	2.43	67%	0.67
6	6.09	2.61	3.48	0	0	3.48	95%	0.95
7	6.80	2.61	4.19	1	0.14	4.05	111%	1.11
8	7.23	2.61	4.62	1	0.14	4.48	122%	1.22
9	7.18	2.61	4.57	1	0.14	4.43	121%	1.21
10	7.46	2.61	4.85	1	0.14	4.71	129%	1.29
11	7.05	2.61	4.44	1	0.14	4.30	118%	1.18
12	6.63	2.61	4.02	1	0.14	3.88	106%	1.06
13	6.73	2.61	4.12	1	0.14	3.98	109%	1.09
14	6.51	2.61	3.90	1	0.14	3.76	103%	1.03
15	6.41	2.61	3.80	1	0.14	3.66	100%	1.00
16	6.62	2.61	4.01	1	0.14	3.87	106%	1.06
17	7.21	2.61	4.60	1	0.14	4.46	122%	1.22
18	7.42	2.61	4.81	0	0	4.81	131%	1.31
19	7.15	2.61	4.54	0	0	4.54	124%	1.24
20	7.36	2.61	4.75	0	0	4.75	130%	1.30
21	7.09	2.61	4.48	0	0	4.48	123%	1.23
22	6.39	2.61	3.78	0	0	3.78	103%	1.03
23	5.86	2.61	3.25	0	0	3.25	89%	0.89
Average	**6.33**				**0.064166667**	**3.66**	**1.00**	**1.00**
Min	**4.28**							
Max.	**7.46**							

Weekday

Min. total flow	4.28	
Average total flow	6.33	
Daily average flow [L/d]	547,081.66	
Subtract trade flows [L/d]	5,544.00	
Residential flow [L/d]	541,537.66	
Metered Pop	839.55	
GWI %	60.93%	
GWI [L/s]= % x Min DWF	2.61	
GWI [L/d]	225,504.00	
Per Capita flow [L/c/d]	376.43	
GWI [L/ha/s]	0.077517078	33.67 ha

Thomas 4a - Weekend Data (2020)

Change Data Source of Pivot table

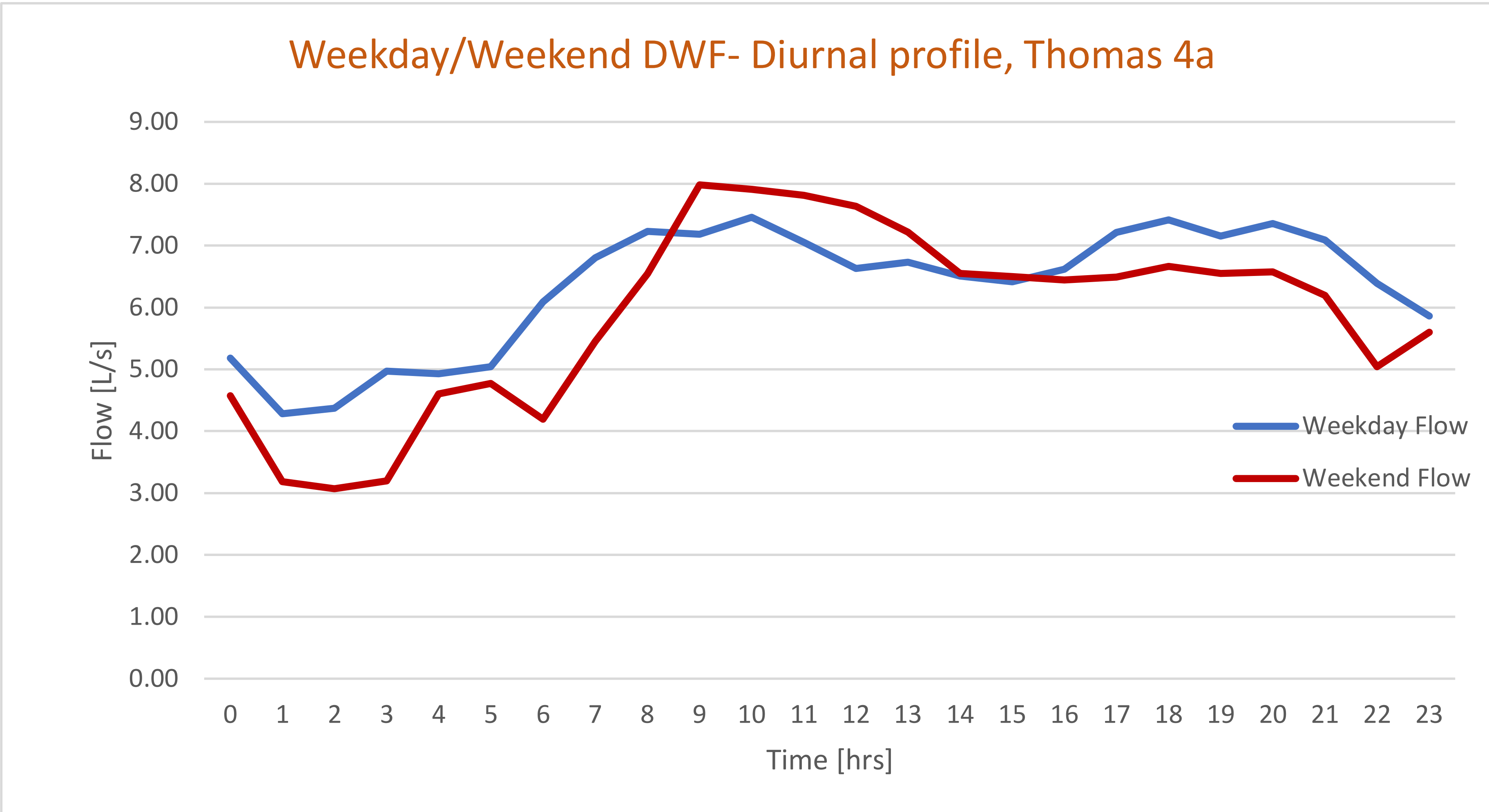
Row Labels	Average of Flow (l/s)
12 AM	4.57
1 AM	3.19
2 AM	3.07
3 AM	3.20
4 AM	4.61
5 AM	4.77
6 AM	4.20
7 AM	5.45
8 AM	6.54
9 AM	7.98
10 AM	7.91
11 AM	7.82
12 PM	7.63
1 PM	7.22
2 PM	6.55
3 PM	6.50
4 PM	6.44
5 PM	6.49
6 PM	6.67
7 PM	6.55
8 PM	6.58
9 PM	6.19
10 PM	5.04
11 PM	5.60
Average	**5.87**

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	4.57	2.61	1.96	60%	60%	0.54
1	3.19	2.61	0.57	18%	18%	0.16
2	3.07	2.61	0.46	14%	14%	0.13
3	3.20	2.61	0.59	18%	18%	0.16
4	4.61	2.61	1.99	61%	61%	0.55
5	4.77	2.61	2.16	66%	66%	0.59
6	4.20	2.61	1.59	49%	49%	0.43
7	5.45	2.61	2.84	87%	87%	0.78
8	6.54	2.61	3.93	121%	121%	1.08
9	7.98	2.61	5.37	165%	165%	1.47
10	7.91	2.61	5.30	163%	163%	1.45
11	7.82	2.61	5.21	160%	160%	1.42
12	7.63	2.61	5.02	154%	154%	1.37
13	7.22	2.61	4.61	141%	141%	1.26
14	6.55	2.61	3.94	121%	121%	1.08
15	6.50	2.61	3.89	119%	119%	1.06
16	6.44	2.61	3.83	118%	118%	1.05
17	6.49	2.61	3.88	119%	119%	1.06
18	6.67	2.61	4.06	125%	125%	1.11
19	6.55	2.61	3.94	121%	121%	1.08
20	6.58	2.61	3.97	122%	122%	1.08
21	6.19	2.61	3.58	110%	110%	0.98
22	5.04	2.61	2.43	75%	75%	0.66
23	5.60	2.61	2.99	92%	92%	0.82
Adjust factor for weekday PCD use in model:						0.89
Average	**5.87**		**3.26**	**1.00**	**1.00**	**0.89**
Min	**3.07**					
Max	**7.98**					

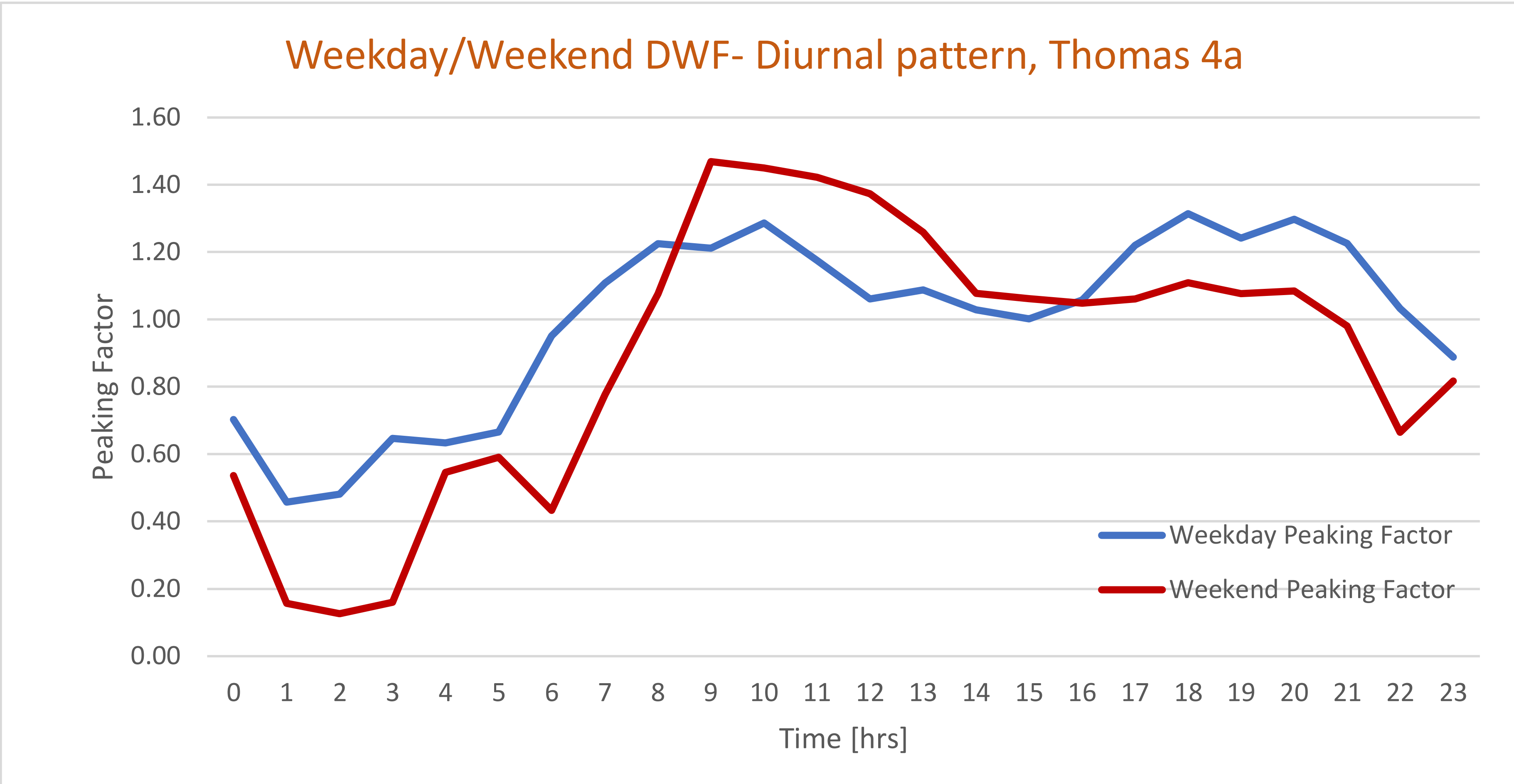
Weekend		
Min.	3.07	
Average	5.87	
Daily average flow [L/d]	506,791.19	
Subtract trade flows [L/d] set t	0.00	
Residential flow [L/d]	506,791.19	
Metered Pop	839.55	
GWI %	85.00%	
GWI [L/s]= % x Min DWF	2.61	
GWI [L/d]	225544.35	
Per Capita flow [L/c/d]	335.00	
GWI [L/ha/s]	0.08	33.67 ha

arva

Thomas 4a - Diurnal Profile (2020)



Thomas 4a - Diurnal Pattern (2020)



Thomas 4b - Weekday Data (2020)

Change Data Source of Pivot table

Trade Flow (l/s): 1.3224

Row Labels	Average of Flow (l/s)
12 AM	11.14
1 AM	9.28
2 AM	8.97
3 AM	8.09
4 AM	8.64
5 AM	8.60
6 AM	9.97
7 AM	13.61
8 AM	16.57
9 AM	15.89
10 AM	16.48
11 AM	15.89
12 PM	15.09
1 PM	14.43
2 PM	14.33
3 PM	12.40
4 PM	13.64
5 PM	14.92
6 PM	16.78
7 PM	17.61
8 PM	18.28
9 PM	17.67
10 PM	13.05
11 PM	14.17
Average	**13.56**

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	11.14	5.75	5.39	0	0	5.39	75%	0.75
1	9.28	5.75	3.53	0	0	3.53	49%	0.49
2	8.97	5.75	3.22	0	0	3.22	45%	0.45
3	8.09	5.75	2.34	0	0	2.34	32%	0.32
4	8.64	5.75	2.89	0	0	2.89	40%	0.40
5	8.60	5.75	2.85	0	0	2.85	40%	0.40
6	9.97	5.75	4.22	0	0	4.22	59%	0.59
7	13.61	5.75	7.86	1	1.3224	6.54	91%	0.91
8	16.57	5.75	10.82	1	1.3224	9.50	132%	1.32
9	15.89	5.75	10.14	1	1.3224	8.82	122%	1.22
10	16.48	5.75	10.73	1	1.3224	9.40	130%	1.30
11	15.89	5.75	10.14	1	1.3224	8.81	122%	1.22
12	15.09	5.75	9.34	1	1.3224	8.02	111%	1.11
13	14.43	5.75	8.68	1	1.3224	7.36	102%	1.02
14	14.33	5.75	8.58	1	1.3224	7.26	101%	1.01
15	12.40	5.75	6.65	1	1.3224	5.33	74%	0.74
16	13.64	5.75	7.89	1	1.3224	6.57	91%	0.91
17	14.92	5.75	9.17	1	1.3224	7.84	109%	1.09
18	16.78	5.75	11.03	0	0	11.03	153%	1.53
19	17.61	5.75	11.86	0	0	11.86	165%	1.65
20	18.28	5.75	12.53	0	0	12.53	174%	1.74
21	17.67	5.75	11.92	0	0	11.92	165%	1.65
22	13.05	5.75	7.30	0	0	7.30	101%	1.01
23	14.17	5.75	8.42	0	0	8.42	117%	1.17
Average	**13.56**				**0.6061**	**7.21**	**1.00**	**1.00**
Min	**8.09**							
Max.	**18.28**							

Weekday		
Min. total flow	8.09	
Average total flow	13.56	
Daily average flow [L/d]	1,171,757.12	
Subtract trade flows [L/d]	52,367.04	
Residential flow [L/d]	1,119,390.08	
Metered Pop	4,322.90	
GWI %	71.11%	
GWI [L/s]= % x Min DWF	5.75	
GWI [L/d]	496,800.00	
Per Capita flow [L/c/d]	144.02	
GWI [L/ha/s]	0.040510075	141.94 ha

Thomas 4b - Weekend Data (2020)

Change Data Source of Pivot table

Row Labels	Average of Flow (l/s)
12 AM	12.13
1 AM	8.44
2 AM	7.16
3 AM	6.76
4 AM	7.91
5 AM	8.09
6 AM	7.10
7 AM	8.46
8 AM	12.86
9 AM	18.92
10 AM	19.37
11 AM	19.90
12 PM	19.80
1 PM	19.97
2 PM	17.04
3 PM	15.20
4 PM	15.43
5 PM	15.42
6 PM	16.40
7 PM	16.75
8 PM	17.08
9 PM	13.07
10 PM	14.47
11 PM	10.40
Average	**13.67**

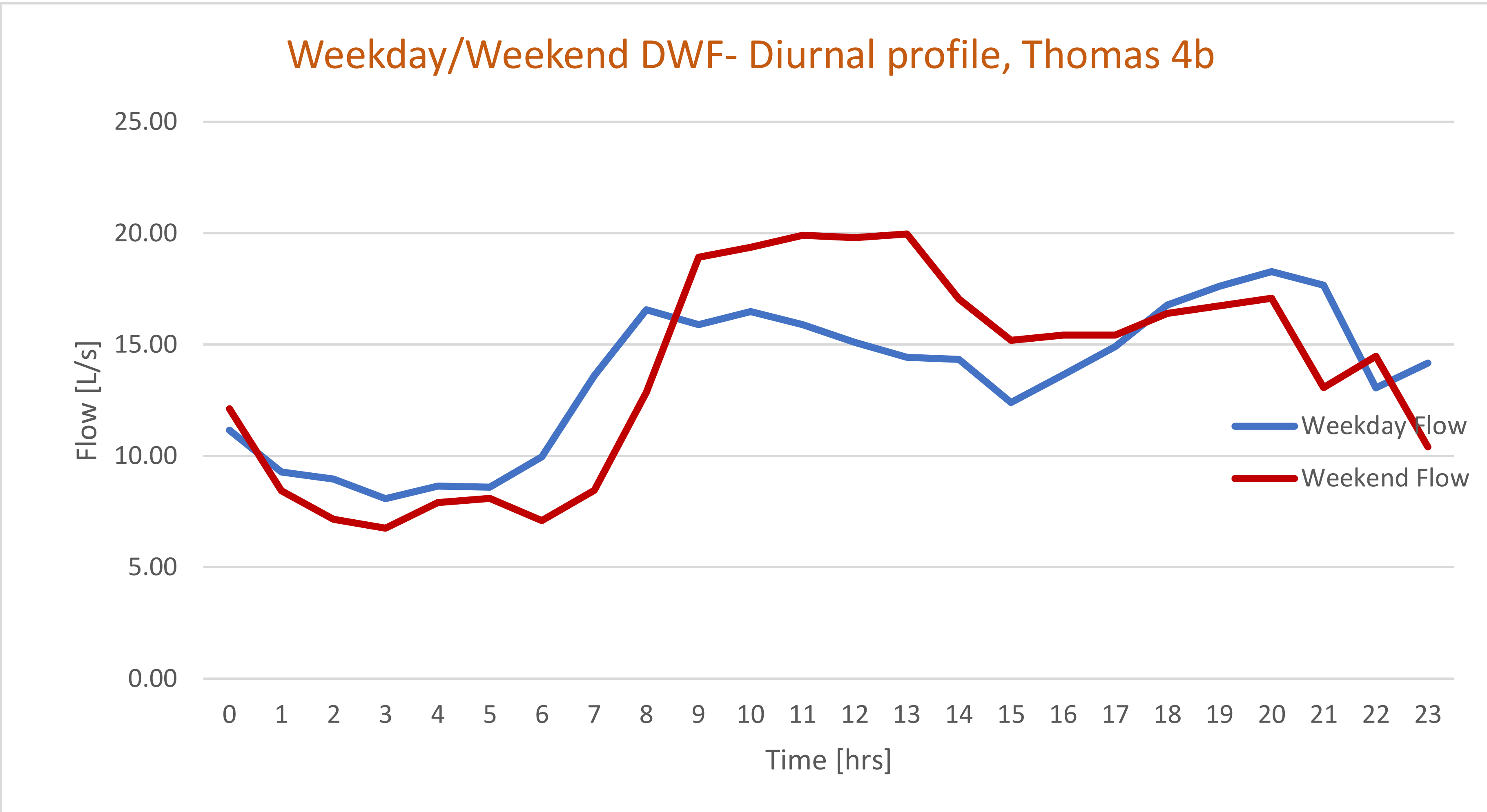
Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	12.13	5.75	6.38	81%	81%	0.89
1	8.44	5.75	2.69	34%	34%	0.37
2	7.16	5.75	1.41	18%	18%	0.20
3	6.76	5.75	1.01	13%	13%	0.14
4	7.91	5.75	2.16	27%	27%	0.30
5	8.09	5.75	2.34	30%	30%	0.33
6	7.10	5.75	1.35	17%	17%	0.19
7	8.46	5.75	2.72	34%	34%	0.38
8	12.86	5.75	7.12	90%	90%	0.99
9	18.92	5.75	13.18	166%	166%	1.83
10	19.37	5.75	13.62	172%	172%	1.89
11	19.90	5.75	14.16	179%	179%	1.96
12	19.80	5.75	14.05	177%	177%	1.95
13	19.97	5.75	14.22	179%	179%	1.97
14	17.04	5.75	11.30	143%	143%	1.57
15	15.20	5.75	9.45	119%	119%	1.31
16	15.43	5.75	9.68	122%	122%	1.34
17	15.42	5.75	9.67	122%	122%	1.34
18	16.40	5.75	10.65	134%	134%	1.48
19	16.75	5.75	11.00	139%	139%	1.53
20	17.08	5.75	11.33	143%	143%	1.57
21	13.07	5.75	7.32	92%	92%	1.02
22	14.47	5.75	8.73	110%	110%	1.21
23	10.40	5.75	4.66	59%	59%	0.65
Adjust factor for weekday PCD use in model:						1.10
Average	**13.67**		**7.93**	**1.00**	**1.00**	**1.10**
Min	**6.76**					
Max	**19.97**					

Weekend

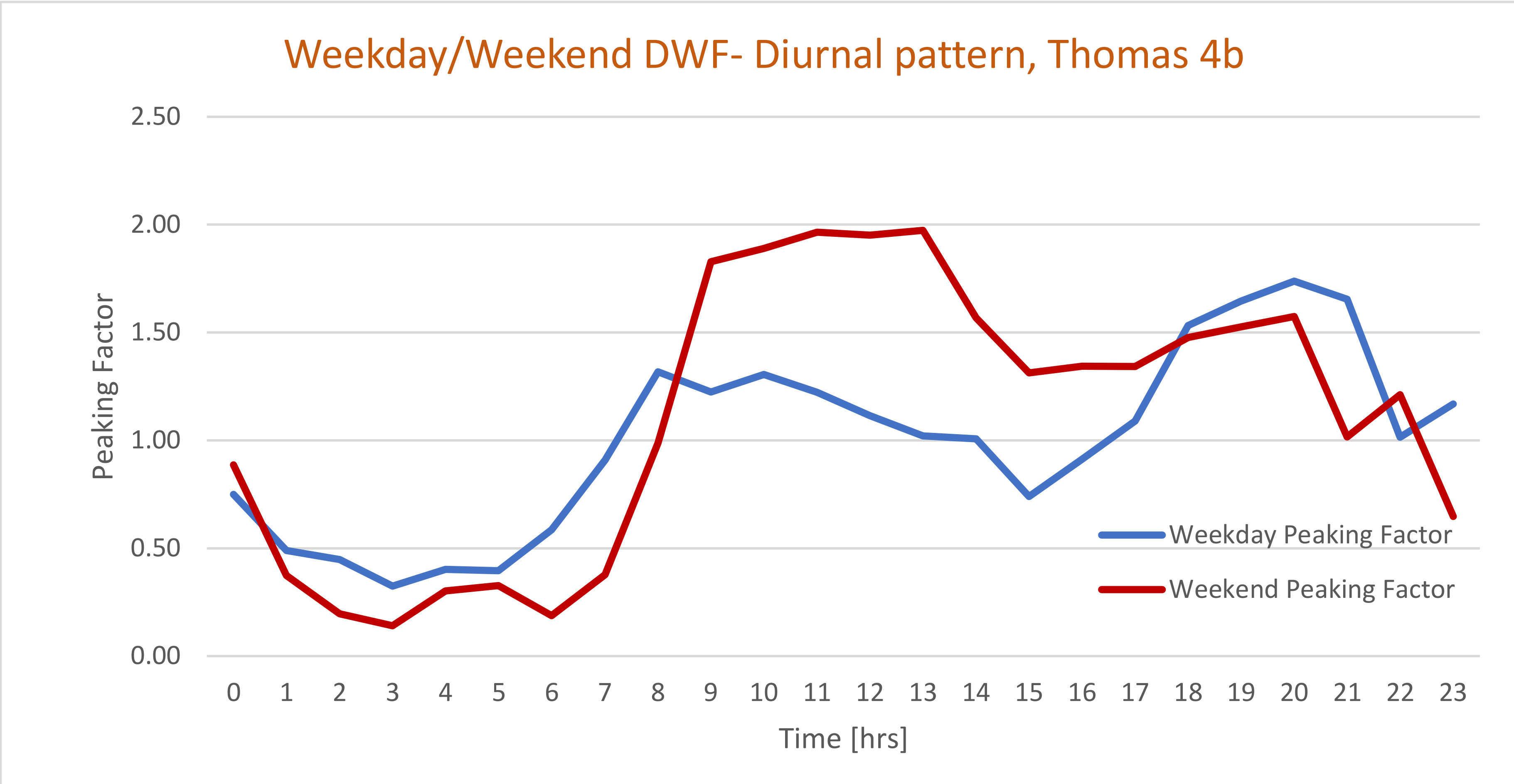
Min.	6.76	
Average	13.67	
Daily average flow [L/d]	1,181,195.04	
Subtract trade flows [L/d] set t	0.00	
Residential flow [L/d]	1,181,195.04	
Metered Pop	4,322.90	
GWI %	85.00%	
GWI [L/s]= % x Min DWF	5.75	
GWI [L/d]	496413.63	
Per Capita flow [L/c/d]	158.41	
GWI [L/ha/s]	0.04	141.94 ha

arva

Thomas 4b - Diurnal Profile (2020)



Thomas 4b - Diurnal Pattern (2020)



Thomas 5 - Weekday Data (2020)

Change Data Source of Pivot table

Trade Flow (l/s): 0.33902

Row Labels	Average of Flow (l/s)
12 AM	12.30
1 AM	10.35
2 AM	10.46
3 AM	10.20
4 AM	9.09
5 AM	9.93
6 AM	13.14
7 AM	20.95
8 AM	23.55
9 AM	21.51
10 AM	18.11
11 AM	17.45
12 PM	16.49
1 PM	16.72
2 PM	15.05
3 PM	14.76
4 PM	15.78
5 PM	17.63
6 PM	24.02
7 PM	22.94
8 PM	23.20
9 PM	20.95
10 PM	17.43
11 PM	13.98
Average	**16.50**

Time	Flow	GWI [L/s]	Flow - GWI [l/s]	Trade flow pattern	Trade flow based on original pattern	Remaining residential flow	% of Average residential flow	Normalized factor
0	12.30	6.53	5.77	0	0	5.77	59%	0.59
1	10.35	6.53	3.82	0	0	3.82	39%	0.39
2	10.46	6.53	3.93	0	0	3.93	40%	0.40
3	10.20	6.53	3.67	0	0	3.67	37%	0.37
4	9.09	6.53	2.56	0	0	2.56	26%	0.26
5	9.93	6.53	3.40	0	0	3.40	35%	0.35
6	13.14	6.53	6.61	0	0	6.61	67%	0.67
7	20.95	6.53	14.42	1	0.33902	14.09	144%	1.44
8	23.55	6.53	17.02	1	0.33902	16.68	170%	1.70
9	21.51	6.53	14.98	1	0.33902	14.64	149%	1.49
10	18.11	6.53	11.58	1	0.33902	11.24	115%	1.15
11	17.45	6.53	10.92	1	0.33902	10.58	108%	1.08
12	16.49	6.53	9.96	1	0.33902	9.62	98%	0.98
13	16.72	6.53	10.19	1	0.33902	9.85	100%	1.00
14	15.05	6.53	8.52	1	0.33902	8.18	83%	0.83
15	14.76	6.53	8.23	1	0.33902	7.89	80%	0.80
16	15.78	6.53	9.25	1	0.33902	8.91	91%	0.91
17	17.63	6.53	11.10	1	0.33902	10.76	110%	1.10
18	24.02	6.53	17.49	0	0	17.49	178%	1.78
19	22.94	6.53	16.41	0	0	16.41	167%	1.67
20	23.20	6.53	16.67	0	0	16.67	170%	1.70
21	20.95	6.53	14.42	0	0	14.42	147%	1.47
22	17.43	6.53	10.90	0	0	10.90	111%	1.11
23	13.98	6.53	7.45	0	0	7.45	76%	0.76
Average	**16.50**				**0.155384167**	**9.81**	**1.00**	**1.00**
Min	**9.09**							
Max.	**24.02**							

Weekday

Min. total flow	9.09
Average total flow	16.50
Daily average flow [L/d]	1,425,569.10
Subtract trade flows [L/d]	13,425.19
Residential flow [L/d]	1,412,143.90
Metered Pop	5,425.10
GWI %	71.86%
GWI [L/s]= % x Min DWF	6.53
GWI [L/d]	564,192.00
Per Capita flow [L/c/d]	156.30
GWI [L/ha/s]	0.039015355

167.37 ha

Thomas 5 - Weekend Data (2020)

Change Data Source of Pivot table

Row Labels	Average of Flow (l/s)
12 AM	13.09
1 AM	9.59
2 AM	8.36
3 AM	7.79
4 AM	8.97
5 AM	7.76
6 AM	8.69
7 AM	11.22
8 AM	18.95
9 AM	23.98
10 AM	27.35
11 AM	26.60
12 PM	24.56
1 PM	21.63
2 PM	21.23
3 PM	19.30
4 PM	18.50
5 PM	20.53
6 PM	21.36
7 PM	23.05
8 PM	20.99
9 PM	21.24
10 PM	15.72
11 PM	12.80
Average	**17.22**

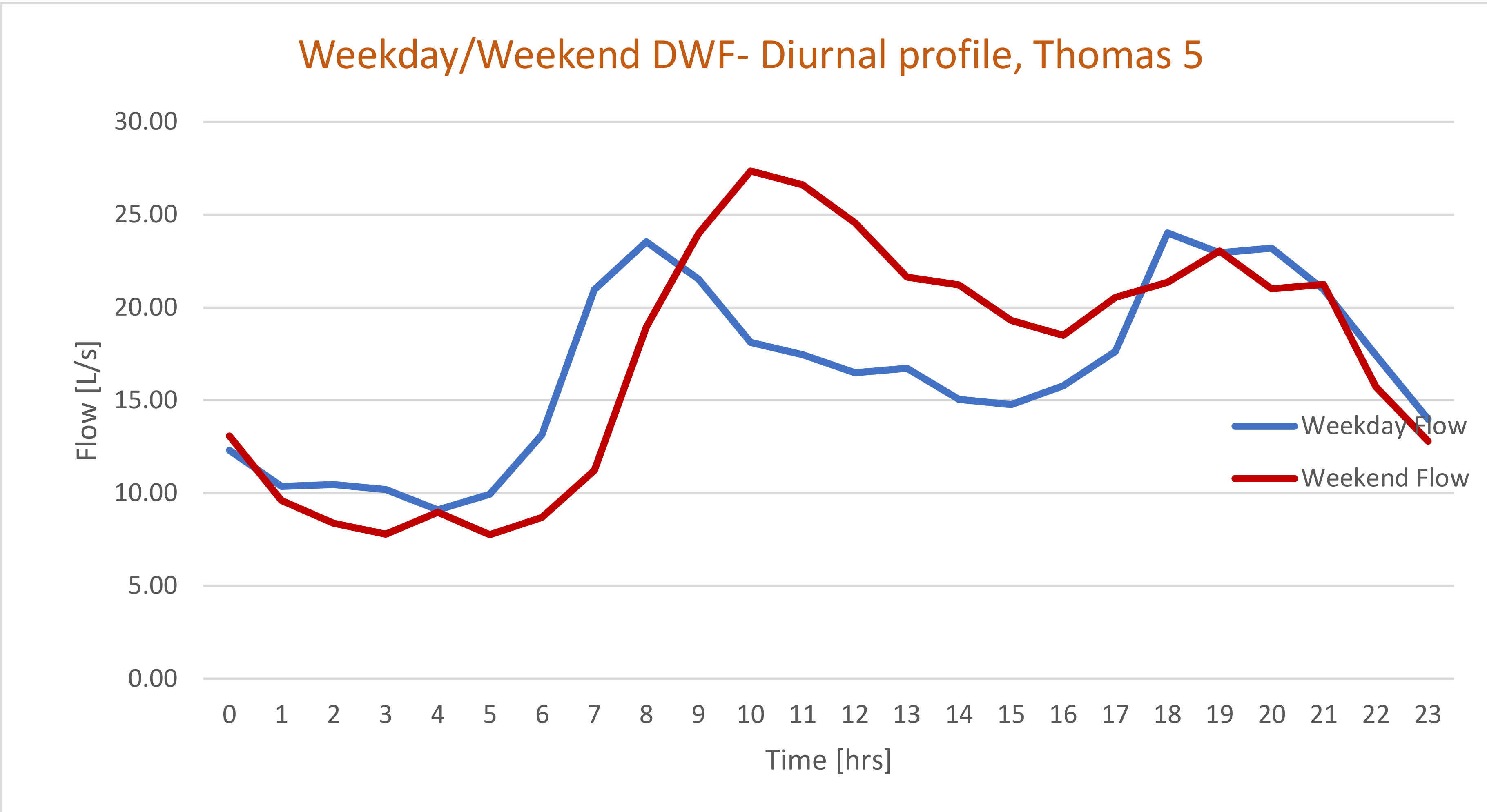
Time	Flow	GWI [L/s]	Flow - GWI [l/s]	% of Average	% Normalized	% Adjusted to weekday PCD
0	13.09	6.59	6.50	61%	61%	0.66
1	9.59	6.59	3.00	28%	28%	0.31
2	8.36	6.59	1.77	17%	17%	0.18
3	7.79	6.59	1.20	11%	11%	0.12
4	8.97	6.59	2.38	22%	22%	0.24
5	7.76	6.59	1.16	11%	11%	0.12
6	8.69	6.59	2.10	20%	20%	0.21
7	11.22	6.59	4.63	44%	44%	0.47
8	18.95	6.59	12.36	116%	116%	1.26
9	23.98	6.59	17.38	164%	164%	1.77
10	27.35	6.59	20.75	195%	195%	2.11
11	26.60	6.59	20.01	188%	188%	2.04
12	24.56	6.59	17.97	169%	169%	1.83
13	21.63	6.59	15.04	142%	142%	1.53
14	21.23	6.59	14.63	138%	138%	1.49
15	19.30	6.59	12.70	120%	120%	1.29
16	18.50	6.59	11.91	112%	112%	1.21
17	20.53	6.59	13.94	131%	131%	1.42
18	21.36	6.59	14.76	139%	139%	1.50
19	23.05	6.59	16.46	155%	155%	1.68
20	20.99	6.59	14.40	136%	136%	1.47
21	21.24	6.59	14.65	138%	138%	1.49
22	15.72	6.59	9.12	86%	86%	0.93
23	12.80	6.59	6.21	58%	58%	0.63
Adjust factor for weekday PCD use in model:						1.08
Average	**17.22**		**10.63**	**1.00**	**1.00**	**1.08**
Min	**7.76**					
Max	**27.35**					

Weekend

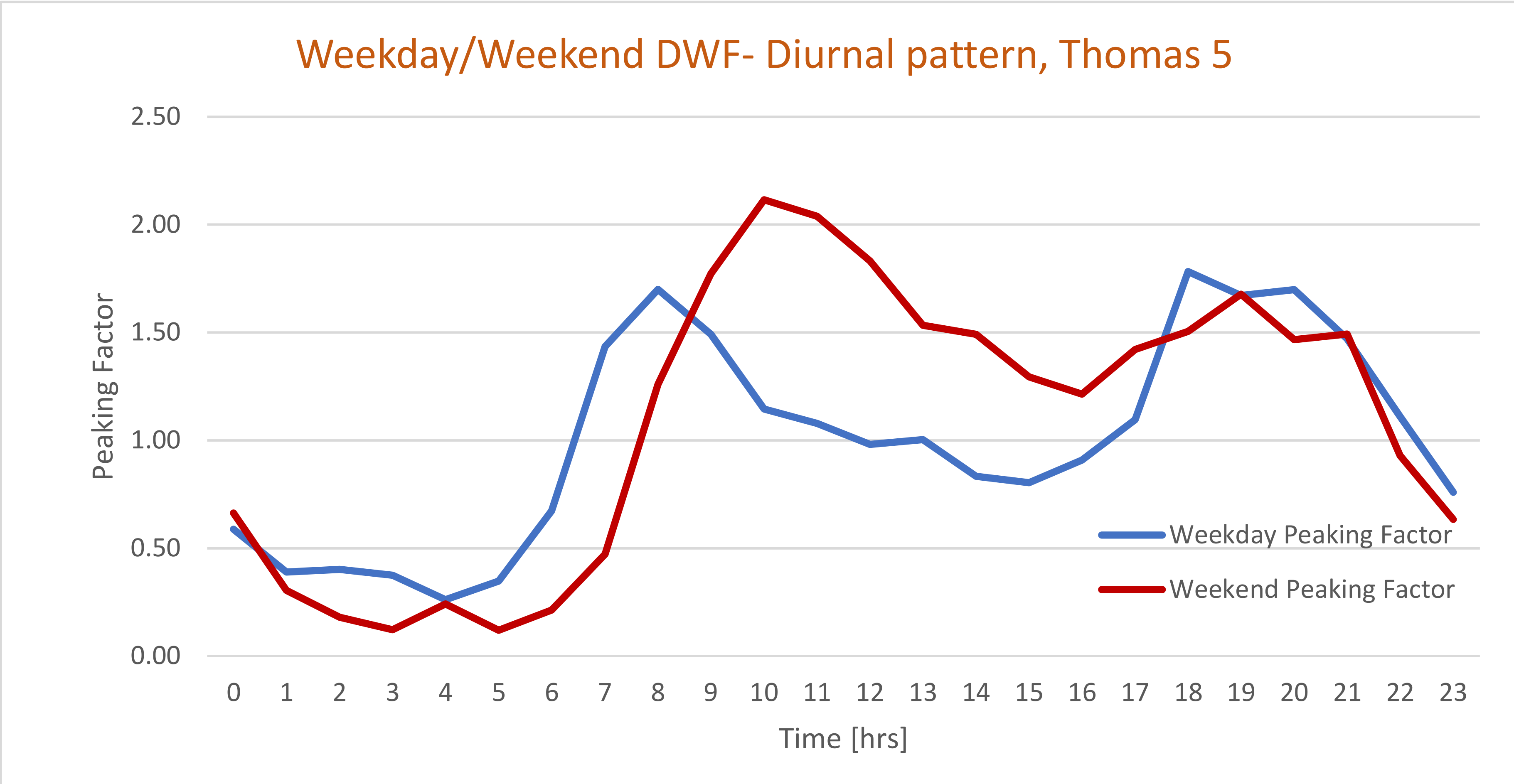
Min.	7.76	
Average	17.22	
Daily average flow [L/d]	1,487,703.99	
Subtract trade flows [L/d] set t	0.00	
Residential flow [L/d]	1,487,703.99	
Metered Pop	5,425.10	
GWI %	85.00%	
GWI [L/s]= % x Min DWF	6.59	
GWI [L/d]	569599.38	
Per Capita flow [L/c/d]	169.23	
GWI [L/ha/s]	0.04	167.37 ha

arva

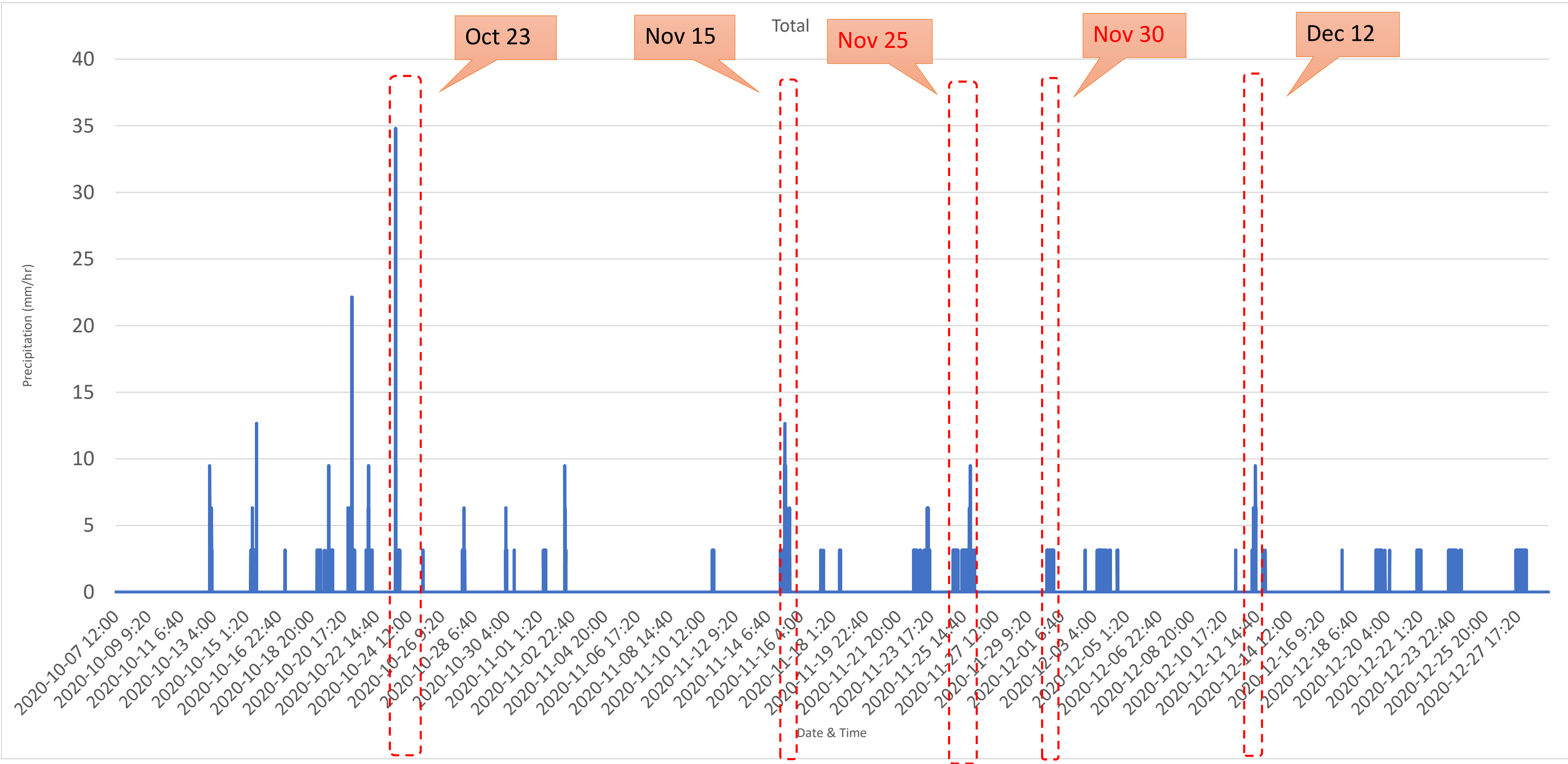
Thomas 5 - Diurnal Profile (2020)



Thomas 5 - Diurnal Pattern (2020)



RAINFALL ASSESSMENT - SELECTED DAYS



Use 5min		Before rain days, at least 3 days of no rain before					for wet weather events		
0		sum of 5 min data		this becomes mm/5min					
0				then multiply by 12					
0									
0		sum of 5 m	mm/hr						
0.263769		0.263769	3.16523						
0					mm	60min	12		
0					5min	1hr			

Event	Date	Min Prior D	Description
Event 1	Oct 23	1	Highest Peak
Event 2	Nov 15	3	Medium Peak Density
Event 3	Nov 25	1	Low rain spread in days
Event 4	Nov 30	3	Low rain half day -->will not pass (less than 6mm/hr)
Event 5	Dec 12	3 (with 1 hour rain)	Medium Peak Density

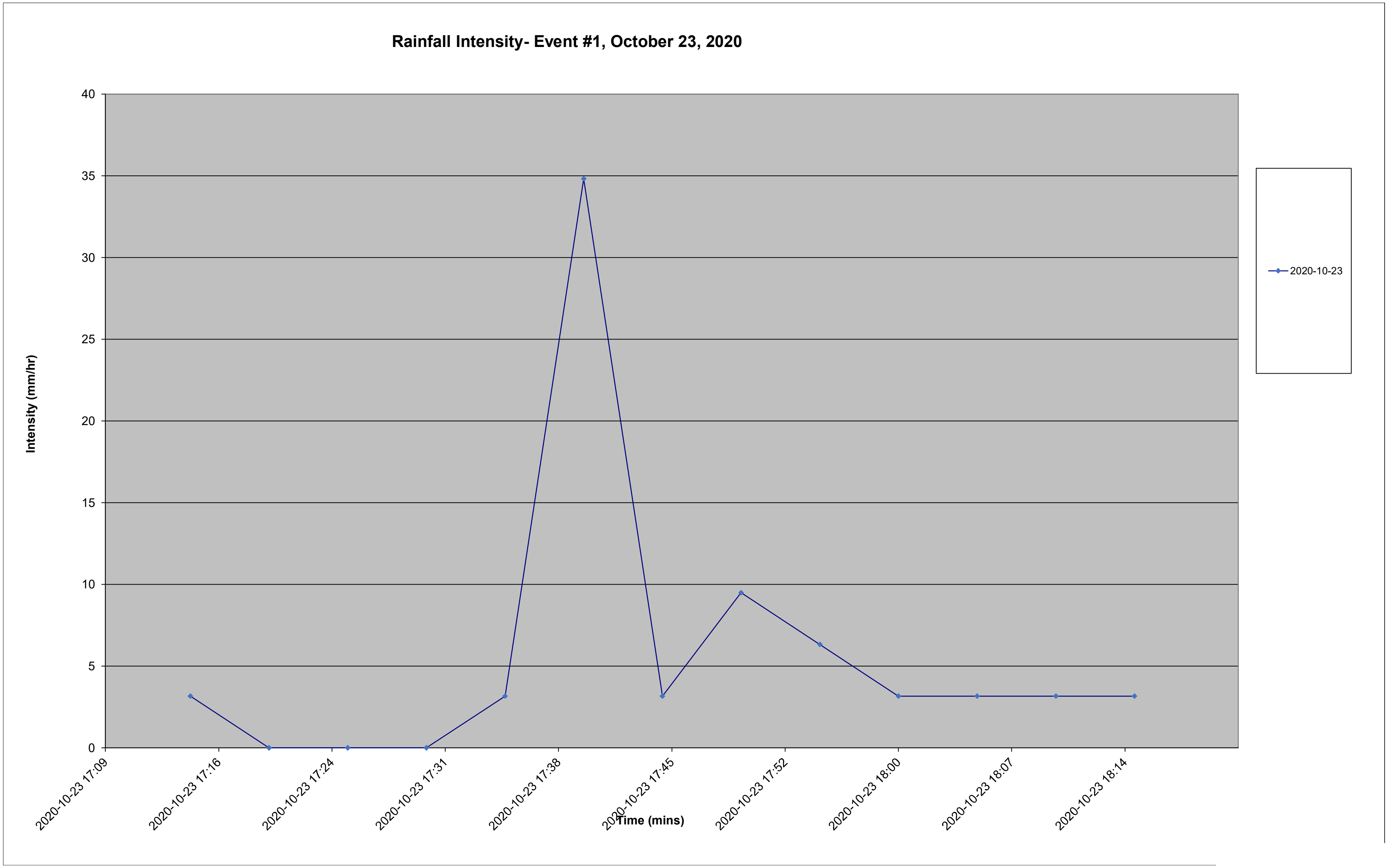
Event Date: 2020-10-23

Conclusion of Assessment = **Pass**

2020-10-23 17:20 to **2020-10-23 18:15**

Week 1	**Intensities (mm/hr)**
Date & Time	**23-Oct**
2020-10-23 17:15	3.1652304
2020-10-23 17:20	0
2020-10-23 17:25	0
2020-10-23 17:30	0
2020-10-23 17:35	3.1652304
2020-10-23 17:40	34.8175392
2020-10-23 17:45	3.1652304
2020-10-23 17:50	9.4956912
2020-10-23 17:55	6.3304608
2020-10-23 18:00	3.1652304
2020-10-23 18:05	3.1652304
2020-10-23 18:10	3.1652304
2020-10-23 18:15	3.1652304
Peak Intensity	**34.8175392**
6mm/hr>4 minutes?	**OK**

Timestep	**Rainfall Depths (mm)**
minutes	
	23-Oct
0	0.2638
5.00	0.0000
10.00	0.0000
15.00	0.0000
20.00	0.2638
25.00	2.9015
30.00	0.2638
35.00	0.7913
40.00	0.5275
45.00	0.2638
50.00	0.2638
55.00	0.2638
60.00	0.2638
Total Depth	**5.8029228**
Depth > 5mm	**OK**


Rainfall Intensity- Event #1, October 23, 2020
Intensity (mm/hr)
40
35
30
25
20
15
10
5
0
2020-10-23 17:09
2020-10-23 17:16
2020-10-23 17:24
2020-10-23 17:31
2020-10-23 17:38
2020-10-23 17:45
2020-10-23 17:52
2020-10-23 18:00
2020-10-23 18:07
2020-10-23 18:14
Time (mins)
2020-10-23

Event Date: 2020-11-15

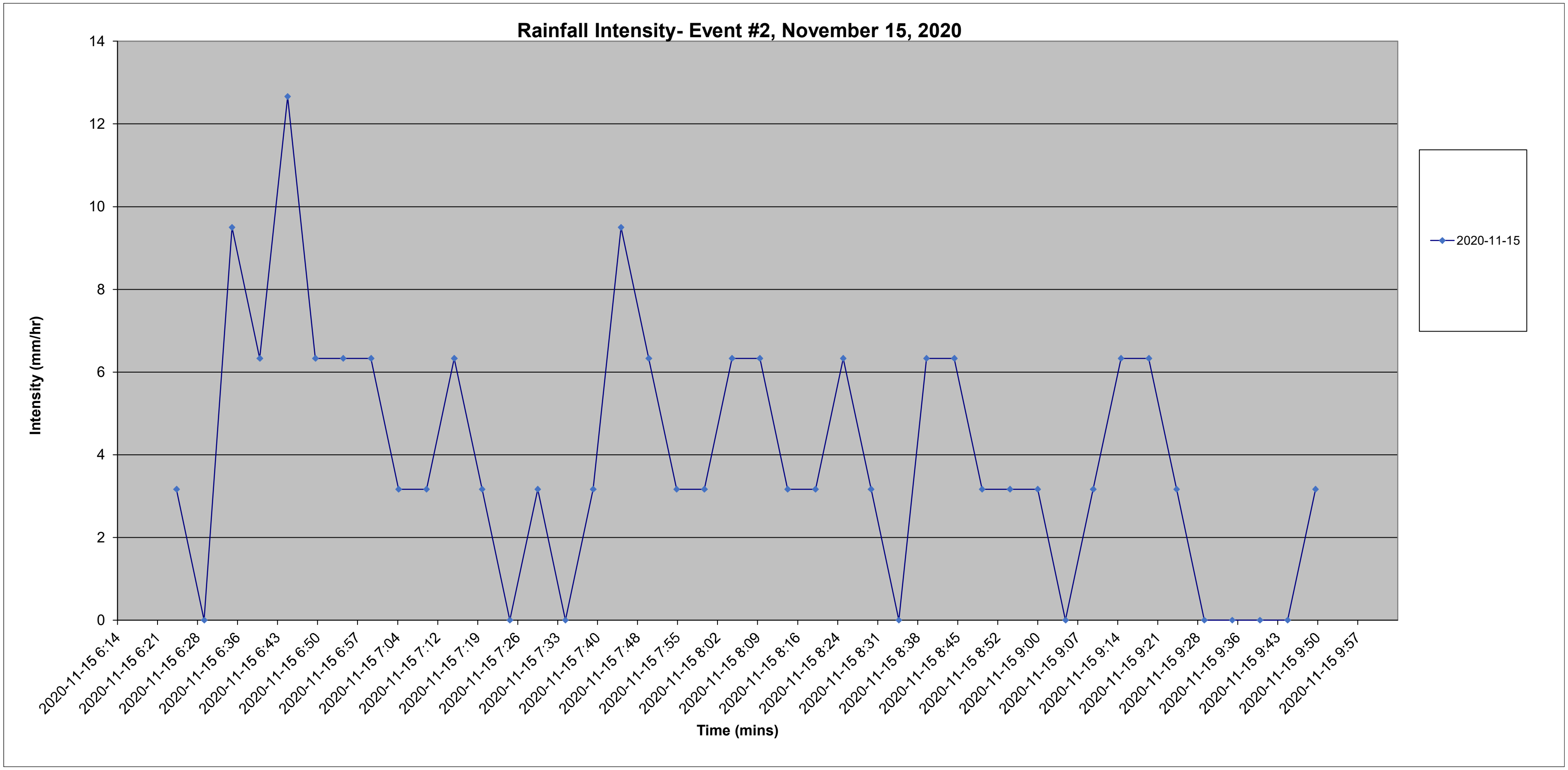
Conclusion of Assessment = **Pass**

2020-11-15 6:30 to **2020-11-15 9:50**

Week 1	**Intensities (mm/hr)**
Date & Time	**15-Nov**
2020-11-15 6:25	3.1652304
2020-11-15 6:30	0
2020-11-15 6:35	9.4956912
2020-11-15 6:40	6.3304608
2020-11-15 6:45	12.6609228
2020-11-15 6:50	6.3304608
2020-11-15 6:55	6.3304608
2020-11-15 7:00	6.3304608
2020-11-15 7:05	3.1652304
2020-11-15 7:10	3.1652304
2020-11-15 7:15	6.3304608
2020-11-15 7:20	3.1652304
2020-11-15 7:25	0
2020-11-15 7:30	3.1652304
2020-11-15 7:35	0
2020-11-15 7:40	3.1652304
2020-11-15 7:45	9.4956912
2020-11-15 7:50	6.3304608
2020-11-15 7:55	3.1652304
2020-11-15 8:00	3.1652304
2020-11-15 8:05	6.3304608
2020-11-15 8:10	6.3304608
2020-11-15 8:15	3.1652304
2020-11-15 8:20	3.1652304
2020-11-15 8:25	6.3304608
2020-11-15 8:30	3.1652304
2020-11-15 8:35	0
2020-11-15 8:40	6.3304608
2020-11-15 8:45	6.3304608
2020-11-15 8:50	3.1652304
2020-11-15 8:55	3.1652304
2020-11-15 9:00	3.1652304
2020-11-15 9:05	0
2020-11-15 9:10	3.1652304
2020-11-15 9:15	6.3304608
2020-11-15 9:20	6.3304608
2020-11-15 9:25	3.1652304
2020-11-15 9:30	0
2020-11-15 9:35	0
2020-11-15 9:40	0
2020-11-15 9:45	0
2020-11-15 9:50	3.1652304
Peak Intensity	**12.6609228**
6mm/hr>4 minutes?	**OK**

Timestep	**Rainfall Depths (mm)**
minutes	
	15-Nov
0	0.2637692
5.00	0
10.00	0.7913076
15.00	0.5275384
20.00	1.0550769
25.00	0.5275384
30.00	0.5275384
35.00	0.5275384
40.00	0.2637692
45.00	0.2637692
50.00	0.5275384
55.00	0.2637692
60.00	0
65.00	0.2637692
70.00	0
75.00	0.2637692
80.00	0.7913076
85.00	0.5275384
90.00	0.2637692
95.00	0.2637692
100.00	0.5275384
105.00	0.5275384
110.00	0.2637692
115.00	0.2637692
120.00	0.5275384
125.00	0.2637692
130.00	0
135.00	0.5275384
140.00	0.5275384
145.00	0.2637692
150.00	0.2637692
155.00	0.2637692
160.00	0
165.00	0.2637692
170.00	0.5275384
175.00	0.5275384
180.00	0.2637692
185.00	0
190.00	0
195.00	0
200.00	0
205.00	0.2637692
Total Depth	**13.7159985**
Depth > 5mm	**OK**

arva


Rainfall Intensity- Event #2, November 15, 2020
Intensity (mm/hr)
Time (mins)
2020-11-15
0
2
4
6
8
10
12
14
2020-11-15 6:14
2020-11-15 6:21
2020-11-15 6:28
2020-11-15 6:36
2020-11-15 6:43
2020-11-15 6:50
2020-11-15 6:57
2020-11-15 7:04
2020-11-15 7:12
2020-11-15 7:19
2020-11-15 7:26
2020-11-15 7:33
2020-11-15 7:40
2020-11-15 7:48
2020-11-15 7:55
2020-11-15 8:02
2020-11-15 8:09
2020-11-15 8:16
2020-11-15 8:24
2020-11-15 8:31
2020-11-15 8:38
2020-11-15 8:45
2020-11-15 8:52
2020-11-15 9:00
2020-11-15 9:07
2020-11-15 9:14
2020-11-15 9:21
2020-11-15 9:28
2020-11-15 9:36
2020-11-15 9:43
2020-11-15 9:50
2020-11-15 9:57

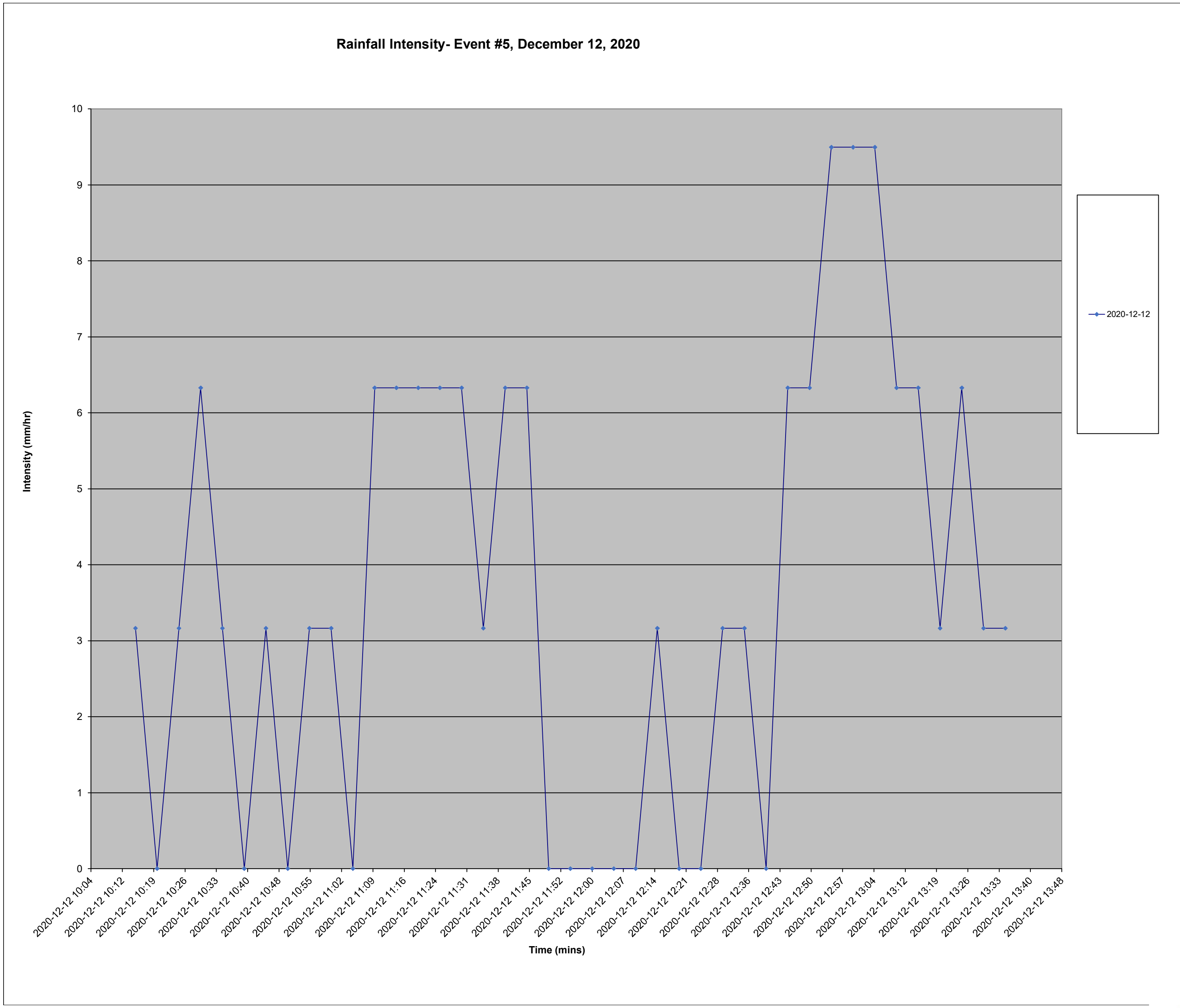
Event Date: 2020-12-12

Conclusion of Assessment = **Pass**

2020-12-12 10:20 to **2020-12-12 13:35**

Week 1	**Intensities (mm/hr)**
Date & Time	**12-Dec**
2020-12-12 10:15	3.1652304
2020-12-12 10:20	0
2020-12-12 10:25	3.1652304
2020-12-12 10:30	6.3304608
2020-12-12 10:35	3.1652304
2020-12-12 10:40	0
2020-12-12 10:45	3.1652304
2020-12-12 10:50	0
2020-12-12 10:55	3.1652304
2020-12-12 11:00	3.1652304
2020-12-12 11:05	0
2020-12-12 11:10	6.3304608
2020-12-12 11:15	6.3304608
2020-12-12 11:20	6.3304608
2020-12-12 11:25	6.3304608
2020-12-12 11:30	6.3304608
2020-12-12 11:35	3.1652304
2020-12-12 11:40	6.3304608
2020-12-12 11:45	6.3304608
2020-12-12 11:50	0
2020-12-12 11:55	0
2020-12-12 12:00	0
2020-12-12 12:05	0
2020-12-12 12:10	0
2020-12-12 12:15	3.1652304
2020-12-12 12:20	0
2020-12-12 12:25	0
2020-12-12 12:30	3.1652304
2020-12-12 12:35	3.1652304
2020-12-12 12:40	0
2020-12-12 12:45	6.3304608
2020-12-12 12:50	6.3304608
2020-12-12 12:55	9.4956912
2020-12-12 13:00	9.4956912
2020-12-12 13:05	9.4956912
2020-12-12 13:10	6.3304608
2020-12-12 13:15	6.3304608
2020-12-12 13:20	3.1652304
2020-12-12 13:25	6.3304608
2020-12-12 13:30	3.1652304
2020-12-12 13:35	3.1652304
Peak Intensity	**9.4956912**
6mm/hr>4 minutes?	**OK**

Timestep	**Rainfall Depths (mm)**
minutes	
	12-Dec
0	0.2637692
5.00	0
10.00	0.2637692
15.00	0.5275384
20.00	0.2637692
25.00	0
30.00	0.2637692
35.00	0
40.00	0.2637692
45.00	0.2637692
50.00	0
55.00	0.5275384
60.00	0.5275384
65.00	0.5275384
70.00	0.5275384
75.00	0.5275384
80.00	0.2637692
85.00	0.5275384
90.00	0.5275384
95.00	0
100.00	0
105.00	0
110.00	0
115.00	0
120.00	0.2637692
125.00	0
130.00	0
135.00	0.2637692
140.00	0.2637692
145.00	0
150.00	0.5275384
155.00	0.5275384
160.00	0.7913076
165.00	0.7913076
170.00	0.7913076
175.00	0.5275384
180.00	0.5275384
185.00	0.2637692
190.00	0.5275384
195.00	0.2637692
200.00	0.2637692
Total Depth	**12.3971524**
Depth > 5mm	**OK**



Rainfall Intensity- Event #5, December 12, 2020
Intensity (mm/hr)
10
9
8
7
6
5
4
3
2
1
0
2020-12-12 10:04
2020-12-12 10:12
2020-12-12 10:19
2020-12-12 10:26
2020-12-12 10:33
2020-12-12 10:40
2020-12-12 10:48
2020-12-12 10:55
2020-12-12 11:02
2020-12-12 11:09
2020-12-12 11:16
2020-12-12 11:24
2020-12-12 11:31
2020-12-12 11:38
2020-12-12 11:45
2020-12-12 11:52
2020-12-12 12:00
2020-12-12 12:07
2020-12-12 12:14
2020-12-12 12:21
2020-12-12 12:28
2020-12-12 12:36
2020-12-12 12:43
2020-12-12 12:50
2020-12-12 12:57
2020-12-12 13:04
2020-12-12 13:12
2020-12-12 13:19
2020-12-12 13:26
2020-12-12 13:33
2020-12-12 13:40
2020-12-12 13:48
Time (mins)
2020-12-12

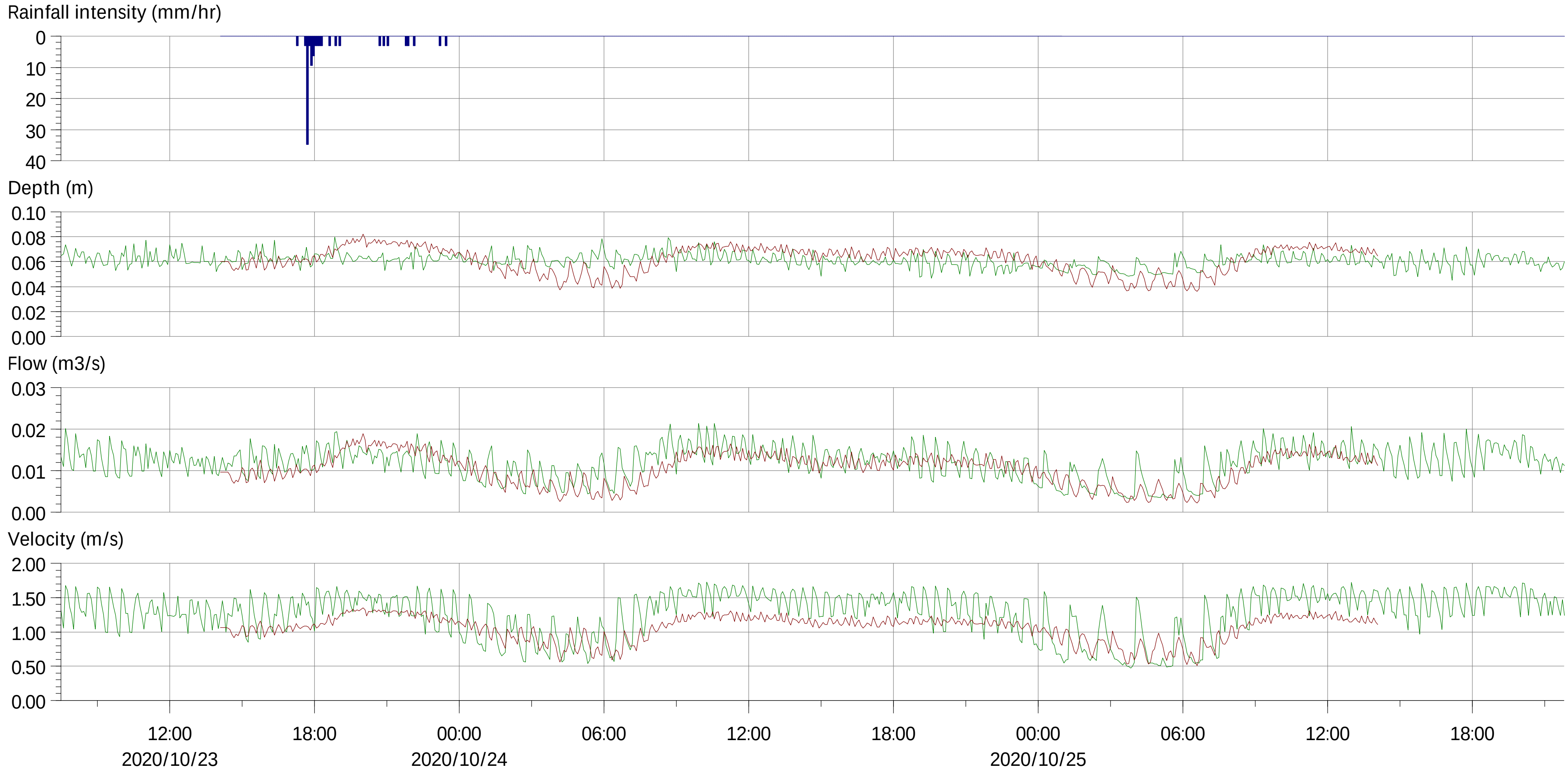
arva

APPENDIX 6

RTK Parameter Validation (comparison graphs monitored vs modelled)

Observed / Predicted Report Produced by Ssoumik (2021-07-29 5:41:01 PM) Page 1 of 6
Flow survey: >Model group>Flow Survey Group - 2>Flow Survey Group -2 (2021-07-21 11:41:01 AM)
Sim: >Model group>Old Run Group>2021-07-21-Wet Weather Profile (Observed Rain _Flow)!>Existing Scenario - RTC - Modified RTK Rainfall Event (Combined) (2021-07-29 5:20:15 PM)

Flow Survey Location (Obs.) SAMH1377.1, Model Location (Pred.) U/S SAMH1377.1, Rainfall Profile: 1



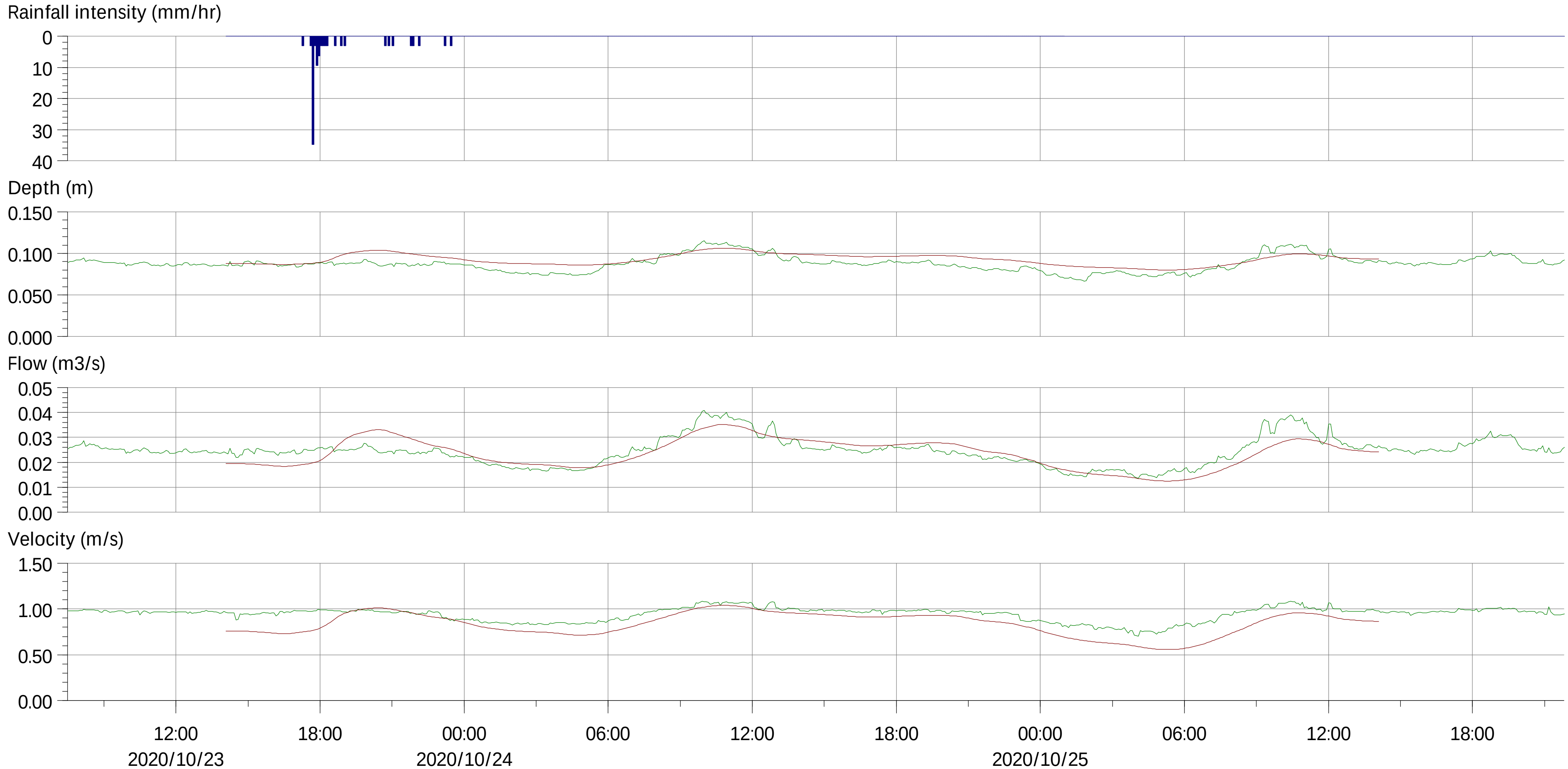
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	8.968	34.818	0.256							
Observed				0.045	0.080	0.003	0.021	2693.134	0.471	1.725
...t (Combined)				0.036	0.082	0.002	0.019	1802.617	0.510	1.348

Observed / Predicted Report - St. Thomas PPCP (2020 Flowmeters) - THOMAS 1

Flow survey: >Model group>Flow Survey Group - 2>Flow Survey Group -2 (2021-07-21 11:41:01 AM)

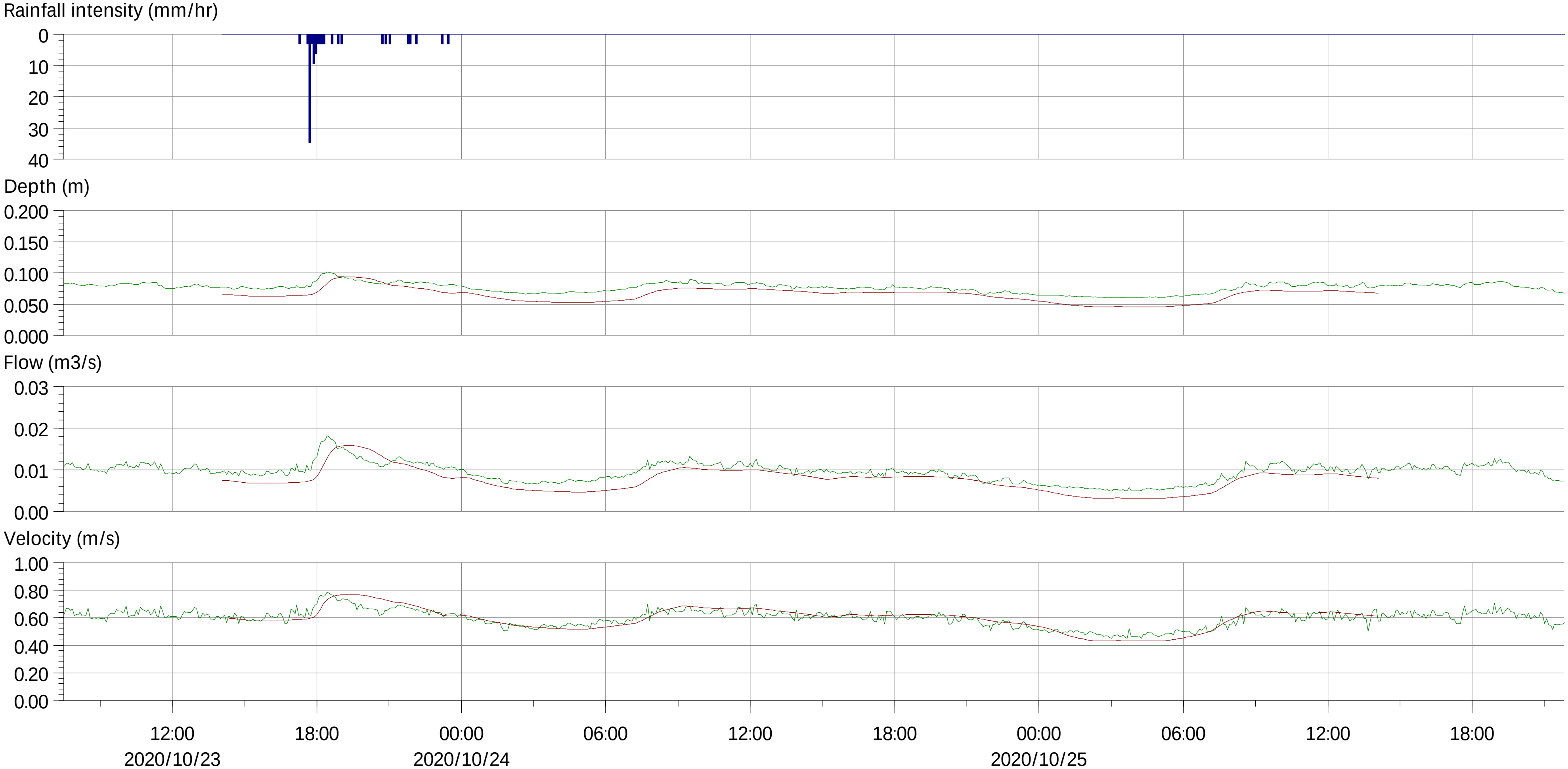
Sim: >Model group>Old Run Group>2021-07-21-Wet Weather Profile (Observed Rain _Flow)!>Existing Scenario - RTC - Modified RTK Rainfall Event (Combined) (2021-07-29 5:20:15 PM)

Flow Survey Location (Obs.) SAMH1987.2, Model Location (Pred.) U/S SAMH1987.2, Rainfall Profile: 1



	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	8.968	34.818	0.256							
Observed				0.067	0.115	0.014	0.041	5505.874	0.706	1.085
...t (Combined)				0.080	0.106	0.012	0.035	4078.333	0.556	1.039

Flow survey: >Model group>Flow Survey Group - 2>Flow Survey Group -2 (2021-07-21 11:41:01 AM)
Sim: >Model group>Old Run Group>2021-07-21-Wet Weather Profile (Observed Rain _Flow)!>Existing Scenario - RTC - Modified RTK Rainfall Event (Combined) (2021-07-29 5:20:15 PM)
Flow Survey Location (Obs.) SAMH1507.1, Model Location (Pred.) U/S SAMH1507.1, Rainfall Profile: 1

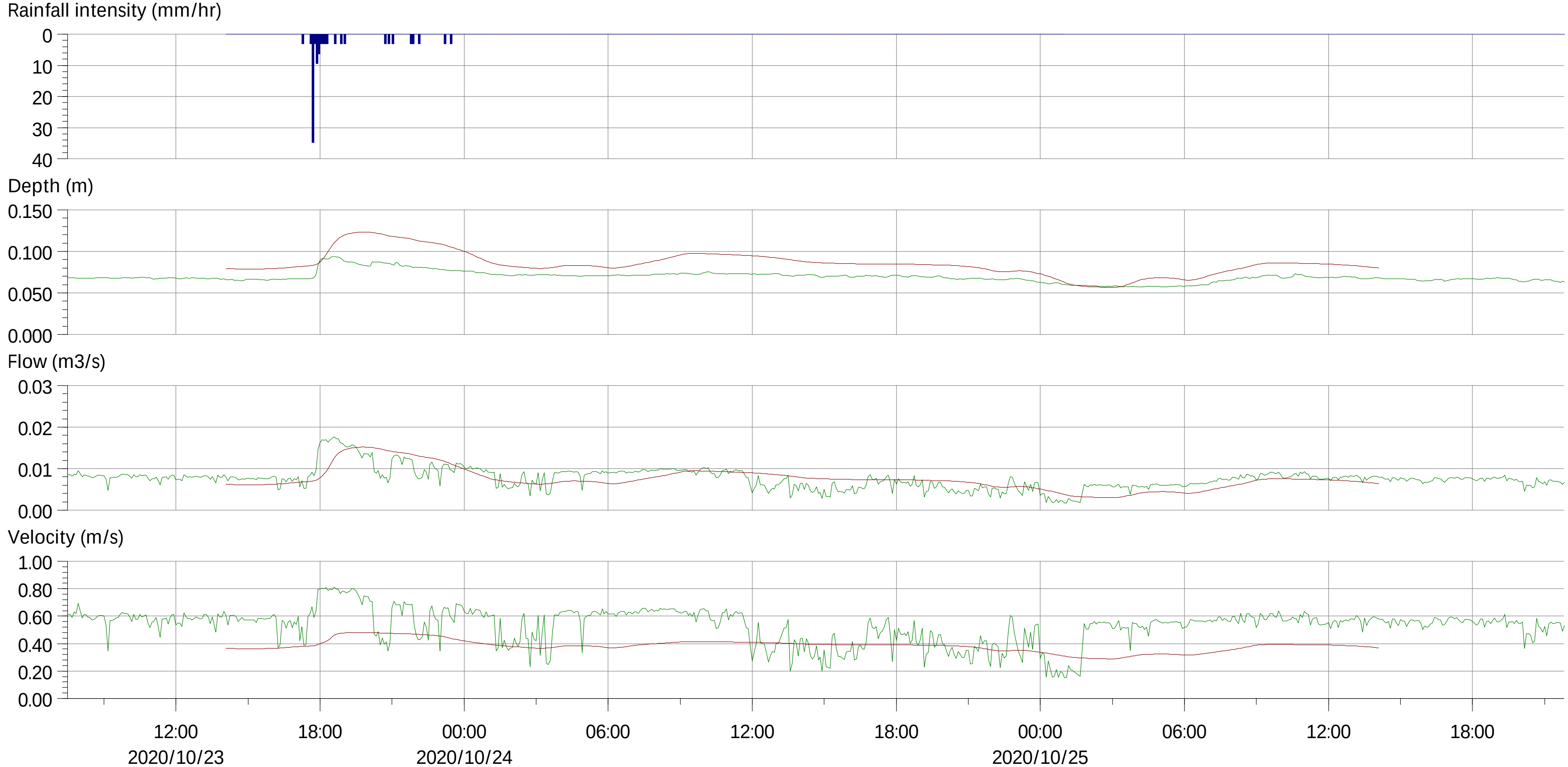


		Rainfall			Depth		Flow			Velocity	
		Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain		8.968	34.818	0.256							
Observed					0.060	0.101	0.005	0.018	2116.670	0.449	0.786
...t (Combined)					0.046	0.093	0.003	0.016	1311.512	0.430	0.769

Flow survey: >Model group>Flow Survey Group - 2>Flow Survey Group -2 (2021-07-21 11:41:01 AM)

Sim: >Model group>Old Run Group>2021-07-21-Wet Weather Profile (Observed Rain _Flow)!>Existing Scenario - RTC - Modified RTK Rainfall Event (Combined) (2021-07-29 5:20:15 PM)

Flow Survey Location (Obs.) SAMH560.1, Model Location (Pred.) U/S SAMH560.1, Rainfall Profile: 1



	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	8.968	34.818	0.256							
Observed				0.057	0.094	0.002	0.018	1703.040	0.150	0.814
...t (Combined)				0.057	0.123	0.003	0.015	1289.226	0.289	0.481

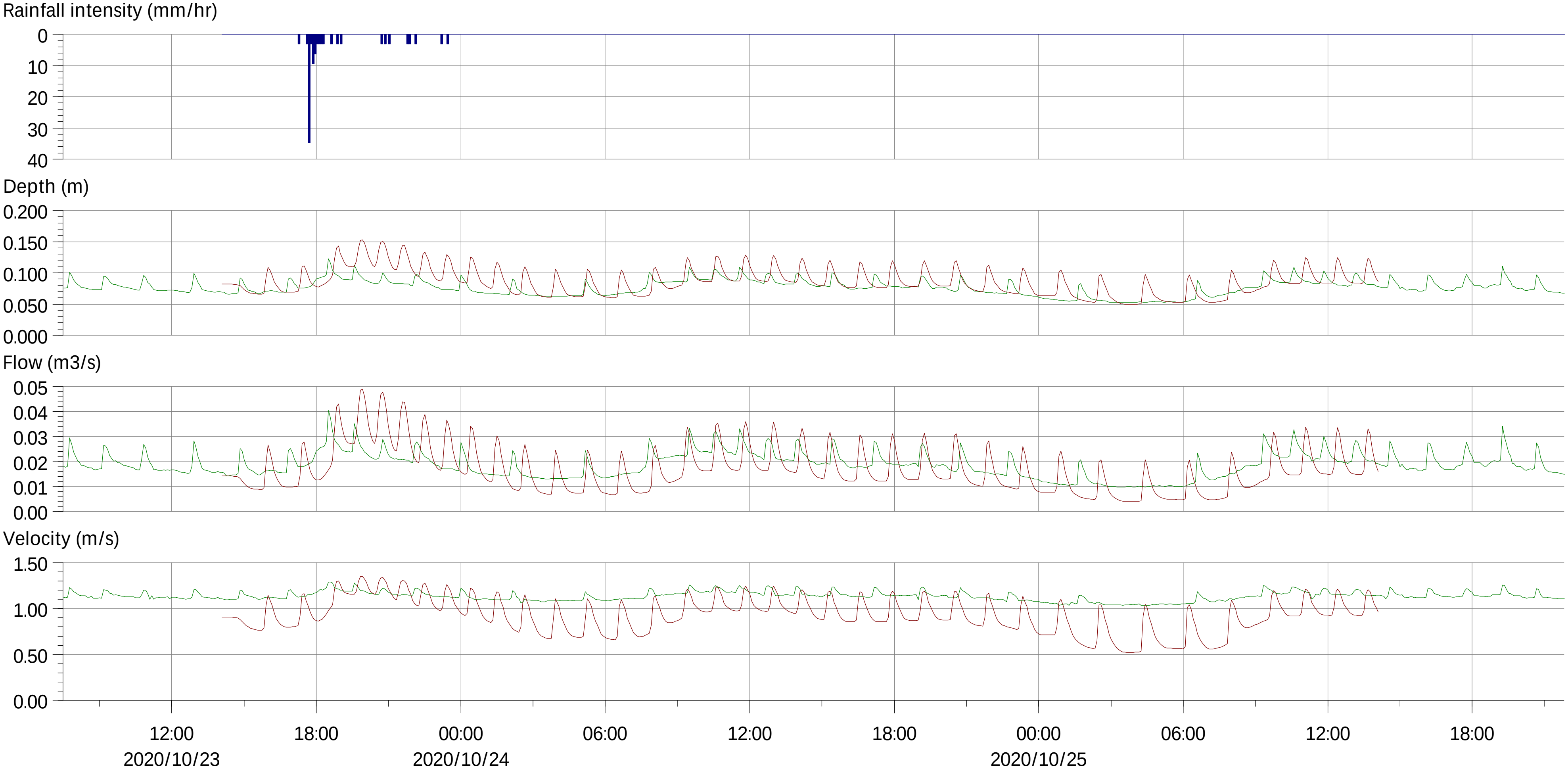
Observed / Predicted Report - St. Thomas PPCP (2020 Flowmeters) - THOMAS 4A

arva

Flow survey: >Model group>Flow Survey Group - 2>Flow Survey Group -2 (2021-07-21 11:41:01 AM)

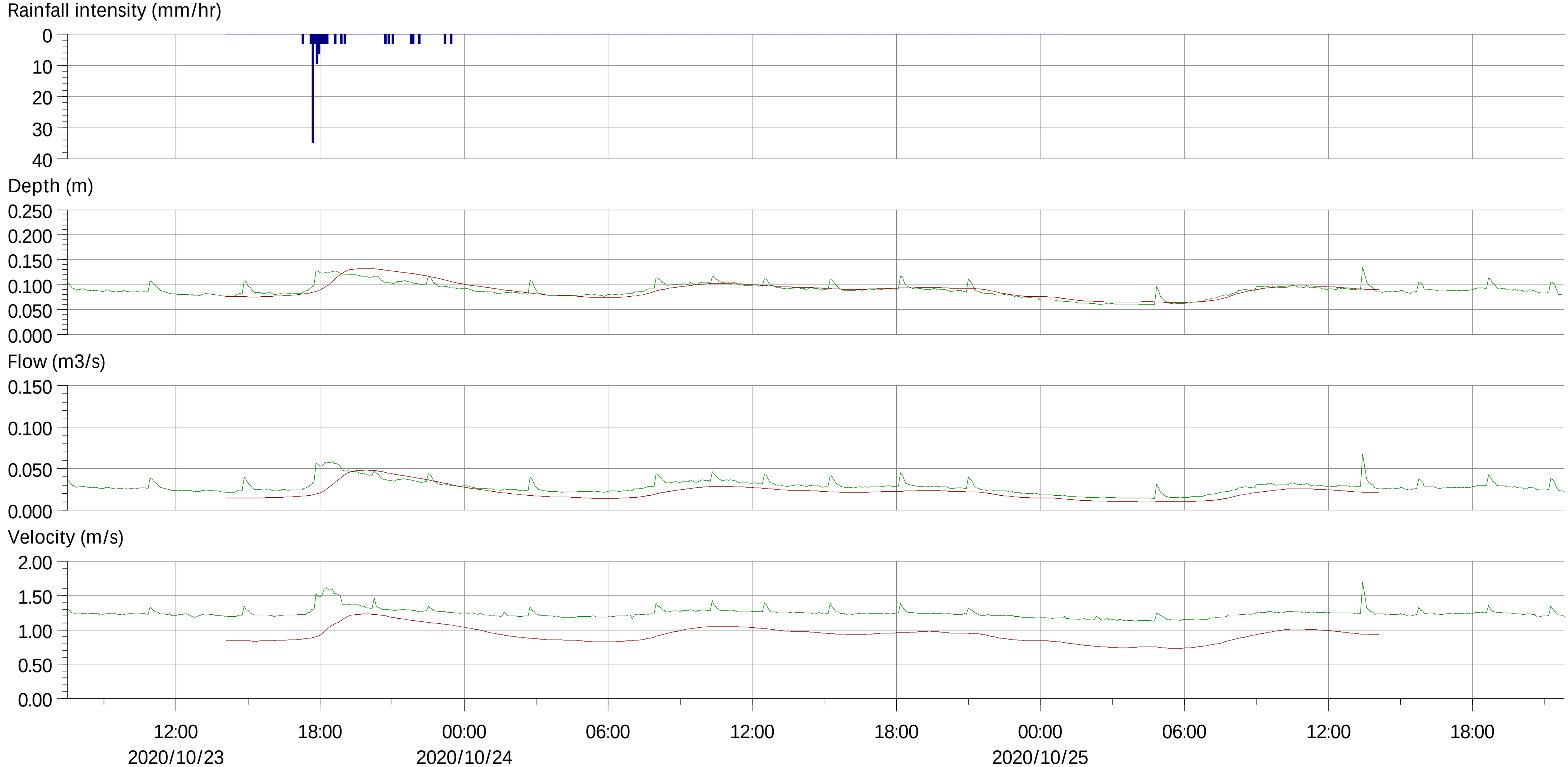
Sim: >Model group>Old Run Group>2021-07-21-Wet Weather Profile (Observed Rain _Flow)!>Existing Scenario - RTC - Modified RTK Rainfall Event (Combined) (2021-07-29 5:20:15 PM)

Flow Survey Location (Obs.) SAMH2616.1, Model Location (Pred.) U/S SAMH2616.1, Rainfall Profile: 1



	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	8.968	34.818	0.256							
Observed				0.052	0.122	0.010	0.040	4216.516	1.034	1.292
...t (Combined)				0.050	0.153	0.004	0.049	2905.719	0.520	1.352

Flow survey: >Model group>Flow Survey Group - 2>Flow Survey Group -2 (2021-07-21 11:41:01 AM)
Sim: >Model group>Old Run Group>2021-07-21-Wet Weather Profile (Observed Rain _Flow)!>Existing Scenario - RTC - Modified RTK Rainfall Event (Combined) (2021-07-29 5:20:15 PM)
Flow Survey Location (Obs.) SAMH144.1, Model Location (Pred.) U/S SAMH144.1, Rainfall Profile: 1



	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	8.968	34.818	0.256							
Observed				0.060	0.134	0.014	0.067	6256.312	1.129	1.692
...t (Combined)				0.064	0.132	0.010	0.048	3730.181	0.731	1.230

Observed / Predicted Report - St. Thomas PPCP (2020 Flowmeters) - THOMAS 5

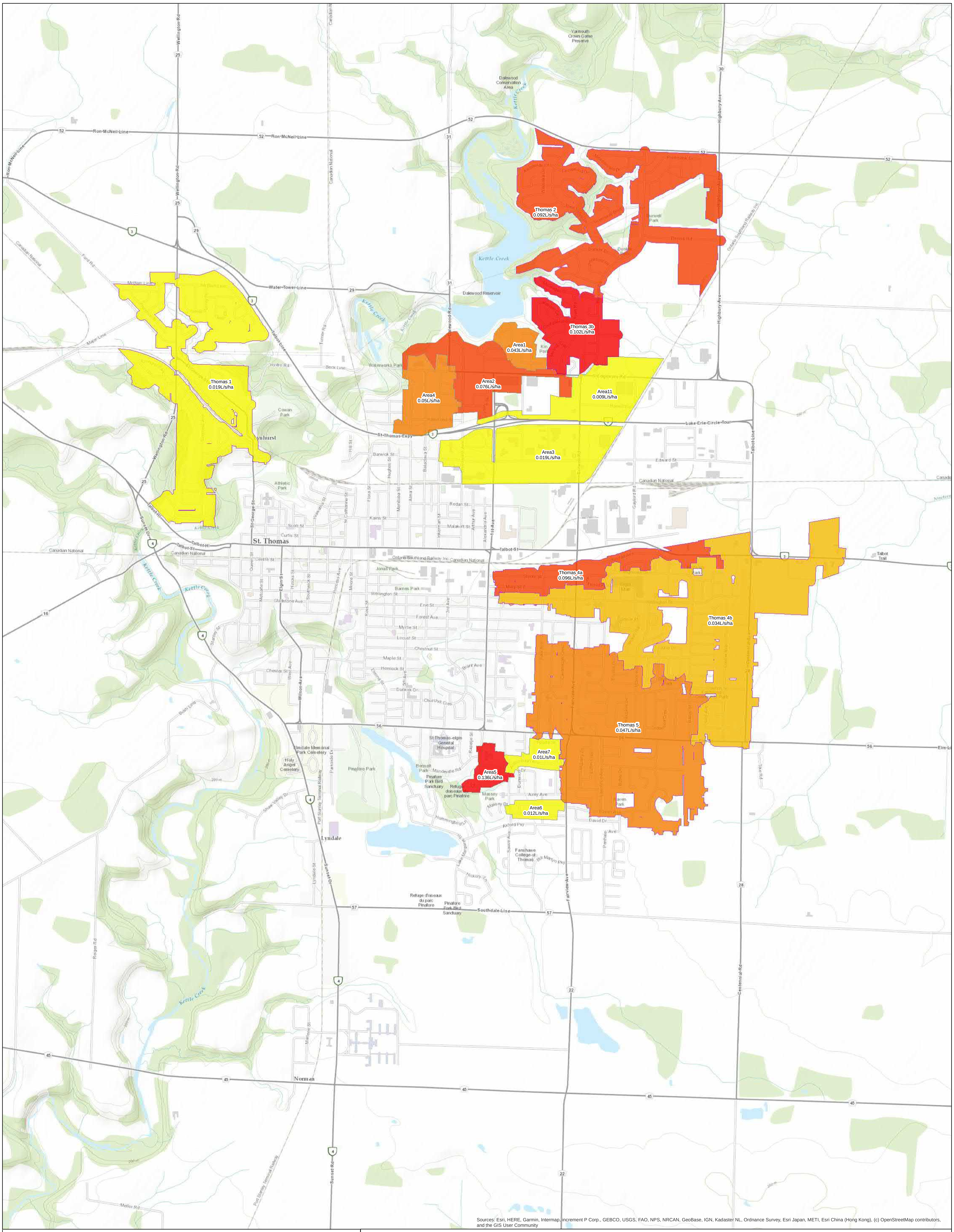
APPENDIX 7

Sewer Network Performance Maps for 2-year to 100-year Design Storm Events

This appendix is to be separately sent to the Client due to the file size

APPENDIX 8

Sewer Catchments with GWI and RDII Rates



Legend

GWI Rate (L/s/ha)

- 0.00 - 0.02
- 0.02 - 0.04
- 0.04 - 0.06
- 0.06 - 0.10
- 0.10 - 0.14
- Metered Catchments

ST. THOMAS POLLUTION PREVENTION CONTROL PLAN

CATCHMENTS GWI RATES

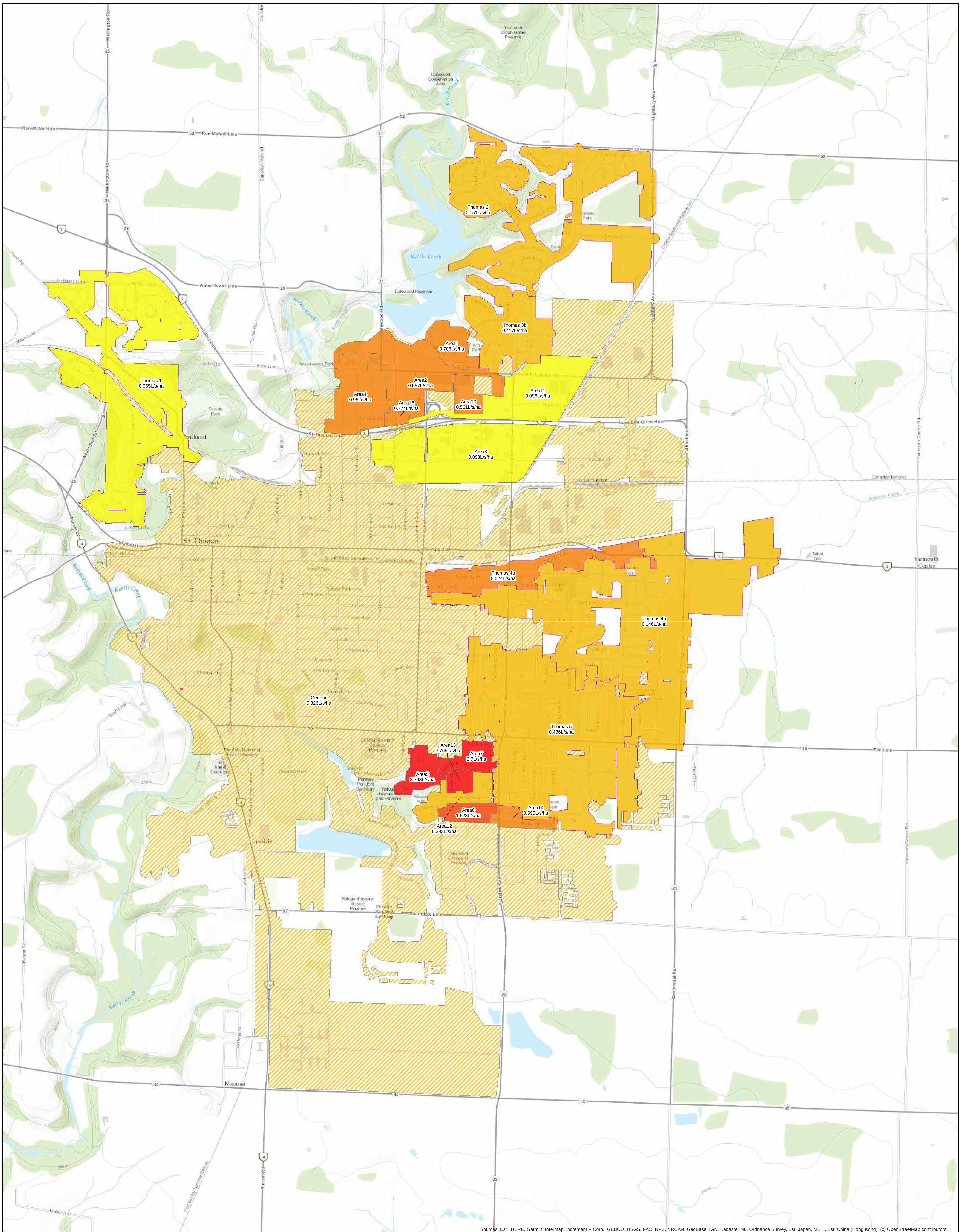
ST. THOMAS THE RAILWAY CITY

arva

0 315 630 1,260 1,890 2,520 Meters

RVA PROJECT NO: 205153

USER NAME: dpaez



Legend

RDII Rate (L/s/ha)

- 0.00 - 0.10
- 0.10 - 0.50
- 0.50 - 1.00
- 1.00 - 2.00
- 2.00 - 4.00
- Metered Catchments
- Generic Catchment

ST. THOMAS POLLUTION PREVENTION CONTROL PLAN

CATCHMENTS RDII RATES

ST. THOMAS THE RAILWAY CITY

rva

0 315 630 1,260 1,890 2,520 Meters

RVA PROJECT NO: 205153

USER NAME: dpaez

APPENDIX 9

PPCP Solution Cost Opinion

APPENDIX 9-1

Cost Opinion Basis

Cost Estimate Methodology

1.0 INTRODUCTION

Our cost estimation is based upon the methodology found in ASTM E 2516-11 (Standard Classification for Cost Estimate Classification System). ASTM E 2516-11 provides a five-level classification system based on several characteristics, with the primary characteristic being the level of project definition (i.e., percentage of design completion). Section 8.5.4 of ASTM E 2516 acknowledges that other “secondary” characteristics impact the accuracy of the estimate, and provides as follows:

> “In summary, estimate accuracy will generally be correlated with estimate classification (and therefore the degree of project definition), all else being equal. However, specific accuracy ranges will typically vary by industry. Also, the accuracy of any given estimate is not fixed or determined by its classification category. Significant variations in accuracy from estimate to estimate are possible if any of the determinants of accuracy, such as differing technological maturity, quality of reference cost data, quality of the estimating process, and skill and knowledge of the estimator vary. Accuracy is also not necessarily determined by the methodology used or the effort expended. Estimate accuracy must be evaluated on an estimate-by-estimate basis, usually in conjunction with some form of risk analysis process.”

2.0 STANDARD RANGE OF COST ESTIMATES

The ASTM standard, shown in Table 1, illustrates the typical accuracy ranges that may be associated with the general building industries.

Table 1 – ASTME 2516-11 Accuracy Range of Cost Opinions for General Building Industries

Cost Estimate Class	**Expressed as % of Design Completion**	**Anticipated Accuracy Range as % of Actual Cost**
5	0-2	-30 to -20/ +30 to +50
4	1-15	-20 to -10/ +20 to +30
3	10-40	-15 to -5/ +10 to +20
2	30-70	-10 to -5/ +5 to +15
1	50-100	-5 to -3/ +3 to +10

3.0 COST ESTIMATE RANGES

3.1 Introduction

Below is a general description of the various classes within a typical five-level cost opinion classification system. Always keep in mind that many factors influence cost opinion accuracy, and any cost opinion accuracy must be evaluated on a case-by-case basis.

3.2 Class 5

This is an order of magnitude cost opinion, also referred to as a parameter or conceptual cost opinion. It is generally used for strategic business or capital planning, assessment of viability, or for comparative purposes to establish a base ranking of alternatives. There is usually a very low level of project definition and limited information available. The cost opinion accuracy can be up to +100%. A Class 5 cost opinion is based upon historical sources, other analogous work, and the experience of the individual. Some percentage breakdown by major work category may be inferred from a review of similar projects that have been completed or estimated in detail. Its basis can be "cost per square meter", "cost per unit" or multiplier of primary equipment cost. Sometimes expression as a range of values is better received and understood than a single number with a stated accuracy of ±50% ($50,000 to $150,000 rather than $100,000 ±50%). This cost opinion is usually not detailed, except perhaps for subtotals of major components and with qualifications as to accuracy. As with all levels, the accuracy must be kept in mind when rounding off the significant figures. For example a $100,000 Class 5 cost opinion would be rounded up to the nearest $10,000 and never the nearest $100 or $1,000.

3.3 Class 4

This is generally referred to as a preliminary, feasibility, schematic design, predesign, authorization or basic system cost opinion. It is used for detailed planning, evaluation of alternatives, confirm economic viability, preliminary budget approval and cash flow projections. At this stage the project concept and scope have been established and enough work completed to define capacities and processes resulting in block schematics, plot plans, process flow diagrams, general arrangement drawings and infrastructure requirements. The cost opinion is based on elemental units using historical costs, standard estimating references, supplier quotes and historical data from similar projects.

3.4 Class 3

This is a target, budget, or control cost opinion, also referred to as a design development cost opinion. It is used for budget authorization and set the design control budget to confirm and monitor design direction. This is the point at which the project begins to have firm definition, and we have begun detailed work. This cost opinion is usually prepared when our work is from 10% to 40% complete. It is based on unit takeoffs from general arrangements, definitive discipline layouts, P & ID's, single lines, block diagrams, preliminary equipment selection, etc. Unit pricing is obtained from supplier quotes, pricing inquiries, historical data from similar work, pricing data books, all viewed toward industry pricing trends and factors.

3.5 Class 2

A Class 2 cost opinion is known as a definitive, detailed or master control, tender/bid or pre-tender/pre-Bid Cost opinion and is based on 90% completion of construction documents. It is prepared using detailed material take-offs and is really a "shadow" cost opinion of what is expected to be bid by the contractors. It is used to:

- Prepare the bid form;
- Anticipate bid prices and update project cost opinion;
- Check pricing during evaluations; and
- Prepare the format for construction progress payments, cost tracking, and change/variation control.

3.6 Class 1

A Class 1 cost opinion is known as a detailed, final execution phase, definitive, current control, or change order cost opinion. It is prepared from fully completed design documentation employing a high level of takeoff breakdown. These may be used for contractor bid negotiations, subcontractors for bid preparation, as the final control base for bid checking, change/variation control, and claim or dispute resolution. These require a significant level of effort and are not typically prepared for all projects. They may only be prepared for critical or selected parts of the project for specific reasons. All levels of cost opinions must be expressed in appropriate significant figures. For example, even a Class 1 cost opinion would be rounded up to at least the next $1,000, or higher depending on project size. A "round off" budget item line can be inserted just above the project total.

APPENDIX 9-2

Detailed Costing

St. Thomas – Pollution Prevention Control Plan
Opinion of Capital Costs **Date:** 14-Jan-22

Component	Cost Estimate Per Activity			Timeframe/ Comment
	Capital	Planning and Engineering	Total	
Recommended Collection System Upgrades				
Sunset SPS Improvements to coordinate with PPCP	$0	$25,000	$25,000	Assume that this may be only a design change in the new PS and not impact the construction cost.
Additional cost to reroute the new Sunset SPS forcemain to the CSO	$100,000	$20,000	$120,000	Undertake following upgrades to WPCP to remove bottlenecks when now Sunset SPS is being built.
Woodworth Ave SPS Upgrades	$2,500,000	$500,000	$3,000,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Woodworth Ave SPS Collection System	$3,849,283	$577,392	$4,426,675	When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Upgrades	$2,000,000	$400,000	$2,400,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Downstream Collection System	$1,500,000	$225,000	$1,725,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Annual Sewer Lining (500 m/year)	$650,000	$65,000	$715,000	It will take 62 years to undertake the lining of the current total of 31 km of fair to poor sanitary sewers in the system.
CSO Operation Optimization				
Improvements to CSO Tank Operation	$0	$100,000	$100,000	2023 - undertake following upgrades to WPCP to remove bottlenecks. Assum that this involves changes in controls only. Does not include costs for removing bottlenecks in WPCP.
Removal of Capacity Constraints at the WPCP				
Remove WPCP Bottlenecks	$2,727,000	$273,000	$3,000,000	2022-23 -Modfy plant flow distribution, remove pipe bottlenecks, twin UV channel and add a new paralel unit, upgrade outfall pipe.
Long Term I & I Mitigation Measures				
Permanent Rain Gauge Installation	$15,000	$4,000	$19,000	Early 2022 installation.
Annual Camera Work in Collection System	$250,000	$0	$250,000	Yearly work ($50,000) over a 5 year period.
Flow Monitor Installation, Maintenance, Removal	$176,000	$0	$176,000	Yearly work ($35,200) over a 5 year period.
Building on the Current Hydraulic Model	$0	$79,000	$79,000	Yearly work (15,800) over a 5 year period.

St. Thomas – Pollution Prevention Control Plan
Opinion of Capital Costs **Date:** **Date:** 14-Jan-22

Component	Cashflow (Years)				Timeframe/ Comment
	1 to 5	6-10	11 to 20	21-40	
Recommended Collection System Upgrades					
Walnut (Sunset) SPS Improvements to coordinate with PPCP	$25,000				Assume that this may be only a design change in the new PS and not impact the construction cost.
Additional cost to reroute the new Walnut SPS forcemain to the CSO	$20,000	$120,000			Undertake following upgrades to WPCP to remove bottlenecks when now Walnut St SPS is being built.
Woodworth Ave SPS Upgrades	$4,800,000				When City deems necessary to do/ High level estimate/ City may look at other options.
Woodworth Ave SPS Collection System	$4,426,675				When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Upgrades		$2,400,000			When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Downstream Collection System		$1,725,000			When City deems necessary to do/ High level estimate/ City may look at other options.
Annual Sewer Lining (500 m/year)	$3,575,000	$3,575,000	$7,150,000	$14,300,000	Start sewer lining in year 3 after 2 years of additional modeling and data city will take 41 years to undertake the lining of the current total of 31 km of fair to poor sanitary sewers in the system.
CSO Operation Optimization					
Improvements to CSO Tank Operation	$100,000				2023 - undertake following upgrades to WPCP to remove bottlenecks. Assume that this involves changes in controls only. Does not include costs for removing bottlenecks in WPCP.
Removal of Capacity Constraints at the WPCP					
Remove WPCP Bottlenecks	$3,000,000				2022-23 -Modify plant flow distribution, remove pipe bottlenecks, twin UV channel and add a new parallel unit, upgrade outfall pipe.
Long Term I & I Mitigation Measures					
Permanent Rain Gauge Installation	$19,000				Early 2022 installation.
Annual Camera Work in Collection System	$250,000	$250,000			Yearly work ($50,000) over a 5 year period. Stop at year 10 when a new MP should be undertaken.
Flow Monitor Installation, Maintenance, Removal	$176,000	$176,000			Yearly work ($35,200) over a 5 year period. Stop at year 10 when a new MP should be undertaken.
Building on the Current Hydraulic Model	$79,000	$79,000			Yearly work ($15,800) over a 5 year period. Stop at year 10 when a new MP should be undertaken.
Total	$16,470,675	$8,325,000	$7,150,000	$14,300,000	
Low (-30%)	$11,529,473	$5,827,500	$5,005,000	$10,010,000	
High(+50%)	$24,706,013	$12,487,500	$10,725,000	$21,450,000	

Total Estimated Costs	**To Year 5**	**To Year 10**	**To Year 20**	**To Year 40**
Estimated Cost	$16,470,675	$24,795,675	$31,945,675	$46,245,675
Low (-30%)	$11,529,473	$17,356,973	$22,361,973	$32,371,973
High(+50%)	$24,706,013	$37,193,513	$47,918,513	$69,368,513

St. Thomas – Pollution Prevention Control Plan
Woodworth PA Sanitary Sewer Servicing Cost Opinion

Date: 14-Jan-22

Method	From	To	Length	Depth	Pipe Dia.	Pipe Cost	Install.	Rest.	Total/m	2014 $ TOTAL	2021 $ TOTAL[1]
New FM	PS	SANMH 1330	957	1.8	600	$360	$1,000	$840	$2,200	$2,105,400	$2,568,588
New FM	N/S RR Xing	S/S RR Xing	50	5	600	$360	$6,295	$0	$6,655	$332,750	$405,955
New SAN	SANMH 1330	Talbot Street	250	3	750	$280	$730	$1,858	$2,868	$717,000	$874,740
New SAN	Talbot Street	SANMH 2091	523	3.5	1050	$535	$794	$1,946	$3,275	$1,712,825	$2,089,647
									TOTAL	**$3,155,150**	**$3,849,283**

1 - Assume total inflation factor of 1.22

TABLE 3-3
CITY OF LONDON
2014 WASTEWATER SERVICING MASTER PLAN UPDATE AND DEVELOPMENT CHARGE BACKGROUND STUDY
UNIT COST TABLE FOR PIPES, CONSTRUCTION AND RESTORATION WORK
(Revised June 3, 2014)

PIPE COSTS
Based on 2014 Concast circular concrete pipe price list, includes pipe and gaskets.
250 mm pipe cost was extrapolated based on other 2014 pipe prices
Diameter

Depth	250	300	375	450	525	600	675	750	825	900	975	1050	1200	1350	1500	1650	1800	1950	2100
2.5	65	75	90	95	105	140	215	280	325	390	430	495	620	755	925	1,110	1,340	1,555	1,780
5.0	65	75	90	95	105	140	215	280	325	390	515	595	745	910	970	1,165	1,410	1,635	1,870
7.5	65	75	90	95	120	160	245	320	415	450	515	595	745	910	1,110	1,330	1,610	1,865	2,140
10.0	65	75	90	95	120	160	245	320	415	450	605	690	865	1,060	1,295	1,550	1,875	2,175	2,495
12.5	65	75	90	120	120	190	245	320	415	450	605	690	865	1,060	1,295	1,550	1,875	2,175	2,495

CONSTRUCTION COSTS - Open Cut - Pipe Cost NOT Included
Based on tender costs as provided by the City over the past 5 years and indexed to 2014
Includes trenching labour and equipment, bedding, backfill, compaction, dewatering, and maintenance holes.
Diameter

Depth	250	300	375	450	525	600	675	750	825	900	975	1050	1200	1350	1500	1650	1800	1950	2100
2.5	405	420	430	455	480	520	535	575	610	640	675	700	735	770	800	835	860	905	935
5.0	605	620	635	670	725	770	805	850	930	935	935	935	970	1,005	1,175	1,230	1,300	1,360	1,460
7.5	710	725	735	795	825	910	945	1,000	1,000	1,080	1,125	1,160	1,255	1,355	1,460	1,555	1,665	1,785	1,890
10.0	1,010	1,075	1,135	1,295	1,435	1,585	1,720	1,875	1,940	2,055	2,085	2,110	2,215	2,315	2,420	2,545	2,675	2,785	2,880
12.5	2,090	2,100	2,110	2,150	2,205	2,225	2,285	2,345	2,340	2,360	2,380	2,415	2,445	2,485	2,555	2,675	2,875	3,085	3,320

CONSTRUCTION COSTS - Tunneling - Pipe Cost NOT Included
Based on 20-year (Stantec) and 50-year (Stantec) study values, indexed to 2014 using tender costs as provided by the City
Includes tunnel shafts @ 150 m spacing for maintenance holes, liners, dewatering, and restoration.
Diameter

Depth	250	300	375	450	525	600	675	750	825	900	975	1050	1200	1350	1500	1650	1800	1950	2100
5.0	3,990	4,755	5,025	5,490	5,895	6,295	6,630	6,900	7,170	7,435	7,705	7,970	8,240	8,505	8,770	9,040	9,305	9,580	9,850
10.0	4,035	4,890	5,160	5,625	5,825	6,495	6,770	7,100	7,435	7,770	8,035	8,305	8,570	8,840	9,105	9,380	9,645	9,915	10,180
15.0	4,180	5,025	5,360	5,760	6,095	6,700	7,035	7,370	7,705	8,105	8,370	8,640	8,905	9,175	9,445	9,715	9,980	10,250	10,515
20.0	4,320	5,160	5,490	5,895	6,360	6,900	7,300	7,705	8,035	8,440	8,705	9,040	9,240	9,515	9,780	10,050	10,315	10,585	10,850
25.0	4,465	5,290	5,625	6,095	6,630	7,100	7,570	7,970	8,370	8,770	9,040	9,305	9,580	9,850	10,115	10,380	10,650	10,915	11,185
30.0	4,570	5,425	5,760	6,295	6,835	7,370	7,835	8,240	8,640	9,040	9,445	9,715	9,980	10,250	10,515	10,785	11,050	11,320	11,585

RESTORATION COSTS
Taken from 20-year (LSSSS) plan (2003), and updated as per 2014 tender and transportation costs for rural and urban restoration.
Open - no restoration; Landscape- minor/boulevard (no roadway restoration); Rural - cross section as per transportation cost table; Urban - cross section as per transportation cost table; Ecosystem - applies to areas adjacent to or within environmentally significant areas

Condition / Depth	Open	Landscape	Rural	Urban	Ecosystem
2.5	0	400	1,600	1,770	840
5.0	0	510	2,040	2,210	1,070
7.5	0	600	2,450	2,610	1,270
10.0	0	710	2,920	3,080	1,480
12.5	0	810	3,400	3,540	1,680

Table 2-7
CITY OF LONDON
WATER SERVICING DC UPDATE 2014
UNIT RATE COSTS FOR WATERMAINS
(Revised June 3, 2014)

PIPE COSTS
400mm to 600mm dia. pipe cost based on 2012 vendor pricing of PVC pipe and including pipe and gaskets. Inflated to 2014 dollars at 5% per year.
Pipe equal to and greater than 750mm dia. based on 2012 vendor pricing of concrete pressure pipe and includes pipe and gaskets. Inflated to 2014 dollars at 5% per year.
Diameter

Type	250	300	400	450	500	600	750	900	1050	1200	1350	1500
CPP	230	280	320	360	440	480	610	740	930	1,120	1,440	1,760
PVC	80	100	155	205	255	360	N/A	N/A	N/A	N/A	N/A	N/A

CONSTRUCTION COSTS - Open Cut - Pipe Cost NOT Included
Based on tender costs as provided by the City over the past 5 years and indexed to 2014.
Includes trenching labor and equipment, installation, bedding, backfill, compaction, chambers, valves, hydrants, etc.
Diameter

Depth	250	300	400	450	500	600	750	900	1050	1200	1350	1500
1.8	690	720	750	780	870	1,000	1,240	1,390	1,690	2,020	2,350	2,750

CONSTRUCTION COSTS - Tunneling - Pipe Cost NOT Included
Based on 20-year (Stantec) and 50-year (Stantec) study values, indexed to 2014 using tender costs as provided by the City.
Diameter

Depth	400	450	500	600	750	900	1050	1200	1350	1500
5.0	5,260	5,580	5,860	6,355	6,955	7,450	7,865	8,225	8,540	8,825
10.0	5,380	5,715	6,015	6,535	7,175	7,695	8,135	8,515	8,850	9,155
15.0	5,575	5,925	6,235	6,775	7,435	7,975	8,435	8,830	9,180	9,490
20.0	5,770	6,130	6,455	7,015	7,705	8,265	8,740	9,150	9,515	9,840
25.0	5,960	6,340	6,675	7,255	7,965	8,545	9,040	9,465	9,840	10,175
30.0	6,145	6,540	6,890	7,500	8,250	8,860	9,375	9,820	10,215	10,570

RESTORATION COSTS
Taken from 20-year (LSSSS) plan (2003), and updated as per 2014 tender and transportation costs for rural and urban restoration.
Open - no restoration; Landscape- minor/boulevard (no roadway restoration); Rural - cross section as per transportation cost table; Urban - cross section as per

Condition / Depth	Open	Landscape	Rural	Urban	Ecosystem
1.8	0	300	1,230	1,310	660

St. Thomas – Pollution Prevention Control Plan
Recommended Ongoing Model Development

Date: 14-Jan-22

	Flow Monitor Installation, Maintenance, Removal	Planning and Engineering	Total
Permanent Rain Gauge Installation	$6,000	$4,000	$10,000
Year 1 Flow Monitoring (3 Flow Monitors)	$30,000	$15,000	$45,000
Year 2 Flow Monitoring (4 Flow Monitors)	$40,000	$15,000	$55,000
Year 3 Flow Monitoring (3 Flow Monitors)	$30,000	$15,000	$45,000
Year 4 Flow Monitoring (4 Flow Monitors)	$40,000	$15,000	$55,000
Year 5 Flow Monitoring (3 Flow Monitors)	$30,000	$15,000	$45,000
TOTAL	**$176,000**	**$79,000**	**$255,000**

APPENDIX 7

Cost Opinion for PPCP

St. Thomas – Pollution Prevention Control Plan
Opinion of Capital Costs

Date: 14-Jan-22

Component	Cost Estimate Per Activity			Timeframe/ Comment
	Capital	Planning and Engineering	Total	
Recommended Collection System Upgrades				
Sunset SPS Improvements to coordinate with PPCP	$0	$25,000	$25,000	Assume that this may be only a design change in the new PS and not impact the construction cost.
Additional cost to reroute the new Sunset SPS forcemain to the CSO	$100,000	$20,000	$120,000	Undertake following upgrades to WPCP to remove bottlenecks when now Sunset SPS is being built.
Woodworth Ave SPS Upgrades	$2,500,000	$500,000	$3,000,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Woodworth Ave SPS Collection System	$3,849,283	$577,392	$4,426,675	When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Upgrades	$2,000,000	$400,000	$2,400,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Downstream Collection System	$1,500,000	$225,000	$1,725,000	When City deems necessary to do/ High level estimate/ City may look at other options.
Annual Sewer Lining (500 m/year)	$650,000	$65,000	$715,000	It will take 62 years to undertake the lining of the current total of 31 km of fair to poor sanitary sewers in the system.
CSO Operation Optimization				
Improvements to CSO Tank Operation	$0	$100,000	$100,000	2023 - undertake following upgrades to WPCP to remove bottlenecks. Assum that this involves changes in controls only. Does not include costs for removing bottlenecks in WPCP.
Removal of Capacity Constraints at the WPCP				
Remove WPCP Bottlenecks	$2,727,000	$273,000	$3,000,000	2022-23 -Modfy plant flow distribution, remove pipe bottlenecks, twin UV channel and add a new paralel unit, upgrade outfall pipe.
Long Term I & I Mitigation Measures				
Permanent Rain Gauge Installation	$15,000	$4,000	$19,000	Early 2022 installation.
Annual Camera Work in Collection System	$250,000	$0	$250,000	Yearly work ($50,000) over a 5 year period.
Flow Monitor Installation, Maintenance, Removal	$176,000	$0	$176,000	Yearly work ($35,200) over a 5 year period.
Building on the Current Hydraulic Model	$0	$79,000	$79,000	Yearly work (15,800) over a 5 year period.

St. Thomas – Pollution Prevention Control Plan
Opinion of Capital Costs **Date:** 14-Jan-22

Component	Cashflow (Years)			Timeframe/ Comment
	1 to 5	6-10	11 to 20	
Recommended Collection System Upgrades				
Sunset SPS Improvements to coordinate with PPCP	$25,000			Assume that this may be only a design change in the new PS and not impact the construction cost.
Additional cost to reroute the new Walnut SPS forcemain to the CSO	$20,000	$120,000		Undertake following upgrades to WPCP to remove bottlenecks when now Walnut St SPS is being built.
Woodworth Ave SPS Upgrades	$4,800,000			When City deems necessary to do/ High level estimate/ City may look at other options.
Woodworth Ave SPS Collection System	$4,426,675			When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Upgrades		$2,400,000		When City deems necessary to do/ High level estimate/ City may look at other options.
Burwell Rd SPS Downstream Collection System		$1,725,000		When City deems necessary to do/ High level estimate/ City may look at other options.
Annual Sewer Lining (500 m/year)	$3,575,000	$3,575,000	$7,150,000	Start sewer lining in year 3 after 2 years of additional modeling and data city will take 41 years to undertake the lining of the current total of 31 km of fair to poor sanitary sewers in the system.
CSO Operation Optimization				
Improvements to CSO Tank Operation	$100,000			2023 - undertake following upgrades to WPCP to remove bottlenecks. Assume that this involves changes in controls only. Does not include costs for removing bottlenecks in WPCP.
Removal of Capacity Constraints at the WPCP				
Remove WPCP Bottlenecks	$3,000,000			2022-23 -Modify plant flow distribution, remove pipe bottlenecks, twin UV channel and add a new parallel unit, upgrade outfall pipe.
Long Term I & I Mitigation Measures				
Permanent Rain Gauge Installation	$19,000			Early 2022 installation.
Annual Camera Work in Collection System	$250,000	$250,000		Yearly work ($50,000) over a 5 year period. Stop at year 10 when a new MP should be undertaken.
Flow Monitor Installation, Maintenance, Removal	$176,000	$176,000		Yearly work ($35,200) over a 5 year period. Stop at year 10 when a new MP should be undertaken.
Building on the Current Hydraulic Model	$79,000	$79,000		Yearly work ($15,800) over a 5 year period. Stop at year 10 when a new MP should be undertaken.
Total	$16,470,675	$8,325,000	$7,150,000	
Low (-30%)	$11,529,473	$5,827,500	$5,005,000	
High(+50%)	$24,706,013	$12,487,500	$10,725,000	

Total Estimated Costs	**To Year 5**	**To Year 10**	**To Year 20**
Estimated Cost	$16,470,675	$24,795,675	$31,945,675
Low (-30%)	$11,529,473	$17,356,973	$22,361,973
High(+50%)	$24,706,013	$37,193,513	$47,918,513

St. Thomas – Pollution Prevention Control Plan
Woodworth PA Sanitary Sewer Servicing Cost Opinion

Date: 14-Jan-22

Method	From	To	Length	Depth	Pipe Dia.	Pipe Cost	Install.	Rest.	Total/m	2014 $ TOTAL	2021 $ TOTAL[1]
New FM	PS	SANMH 1330	957	1.8	600	$360	$1,000	$840	$2,200	$2,105,400	$2,568,588
New FM	N/S RR Xing	S/S RR Xing	50	5	600	$360	$6,295	$0	$6,655	$332,750	$405,955
New SAN	SANMH 1330	Talbot Street	250	3	750	$280	$730	$1,858	$2,868	$717,000	$874,740
New SAN	Talbot Street	SANMH 2091	523	3.5	1050	$535	$794	$1,946	$3,275	$1,712,825	$2,089,647
									TOTAL	**$3,155,150**	**$3,849,283**

1 - Assume total inflation factor of 1.22

TABLE 3-3
CITY OF LONDON
2014 WASTEWATER SERVICING MASTER PLAN UPDATE AND DEVELOPMENT CHARGE BACKGROUND STUDY
UNIT COST TABLE FOR PIPES, CONSTRUCTION AND RESTORATION WORK
(Revised June 3, 2014)

PIPE COSTS
Based on 2014 Concast circular concrete pipe price list, includes pipe and gaskets.
250 mm pipe cost was extrapolated based on other 2014 pipe prices

Diameter

Depth	250	300	375	450	525	600	675	750	825	900	975	1050	1200	1350	1500	1650	1800	1950	2100
2.5	65	75	90	95	105	140	215	280	325	390	430	495	620	755	925	1,110	1,340	1,555	1,780
5.0	65	75	90	95	105	140	215	280	325	390	515	595	745	910	970	1,165	1,410	1,635	1,870
7.5	65	75	90	95	120	160	245	320	415	450	515	595	745	910	1,110	1,330	1,610	1,865	2,140
10.0	65	75	90	95	120	160	245	320	415	450	605	690	865	1,060	1,295	1,550	1,875	2,175	2,495
12.5	65	75	90	120	120	190	245	320	415	450	605	690	865	1,060	1,295	1,550	1,875	2,175	2,495

CONSTRUCTION COSTS - Open Cut - Pipe Cost NOT Included
Based on tender costs as provided by the City over the past 5 years and indexed to 2014
Includes trenching labour and equipment, bedding, backfill, compaction, dewatering, and maintenance holes.

Diameter

Depth	250	300	375	450	525	600	675	750	825	900	975	1050	1200	1350	1500	1650	1800	1950	2100
2.5	405	420	430	455	480	520	535	575	610	640	675	700	735	770	800	835	860	905	935
5.0	605	620	635	670	725	770	805	850	930	935	935	935	970	1,005	1,175	1,230	1,300	1,360	1,460
7.5	710	725	735	795	825	910	945	1,000	1,000	1,080	1,125	1,160	1,255	1,355	1,460	1,555	1,665	1,785	1,890
10.0	1,010	1,075	1,135	1,295	1,435	1,585	1,720	1,875	1,940	2,055	2,085	2,110	2,215	2,315	2,420	2,545	2,675	2,785	2,880
12.5	2,090	2,100	2,110	2,150	2,205	2,225	2,285	2,345	2,340	2,360	2,380	2,415	2,445	2,485	2,555	2,675	2,875	3,085	3,320

CONSTRUCTION COSTS - Tunneling - Pipe Cost NOT Included
Based on 20-year (Stantec) and 50-year (Stantec) study values, indexed to 2014 using tender costs as provided by the City
Includes tunnel shafts @ 150 m spacing for maintenance holes, liners, dewatering, and restoration.

Diameter

Depth	250	300	375	450	525	600	675	750	825	900	975	1050	1200	1350	1500	1650	1800	1950	2100
5.0	3,990	4,755	5,025	5,490	5,895	6,295	6,630	6,900	7,170	7,435	7,705	7,970	8,240	8,505	8,770	9,040	9,305	9,580	9,850
10.0	4,035	4,890	5,160	5,625	5,825	6,495	6,770	7,100	7,435	7,770	8,035	8,305	8,570	8,840	9,105	9,380	9,645	9,915	10,180
15.0	4,180	5,025	5,360	5,760	6,095	6,700	7,035	7,370	7,705	8,105	8,370	8,640	8,905	9,175	9,445	9,715	9,980	10,250	10,515
20.0	4,320	5,160	5,490	5,895	6,360	6,900	7,300	7,705	8,035	8,440	8,705	9,040	9,240	9,515	9,780	10,050	10,315	10,585	10,850
25.0	4,465	5,290	5,625	6,095	6,630	7,100	7,570	7,970	8,370	8,770	9,040	9,305	9,580	9,850	10,115	10,380	10,650	10,915	11,185
30.0	4,570	5,425	5,760	6,295	6,835	7,370	7,835	8,240	8,640	9,040	9,445	9,715	9,980	10,250	10,515	10,785	11,050	11,320	11,585

RESTORATION COSTS
Taken from 20-year (LSSSS) plan (2003), and updated as per 2014 tender and transportation costs for rural and urban restoration.
Open - no restoration; Landscape- minor/boulevard (no roadway restoration); Rural - cross section as per transportation cost table; Urban - cross section as per transportation cost table; Ecosystem - applies to areas adjacent to or within environmentally significant areas

Condition / Depth	Open	Landscape	Rural	Urban	Ecosystem
2.5	0	400	1,600	1,770	840
5.0	0	510	2,040	2,210	1,070
7.5	0	600	2,450	2,610	1,270
10.0	0	710	2,920	3,080	1,480
12.5	0	810	3,400	3,540	1,680

Table 2-7
CITY OF LONDON
WATER SERVICING DC UPDATE 2014
UNIT RATE COSTS FOR WATERMAINS
(Revised June 3, 2014)

PIPE COSTS
400mm to 600mm dia. pipe cost based on 2012 vendor pricing of PVC pipe and including pipe and gaskets. Inflated to 2014 dollars at 5% per year.
Pipe equal to and greater than 750mm dia. based on 2012 vendor pricing of concrete pressure pipe and includes pipe and gaskets. Inflated to 2014 dollars at 5% per year.

Diameter

Type	250	300	400	450	500	600	750	900	1050	1200	1350	1500
CPP	230	280	320	360	440	480	610	740	930	1,120	1,440	1,760
PVC	80	100	155	205	255	360	N/A	N/A	N/A	N/A	N/A	N/A

CONSTRUCTION COSTS - Open Cut - Pipe Cost NOT Included
Based on tender costs as provided by the City over the past 5 years and indexed to 2014.
Includes trenching labor and equipment, installation, bedding, backfill, compaction, chambers, valves, hydrants, etc.

Diameter

Depth	250	300	400	450	500	600	750	900	1050	1200	1350	1500
1.8	690	720	750	780	870	1,000	1,240	1,390	1,690	2,020	2,350	2,750

CONSTRUCTION COSTS - Tunneling - Pipe Cost NOT Included
Based on 20-year (Stantec) and 50-year (Stantec) study values, indexed to 2014 using tender costs as provided by the City.

Diameter

Depth	400	450	500	600	750	900	1050	1200	1350	1500
5.0	5,260	5,580	5,860	6,355	6,955	7,450	7,865	8,225	8,540	8,825
10.0	5,380	5,715	6,015	6,535	7,175	7,695	8,135	8,515	8,850	9,155
15.0	5,575	5,925	6,235	6,775	7,435	7,975	8,435	8,830	9,180	9,490
20.0	5,770	6,130	6,455	7,015	7,705	8,265	8,740	9,150	9,515	9,840
25.0	5,960	6,340	6,675	7,255	7,965	8,545	9,040	9,465	9,840	10,175
30.0	6,145	6,540	6,890	7,500	8,250	8,860	9,375	9,820	10,215	10,570

RESTORATION COSTS
Taken from 20-year (LSSSS) plan (2003), and updated as per 2014 tender and transportation costs for rural and urban restoration.
Open - no restoration; Landscape- minor/boulevard (no roadway restoration); Rural - cross section as per transportation cost table; Urban - cross section as per

Condition / Depth	Open	Landscape	Rural	Urban	Ecosystem
1.8	0	300	1,230	1,310	660

St. Thomas – Pollution Prevention Control Plan
Recommended Ongoing Model Development

Date: 14-Jan-22

	Flow Monitor Installation, Maintenance, Removal	Planning and Engineering	Total
Permanent Rain Gauge Installation	$6,000	$4,000	$10,000
Year 1 Flow Monitoring (3 Flow Monitors)	$30,000	$15,000	$45,000
Year 2 Flow Monitoring (4 Flow Monitors)	$40,000	$15,000	$55,000
Year 3 Flow Monitoring (3 Flow Monitors)	$30,000	$15,000	$45,000
Year 4 Flow Monitoring (4 Flow Monitors)	$40,000	$15,000	$55,000
Year 5 Flow Monitoring (3 Flow Monitors)	$30,000	$15,000	$45,000
TOTAL	**$176,000**	**$79,000**	**$255,000**