Water

City of Lloydminster

Sanitary Sewer Master Plan

Prepared by:

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Project Number:

60342706 (402.29)

Date:

March, 2016

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March 15, 2016

Craig Anderson
Project Lead, Utilities Branch
Operations Centre
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Dear Craig:

Project No: 60342706 (402.39)

Regarding: Sanitary Sewer Master Plan

Please find enclosed the final report for the above noted project and study. The report has been updated following our discussion in the review meeting on January 20, 2016.

We truly enjoyed completing this assignment for the City. We look forward to further collaboration with the City for many years to come.

Please call the undersigned if you have any questions or concerns about this report.

Sincerely,
AECOM Canada Ltd.

Ryan King
Municipal Infrastructure Manager
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Revision Log

Revision #	Revised By	Date	Issue / Revision Description
A	Ryan King	November 20, 2015	Issued as Draft for Client Review
0	Ryan King	March 15, 2016	Issued as Final

AECOM Signatures

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Executive Summary

The City of Lloydminster has experienced significant population growth since its previous Sanitary Sewer Master Plan study was completed in 2009 and recognized the need for an updated Master Plan. The present Sanitary Sewer Master Plan study is founded on the development and calibration of a dynamic computer model of the city-wide sanitary sewer system.

A second main component of the Sanitary Sewer Master Plan study was the establishment of Level of Service criteria that are practical, achievable, and that identify realistic upgrade and future servicing strategies. As presented in this report and through considerable discussion between AECOM and the City, the Level of Service criteria were established to have sufficient capacity in the sanitary sewer system to mitigate basement flooding in a 25 year return period storm.

The 25 year return period storm used in the study was a synthetic design storm, scaled up from the August 9, 2008 historical storm. An ideal design rainfall event is a real historical rainfall event, coupled with flow monitoring data; however, as the City is presently in the initial stages of a flow monitoring and modelling program, this data has not yet been collected. Therefore, the modelling analysis was completed with the best data presently available. As subsequent years of flow data are collected and the model calibration further refined, greater confidence will be gained in understanding the wet weather performance of the sanitary sewer system and the Level of Service criteria.

The sanitary sewer model proved to be a valuable tool in assessing the performance of the existing sanitary sewer system, identifying areas of concern, and developing upgrade concepts to alleviate these areas of concern. It was also used to determine the off-site servicing needs for future development areas and upgrade requirements for the existing sanitary sewer system in order to intercept the additional future flows.

Off-site servicing requirements and existing sanitary sewer system upgrades were developed for the Present-Day (2015), Three Year (2018), Five Year (2020), Ten Year (2025), Twenty Year (2035), and Forty Year (2055) growth horizons. AECOM developed construction capital cost estimates for each of the timelines and provided recommendations for the preferred solutions.

- Present-Day – Upgrades to the existing sanitary sewer system were identified in two locations. In the area around 36 Street and 47 Avenue, a new sanitary sewer to re-route flow to the new Larsen Grove neighbourhood was recommended. Secondly, a new sanitary sewer along 56 Avenue from 44 Street to 45 Street was identified.
- Three year growth horizon – Two new trunk sewers are proposed (19 Street Trunk and South Trunk) to re-route a portion of the flow from the Southeast Trunk.
- Five year growth horizon – A portion of the Southeast Trunk is proposed to be twinned to alleviate surcharge.
- Ten year growth horizon – Off-site sanitary sewer servicing is provided by an extension of the South Trunk and introduction of a new trunk (CN Rail Trunk). For the existing system upgrades, it is proposed to twin the East Trunk.
- Twenty year growth horizon – The off-site sanitary sewer servicing requirements are met by extending the South Trunk and CN Rail Trunk, and introducing a new trunk (Highway 16 Trunk).
- Forty year growth horizon – Off-site sanitary sewer servicing is provided by extension of the South Trunk and Highway 16 Trunk and introduction of the Northwest Trunk.

From these recommendations, AECOM developed a Construction Capital Cost Plan for implementation of the off-site sanitary sewer servicing and existing system upgrades, summarized below in Table ES.1.

Table ES.1: Construction Capital Cost Plan

	Off-Site Sanitary Sewer Servicing	Existing Sanitary Sewer System Upgrades	Total (Present Value)	Total (Future Value)
Present-Day (2015)	$0	$4,180,000	$4,180,000	
Three Year Growth Horizon (2018)	$0	$6,970,000	$6,970,000	$7,400,000
Five Year Growth Horizon (2020)	$0	$3,500,000	$3,500,000	$3,860,000
Ten Year Growth Horizon (2025)	$23,480,000	$30,470,000	$53,950,000	$65,770,000
Twenty Year Growth Horizon (2035)	$5,720,000	$0	$5,720,000	$8,490,000
Forty Year Growth Horizon (2055)	$43,090,000	$0	$43,090,000	$95,120,000
Total	**$72,290,000**	**$45,120,000**	**$117,410,000**	

The Sanitary Sewer Master Plan study establishes the foundation and principles for developing and using flow monitoring data and dynamic modelling. The findings of this study and the content developed within present significant benefits to the management of the City's sanitary sewer system; however, the need for further refinement should be recognized. The Level of Service criteria is founded on a synthetic design storm, as there is not yet sufficient flow monitoring data available to establish a historical rainfall event as the critical design storm. This can only be achieved through collecting subsequent years of flow monitoring data and continued model calibration to as future data is made available. Thus, a key recommendation of this study is for the City to implement an annual flow monitoring program and related model calibration and verification of the sanitary sewer system performance and Level of Service criteria.

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1. Introduction

The City of Lloydminster, located on the provincial border between Saskatchewan and Alberta, is known as Canada's Border City. The City has experienced significant and sustained population growth over the past few decades and has a current population of approximately 32,000 people. The City has a vibrant oil and gas industry, with large amounts of exploration, drilling, and extraction in the surrounding area as well as the Husky Upgrader, located on the eastern edge of the city. The City also acts as an important centre for the local agricultural industry and consumer market for the region.

Since the last Sanitary Sewer Master Plan study, completed in 2009, the City has grown significantly. As well, a number of important sanitary sewer system upgrades have been implemented or planned for construction in the near future. In response to this, the City recognized the need to develop an updated Sanitary Sewer Master Plan study, which centres on the development and calibration of a system-wide dynamic model of the City's sanitary sewer system.

The objectives of the Sanitary Sewer Master Plan study are as follows:

- Implement a flow monitoring program
- Develop a system-wide model of the City's sanitary sewer system
- Calibrate the model for Dry Weather Flow and Wet Weather Flow based on available flow monitor data collected in 2008 and 2015
- Establish Level of Service criteria for the sanitary sewer system
- Assess the current conveyance capacity of the sanitary sewer system
- Identify system deficiencies and develop upgrade concepts to address these deficiencies
- Determine future growth boundaries for the three year, five year, ten year, twenty year, and forty year growth horizons
- Identify off-site sanitary sewer servicing and existing sanitary sewer system upgrades required to service the future growth areas
- Develop a construction capital cost plan for each of the growth horizons identified above

The subsequent report sections address all of the above objectives.

2. Sanitary System Overview

The City of Lloydminster sanitary sewer system consists of a gravity sanitary sewer system and one lift station that services the Husky Upgrader, shown in Figure A1 (Appendix A). The sanitary sewer system contains a total of 1932 manholes and 2065 pipes, with a total length of 177 km. A summary of the pipe system is shown below in Table 2.1 and on Figure A2 (Appendix A).

Table 2.1: Sanitary Sewer Pipe Summary

Size	Pipe Count	Length
200 mm and smaller	1100	89.4 km
250 mm	231	21.6 km
300 mm	145	12.3 km
375 mm	171	13.8 km
450 mm	119	11.3 km
525 mm	50	4.1 km
600 mm	61	5.9 km
675 mm	9	0.9 km
750 mm	49	4.8 km
900 mm	30	3.1 km
1050 mm	48	6.3 km
1200 mm	30	2.7 km
1350 mm	3	0.3 km

The sanitary sewer system generally drains northeast, following the overall topography of the city, to the Wastewater Treatment Plant located along 67 Street, approximately 800 m east of 40 Avenue. The sanitary sewer system has two main sanitary trunk sewers that drain to the Wastewater Treatment Plant. The first, called the East Trunk, drains north from the intersection of 36 Street and 40 Avenue to the Wastewater Treatment Plant. The second is called the North Trunk, and drains east from 62 Street and 51 Avenue along 67 Street and connects to the East Trunk just south of the Wastewater Treatment Plant.

The East and North Trunks collect flow from a number of smaller trunk sewers that extend into and service the city itself: the 52 Street Trunk, the 47 Street Trunk, the 36 Street Trunk, the 25 Street Trunk, and the Southeast Trunk, which connect to the East Trunk; and the 62 Street Trunk and West Trunk that connect to the upstream end of the North Trunk. Each of the seven smaller trunk sewers service an individual collection area, shown on Figure A3 (Appendix A). It is noted that the 62 Street Trunk and West Trunk have been constructed larger than required for the current collection area and provide capacity for future development. Details of the two main trunk sewers and the seven smaller trunk sewers are shown below in Table 2.2 and Figure 2.1.

Table 2.2: Summary of Sanitary Trunk Sewers

	Size	Length	Collection Area
East Trunk	750 mm – 1200 mm	5086 m	824 ha
North Trunk	900 mm – 1050 mm	3180 m	302 ha
52 Street Trunk	450 mm – 600 mm	2603 m	172 ha
47 Street Trunk	375 mm – 675 mm	1743 m	120 ha
36 Street Trunk	375 mm – 750 mm	2207 m	507 ha
25 Street Trunk	300 mm – 750 mm	2440 m	164 ha
Southeast Trunk	375 mm – 900 mm	3101 m	316 ha
62 Street Trunk	450 mm – 1200 mm	2061 m	30 ha
West Trunk	600 mm – 1200 mm	5228 m	248 ha

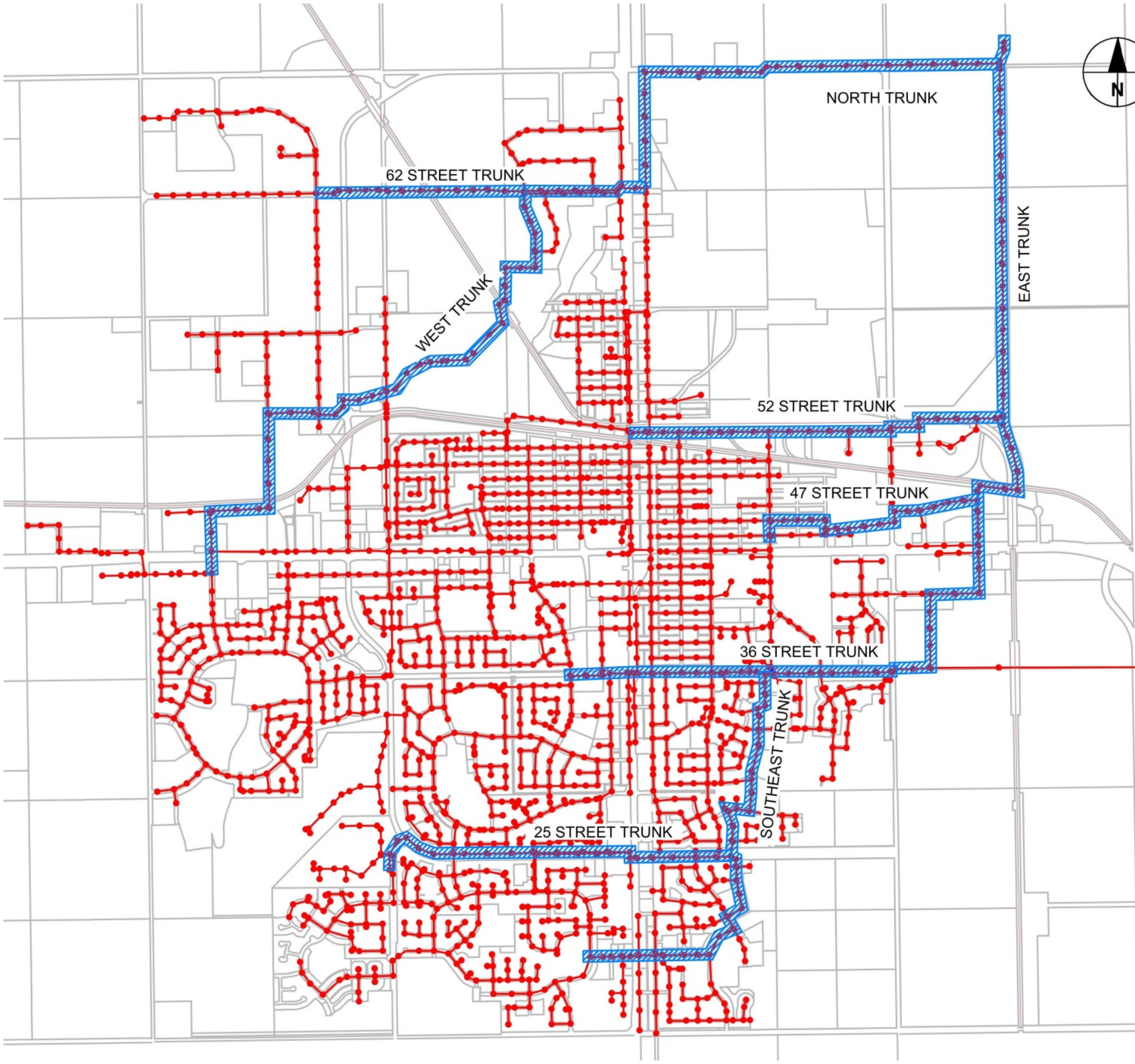


Figure 2.1: Overall Plan – Main Sanitary Trunk Sewers

The city-wide sanitary sewer system contains a total of 123 interconnections and flow-splits, for which the hydraulics of the sanitary sewer are very complex and nearly impossible to represent by using a steady state spreadsheet model. On the other hand, a dynamic computer model is much better suited to capture the complexities of the system performance. Based on the pipe inverts at the location of the flow-split, the sewer may flow in one direction during a dry weather flow scenario and flow in two directions during times of high flow. Accordingly, the collection areas presented in Table 2.2 and Figure A3 (Appendix A) for each trunk sewer are applicable to the dry weather flow and the boundaries of each area may shift during times of high wet weather flow.

The city-wide sanitary sewer system is typically installed relatively deep, with an average depth of cover from top of pipe to ground surface of 4.0 m. A sanitary sewer is usually considered to be overwhelmed when basement flooding occurs. Basements are typically constructed as 2.1 m deep and, assuming that the house is 0.3 m above the street centreline, the basement floor elevation is approximately 1.8 m below street centreline. Accounting for a freeboard allowance of 0.6 m, this means that the sanitary sewer system may be able to experience a certain degree of pipe surcharge (where the HGL is above the top of the pipe) before issues would occur.

The city-wide sanitary sewer system is predominately separate from the storm sewer system, except for a small area of combined sewer in the downtown core. The City advised that, when encountered during their annual capital works upgrades, the combined sewers are disconnected and separate storm and sanitary sewer are installed.

The Husky Upgrader is located approximately 1.6 km east of the city, south of Highway 16. The Husky Upgrader lift station pumps to a 250 mm diameter forcemain, approximately 2200 m long, and connects to the East Trunk at 36 Street, 240 m east of 40 Avenue.

3. Flow Monitoring

The City of Lloydminster owns ten flow monitors (ISCO 2150 Area Velocity Flow Meters) and three tipping bucket rain gauges (Telog RG-32). The flow monitors consist of a probe that is placed in the invert of a sanitary sewer pipe and measures velocity and depth of flow, which is converted to flow rate. The flow meter logs data on 5 minute intervals. The tipping bucket rain gauges are placed at locations throughout the city (commonly on the roof of city-owned facilities) and measure rainfall depths on 5 minute intervals, which provide rainfall intensity during a storm.

The City implemented a flow monitoring program in 2008 and 2015. The 2008 flow monitoring program was completed from May 3 to October 2 and the 2015 flow monitoring program was completed from May 6 to September 9. Both flow monitoring programs were conducted by SFE Global of Edmonton, Alberta.

The purpose of the flow monitoring program was two-fold. The first objective of the flow monitoring programs was to collect real-time flow in the sanitary sewer system for a number of consecutive dry days to determine the regular and repeatable flow pattern, defined as Dry Weather Flow (DWF). The second objective was to capture flow data and rainfall intensity for significant rainfall events that resulted in a large amount of Wet Weather Flow (WWF) in the sanitary sewer system. Both data sets were used for subsequent calibration of the city-wide sanitary sewer model.

AECOM completed a thorough review of both years of data, presented in two technical memoranda:

- Lloydminster Sanitary Master Plan Progress Update, dated August 7, 2015
- 2015 Flow Monitoring Program and Model Calibration, dated November 6, 2015

The information presented below is a summary of the 2008 and 2015 Flow Monitoring Programs and the reader is referred to these two technical memoranda, included in Appendices B and C, for more detailed information.

3.1 2008 Flow Monitoring Program

The 2008 flow monitoring program was conducted from May 3 to October 2. The program yielded a good data set of dry weather flow and had captured a significant rainfall event on August 9, 2008.

The ten flow monitors were installed throughout the sanitary sewer system and measured flow from different types of land-uses (residential and commercial) of varying sewershed sizes. A summary of the flow monitor sites is presented in Table 3.1 and Figure A4 (Appendix A).

Table 3.1: Summary – 2008 Flow Monitoring Sites

Flow Monitoring Site	Manhole ID	Pipe ID	Pipe Size (mm)	Sewershed Area (ha)	Proportion Residential	Proportion Commercial
1	354	935	375	172	20%	80%
2	29	1034	450	316	49%	51%
3	23	1040	600	835	50%	50%
4	1086	115	1050	266	28%	72%
5	2010	510	525	83	69%	31%
6	471	1113	375	70	85%	15%
7	442	1110	375	52	99%	1%
8	975	186	750	471	69%	31%
9	854	213	450	131	65%	35%
10	1766	1930	450	66	59%	41%

The three rain gauges were located throughout the city on the roofs of civic buildings. Rain Gauge 1 was located in the centre of the city on the roof of City Hall, but was inoperable for the majority of the time period. Rain Gauge 2 was located on the southern edge of the City on the roof of the Servus Sports Centre. Rain Gauge 3 was located on the western edge of the City at the West Water Reservoir.

The Dry Weather Flow data was used to calibrate the average dry weather flow in the model and to develop the diurnal pattern for the two types of flow contributors (residential and commercial).

A significant rainfall event was captured on August 9, 2008. The rain gauge at the West Reservoir had recorded 42 mm of rainfall over 5 hours and 20 minutes, which approximated a 5 year return period storm. The rain gauge at the Servus Sports Centre measured 51 mm of rainfall over 5 hours and 20 minutes, which is approximately equivalent to a 10 year return period storm. The third rain gauge was inoperable at the time of the rainfall event. The August 9, 2008 rainfall event is shown below in Figure 3.1.

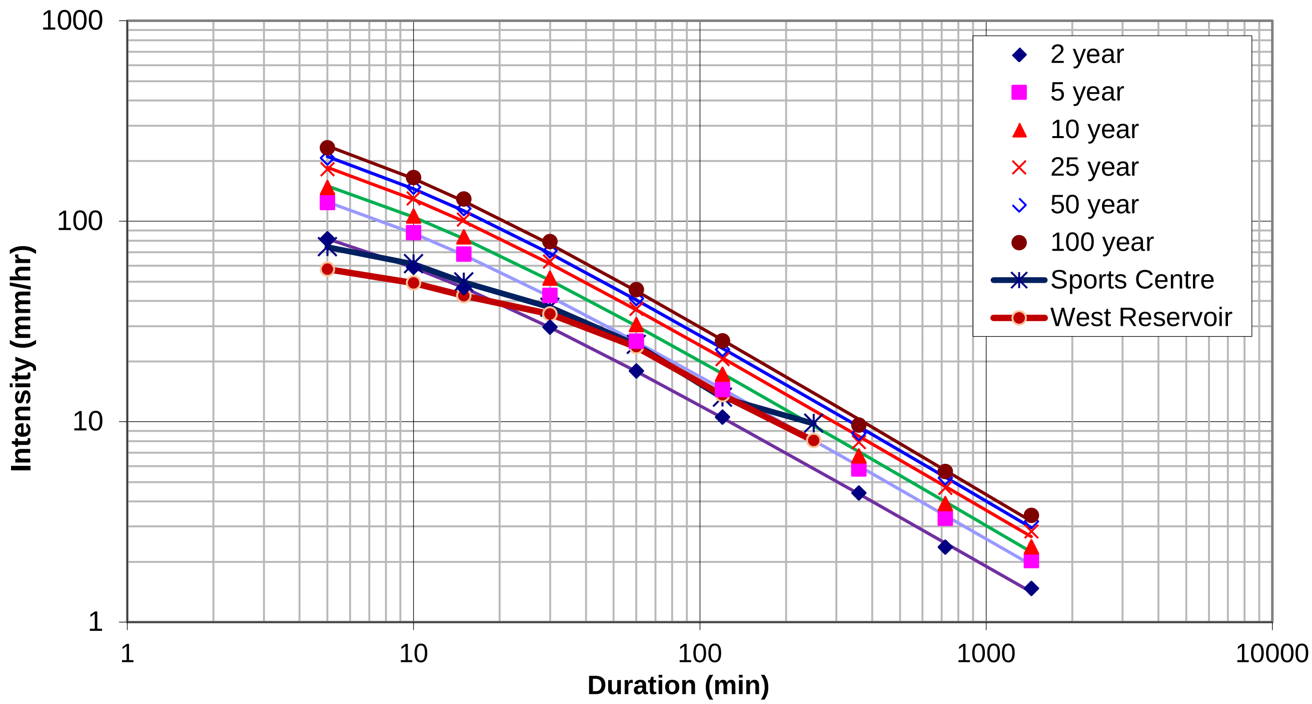


Figure 3.1: August 9, 2008 Rainfall Event

3.2 2015 Flow Monitoring Program

The 2015 flow monitoring program was in place from May 6 to September 9 and collected a robust data set of dry weather and wet weather flow. Ten flow monitors were placed through the sanitary sewer system. The locations were selected based on the following criteria:

- Measure flow in the main sanitary trunk sewers to provide a total flow in the system (sum of flow in the East Trunk and North Trunk)
- Collect data from sewersheds of varying land-uses and sizes
- Monitor flow from the section of combined sewer, located in the downtown core

A summary of the flow monitor sites is presented in Table 3.2 and Figure A5 (Appendix A).

Table 3.2: Summary – 2015 Flow Monitoring Sites

Flow Monitoring Site	Manhole ID	Pipe ID	Pipe Size (mm)	Sewershed Area (ha)	Proportion Residential	Proportion Commercial
1	999	148	1050	772	61%	39%
2	1034	101	900	302	28%	72%
3	882	999	900	198	34%	66%
4	30	1369	300	38	22%	75%
5	36	968	375	6.2	100%	0%
6	138	1246	250	5.1	19%	81%
7	975	186	750	471	69%	31%
8	386	1043	375	136	80%	20%
9	445	1435	375	47	100%	0%
10	854	213	450	131	65%	35%

The rain gauges were placed around the city as follows:

- Rain Gauge 1, located on the roof of the Civic Operations Centre, along the city's western edge
- Rain Gauge 2, located on the roof of the Water Treatment Plant, along the city's northern edge
- Rain Gauge 3, located on the roof of the Servus Sports Centre, along the city's southern edge

There were three significant rainfall events captured: June 21, August 16, and September 5 – 7. The three significant rainfall events were quite distinct, as presented in Table 3.3.

Table 3.3: Summary – 2015 Significant Rainfall Events

Date		RG1 Civic Operations Centre	RG2 Water Treatment Plant	RG3 Servus Sports Centre
June 21, 2015	Depth Duration	15 mm 2 hr 10 min	10 mm 20 min	63 mm 4 hr 10 min
August 15, 2015	Depth Duration	61 mm 12 hr 10 min	58 mm 12 hr 10 min	60 mm 12 hr 10 min
September 5 – 7, 2015	Depth Duration	66 mm 42 hr 15 min	70 mm 44 hr 30 min	64 mm 45 hr 20 min

The June 21, 2015 rainfall event was quite localized; both the rainfall amounts and storm duration were significantly less at the Civic Operations Centre and Water Treatment Plant, compared to the Servus Sports Centre. The severity of the rainfall measured at the Civic Operations Centre and Water Treatment Plant was relatively minor (less than a 2 year return period). However, the severity of the rainfall measured at the Servus Sports Centre is estimated as a 100 year return period. The June 21, 2015 rainfall event relative to the City of Lloydminster IDF curves is presented below in Figure 3.2.

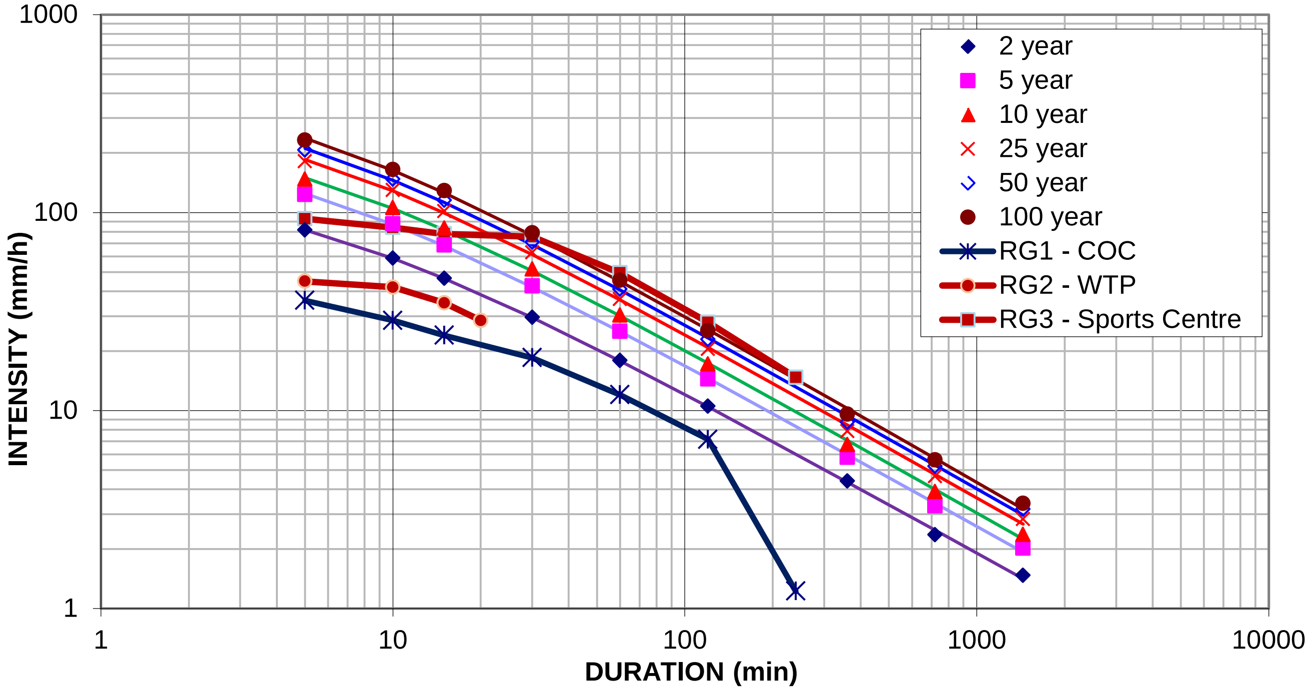


Figure 3.2: June 21, 2015 Rainfall Event

The August 15, 2015 rainfall event was consistent amongst the three rain gauge sites, measuring 58 to 61 mm depth of precipitation and all having the same duration (12 hours and 10 minutes). This rainfall event is estimated to have a 25 year return period (Figure 3.3).

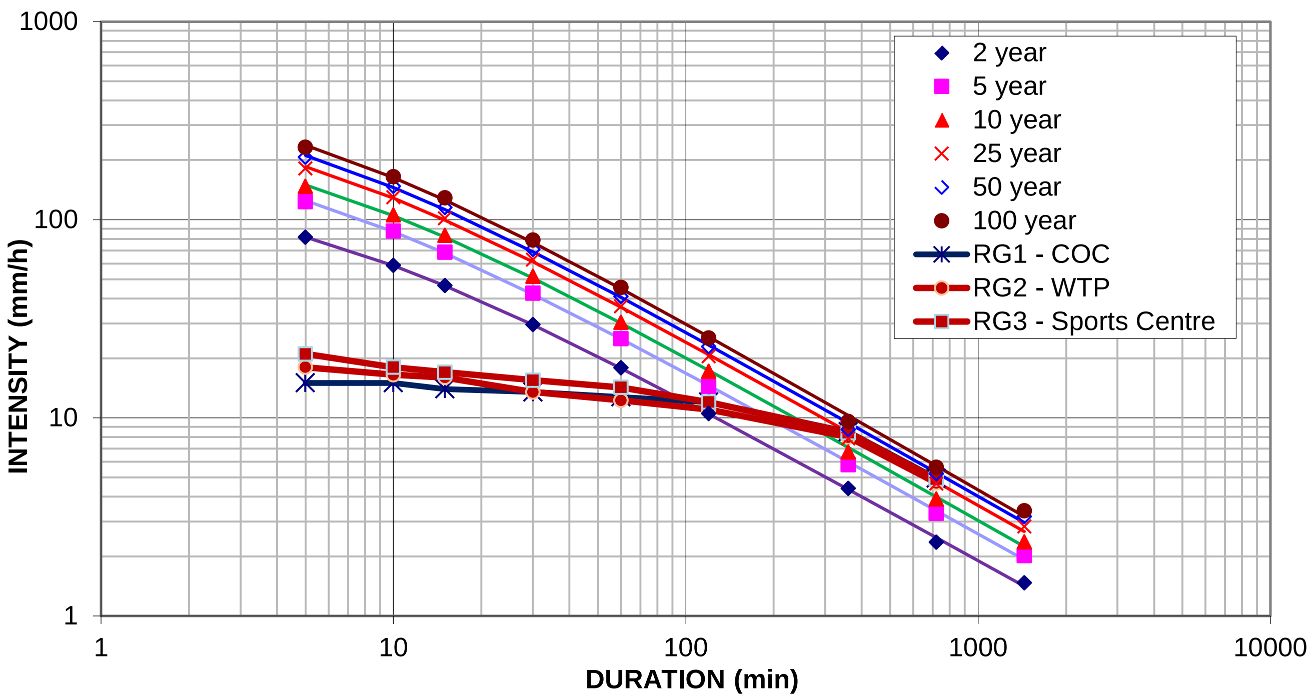


Figure 3.3: August 15, 2015 Rainfall Event

Similarly, the September 5 – 7, 2015 rainfall event had similar precipitation depths (64 to 70 mm) and storm durations (42 to 45 hours) amongst the three rain gauge sites. This rainfall event is estimated to have a 5 – 10 year return period (Figure 3.4).

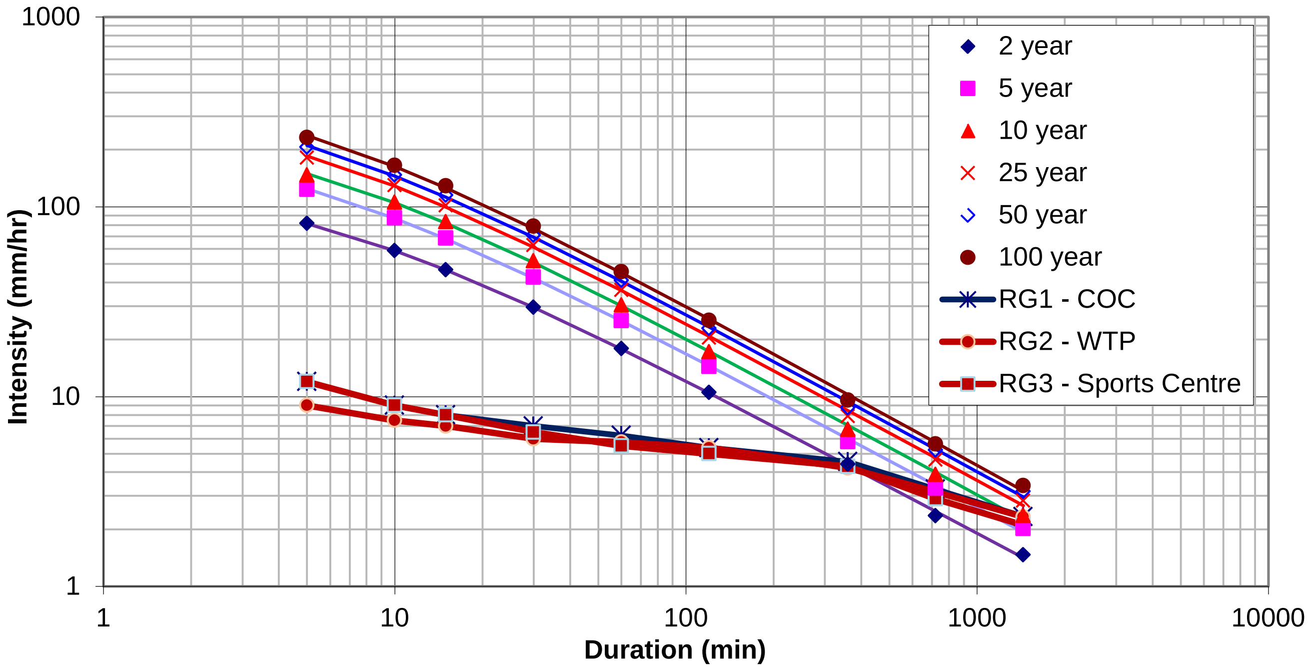


Figure 3.4: September 5 - 7, 2015 Rainfall Event

Although the June 21 rainfall event was the most severe, approaching a 100 year return period on the southern edge of the city, the wet weather flow response in the sanitary sewer system was less than would be expected. There were two possible causes for this. First, the rainfall event was quite localized, such that only a small portion of the city-wide sanitary sewer system would have experienced the most severe rainfall. Secondly, the preceding two months were considerably dry, meaning the soil likely held a large portion of the rainfall that did not infiltrate into the sanitary sewer system.

The August 15 and September 5 – 7 rainfall events were fairly widespread with small variation in rainfall depth and duration amongst the three rain gauge sites. The severity of the September 5 – 7 rainfall event was small (estimated as a 5 – 10 year return period event) and the duration was quite lengthy (42 – 45 hours), meaning that the wet weather flow response in the sewer was relatively small.

Considering the above, the August 9, 2008 and August 15, 2015 rainfall events were considered to be the most suitable rainfall event for further model calibration. Both rainfall events had a reasonable return period, rainfall depth, and duration.

4. Model Development

A dynamic computer model was developed for the city-wide sanitary sewer system using XPSWMM software (Version 2013 SP1). The sanitary sewer system is built as a link/node model, with the links representing pipes and the nodes representing manholes. All flows are input at the nodes in the model.

The XPSWMM software solves two "layers": the Runoff layer and the Hydraulics layer. The Runoff layer simulates the hydrology and the Rainfall-Derived Inflow and Infiltration (RDII), which is exported as a hydrograph at each node that the model then inputs into the nodes in the Hydraulics layer. The Hydraulics layer simulates the domestic dry weather flow at each of the nodes and adds the RDII hydrograph solved in the Runoff layer. The flow is then routed through the pipe system.

4.1 Development of Link/Node Model

As described above, the links and nodes in the model correspond to pipes and manholes. The required physical pipe properties include diameter, upstream invert, downstream invert, length, slope, and roughness. The required physical manhole properties are northing and easting coordinates, rim elevation, and invert elevation.

The City provided their GIS database of the sanitary sewer system in AutoCAD format, which contained 2268 pipes and 1820 manholes. The required pipe and manhole properties described above were exported from the AutoCAD drawing and imported into the XPSWMM model. The pipe and manhole identifiers (ID's) in the City's GIS database were retained in the XPSWMM model.

The GIS database defined pipes as being from manhole to manhole or coupler, which was used where a pipe was originally constructed with a stub that was later extended. Conversely, in the XPSWMM model, a pipe is defined as being from manhole to manhole only. In cases where a coupler was used (meaning that there was more than one pipe between manholes), the multiple pipes were combined into a single pipe with the Pipe ID of the downstream pipe retained. Additionally, existing pipe stubs that are installed for future extension were excluded from the XPSWMM model. The instances of pipe couplers and stubs were encountered in approximately 320 pipes.

The GIS database showed the gravity pipe consisted of Vitrified Clay Tile (VCT), concrete, and PVC. A roughness value of 0.013 was applied to the VCT and concrete pipes and a roughness value of 0.011 was used for the PVC pipes.

The sanitary sewer model terminated at the Wastewater Treatment Plant, which was configured as an outfall with flow exiting the model at normal depth.

The manhole rim elevation was missing for 316 manholes. A Civil 3D surface was created from the City's LIDAR information to determine approximate ground elevation at these manholes.

A small portion of the GIS database was observed to have incorrect/questionable/missing data and was reviewed against City record drawings. As well, portions of the College Park, Larsen Grove, and Wallace Field neighbourhoods were missing from the GIS database and this data was recorded manually from record drawings.

The pipe data was subjected to a Quality Assurance review by comparing the pipe length calculated from the manhole coordinates and pipe slope calculated from this length and the pipe inverts. Questionable pipe length and slope data was overwritten by the calculated values. Following the pipe and manhole data import into XPSWMM, a profile of each pipe section was drawn in the model to evaluate the quality of the data.

4.2 Dry Weather Flow Generation

The dry weather flow portion of sewer flow is composed of groundwater infiltration and domestic sewage flow. Groundwater infiltration is a relatively constant flow and varies on a seasonal time scale. The domestic sewage flow follows a regular and repetitive pattern throughout a 24 hour period. The dry weather flow contribution is input into the XPSWMM model at the manholes on the Hydraulics layer as an average daily flow.

The first step in generating the dry weather flow is to distribute the total average daily flow throughout the model. The City of Edmonton sanitary sewer design standards were used to develop the average daily flow at each manhole. These design standards were chosen as they provide detailed design values for residential, commercial, and institutional land-uses, which is valuable considering the wide range of flows generated by different types of business (such as a warehouse and a medical clinic).

The collection area serviced by each section of pipe was delineated and assigned to its upstream manhole. In residential areas, the number of single family homes and multi-family units was counted and a population density applied to determine the population for each manhole's collection area. The system services a total of 8350 single family homes and 3511 multi-family units. Using a total current population of 32,000, the population density was calculated to be 3.2 persons per household for single-family homes and 1.6 persons per household for multi-family units. A per capita flow of 300 lpcd was used to determine the total average daily flow for the manhole.

For collection areas that contained businesses and institutions (school, church, library, etc.), the type of building was recorded and its floor area was measured in AutoCAD. The average daily flow was calculated by multiplying the floor area by the relative flow generation for that type of building. The average daily flow for different types of buildings is presented below in Table 4.1.

Table 4.1: Commercial and Institutional Design Values

Type of Building	Average Daily Flow (l/day/m^2 of floor area)
Shopping Centres	4
Car Dealers, Repair, and Service	6
Office Building, Neighbourhood Stores, Service Stations, Supermarkets, Trade Businesses (e.g. Plumber)	8
Banks, Medical Clinics, Lounges	12
Restaurants	20
Schools, Churches, Libraries, and other Places of Assembly	24
Dry Cleaners	41
Carwashes	77
Hospitals	1700 L/bed/day

There were a total of 1276 individual collection areas throughout the city, which had a total average daily flow of 243.5 L/s, based on design values. From the 2015 flow monitoring program, the total average daily flow (the sum of the flow from Site 1 in the East Trunk and Site 2 in the North Trunk) was 117.3 L/s, which is a 52% reduction from the design value. To calibrate the average daily flow in the XPSWMM model, the design values were reduced by this amount.

The five largest flow contributors are presented below in Table 4.2. The sum of these average daily flows is equal to 10.12 L/s, which is 9% of the total average daily flow for the entire city.

Table 4.2: Five Largest Flow Contributors

Building	Average Daily Flow
ADM Crushing Plant	3.27 L/s
Exhibition Grounds	1.81 L/s
Lakeland College	1.75 L/s
Servus Sports Centre	1.69 L/s
Hospital	1.60 L/s

The above methodology aided in distributing the average daily flow amongst the manholes in the model; however, it is recognized that the flow contributions are not perfectly distributed in reality. To address this, the dry weather flow data from the 2015 flow monitoring program was evaluated.

For each of the ten flow monitoring sites, an adjustment factor for the average daily flow was used within the collection system that was specific to each individual flow monitor site to match the model results to the flow monitor data. For example, an adjustment factor was determined to fit the average daily flow in Site 3. A different factor was then determined for the flow contributors specific to only Site 2, which is downstream of Site 3. As such, the average daily flow at Site 2 was calculated from a combination of the adjustment factors specific to only Site 2 and Site 3. The average daily flow based on design values and subsequent calibration and the adjustment factors used at each flow monitor site are shown below in Table 4.3.

Table 4.3: Average Dry Weather Flow Peaking Factors

Flow Monitoring Site	Average Daily Flow				
	Design Values	Reduction in Design Value	Adjustment Factor	Model Calibration Results	Flow Monitor Data
1	195.9 L/s	0.52	0.904	85.0 L/s	84.4 L/s
2	86.7 L/s	0.52	0.795	33.1 L/s	32.9 L/s
3	45.4 L/s	0.52	1.050	22.9 L/s	22.7 L/s
4	35.7 L/s	0.52	0.216	3.7 L/s	3.8 L/s
5	0.96 L/s	0.52	1.700	0.78 L/s	0.78 L/s
6	2.22 L/s	0.52	1.260	1.34 L/s	1.36 L/s
7	66.8 L/s	0.52	1.400	44.9 L/s	49.1 L/s
8	33.8 L/s	0.52	0.548	8.9 L/s	9.0 L/s
9	4.44 L/s	0.52	1.220	2.6 L/s	2.6 L/s
10	20.8 L/s	0.52	1.359	13.6 L/s	13.6 L/s

The total average daily flow for residential and commercial is 60.7 L/s and 56.6 L/s, respectively. It is noted that, using a current population of 32,000 people, the average flow generation is 316 lpcd over the whole city, and 164 lpcd for only the residential contribution.

The next step in developing the dry weather flow portion of the sanitary model was to determine the diurnal patterns. Domestic sewage flow usually follows a regular and repetitive pattern throughout a 24 hour period, defined as the diurnal pattern, and different types of land-uses have distinct diurnal patterns. For example, a typical residential diurnal pattern has a maximum peak flow occurring at approximately 7:00 am, a second smaller peak flow at approximately 7:00 pm, and a minimum flow at approximately 3:00 am. A commercial diurnal pattern is characterized by a peak flow that is relatively sustained over the course of regular business hours and a sustained minimum flow through the early morning hours.

The diurnal pattern is expressed as a set of hourly multipliers, which the average daily flow is multiplied by to produce a daily flow hydrograph. It is important to note that the average of the hourly multipliers is equal to 1.00, meaning the volume of the flow in the daily flow hydrograph is equal to the average daily flow. Three distinct diurnal patterns were developed to fit the dry weather flow hydrographs – residential, commercial, and industrial (applied to the northwest industrial area of the city). The maximum peaking factor for the residential, commercial, and northwest industrial diurnal patterns corresponded to 1.63, 1.47, and 2.10. The three diurnal patterns are presented below in Table 4.4 and Figure 4.1.

Table 4.4: Diurnal Patterns

Time	Average Daily Flow - Hourly Multipliers		
	Residential	Commercial	Northwest Industrial
0:00	0.61	0.58	0.30
1:00	0.51	0.52	0.30
2:00	0.48	0.53	0.30
3:00	0.51	0.54	0.30
4:00	0.60	0.54	0.30
5:00	0.87	0.57	0.30
6:00	1.38	0.70	0.30
7:00	1.63	0.88	0.80
8:00	1.18	1.05	1.80
9:00	1.11	1.27	2.00
10:00	1.03	1.27	2.10
11:00	1.00	1.38	2.10
12:00	0.98	1.39	2.10
13:00	0.93	1.47	1.60
14:00	0.89	1.18	1.00
15:00	0.97	1.34	1.40
16:00	1.05	1.37	1.20
17:00	1.25	1.35	1.20
18:00	1.25	1.31	1.20
19:00	1.32	1.17	1.20
20:00	1.37	1.19	0.80
21:00	1.24	0.98	0.60
22:00	1.06	0.82	0.40
23:00	0.78	0.60	0.40
Average	**1.00**	**1.00**	**1.00**

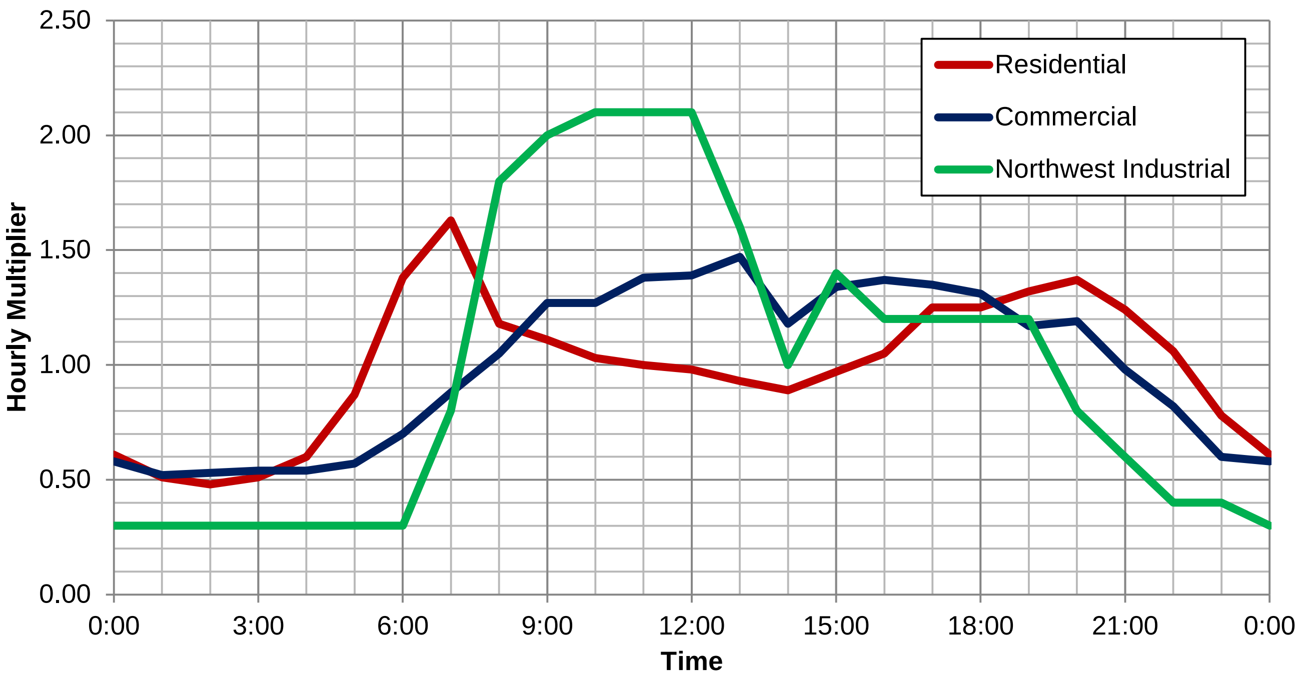


Figure 4.1: Dry Weather Flow Diurnal Patterns

4.3 Wet Weather Flow Generation

The sewershed areas draining to each individual manhole were delineated and included the full residential lots to account for RDII flow through the service connections. For commercial and industrial areas with large parking lots and compounds, the sewershed was drawn to include the building and service connection (excluding the parking lot and compound). For sections of sanitary sewer without services, a nominal width approximately equal to the roadway was assigned to its upstream manholes to allow for RDII flow into the sanitary sewer and manhole. There were 1753 individual sewershed areas which amounted to a total sewershed area of 1126 ha.

The Wet Weather Flow in a sanitary sewer system is composed of Dry Weather Flow and the Rainfall-Derived Inflow and Infiltration (RDII) flow. The RDII flow was simulated in the XPSWMM model with the RTK method, which develops an RDII hydrograph from the sum of three unit hydrographs, shown below in Figure 4.2.

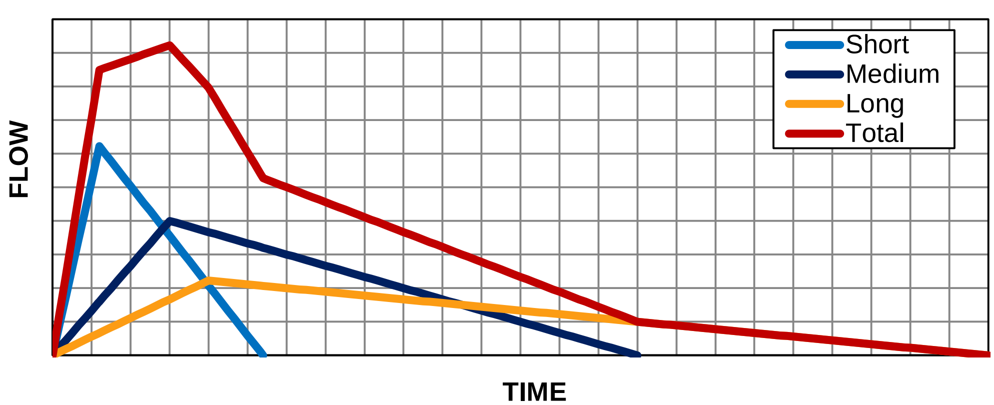


Figure 4.2: RTK Methodology

Each unit hydrograph represents the fast, medium, and slow response of the inflow to the system. The fast response may indicate direct inflows to the system through low-lying manhole lids, storm sewer cross-connections, and direct drainage connections. The medium response is composed of flows that take longer to develop, such as weeping tile flows. Finally, the slow response is representative of longer-lasting inflows such as increased infiltration (above the relatively constant groundwater inflow) into the system through pipe and manhole joints.

Each of the three unit hydrographs are drawn based on the R, T, and K parameters. The R value represents the proportion of the rainfall that enters the sanitary sewer, T is the time of the peak in the hydrograph, and K describes the recession limb of the hydrograph (ratio of time to subside to time to peak). The flow rate at a given time interval is calculated by multiplying the R value by the rainfall intensity and the sewershed area of the node.

The RTK method allows for an initial abstraction loss, which is the rainfall depth at the start of the rainfall that does not contribute to RDII flow. This accounts for losses such as the volume of rainfall that is held by the soil and does not infiltrate to the sanitary sewer system. This value is meant to represent antecedent moisture conditions and may have a significant impact on RDII simulations.

The flow monitor and rain gauge data from the August 9, 2008 rainfall event was used to determine the RTK parameters as presented below in Table 4.5. It was found that three sets of RTK parameters resulted in the best fit to the flow monitor data. As would be expected, the combined sewer located in the City's downtown resulted in exceptionally high RDII flows, compared to the remainder of the city.

Table 4.5: August 9, 2008 Rainfall Event – RTK Parameters

	Residential/Commercial			Downtown (Combined Sewer)			Northwest Industrial		
	Fast	Medium	Slow	Fast	Medium	Slow	Fast	Medium	Slow
R	2.5%	4.5%	6.0%	7.0%	7.5%	10.0%	2.0%	2.0%	4.5%
T	0.7 h	3.0 h	9.0 h	1.0 h	3.0 h	9.0 h	0.7 h	3.0 h	9.0 h
K	3	4	6	3	4	6	2	4	6
Initial Abstraction	2 mm			2 mm			2 mm		

A second set of RTK parameters was developed for the August 15, 2015 rainfall event, shown below in Table 4.6. It was found that the area of combined sewer no longer had a significantly higher RDII than the remainder of the city. This indicates that a large portion of the combined sewer has been separated since the 2008 flow data was collected.

Table 4.6: August 15, 2015 Rainfall Event – RTK Parameters

	Residential/Commercial			Northwest Industrial		
	Fast	Medium	Slow	Fast	Medium	Slow
R	2.0%	0.5%	1.5%	2.1%	1.5%	2.0%
T	1.5 h	3.0 h	9.0 h	1.2 h	3.0 h	9.0 h
K	3.5	4.0	6.0	3.5	4.0	6.0
Initial Abstraction	10 mm			10 mm		

It is interesting to note the difference in R values between the August 9, 2008 and August 15, 2015 rainfall events (excluding the combined sewer RTK parameters). Although the August 9, 2008 rainfall event was found to have a greater frequency (10 year return period) than the August 15, 2015 rainfall event (25 year return period), the total R values are considerably higher and the initial abstraction is significantly less. This shows that the RDII flow to the sanitary sewer system is quite complex and that a lower frequency storm (i.e. higher return period) does not necessarily correlate to a greater wet weather flow response.

For further details on the model calibration, the reader is referred to two previous technical memoranda submitted by AECOM:

- Lloydminster Sanitary Master Plan Progress Update, dated August 7, 2015 (included in Appendix B)
- 2015 Flow Monitoring Program and Model Calibration, dated November 6, 2015 (included in Appendix C)

4.4 Model Simulation Setup

The Hydraulics layer in the XPSWMM model was set up to simulate a three day period: one day of dry weather flow, a second day for the wet weather flow event, and a third day to return to the dry weather flow pattern. The time step was set to 10 seconds and model results were saved every 60 seconds. The model was configured to start “hot”, which means the simulation initiated with the actual flow and depth in the links and nodes that would be present at that particular time of the day. This is preferred to starting “cold” (no flow in the system) as the sanitary system takes three hours to reach normal flow conditions.

The Runoff layer was set up to solve a two day period to encompass the full duration of the long-term RTK hydrograph. The time step was set to 60 seconds.

The continuity error was 0.014%, which is considered to be minimal. A three day simulation would take approximately 27 minutes to complete.

5. Level of Service Criteria

The Level of Service criteria is a vital key to evaluating the performance of a sanitary sewer system and can be defined as the policy objectives that a municipality commits to achieve in their sanitary sewer system. By establishing practical criteria to measure the performance of a sanitary sewer system, one may determine the capacity of the existing system to convey an accepted design event, identify areas of concern, develop mitigation strategies to address these areas of concern, and ascertain locations of available capacity to assist in devising servicing plans for future development.

The Level of Service criteria has two components – the design rainfall event and the performance characteristics of the sanitary sewer system.

5.1 Design Rainfall Event

The design rain event can be either a theoretical storm distribution of a selected return period or a historical event that approximates a return period of sufficient severity. The ideal design rainfall event is a historical event of reasonable severity that results in a critical wet weather flow response in the sanitary sewer system and for which tipping bucket rainfall data and flow monitor data is available. However, until field data is collected for a rainfall event of this magnitude, one must apply the most suitable data available.

A design rain event with a 25 year return period is an objective commonly used by other municipalities as a sufficient risk management tool. A lesser return period is seen as occurring too often while a greater return period may result in performance assessments and upgrade options that are impractical and too costly to reasonably implement. It was agreed by the City to adopt this return period as a Level of Service criterion.

Another consideration in selecting the design rain event is the duration of the storm. Shorter durations (less than 2 hours) often have a high rainfall intensity that produces a high amount of surface runoff, which in turn results in a small RDII response in the sanitary sewer. Longer durations (more than 12 hours) typically have a smaller rainfall intensity that is sustained over a number of hours. This may produce a large volume of RDII, but the RDII flow is too small to generate a critical response in the sanitary sewer.

The August 23 to 25, 2005 storm is the most severe storm that has occurred in recent history. There was 135 mm of rainfall over the three day period and 99 mm of rainfall on August 24 alone. By extrapolating the IDF data, this event was roughly approximated to a 65 year return period. Although this storm was much more severe than could be reasonably evaluated for the performance of the sanitary sewer, the records of basement flooding provided a valuable indicator of possible restrictions in the sewer system.

The August 9, 2008 storm yielded 42 mm of rainfall, measured at the West Reservoir, and approximated a 5 year return period event. This level of return period was considered to be too minor to establish as a design rain event.

The August 15, 2015 storm yielded 60 mm of rainfall over 12 hours and 10 minutes and approximated a 25 year return period event. However, despite its return period, the rainfall event did not result in a considerable response in the sanitary sewer system. This is likely due to the high level of initial abstraction (10 mm) and the relatively long duration of the event; both of which would have caused a greater proportion of the precipitation to be held by the soil and not reach the sanitary sewer system. Although this historical storm approximates the established level of service, it was decided that it should not be selected as the design rainfall event as the sanitary sewer system did not present a critical response to the wet weather flow. At the present time, there is not sufficient confidence and understanding of the sanitary sewer system's response to wet weather flow to ascertain the August 15, 2015 historical storm as a critical design rainfall event.

Taking into account the above discussion, it was decided to establish the design rainfall event by scaling the August 9, 2008 rainfall event to a 25 year return period by multiplying the normalized rainfall intensity by the 25 year 5 hour 20 minute rainfall depth of 50 mm. As the City has undergone significant growth since 2008, it was decided to use the 2015 dry weather flow calibration with the August 9, 2008 RTK parameters (excluding the combined sewer RTK parameters).

In addition to establishing the 25 year return period as the target Level of Service, the City requested analysis be completed for the 50 year and 100 year design storms. Similar to 25 year design storm, the 50 year and 100 year design storms were developed by scaling the August 9, 2008 storm to the 5 hour 20 minute rainfall depths (57 mm and 88 mm, respectively). The rainfall hyetographs for the August 9, 2008 storm and the scaled 25, 50, and 100 year design storms are shown below in Figure 5.1.

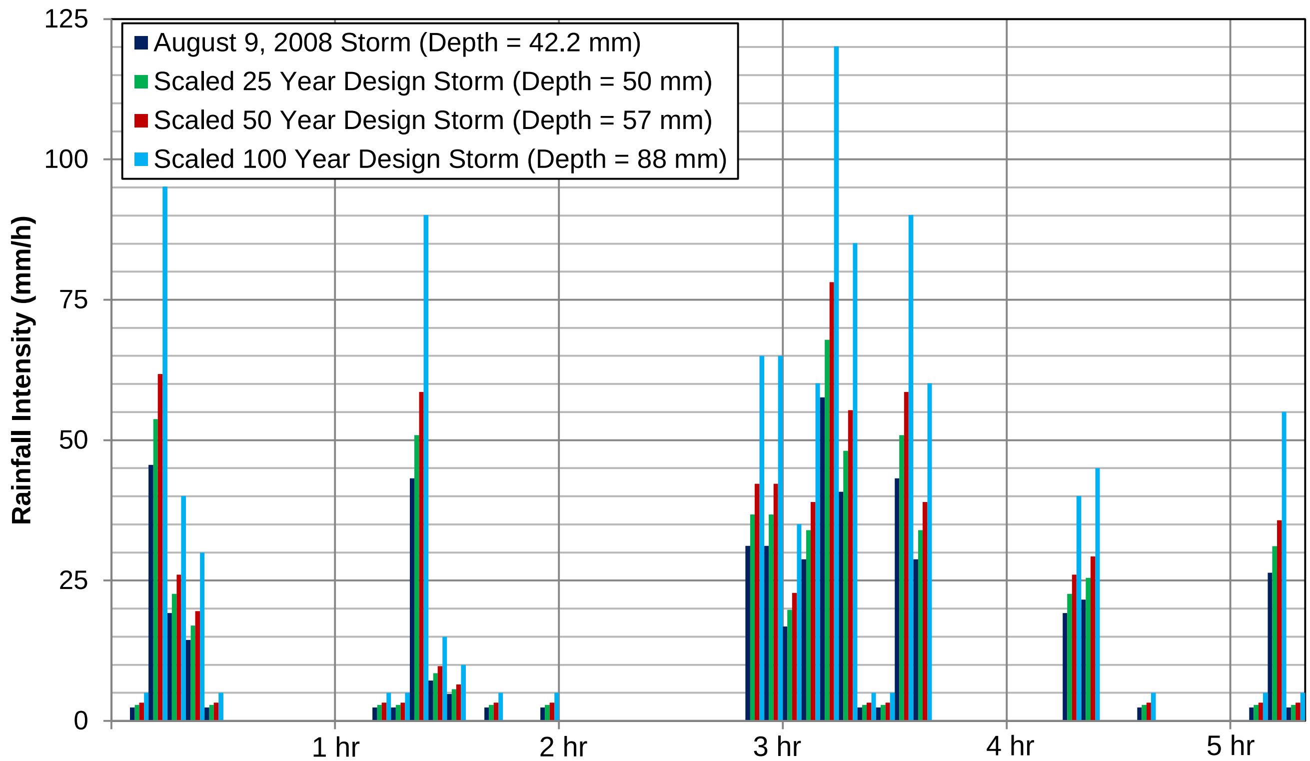


Figure 5.1: Design Storm Hyetographs

The wet weather flow response in the sanitary sewer system is complex and often requires rainfall and flow monitoring data to better understand the dynamics. The 25 year design storm was developed by scaling the August 9, 2008 storm, which was estimated as a 5 year return period storm. This was deemed to be a reasonable amount of extrapolation of the wet weather flow data. It is recognized that there is a limit to the degree of extrapolation that should be made, as the wet weather flow response in the sanitary sewer system is not directly dependent on the return period of the storm. For instance, the soil may not have reached full saturation in the actual storm. However, in a more-severe storm, the soil would reach full saturation so that its infiltration capacity would limit the amount of precipitation that is absorbed (and enters the sanitary sewer system), with the excess becoming surface runoff. This effect would not be apparent by simply extrapolating the design storm to a 50 year or 100 year return period.

Further the selection of a design rainfall event should be considered as an interim-basis. Greater confidence in the selection of a design rainfall event will be acquired only after collecting sufficient years of flow monitor data that captured a number of critical wet weather flow events.

5.2 Sanitary Sewer Performance Characteristics

A sanitary sewer system is typically considered to be overwhelmed when the Hydraulic Grade Line (HGL) in the pipe rises to a level where basement flooding may occur. As it is assumed that the depths of basement are typically 1.8 m below road centreline, basement flooding would occur when the HGL rises above this depth. A 0.6 m freeboard allowance was then added to the depth of HGL criterion. The XPSWMM model was set up to identify nodes based on the following criteria:

- Basement flooding unlikely (HGL more than 2.4 m below surface)
- Basement flooding possible (HGL between 1.8 m and 2.4 m below surface)
- Basement flooding likely (HGL less than 1.8 m below surface)

Sanitary sewers are typically installed at a minimum of 2.9 m depth to be below the frost line. This means that the above criteria allows for some amount of pipe surcharge (more than full-flow capacity). While this is acceptable in some cases, the proportion of full-flow capacity is often used to identify restrictions in the system. In addition, this criterion also identifies pipes where under surcharge where the HGL is clearly steeper than the pipe slope, which further indicates the presence of a restriction. A second evaluation of performance was set up as the percentage of full-flow capacity:

- Peak wet weather flow at less than 90% of full-flow pipe capacity
- Peak wet weather flow at 90% to 150% of full-flow pipe capacity
- Peak wet weather flow at more than 150% of full-flow pipe capacity

The above criteria were used to assess the performance of the existing sanitary sewer system, identify system constraints, develop upgrade options, and to address servicing for future development areas.

6. Performance of Existing Sanitary Sewer System

The performance of the existing sanitary sewer system was evaluated based on the Level of Service criteria established in the interim and presented in the preceding section. As agreed with the City, the critical design rainfall event was the 25 year design storm scaled from the August 9, 2008 rainfall event and the August 9, 2008 RTK parameters.

In addition, the City requested an analysis of the sanitary sewer system performance for the 50 year and 100 year design storms. The City recognized that this is more-severe than the established level of service objectives and the limitations of extrapolating the flow data to this degree, described in the previous section.

6.1 Model Results (25 Year Design Storm)

The performance of the sanitary sewer system, in terms of locations of possible basement flooding and pipe surcharge, is presented in Figure A6 (Appendix A).

There was widespread pipe surcharge observed in the sanitary sewer system; however the extent of possible basement flooding was fairly limited to an area approximately bound by 41 Street to the north, 45 Avenue to the east, 33 Street to the south, and 51 Avenue to the west. It is noted that the August 25, 2005 rainfall event had caused basement flooding in the same general vicinity. There was also an isolated location at 44 Street and 56 Avenue that showed possible basement flooding. Both of these areas were identified for development of sanitary sewer upgrades, presented in the following section.

The trunk sewers were mostly able to convey the peak flow without concern. There were some locations of surcharge in the trunk sewers; however, the HGL was maintained well below ground surface, except for locations in the 47 Street Trunk and 36 Street Trunk. The 47 Street Trunk and 36 Street Trunk had locations where the HGL reached a depth of 2.12 m and 2.17 m below ground, respectively, which is related to the general location of concern described above.

A summary of the flows in each of the trunk sewers is presented below (Table 6.1) and the peak HGL profile in each of the trunks is shown below in Figure 6.1 to Figure 6.9. Although the HGL profile in each of the figures show the basement floor elevation profile (1.8 m to 2.4 m below ground surface), it is recognized that a number of the trunk sewers do not have direct service connections.

Table 6.1: Summary of Present Day Flows - Trunk Sewers

	Downstream End of Trunk Sewer					Minimum Depth of HGL Below Ground (2)
	Size	Full-Flow Capacity(1)	Average Dry Weather Flow	Peak Dry Weather Flow	Peak Wet Weather Flow	
52 Street Trunk	600 mm	340 L/s	20 L/s	25 L/s	274 L/s	2.64 m
47 Street Trunk	675 mm	410 L/s	15 L/s	19 L/s	235 L/s	2.12 m
36 Street Trunk	750 mm	480 L/s	49 L/s	64 L/s	642 L/s	2.17 m
25 Street Trunk	750 mm	470 L/s	14 L/s	19 L/s	220 L/s	4.96 m
Southeast Trunk	750 mm	630 L/s	36 L/s	46 L/s	582 L/s	2.48 m
East Trunk	1050 mm	1450 L/s	84 L/s	107 L/s	1181 L/s	3.02 m
62 Street Trunk	450 mm	180 L/s	3 L/s	7 L/s	40 L/s	4.54 m
West Trunk	1050 mm	1950 L/s	28 L/s	39 L/s	320 L/s	4.03 m
North Trunk	900 mm	1650 L/s	33 L/s	48 L/s	377 L/s	4.46 m

(1) The full-flow capacity is calculated at the downstream end of the trunk sewer. In cases where the downstream pipe section slopes steeply to its outlet (relative to the overall trunk profile), the next upstream section was considered for the full-flow capacity.

(2) Minimum depth across the length of trunk sewer shown below

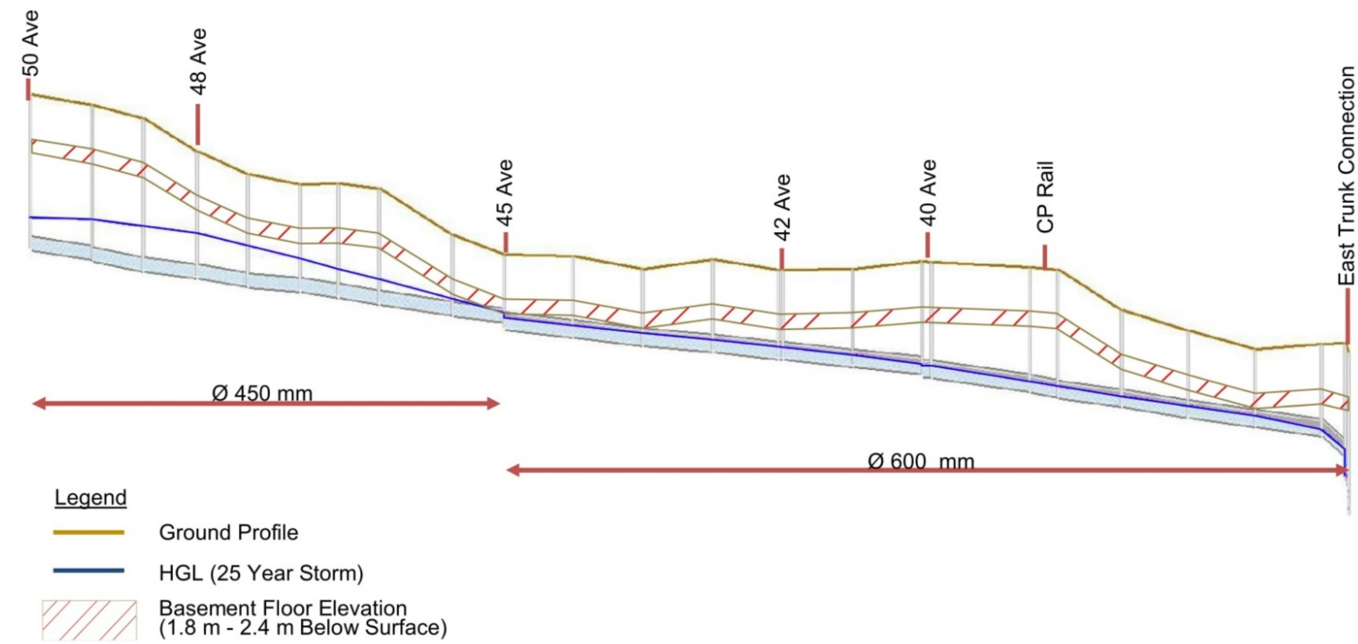


Figure 6.1: 52 Street Trunk – 25 Year Storm HGL Profile

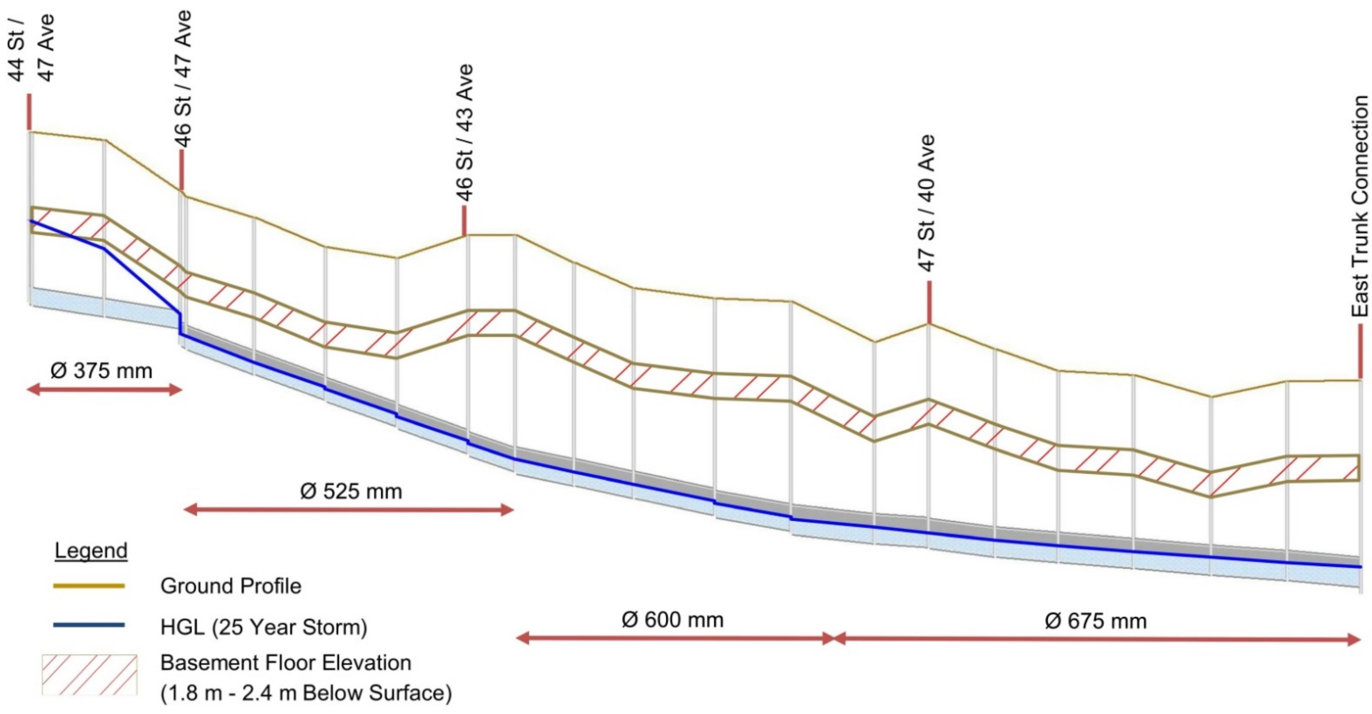


Figure 6.2: 47 Street Trunk – 25 Year Storm HGL Profile

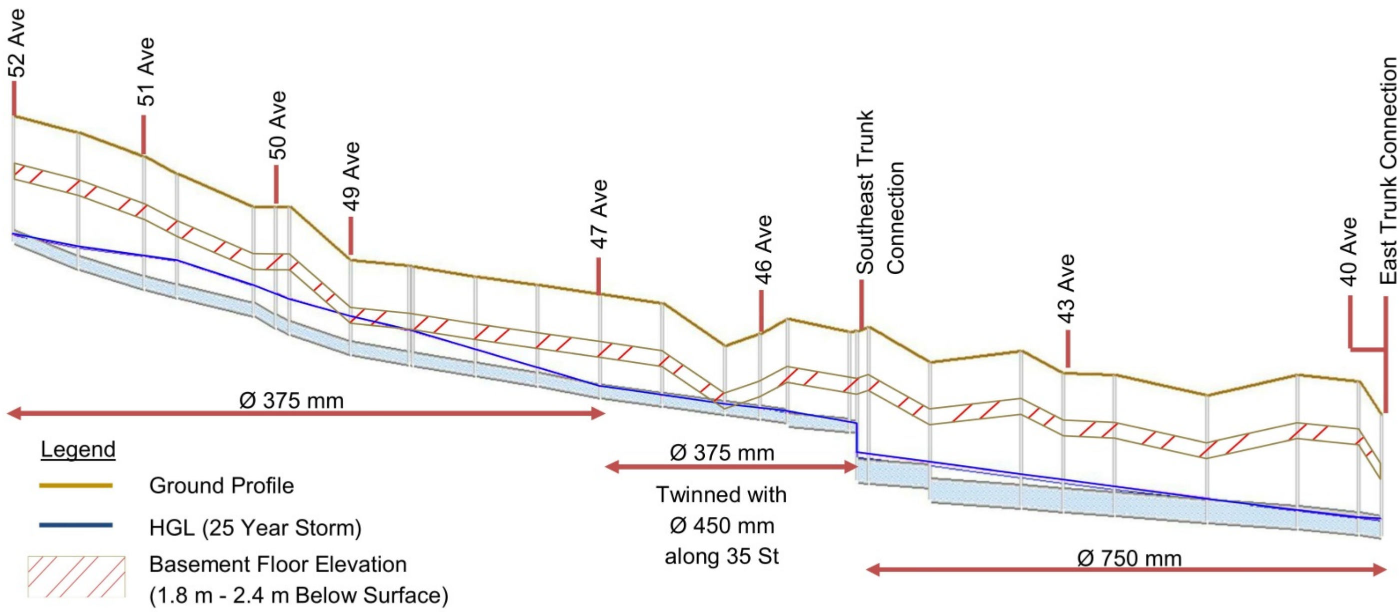


Figure 6.3: 36 Street Trunk – 25 Year Storm HGL Profile

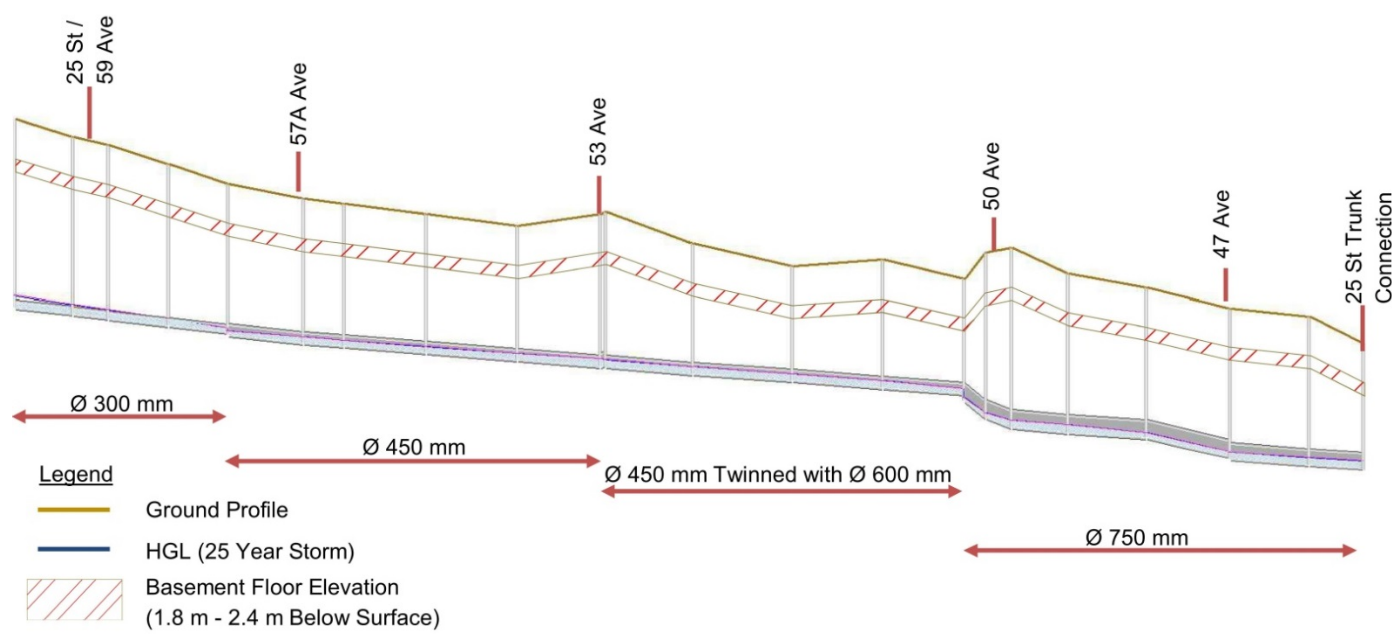


Figure 6.4: 25 Street Trunk – 25 Year Storm HGL Profile

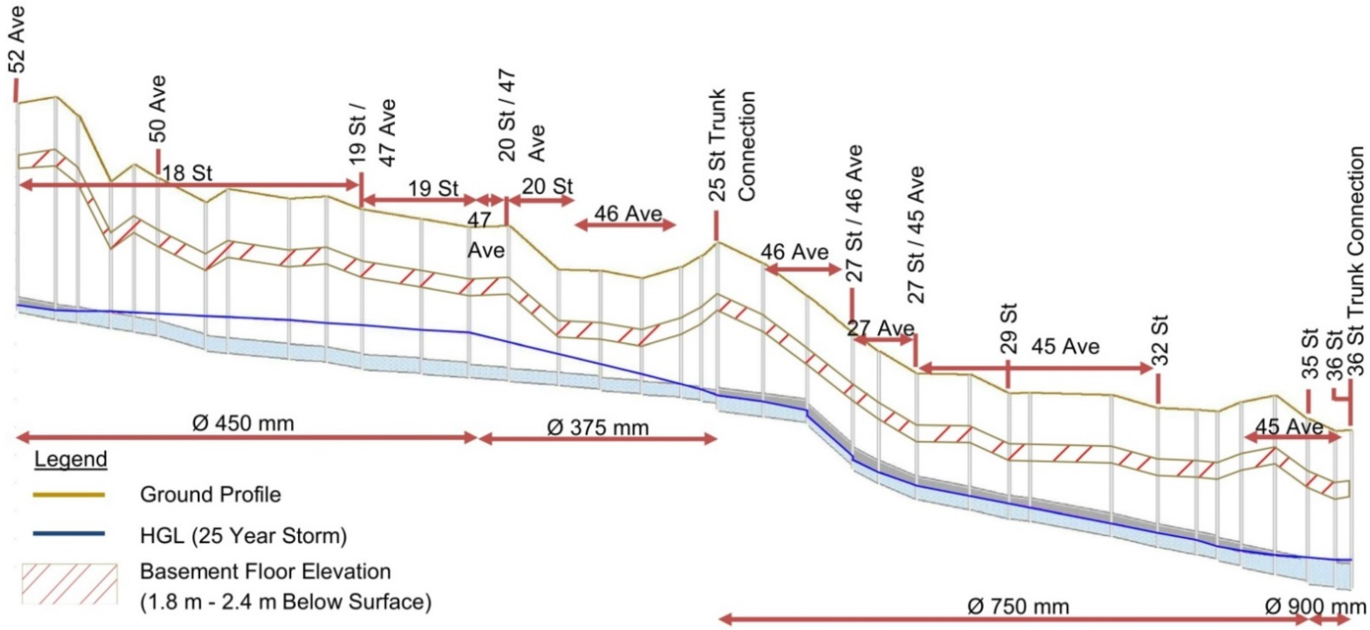


Figure 6.5: Southeast Trunk – 25 Year Storm HGL Profile

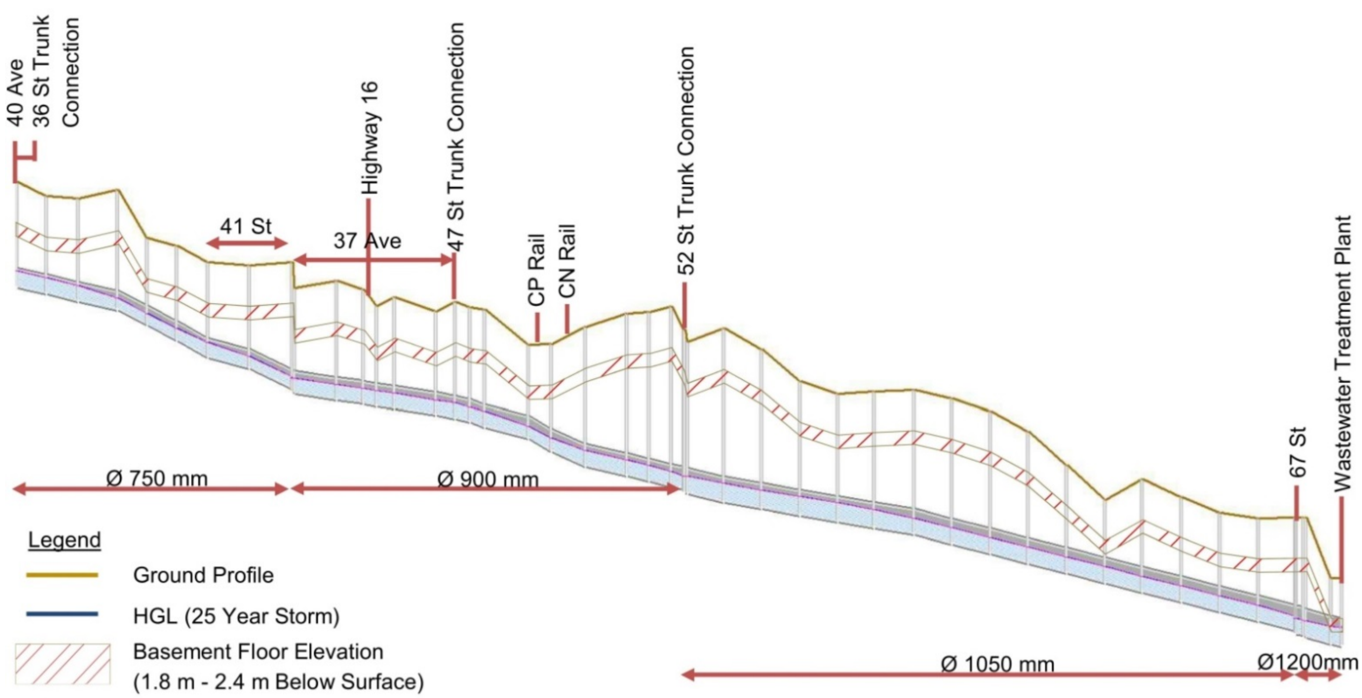


Figure 6.6: East Trunk – 25 Year Storm HGL Profile

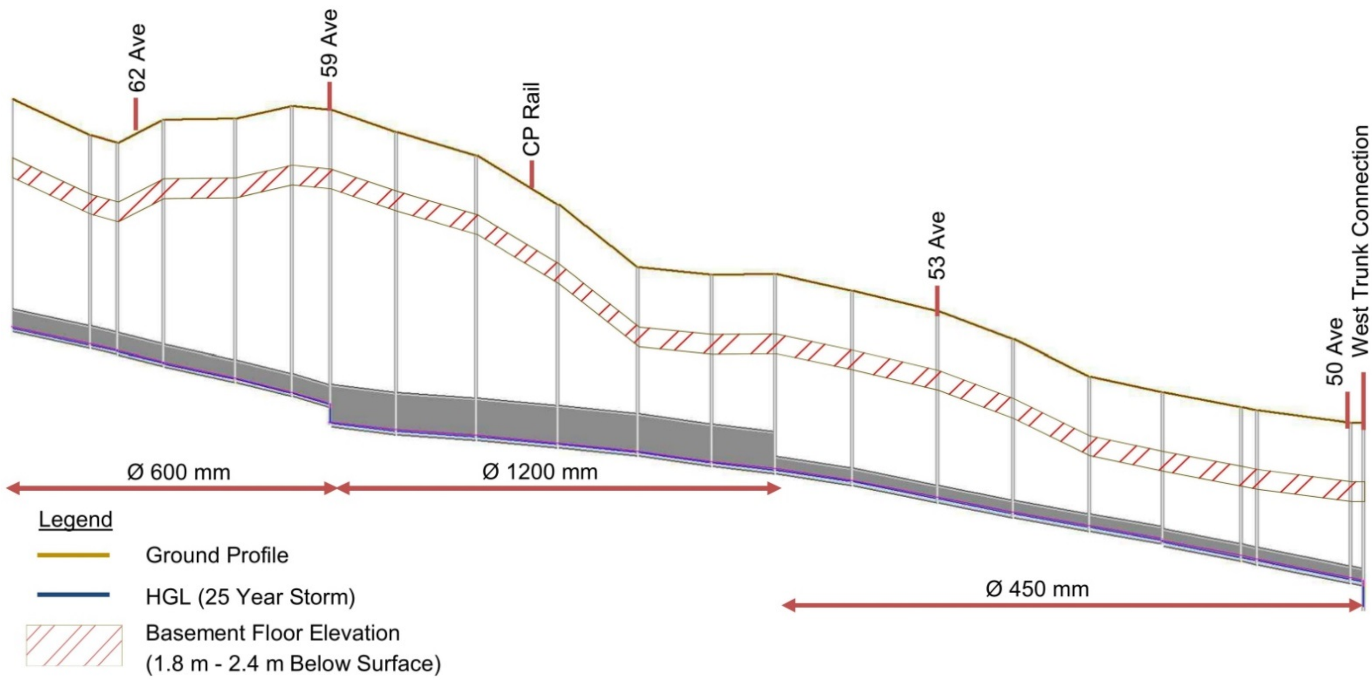


Figure 6.7: 62 Street Trunk – 25 Year Storm HGL Profile

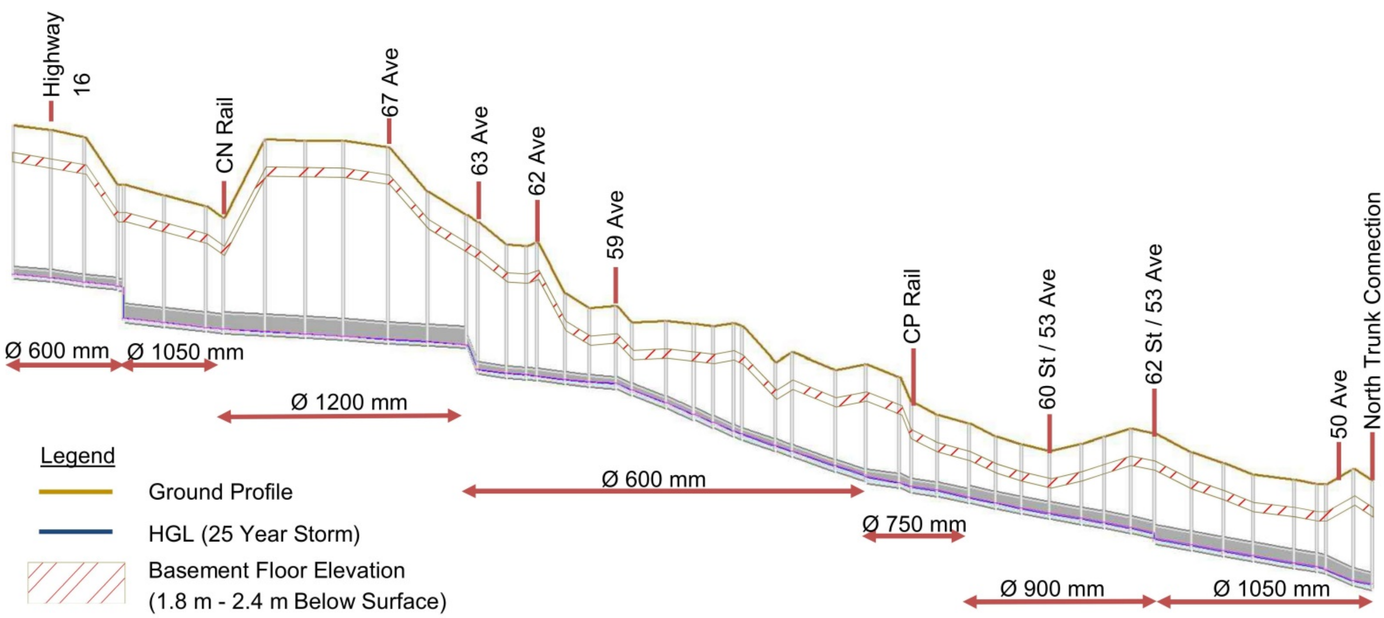


Figure 6.8: West Trunk – 25 Year Storm HGL Profile

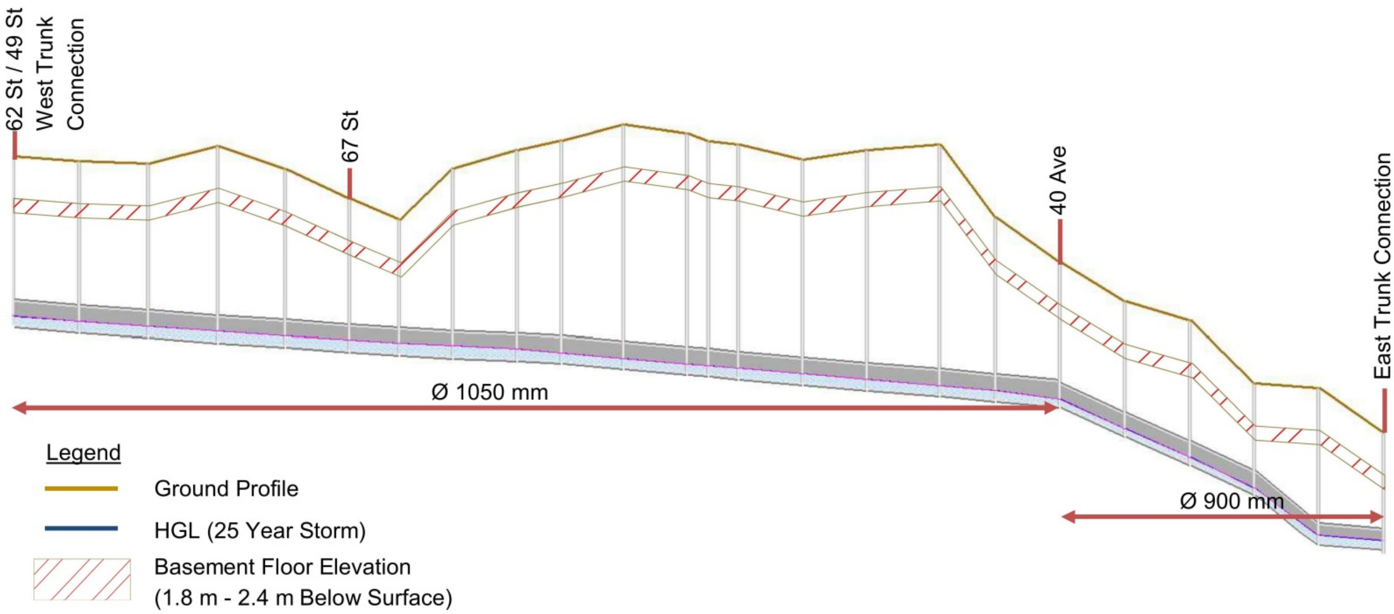


Figure 6.9: North Trunk – 25 Year Storm HGL Profile

6.2 Model Results (50 Year and 100 Year Design Storms)

The City requested an analysis be completed for the 50 year and 100 year design storms. These storms are more severe than the established level of service objective, set as the 25 year design storm. In addition, the extrapolation of the flow and rainfall data to this level of return period provides a greater degree of uncertainty.

The performance of the existing sanitary sewer system for the 50 year and 100 year design storms is shown in Appendix A – Figures A7 and A8, respectively. As expected, the area of possible and probable basement flooding is much larger compared to the 25 year design storm.

6.3 Present Day (2015) - Proposed Upgrades

6.3.1 Overview of Proposed Upgrades

Considerations of sanitary sewer upgrades within an existing system typically abide by the following concepts:

- Flow re-routing – changing the path of a certain amount of flow by re-routing gravity flow or by pumping to a different area of the sanitary sewer system where capacity is available
- Conveyance upgrades – twinning existing sanitary sewer or new trunk sewers
- Inline Storage – Inline storage (commonly large diameter pipe) is located a nominal height above the obvert of the sanitary sewer pipe so that it only fills when the pipe surcharges during a wet weather flow. Inline storage is placed above the sanitary sewer and fills by the rising HGL. It subsequently empties by gravity into the sanitary sewer as the wet weather flow recedes and sewer capacity becomes available. Evaluation of the suitability of inline storage requires careful consideration that the HGL is maintained below basements, as the HGL is allowed surcharge above the pipe in order to fill the tank. An important advantage of inline storage is that the storage tank drains by gravity, meaning that it is maintained empty and the full storage volume is available when a wet weather flow event occurs.

- Offline Storage – Offline storage is implemented where the sanitary sewer system does not allow for the HGL to rise to a sufficient height above the sewer to permit inline storage. In this case, a pipe placed a nominal height above the sanitary sewer and fills the offline storage tank, which is placed below the invert of the sanitary sewer. It then requires pumping to empty the tank. It is important that the offline storage tank is maintained empty such that the full storage volume is available when a wet weather flow event occurs.
- Lift stations and forcemains – In locations where gravity flow to a downstream sewer is not possible, a lift station and forcemain may be used to convey the flow to a different location in the sanitary sewer system where capacity is available. There are two main advantages of this concept. First, the upgrades are not dependent on maintaining gravity flow, meaning the flow may be conveyed across sewersheds. Secondly, because the flow is pumped, the forcemain may be shallower and smaller than a gravity main.

While evaluating the inline storage tank options, it is important to recognize the unique considerations that a storage option presents. As mentioned earlier, the Level of Service criteria and wet weather flow modelling should be considered as interim at the present time and subject to further evaluation and refinement as future years of flow monitoring and wet weather flow calibration is completed. An inline storage tank is relatively more dependent on the design rainfall event and wet weather flow model simulations than a conveyance upgrade. A storage tank requires meticulous modelling to determine its storage requirement, which is a finite volume, while a conveyance upgrade is more independent of the wet weather flow response. As such, there is a certain degree of uncertainty at the present time for the consideration of a storage tank solution.

Secondly, the City's sanitary sewer system is predominately gravity sewer and presently does not have any storage tanks. There is some amount of familiarity to be gained by city technical, operations, and management staff on the implementation of storage tanks before this may be considered as a viable option.

A lift station and forcemain was not strongly considered as an upgrade option. The sanitary sewer system generally follows the overall topography of the city, such that there is mostly adequate grade to drain northeast.

It has been previously mentioned that the City's sanitary sewer system contains a number of flow-splits and interconnections, which makes flow re-routing a viable alternative. Secondly, the City's sanitary sewer system is generally constructed deep and with shallow slope (compared to the ground topography) such that there may be sufficient height between the sanitary sewer and basements to permit inline storage.

The areas of concern in the sanitary sewer system are rather limited to an area approximately bound by 41 Street to the north, 45 Avenue to the east, 33 Street to the south, and 51 Avenue to the west and to a surcharged manhole at 44 Street and 56 Avenue.

In the first area, much of the concern in the sanitary sewer is caused by the surcharged HGL in the 36 Street Trunk between 47 Avenue and 50 Avenue. The HGL backs up the local sewers south of 36 Street, which creates basement flooding on 47 Avenue and 48 Avenue. Secondly, the HGL in the 36 Street Trunk causes flow to escape to the sanitary sewer system north of 36 Street between 47 Avenue and 50 Avenue. This sanitary sewer system, meant to function as a local collection system, is composed of 200 mm diameter sewer and is readily overwhelmed by the excess flow from the 36 Street Trunk.

Upgrades to the 36 Street Trunk were excluded, as increasing the capacity in the section of trunk sewer between 47 Avenue and 50 Avenue would route increased flow further east, which would likely require upgrades to the trunk sewer for its entire downstream length. Secondly, 36 Street is an important arterial roadway so construction would be considerably more complex. Alternatively, an upgrade concept was developed based on a combination of flow re-routing and increased conveyance.

6.3.2 Upgrades – 32 Street and 33 Street

Firstly, the flow in the 250 mm diameter sanitary sewer along 49 Avenue south of 36 Street is able to split and drain to the 200 mm diameter sanitary sewer in two locations (along 32 Street and 33 Street) where it causes the HGL to rise further downstream. A plug in the east sewer in the manhole at both of these flow split locations (MH 427 and MH 428) is recommended to maintain flow in the 250 mm diameter sewer. The reduction in the HGL profile along 48 Avenue is shown below in Figure 6.10.

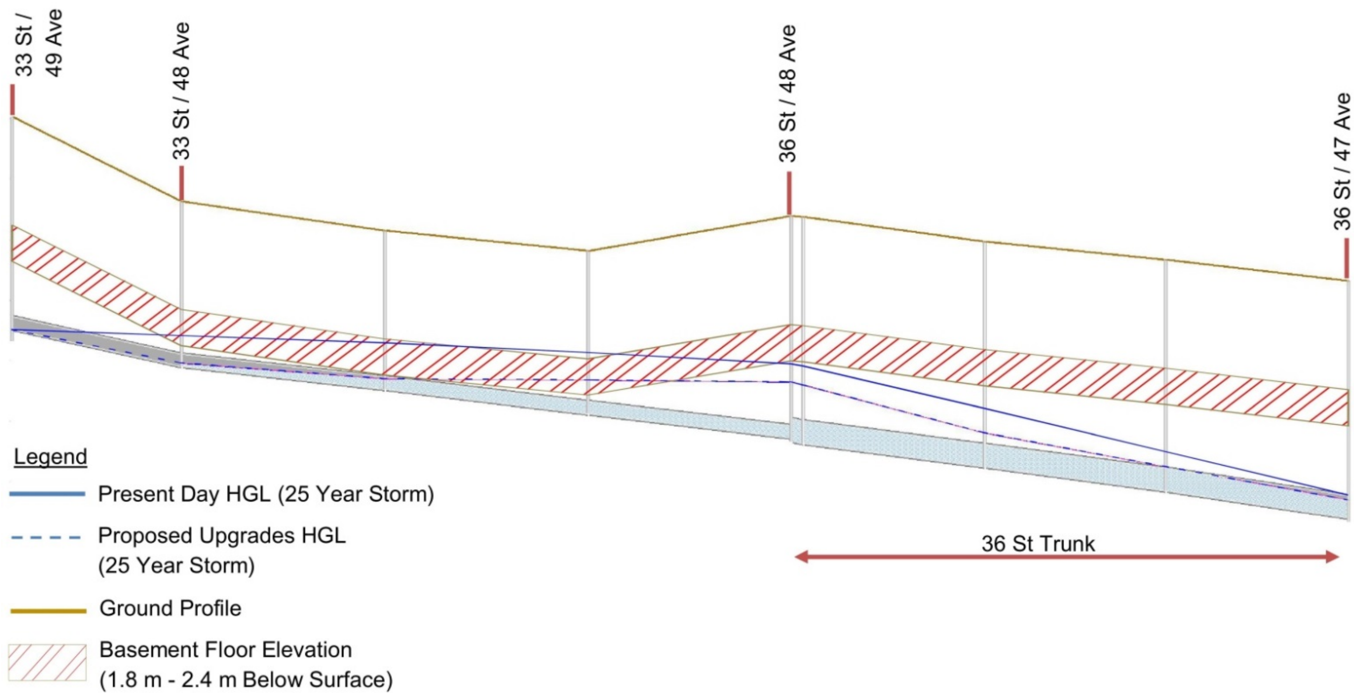


Figure 6.10: Sanitary Sewer between 33 Street/49 Avenue and 36 Street/47 Avenue – HGL Profile

It is noted that there is one manhole along 48 Avenue that shows possible basement flooding. The depth of the HGL in this location is 2.20 m below surface, which is below the expected basement floor elevation (1.8 m below surface) and close to the freeboard allowance. The rise in the HGL in this section of pipe is caused by a backwater effect from the 36 Street Trunk. In order to further alleviate the HGL surcharge, fairly extensive and costly upgrades would be required in the downstream trunk sewer (beyond the upgrades presented in the following section), while benefitting a relatively small area (approximately twenty residences). Although this section of the sewer indicates possible basement flooding, it is suggested that this is deemed acceptable.

6.3.3 Upgrades – 48 Avenue and 49 Avenue

Secondly, a new 300 mm diameter sanitary sewer is required along 48 Avenue from 36 Street to 40 Street and along 40 Street from 48 Avenue to 47 Avenue (total length of 600 m). Additionally, a plug in the north sewer in the manhole at the intersection of 40 Street and 48 Avenue (MH 76) and a plug in the east sewer in the manhole at the intersection of 39 Street and 47 Avenue (MH 79) will ensure the flow remains in the upgraded sewer, which also reduces the flow and mitigates possible basement flooding further downstream.

As this upgrade is routing additional flow to the intersection of 40 Street and 47 Avenue, the excess flow needs to be addressed so that the performance of the downstream sanitary sewer system is not compromised. To achieve this, two options were considered.

Option 1 is for an inline storage tank located in the open space east of 47 Avenue and north of First Baptist Church. The inline storage tank requires a capacity of 440 m^3 and could be achieved with 175 m of 1800 mm diameter pipe.

Option 2 is a conveyance upgrade and requires construction of a 300 mm diameter sewer from the intersection of 40 Street and 47 Avenue, through the open space and campgrounds north of First Baptist Church (length of 500 m). The proposed sanitary sewer connects to an existing manhole at 41 Street and 41 Avenue in the new Larsen Grove neighbourhood (length of 600 m). Note that this option requires a section of existing 250 mm diameter sanitary sewer in the Larsen Grove neighbourhood (length of 200 m) to be replaced with a new 300 mm diameter sanitary sewer. Additionally, the proposed sanitary sewer passes through land presently owned by Husky Energy, such that land acquisition may add a considerable cost to the upgrade.

The reduction in the HGL profile from the intersection of 39 Street and 45 Avenue to the intersection of 36 Street and 49 Avenue is shown below in Figure 6.11.

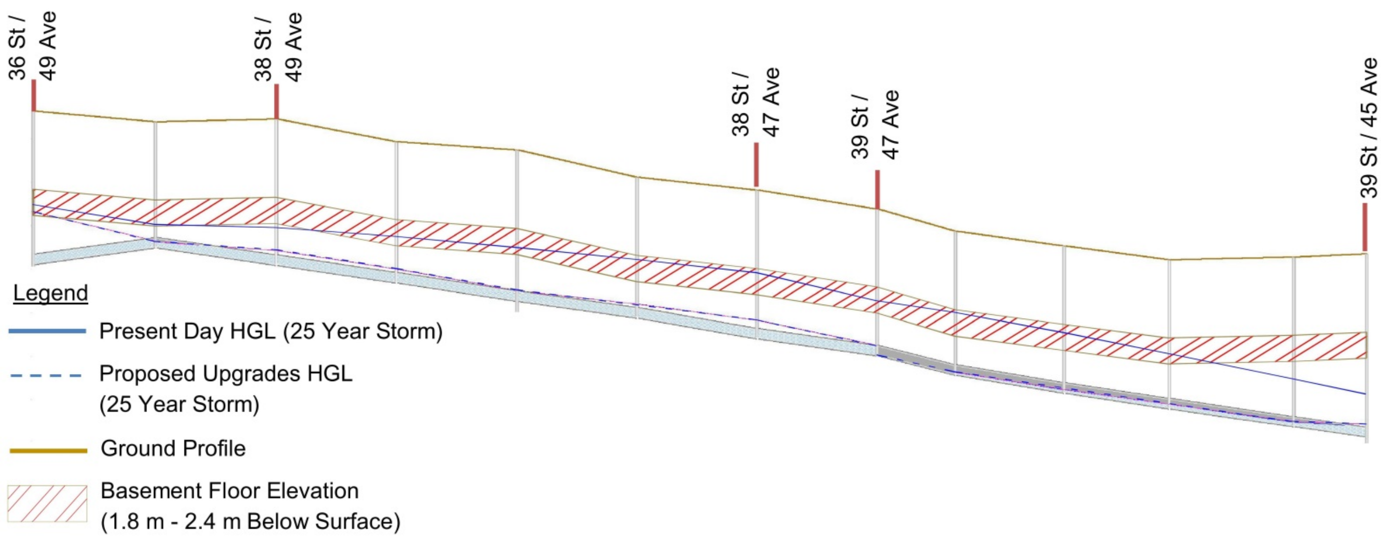


Figure 6.11: Sanitary Sewer from 36 Street/49 Avenue to 39 Street/45 Avenue – HGL Profile

An overall plan of the upgrades described above, along with Option 1 and Option 2, is presented in Figure A9 (Appendix A).

6.3.4 Upgrades – 44 Street and 56 Avenue

The second location of possible basement flooding is located at the intersection of 44 Street and 56 Avenue. At this intersection, there are two 200 mm diameter sanitary sewers that combine into a single 200 mm diameter sanitary sewer that continues east along 44 Street. This section of 200 mm diameter sanitary sewer is undersized until it reaches 54 Avenue (length of 270 m) where it increases in size to 250 mm diameter.

This location is one of the busiest and most important roadways in the city and sanitary sewer upgrades along 44 Street would be significantly disruptive. Alternatively, a flow-split was investigated that involved installation of a new 250 mm diameter sanitary sewer along 56 Avenue from 44 Street to 45 Street (length of 110 m) where there is capacity available in the downstream sanitary sewer. The new sewer at 44 Street is set above the manhole invert and would function as an overflow, such that sufficient pipe slope is available to drain north along 56 Avenue. Figure 6.12 shows the reduction in the height of the HGL along 44 Street and further details are provided in Figure A9 (Appendix A).

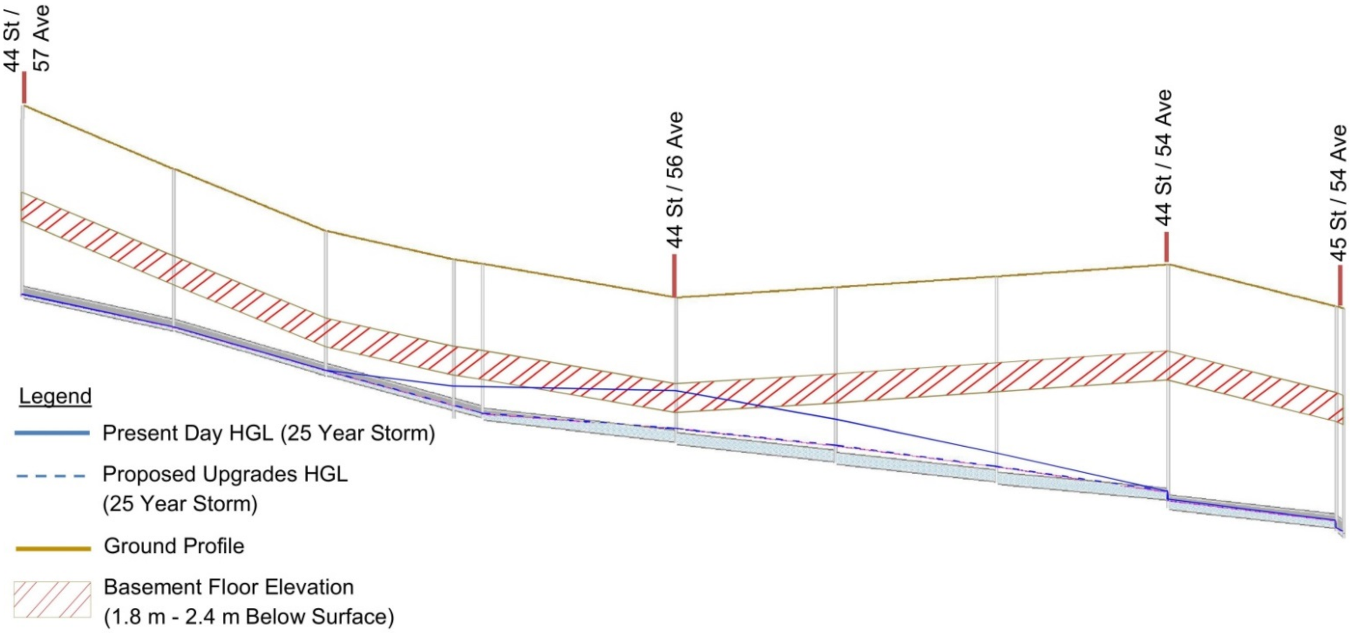


Figure 6.12: Sanitary Sewer along 44 Street between 57 Avenue and 54 Avenue – HGL Profile

The performance of the sanitary sewer system with the implementation of the proposed upgrades is presented in Figure A10 (Appendix A). It is shown that the Level of Service objectives are achieved throughout the city, with minor locations of possible basement flooding shown. These include two locations along the 36 Street Trunk between 46 Avenue and 49 Avenue, where the HGL rises to 2.27 m below surface. There are no service connections along this section of the 36 Street Trunk, such that there should not be any concern.

7. Future Servicing Considerations

The future servicing requirements for the three, five, ten, twenty, and forty year growth horizons were evaluated and included off-site sanitary sewer servicing and upgrades to the existing sanitary sewer system where required to accept additional flow from future growth.

7.1 Methodology

The future growth for the city was assessed for the three, five, ten, twenty, and forty year growth horizons, corresponding to the years 2018, 2020, 2025, 2035, and 2055. The City provided a Master Plans – Future Staging plan (dated June 15, 2015) that delineated the growth areas and land-use for each of the growth horizons. The future development staging plan for residential, commercial, and industrial land-uses and for the 3 year, 5 year, 10 year, and 20 year growth horizons is presented in Figure A11 (Appendix A).

The Present-Day level of residential, commercial, and industrial was established. For residential, a 2015 population of 32,000 was used. For commercial and industrial, the areas of the existing built-up commercial and industrial zones were delineated based on the most-recent aerial image provided by the City. The 2015 commercial and industrial areas were measured to be 203 ha and 688 ha, respectively. Note that these are measured as gross areas and include streets, railway, municipal reserve, and other incidental land-uses within the respective zones.

Using the Present-Day population and commercial and industrial areas, the annual growth rate was determined based on the area increase for each of the growth timelines. The net increase in population was calculated by assuming a population density of 60 p/ha based on net area. It was determined that, on average, the gross area is approximately 180% of the net area for new residential development, based on the land-use summaries provided in the Area Structure Plans for Parkview Estates and Colonial Park and AECOM's experience in neighbourhood master plans in other communities of similar size in Alberta and Saskatchewan. Note that this equates to a population density of 33 p/ha based on gross area. The population growth, area increase for residential, commercial, and industrial land-uses, and corresponding annual growth rate is presented below in Table 7.1.

Table 7.1: Residential, Commercial, and Industrial Growth

	Residential Present Area 960 ha Present Population 32,000 p				Commercial Present Area 203 ha			Industrial Present Area 688 ha		
	Increase in Area	Increase in Population	Total Population	Annual Growth Rate	Increase in Area	Total Area	Annual Growth Rate	Increase in Area	Total Area	Annual Growth Rate
Three Year Growth Horizon (2018)	63 ha	2,079 p	34,079 p	2.1%	18 ha	221 ha	2.9%	44 ha	732 ha	2.1%
Five Year Growth Horizon (2020)	137 ha	4,521 p	38,600 p	3.8%	37 ha	258 ha	4.9%	58 ha	790 ha	2.8%
Ten Year Growth Horizon (2025)	172 ha	5,676 p	44,276 p	3.3%	44 ha	302 ha	4.0%	86 ha	876 ha	2.4%
Twenty Year Growth Horizon (2035)	353 ha	11,649 p	55,925 p	2.8%	95 ha	397 ha	3.4%	180 ha	1056 ha	2.2%
Forty Year Growth Horizon (2055)	727 ha	23,991 p	79,916 p	2.3%	176 ha	573 ha	2.6%	407 ha	1463 ha	1.9%

Once the future development areas were established, the sanitary sewer servicing was evaluated. The off-site sanitary sewer servicing, defined as the main trunk sewers required to service multiple developments, considered the servicing plan already developed in the neighbourhood Area Structure Plans, where available, along with consideration of the existing ground topography and available capacity at the tie-in locations in the downstream sanitary sewer system.

The design flows for the sanitary sewer system were developed in part with the City of Lloydminster Municipal Development Standards as well as the design criteria established in the present study.

For dry weather flow generation, the City of Lloydminster Municipal Development Standards design values for average daily flow for residential, commercial, and industrial land-uses were used because this would be rational used in future design. For residential development, a per capita flow of 360 lpcd was used. A population density of 108 p/ha (net area) was used to develop the residential flow contribution, which equates to 0.45 L/s/ha. This population density allows for some proportion of medium and high density residential development. For commercial and industrial land-uses, an average daily flow of 0.2 L/s/ha (gross area) was used. Although the dry weather flow calibration had found that the actual average daily flow was considerably smaller than the design value (described in Section 4.2), it was deemed appropriate to use the full design value (without reduction) for the future servicing design to allow for potential changes. The residential and commercial diurnal patterns developed from the 2015 flow monitoring data calibration were used.

The sewershed areas were delineated and the RDII flow was modelled with the RTK parameters presented earlier.

The on-site sanitary sewer servicing within the future development areas, which includes the local collection system, was not included in the analysis. Only the off-site sanitary sewer servicing, and the existing sanitary sewer system downstream of the future tie-ins were considered. In addition, the off-site sanitary sewer servicing and existing system upgrades developed for each growth horizon (pipe size and routing) considered the requirements of future growth beyond the specific growth horizon.

7.2 Three Year Growth Horizon (2018)

The three year growth horizon estimates a net increase of 2079 people, requiring 63 ha of residential development spread amongst the Colonial Park, Wallace Field, College Park, Lakeside, and Parkview neighourhoods. The increase in commercial area is 18 ha and is assigned to an area along the north side of 44 Street between 62 Avenue and 75 Avenue. The net increase in industrial area is estimated at 44 ha and is allotted mostly to infill areas north of the future commercial expansion area, alongside 50 Avenue, and north of 44 Street and east 40 Avenue.

The residential areas in the Colonial Park and Parkview neighbourhoods and the commercial and industrial areas may be intercepted into the existing downstream sanitary sewer system with no required upgrades. The residential development in the southern portion of the city that drains to the Southeast Trunk and causes possible basement flooding in the downstream trunk.

To alleviate possible basement flooding, it is proposed to reroute a portion of the flow in the Southeast Trunk to a new trunk, called the 19 Street Trunk. The 19 Street Trunk ranges in size from 600 mm to 750 mm diameter and connects to the Southeast Trunk at the intersection of 19 Street and 47 Avenue. It passes east through future stages of the Wallace Field neighbourhood and connects to a future trunk, identified as the South Trunk, close to 40 Avenue. The South Trunk is 1200 mm diameter and drains north to connect to the existing 1350 mm diameter trunk at 29 Street Close and 43 Avenue. Both the 19 Street Trunk and South Trunk provide service for future growth areas, described in subsequent sections.

The HGL profile for the Southeast Trunk showing the existing flow, the future flow without upgrades, and the future flow with the proposed rerouting is presented below in Figure 7.1.

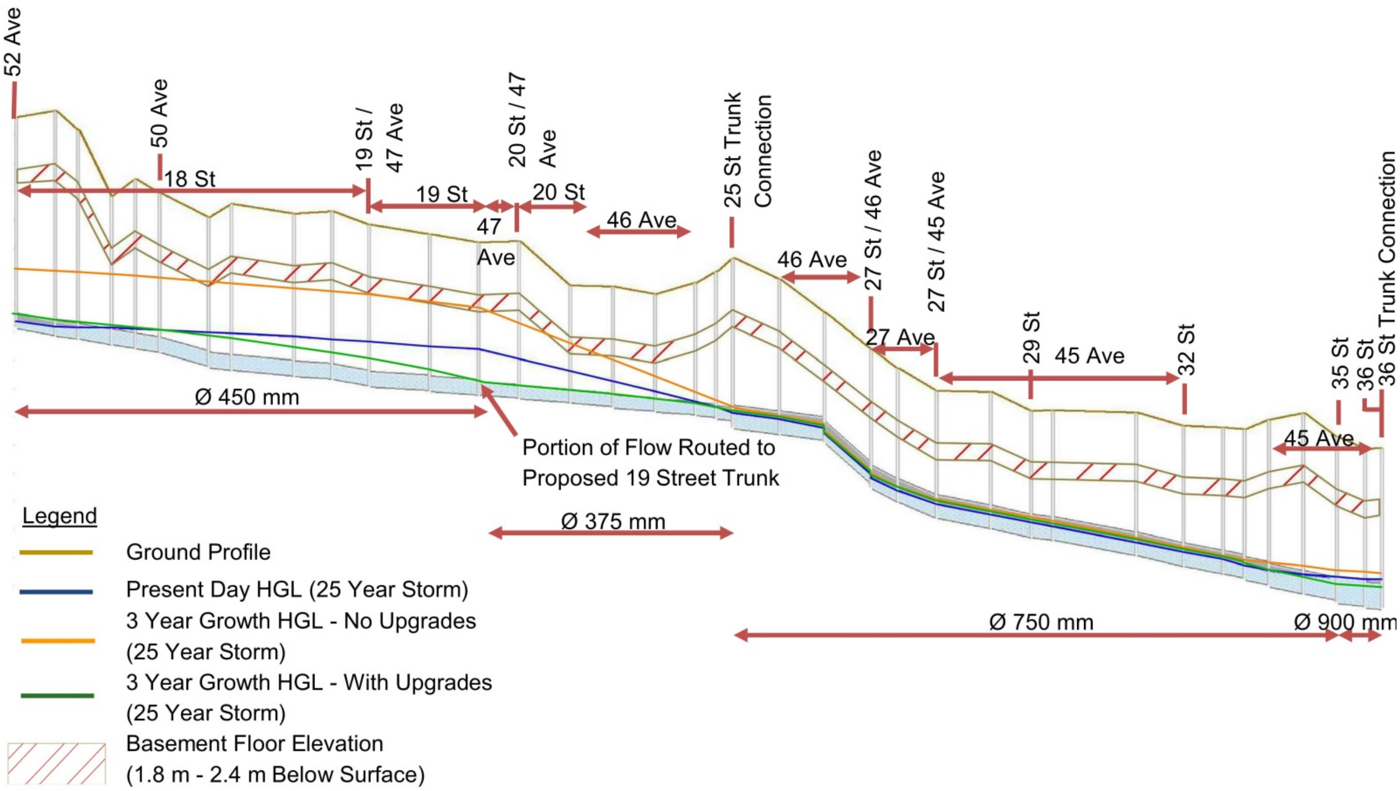


Figure 7.1: Southeast Trunk HGL Profile - Three Year Growth Horizon

The staging plan for the three year growth horizon, on-site and off-site sanitary sewer servicing, and existing sanitary sewer upgrades are presented in Figure A12 (Appendix A).

7.3 Five Year Growth Horizon (2020)

The five year growth horizon estimates an increase of 4,521 people (137 ha), 37 ha of commercial land, and 58 ha of industrial land. The residential development was spread throughout the Colonial Park, Wallace Field, College Park, Lakeside, and Parkview neighbourhoods. The Colonial Park and College Park both reached full build-out. Commercial area was designated south of 44 Street on the western edge of the city and industrial area was allocated in a portion of the Hill Industrial area and an infill area east of 50 Avenue.

The future growth areas may be serviced entirely with on-site sanitary sewer servicing, including a portion of area in the Wallace Field neighbourhood that is serviced by the 19 Street Trunk and South Trunk (presented in the three year growth horizon). The off-site servicing requirements are provided by the 19 Street Trunk and South Trunk.

The additional flow in the College Park and Lakeside neighbourhoods further overwhelm the Southeast Trunk. To relieve the occurrence of possible basement flooding, it is proposed to twin the Southeast Trunk with a 525 mm diameter sewer along 18 Street and 19 Street between 47 Avenue and 49 Avenue. The HGL profile in the Southeast Trunk, showing the existing flow, future flow without upgrades, and future flow with upgrades, is presented below in Figure 7.2.

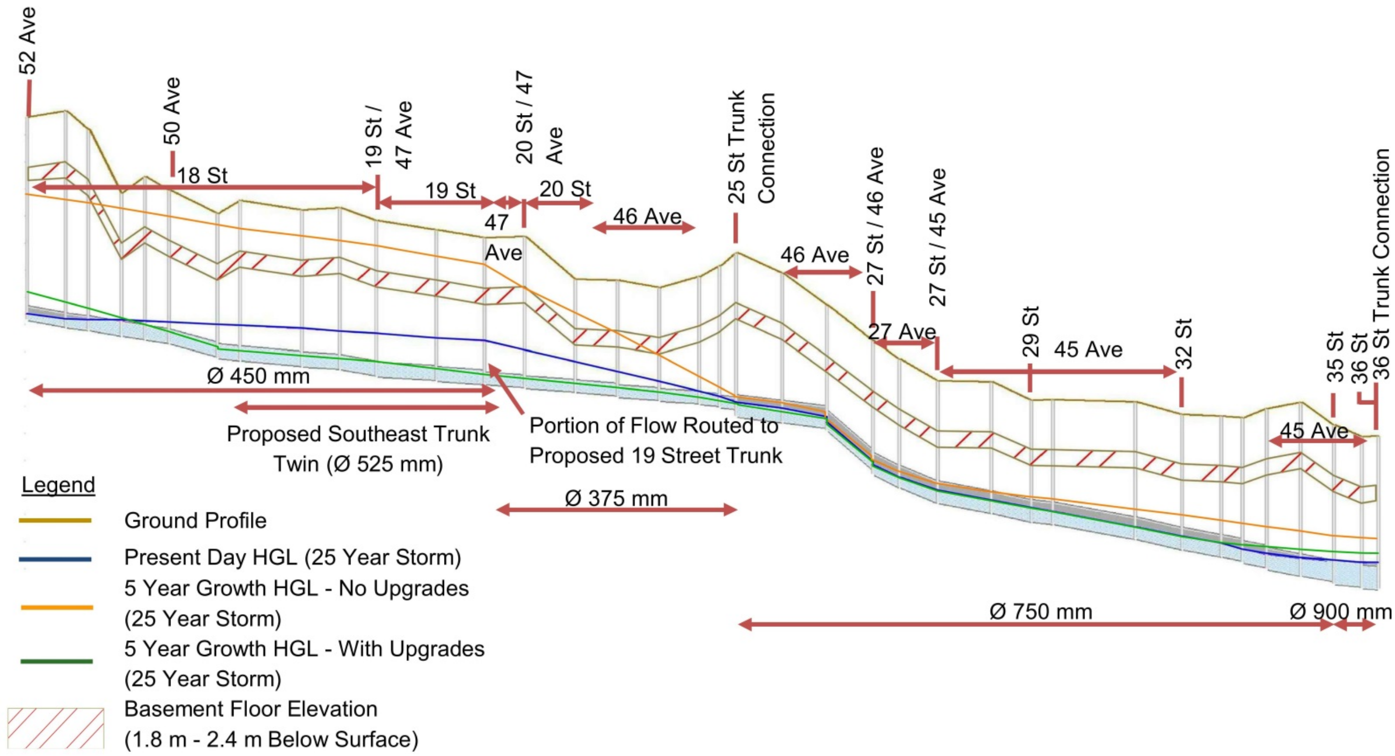


Figure 7.2: Southeast Trunk HGL Profile - Five Year Growth Horizon

The five year staging plan, on-site and off-site sanitary sewer servicing, and existing sanitary sewer upgrades are presented in Figure A13 (Appendix A).

7.4 Ten Year Growth Horizon (2025)

The ten year growth horizon estimates a net increase in population of 5,676 people (137 ha) and is allocated to the Wallace Field, Lakeside, and Parkview neighbourhoods (reaching full build-out) and to an area south of 12 Street between 50 Avenue and 62 Avenue. The increase in commercial area is 44 ha and is allocated to an area along 44 Street on the west edge of the city and along 50 Avenue south of 12 Street. There is 86 ha of industrial area, assigned to the Hill Industrial and Wigfield Industrial areas and infill area along 50 Avenue.

The on-site sanitary sewer servicing may connect directly to the existing sanitary sewer system except for the residential/commercial area south of 12 Street and the commercial area north of 44 Street, which requires off-site sanitary sewer servicing. The area south of 12 Street requires extension of the South Trunk (ranging in size from 900 mm to 1200 mm diameter). The area north of 44 Street requires the construction of a new trunk, called the CN Rail Trunk, and ranges in size from 375 mm to 450 mm diameter. Note that the proposed CN Rail Trunk follows a similar alignment as the 75 Avenue Trunk Extension project.

In addition to on-site and off-site sanitary sewer servicing, the future flow overwhelms the capacity of the existing East Trunk. To alleviate the flow in the existing East Trunk, it is proposed to twin the East Trunk with a new trunk sewer, identified as the East Trunk Twin. The East Trunk Twin is 1200 mm diameter and would connect to the existing 1200 mm diameter trunk at 36 Street and 40 Avenue. The trunk would travel north along 40 Avenue to north of the CN Rail, where it would travel northeast and then north adjacent to the existing East Trunk to the WWTP.

Note that the existing East Trunk would continue to service its present collection area but would be relieved of the flow in the 19 Street Trunk and the South Trunk (presented in the 3 year and 5 year growth horizons). This provides capacity in the existing East Trunk that would be available for future development in the Wigfield Industrial area.

The ten year staging plan, on-site and off-site sanitary sewer servicing, and existing system upgrades are presented in Figure A14 (Appendix A).

7.5 Twenty Year Growth Horizon (2035)

The twenty year growth horizon estimates a net increase of 11,649 residents (353 ha), 95 ha of commercial land, and 180 ha of industrial land.

The residential area is assigned to south of 12 Street, east of Larsen Grove, west of Parkview, and along 40 Avenue south of 67 Street. The commercial area is distributed to south of 12 Street, north of 44 Street, and along 50 Avenue north of 67 Street. The industrial area is allocated to the Wigfield Industrial area (now fully occupied), a parcel of land north of the CN Rail and east of 40 Avenue, and a parcel of land west of 75 Avenue and south of 52 Street.

The future development requires extension of the South Trunk (750 mm diameter), CN Rail Trunk (300 mm diameter), and a new trunk called the Highway 16 Trunk, south of 44 Street on the western edge of the city (450 mm diameter).

The twenty year staging plan and on-site and off-site sanitary sewer servicing are presented in Figure A15 (Appendix A).

7.6 Forty Year Growth Horizon (2055)

The forty year growth horizon estimates a net increase in population of 23,991 people (727 ha), 176 ha of commercial area, and 407 ha of industrial area. The residential area is allocated south of 12 Street, east of 40 Avenue, and north of 67 Street. The commercial area is assigned to south of 12 Street and along 50 Avenue north of 67 Street. The industrial area is distributed to Hill Industrial area (now fully occupied), south of 44 Street on the western edge of the city, and west of 75 Avenue, and north of 67 Street.

The forty year growth horizon may be serviced solely by on-site and off-site sanitary sewer, with no upgrades required to the downstream existing sanitary sewer system. The proposed South Trunk (525 mm diameter), Highway 16 Trunk (375 mm and 450 mm diameter) would require extension to the limits of the 40 year development areas. In addition, a new trunk (identified as the Northwest Trunk) would be required to extend from the western edges of the future industrial area (along 52 Street, 1.6 km west of 75 Avenue, along 75 Avenue, through the residential development north of 67 Street, and end at the WWTP. The Northwest Trunk ranges in size from 375 mm diameter to 1050 mm diameter.

The forty year staging plan and on-site and off-site sanitary sewer servicing are presented in Figure A16 (Appendix A).

The HGL profiles in each of the existing trunks for the present-day flow and the forty year flow are presented in Figures 7.3 to 7.11 below.

The HGL in the 52 Street Trunk is shown to rise slightly in the 40 year development scenario (Figure 7.3), which is caused by the infill development areas directly connecting to the existing sanitary sewer system. The HGL is maintained below the basement floor elevation profile.

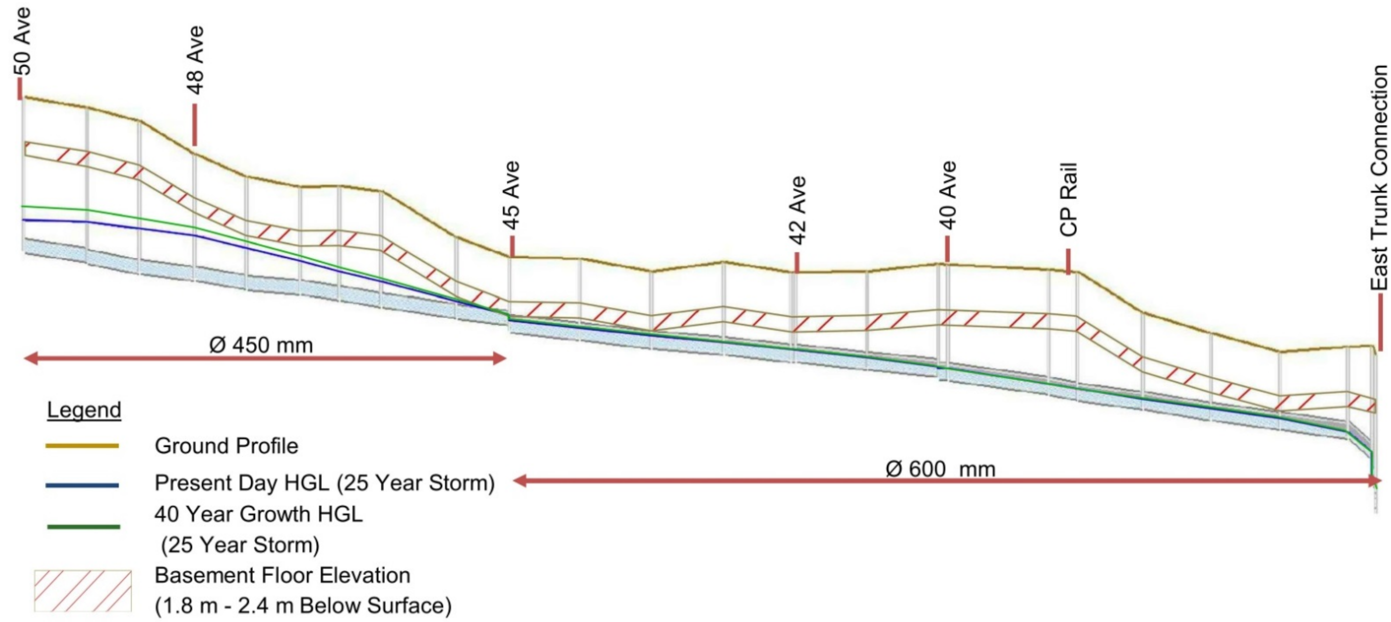


Figure 7.3: 52 Street Trunk HGL Profile – Forty Year Growth Horizon

The 47 Street Trunk HGL shows a slight reduction in surcharge in the forty year growth horizon (Figure 7.4) and is lowered to below the basement floor elevation profile. The reduction in surcharge is caused by the conveyance upgrade introduced in the Present-Day scenario.

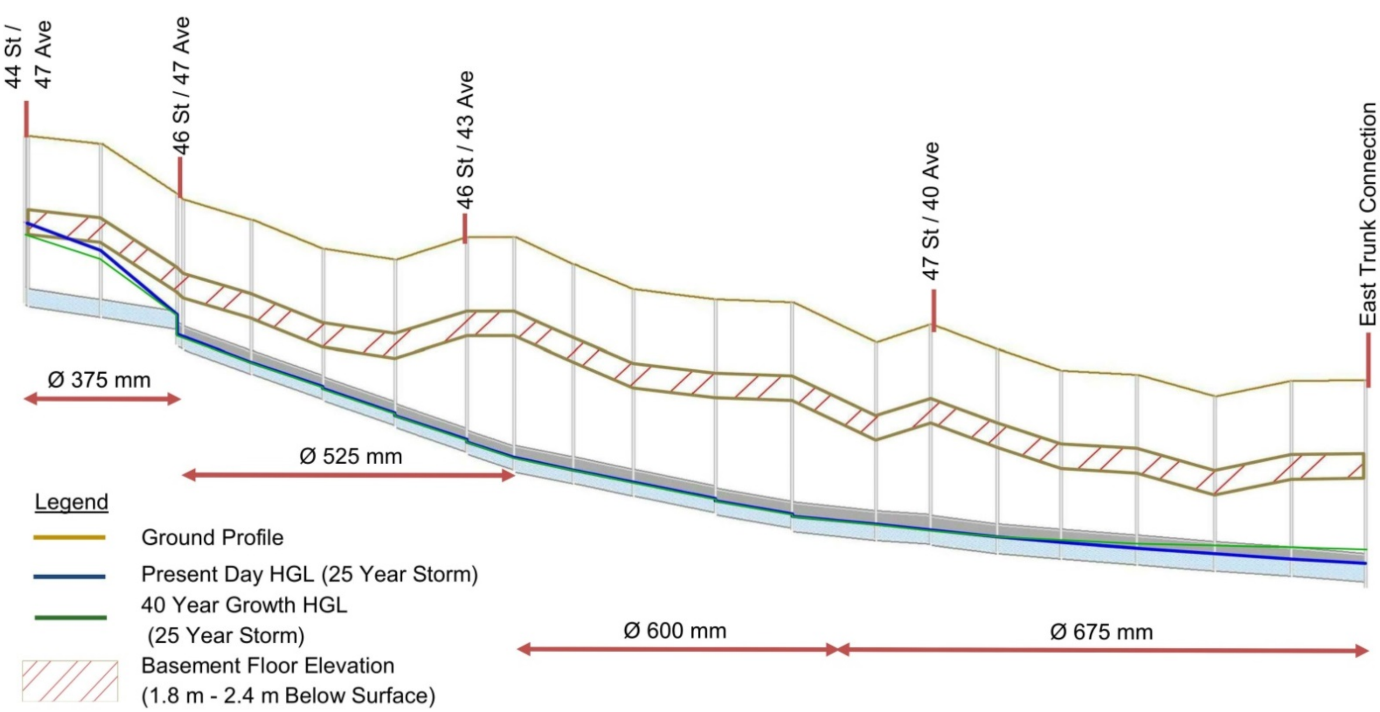


Figure 7.4: 47 Street Trunk HGL Profile – Forty Year Growth Horizon

The 36 Street Trunk HGL profile, shown in Figure 7.5, shows a slight reduction in the HGL profile. This is caused by the conveyance upgrade in the Present-Day scenario. There are two isolated locations where the HGL rises to a small height above the basement floor profile; however, as discussed previously, there are no basement connections in these locations.

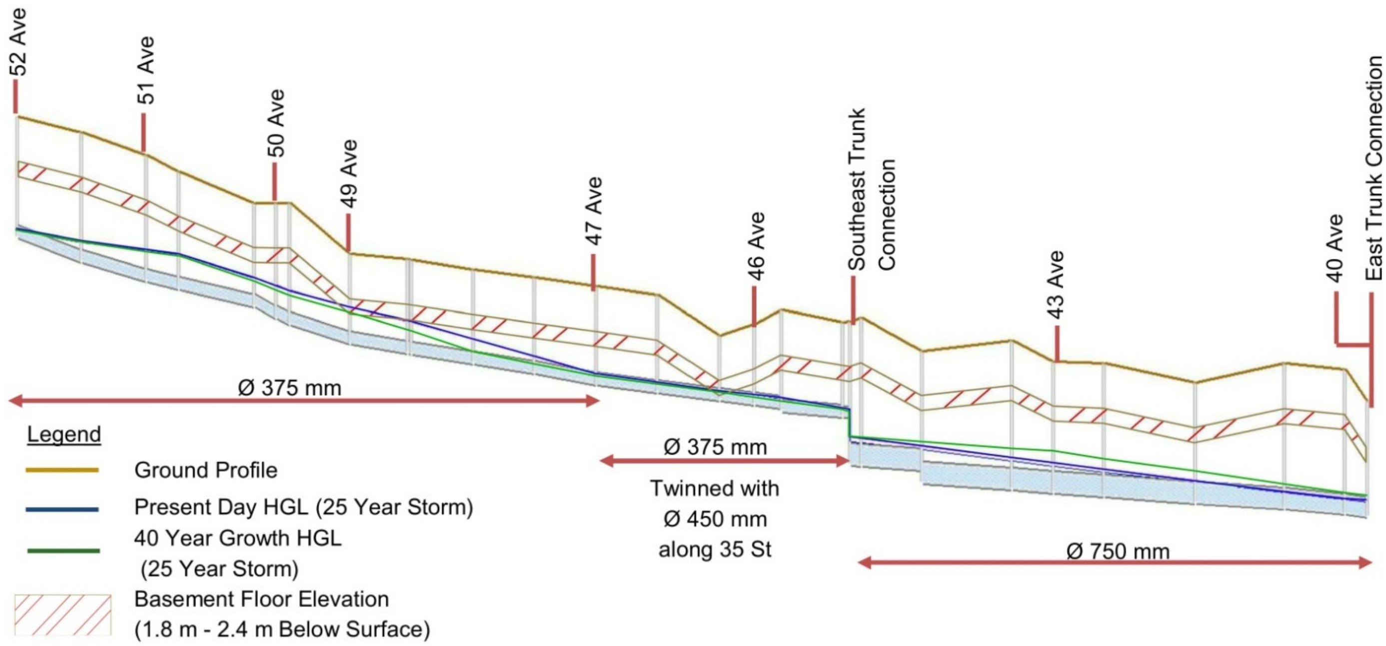


Figure 7.5: 36 Street Trunk HGL Profile – Forty Year Growth Horizon

The 25 Street Trunk HGL profile (Figure 7.6) shows a rise in the forty year HGL at its upstream end, which is caused by the on-site sanitary sewer in the development areas directly connecting to the existing sanitary sewer system.

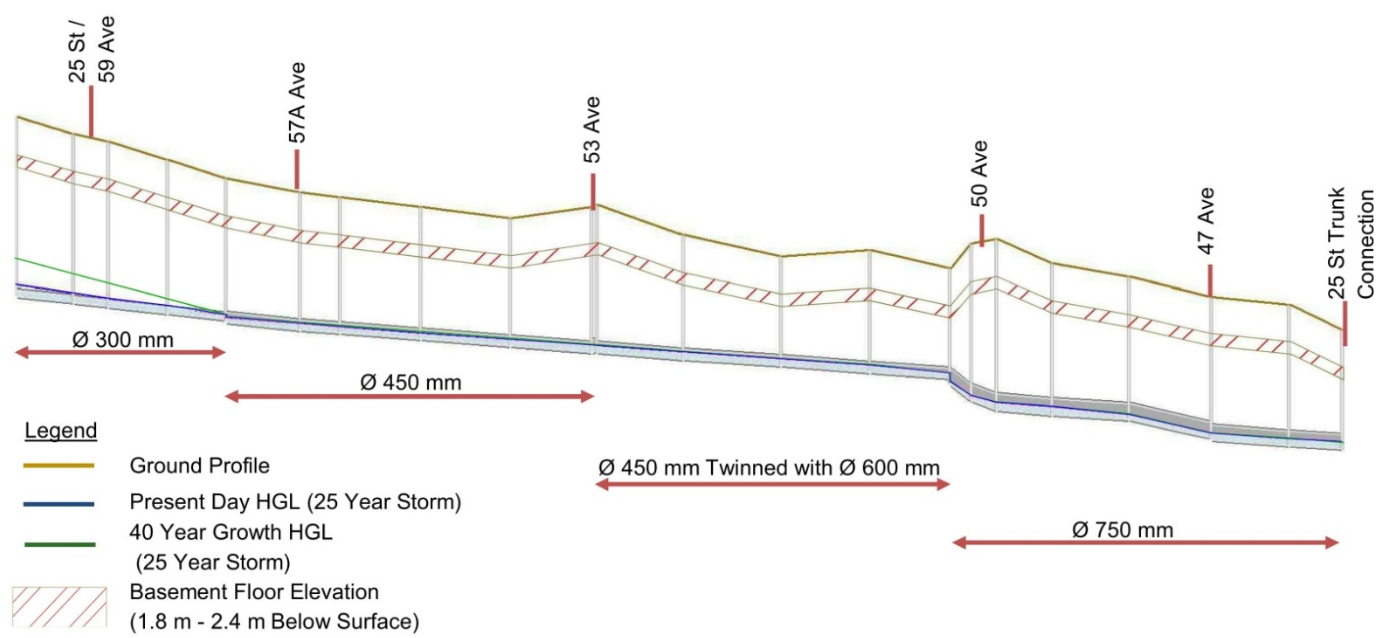


Figure 7.6: 25 Street Trunk HGL Profile – Forty Year Growth Horizon

The Southeast Trunk HGL profile, shown in Figure 7.7, shows that the forty year HGL profile is higher than the present day HGL at the upstream end of the trunk, caused by the direct connection of the development areas. The forty year HGL is lower than the present-day HGL further downstream, due to the proposed upgrades presented in the three year and five year growth horizons.

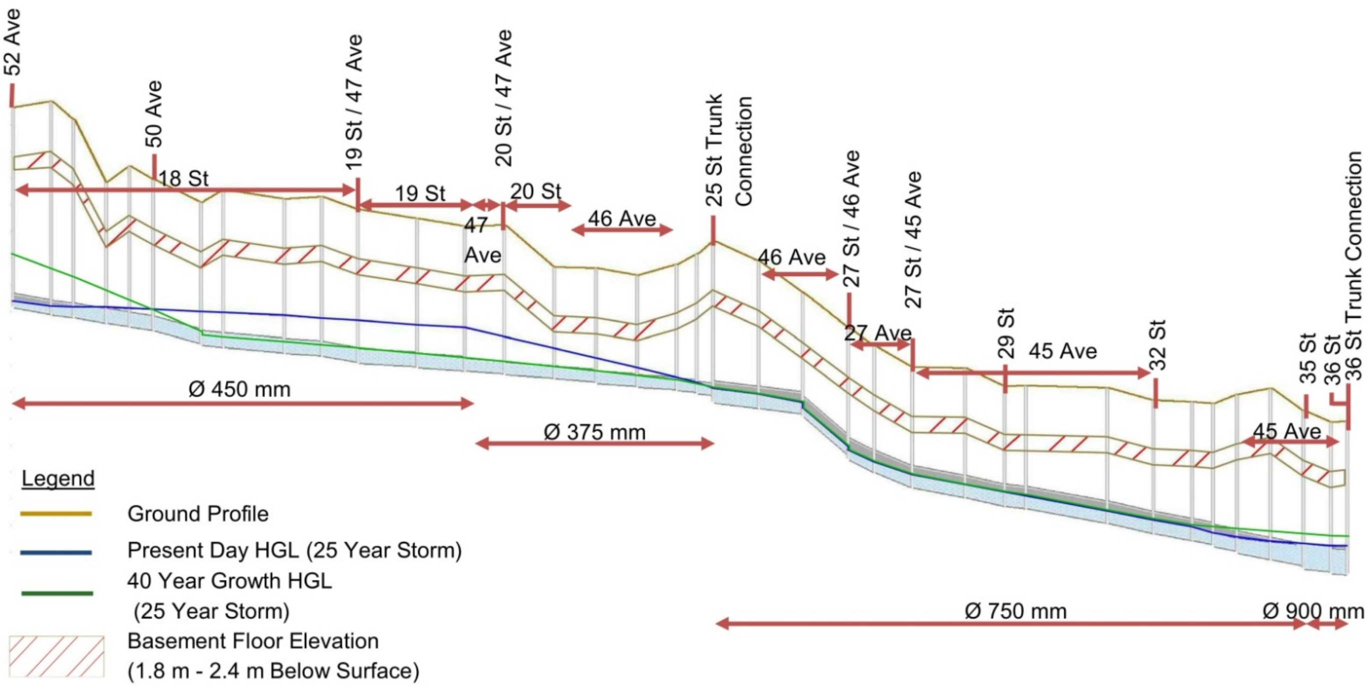


Figure 7.7: Southeast Trunk HGL Profile – Forty Year Growth Horizon

The East Trunk HGL profile (Figure 7.8) is slightly higher in the forty year growth horizon. Although the East Trunk is proposed to be twinned in the ten year growth horizon, future development area (such as the Wigfield Industrial area) is directly connected to existing East Trunk.

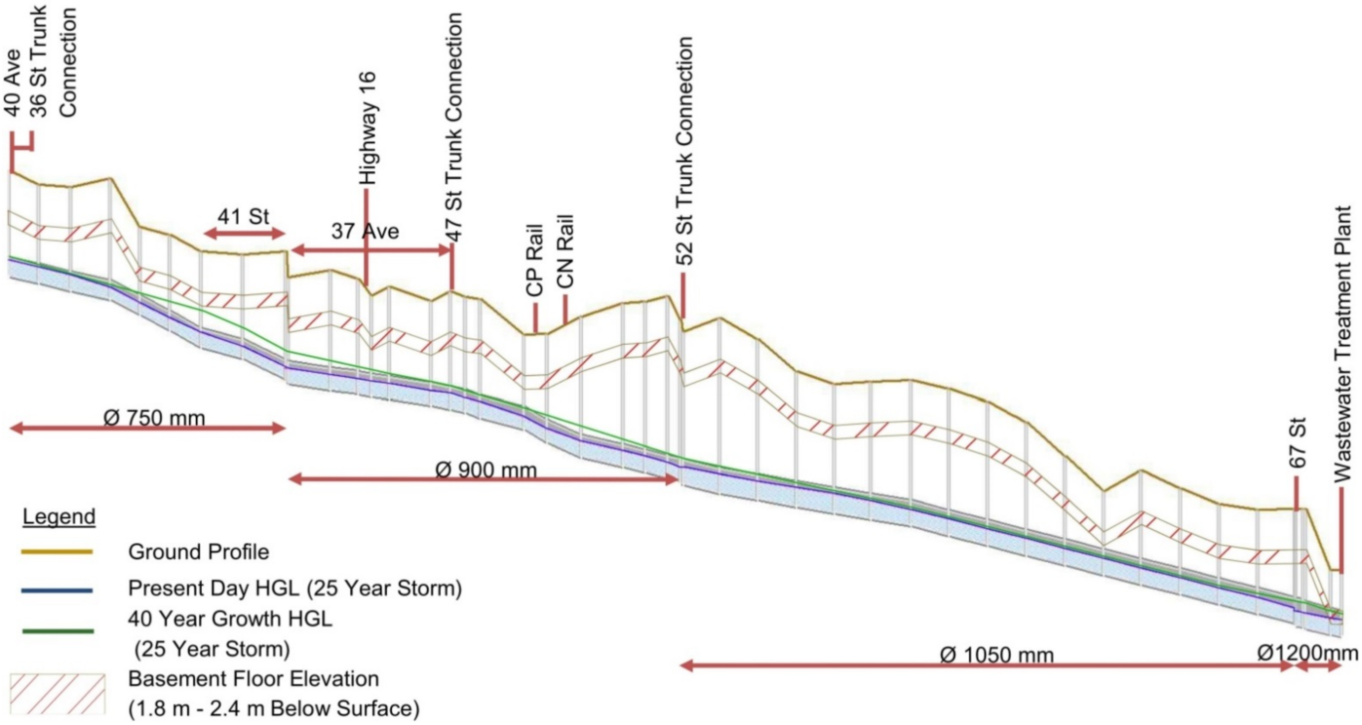


Figure 7.8: East Trunk HGL Profile – Forty Year Growth Horizon

The 62 Street Trunk, shown in Figure 7.9, shows a higher HGL in the forty year growth horizon, as it services a large portion of future development area. There appears to be excess flow capacity that is not fully-used in the upstream sections of the trunk. However, the downstream portion of the 62 Street Trunk and the North Trunk have both reached full capacity and cannot service additional land.

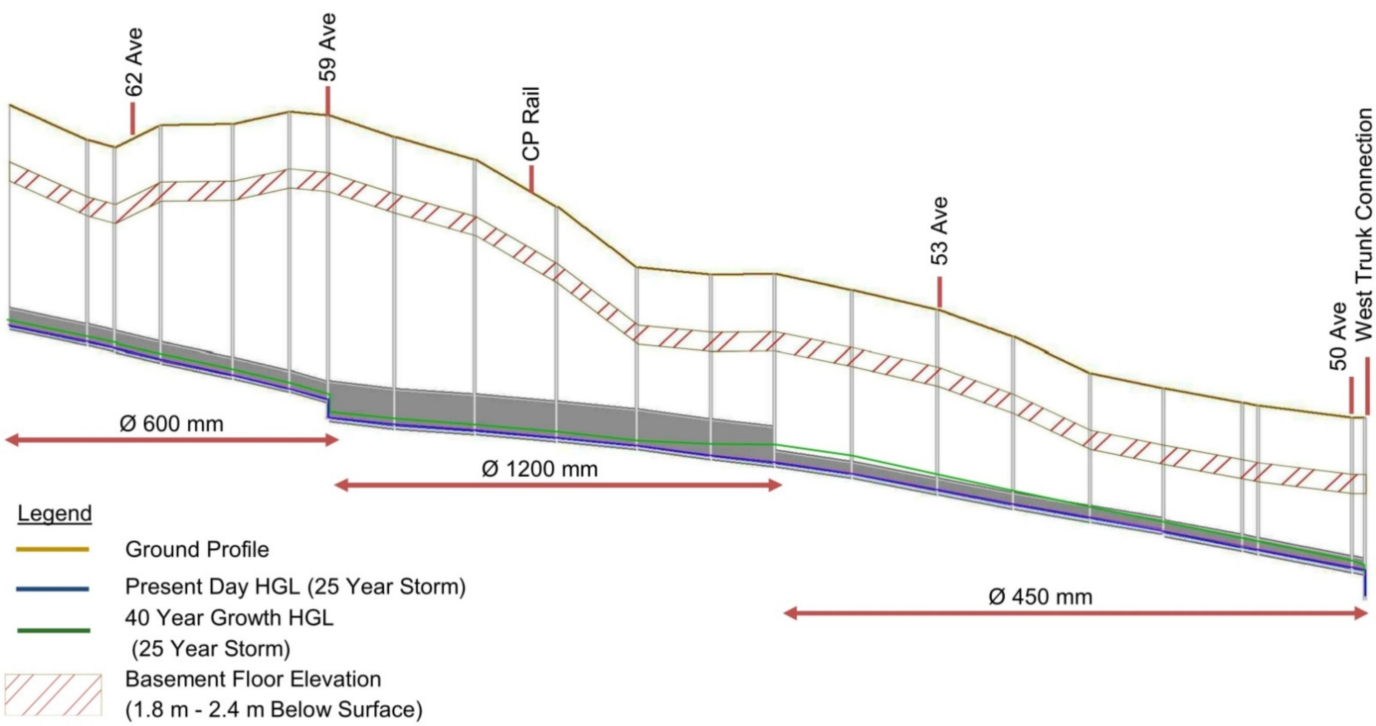


Figure 7.9: 62 Street Trunk HGL Profile – Forty Year Growth Horizon

The West Trunk HGL (Figure 7.10) shows a high amount of surcharge between 63 Avenue and 59 Avenue and indicates possible basement flooding. This is not concerning, as the trunk in this location passes through open field without direct service connections. Additionally, this section of the West Trunk passes adjacent to a drainage channel, such that the ground elevation is lower than the surrounding area.

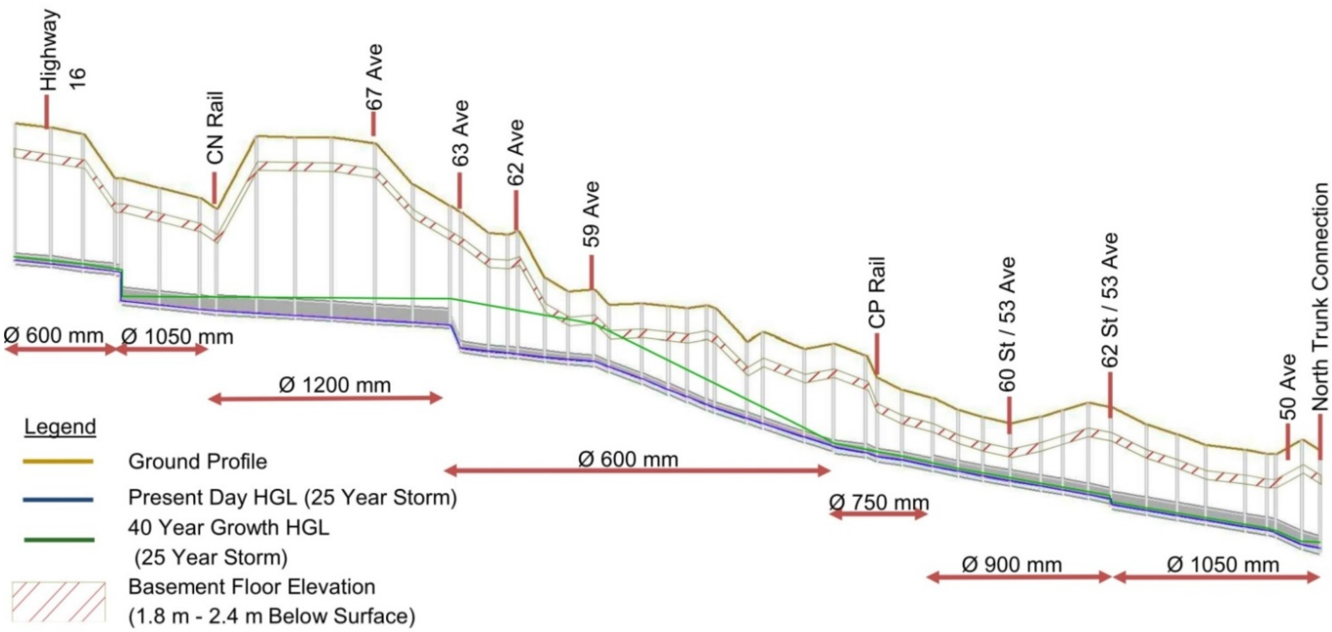


Figure 7.10: West Trunk HGL Profile – Forty Year Growth Horizon

The North Trunk HGL (Figure 7.11) is higher in the forty year growth horizon than the present-day flow, as a considerable development area is directly serviced by the North Trunk and its upstream trunks (62 Street Trunk and West Trunk). The North Trunk has reached its full capacity in the forty year growth horizon.

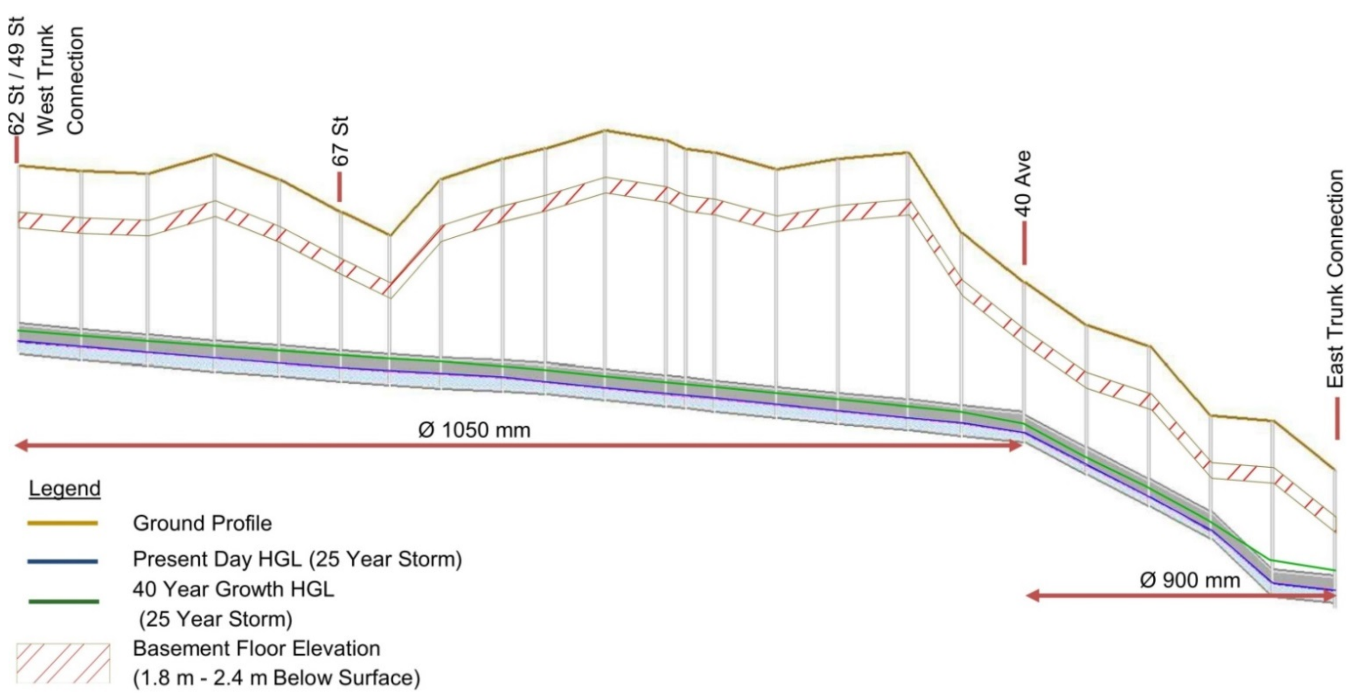


Figure 7.11: North Trunk HGL Profile – Forty Year Growth Horizon

8. Construction Capital Cost Plan

A construction capital cost plan was developed for the upgrades presented in the preceding two sections for the present day, three year, five year, ten year, twenty year, and forty year growth horizons. AECOM prepared the cost estimates by applying the Association for the Advancement of Cost Engineering (AACE) International Recommended Practice No. 18R-97 “Cost Estimate Classification System – As Applied in Engineering, Procurement and Construction for Process Industries” Cost Estimate Classification Matrix. The estimates were based on a Class 5 Estimate Class as described below in Table 8.1.

Table 8.1: Construction Capital Cost Plan

	Primary Characteristic	**Secondary Characteristics**		
Estimate Class	**Maturity Level of Project Definition Deliverables** Expressed as % of complete definition	**End Usage** Typical purpose of estimate	**Methodology** Typical estimating method	**Expected Accuracy Range** Typical variation in low and high ranges (1)
Class 5	0% to 2%	Functional area or concept screening	SF or m^2 factoring, parametric models, judgment, or analogy	Low: -20% to -30% High: +30% to +50%
Class 4	1% to 15%	Schematic design or concept study	Parametric models, assembly driven models	Low: -10% to -20% High: +20% to +30%
Class 3	10% to 40%	Design development, budget authorization, feasibility	Semi-detailed unit costs with assembly level line items	Low: -5% to -15% High: +10% to +20%
Class 2	30% to 75%	Control or bid/tender, semi-detailed	Detailed unit cost with forced detailed take-off	Low: -5% to -10% High: +5% to +15%
Class 1	65% to 100%	Check estimate or pre-bid/tender, change order	Detailed unit cost with detailed take-off	Low: -3% to -5% High: +3% to +10%

(1) *The state of construction complexity and availability of applicable reference cost data affect the range markedly. The +/- value represents typical percentage variation of actual cost from the cost estimate after application of contingency (typically at a 50% level of confidence) for given scope.*

Adapted from AACE International Recommended Practice No. 56R-08 “Cost Estimate Classification System – As Applied For the Building and General Construction Industries” (2012)

Any estimates or opinions regarding probable construction costs provided by AECOM represent AECOM's professional judgement in light of its experience and the knowledge and information available to it at the time of preparation. Since AECOM has no control over market or economic conditions, prices for construction labour, equipment or materials or bidding procedures, AECOM, its directors, officers and employees are not able to, nor do they, make any representations, warranties or guarantees whatsoever, whether express or implied, with respect to such estimates or opinions, or their variance from actual construction costs, and accept no responsibility for any loss or damage arising therefrom or in any way related thereto. Persons relying on such estimates or opinions do so at their own risk.

A contingency of 30% was included in the Class 5 cost estimates. An annual inflation rate of 2.0% was assumed for future construction. The reader is referred to Appendix D for quantity breakdowns and cost estimate sheets.

The cost estimates were developed based on historical construction costs provided by the City for the period of 2011 to 2013 and AECOM’s past experience in the installation of sanitary sewer and storage tanks in other municipalities.

As mentioned previously, consideration was made for off-site sanitary sewer servicing and upgrades to the existing sanitary sewer system. The on-site servicing, which includes the local collection system, was not included in the analysis, and therefore is excluded from the cost estimates. The estimates do not include project administration, engineering or land acquisition costs.

The cost estimates included a 20% allowance for general requirements, which includes contract insurance and bonding, material testing, mobilization/demobilization, and a 20% allowance for appurtenances and incidentals, which include manholes, connections to existing sanitary sewer, utility crossings, and unforeseen and subsurface ground conditions. Consideration was made in the cost estimates for sanitary sewer installation through existing streets, along rail right-of-way, and open field, as construction along existing streets and rail right-of-way would require more stringent backfill and trench requirements and increased handling of excavation and backfill material.

8.1 Present-Day (2015) – Sanitary Sewer Upgrades

As described in Section 6, there were two locations identified that required sanitary sewer upgrades to mitigate basement flooding.

For the first location, approximately abound by 41 Street to the north, 45 Avenue to the east, 33 Street to the south, and 51 Avenue to the west, two options were developed. The two options both required installation of 300 mm diameter sanitary sewer to reroute convey flow to 47 Avenue. The cost of the work common to the two options presented below is estimated at approximately $2,120,000.

Option 1 was an inline storage tank solution, which temporarily stored the peak wet weather flow and slowly drained into the existing downstream sanitary sewer system as the wet weather flow receded and flow capacity became available. This option is estimated to have a capital cost of $1,840,000. The total cost for implementation of this option, which includes the 300 mm diameter sanitary sewer described above, is $3,960,000.

Option 2 was a conveyance option that re-routed the flow to the existing sanitary sewer from 47 Avenue to the intersection of 41 Street and 41 Avenue in the Larsen Grove neighbourhood. The capital cost of Option 2 is estimated at $1,690,000. Including the 300 mm diameter sanitary sewer described above, the total cost for Option 2 is $3,810,000.

The cost estimate for the conveyance upgrade (Option 2) is approximately $150,000 less than the storage option (Option 1). However, the conveyance option would have additional cost for land acquisition, as the proposed sanitary sewer passes through land presently owned by Husky Energy.

The City's sanitary sewer system is predominantly gravity sewer and the introduction of a storage facility would present a unique component to the operation and maintenance of the system. It is expected that there would be some amount of familiarity to be gained by the City's technical, operations, and management staff on the implementation of storage facilities before this may be considered as a viable option.

Considering this, the conveyance option is recommended as the preferred option.

The upgrade for 44 Street and 56 Avenue has an estimated cost of $370,000.

Thus, the total cost of present-day upgrades is estimated at $4,180,000.

8.2 Three Year Growth Horizon (2018) – Sanitary Sewer Upgrades

For the three year growth horizon, sanitary sewer upgrades were proposed to alleviate the surcharge in the Southeast Trunk. The upgrades included the 19 Street Trunk and the South Trunk; both of which would be required at some point in the future as development expands. Thus, these two trunks may be considered as partly sanitary sewer upgrades required for the three year growth horizon as well as partly off-site sanitary sewer servicing required for future growth beyond the three year growth horizon.

The 19 Street Trunk and South Trunk should be considered as a single project in order to reroute flow from the Southeast Trunk. However, the two trunks are presented separately, as each provides on-site and off-site sanitary sewer service to different development areas.

The capital cost for the 19 Street Trunk is estimated as $1,440,000 at present value, which equates to a future value of $1,530,000. The section of South Trunk required in the three year growth horizon has a present value of $5,530,000 and a future value of $5,870,000.

The total capital cost for the three year growth horizon is $6,970,000 (present value). The future value is estimated at $7,400,000.

8.3 Five Year Growth Horizon (2020) –Sanitary Sewer Upgrades

The additional flow generated in the five year growth horizon further stresses the hydraulic capacity in the Southeast Trunk. To alleviate surcharge, a section of existing sewer is proposed to be twinned with a 525 mm diameter sanitary sewer. The present value of this upgrade is estimated at $3,500,000 and has a future value of $3,860,000.

The on-site sanitary sewer servicing in the development in the Colonial Park and Wallace Field neighbourhoods is serviced by the 19 Street Trunk and South Trunk that was proposed in the three year growth horizon.

8.4 Ten Year Growth Horizon (2025) – Off-Site Sanitary Sewer Servicing and Upgrades

The ten year growth horizon identified both off-site sanitary sewer servicing and existing system upgrade requirements.

For off-site sanitary sewer servicing, there are two components. The first component requires extension of the South Trunk (initially presented in the three year growth horizon) in order to service development south of 12 Street. The capital cost estimate for the South Trunk extension is $19,250,000 (present value), or $23,470,000 (future value).

The second component of the off-site sanitary sewer servicing is the implementation of the CN Rail Trunk, which connects to the existing sanitary sewer at 75 Avenue, south of the CN Rail. The portion of the CN Rail Trunk required in the ten year growth horizon is estimated to have a capital cost of $4,230,000 (present value), or $5,150,000 (future value).

For the existing system upgrade requirements, it is proposed to twin the East Trunk from 36 Avenue to the WWTP. This is estimated to have a present cost of $30,470,000 and a future cost of $37,150,000.

8.5 Twenty Year Growth Horizon (2035) – Off-Site Sanitary Sewer Servicing

The twenty year growth horizon identified only off-site sanitary sewer servicing requirements.

The South Trunk is extended further west to service additional development south of 12 Street. The capital cost estimate for the extension is $1,770,000 (present value) and $2,630,000 (future value).

The CN Rail Trunk presented in the ten year growth horizon extends further west to service additional development. This is estimated to have a capital cost of $2,600,000 (present value) and $3,860,000 (future value).

A new trunk is proposed, called the Highway 16 Trunk, to service development south of Highway 16 on the western edge of the city. The first section of this trunk, required in the twenty year growth horizon, is estimated to have a present cost of $1,350,000. The future value is estimated as $2,000,000.

8.6 Forty Year Growth Horizon (2055) – Off-Site Sanitary Sewer Servicing

The forty year growth horizon identified only off-site sanitary sewer servicing requirements in three separate locations.

The South Trunk is extended further west to 75 Avenue. This is estimated to have a capital cost of $1,710,000 (present value) or $3,770,000 (future value).

The Highway 16 Trunk (presented in the twenty year growth horizon) is extended to service additional development on the western edge of the city. The capital cost estimate for the trunk extension has a present value of $2,020,000 and a future value of $4,450,000.

A new trunk, called the Northwest Trunk, is proposed to service a significant area of development along the western and northern fringes of the city. The trunk runs from the western edge of the city, through the development area on the west and north sides of the city and ends at the WWTP. The capital cost estimate for the Northwest Trunk is $39,360,000 (present value) and $86,900,000 (future value).

8.7 Capital Cost Plan – Summary

A capital cost plan was developed based on the recommendations presented above for each of the growth horizons and is summarized below in Table 8.2.

Table 8.2: Construction Capital Cost Plan

	Off-Site Sanitary Sewer Servicing	Existing Sanitary Sewer System Upgrades	Total (Present Value)	Total (Future Value)
Present-Day (2015)	$0	$4,180,000	$4,180,000	
Three Year Growth Horizon (2018)	$0	$6,970,000	$6,970,000	$7,400,000
Five Year Growth Horizon (2020)	$0	$3,500,000	$3,500,000	$3,860,000
Ten Year Growth Horizon (2025)	$23,480,000	$30,470,000	$53,950,000	$65,770,000
Twenty Year Growth Horizon (2035)	$5,720,000	$0	$5,720,000	$8,490,000
Forty Year Growth Horizon (2055)	$43,090,000	$0	$43,090,000	$95,120,000
Total	**$72,290,000**	**$45,120,000**	**$117,410,000**	

9. Summary and Recommendations

The City of Lloydminster Sanitary Sewer Master Plan study was completed with the primary goal of identifying the performance of the existing sanitary sewer system and to establish the framework for servicing future development lands. This primary goal is founded on the development and calibration of a dynamic XPSWMM model of the city-wide sanitary sewer system.

The City conducted a flow monitoring program during the summer months of 2008 and 2015. Both years collected sound datasets of real-time flow data and captured two significant rainfall events that resulted in observable wet weather flow response in the sanitary sewer system. These datasets were used for subsequent calibration of the XPSWMM model for both dry weather and wet weather flows.

A key component in the evaluation of the performance of the sanitary sewer system is the Level of Service criteria. It is vital to identify Level of Service criteria that are practical and achievable such that upgrades are realistic and reasonably implemented. As part of the Level of Service criteria, the 25 year design storm was adopted as the critical rainfall event. The second Level of Service criterion was the mitigation of basement flooding, identified as the HGL rising to less than 2.4 m below ground surface.

The design rainfall event was selected on the basis of the best available data; however, it should be considered as preliminary at this time. The City has embarked on a flow monitoring program and dynamic modelling that will evolve as subsequent years of data is collected. The design rainfall event and the understanding of the wet weather flow response of the sanitary sewer system will be further enhanced as more critical rainfall events are captured.

The performance of the existing sanitary sewer system was then evaluated. There were two locations where possible basement flooding was encountered. Upgrade options, which included conveyance and storage upgrades, were developed to alleviate the occurrence of basement flooding. A conveyance upgrade to re-route flow from the 36 Street Trunk to the Larsen Grove neighbourhood was recommended as the preferred option for the first location. For the second location at 44 Street and 56 Avenue, re-routing of the sanitary sewer system was identified to alleviate the surcharge in the sanitary sewer.

The future development areas for the three year, five year, ten year, twenty year, and forty year growth horizons were determined. For each growth horizon, AECOM devised an off-site servicing scheme, assessed the impact to the existing downstream sanitary sewer system, and developed upgrade concepts to address the areas of concern in the sanitary sewer system that arose from the additional future flow. A summary of the recommendations for off-site sanitary sewer servicing and existing sanitary sewer system upgrades is presented below:

- Three year growth horizon – Existing system upgrades are required to alleviate surcharge in the Southeast Trunk. The proposed upgrades include the 19 Street Trunk and the South Trunk, and provide relief to the Southeast Trunk as well as provide off-site sanitary sewer servicing for future growth horizons.
- Five year growth horizon – Existing system upgrades are required to further alleviate surcharge in the Southeast Trunk. A portion of the Southeast Trunk is proposed to be twinned with a new sanitary sewer. The off-site sanitary sewer servicing requirements are met by the 19 Street Trunk and South Trunk proposed in the three year growth horizon.
- Ten year growth horizon – Existing system upgrades include twinning of the East Trunk from 40 Avenue to the WWTP. Off-site sanitary sewer servicing requirements include extending the South Trunk and introducing the CN Rail Trunk.
- Twenty year growth horizon – No existing system upgrades are required. The off-site sanitary sewer servicing requirements are met by extending the South Trunk and CN Rail Trunk and introducing the Highway 16 Trunk.

- Forty year growth horizon – No existing system upgrades are required. For off-site sanitary sewer servicing, the extension of the South Trunk and Highway 16 Trunk and the introduction of the Northwest Trunk is proposed.

AECOM developed a construction capital cost plan for the recommended off-site servicing and upgrade options for present day and each of the future growth horizons identified above. A summary of the construction capital cost plan is provided in Table 9.1.

Table 9.1: Construction Capital Cost Plan

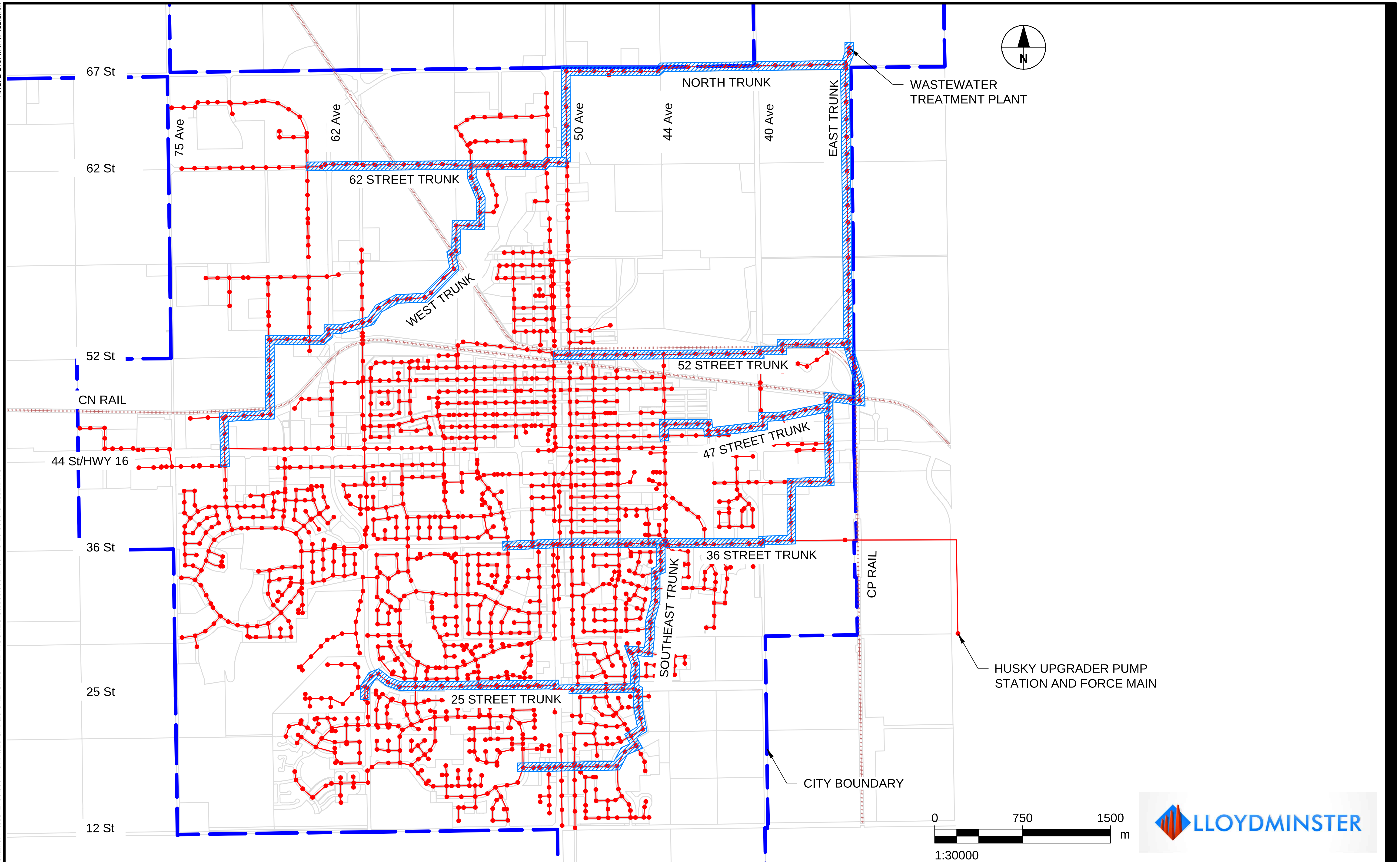
	Off-Site Sanitary Sewer Servicing	Existing Sanitary Sewer System Upgrades	Total (Present Value)	Total (Future Value)
Present-Day (2015)	$0	$4,180,000	$4,180,000	
Three Year Growth Horizon (2018)	$0	$6,970,000	$6,970,000	$7,400,000
Five Year Growth Horizon (2020)	$0	$3,500,000	$3,500,000	$3,860,000
Ten Year Growth Horizon (2025)	$23,480,000	$30,470,000	$53,950,000	$65,770,000
Twenty Year Growth Horizon (2035)	$5,720,000	$0	$5,720,000	$8,490,000
Forty Year Growth Horizon (2055)	$43,090,000	$0	$43,090,000	$95,120,000
Total	**$72,290,000**	**$45,120,000**	**$117,410,000**	

In consideration of the information presented in this study, AECOM offers the following recommendations:

- Initiate an annual flow monitoring program
- Conduct additional model calibration and verification as subsequent wet weather flow events are captured to garner a better understanding of the critical design rainfall event and wet weather flow response in the sanitary sewer system
- Continue to evaluate and potentially revise the Level of Service criteria and design rainfall event
- Implement the Present-Day upgrades
- Plan for the implementation of future off-site servicing and sanitary system upgrades

Appendix A

Figure Drawings



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SANITARY SEWER MASTER PLAN

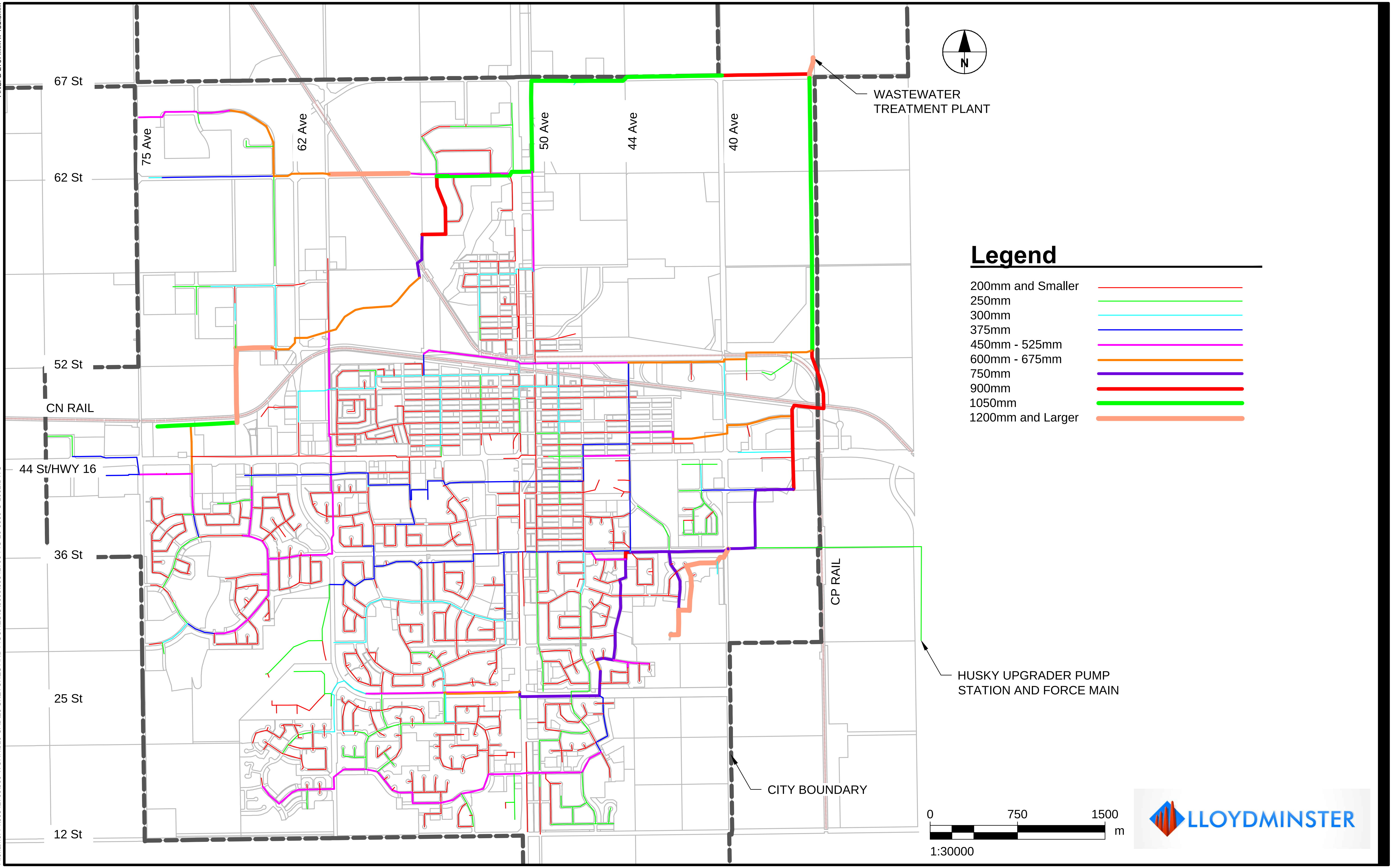
CITY OF LLOYDMINSTER

Project No.: 60342706 Date: 2016-03-15

SANITARY SEWER SYSTEM OVERVIEW

AECOM

Figure: A1



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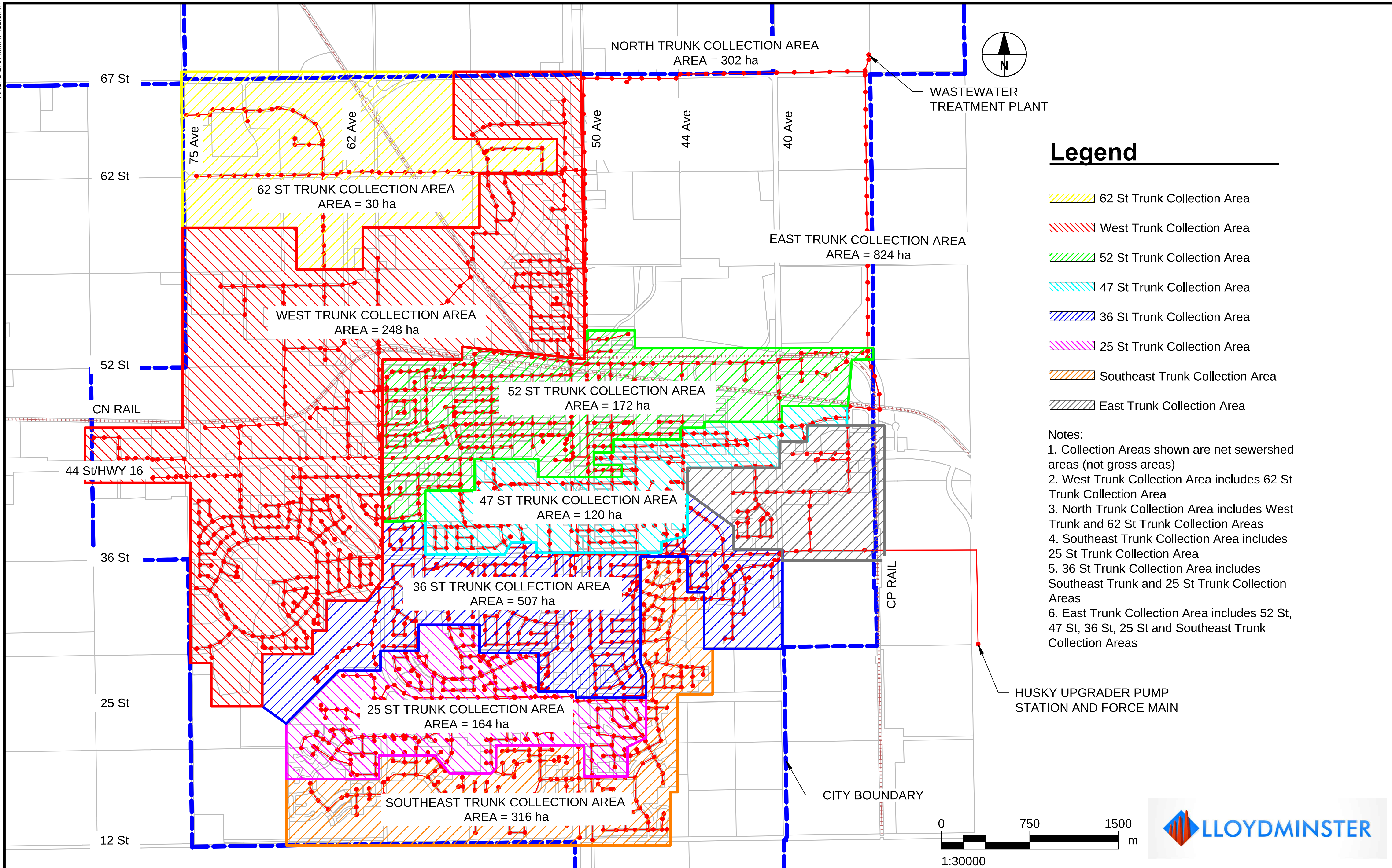
SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

SANITARY SEWER SYSTEM - PIPE OVERVIEW

AECOM

Figure: A2



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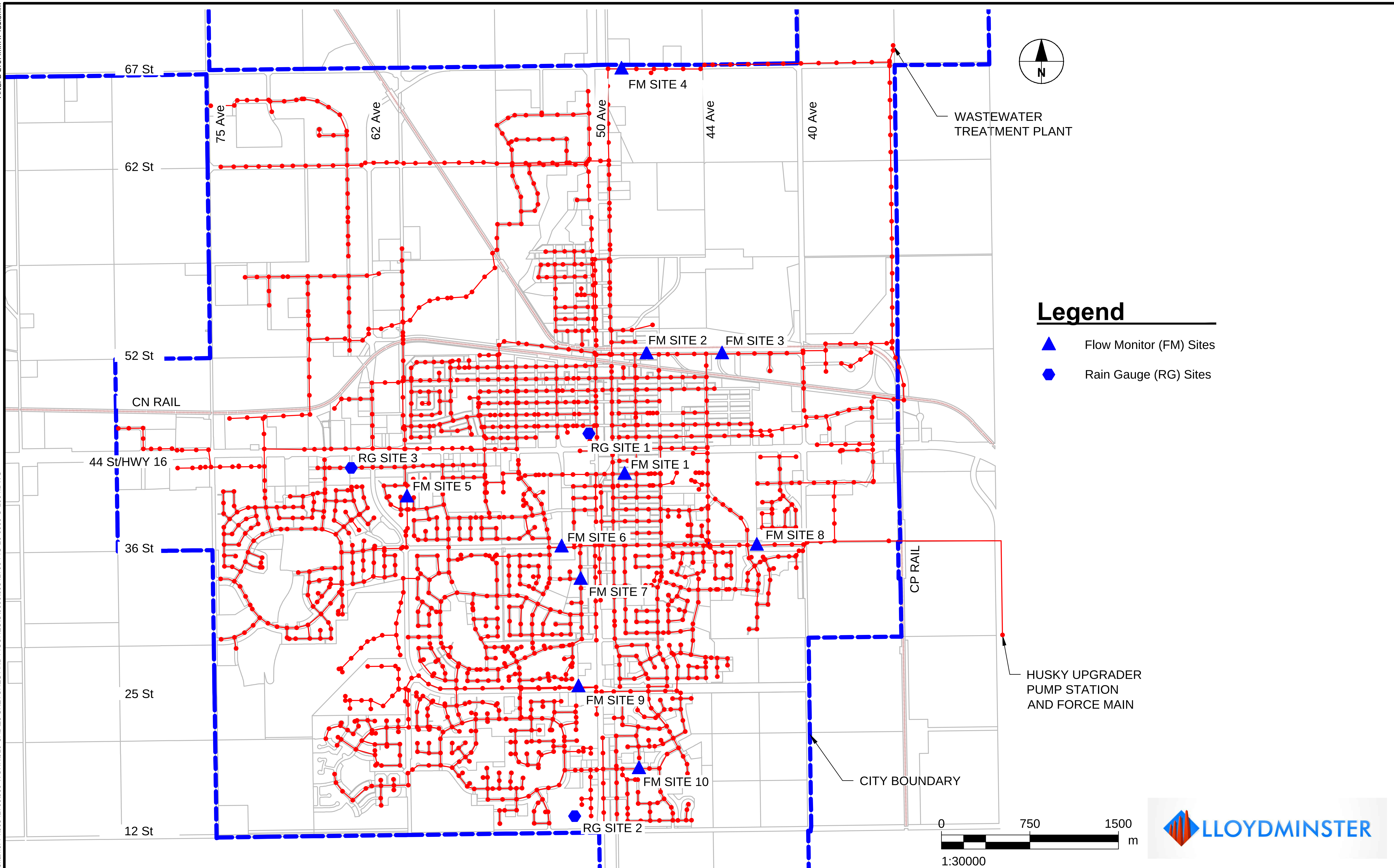
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

SANITARY SEWER COLLECTION AREAS

AECOM
Figure: A3



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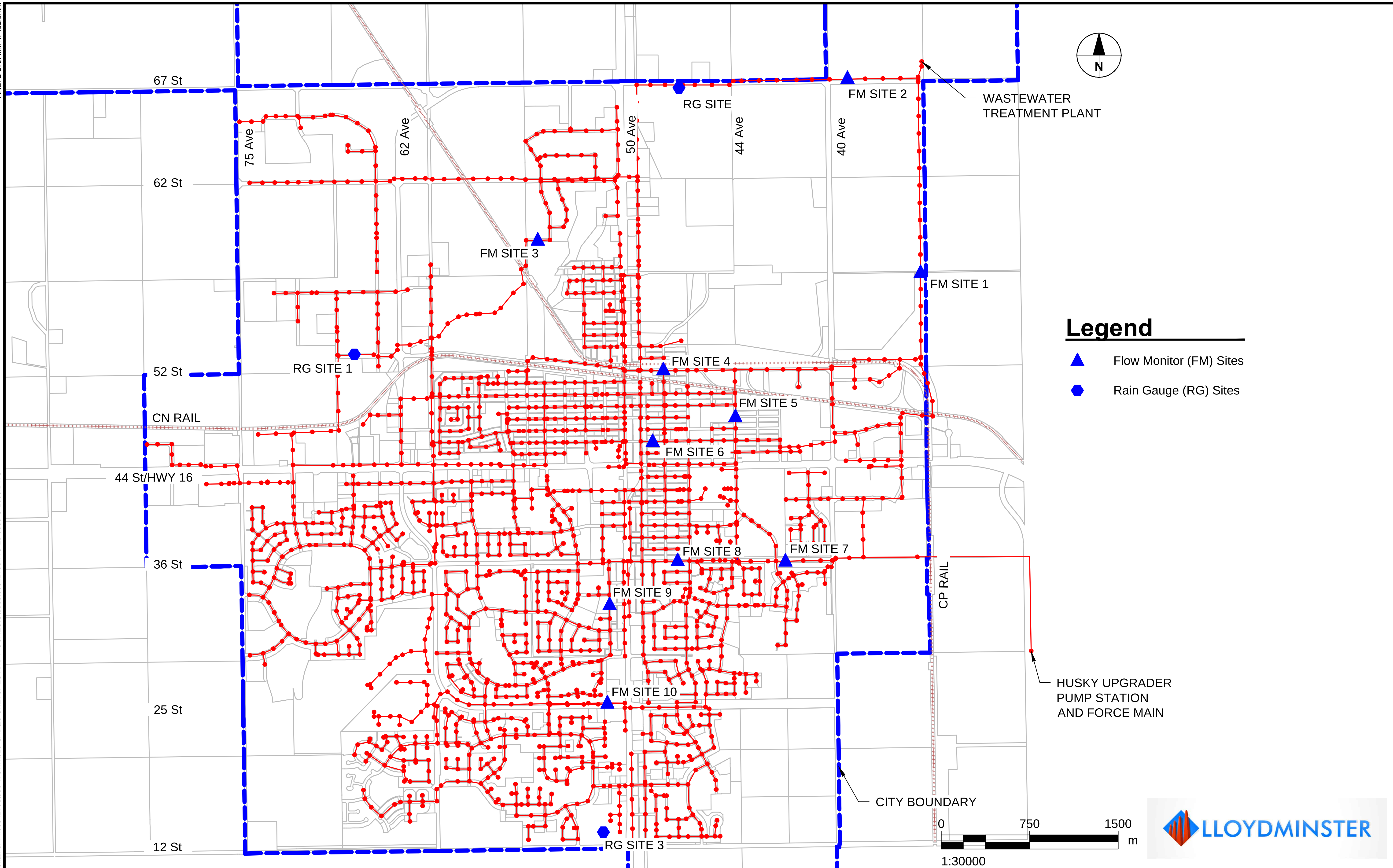
SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

2008 FLOW MONITORING PROGRAM

AECOM
Figure: A4

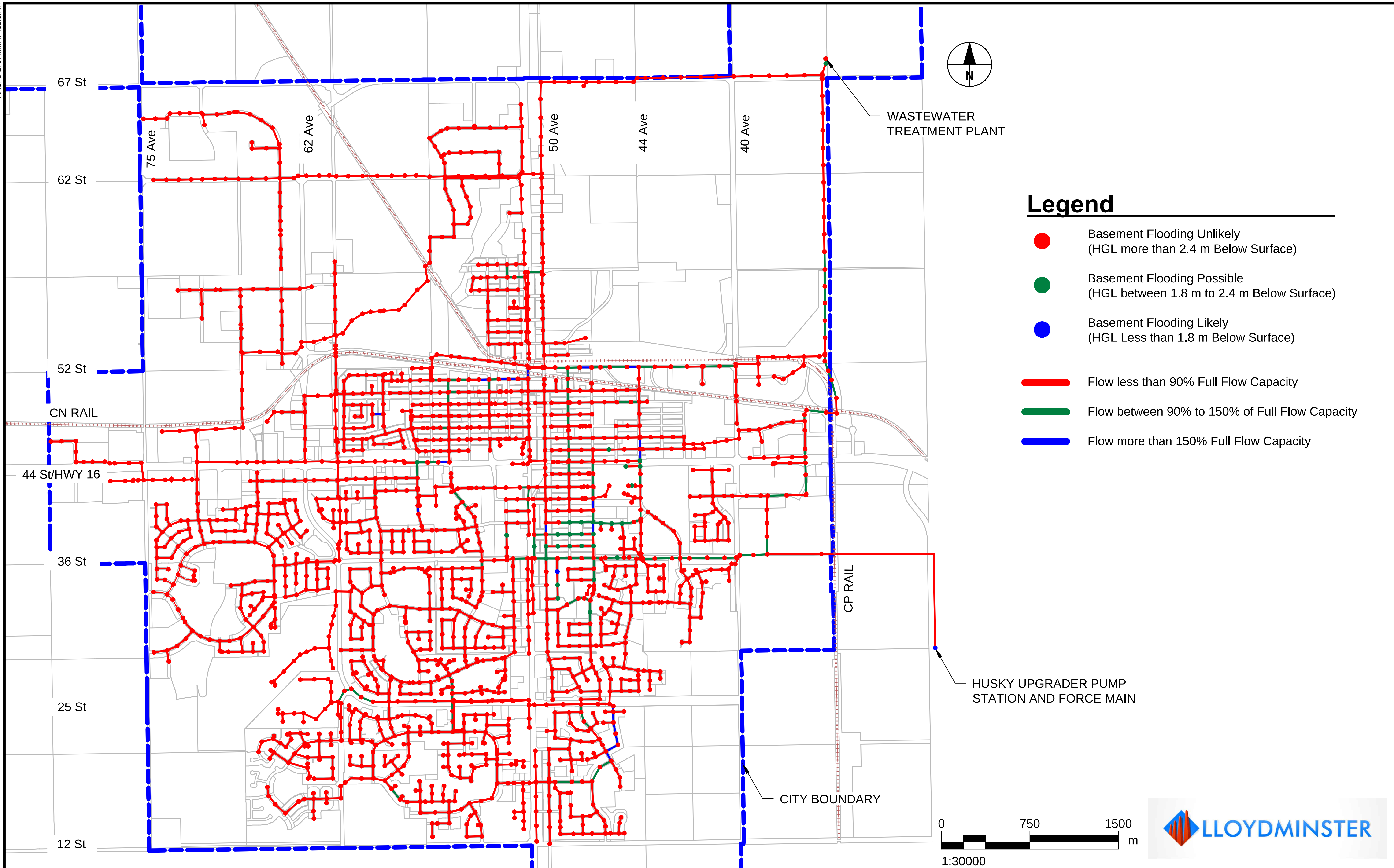


SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

2015 FLOW MONITORING PROGRAM

AECOM

Figure: A5



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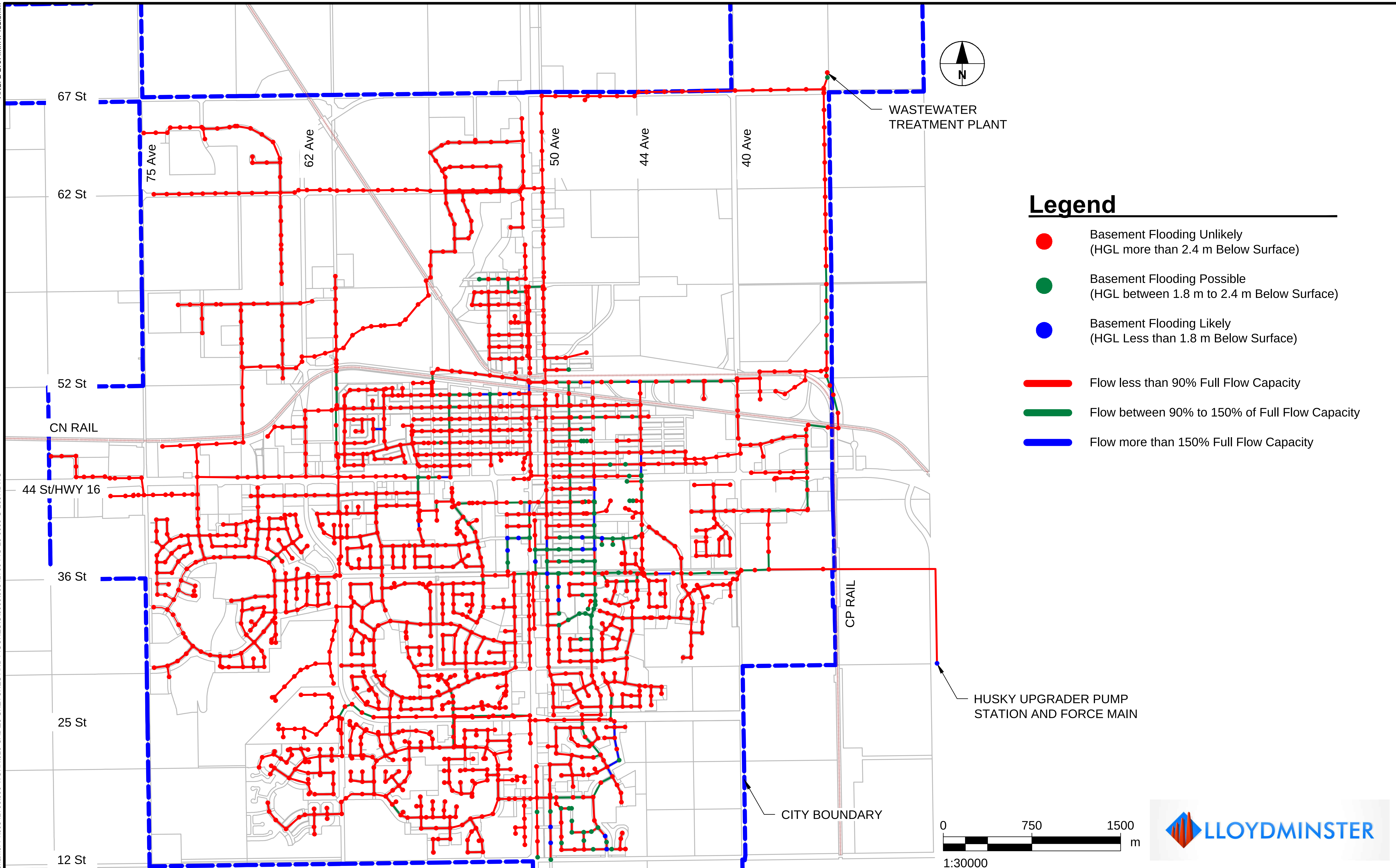
SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

EXISTING SANITARY SEWER SYSTEM PERFORMANCE (25 YEAR DESIGN STORM)

AECOM
Figure: A6

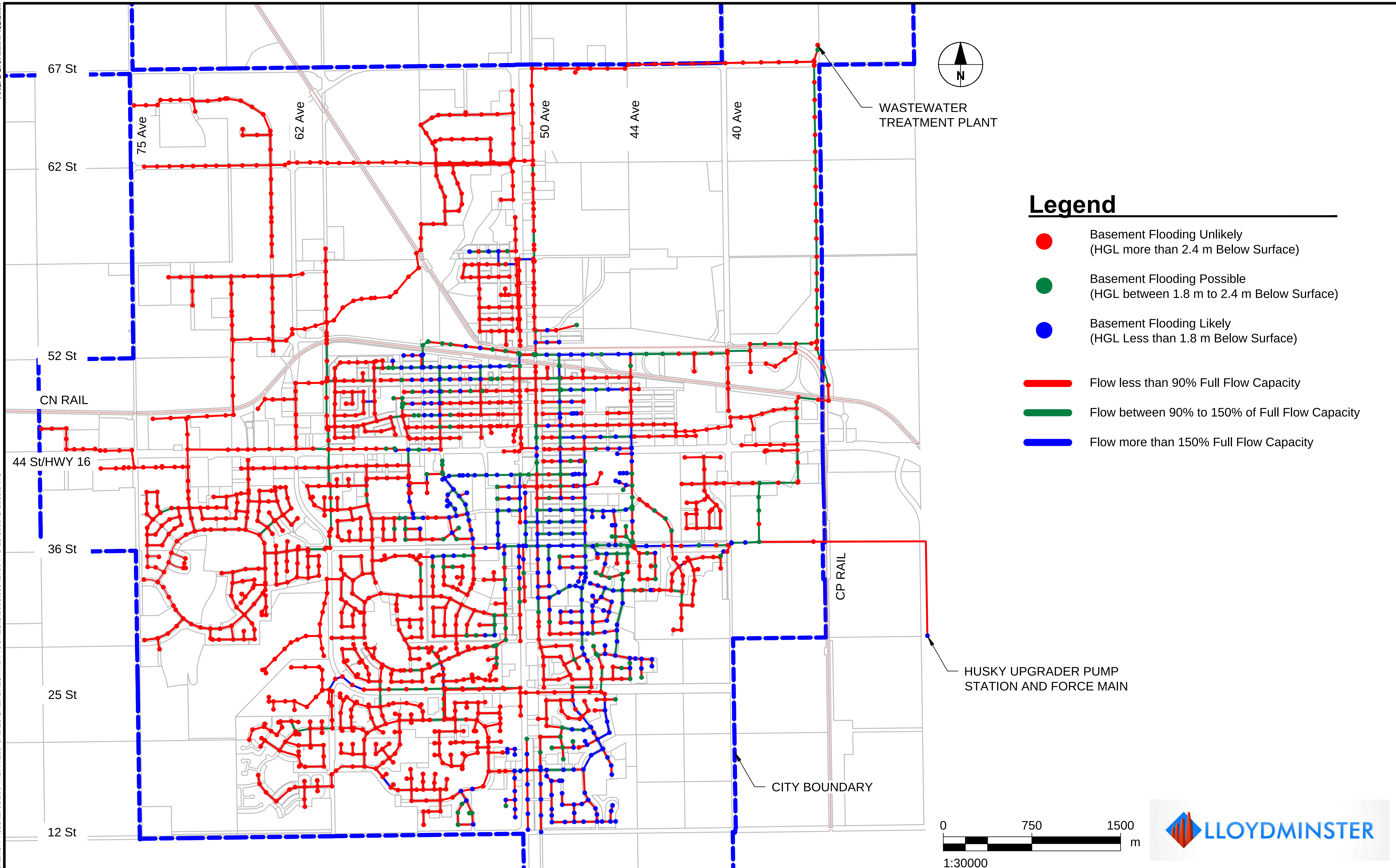


SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

EXISTING SANITARY SEWER SYSTEM PERFORMANCE (50 YEAR DESIGN STORM)

AECOM

Figure: A7



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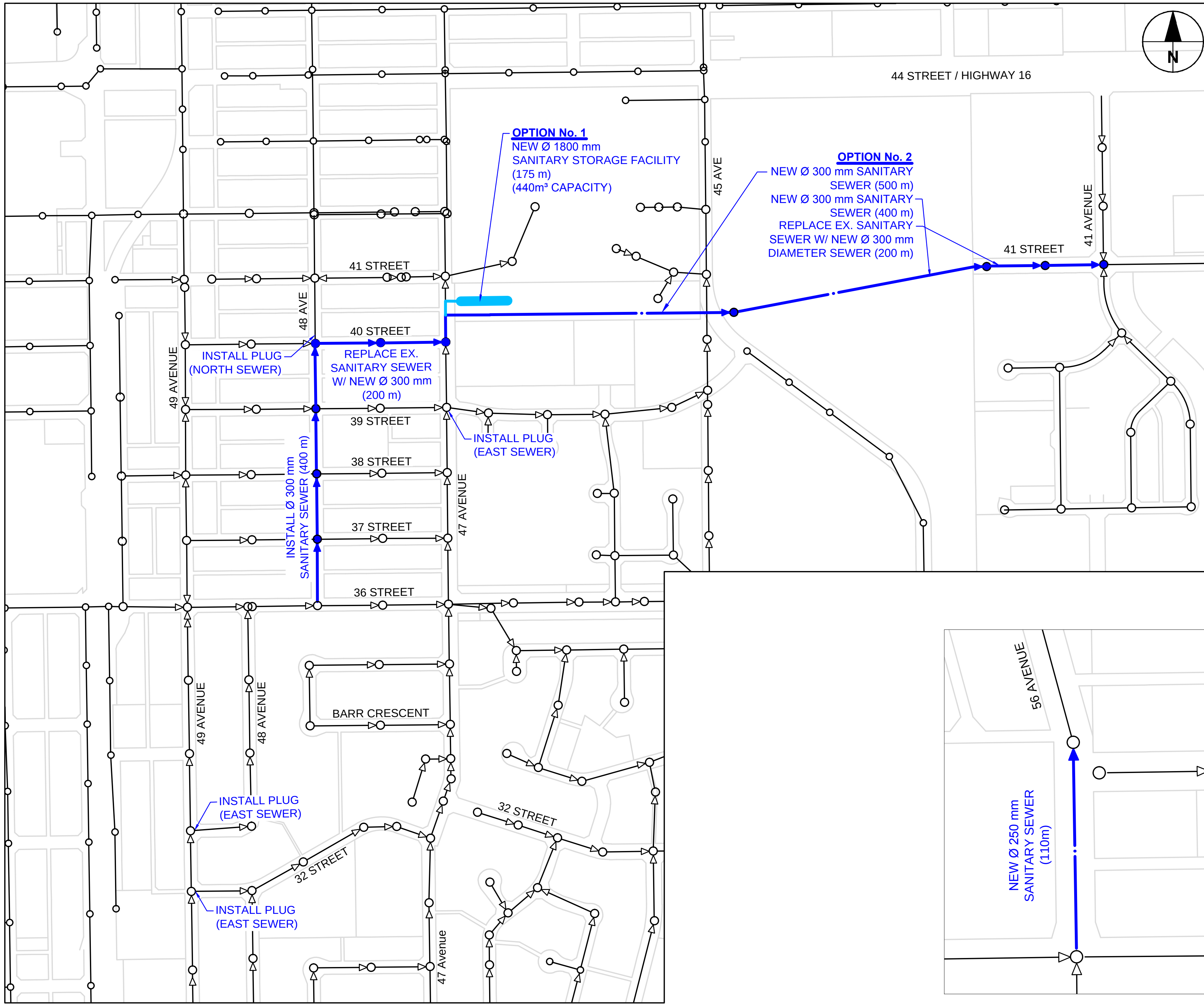
SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

EXISTING SANITARY SEWER PERFORMANCE (100 YEAR DESIGN STORM)

AECOM
Figure: A8



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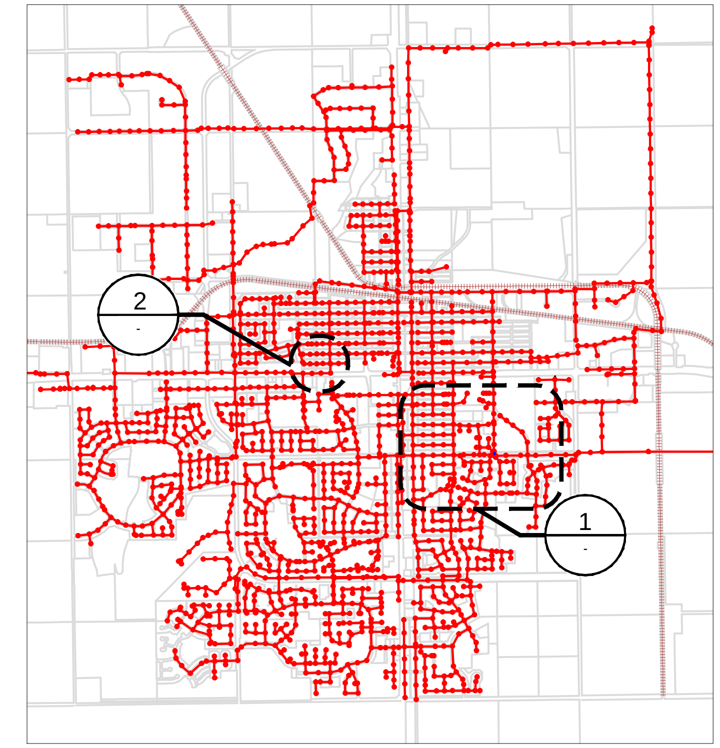

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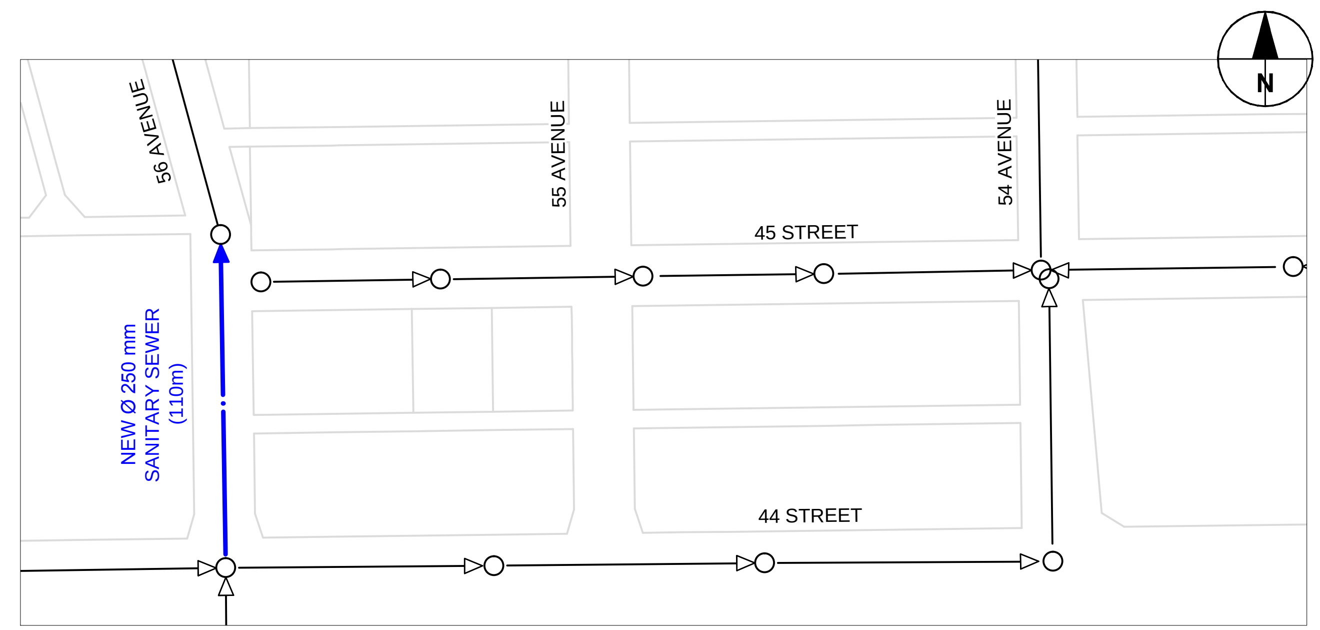

2 | 56 AVENUE UPGRADES
Scale 1:2500

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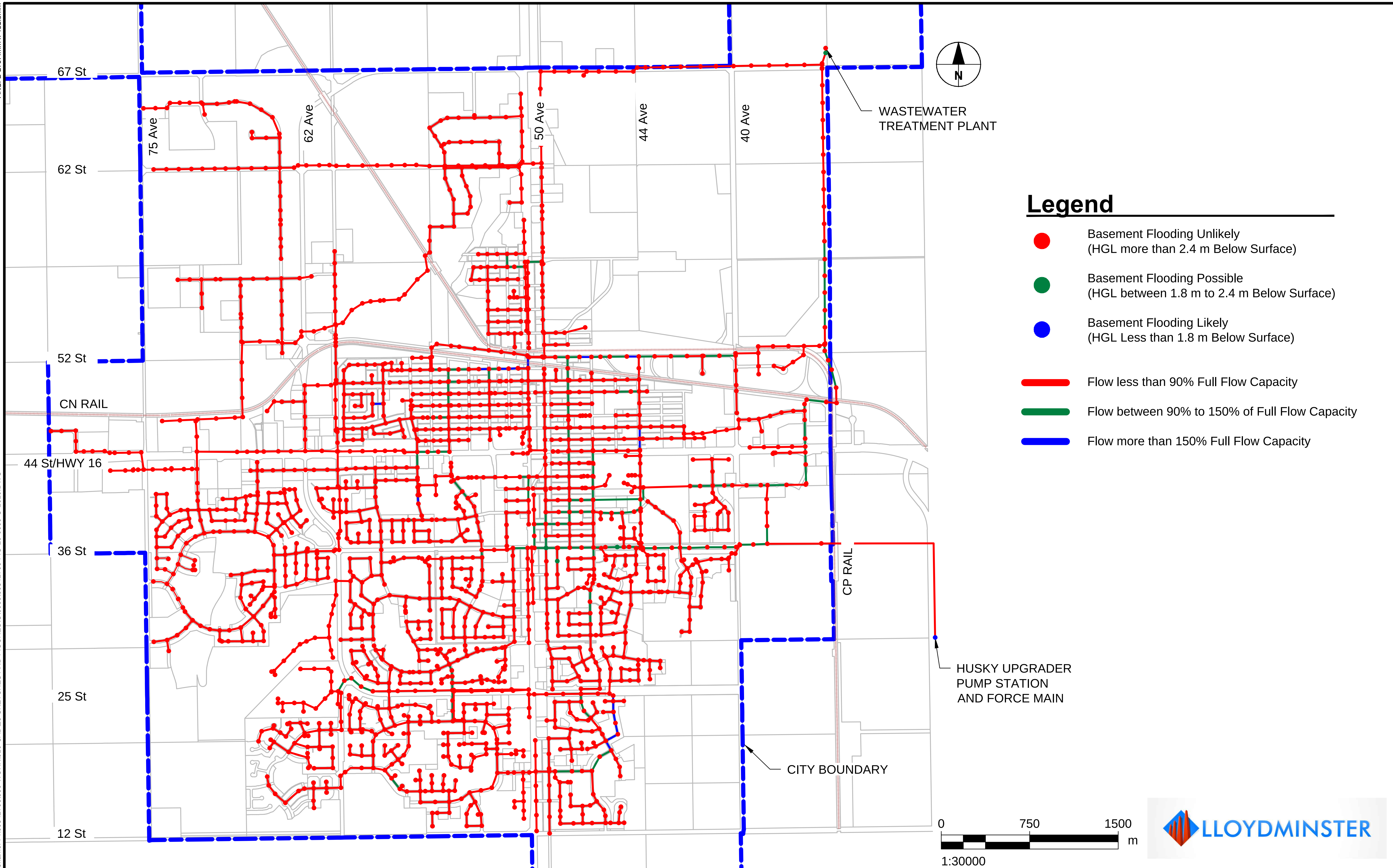
SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

PRESENT DAY (2015) - PROPOSED SANITARY SEWER SYSTEM UPGRADES

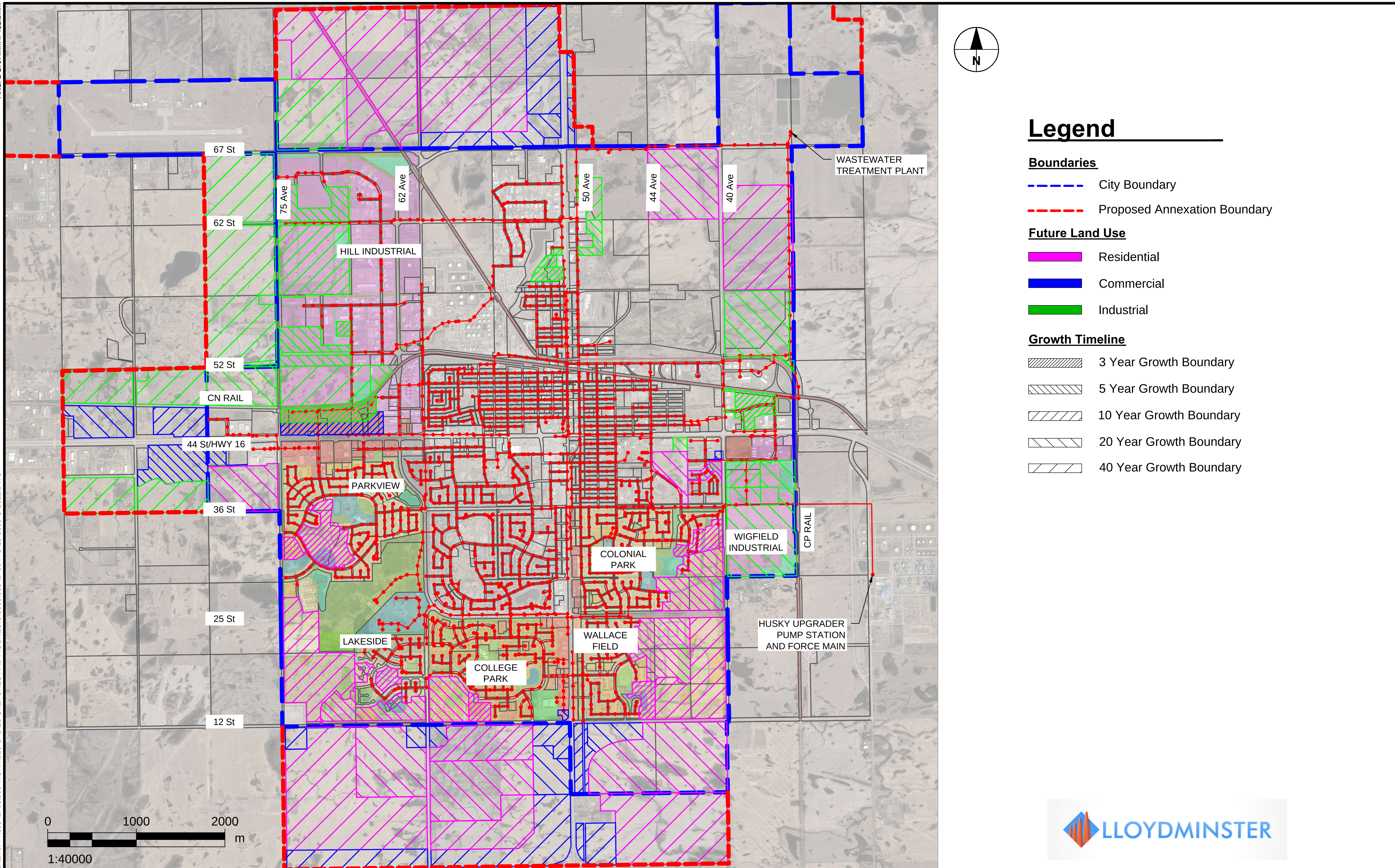
AECOM
Figure: A9



SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

EXISTING SANITARY SEWER SYSTEM PERFORMANCE - 25 YEAR DESIGN STORM
IMPLEMENTATION OF PROPOSED UPGRADES

AECOM
Figure: A10



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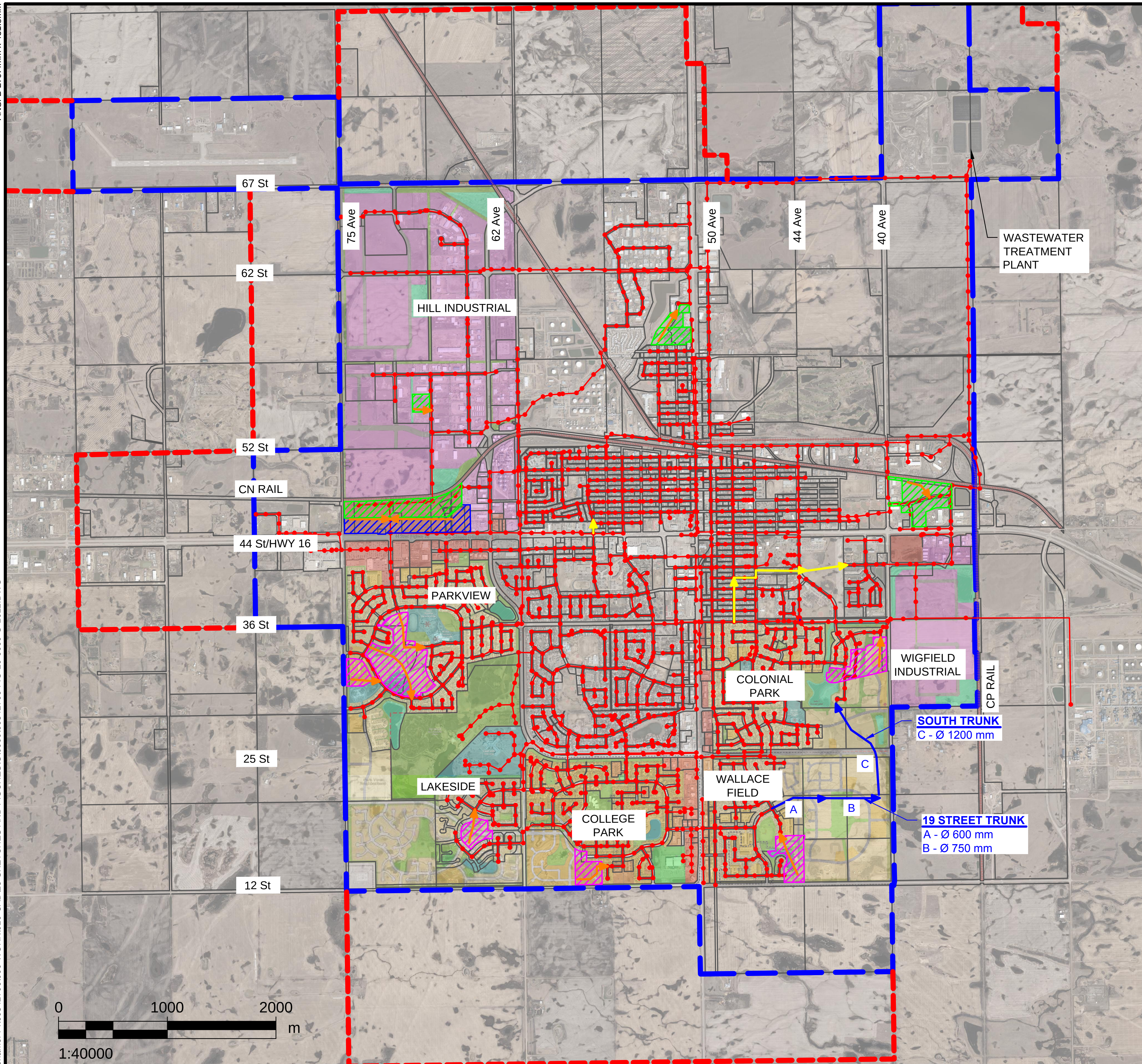
SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

FUTURE DEVELOPMENT STAGING PLAN

AECOM
Figure: A11



Legend

Boundaries

- City Boundary
- Proposed Annexation Boundary

Future Land Use

- Residential
- Commercial
- Industrial

Growth Timeline

- 3 Year Growth Boundary

Proposed Sanitary Servicing and Upgrades

- On-Site Collection System
- Present Day (2015) Sanitary Upgrades
- 3 Year Off-Site Sanitary Sewer Servicing and Existing System Upgrades

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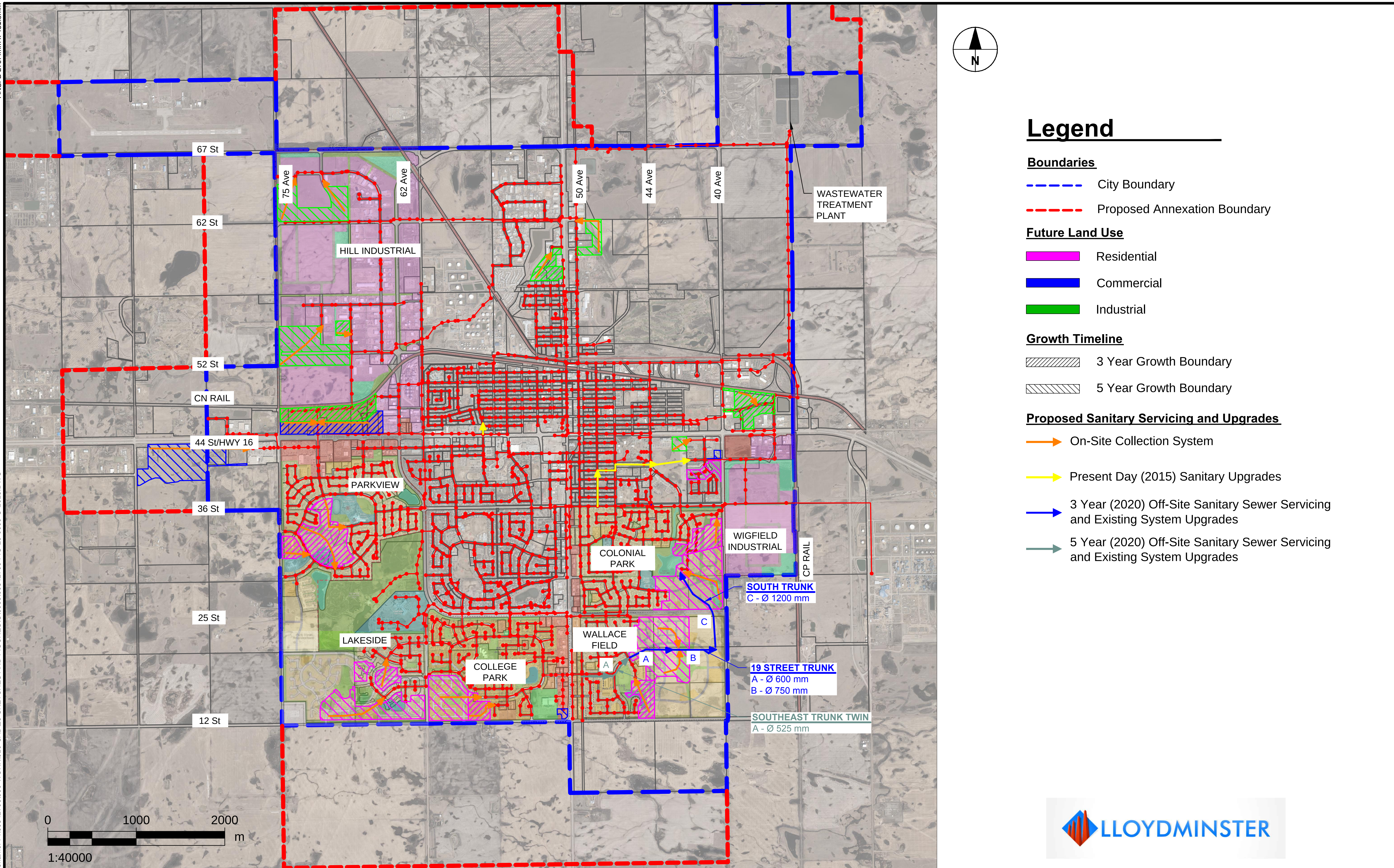
SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

THREE YEAR GROWTH HORIZON (2018) - PROPOSED SANITARY SEWER SERVICING AND UPGRADES

AECOM

Figure: A12



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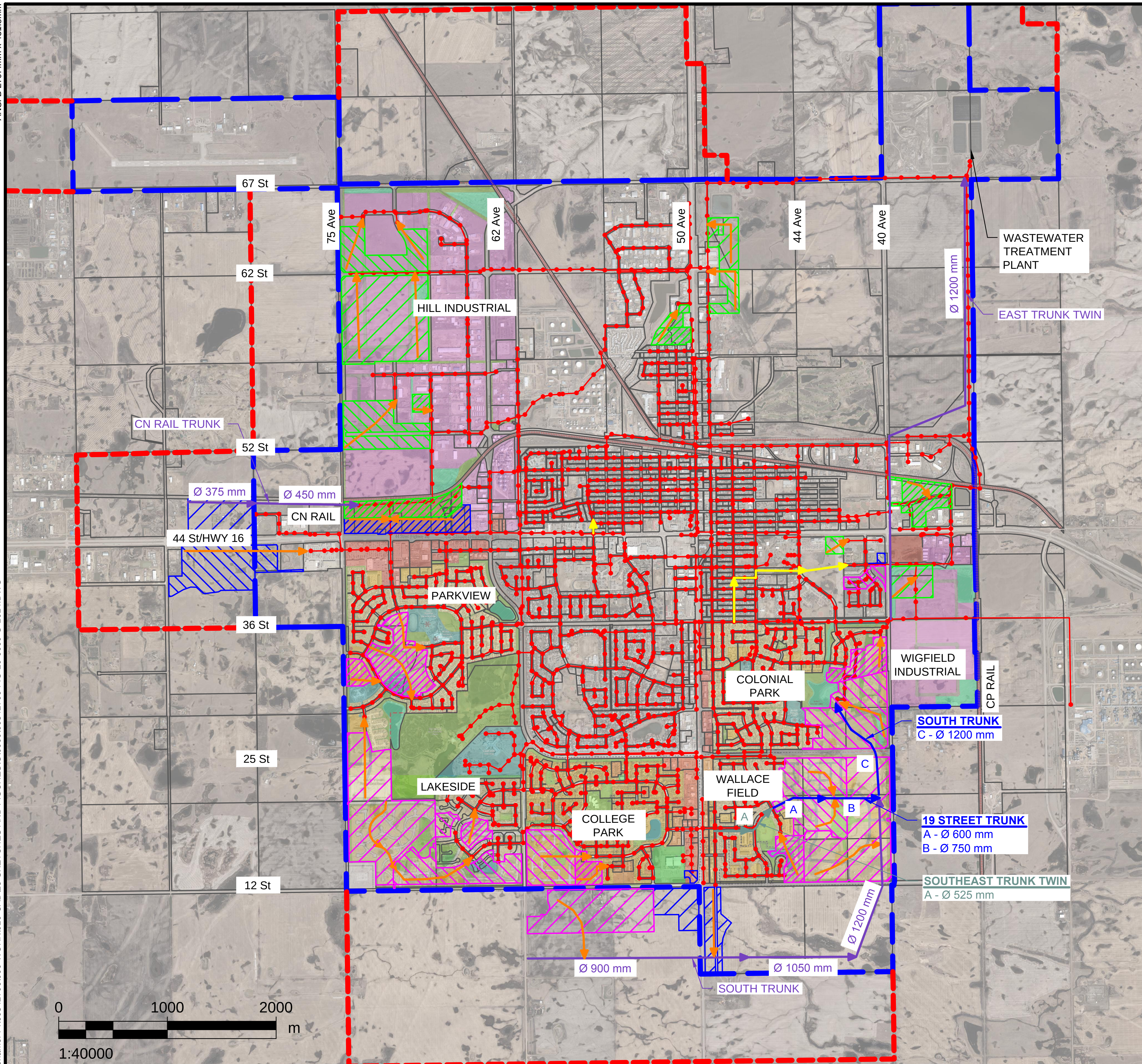
SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

FIVE YEAR GROWTH HORIZON (2020) - PROPOSED SANITARY SEWER SERVICING AND UPGRADES

AECOM
Figure: A13



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Legend

Boundaries

- City Boundary
- Proposed Annexation Boundary

Future Land Use

- Residential
- Commercial
- Industrial

Growth Timeline

- 3 Year Growth Boundary
- 5 Year Growth Boundary
- 10 Year Growth Boundary

Proposed Sanitary Servicing and Upgrades

- On-Site Collection System
- Present Day (2015) Sanitary Upgrades
- 3 Year (2018) Off-Site Sanitary Sewer Servicing and Existing System Upgrades
- 5 Year (2020) Off-Site Sanitary Sewer Servicing and Existing System Upgrades
- 10 Year (2025) Off-Site Sanitary Sewer Servicing and Existing System Upgrades

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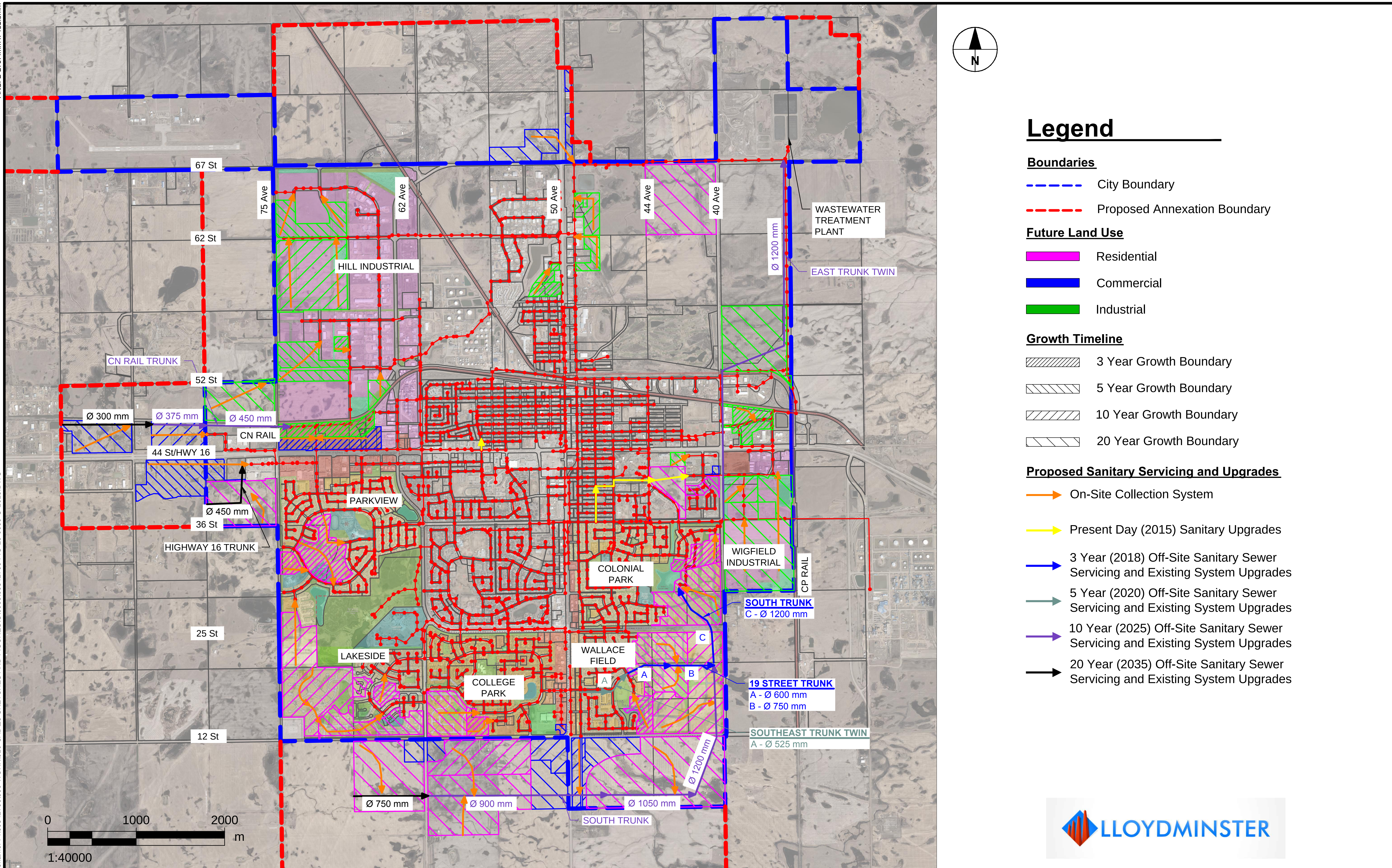
SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-01-29

TEN YEAR GROWTH HORIZON (2025) - PROPOSED SANITARY SEWER SERVICING AND UPGRADES

AECOM

Figure: A14



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SANITARY SEWER MASTER PLAN

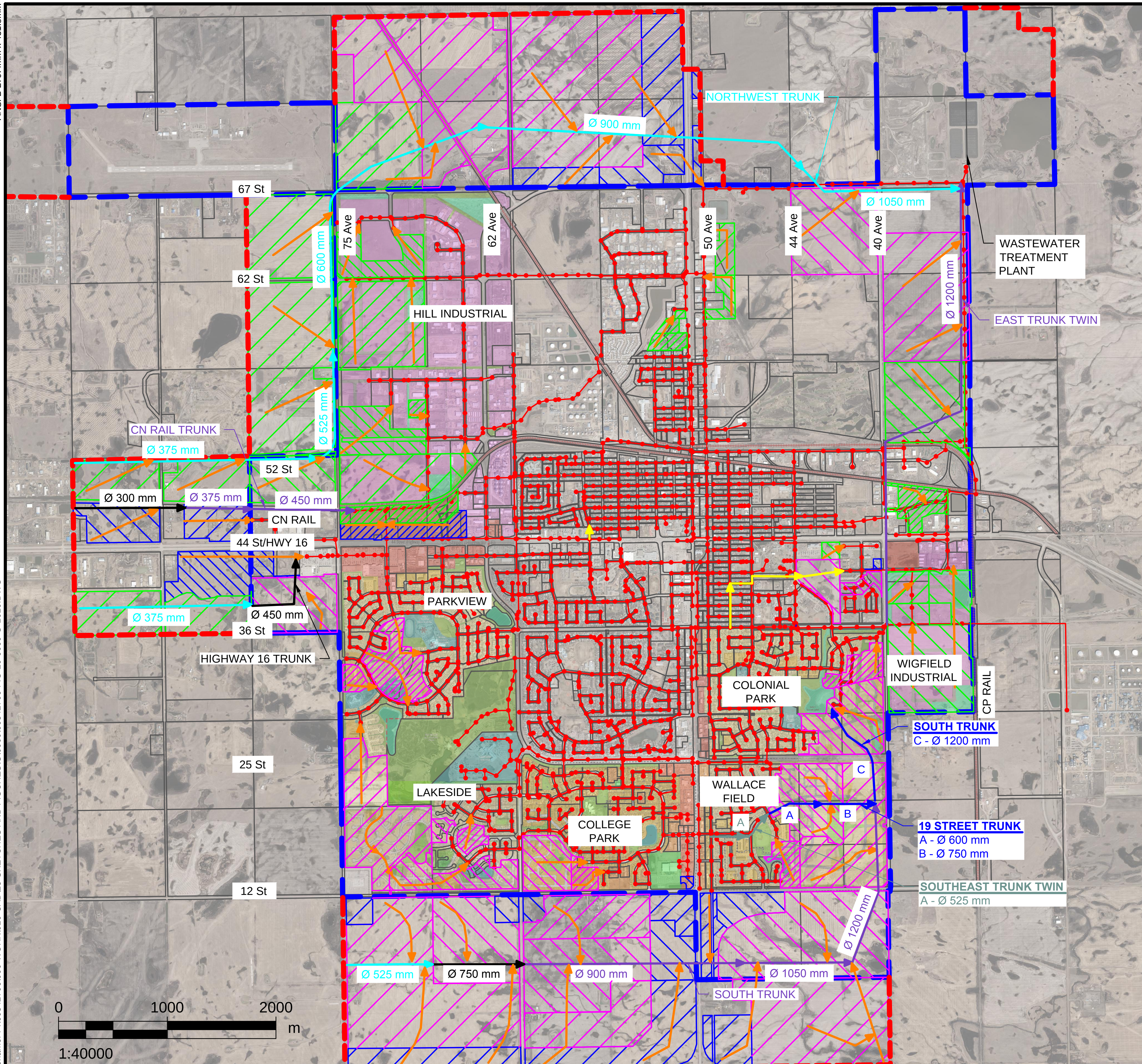
CITY OF LLOYDMINSTER

Project No.: 60342706 Date: 2016-03-15

TWENTY YEAR GROWTH HORIZON (2035) - PROPOSED SANITARY SEWER SERVICING AND UPGRADES

AECOM

Figure: A15



N

Legend

Boundaries

- City Boundary
- Proposed Annexation Boundary

Future Land Use

- Residential
- Commercial
- Industrial

Growth Timeline

- 3 Year Growth Boundary
- 5 Year Growth Boundary
- 10 Year Growth Boundary
- 20 Year Growth Boundary
- 40 Year Growth Boundary

Proposed Sanitary Servicing and Upgrades

- On-Site Collection System
- Present Day (2015) Sanitary Upgrades
- 3 Year (2018) Off-Site Sanitary Sewer Servicing and Existing System Upgrades
- 5 Year (2020) Off-Site Sanitary Sewer Servicing and Existing System Upgrades
- 10 Year (2025) Off-Site Sanitary Sewer Servicing and Existing System Upgrades
- 20 Year (2035) Off-Site Sanitary Sewer Servicing and Existing System Upgrades
- 40 Year (2055) Off-Site Sanitary Sewer Servicing and Existing System Upgrades

LLOYDMINSTER

Approved:
Checked:
Designer:
Project Management Initials:

ANSI B 279.4mm x 431.8mm

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SANITARY SEWER MASTER PLAN
CITY OF LLOYDMINSTER
Project No.: 60342706 Date: 2016-03-15

FORTY YEAR GROWTH HORIZON (2055) - PROPOSED SANITARY SEWER SERVICING AND UPGRADES

AECOM

Figure: A16

Appendix B

Technical Memorandum

Lloydminster Sanitary Master Plan Progress Update

(August 7, 2015)

AECOM
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Memorandum

To	Craig Anderson (City of Lloydminster)	Page	Page 1 of 23
CC	Abdelqader Abdelqader, Niki Burkinshaw (City of Lloydminster) Ryan King, Ryan Cadieux (AECOM)		
Subject	Lloydminster Sanitary Master Plan Progress Update		
From	Jonathan Peterson & Brock King (AECOM)		
Date	August 7, 2015	Project Number	60342706 (402.39)

1. Introduction

This technical memorandum is intended to summarize AECOM's progress on the Lloydminster Sanitary Sewer Master Plan study. The technical memorandum is an interim deliverable and is meant to present the City of Lloydminster (City) with AECOM's progress to date, initiate discussion between the City and AECOM, and establish agreement with the City on AECOM's approach. This will then lead to defined criteria to guide the completion of the study.

An overview of the City's sanitary sewer system is presented below, followed by discussion of the analysis completed on the 2008 flow monitoring data. The next section describes the development of the sanitary sewer model in XPSWMM software and the initial model calibration, completed with the 2008 flow monitoring data. This is followed by a description of the Level of Service criteria. The next section presents preliminary model results. The memorandum is concluded with a description of the next steps to be undertaken by AECOM.

2. Sanitary Sewer System Overview

The City of Lloydminster sanitary sewer system services a sewershed area of approximately 1249 ha and consists predominately of gravity sewer mains, with a total length of 135 km of pipe. There is one lift station that services the Husky Upgrader.

The sanitary sewer system is serviced by two main trunk sewers. The North Trunk is a 1050 mm diameter pipe and runs along 67 Street from 50th Avenue to the WWTP. The North Trunk services the northwest portion of the city (approximately 328 ha). The East Trunk is a 1050 mm diameter pipe and runs 800 m east of 40th Avenue. The East Trunk services the remainder of the city (approximately 921 ha).

The North and East trunks combine into a single 1200 mm diameter trunk (approximately 150 m in length) that connects to the WWTP.

There are three smaller trunks that connect to the East Trunk: the 52 Street Trunk (600 mm diameter), the 47 Street Trunk (675 mm diameter), and the 36 Street Trunk (750 mm diameter).

The hydraulics of the sanitary sewer system is complex. The city-wide system contains a total of 123 interconnections and flow-splits, which makes the system very difficult to properly analyse with manual/spreadsheet methods. The XPSWMM model simulations are able to accurately compute the hydraulics through the interconnections and flow-splits and will serve to be a valuable tool in assessing the interactions between different sewersheds and in evaluating potential upgrade options.

The city-wide sanitary sewer system is predominately a separate sewer system, meaning that stormwater is directed to a separate storm sewer system. However, there is a small section of combined sewer (which has stormwater draining to the sanitary sewer system), located in the downtown core. The total extent of the combined sewer is unknown; however, as part of the annual capital upgrades program, the City has been separating the combined sewer as it is encountered.

The City has embarked on an ambitious sewer upgrade strategy, based on the recommendations from the previous Sanitary Master Plan. Presently, the City has completed the 25 Street upgrades, which included twinning the 450 mm diameter 25th Street trunk sewer with a 600 mm diameter sewer between 50 Avenue and 52 Avenue and upsizing and rerouting the sewer on 23 Street and 59 Avenue.

The City has also completed detailed design on the 75th Sanitary Sewer Extension project. This project has not commenced construction at this time.

3. 2008 Flow Monitoring Data Analysis

The flow monitoring data analysis used the 2008 flow monitoring data to initially calibrate the XPSWMM model of the City's sanitary sewer system. The 2008 flow monitoring program was implemented by SFE Global of Edmonton, Alberta and conducted through the summer months of 2008 (May 3 to October 2) The intention was to capture dry weather flow and any significant wet weather events that may occured during severe summer rainstorms. The flow monitoring program consisted of ten flow monitors that were installed in the sanitary sewer throughout the City and three tipping bucket rain gauges.

The flow in the sewer pipes with diameter less than 525 mm was measured using the Isco 2150 Ultra Sonic Flow Module instrument. Sites that had a pipe diameter larger than 525 mm used the Isco 2150 Average Velocity Flow Module Instrument. The instruments consisted of a small probe that was mounted on the bottom of the upstream pipe within the manhole. The probe would transmit the readings to a data logger hung near the top of the manhole, where it was regularly connected to a laptop computer to download the flow data. The data logger was configured to record 5 minute readings of flow depth and velocity, which was then converted to flow. Figure 3.1 shows the flow monitor probe and Figure 3.2 shows the data logger installed in a manhole.

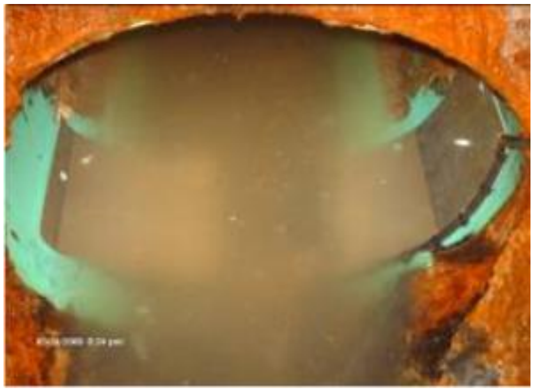

Figure 3.1: Flow Monitor Probe

Figure 3.2: Flow Monitor Data Logger

To measure the volume and intensity of rainfall during a storm, tipping bucket rain gauges were installed at City Hall, the Commonwealth Centre, and the West Reservoir. The rain gauge had a bucket that was mounted so that it automatically tipped when the bucket filled with a 0.1 mm depth of rainfall. The intensity of the rainfall was calculated as the number bucket tips in a 5 minute interval. Figure 3.3 shows the tipping bucket rain gauge on the roof of the West Reservoir.

Figure 3.3: Tipping Bucket Rain Gauge

3.1 2008 Flow Monitoring Sites

The flow monitoring program used ten flow monitors and three tipping bucket rain gauges. The ten flow monitors provided real-time flow measurements in the sanitary system at a specific point in the system. A summary for each of the flow monitoring sites is summarized below in Table 3.1.

Table 3.1: Flow Monitoring Analysis

Flow Monitoring Site	Manhole ID	Type of Flow	Pipe Size (mm)	Sewershed Area (ha)	Average Daily Flow (L/s)
1	0354	Commercial	375	172	15
2	0029	Commercial	450	316	26
3	0023	Commercial	600	835	34
4	1086	Commercial	1050	266	27
5	2010	Residential	525	83	10
6	0471	Residential	375	70	10
7	0442	Residential	375	52	12
8	0975	Residential	750	471	60
9	0854	Residential	450	131	19
10	1766	Residential	450	66	6

Three tipping bucket rain gauges were set up to provide the rate-of-rainfall that is required to analyse the Wet Weather Flows and calibrate the hydraulic model. A summary of the locations where the tipping bucket rain gauges were set up is summarized below.

- Rain Gauge 1: Located at the centre of the city, on the roof of City Hall (this site was inoperable for the majority of the program);
- Rain Gauge 2: Located on the south boundary of the City on the roof of the Commonwealth Recreational Centre, and;
- Rain Gauge 3: Located on the west side of the City at the West Water Reservoir.

Assessing the 2008 flow monitoring program, the total DWF for the entire City appears to be slightly underestimated. The summation of Sites 3, 4, and 8 does not include a small portion of the City (35 ha) and was not considered as part of the actual total flow.

3.2 Dry Weather Flow Data

The first step of the DWF analysis was to identify a seven day period where there was no rainfall during the week. During the time of the 2008 flow monitoring program, there was week in each month of June, July, August, and September that met this criterion. The flow monitoring data from the 2008 flow monitoring program included the flow rate at the ten different sites on five minute intervals. The flow at the different intervals was averaged to define the average hourly flow rate for each hour of every day throughout the week. As the population’s daily routine shifts from weekdays to weekends, the daily flow pattern also changed.

The flow contributors were split into to two types of flow (residential and commercial/industrial), considering the difference in diurnal patterns for the residential and commercial/industrial areas. This is because commercial contributors generally have a higher and more consistent flow during business hours and lower flows outside of work hours and therefore do not see as pronounced of a peak. The DWF diurnal patterns from the 2008 Flow Monitoring Data are shown in Figure 3.4 and Figure 3.5.

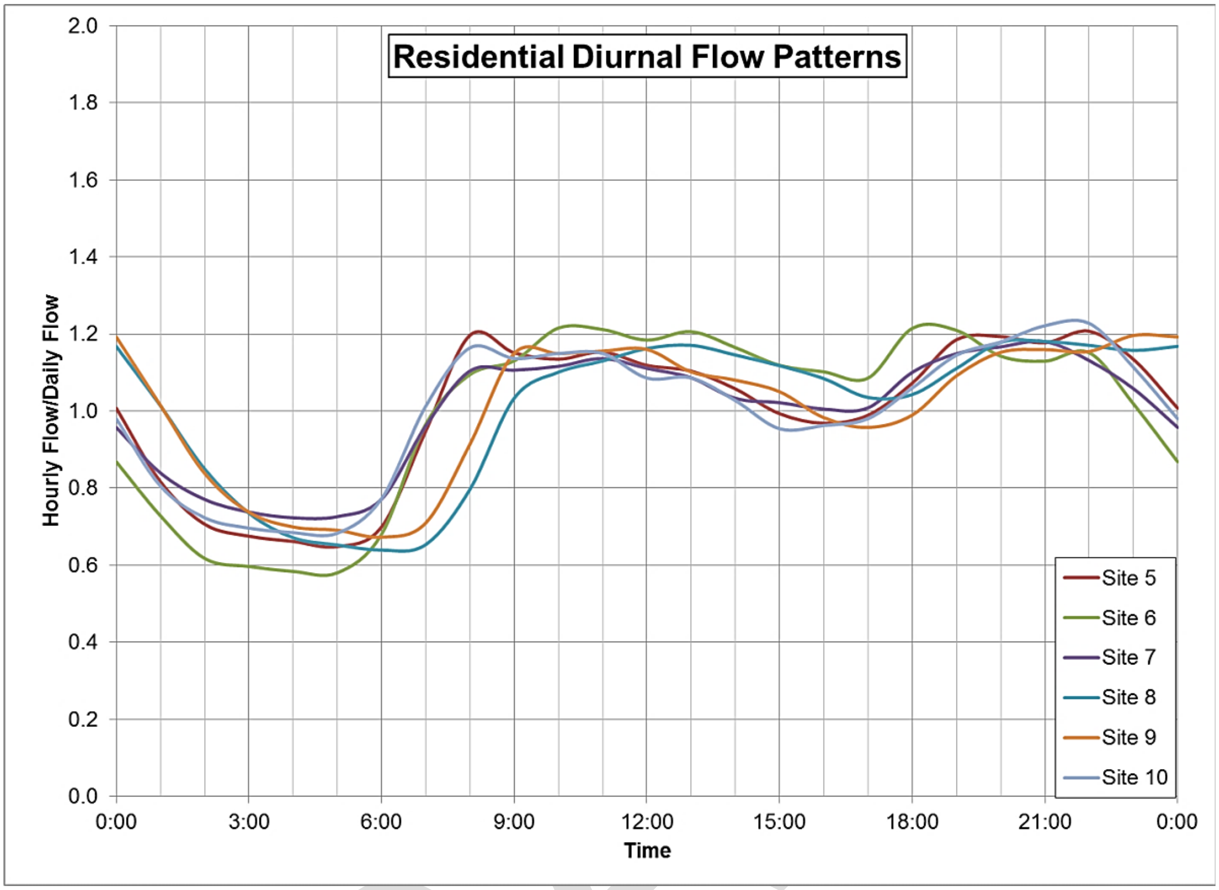

Figure 3.4: Residential Diurnal Flow Pattern

The climbing limb of the residential diurnal patterns for Site 8 and Site 9 occur approximately one hour after the climbing limb for the other residential sites. This is because Site 8 and Site 9 are main trunks that service a large upstream collection area. The one hour delay is a due to the travel time required for the wastewater to flow through the sanitary system in the upstream collection system to the trunk. This trend is confirmed by comparing Site 8 and Site 9. The peak at Site 8 occurs a short time after the peak at Site 9. Site 8 is downstream of Site 9 so that the wastewater flow takes longer to travel from the upstream collection areas to the Site 8 flow monitoring.

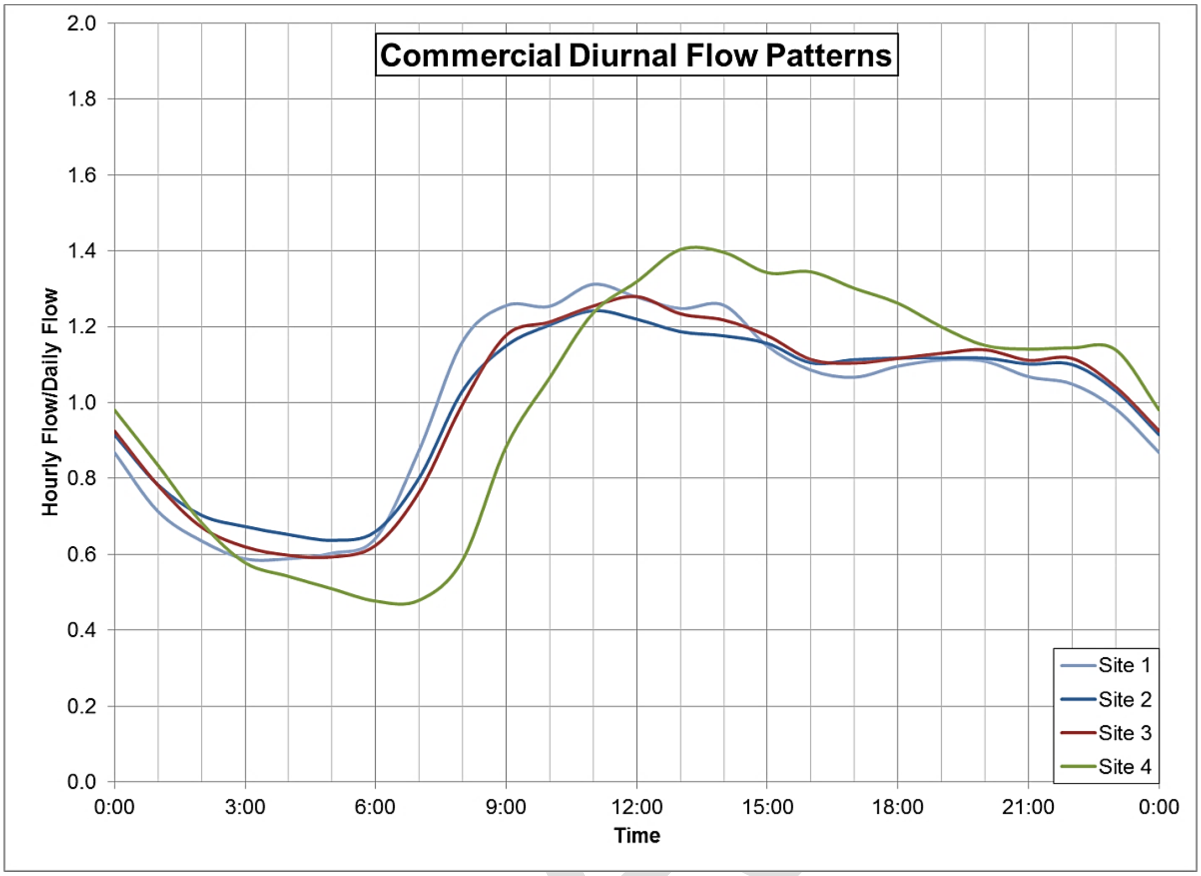

Figure 3.5: Commercial Diurnal Flow Patterns

The diurnal patterns for the commercial flow sites show a consistent flow from 9am until 3pm. Site 4 shows that the flow climbs and recedes two hours after Sites 1, 2, and 3, but follows the same general pattern. Site 4 is further downstream from the other sites, so this lag could be due to the increased travel time (as discussed above).

The diurnal patterns for the residential and commercial contributors were developed into hourly multipliers for the Average Dry Weather Flow (ADWF). This was done in order to simulate the dual peaks for the residential flow and a continual flow during work hours for commercial contributors. The diurnal multipliers were used to calibrate the type of flow and the estimated volume of flow from the City's delineated contribution areas and the City of Edmonton sanitary sewer design standards. The estimated design flow was then calibrated to the 2008 flow monitoring data that was provided by the City. Table 3.2 shows the diurnal pattern multipliers that were used for the residential and commercial areas.

Table 3.2: Diurnal Pattern

Time	Multiplier	
	Residential	Commercial
12:00 AM	0.54	0.90
1:00 AM	0.50	0.76
2:00 AM	0.36	0.67
3:00 AM	0.34	0.63
4:00 AM	0.40	0.61
5:00 AM	0.55	0.61
6:00 AM	1.00	0.64
7:00 AM	1.60	0.81
8:00 AM	1.32	1.06
9:00 AM	1.30	1.20
10:00 AM	1.29	1.22
11:00 AM	1.28	1.27

Time	Multiplier	
	Residential	Commercial
12:00 PM	1.26	1.26
1:00 PM	1.20	1.22
2:00 PM	1.18	1.22
3:00 PM	1.15	1.16
4:00 PM	1.20	1.10
5:00 PM	1.40	1.10
6:00 PM	1.22	1.11
7:00 PM	1.19	1.12
8:00 PM	1.16	1.12
9:00 PM	1.06	1.09
10:00 PM	0.80	1.09
11:00 PM	0.80	1.09

3.3 Wet Weather Flow Data

There was one significant rainfall event, occurring on August 9. 2008, that provided wet weather flow (WWF) response in the sanitary sewer. The West Reservoir measured 42.2 mm of rainfall over 4 hours and 10 minutes and is approximately equivalent to a 5 year return period rain event. The Commonwealth Centre measured 51.4 mm of rain over 4 hours and 10 minutes and is approximately equivalent to a 10 year return period rain event.

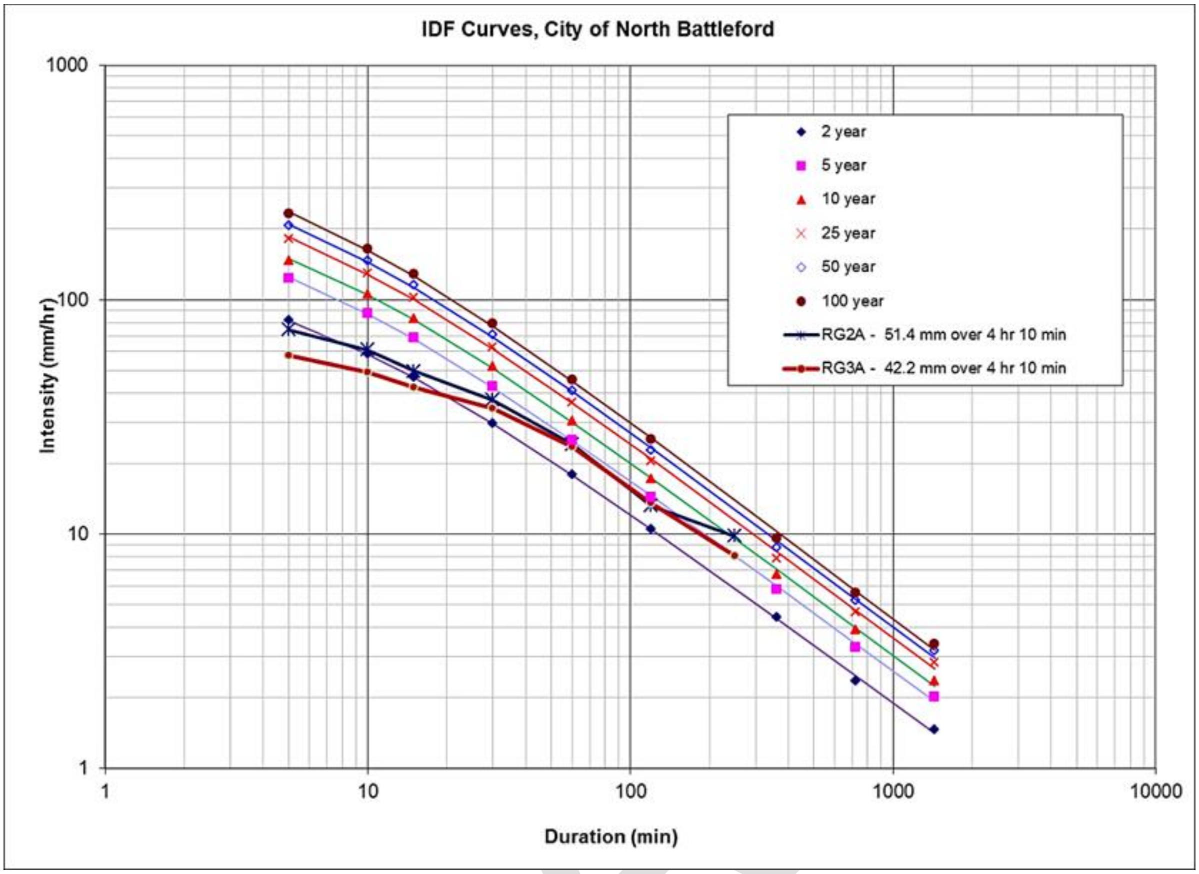

Figure 3.6: IDF Curves with the Two Rainfalls

The results of the flow monitoring data analysis are shown below in Table 3.3.

Table 3.3: Flow Monitoring Data Analysis Results

Flow Monitoring Site	Manhole ID	ADWF (L/s)	Peak DWF (L/s)	Peak WWF (L/s)	Infiltration Volume (m^3)
1	0354	15	20	104	861
2	0029	26	43	209	3198
3	0023	34	44	227	4502
4	1086	27	43	232	3444
5	2010	10	13	129	1618
6	0471	10	13	118	1617
7	0442	12	14	149	667
8	0975	60	73	512	6840
9	0854	19	23	227	3826
10	1766	6	8	78	933

4. Model Development

The city-wide sanitary sewer model was developed in XPSWMM software (Version 2013 SP1). The model is built as nodes and links, with the nodes representing manholes and points of inflow and the links representing pipes. The city-wide model contains 1846 nodes (manholes) and 1968 links (pipes).

In a sanitary sewer model, the flow consists of two components: the Dry Weather Flow (DWF) and the Wet Weather Flow (WWF). The DWF is made up of groundwater infiltration and domestic sewage flow. Groundwater infiltration is a relatively constant flow and varies on a seasonal time scale. The domestic sewage flow follows a regular and repetitive pattern throughout a 24 hour period, with a maximum peak occurring at approximately 7:00 am, a second smaller peak at approximately 7:00 pm, and a minimum flow at approximately 3:00 am.

The WWF is composed of the DWF and the Rainfall-Derived Inflow and Infiltration (RDII) flow. The RDII flow was simulated with the RTK method, which uses three unit hydrographs. Each unit hydrograph represents the fast, medium, and slow response of the inflow to the system. The fast response may indicate direct inflows to the system through low-lying manhole lids and direct drainage connections. The medium response may be composed of slower flows to the system, such as weeping tile flows. Finally, the slow response may be representative of longer-lasting inflows such as increased infiltration (above the relatively constant groundwater inflow) into the system through the pipe and manhole joints.

Each of the three unit hydrographs are drawn based on the R, T, and K parameters. The R value represents the proportion of the rainfall that enters the sanitary sewer, T is the time of the peak in the hydrograph, and K describes the recession limb of the hydrograph. The flow rate at a given time interval is calculated by the multiplying the R value by the rainfall intensity and the collection area of the node.

The XPSWMM software solves two “layers”: the Hydraulics layer and the Runoff layer. The Hydraulics layer contains the pipe system as well as the DWF contribution and routes the flow through the pipe system. The Runoff layer simulates the hydrology and the Rainfall-Derived Inflow and Infiltration (RDII), which the software then inputs into the nodes in the Hydraulics layer to route the RDII flow through the pipe system.

The City provided a GIS database of their sanitary sewer manholes and pipes in AutoCAD format. The attributes of the GIS database, which included manhole rim and invert elevation and pipe inverts, size, material, length, and slope, was exported to an Excel database. The Excel database was imported into the XPSWMM software to automatically build the model. The model data was manually reviewed and verified for validity. The City provided a subset of plan/profile record drawings to confirm and correct any curious data. Also, it was found that a number of manhole rim elevations were either missing or erroneous. The City’s LIDAR data was used to correct this data.

The upgrades implemented in the 25 Street project were manually added to the model. The 75 Avenue Sanitary Sewer Extension has been designed; however, construction has not begun. As such, this information was not added to the model.

The sanitary sewer collection areas draining to each section of pipe were delineated and transferred to the upstream nodes. The domestic sewer flow from residential areas was calculated by determining the number of single-family homes and multi-family units, multiplying by a population density of 3.5 persons/household for single-family and 2.5 persons/household for multi-family, and multiplying by an unit Average Dry Weather Flow (ADWF) of 290 l/person/day.

For commercial, industrial, and institutional (schools, churches, hospital), the ADWF was calculated by determining the type of business/institution from Google Street View, measuring the floor area of the building, and multiplying by a unit ADWF based on City of Edmonton design standards.

The sewershed area draining to each section of pipe was measured from AutoCAD and imported to each node in the model. The model then calculates the RDII flow by multiplying the area by the proportion of the rainfall entering the sanitary sewer system at each time interval.

5. Model Calibration

Once the XPSWMM model of the city-wide sanitary sewer system was developed, the model was calibrated with the 2008 flow monitoring data. The calibration occurred in two steps. First, the DWF component of the model was calibrated, followed by the WWF.

5.1 Dry Weather Flow Calibration

The ADWF was calculated for each individual collection area based on City of Edmonton design standards for different land-uses. The design ADWF was then calibrated based on the measured ADWF from the flow monitoring data.

The DWF design flows were calibrated in the XPSWMM model to the 2008 DWF data. Adjustments to the diurnal patterns and design value for type of flow contributors were made until the XPSWMM dry weather flow hydrographs looked provided a close fit to the 2008 flow. The final step of the calibration was to make sure the total system flow for the model, WWTP flow meter, and flow monitoring data. Table 5.1 compares the total flow out of the system.

Table 5.1: Comparison of Total Dry Weather Flow

	Model	WWTP Flow Meter	Total Flow From Sites 3,4,& 8
Total DWF Outflow	9,800 m^3	11,600 m^3	10,400 m^3

A specific diurnal variation was used for the residential and commercial/industrial areas. The development of the diurnal variation is a rigorous exercise, as the flow pattern input at the nodes differs from the pattern measured at the flow monitor, due to travel time through the pipe, flow attenuation, and the range of locations that the flow enters the system (at nearly all locations in the upstream collection system). The residential diurnal variation was developed based on AECOM's experience in other municipalities. The commercial/industrial diurnal variation was developed based on the flow monitoring data. Through an iterative process, the DWF patterns were adjusted in the XPSWMM simulations to closely match the measured DWF patterns.

After the DWF calibration was deemed satisfactory, AECOM proceeded with the WWF calibration.

5.2 WWF Calibration

The R, T, and K parameters were developed by calibrating the model to flow monitoring data. The R value is specific to the storm event and is influenced by a number of factors (e.g. rainfall intensity, duration, antecedent moisture conditions). Thus, each WWF event typically requires the R value to be adjusted to calibrate the model to the flow monitoring data. The T and K parameters are characteristics of the physical system, and may be considered to be common amongst different WWF events.

Tipping bucket rain gauges collected rainfall data for the period of May 3, 2008 to October 2, 2008. During this period there was one significant rainstorm that caused a flow response in the sanitary sewer. The rainstorm took place on August 9, 2008 and the rainfall depth varied between the bucket rain gauges.

- Rain Gauge #1 was not in working condition during the rain event,
- Rain Gauge #2 recorded 51.4 mm over four hours and 10 minutes (equivalent to a 5 year return period), and
- Rain Gauge #3 recorded 42.2 mm over four hours and 10 minutes (equivalent to a 10 year return period).

The storm resulted in a noticeable WWF response in the sewer system. To assess the response to the system, R, T, K, parameters for the fast, medium, and slow response hydrographs were adjusted until the simulated WWF closely fit the flow monitor data. In order to achieve a better fit to the flow monitoring data, three sets of RTK parameters were developed. The downtown area with combined sewer was given a higher value of R to account the for the storm runoff that would directly enter the sanitary sewer system. The data from the flow monitoring program rainfall event on August 9, 2008 showed a sharp peak and quick recession of the WWF in the City's northwest industrial area with a smaller total R value. Finally, for the remaining area of the City, the R value remained consistent throughout. The results of the initial calibration are shown below in Table 5.2.

Table 5.2: Calibrated RDII (R,T,K) - August 9, 2008

	Residential			Downtown (Combined Sewer)			Northwest Industrial		
	Fast	Medium	Slow	Fast	Medium	Slow	Fast	Medium	Slow
R	0.025	0.045	0.060	0.070	0.075	0.100	0.020	0.020	0.045
T	0.7	3	9	1	3	9	0.7	3	9
K	3	4	6	3	4	6	2	4	6

The rainfall data from the West Reservoir was chosen to calibrate the WWF. Although this rainfall event was less severe than measured at the Commonwealth Centre, it is deemed to be more representative of the city-wide system as it is located nearer to the middle of the city. There is not sufficient data available to determine a variable distribution of the storm event throughout the city. Further, this is a conservative approach to the calibration. By using a smaller storm event to calibrate the WWF, the model will predict greater flow when scaling up to a more severe design event (discussed later).

A comparison of the DWF and WWF hydrographs for the measured flow and simulated flow for each flow monitor site are presented below in Figure 5.1 through Figure 5.10.

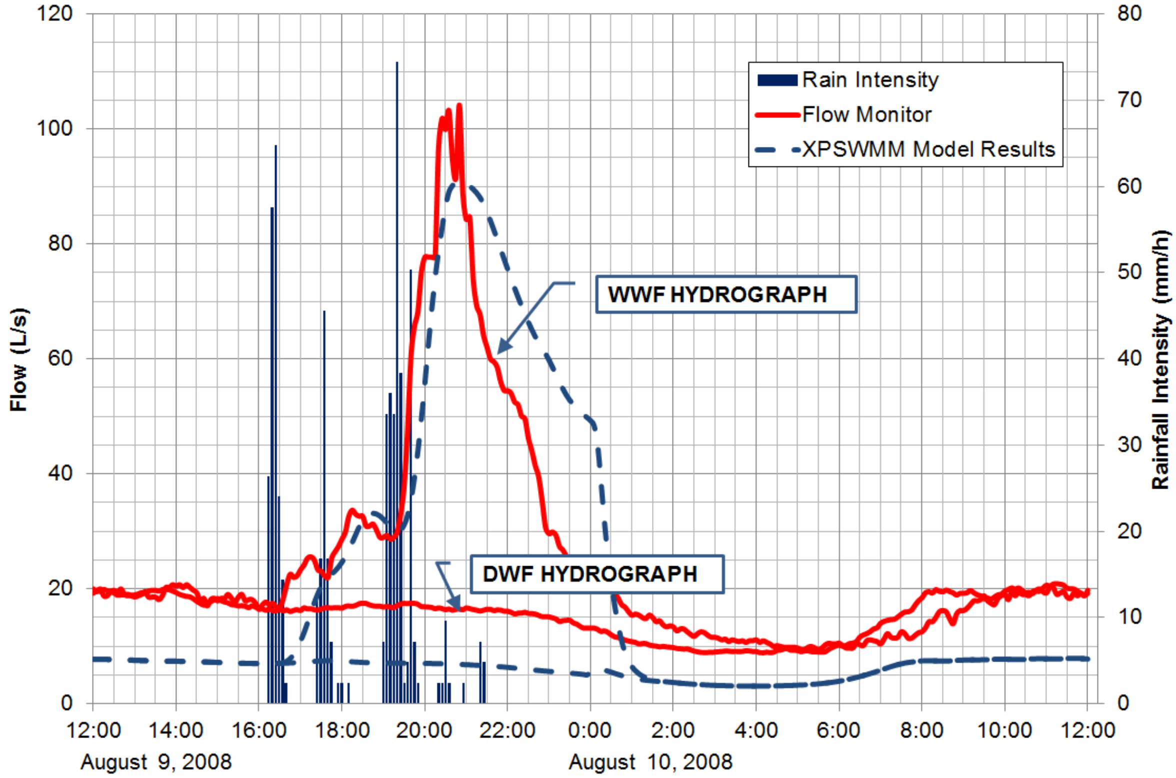

Figure 5.1: Site #1 Hydrograph

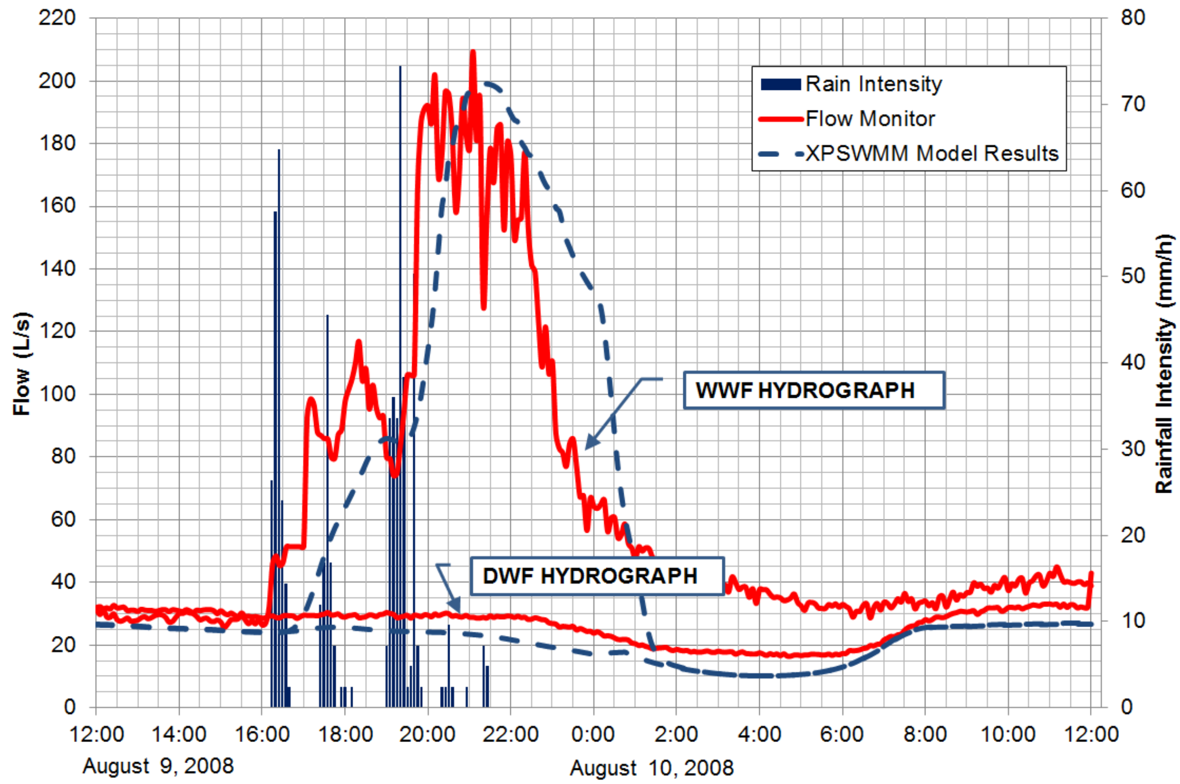

Figure 5.2: Site #2 Hydrograph

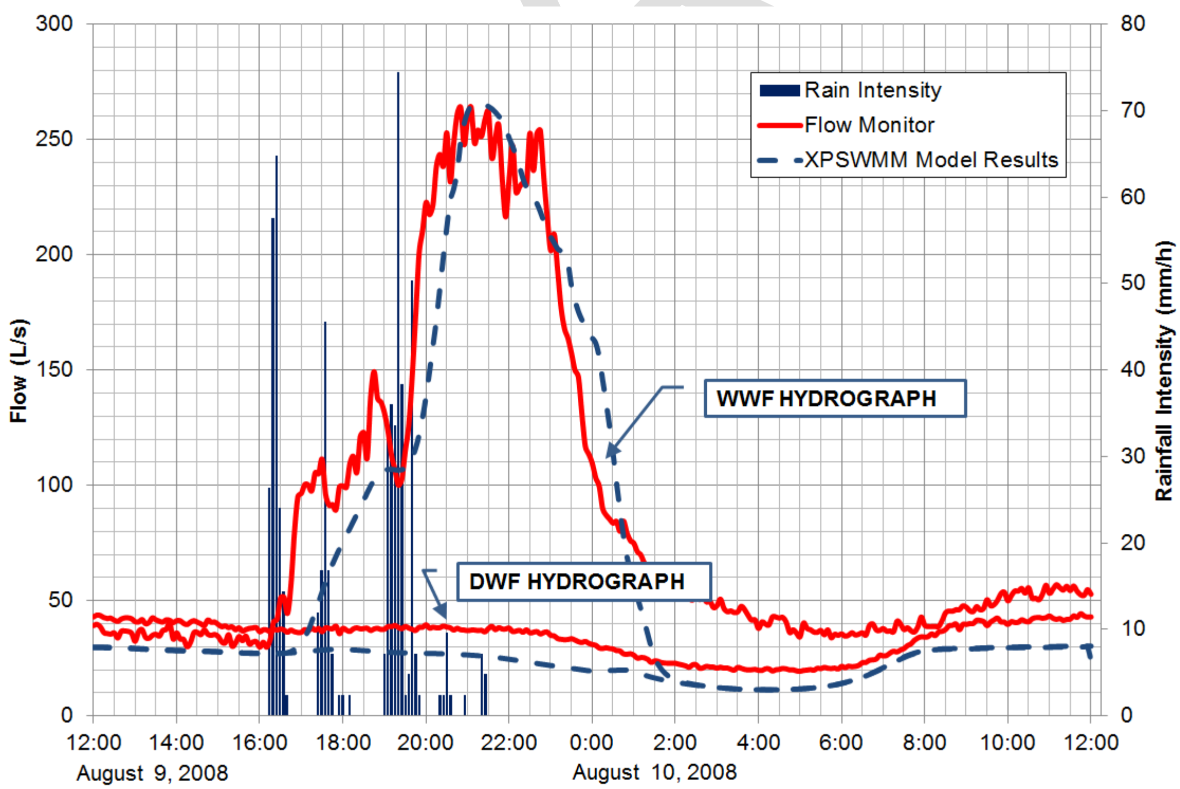

Figure 5.3: Site #3 Hydrograph

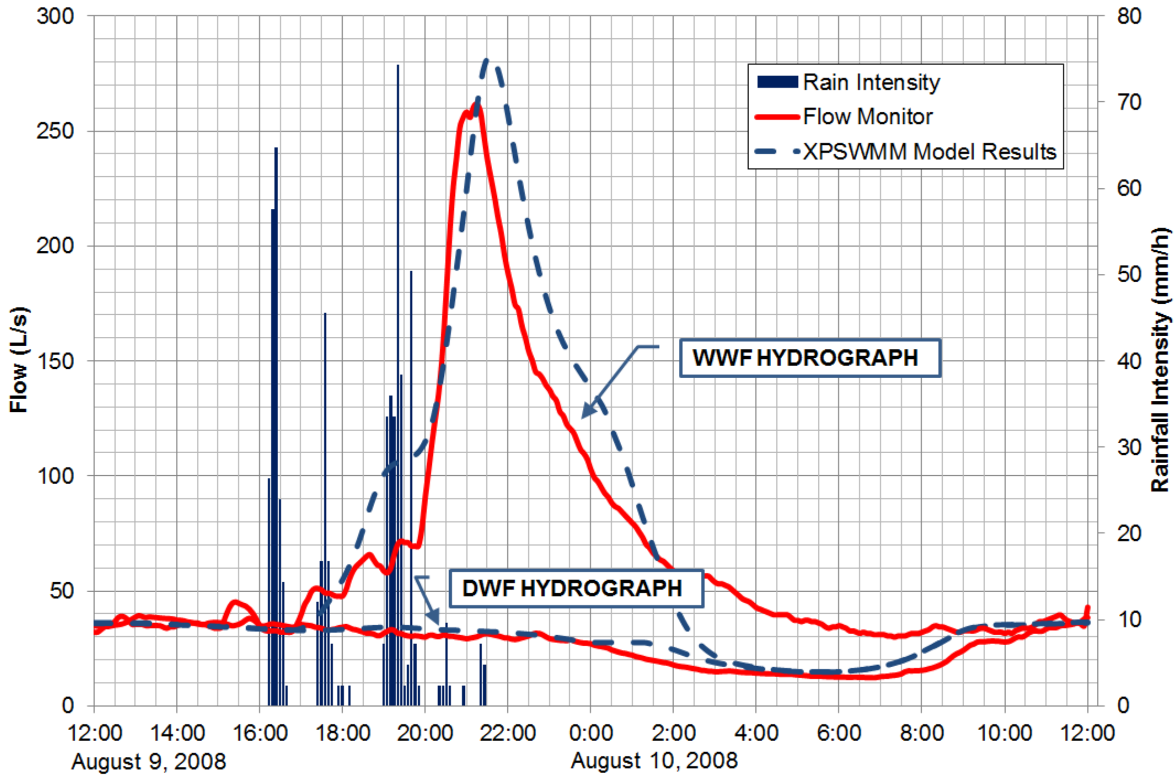

Figure 5.4: Site #4 Hydrograph

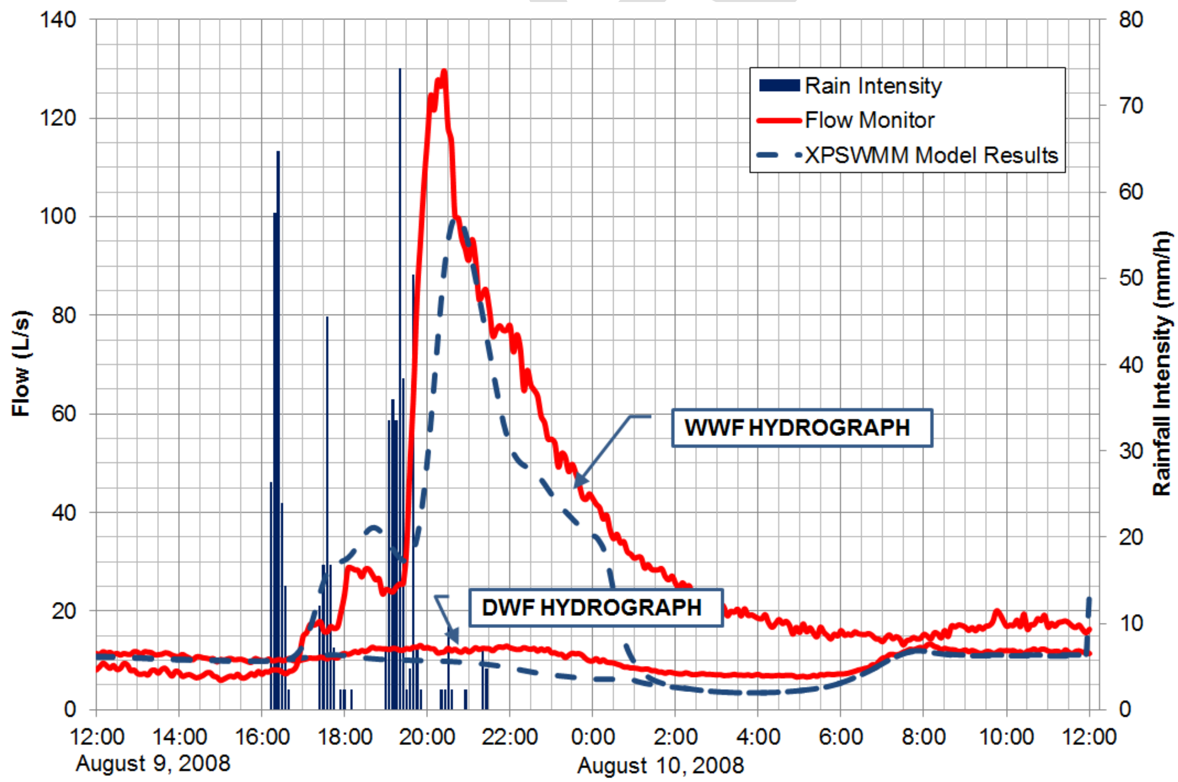

Figure 5.5: Site #5 Hydrograph

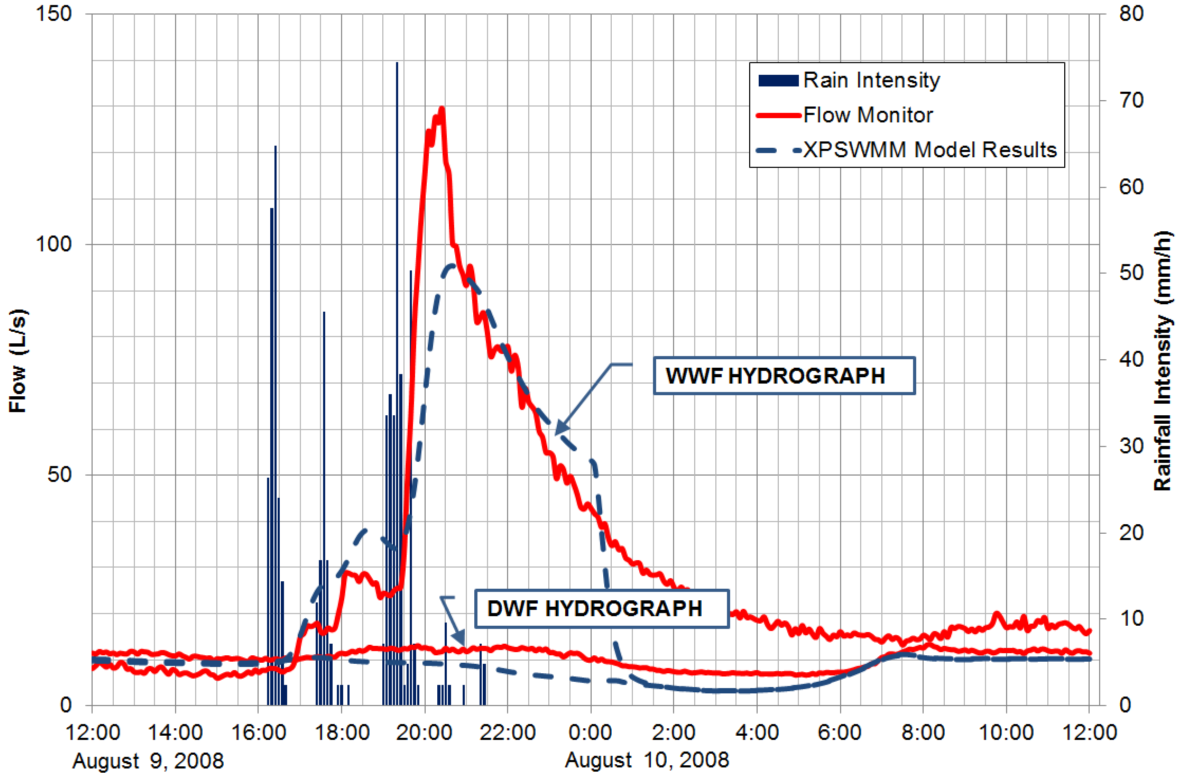

Figure 5.6: Site #6 Hydrograph

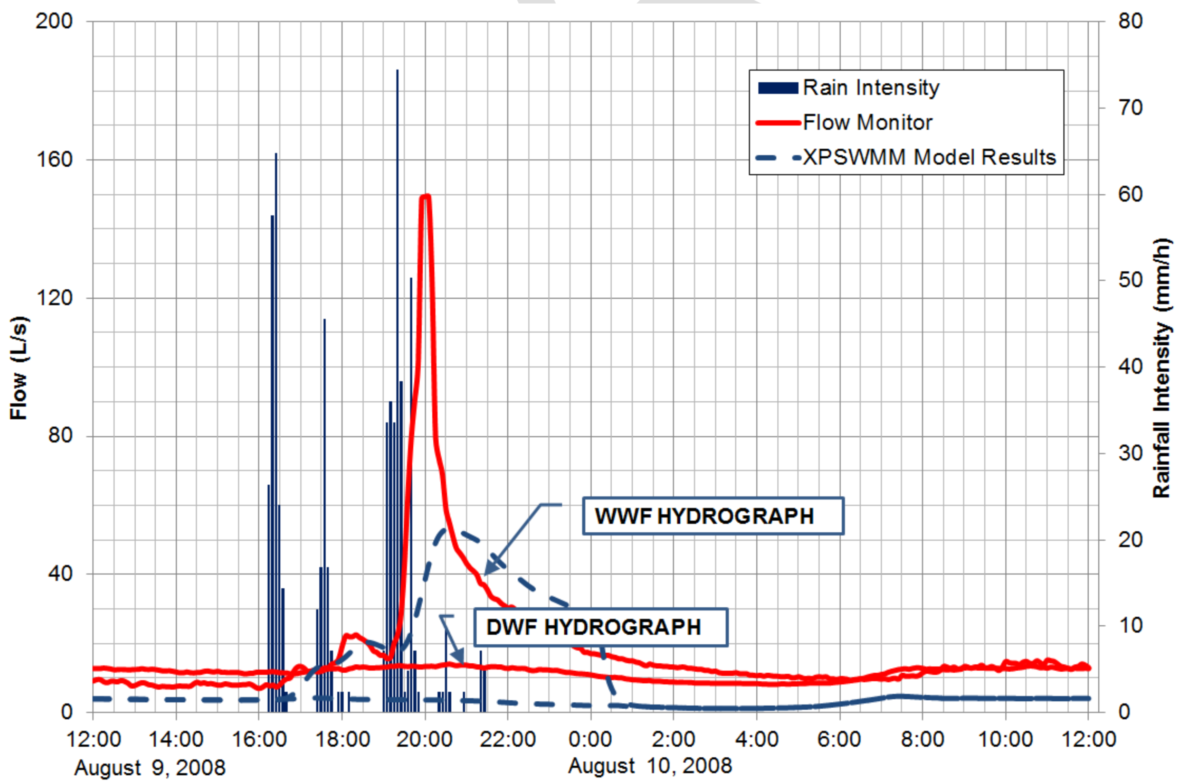

Figure 5.7: Site #7 Hydrograph

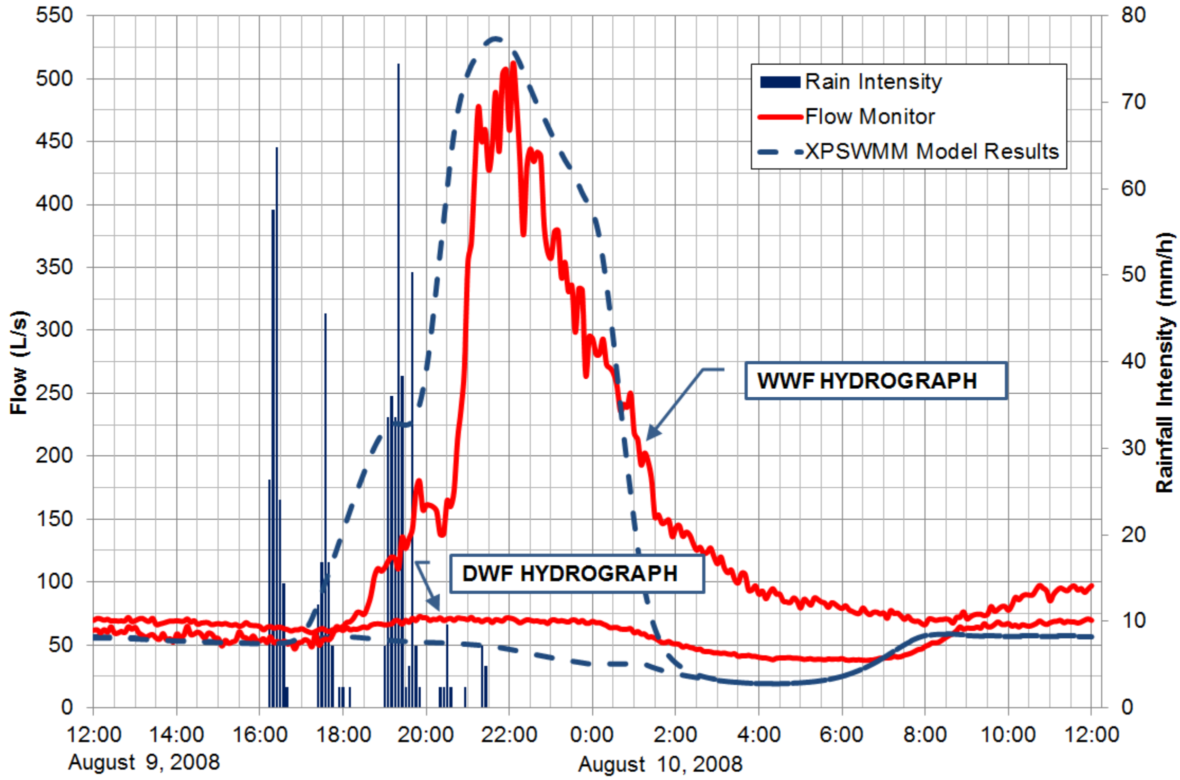

Figure 5.8: Site #8 Hydrograph

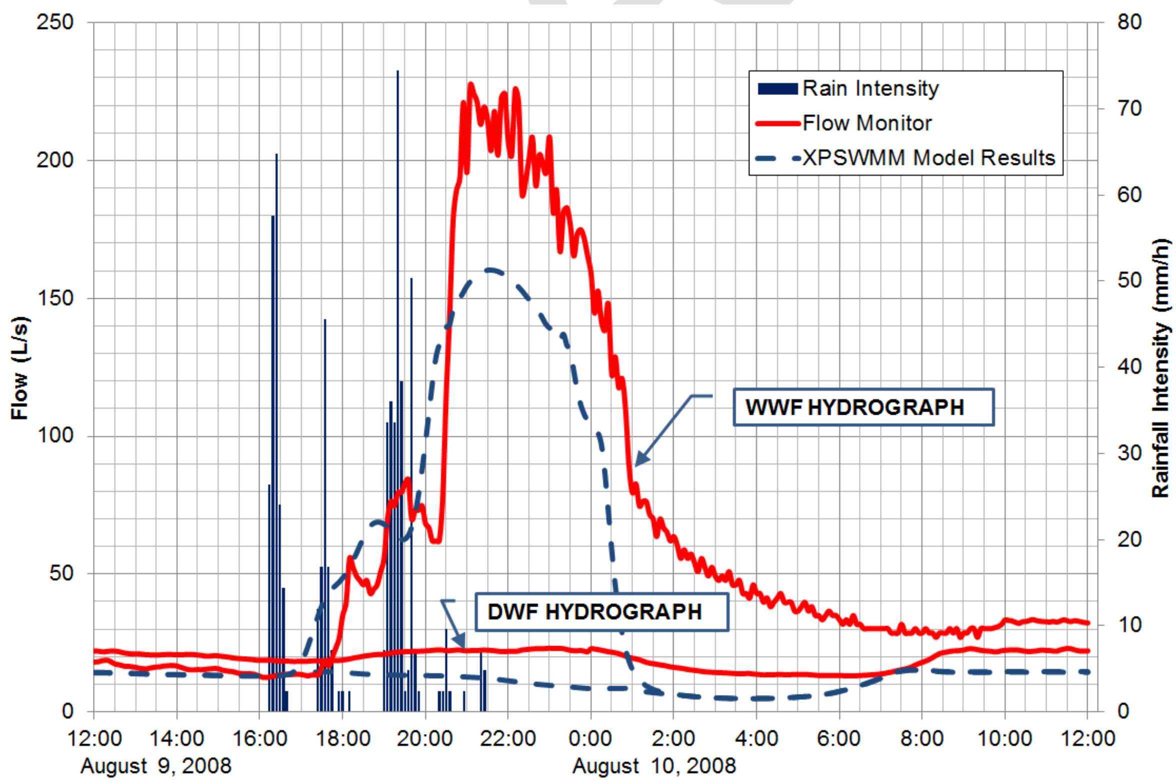

Figure 5.9: Site #9 Hydrograph

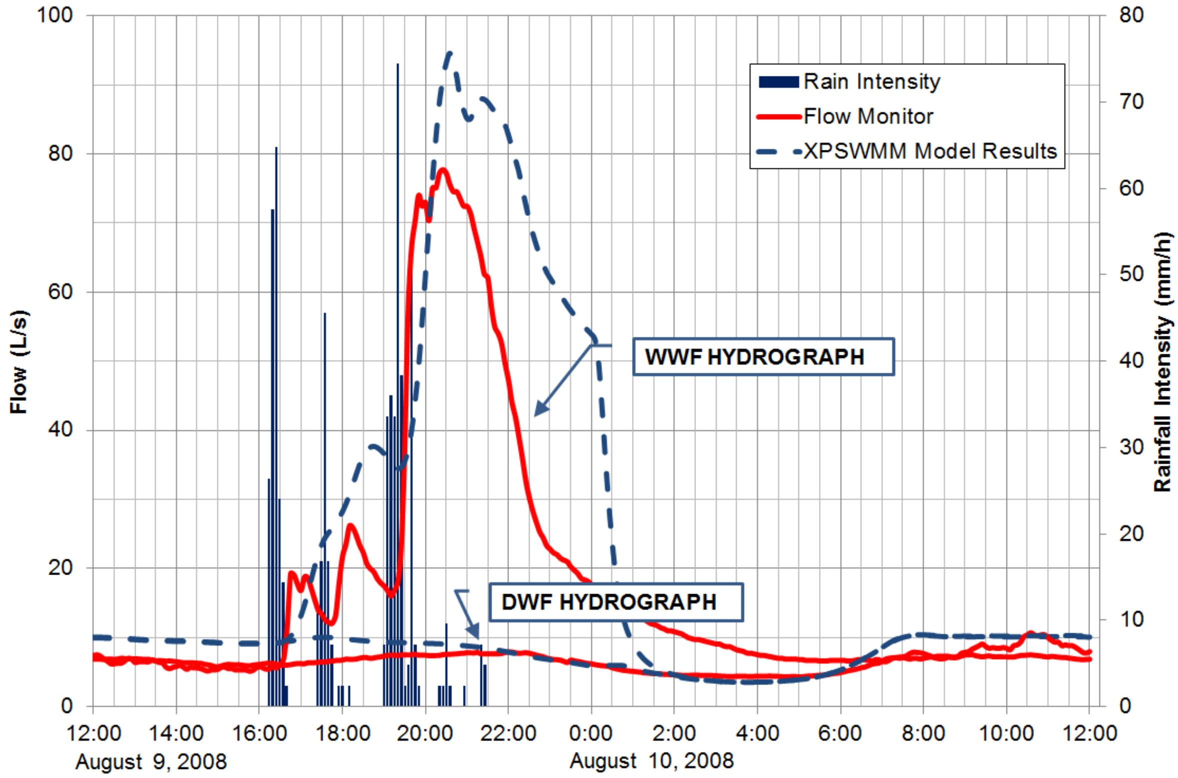

Figure 5.10: Site #10 Hydrograph

In reviewing the model calibration presented in the preceding figures, it is noted that a poorer fit is achieved in a portion of the flow monitor sites. The calibration of a model of this scale is difficult. It was determined that the overall best fit of all the flow monitor sites was more important than a best fit for each individual site. As well, some of the poorer fits could be due to flow monitor error or other unexplainable consequences. Considering the data set available, AECOM concluded that there is not sufficient evidence to further manipulate the calibration data upstream of the individual flow monitoring sites.

In assessing the fit of the simulation to the measured data, the important features to consider are the rising limb and peak of the WWF hydrograph, as this is the most critical in the WWF simulation. After the peak flow has occurred (and possibly overwhelming the system), the recession limb is less important as the flow in the system is subsiding.

It is important to note that this analysis should be considered an initial calibration and the beginning of an ongoing exercise. The model has been calibrated to a single observed WWF event. As future WWF events are captured, with rainstorms of varying magnitude (rainfall depth, intensity, duration), the model calibration will become more robust, lending increased confidence in simulating critical WWF events and recommending upgrades. The present 2015 flow monitoring program will contribute to this increased confidence, as well as subsequent flow monitoring programs in future years.

6. Establish Level of Service Criteria

After the model has been calibrated to flow monitoring data, the performance of the existing system can be assessed. In assessing the system performance, it is vital to establish the acceptable Level of Service to measure the system performance. The Level of Service criteria has two components – the design rain event and the performance characteristics of the sanitary sewer.

6.1 Design Rain Event

First, the design rain event is selected for completing the model simulations. The design rain event can be either a theoretical storm distribution of a selected return period or a historical event that approximates a return period of sufficient severity. A design rain event with a 25 year return period is commonly used in other municipalities; a lesser return period is often seen as occurring too often while a greater return period may result in performance assessments and upgrade options that are too costly to reasonably implement.

The August 9, 2008 storm yielded 42.2 mm of rainfall, measured at the West Reservoir. The West Reservoir rain gauge data approximated a 5 year return period and was used to calibrate the XPSWMM model (described earlier). This level of return period is too minor to consider as a design rain event.

The August 23 to 25, 2005 storm is the most severe storm that has occurred in recent history. There was 134.9 mm over the three day period and 98.9 mm of rainfall on August 24 alone. By comparison, the 100 year 24 hour storm yields 76.8 mm. By extrapolating the IDF data, this event is roughly approximated to a 500 year return period. Although this storm is much more severe than can be reasonably evaluated for the performance of the sanitary sewer, the records of basement flooding provides a valuable indication of possible restrictions in the sewer system.

Another consideration in selecting the design rain event is the duration of the storm. Shorter durations (less than 2 hours) often have a high rainfall intensity that produces a high amount of surface runoff, which in turn results in a small RDII response in the sanitary sewer. Longer durations (more than 12 hours) typically have a smaller rainfall intensity that is sustained over a number of hours. There may be a large volume of RDII, but the RDII flow is too small to generate a critical response in the sanitary sewer. The August 9, 2008 storm had a duration of approximately 4 hour 10 minutes, which is considered satisfactory.

Taking into account the above discussion, it was determined to establish the design rain event by scaling the August 9, 2008 rainfall distribution to a 25 year and 100 year return by multiplying the normalized rainfall intensity by the total rainfall depth (49.8 mm and 60.3 mm, respectively).

The following section on the performance of the existing system presents results for both the 25 year and 100 year design rain events, which is meant to initiate discussion to select the appropriate design rain event. The chosen design rain event will then be implemented in subsequent simulations.

6.2 Sanitary Sewer Performance Characteristics

A sanitary sewer system is typically considered to be overwhelmed when the Hydraulic Grade Line (HGL) in the pipe rises to a level where basement flooding may occur. Assuming that the depths of basement are 1.8 to 2.4 m, basement flooding would occur when the HGL rises above this depth. The XPSWMM model was set up to identify nodes based on the following criteria:

- Basement flooding unlikely (HGL more than 2.4 m below surface)
- Basement flooding possible (HGL between 1.8 m and 2.4 m below surface)
- Basement flooding likely (HGL less than 1.8 m below surface)

Sanitary sewers are typically installed at a minimum of 2.9 m depth to be below the frost line. This means that the above criteria allows for some amount of pipe surcharge (more than full-flow capacity). While this is acceptable in some cases, the proportion of full-flow capacity is often used to identify restrictions in the system. A second evaluation of performance was set up as the percentage of full-flow capacity:

- Peak flow at less than 90% of full-flow pipe capacity
- Peak flow approximately at 90% - 150% of full-flow pipe capacity
- Peak flow at more than 150% of full-flow pipe capacity

7. Existing System Performance

The XPSWMM model was simulated with the 25 year and 100 year design rain event. The performance of the sanitary sewer system in the majority of the city was satisfactory. In the 25 year storm, there was only possible basement flooding predicted, with the HGL approaching a depth of 1.9 m below ground. As expected, the severity and extent of basement flooding increased from the 25 year to the 100 year design rain event. The likelihood of basement flooding increased, with the HGL approaching to a depth of less than 1.0 m in a number of locations.

The occurrence of basement flooding was isolated to an area approximately bound by 45 Avenue to the east, 36 Street to the south, 50 Avenue to the west, and 52 Street to the north. It is noted that the August 23 to 25, 2005 rain event resulted in basement flooding in approximately the same area. The occurrence of basement flooding for this area for both the 25 year and 100 year storms are shown below in Figure 7.1.

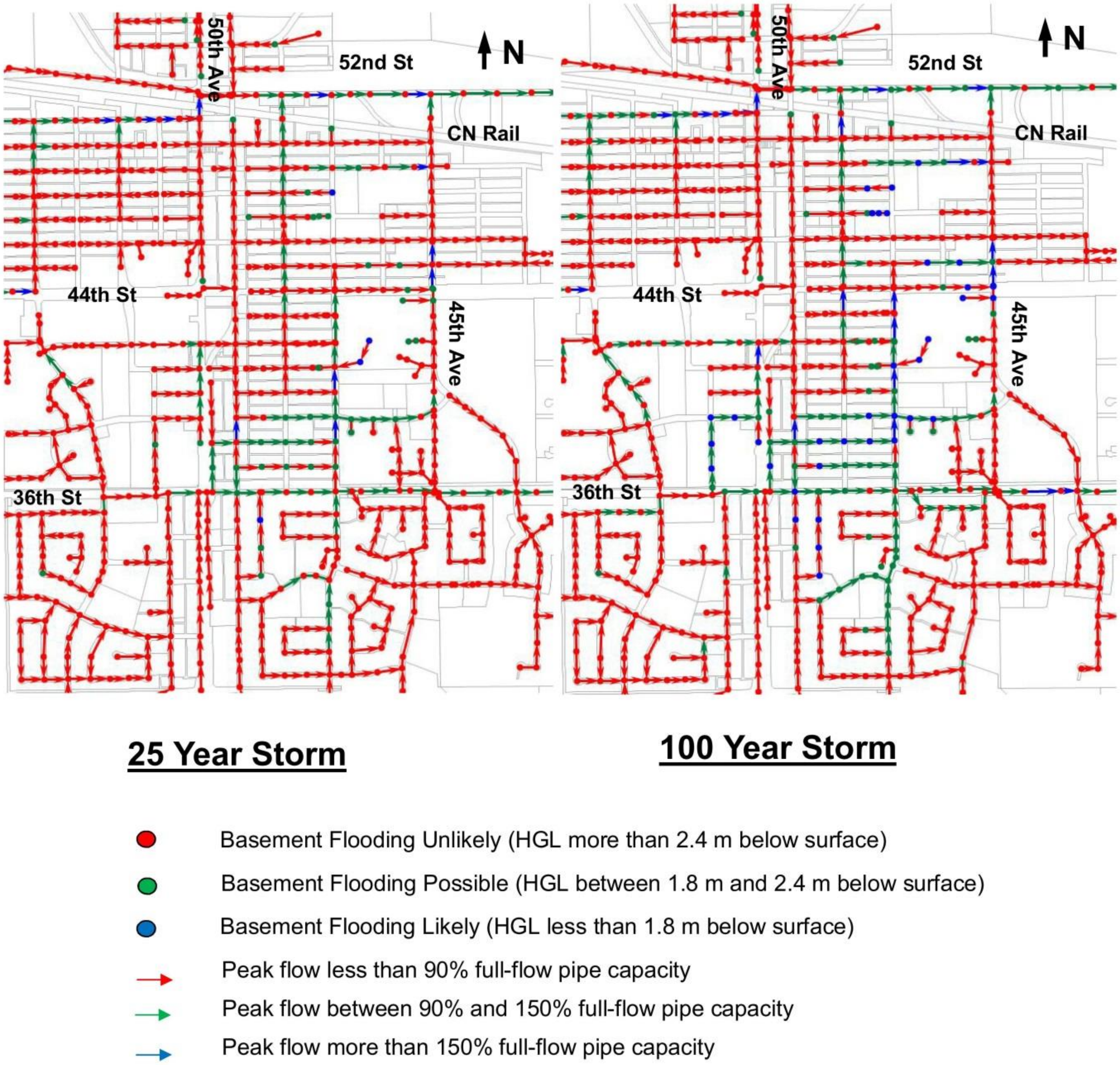

Figure 7.1: XPSWMM Model Simulation Results (25 year and 100 year Storm)

There are a number of pipes throughout the city shown to be overcapacity and surcharged. However, in locations where there is not likelihood of basement flooding, the surcharge should not be a concern, considering the depth of the HGL.

The 25 year and 100 year storm HGL profiles for a number of important trunk sewers are shown below in Figure 7.2 through Figure 7.6.

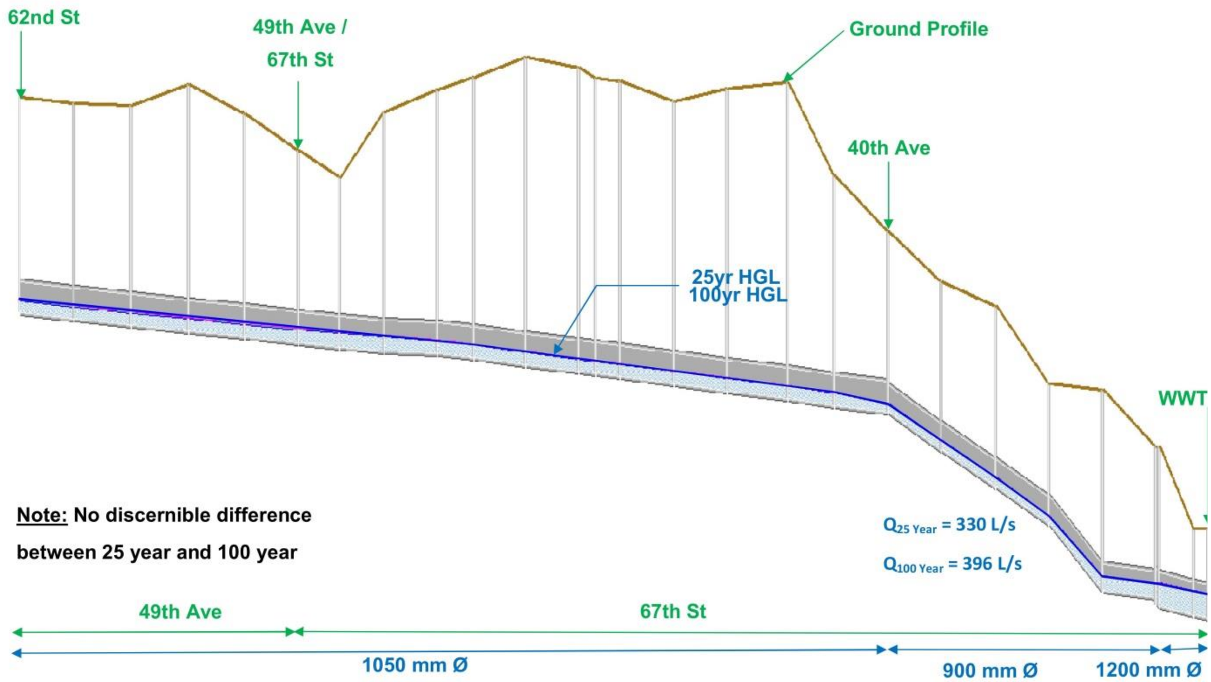

Figure 7.2: North Trunk - Hydraulic Grade Line Profile

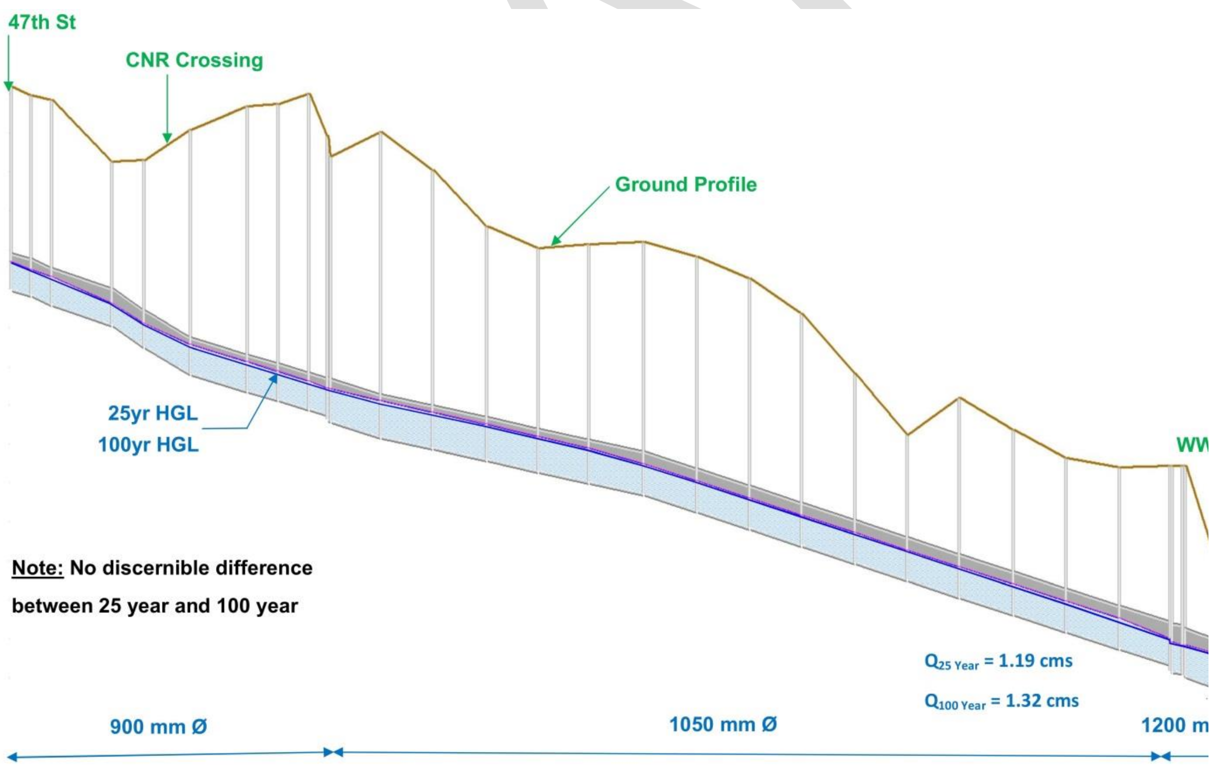

Figure 7.3: East Trunk – Hydraulic Grade Line Profile

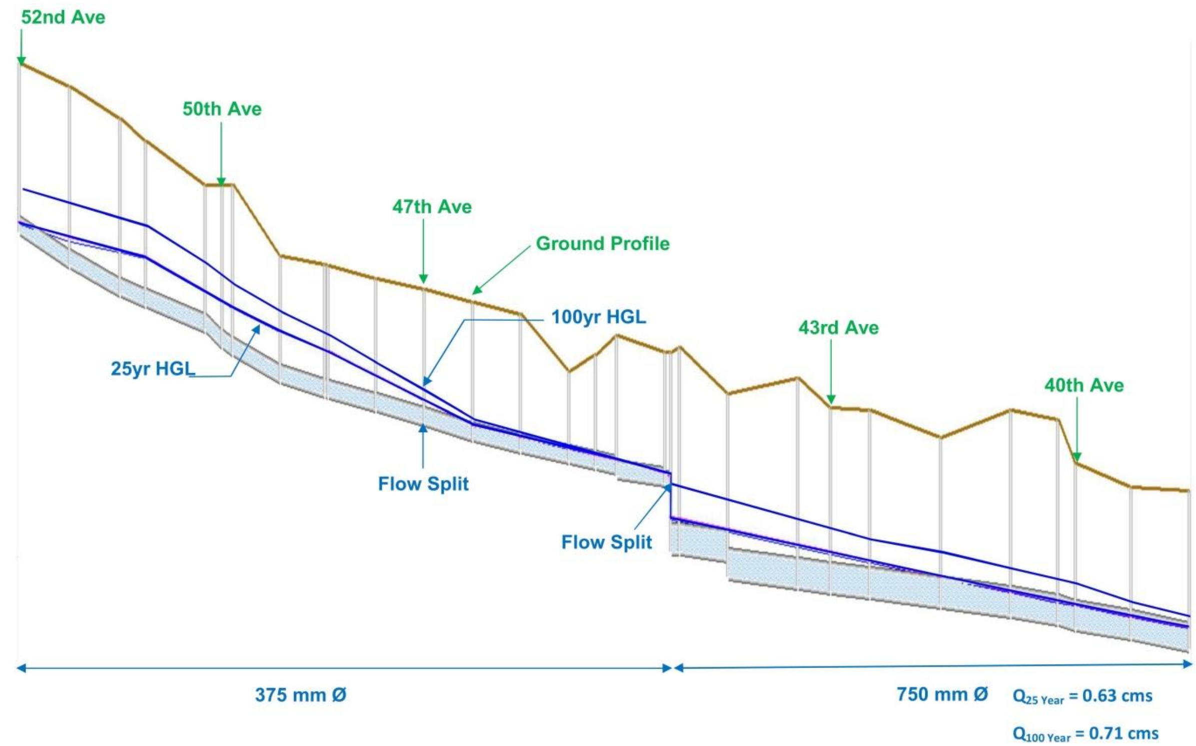

Figure 7.4: 52 Street Trunk – Hydraulic Grade Line Profile

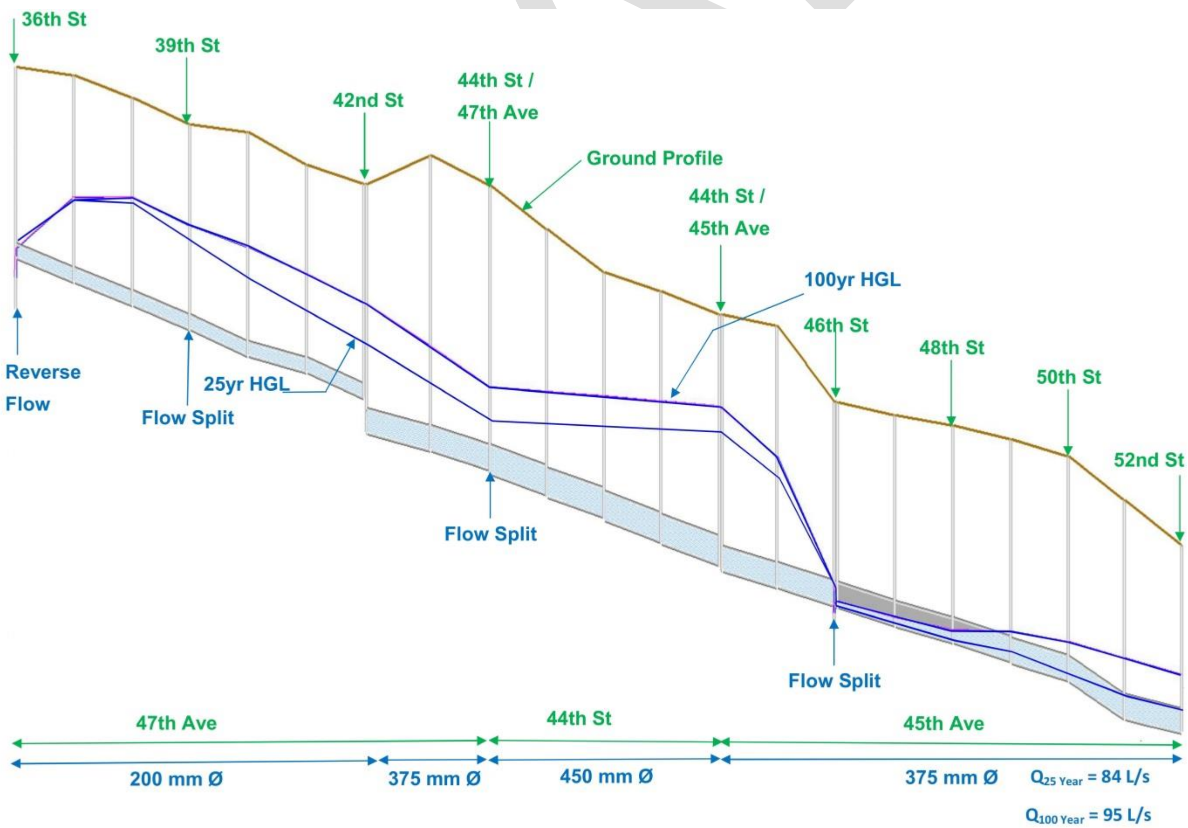

Figure 7.5: 47 Avenue/44 Street/45 Avenue Trunk – Hydraulic Grade Line Profile

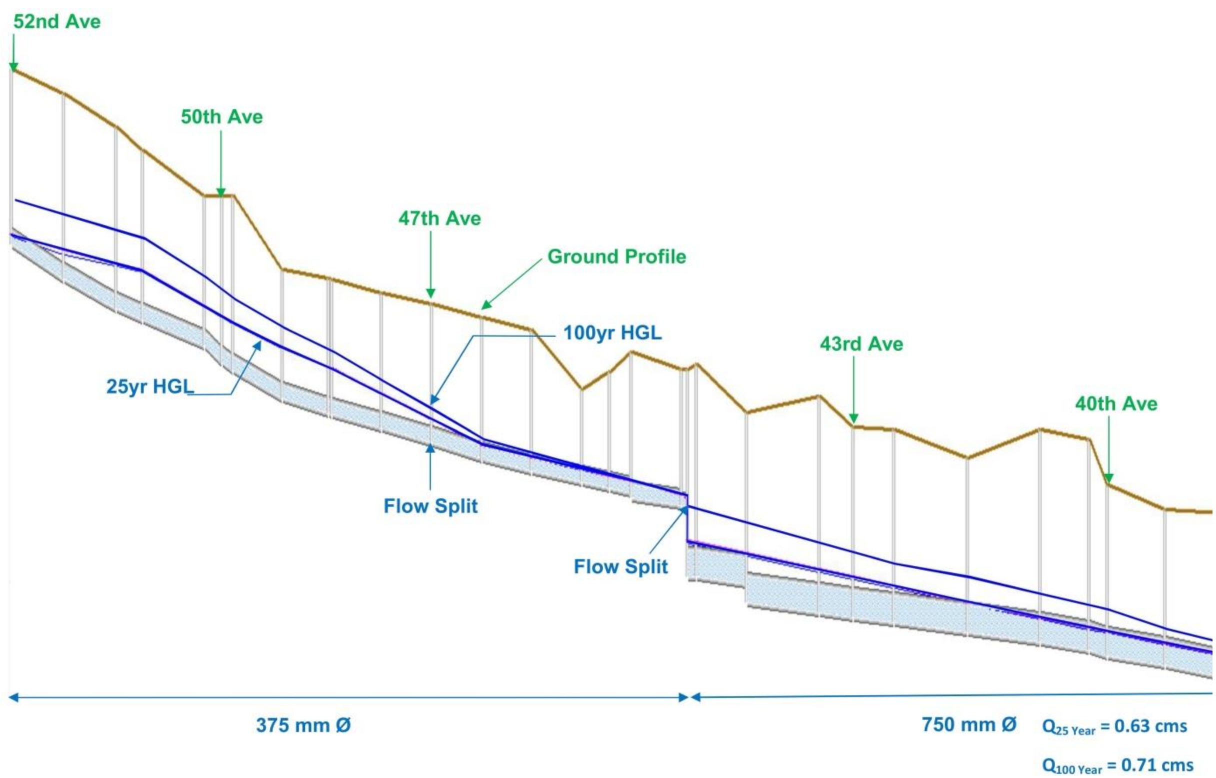

Figure 7.6: 36 Street Trunk – Hydraulic Grade Line Profile

After assessing the performance of the sanitary sewer system and identifying problematic areas, the next step is to analyze different upgrade options to alleviate the concerns. However, the Level of Service criteria, specifically design rain event and performance characteristics, must be defined to guide the evaluation.

8. Summary

The preceding technical memorandum presented AECOM's analysis and results for the 2008 flow monitoring data analysis, model development and calibration, Level of Service discussion, and preliminary model results.

The purpose of the technical memorandum is to guide discussion with the City to arrive at an agreement on the model approach and for establishing the Level of Service criteria. This will then facilitate completion of the remainder of the Sanitary Master Plan study.

The remaining tasks to complete the Sanitary Master Plan study include:

- Determine sanitary sewer servicing concepts for the future growth areas
- Develop upgrades required for the Present, 3, 5, 10, and 20 year timelines
- Review the City Development Standards for Sanitary Sewerage Systems (Section 4) and recommend any changes
- Complete secondary model calibration after the 2015 flow monitoring program is complete (September 2015)

Appendix C

Technical Memorandum

Lloydminster Sanitary Master Plan

2015 Flow Monitoring Program and Model Calibration

November 6, 2015

AECOM
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Saskatoon, SK, Canada S7H 0V1
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Memorandum

To	Craig Anderson (City of Lloydminster)	Page	Page 1 of 36
CC	Abdelqader Abdelqader (City of Lloydminster) Ryan King, Ryan Cadieux (AECOM)		
Subject	Lloydminster Sanitary Master Plan 2015 Flow Monitoring Program and Model Calibration		
From	Jonathan Peterson (AECOM)		
Date	November 6, 2015	Project Number	60342706 (402.39)

The purpose of this technical memorandum is to present a summary of the 2015 Flow Monitoring Program and subsequent data analysis and model calibration. The data analysis and model calibration are integral components of the Sanitary Sewer Master Plan study, currently underway by AECOM.

The technical memorandum presents an overview of the 2015 Flow Monitoring Program and specific discussion for each of the flow monitor and rain gauge sites. The next sections present the analysis completed for the rainfall, Dry Weather Flow, and Wet Weather flow data sets and subsequent model calibration. The memorandum is concluded with recommendations for future flow monitoring.

1. Overview – 2015 Flow Montoring Program

The flow monitoring program was implemented by SFE Global (SFE) of Edmonton, Alberta during the summer months of 2015 (May 6 to September 9). The intention of the flow monitoring program was to acquire a robust data set of dry weather flow and capture significant wet weather flow events that occur during severe summer rainstroms.

The flow monitoring program consisted of ten flow meters (ISCO 2150 Area Velocity Flow Meters) and three tipping bucket rain gauges (Telog RG-32). All equipment is owned by the City. The flow meters and rain gauges were configured to log data on 5 minute intervals.

The flow meters and rain gauges were located strategically throughout the City, as shown in Figure 1. The rationale for determining the flow meter locations were to measure flow in the major trunk sewers and to sample sewersheds of different sizes and land-uses (residential, commercial, industrial). The rain gauges were located in the northern, western, and southern edges of the city.

A description of each of the individual flow monitor (FM) and rain gauge (RG) sites follows below.

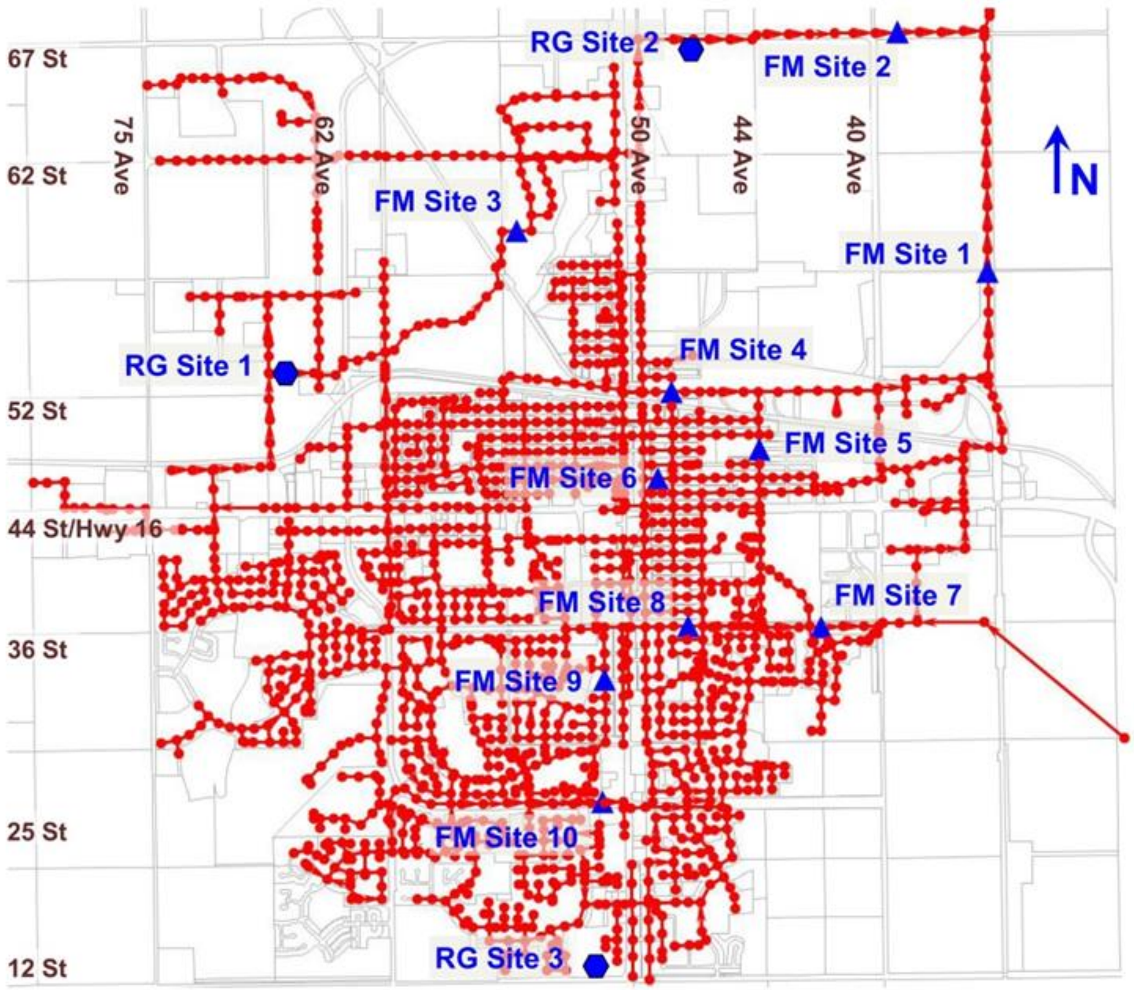

Figure 1.1: Overall Plan of Flow Monitor (FM) and Rain Gauge (RG) Sites

1.1 Flow Monitor (FM) Site 1

FM Site 1 was located on the East Trunk, approximately 1.6 km upstream of the WasteWater Treatment Plant. The general characteristics of FM Site 1 are shown below in Table 1.1 and its location and upstream sewershed area are shown in Figure 1.2.

Table 1.1: FM Site 1 General Characteristics

Manhole ID	999
Pipe ID	148
Pipe Diameter	1050 mm
Sewershed Area	772 ha
Proportion Residential	61%
Proportion Commercial/Industrial/Institutional	39%

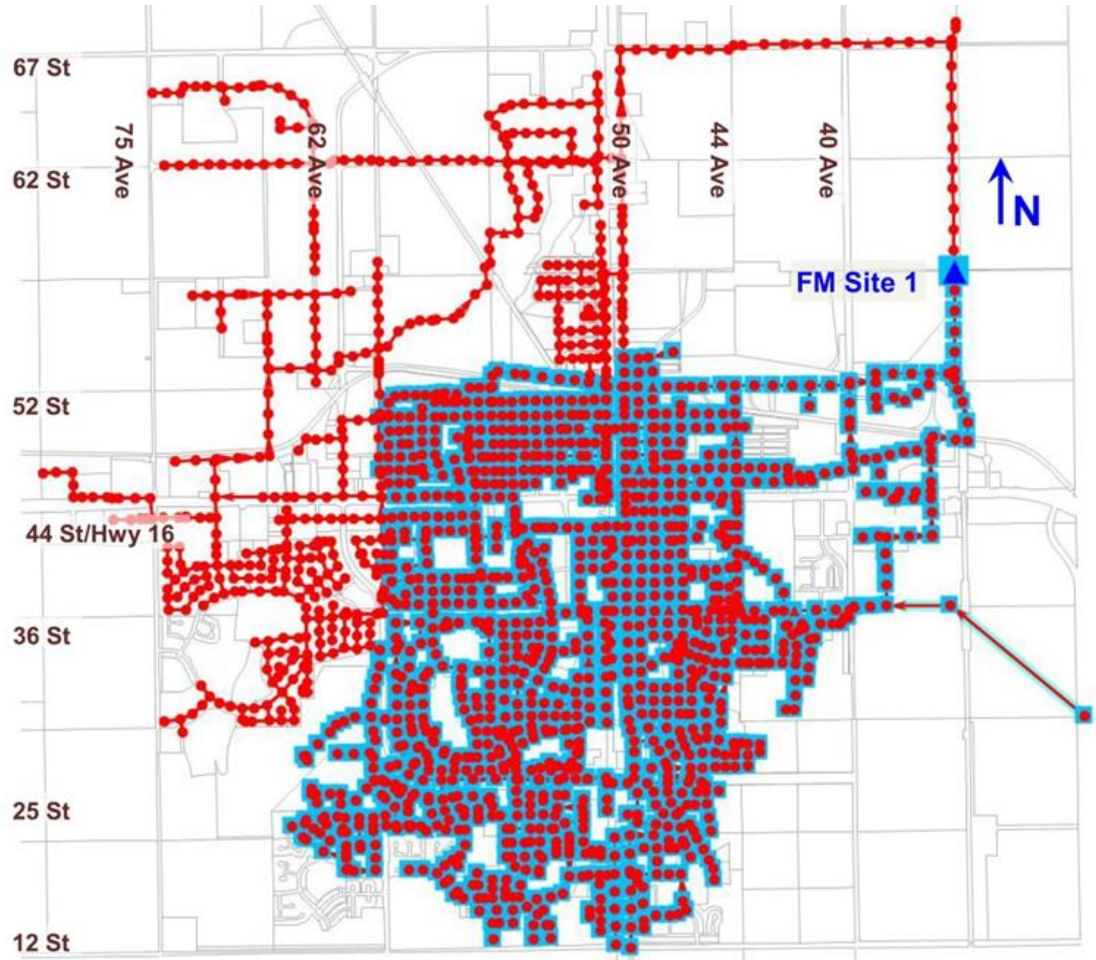

Figure 1.2: Flow Monitor Site 1 Location and Upstream Sewershed

There were no issues reported with FM Site 1 and the data appears sound.

1.2 Flow Monitor (FM) Site 2

FM Site 2 was located on the North Trunk in the north ditch of 67 St, just east of 40 Ave. The general characteristics of FM Site 2 are shown below in Table 1.2 and its location and upstream sewershed area are shown in Figure 1.3.

Table 1.2: FM Site 2 General Characteristics

Manhole ID	1034
Pipe ID	101
Pipe Diameter	900 mm
Sewershed Area	302 ha
Proportion Residential	28%
Proportion Commercial/Industrial/Institutional	72%

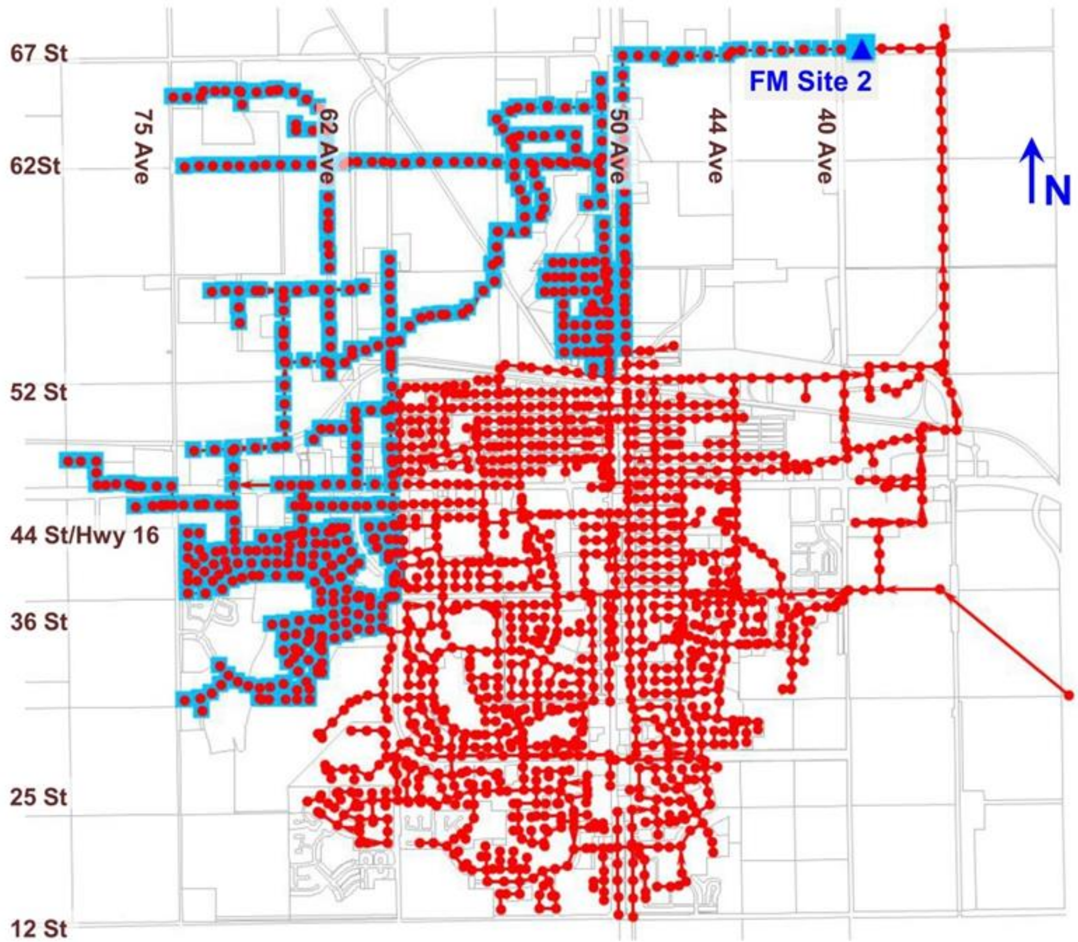

Figure 1.3: Flow Monitor Site 2 Location and Upstream Sewershed

FM Site 2 experienced issues with ragging of the sensor, which means solids suspended in the flow are caught on the flow probe and distort the readings. SFE adjusted the flow data to correct this, and the data set appears to be sound.

1.3 Flow Monitor (FM) Site 3

FM Site 3 was located on the West Trunk on 59 St west of 53 Ave. The general characteristics of FM Site 3 are shown below in Table 1.3 and its location and upstream sewershed area are shown in Figure 1.4.

Table 1.3: FM Site 3 General Characteristics

Manhole ID	882
Pipe ID	999
Pipe Diameter	900 mm
Sewershed Area	198 ha
Proportion Residential	34%
Proportion Commercial/Industrial/Institutional	66%

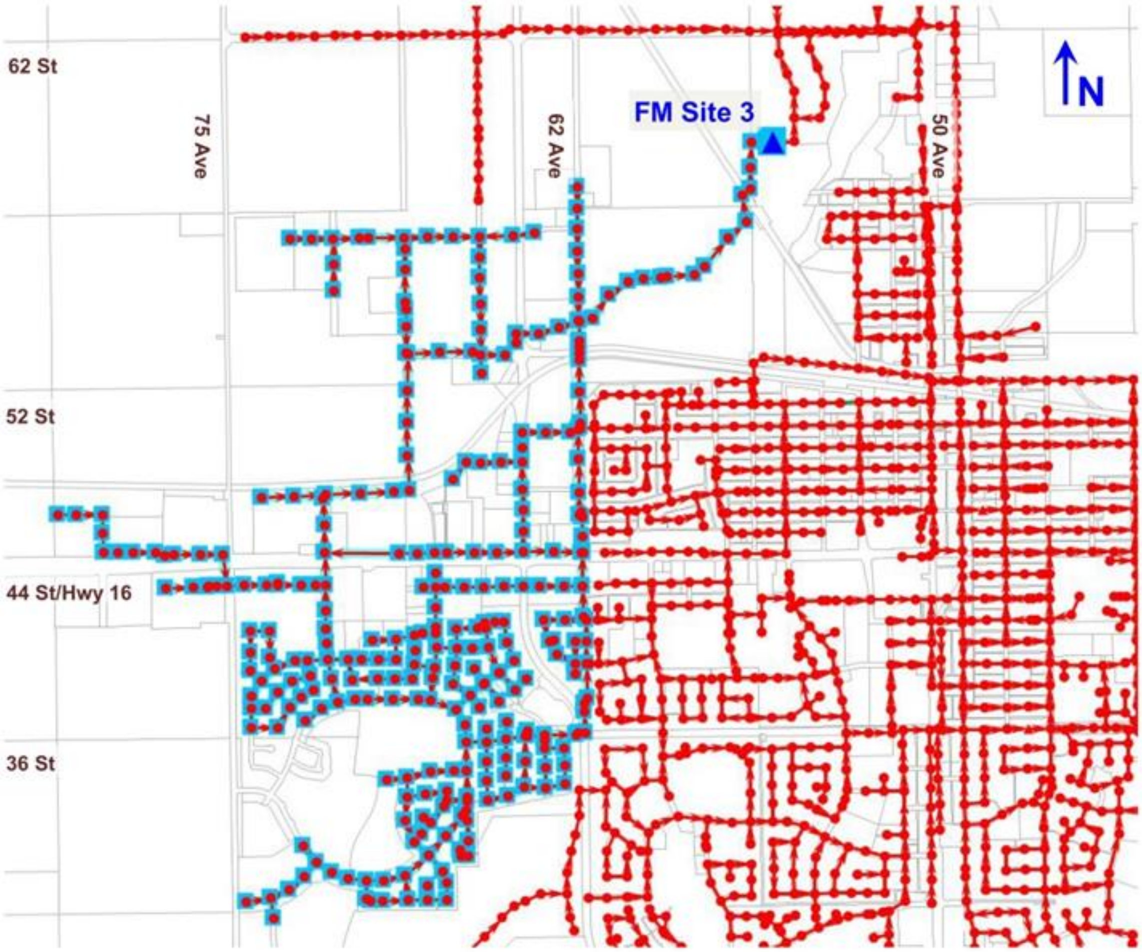

Figure 1.4: Flow Monitor Site 3 Location and Upstream Sewershed

Ragging of the sensor was reported at times for FM Site 3. There is noise observed in the data, but overall the data set appears to be sound.

1.4 Flow Monitor (FM) Site 4

FM Site 4 was located in the intersection of 52 St and 48 Ave. The general characteristics of FM Site 4 are shown below in Table 1.4 and its location and upstream sewershed area are shown in Figure 1.5.

Table 1.4: FM Site 4 General Characteristics

Manhole ID	30
Pipe ID	1369
Pipe Diameter	300 mm
Sewershed Area	38.2 ha
Proportion Residential	22%
Proportion Commercial/Industrial/Institutional	78%

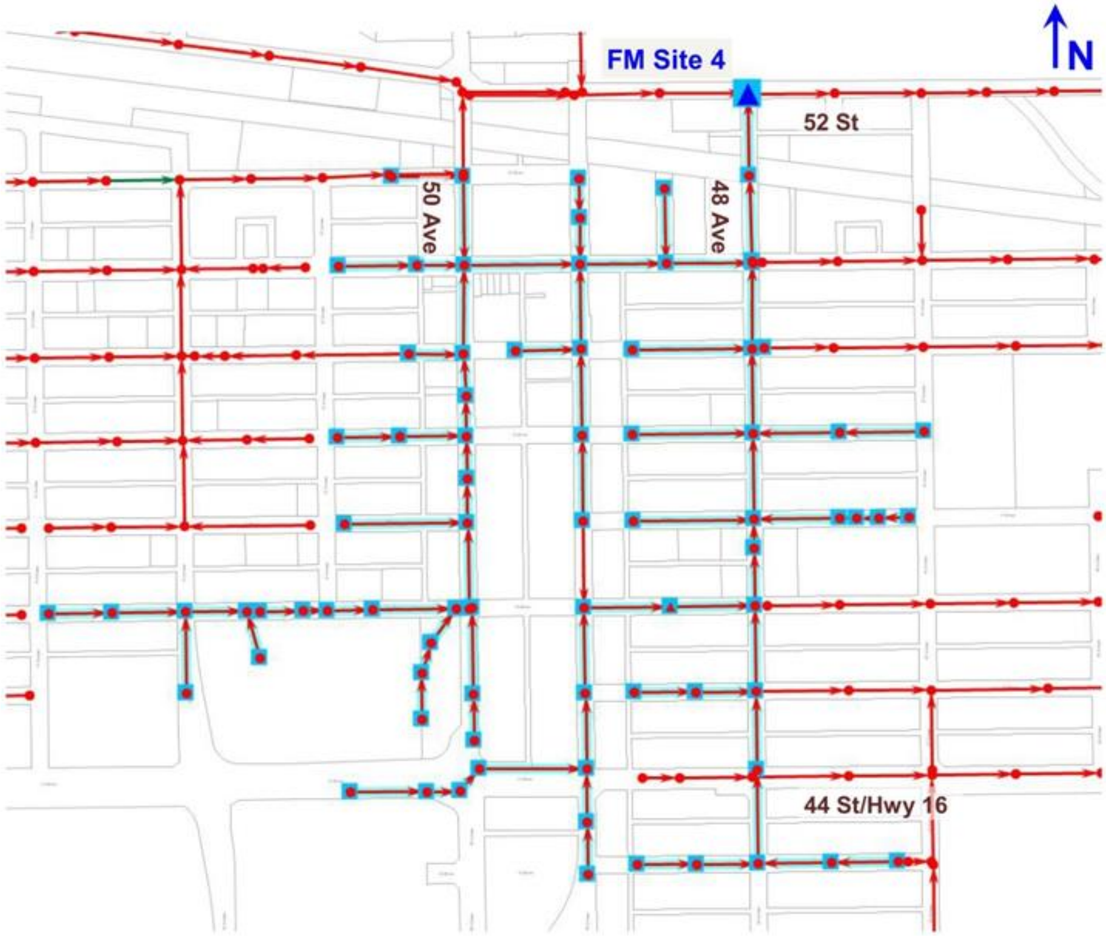

Figure 1.5: Flow Monitor Site 4 Location and Upstream Sewershed

FM Site 4 experienced frequent issues with ragging and silting of the sensor. This was compounded by the low depth of flow in the sewer, which limited the degree that the sensor could be offset from the pipe invert. Also, the flow data would tend to decrease over time until the next maintenance visit, when the sensor would be cleaned and readjusted. The amount of sound data available was limited to the first two weeks of July.

1.5 Flow Monitor (FM) Site 5

FM Site 5 was located in the intersection of 48 St and 45 Ave. The general characteristics of FM Site 5 are shown below in Table 1.5 and its location and upstream sewershed area are shown in Figure 1.6.

Table 1.5: FM Site 5 General Characteristics

Manhole ID	36
Pipe ID	968
Pipe Diameter	375 mm
Sewershed Area	6.2 ha
Proportion Residential	100%
Proportion Commercial/Industrial/Institutional	0%

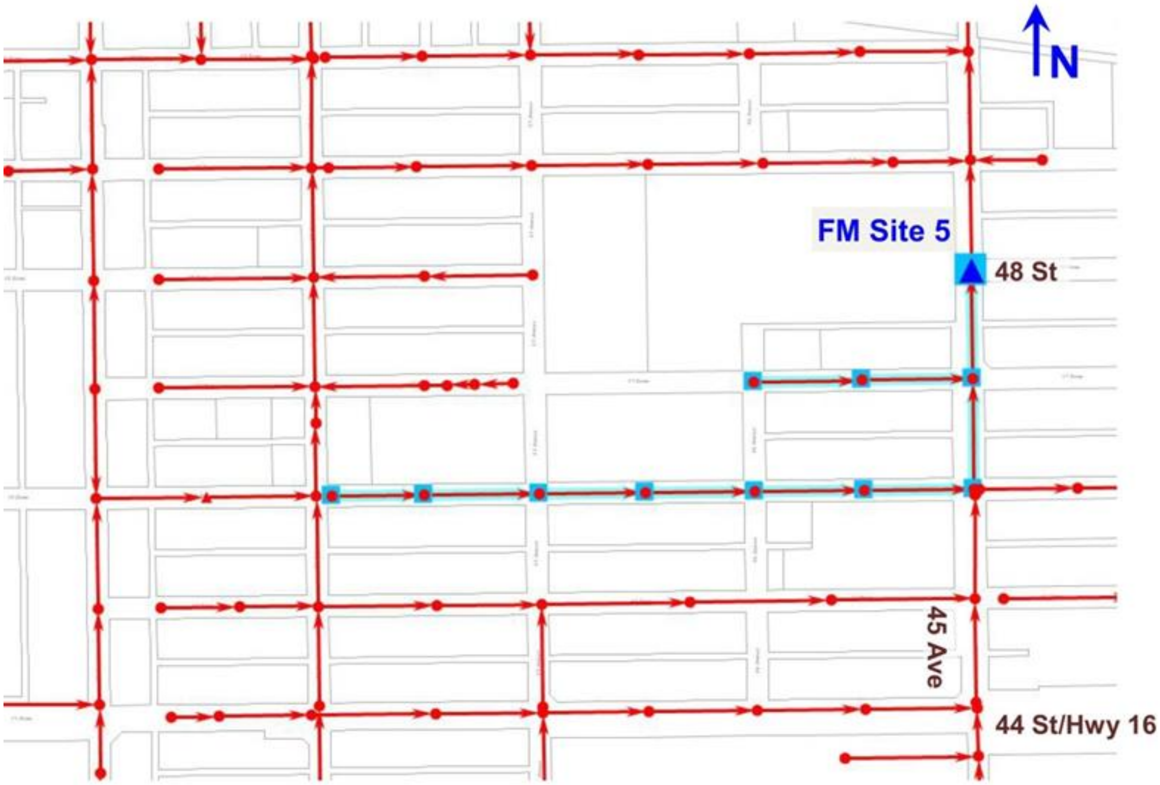

Figure 1.6: Flow Monitor Site 5 Location and Upstream Sewershed

FM Site 5 experienced ragging and silting of the sensor throughout the monitoring period. There was loss of data from May 22 to 24. There is noise observed in the data, but overall the data set appears to be sound.

1.6 Flow Monitor (FM) Site 6

FM Site 6 was located along 46 St between 48 Ave and 49 Ave. The general characteristics of FM Site 6 are shown below in Table 1.6 and its location and upstream sewershed area are shown in Figure 1.7.

Table 1.6: FM Site 6 General Characteristics

Manhole ID	138
Pipe ID	1246
Pipe Diameter	250 mm
Sewershed Area	5.1 ha
Proportion Residential	19%
Proportion Commercial/Industrial/Institutional	81%

Figure 1.7: Flow Monitor Site 6 Location and Upstream Sewershed

FM Site 6 experienced velocity drop out which was corrected by SFE. There is noise observed in the data, but overall the data set appears to be sound.

1.7 Flow Monitor (FM) Site 7

FM Site 7 was located along 36 St just east of 47 Ave. The general characteristics of FM Site 7 are shown below in Table 1.7 and its location and upstream sewershed area are shown in Figure 1.8.

Table 1.7: FM Site 7 General Characteristics

Manhole ID	975
Pipe ID	186
Pipe Diameter	750 mm
Sewershed Area	471 ha
Proportion Residential	69%
Proportion Commercial/Industrial/Institutional	31%

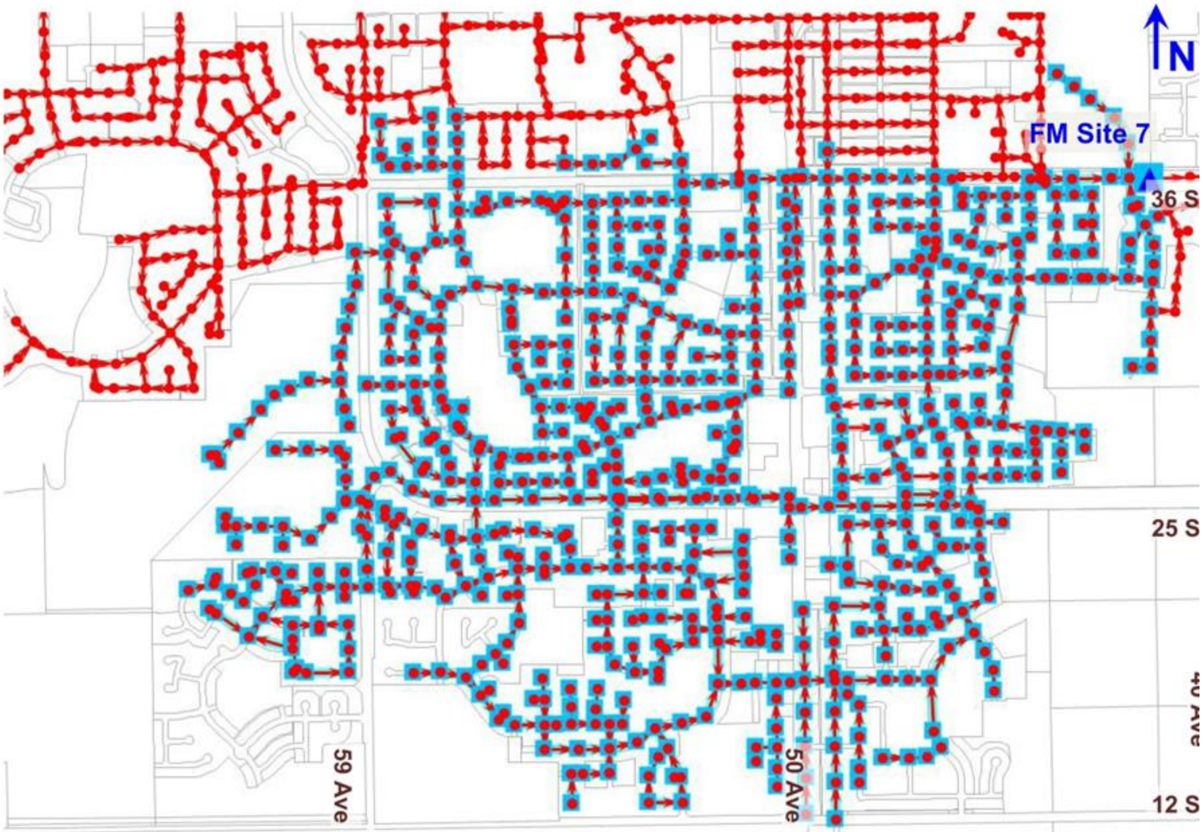

Figure 1.8: Flow Monitor Site 7 Location and Upstream Sewershed

There were no issues reported with FM Site 7 and the data appears sound.

1.8 Flow Monitor (FM) Site 8

FM Site 8 was located along 36 St between 47 Ave and 48 Ave. The general characteristics of FM Site 8 are shown below in Table 1.8 and its location and upstream sewershed area are shown in Figure 1.8.

Table 1.8: FM Site 8 General Characteristics

Manhole ID	386
Pipe ID	1043
Pipe Diameter	375 mm
Sewershed Area	136 ha
Proportion Residential	80%
Proportion Commercial/Industrial/Institutional	20%

Figure 1.9: Flow Monitor Site 8 Location and Upstream Sewershed

FM Site 8 experienced velocity drop out which was corrected by SFE. Due to battery drain, there was loss of data from June 9 to 12. Outside of these dates, the data set appears to be sound.

1.9 Flow Monitor (FM) Site 9

FM Site 9 was located in the intersection of 33 St and 51 Ave. The general characteristics of FM Site 9 are shown below in Table 1.9 and its location and upstream sewershed area are shown in Figure 1.10.

Table 1.9: FM Site 9 General Characteristics

Manhole ID	445
Pipe ID	1435
Pipe Diameter	375 mm
Sewershed Area	47.0 ha
Proportion Residential	100%
Proportion Commercial/Industrial/Institutional	0%

Figure 1.10: Flow Monitor Site 9 Location and Upstream Sewershed

FM Site 9 experienced velocity drop out which was corrected by SFE. Due to battery drain, there was loss of data from August 11 to 25. Outside of these dates, the data set appears to be sound.

1.10 Flow Monitor (FM) Site 10

FM Site 10 was located along 25 St between 50 Ave and 53 Ave. This section of sanitary sewer was recently twinned with a 600 mm diameter sewer as part of the 25th Street Sanitary Sewer Extension project and the flow meter was installed in the original 450 mm diameter sewer. The general characteristics of FM Site 10 are shown below in Table 1.10 and its location and upstream sewershed area are shown in Figure 1.11.

Table 1.10: FM Site 10 General Characteristics

Manhole ID	854
Pipe ID	213
Pipe Diameter	450 mm
Sewershed Area	131.4 ha
Proportion Residential	65%
Proportion Commercial/Industrial/Institutional	35%

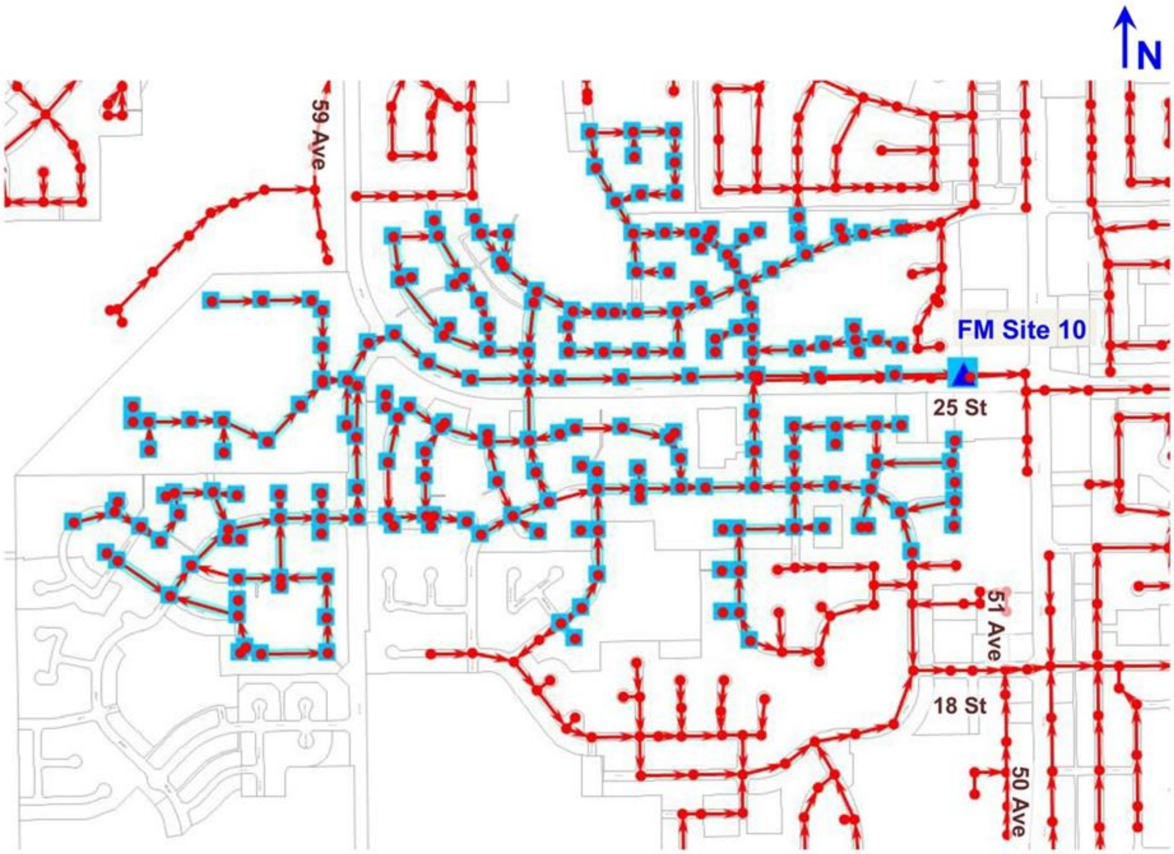

Figure 1.11: Flow Monitor Site 10 Location and Upstream Sewershed

FM Site 10 experienced velocity drop out which was corrected by SFE. There was loss of data from June 2 to 10. Outside of these dates, the data set appears to be sound.

1.11 Rain Gauge (RG) Sites

There were three rain gauge sites set up around the city, as shown on Figure 1. RG1 was set on the roof of the Civic Operations Centre, located along 52 St west of 63 Ave. RG2 was set on the roof of the Water Treatment Plan, located along 67 St east of 50 Ave. RG3 was set on the roof of the Servus Sports Centre, located at the intersection of 15 St and 51 Ave.

There were no issues reported for any of the rain gauge sites.

2. Rainfall Data Analysis

There were three significant rainfall events observed during the flow monitoring program – occurring on June 21, August 15, and September 5 to 7. The details of the discrete rainfall events are presented below in Table 2.1.

Table 2.1: 2015 Significant Rainfall Events

Date		RG1 Civic Operations Centre	RG2 Water Treatment Plant	RG3 Servus Sports Centre
June 21, 2015	Depth	15 mm	10 mm	63 mm
	Duration	2 hr 10 min	20 min	4 hr 10 min
Aug 15, 2015	Depth	61 mm	58 mm	60 mm
	Duration	12 hr 10 min	12 hr 10 min	12 hr 10 min
Sept 5-7, 2015	Depth	66 mm	70 mm	64 mm
	Duration	42 hr 15 min	44 hr 30 min	45 hr 20 min

The June 21, 2015 rainfall event was quite variable; both the rainfall amounts and storm duration were significantly different at the Civic Operations Centre and Water Treatment Plant, compared to the Servus Sports Centre. The severity of the rainfall measured at the Civic Operations Centre and Water Treatment Plant was relatively minor (less than a 2 year return period). However, the severity of the rainfall measured at the Servus Sports Centre is estimated as a 100 year return period. The June 21, 2015 rainfall event relative to the City of Lloydminster IDF curves, is presented below in Figure 2.1.

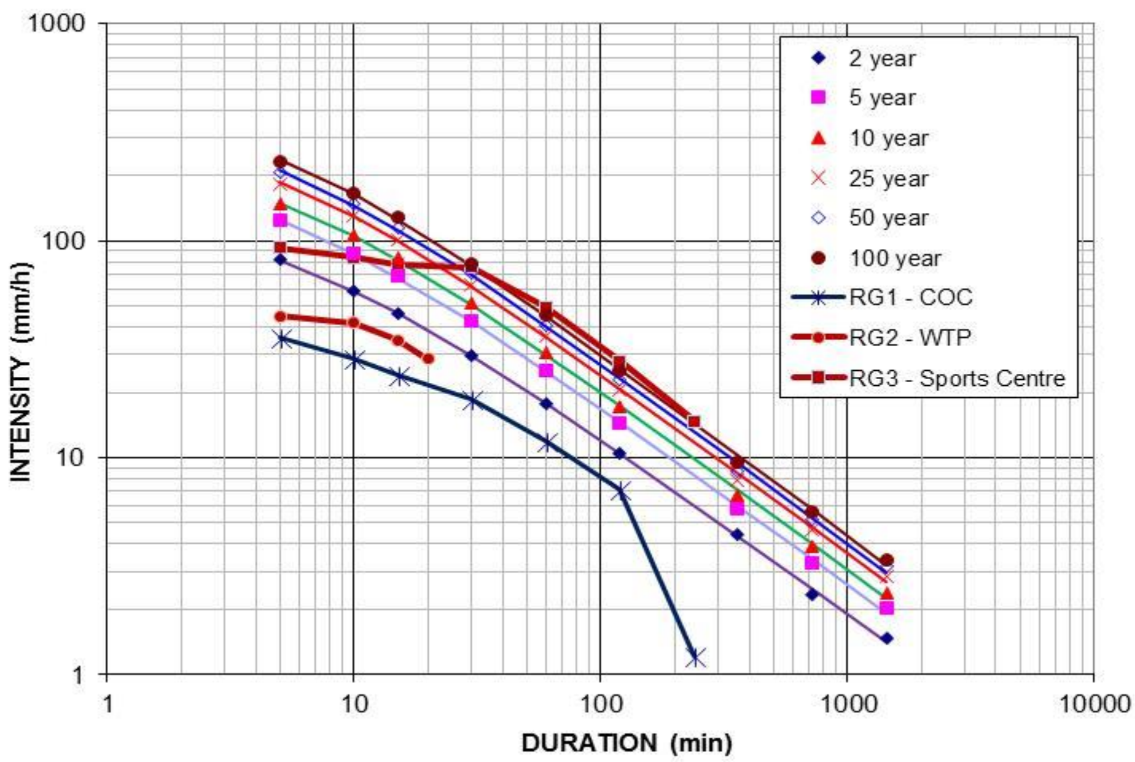

Figure 2.1: June 21, 2015 Rainfall Event

The August 15, 2015 rainfall event was consistent amongst the three rain gauge sites, measuring 58 to 61 mm depth of precipitation and all having the same duration. This rainfall event is estimated to have a 25 year return period (Figure 2.2).

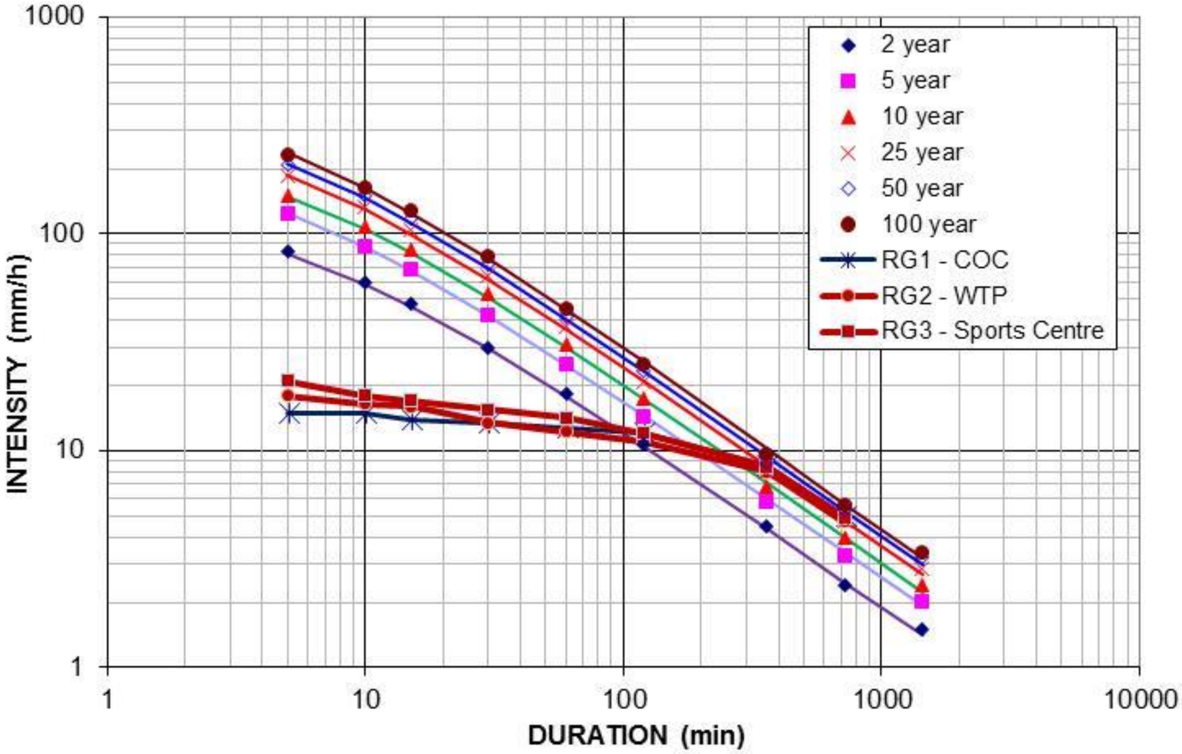

Figure 2.2: August 15, 2015 Rainfall Event

Similarly, the September 5 – 7, 2015 rainfall event had similar precipitation depths (64 to 70 mm) and storm durations amongst the three rain gauge sites. This rainfall event is estimated to have a 5 – 10 year return period (Figure 2.3).

The impact to the flow in the sanitary sewer from each of the significant rainfall events was then evaluated. The rainfall event and associated wet weather flow response in the sanitary sewer is a complex relationship, dependent on a number of factors that include antecedent moisture conditions and rainfall intensity, depth, and duration. Accordingly, each rainfall event produces a different wet weather flow response. To assist in evaluating the wet weather response for each of the significant rainfall events, the average daily flow in the sewer was plotted against the daily rainfall depths (Figures 2.4 to 2.13).

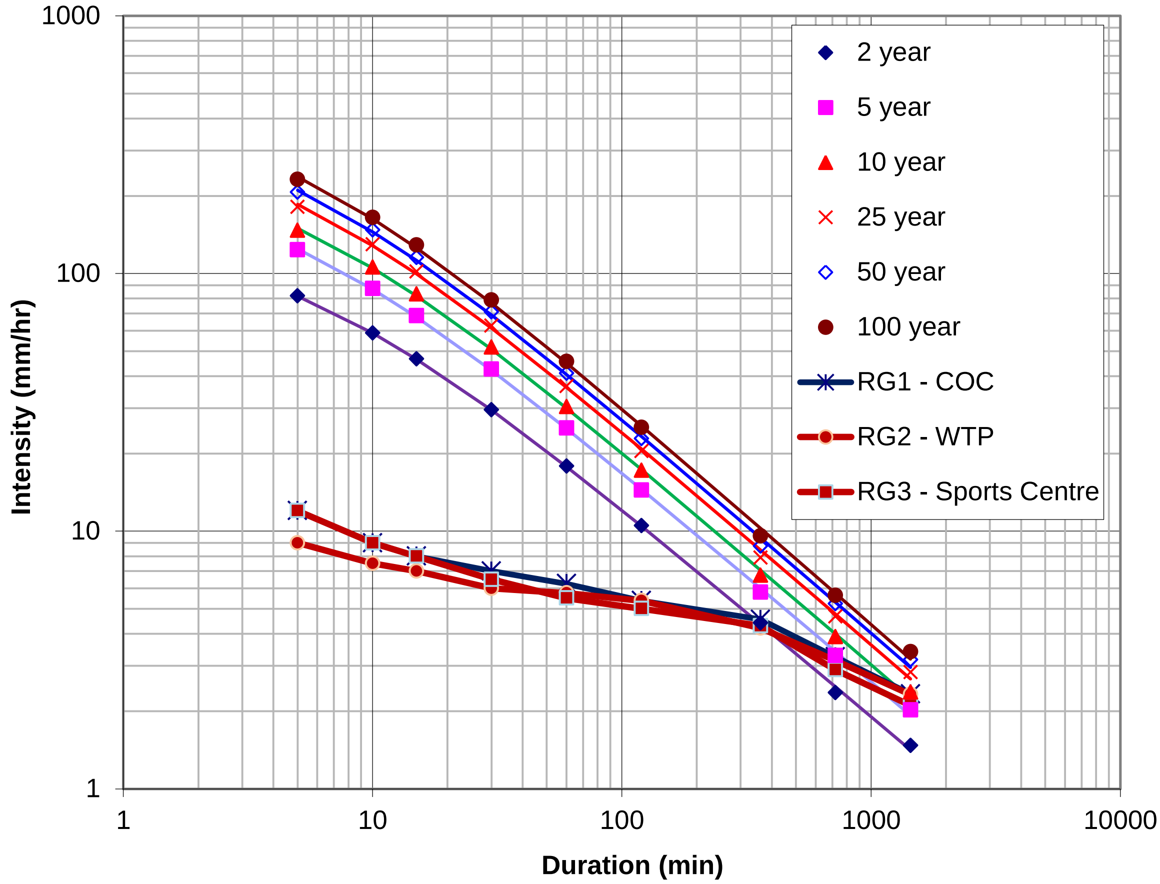

Figure 2.3: September 5 - 7, 2015 Rainfall Event

Figure 2.4: FM Site 1 – Daily Flow and Rainfall Depth

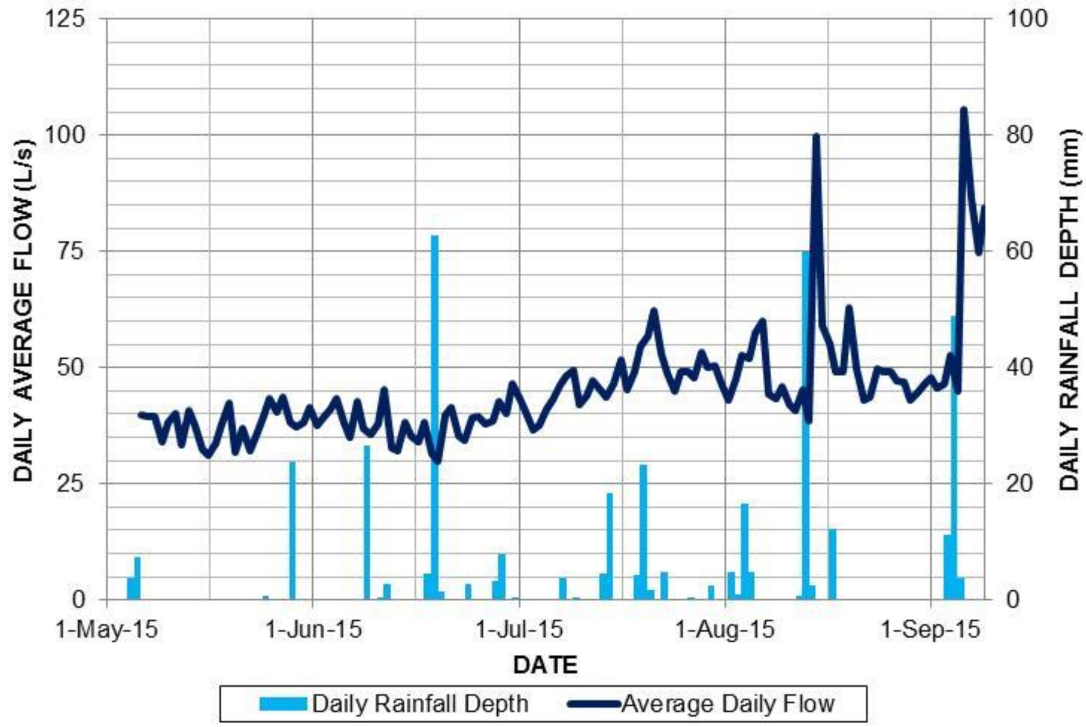

Figure 2.5: FM Site 2 – Daily Flow and Rainfall Depth

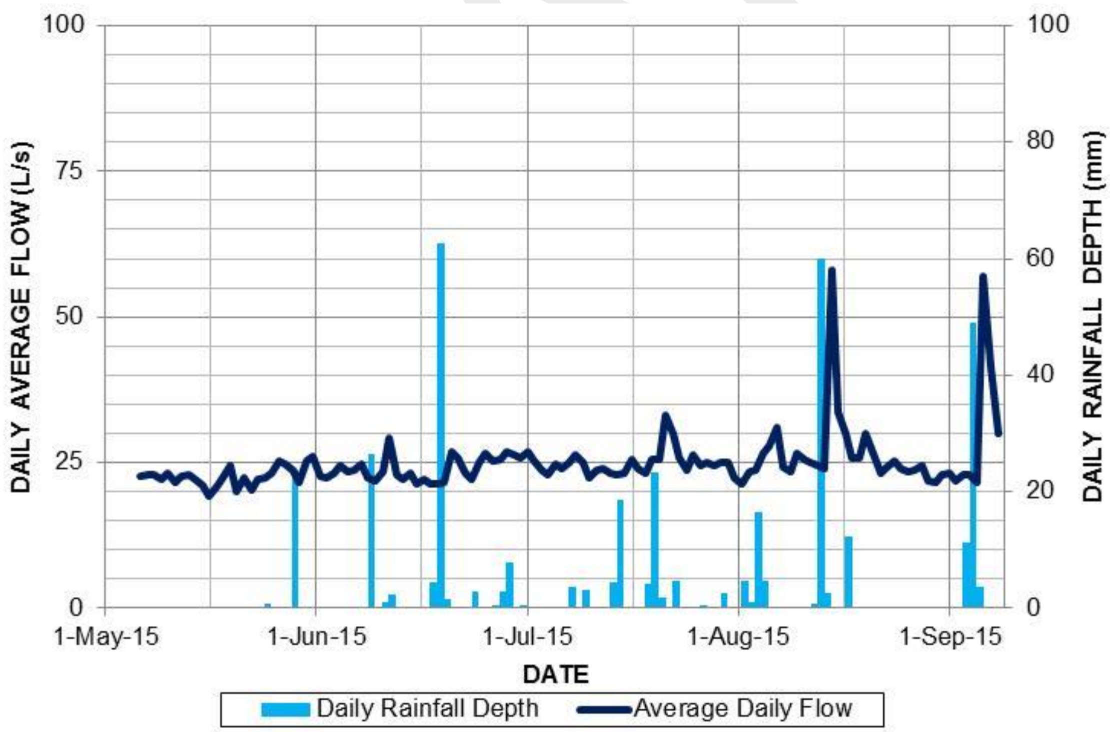

Figure 2.6: FM Site 3 – Daily Flow and Rainfall Depth

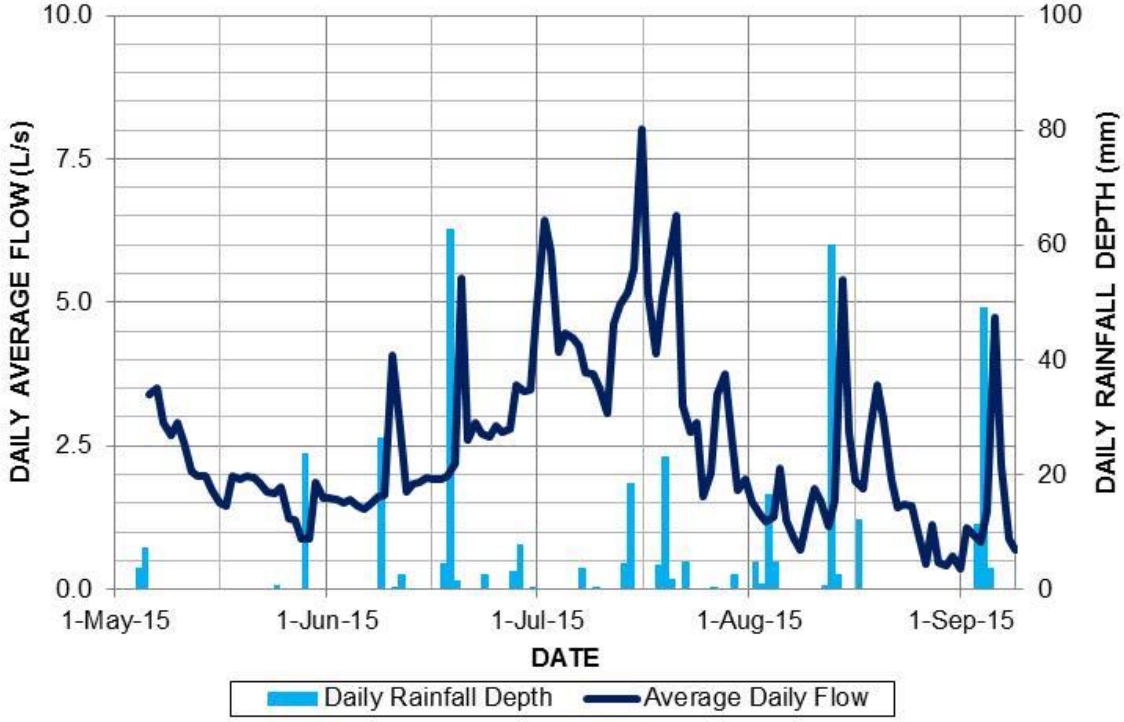

Figure 2.7: FM Site 4 – Daily Flow and Rainfall Depth

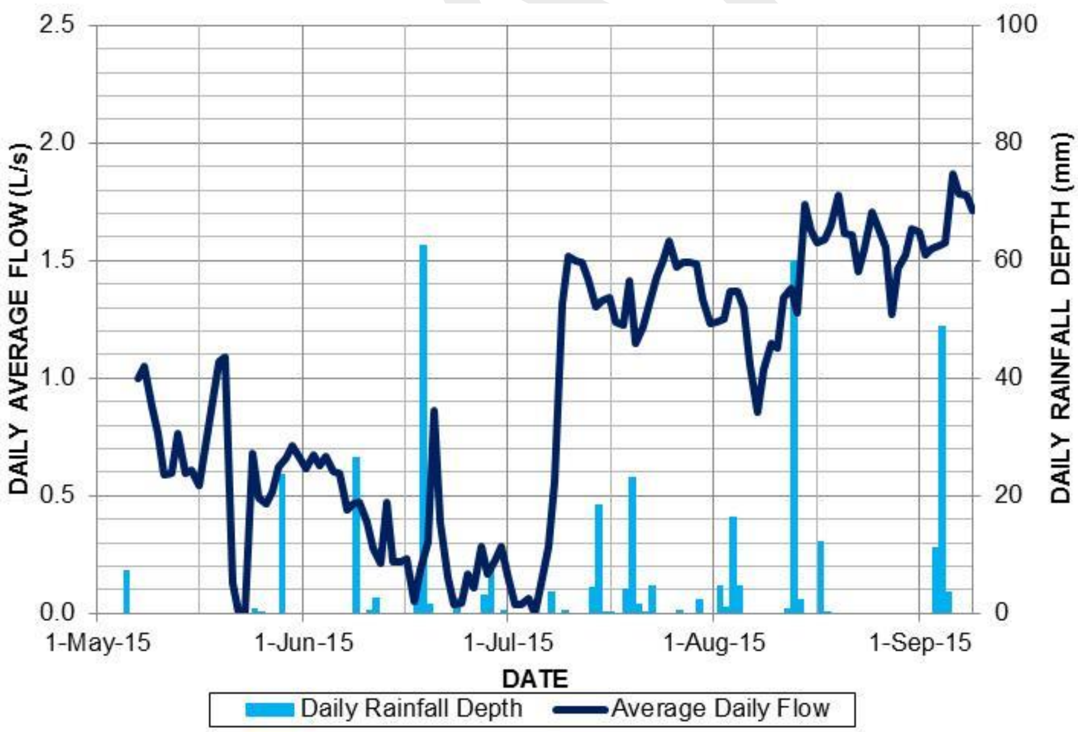

Figure 2.8: FM Site 5 – Daily Flow and Rainfall Depth

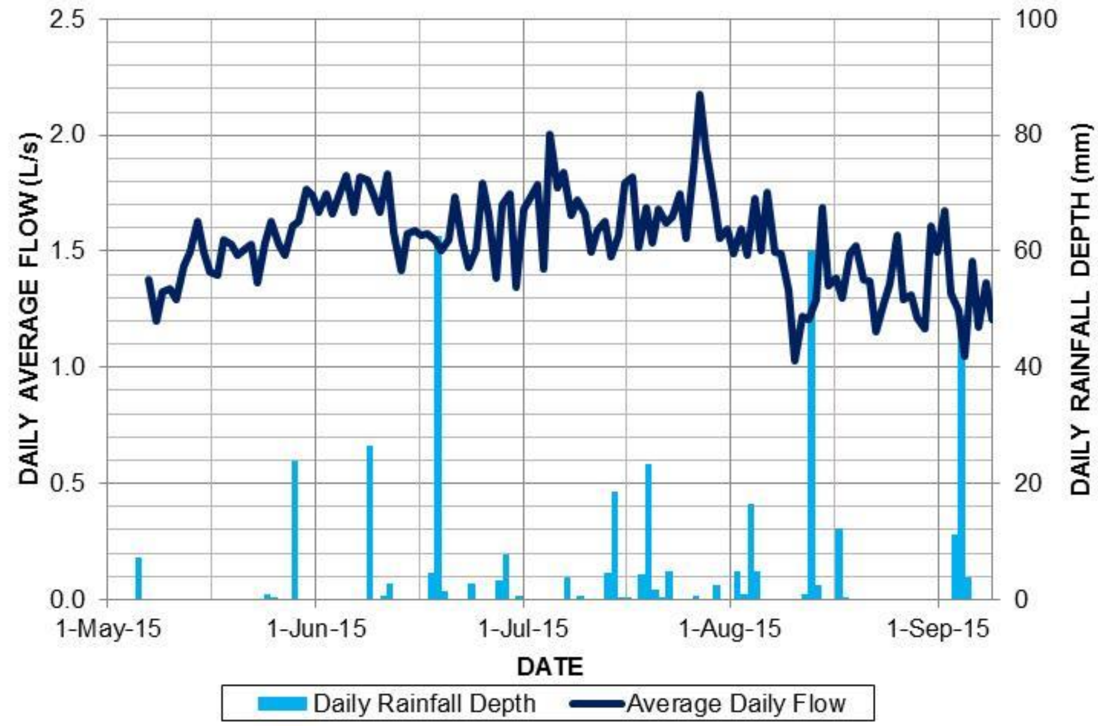

Figure 2.9: FM Site 6 – Daily Flow and Rainfall Depth

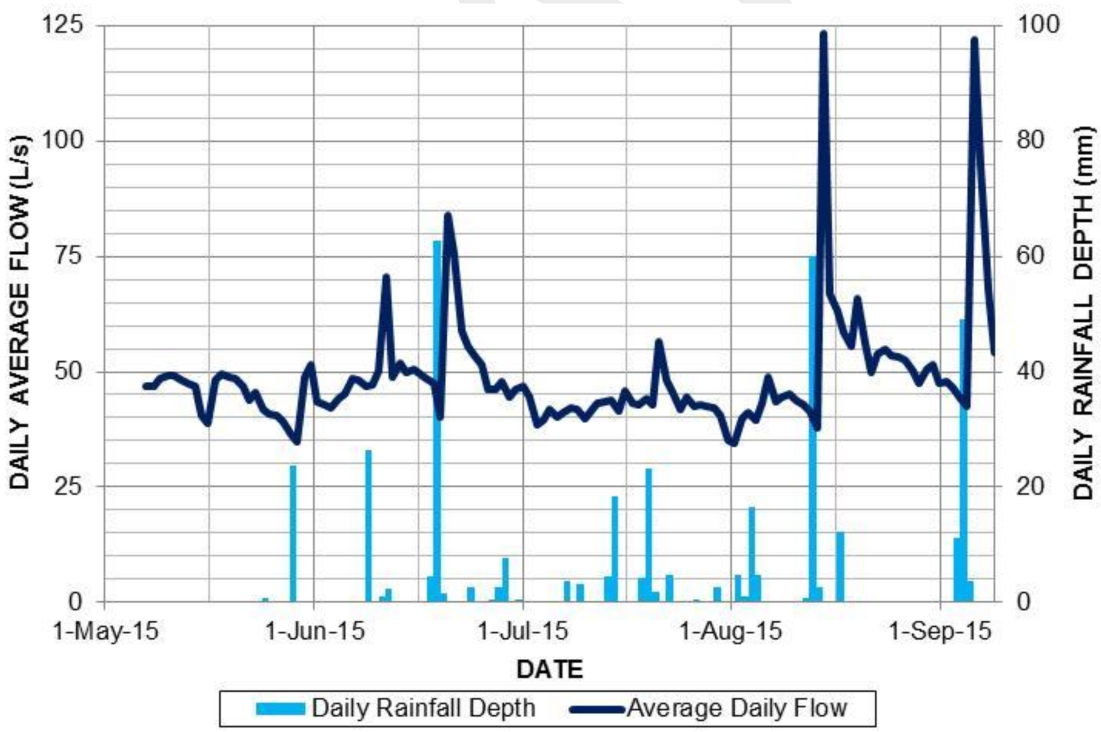

Figure 2.10: FM Site 7 – Daily Flow and Rainfall Depth

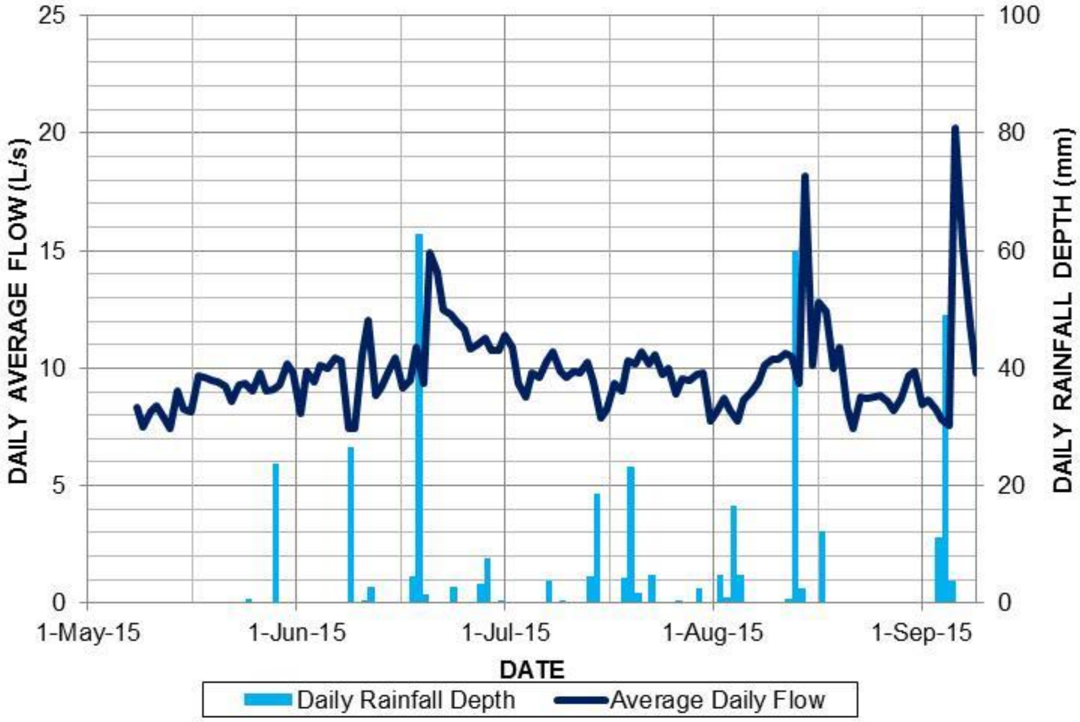

Figure 2.11: FM Site 8 – Daily Flow and Rainfall Depth

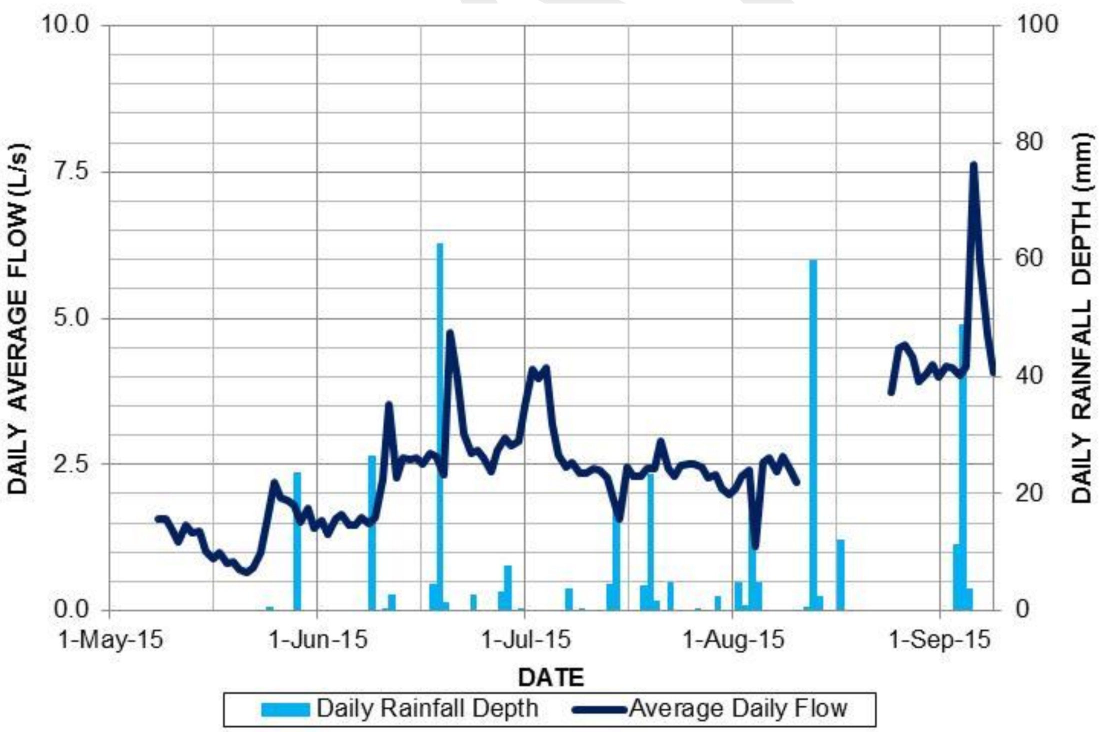

Figure 2.12: FM Site 9 – Daily Flow and Rainfall Depth

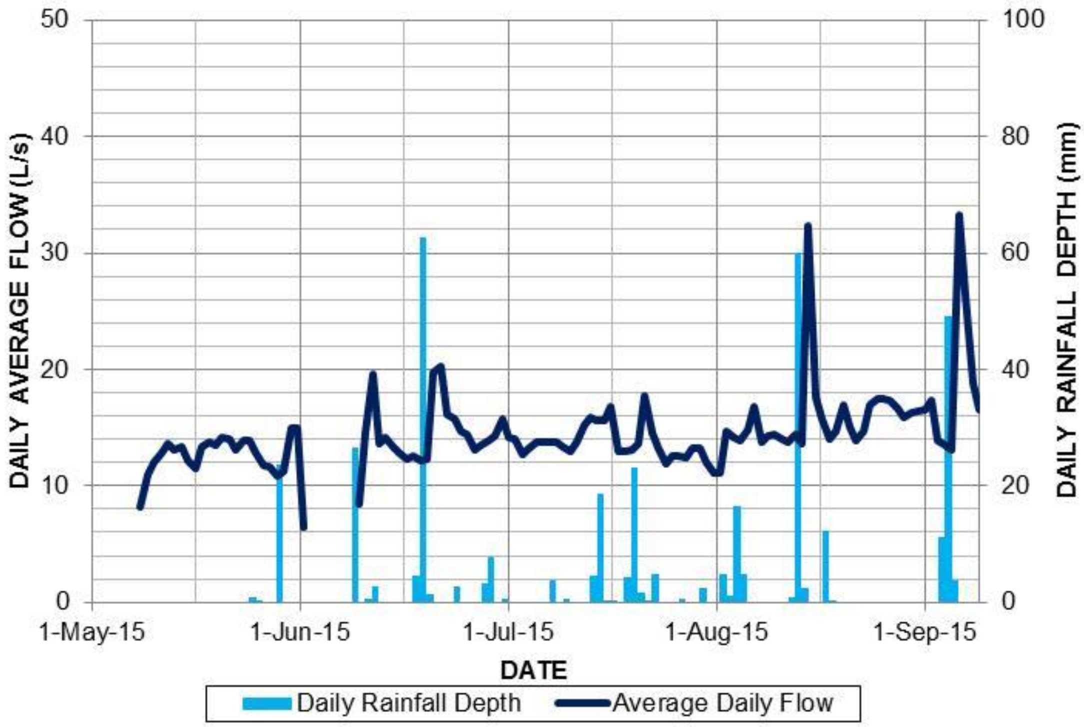

Figure 2.13: FM Site 10 – Daily Flow and Rainfall Depth

For the June 21 rainfall event, the wet weather flow response is not consistent amongst the flow monitor sites. FM Sites 1, 7, 8, 9, and 10 show a wet weather flow response to this rainfall, while FM Sites 2, 3, and 6 do not. This correlates to the location of the flow monitors and the variability of the storm – the sites downstream of the sewershed in the southern portion of the city shows the greatest wet weather flow response, which is where the more severe rainfall occurred. Further, FM Sites 4 and 5 are not reliable as the daily flow is not following a regular pattern leading up to the storm event.

The wet weather flow response in the June 21, 2015 rainfall is comparatively small, especially considering the severity of the storm. May and June were extremely dry months, with a total rainfall of 55 mm recorded. In comparison, the average rainfall for these two months is 116 mm, respectively. The soil moisture conditions prior to the June 21 rainfall event was likely very dry, meaning that a large portion of the infiltration was held by the soil and did not enter the sanitary sewer system.

For both the August 15 and September 5 -7 rainfall events, all the flow monitor sites show similar wet weather flow response. This could be expected, as the three rainfall gauges recorded similar rainfall depths and durations for both of the rainfall events. Also, the wet weather flow response in the flow monitors is greater in both of these storms, relative to the June 21 rainfall event.

Although the June 21 rainfall event, with a 100 year return period, is quite severe, the rainfall depth and duration was highly variable. Further, the flow monitoring data shows that the rainfall was isolated to the southern portion of the city. The September 5 – 7 rainfall event did exhibit notable wet weather flow response, but its severity (5 – 10 year return period) was expected to be too minor to cause critical flow in the sanitary sewer.

The August 15 rainfall event approximated a 25 year return period event, which was previously defined as the Level of Service return period. Also, this rainfall event was fairly consistent throughout the entire city, as shown in the rain gauge and flow monitor data. Considering the above, the August 15 rainfall event was selected as the most appropriate for subsequent wet weather flow calibration of the sanitary sewer model.

3. Dry Weather Flow Analysis and Calibration

The main objective of the dry weather flow analysis is to produce a model calibration that accounts for the known variables and provides a best fit to the flow monitor data amongst all ten flow monitor sites. The flow at each individual flow monitor site is a sum of many flow contributors, which contain a combination of different land-uses, located throughout the upstream collection system. One cannot expect the calibration to be a perfect fit to the flow monitor data, as the data analysis and calibration is only able to account for a finite number of variables. There is always the possibility for influences that cannot be readily explained, such as partial flow blockages, minute changes in pipe roughness, individual differences in flow contributions, flow monitor problems, and many other things.

The dry weather flow in a sanitary sewer system typically follows a regular and repeatable pattern throughout the day, with variations occurring between weekdays and weekends and through the seasons. For example, the groundwater infiltration to the sanitary sewer, represented as a constant base flow, may vary slowly through the summer to winter months. Secondly, the daily flow pattern may change during the summer months when schools are on break and portions of the population are on holiday.

The initial step in evaluating the dry weather flow was isolating periods of consecutive days of data that were not influenced by wet weather flow or the above factors. The 5 minute flow data was then smoothed out to determine the overall flow pattern by reducing the influence of individual outlying data points. First, the average daily and average hourly flows were calculated from the 5 minute flow data. The average hourly flow was normalized by the average daily flow to determine the hourly multiplier. The hourly multipliers and average daily flow for each day were then averaged across the individual days of data and the hourly flow recalculated by multiplying the average hourly multiplier by the average daily flow. This produced dry weather hydrographs for each of the flow monitor sites that were used in the model calibration.

The next step was to input the dry weather hydrographs into the sanitary sewer model and calibrate the average daily flow in the collection area of each flow monitor site to match the monitor data. This was achieved by adjusting average daily flow peaking factors within the collection system that were specific to only the given flow monitor site. For example, a peaking factor was determined to fit the average daily flow in FM Site 3. A different factor was then determined for the flow contributors specific only FM Site 2, which is downstream of FM Site 3. As such, the average daily flow at FM Site 2 was calculated from a combination of the peaking factors specific to only FM Site 2 and FM Site 3. The average daily flow peaking factors used for each flow monitor site and the average daily flow for the monitor data and calibration results are presented below in Table 3.1.

Table 3:1: Average Dry Weather Flow Peaking Factors

Flow Monitoring Site	Average Daily Flow Peaking Factor	Average Daily Flow Monitor Data	Average Daily Flow Calibration Results
FM Site 1	0.904	84.4 L/s	85.0 L/s
FM Site 2	0.795	32.9 L/s	33.1 L/s
FM Site 3	1.050	22.7 L/s	22.9 L/s
FM Site 4	0.216	3.8 L/s	3.7 L/s
FM Site 5	1.700	0.78 L/s	0.78 L/s
FM Site 6	1.260	1.36 L/s	1.34 L/s
FM Site 7	1.400	49.1 L/s	44.9 L/s
FM Site 8	0.548	9.0 L/s	8.9 L/s
FM Site 9	1.220	2.6 L/s	2.6 L/s
FM Site 10	1.359	13.6 L/s	13.6 L/s

The next step in the dry weather flow calibration was to determine flow pattern throughout the day, defined as the diurnal pattern. The diurnal pattern is expressed as a set of hourly multipliers, which the average daily flow is multiplied to produce a daily flow hydrograph. It is important to note that the average of the hourly multipliers is equal to 1.00, meaning the volume of the flow in the daily flow hydrograph is equal to the average daily flow. Different land-uses, such as residential and commercial, follow different diurnal patterns. Three distinct diurnal patterns were developed to fit the dry weather flow hydrographs – residential, commercial, and industrial (applied to the northwest industrial area of the city). The three diurnal patterns are presented below in Figure 3.1 and Table 3.2.

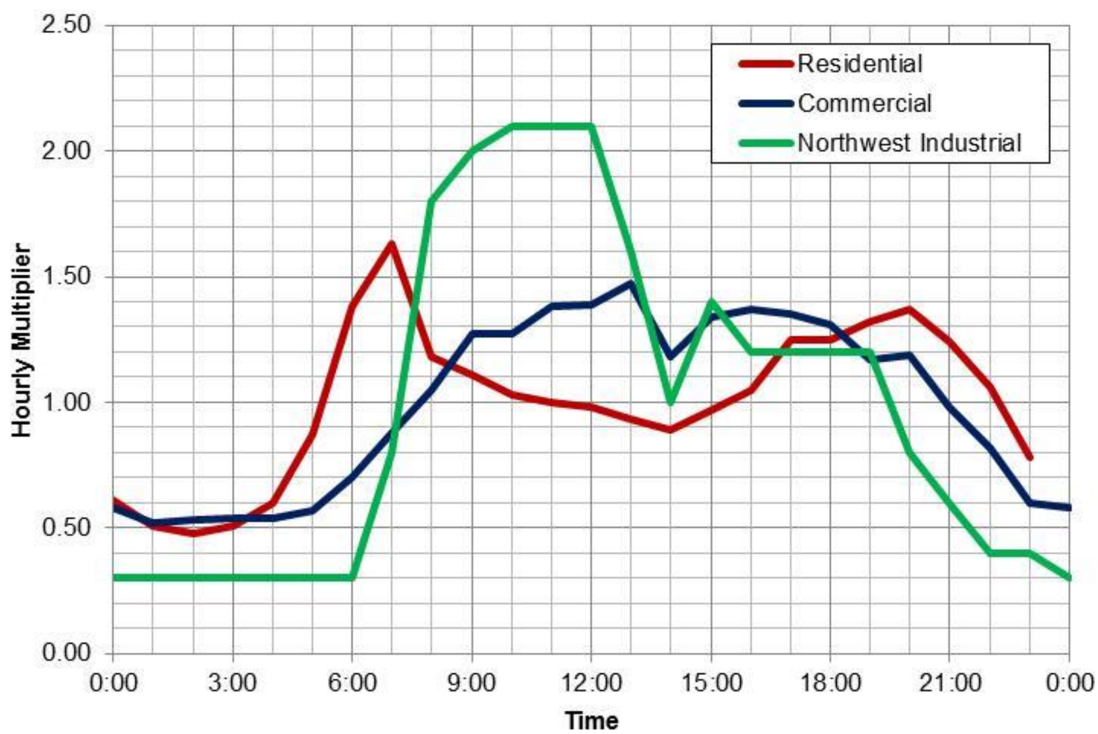

Figure 3.1: Diurnal Patterns

Table 3:2: Diurnal Patterns

Time	Average Daily Flow - Hourly Multipliers		
	Residential	Commercial	Northwest Industrial
0:00	0.61	0.58	0.30
1:00	0.51	0.52	0.30
2:00	0.48	0.53	0.30
3:00	0.51	0.54	0.30
4:00	0.60	0.54	0.30
5:00	0.87	0.57	0.30
6:00	1.38	0.70	0.30
7:00	1.63	0.88	0.80
8:00	1.18	1.05	1.80
9:00	1.11	1.27	2.00
10:00	1.03	1.27	2.10
11:00	1.00	1.38	2.10
12:00	0.98	1.39	2.10
13:00	0.93	1.47	1.60
14:00	0.89	1.18	1.00
15:00	0.97	1.34	1.40
16:00	1.05	1.37	1.20
17:00	1.25	1.35	1.20
18:00	1.25	1.31	1.20
19:00	1.32	1.17	1.20
20:00	1.37	1.19	0.80
21:00	1.24	0.98	0.60
22:00	1.06	0.82	0.40
23:00	0.78	0.60	0.40
Average	**1.00**	**1.00**	**1.00**

The diurnal patterns were adjusted (while always maintaining an average value of 1.00) to achieve a best fit amongst the ten flow monitor sites in terms of hydrograph shape, peak flow, and minimum flow. The peak and minimum flows at each of the flow monitor sites are presented below in Table 3.3. and the dry weather flow hydrographs for each of the flow monitors sites are presented in Figures 3.2 to 3.11.

Table 3:3: Dry Weather Flow Calibration – Peak and Minimum Flows

Flow Monitoring Site	Peak Flow		Minimum Flow	
	Monitor Data	Model Calibration	Monitor Data	Model Calibration
FM Site 1	107.4 L/s	111.6 L/s	43.23 L/s	39.0 L/s
FM Site 2	47.6 L/s	47.5 L/s	15.3 L/s	16.0 L/s
FM Site 3	29.3 L/s	29.0 L/s	11.9 L/s	12.0 L/s
FM Site 4	5.0 L/s	4.9 L/s	1.9 L/s	2.4 L/s
FM Site 5	1.23 L/s	1.08 L/s	0.38 L/s	0.32 L/s
FM Site 6	1.89 L/s	1.93 L/s	0.71 L/s	0.71 L/s
FM Site 7	63.0 L/s	63.6 L/s	18.1 L/s	25.2 L/s
FM Site 8	12.9 L/s	12.1 L/s	4.5 L/s	3.4 L/s
FM Site 9	4.2 L/s	4.2 L/s	1.3 L/s	1.4 L/s
FM Site 10	17.9 L/s	17.8 L/s	6.8 L/s	8.2 L/s

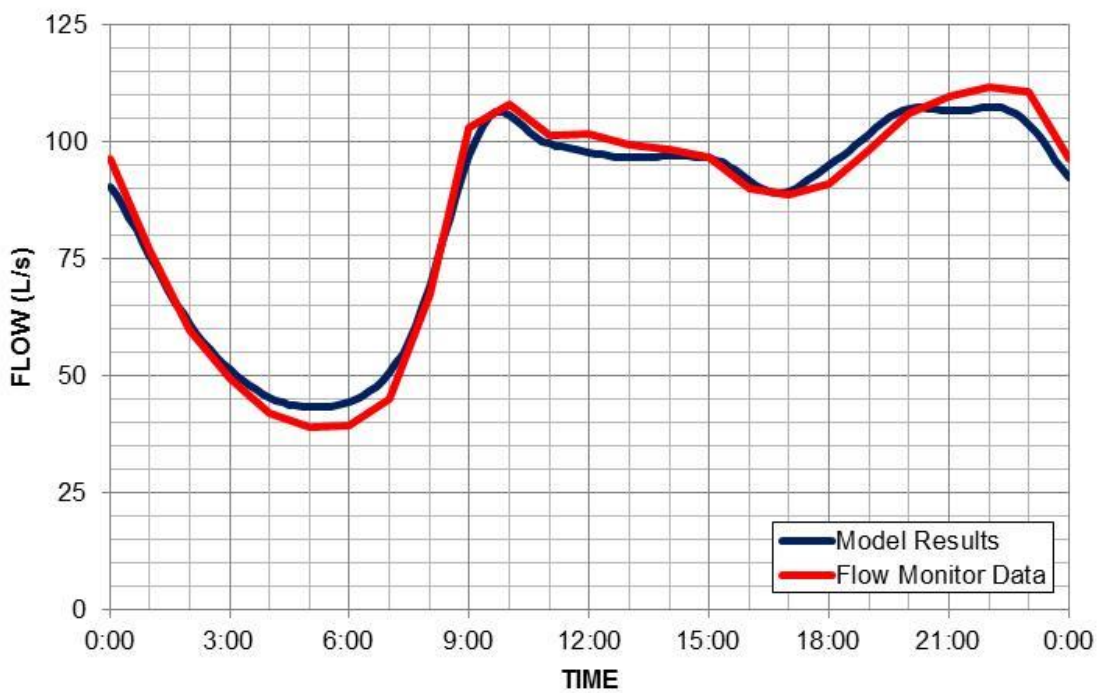

Figure 3.2: FM Site 1 – Dry Weather Flow Hydrograph

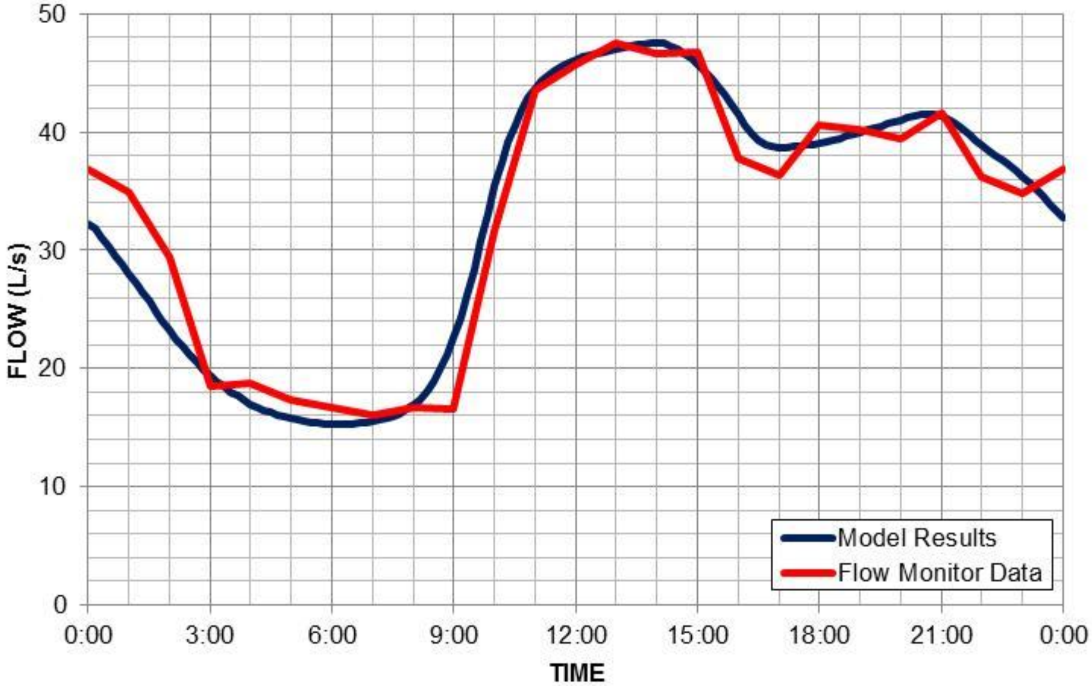

Figure 3.3: FM Site 2 – Dry Weather Flow Hydrograph

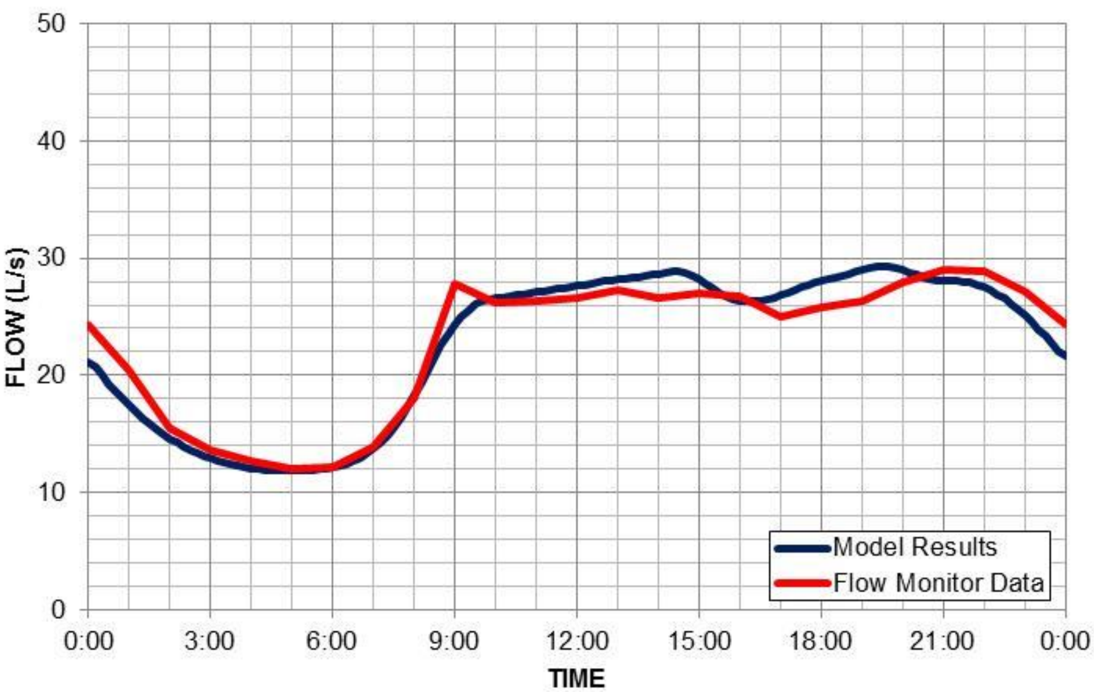

Figure 3.4: FM Site 3 – Dry Weather Flow Hydrograph

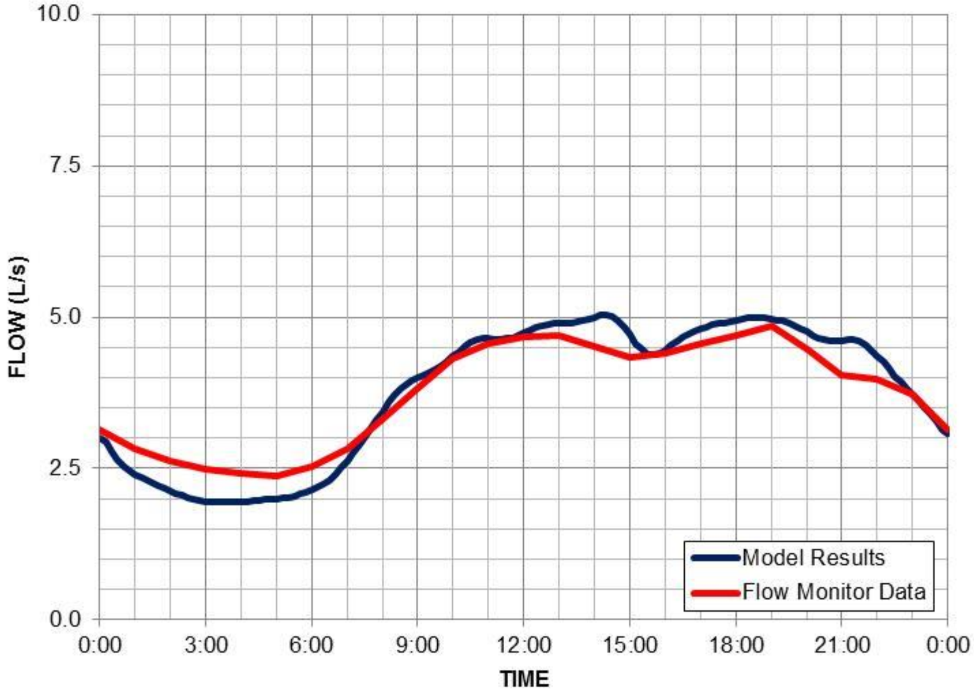

Figure 3.5: FM Site 4 – Dry Weather Flow Hydrograph

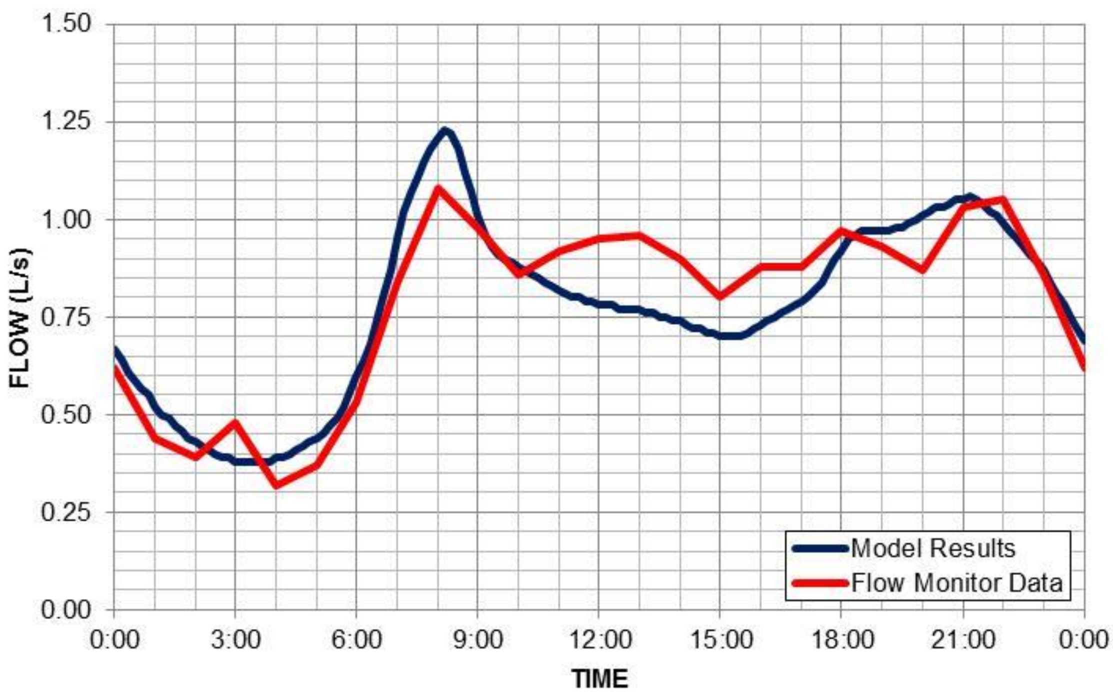

Figure 3.6: FM Site 5 – Dry Weather Flow Hydrograph

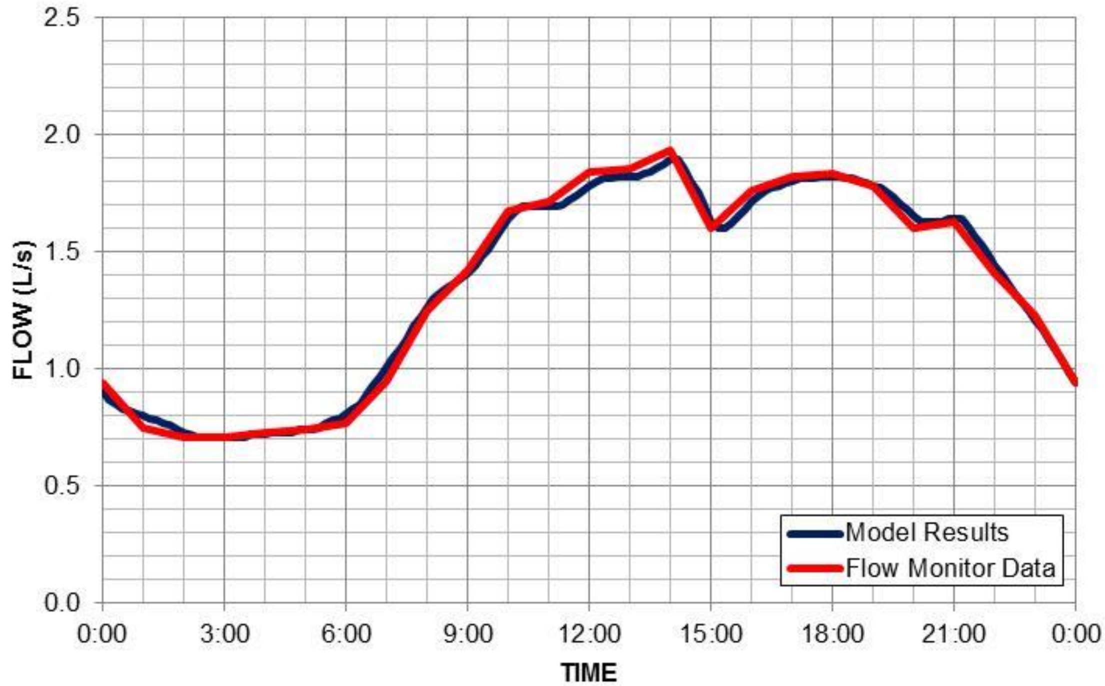

Figure 3.7: FM Site 6 – Dry Weather Flow Hydrograph

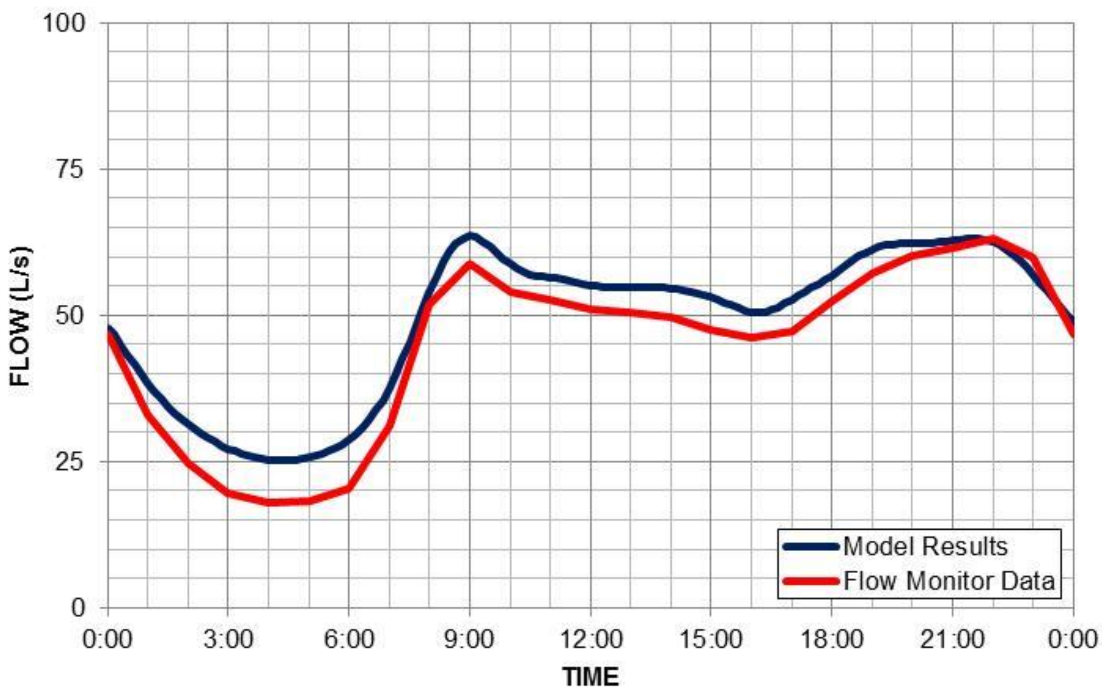

Figure 3.8: FM Site 7 – Dry Weather Flow Hydrograph

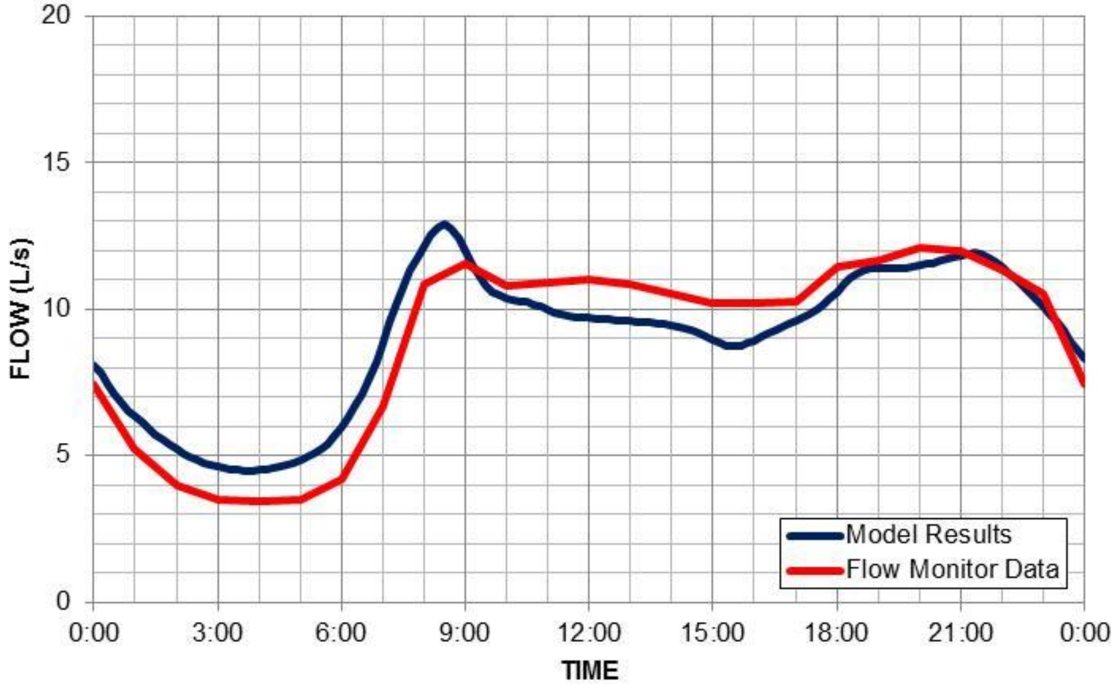

Figure 3.9: FM Site 8 – Dry Weather Flow Hydrograph

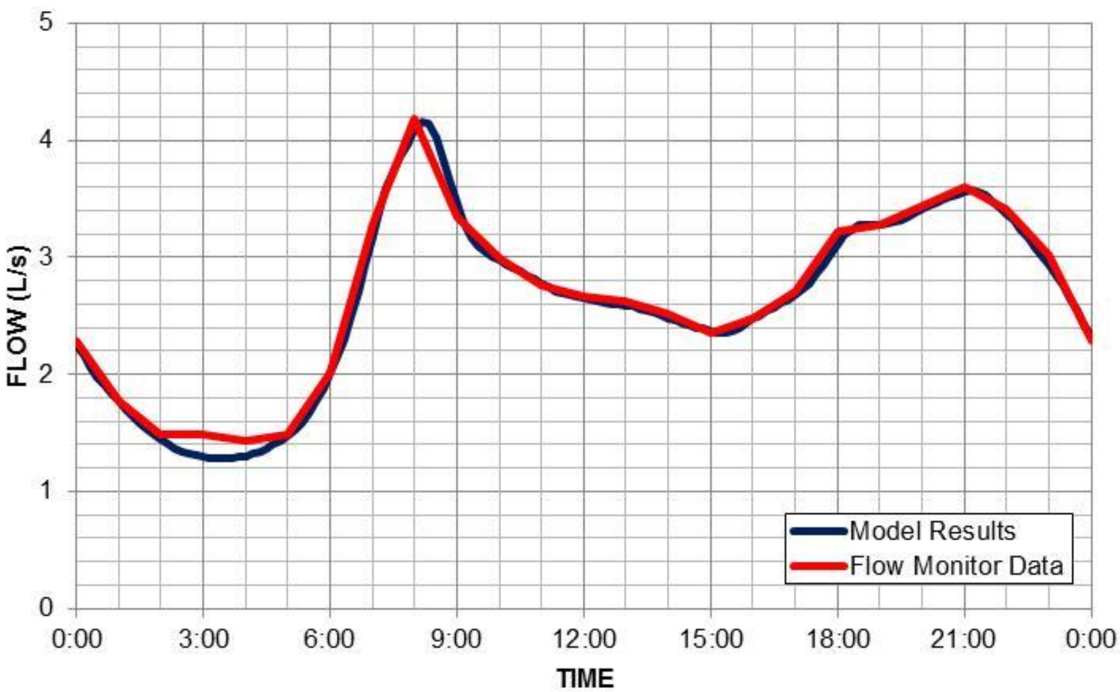

Figure 3.10: FM Site 9 – Dry Weather Flow Hydrograph

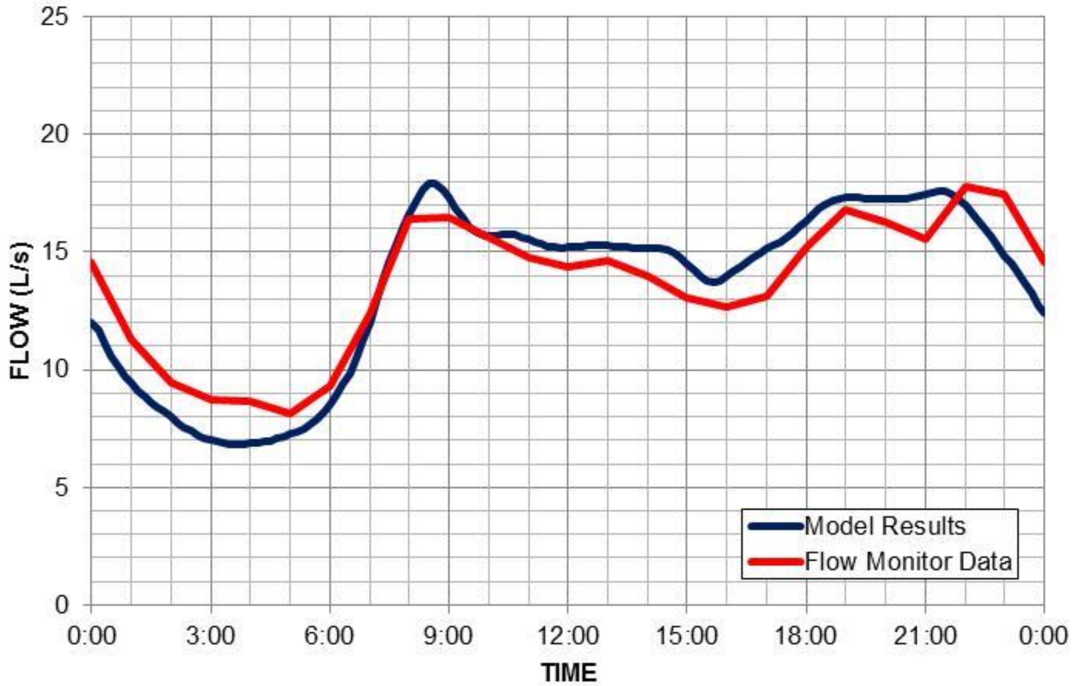

Figure 3.11: FM Site 10 – Dry Weather Flow Hydrograph

4. Wet Weather Flow Analysis and Model Calibration

The next step in the model calibration was to evaluate the wet weather flow, which is the sum of dry weather flow (described in the preceding section) and rainfall-derived inflow and infiltration (RDII). The RDII was simulated using the RTK method, which generates three unit hydrographs that represent the short, medium, and long-term flow response. Each of the three unit hydrographs have a unique set of values for the RTK parameters, where R represents the percentage of the rainfall volume that enters the sanitary sewer system, T is the time to peak, and K is used to describe the receding limb of the hydrograph. There is also an allowance for initial abstraction, which is the initial depth of rainfall that is held by the soil and does not infiltrate into the sanitary sewer.

As described in Section 2, the August 15, 2015 rainfall event, which approximated a 25 year return period event, yielded a noticeable wet weather flow response in the sanitary sewer system at all flow monitor sites. This rainfall event was used to calibrate the model to wet weather flow.

The three sets of RTK parameters and the initial abstraction was adjusted in the model such that a best fit was found between the model simulation and the monitored data amongst all ten flow monitor sites. In order to achieve a good fit, two sets of RTK parameters were derived; one for the northwest industrial portion of the city and the other for the remainder of the city (designated as residential/commercial). Note that the previously-completed 2008 wet weather flow calibration had found a significant response from the combined sewer in the downtown. This was not found in the 2015 wet weather flow calibration, which suggests a large portion of combined sewer had been separated since 2008. The two sets of RTK parameters for the 2015 wet weather flow calibration is presented below in Table 4.1.

Table 4:1: Wet Weather Flow Calibration – RTK Parameters

	Residential/Commercial			Northwest Industrial		
	Short	Medium	Long	Short	Medium	Long
R	2.0%	0.5%	1.5%	2.1%	1.5%	2.0%
T	1.5 h	3.0 h	9.0 h	1.2 h	3.0 h	9.0 h
K	3.5	4.0	6.0	3.5	4.0	6.0
Initial Abstraction	10 mm			10 mm		

Note that in the residential/commercial and the northwest industrial areas, a total of 4.0% and 5.6% of the rainfall volume, less the volume of initial abstraction, enter the sanitary sewer system.

In calibrating the model to the wet weather flow, the important considerations are the peak flow, RDII volume, and a visual assessment of the flow hydrograph. A summary of the peak flow, RDII volume, and peak infiltration rate for each of the flow monitor sites is presented below in Table 4.2.

Table 4:2: Wet Weather Flow Calibration – Peak Flow, RDII Volume, Peak Infiltration Rate

Flow Monitoring Site	Peak Flow		RDII Volume		Peak Infiltration Rate	
	Monitor Data	Model Calibration	Monitor Data	Model Calibration	Monitor Data	Model Calibration
FM Site 1	573 L/s	527 L/s	10,850 m^3	12,200 m^3	0.74 L/s/ha	0.68 L/s/ha
FM Site 2	252 L/s	247 L/s	5,300 m^3	6,130 m^3	0.83 L/s/ha	0.82 L/s/ha
FM Site 3	150 L/s	156 L/s	2,940 m^3	3,890 m^3	0.76 L/s/ha	0.79 L/s/ha
FM Site 4	20 L/s	32 L/s	330 m^3	780 m^3	0.52 L/s/ha	0.84 L/s/ha
FM Site 5	5.8 L/s	4.1 L/s	40 m^3	91 m^3	0.94 L/s/ha	0.66 L/s/ha
FM Site 6	3.5 L/s	4.0 L/s	34 m^3	70 m^3	0.67 L/s/ha	0.78 L/s/ha
FM Site 7	344 L/s	295 L/s	7370 m^3	6740 m^3	0.73 L/s/ha	0.63 L/s/ha
FM Site 8	57.5 L/s	81.3 L/s	762 m^3	2010 m^3	0.42 L/s/ha	0.60 L/s/ha
FM Site 9	No flow data available due to battery loss					
FM Site 10	80.7 L/s	64.3 L/s	1620 m^3	1640 m^3	0.62 L/s/ha	0.49 L/s/ha

It is noted that the model calibration is considerably different from the flow monitor data for a number of sites and the calibrated RDII volume is higher than the monitored data at all sites. Unlike the Dry Weather Flow calibration, which filters much of the noise out of the data by sampling a number of days of data, the wet weather flow calibration uses the only single data set available for this specific rainfall event. Some of the flow monitor sites, particularly with low flows, have considerable noise in the data.

Secondly, the RDII volume is calculated by subtracting the volume of the wet weather flow hydrograph from the dry weather flow hydrograph. The calibrated dry weather flow was developed from averaging over a number of days, and provides a typical flow hydrograph. The dry weather flow monitor data was taken from August 14 (the day preceding the rainfall event), which could be an atypical day of dry weather flow, compared to the calibrated dry weather flow.

The suitability of the flow hydrograph was completed through a visual assessment. The wet weather flow hydrographs are presented below in Figures 4.1 to 4.9.

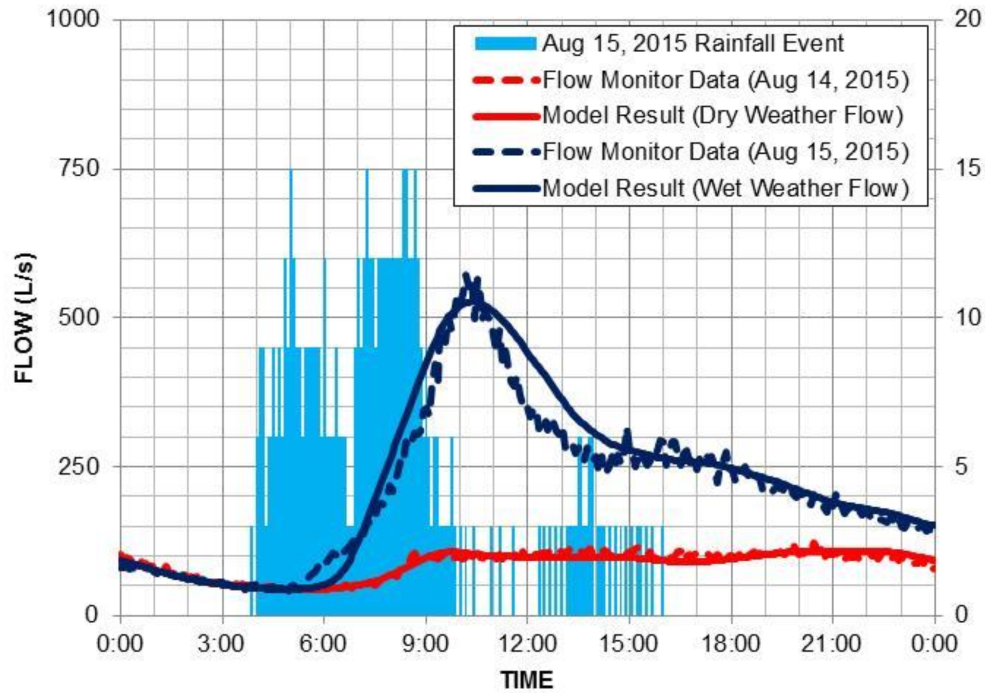

Figure 4.1: FM Site 1 – Wet Weather Flow Hydrograph

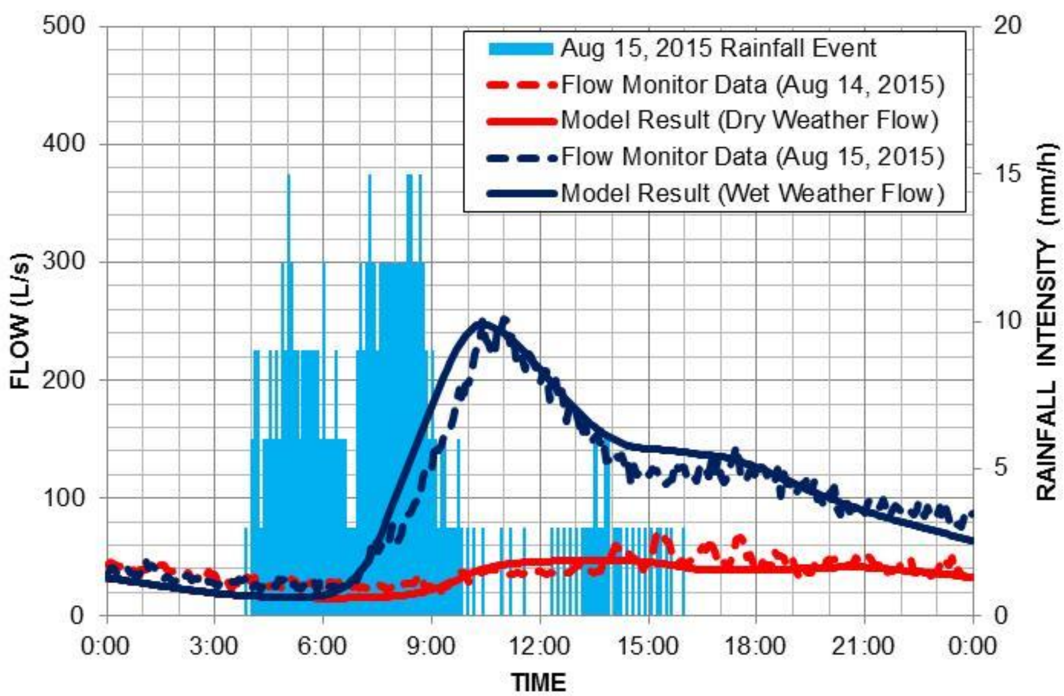

Figure 4.2: FM Site 2 – Wet Weather Flow Hydrograph

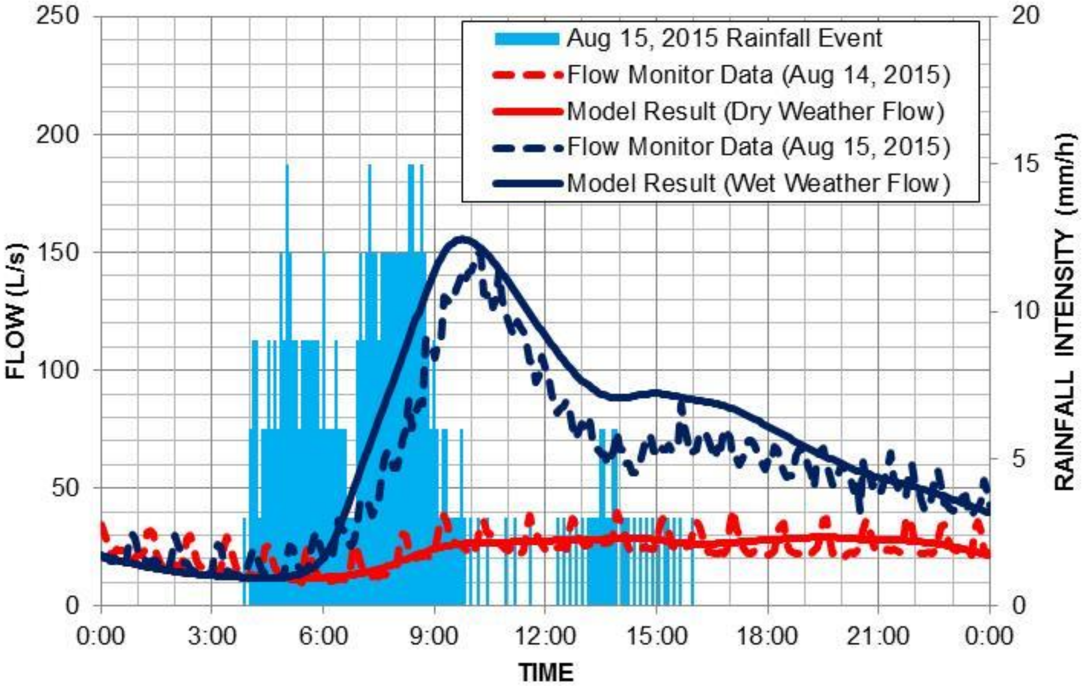

Figure 4.3: FM Site 3 – Wet Weather Flow Hydrograph

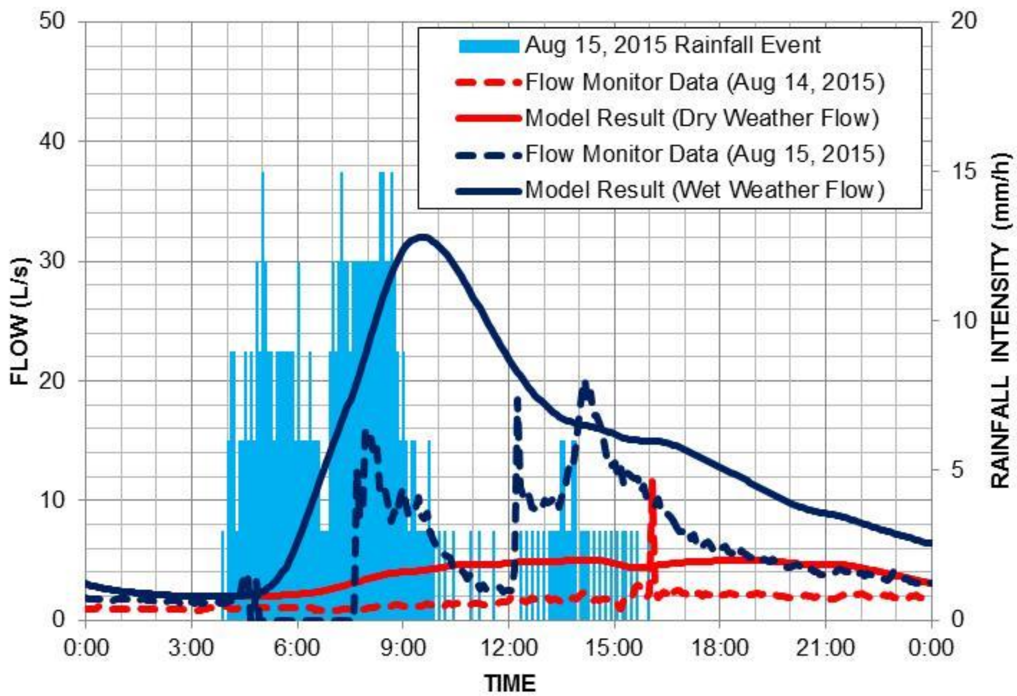

Figure 4.4: FM Site 4 – Wet Weather Flow Hydrograph

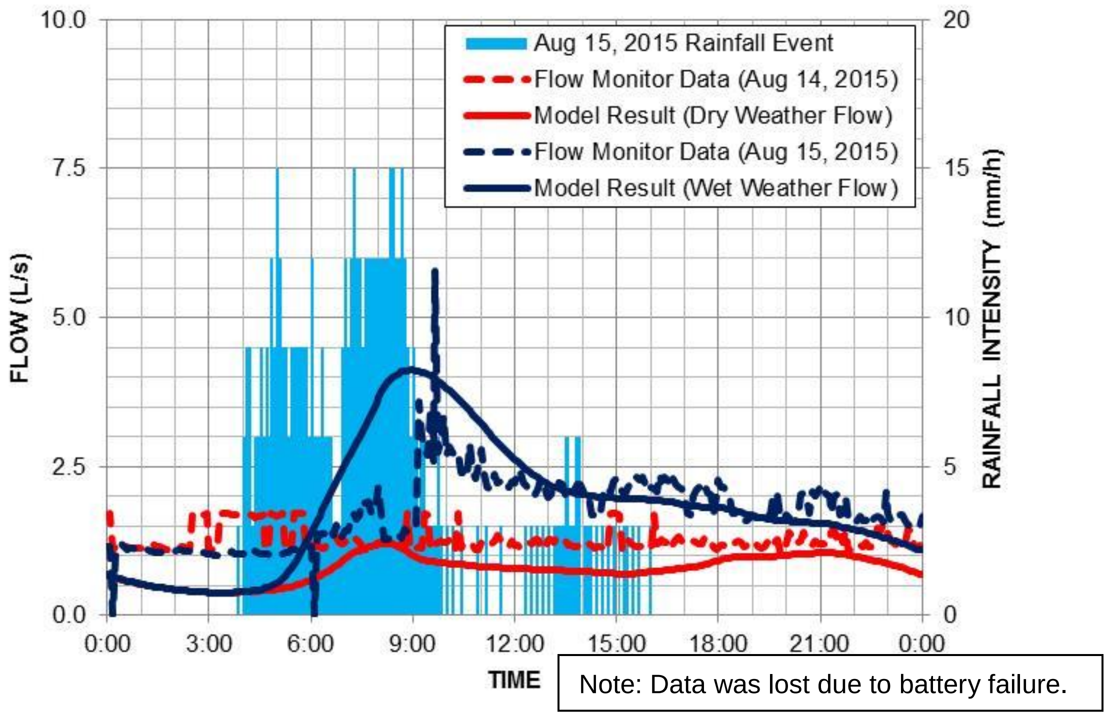

Figure 4.5: FM Site 5 – Wet Weather Flow Hydrograph

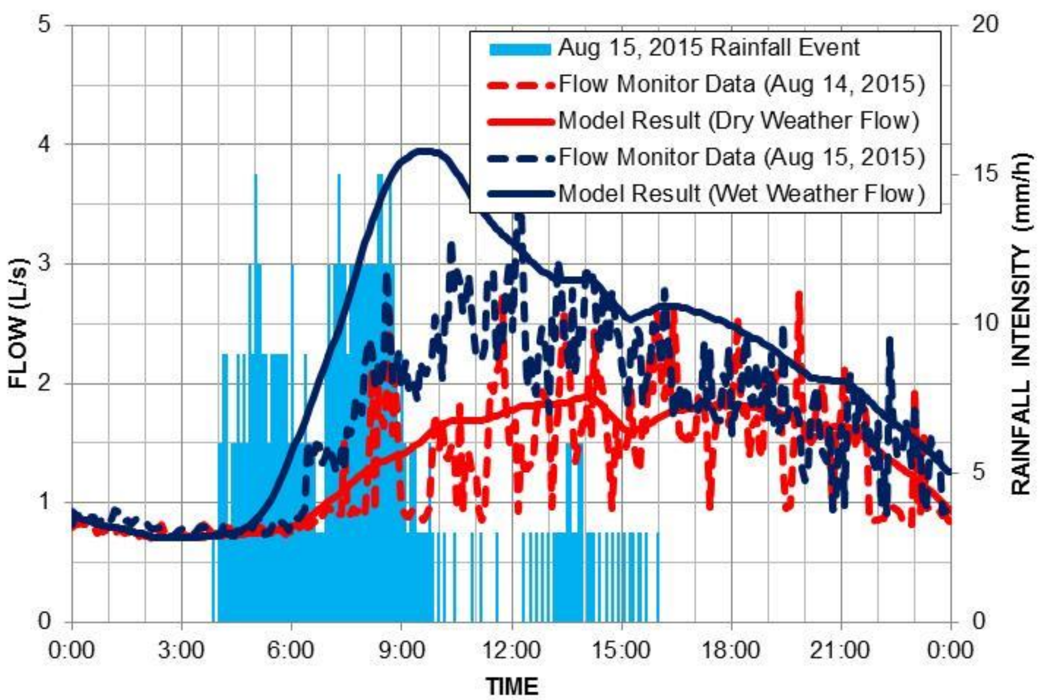

Figure 4.6: FM Site 6 – Wet Weather Flow Hydrograph

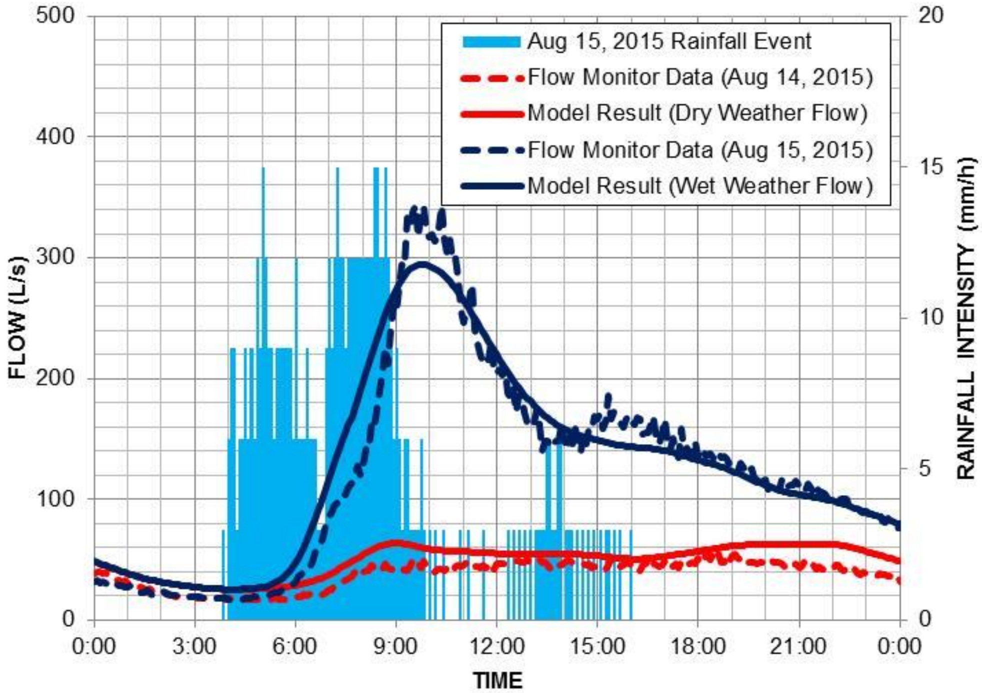

Figure 4.7: FM Site 7 – Wet Weather Flow Hydrograph

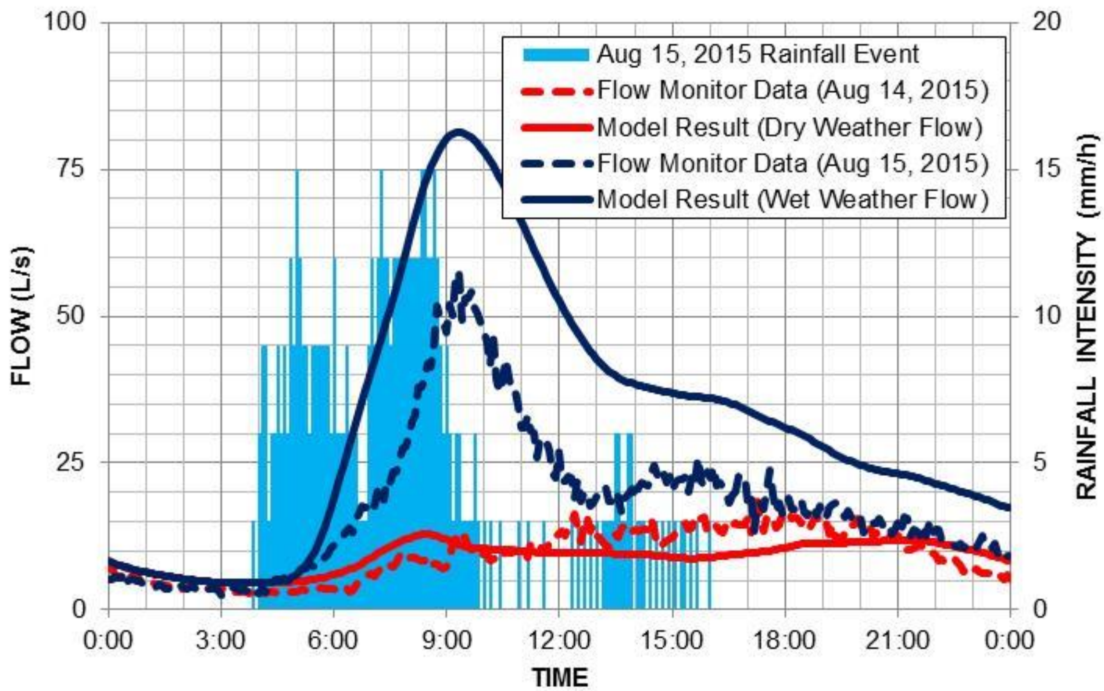

Figure 4.8: FM Site 8 – Wet Weather Flow Hydrograph

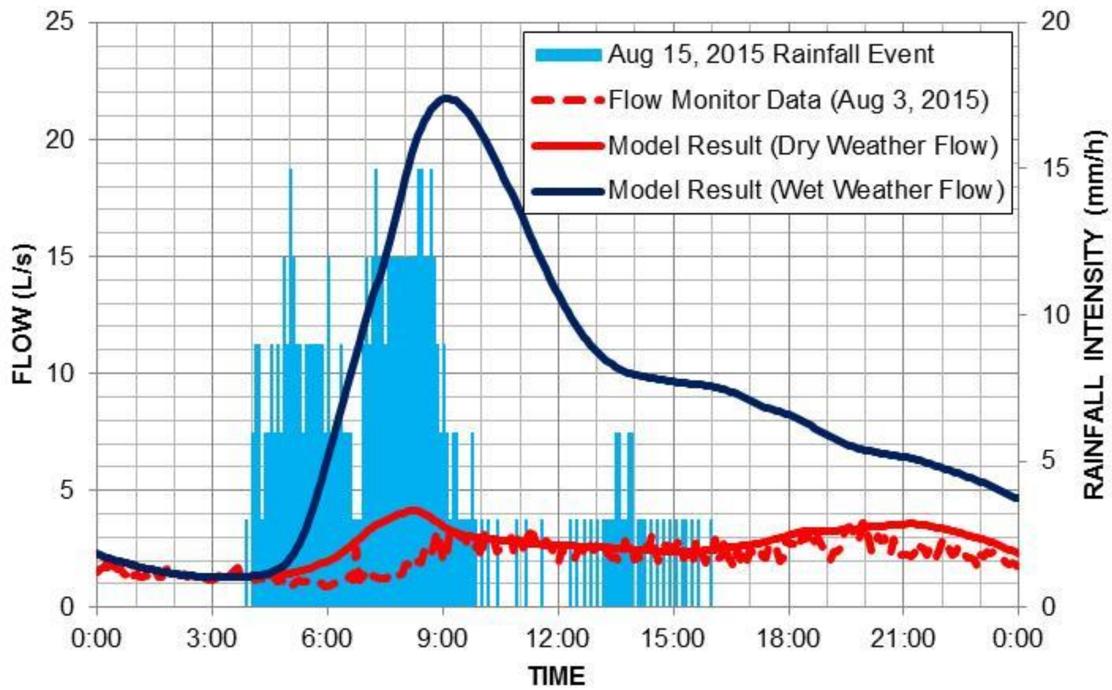

Figure 4.9: FM Site 9 – Wet Weather Flow Hydrograph

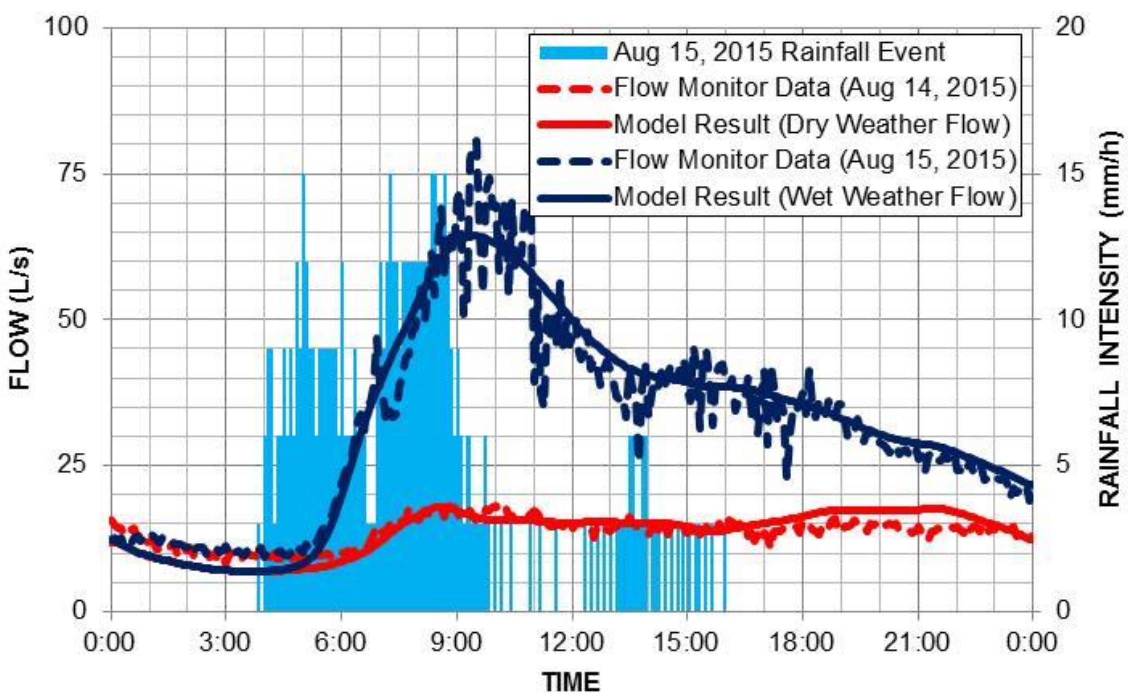

Figure 4.10: FM Site 10 – Wet Weather Flow Hydrograph

5. Summary

The City conducted a flow monitoring program from May 6 to September 9, 2015. The purpose of the flow monitoring program was to collect real-time flow data in the sanitary sewer system to subsequently the city-wide sanitary sewer model, presently under development by AECOM.

A total of ten flow monitoring sites and three rain gauge sites were strategically located throughout the city. The locations of the flow monitoring sites were intended to record flows in the main sanitary trunks and from a variety of land-uses. The rain gauges were placed on the north and south edges and approximate mid-point of the city.

The preceding technical memorandum provided a summary of the flow monitoring program, data analysis, and model calibration. Once the model has been satisfactorily calibrated, it will be used to assess the performance of the existing sanitary sewer system, identify upgrades, and determine future servicing requirements.

The 2015 flow monitoring program produced a valuable, sound, and robust data set, which fostered an effective calibration of the sanitary sewer model. However, there were lessons learned that should be implemented in future flow monitoring programs.

The nature of flow monitoring, sanitary sewer modelling, and the interface between the two by way of model calibration means that there is always opportunity for future work. The recommendations for future work include:

- **Implement an annual flow monitoring program.** This would reinforce the dry weather flow and wet weather flow calibration. Also, the most useful wet weather flow data is captured in a severe rainfall event that results in overwhelming of the sanitary sewer system. The occurrence of such a type of rainfall event cannot be predicted, and only addressed by ensuring the flow monitors are in place on an annual basis.

- **Use the sanitary sewer model to aid in flow monitor site selection.** Due to the timing of the Sanitary Master Plan study, AECOM had not yet garnered a thorough understanding of the sanitary system when the flow monitor sites were selected in April. The city-wide sanitary sewer system has many flow-splits and interconnections, such that the flow routing proved difficult to ascertain from plan drawings alone. A key consideration in selecting the sites was the pipe size (larger than 250 mm diameter), which was assumed to indicate a sustained and measurable flow in the sewer. In reality, this was not always the case. Going forward, it is recommended to use the sanitary sewer model to verify the locations of proposed flow monitor sites provide a specified minimum average daily flow.

- **Continue to sample main sanitary trunks and varying and unique land-uses.** The number of available flow monitors limits the number of varying and unique land-uses that may be sampled. It is recommended to always sample the main sanitary trunks to gain an understanding of the total flow to the system. This may be beneficial for other uses, such as work at the Waste Water Treatment Plant or environmental reporting. It is always important to sample different types of land-uses, including residential, commercial, industrial, and institutional. Large, unique flow contributors, such as the ADM processing plant and Lakeland College, have distinct diurnal patterns and potentially large influences on the flow in the sanitary sewer.

Appendix D

Capital Cost Estimate Tables

PRESENT DAY UPGRADES

Capital Cost Estimate

NO.	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	AMOUNT
A.	**44 STREET UPGRADES**				
.1	General Requirements	lump sum			$47,080.00
.2	250 mm Dia. Sewer (3 - 4 m Deep) in street	lin. m.	$950.00	110	$104,500.00
.3	Appurtenances and Incidentals	lump sum			$20,900.00
.4	Surface Restoration	lin. m.	$1,000.00	110	$110,000.00
	SUB-TOTAL				**$282,480.00**
	Contingency (30%)				$84,744.00
	TOTAL				**$367,224.00**
B.	**47 AVENUE AND 48 AVENUE UPGRADES (Common to both options)**				
.1	General Requirements	lump sum			$271,200.00
.2	300 mm Dia. Sewer (3 - 4 m Deep) in street	lin. m.	$1,050.00	600	$630,000.00
.3	Appurtenances and Incidentals	lump sum			$126,000.00
.4	Surface Restoration	lin. m.	$1,000.00	600	$600,000.00
	SUB-TOTAL				**$1,627,200.00**
	Contingency (30%)				$488,160.00
	TOTAL				**$2,115,360.00**
C.	**OPTION 1 - INLINE STORAGE**				
.1	General Requirements	lump sum			$235,550.00
.2	1800 mm dia. Storage Tank	lin. m.	$5,400.00	175	$945,000.00
.3	Appurtenances and Incidentals	lump sum			$189,000.00
.4	Surface Restoration	lin. m.	$250.00	175	$43,750.00
	SUB-TOTAL				**$1,413,300.00**
	Contingency (30%)				$423,990.00
	TOTAL				**$1,837,290.00**
D.	**OPTION 2 - CONVEYANCE**				
.1	General Requirements	lump sum			$216,280.00
.2	300 mm Dia. Sewer (3 - 4 m Deep) in field	lin. m.	$500.00	730	$365,000.00
.3	300 mm Dia. Sewer (4 - 5 m Deep) in field	lin. m.	$600.00	170	$102,000.00
.4	300 mm Dia. Sewer (4 - 5 m Deep) in street	lin. m.	$1,150.00	200	$230,000.00
.5	Appurtenances and Incidentals	lump sum			$139,400.00
.6	Surface Restoration (Field)	lin. m.	$50.00	900	$45,000.00
.7	Surface Restoration (Street)	lin. m.	$1,000.00	200	$200,000.00
	SUB-TOTAL				**$1,297,680.00**
	Contingency (30%)				$389,304.00
	TOTAL				**$1,686,984.00**

3-YEAR GROWTH HORIZON UPGRADES (2018)
Capital Cost Estimate

NO.	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	AMOUNT
A.	**19 STREET TRUNK**				
.1	General Requirements	lump sum			$ 184,300.00
.2	600 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 850.00	50	$ 42,500.00
.3	600 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 1,000.00	120	$ 120,000.00
.4	750 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 1,000.00	190	$ 190,000.00
.5	750 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 1,250.00	310	$ 387,500.00
.6	Appurtenances and Incidentals	lump sum			$ 148,000.00
.7	Surface Restoration	lin. m.	$ 50.00	670	$ 33,500.00
	SUB-TOTAL				**$ 1,105,800.00**
	Contingency (30%)				$ 331,740.00
	Total (2015 Value)				**$ 1,437,540.00**
	Inflation (2% annually)				$ 87,989.00
	Total (2018 Value)				**$ 1,525,529.00**
B.	**SOUTH TRUNK**				
.1	General Requirements	lump sum			$ 707,980.00
.2	3 - 4 m Deep Excavation	lin. m.	$ 1,700.00	230	$ 391,000.00
.3	4 - 5 m Deep Excavation	lin. m.	$ 2,100.00	200	$ 420,000.00
.4	5 - 6 m Deep Excavation	lin. m.	$ 2,700.00	270	$ 729,000.00
.5	6 - 7 m Deep Excavation	lin. m.	$ 4,100.00	150	$ 615,000.00
.6	7 - 8 m Deep Excavation	lin. m.	$ 4,600.00	160	$ 736,000.00
.7	Appurtenances and Incidentals	lump sum			$ 578,200.00
.8	Surface Restoration	lin. m.	$ 70.00	1010	$ 70,700.00
	SUB-TOTAL				**$ 4,247,880.00**
	Contingency (30%)				$ 1,274,364.00
	Total (2015 Value)				**$ 5,522,244.00**
	Inflation (2% annually)				$ 338,006.00
	Total (2018 Value)				**$ 5,860,250.00**

5-YEAR GROWTH HORIZON UPGRADES (2020)

Capital Cost Estimate

NO.	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	AMOUNT
A.	**SOUTHEAST TRUNK TWIN**				
.1	General Requirements	lump sum			$ 447,600.00
.2	525 mm Dia. Sewer (5 - 6 m Deep) in street	lin. m.	$ 2,000.00	50	$ 100,000.00
.3	525 mm Dia. Sewer (6 - 7 m Deep) in street	lin. m.	$ 2,300.00	550	$ 1,265,000.00
.4	Appurtenances and Incidentals	lump sum			$ 273,000.00
.5	Surface Restoration (Street)	lin. m.	$ 1,000.00	600	$ 600,000.00
	SUB-TOTAL				**$ 2,685,600.00**
	Contingency (30%)				$ 805,680.00
	TOTAL (2015 Value)				**$ 3,491,280.00**
	Inflation (2% Annually)				$ 363,375.00
	TOTAL (2020 Value)				**$ 3,854,655.00**

10-YEAR GROWTH HORIZON UPGRADES (2025)

Capital Cost Estimate

NO.	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	AMOUNT
A.	**SOUTH TRUNK**				
.1	General Requirements	lump sum			$ 2,467,920.00
.2	750 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 1,000.00	140	$ 140,000.00
.3	750 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 1,250.00	480	$ 600,000.00
.4	750 mm Dia. Sewer (5 - 6 m Deep)	lin. m.	$ 1,500.00	130	$ 195,000.00
.5	900 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 1,200.00	160	$ 192,000.00
.6	900 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 1,300.00	520	$ 676,000.00
.7	900 mm Dia. Sewer (5 - 6 m Deep)	lin. m.	$ 1,600.00	1320	$ 2,112,000.00
.8	1050 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 1,900.00	590	$ 1,121,000.00
.9	1050 mm Dia. Sewer (5 - 6 m Deep)	lin. m.	$ 2,400.00	400	$ 960,000.00
.10	1200 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 1,700.00	250	$ 425,000.00
.11	1200 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 2,100.00	190	$ 399,000.00
.12	1200 mm Dia. Sewer (5 - 6 m Deep)	lin. m.	$ 2,700.00	100	$ 270,000.00
.13	1200 mm Dia. Sewer (6 - 7 m Deep)	lin. m.	$ 3,800.00	290	$ 1,102,000.00
.14	1200 mm Dia. Sewer (7 - 8 m Deep)	lin. m.	$ 4,200.00	210	$ 882,000.00
.15	1200 mm Dia. Sewer (8 - 9 m Deep)	lin. m.	$ 4,800.00	200	$ 960,000.00
.16	Appurtenances and Incidentals	lump sum			$ 2,006,800.00
.17	Surface Restoration	lin. m.	$ 60.00	4980	$ 298,800.00
	SUB-TOTAL				**$ 14,807,520.00**
	Contingency (30%)				$ 4,442,256.00
	TOTAL (2015 Value)				**$ 19,249,776.00**
	Inflation (2% Annually)				$ 4,215,594.00
	TOTAL (2020 Value)				**$ 23,465,370.00**
B.	**EAST TRUNK TWIN**				
.1	General Requirements	lump sum			$ 3,906,240.00
.2	1200 mm Dia. Sewer (3 - 4 m Deep) in field	lin. m.	$ 1,700.00	200	$ 340,000.00
.3	1200 mm Dia. Sewer (3 - 4 m Deep) in street	lin. m.	$ 3,200.00	140	$ 448,000.00
.4	1200 mm Dia. Sewer (4 - 5 m Deep) in field	lin. m.	$ 2,100.00	1090	$ 2,289,000.00
.5	1200 mm Dia. Sewer (4 - 5 m Deep) in street	lin. m.	$ 3,600.00	2270	$ 8,172,000.00
.6	1200 mm Dia. Sewer (5 - 6 m Deep) in field	lin. m.	$ 2,700.00	390	$ 1,053,000.00
.7	1200 mm Dia. Sewer (5 - 6 m Deep) in street	lin. m.	$ 3,200.00	470	$ 1,504,000.00
.8	Appurtenances and Incidentals	lump sum			$ 2,761,200.00
.9	Surface Restoration (Field)	lin. m.	$ 50.00	1680	$ 84,000.00
.10	Surface Restoration (Street)	lin. m.	$ 1,000.00	2880	$ 2,880,000.00
	SUB-TOTAL				**$ 23,437,440.00**
	Contingency (30%)				$ 7,031,232.00
	TOTAL (2015 Value)				**$ 30,468,672.00**
	Inflation (2% Annually)				$ 6,672,469.00
	TOTAL (2020 Value)				**$ 37,141,141.00**

10-YEAR GROWTH HORIZON UPGRADES (2025)

Capital Cost Estimate

NO.	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	AMOUNT
C.	**CN RAIL TRUNK**				
.1	General Requirements	lump sum			$ 541,660.00
.2	375 mm Dia. Sewer (3 - 4 m Deep) along CNR	lin. m.	$ 1,350.00	530	$ 715,500.00
.3	375 mm Dia. Sewer (4 - 5 m Deep) along CNR	lin. m.	$ 1,450.00	70	$ 101,500.00
.4	450 mm Dia. Sewer (3 - 4 m Deep) along CNR	lin. m.	$ 1,400.00	280	$ 392,000.00
.5	450 mm Dia. Sewer (4 - 5 m Deep) along CNR	lin. m.	$ 1,550.00	90	$ 139,500.00
.6	450 mm Dia. Sewer (3 - 4 m Deep) along CNR	lin. m.	$ 1,700.00	320	$ 544,000.00
.7	450 mm Dia. Sewer (4 - 5 m Deep) along CNR	lin. m.	$ 1,900.00	160	$ 304,000.00
.8	Appurenances and Incidentals	lump sum			$ 439,300.00
.9	Surface Restoration	lin. m.	$ 50.00	1450	$ 72,500.00
	SUB-TOTAL				**$ 3,249,960.00**
	Contingency (30%)				$ 974,988.00
	TOTAL (2015 Value)				**$ 4,224,948.00**
	Inflation (2% Annually)				$ 925,240.00
	TOTAL (2020 Value)				**$ 5,150,188.00**

20-YEAR GROWTH HORIZON UPGRADES (2035)
Capital Cost Estimate

AECOM

NO.	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	AMOUNT
A.	**SOUTH TRUNK**				
.1	General Requirements	lump sum			$ 226,620.00
.2	750 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 1,000.00	140	$ 140,000.00
.3	750 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 1,150.00	670	$ 770,500.00
.4	Appurtenances and Incidentals	lump sum			$ 182,100.00
.5	Surface Restoration	lin. m.	$ 50.00	810	$ 40,500.00
	SUB-TOTAL				**$ 1,359,720.00**
	Contingency (30%)				$ 407,916.00
	TOTAL (2015 Value)				**$ 1,767,636.00**
	Inflation (2% Annually)				$ 858,978.00
	TOTAL (2020 Value)				**$ 2,626,614.00**
B.	**CN RAIL TRUNK**				
.1	General Requirements	lump sum			$ 332,920.00
.2	300 mm Dia. Sewer (3 - 4 m Deep) along CNR	lin. m.	$ 1,300.00	570	$ 741,000.00
.3	300 mm Dia. Sewer (4 - 5 m Deep) along CNR	lin. m.	$ 1,400.00	180	$ 252,000.00
.4	375 mm Dia. Sewer (3 - 4 m Deep) along CNR	lin. m.	$ 1,350.00	100	$ 135,000.00
.5	375 mm Dia. Sewer (4 - 5 m Deep) along CNR	lin. m.	$ 1,450.00	150	$ 217,500.00
.6	Appurenances and Incidentals	lump sum			$ 269,100.00
.7	Surface Restoration	lin. m.	$ 50.00	1000	$ 50,000.00
	SUB-TOTAL				**$ 1,997,520.00**
	Contingency (30%)				$ 599,256.00
	TOTAL (2015 Value)				**$ 2,596,776.00**
	Inflation (2% Annually)				$ 1,261,897.00
	TOTAL (2020 Value)				**$ 3,858,673.00**
C.	**HIGHWAY 16 TRUNK**				
.1	General Requirements	lump sum			$ 172,580.00
.2	450 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 750.00	170	$ 127,500.00
.3	450 mm Dia. Sewer (5 - 6 m Deep)	lin. m.	$ 900.00	470	$ 423,000.00
.4	450 mm Dia. Sewer (6 - 7 m Deep)	lin. m.	$ 1,050.00	130	$ 136,500.00
.5	Appurenances and Incidentals	lump sum			$ 137,400.00
.6	Surface Restoration	lin. m.	$ 50.00	770	$ 38,500.00
	SUB-TOTAL				**$ 1,035,480.00**
	Contingency (30%)				$ 310,644.00
	TOTAL (2015 Value)				**$ 1,346,124.00**
	Inflation (2% Annually)				$ 654,145.00
	TOTAL (2020 Value)				**$ 2,000,269.00**

40-YEAR GROWTH HORIZON UPGRADES (2055)

Capital Cost Estimate

NO.	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	AMOUNT
A.	**NORTH TRUNK**				
.1	General Requirements	lump sum			$ 5,045,500.00
.2	375 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 550.00	1310	$ 720,500.00
.3	375 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 650.00	360	$ 234,000.00
.4	375 mm Dia. Sewer (5 - 6 m Deep)	lin. m.	$ 750.00	340	$ 255,000.00
.5	375 mm Dia. Sewer (6 - 7 m Deep)	lin. m.	$ 850.00	260	$ 221,000.00
.6	375 mm Dia. Sewer (7 - 8 m Deep)	lin. m.	$ 950.00	150	$ 142,500.00
.7	375 mm Dia. Sewer (8 - 9 m Deep)	lin. m.	$ 1,050.00	270	$ 283,500.00
.8	525 mm Dia. Sewer (6 - 7 m Deep)	lin. m.	$ 1,100.00	130	$ 143,000.00
.9	525 mm Dia. Sewer (7 - 8 m Deep)	lin. m.	$ 1,350.00	150	$ 202,500.00
.10	525 mm Dia. Sewer (8 - 9 m Deep)	lin. m.	$ 1,600.00	270	$ 432,000.00
.11	525 mm Dia. Sewer (9 - 10 m Deep)	lin. m.	$ 1,900.00	440	$ 836,000.00
.12	600 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 850.00	430	$ 365,500.00
.13	600 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 1,000.00	580	$ 580,000.00
.14	600 mm Dia. Sewer (5 - 6 m Deep)	lin. m.	$ 1,250.00	250	$ 312,500.00
.15	600 mm Dia. Sewer (6 - 7 m Deep)	lin. m.	$ 1,500.00	490	$ 735,000.00
.16	600 mm Dia. Sewer (7 - 8 m Deep)	lin. m.	$ 1,750.00	320	$ 560,000.00
.17	600 mm Dia. Sewer (8 - 9 m Deep)	lin. m.	$ 2,000.00	540	$ 1,080,000.00
.18	900 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 1,300.00	470	$ 611,000.00
.19	900 mm Dia. Sewer (5 - 6 m Deep)	lin. m.	$ 1,600.00	1100	$ 1,760,000.00
.20	900 mm Dia. Sewer (6 - 7 m Deep)	lin. m.	$ 2,000.00	330	$ 660,000.00
.21	900 mm Dia. Sewer (7 - 8 m Deep)	lin. m.	$ 2,500.00	830	$ 2,075,000.00
.22	1050 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 1,500.00	290	$ 435,000.00
.23	1050 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 1,900.00	40	$ 76,000.00
.24	1050 mm Dia. Sewer (5 - 6 m Deep)	lin. m.	$ 2,400.00	50	$ 120,000.00
.25	1050 mm Dia. Sewer (6 - 7 m Deep)	lin. m.	$ 3,400.00	210	$ 714,000.00
.26	1050 mm Dia. Sewer (7 - 8 m Deep)	lin. m.	$ 3,900.00	150	$ 585,000.00
.27	1050 mm Dia. Sewer (8 - 9 m Deep)	lin. m.	$ 4,200.00	120	$ 504,000.00
.28	1050 mm Dia. Sewer (9 - 10 m Deep)	lin. m.	$ 5,000.00	210	$ 1,050,000.00
.29	1050 mm Dia. Sewer (10 - 11 m Deep)	lin. m.	$ 5,600.00	130	$ 728,000.00
.30	1050 mm Dia. Sewer (11 - 12 m Deep)	lin. m.	$ 6,100.00	420	$ 2,562,000.00
.31	1050 mm Dia. Sewer (12 - 13 m Deep)	lin. m.	$ 6,700.00	210	$ 1,407,000.00
.32	Appurtenances and Incidentals	lump sum			$ 4,078,000.00
.33	Surface Restoration	lin. m.	$ 70.00	10850	$ 759,500.00
	SUB-TOTAL				**$ 30,273,000.00**
	Contingency (30%)				$ 9,081,900.00
	TOTAL (2015 Value)				**$ 39,354,900.00**
	Inflation (2% Annually)				$ 47,542,280.00
	TOTAL (2020 Value)				**$ 86,897,180.00**

40-YEAR GROWTH HORIZON UPGRADES (2055)
Capital Cost Estimate

NO.	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	AMOUNT
B.	**HIGHWAY 16 TRUNK**				
.1	General Requirements	lump sum			$ 258,020.00
.2	375 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 550.00	580	$ 319,000.00
.3	375 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 650.00	220	$ 143,000.00
.4	375 mm Dia. Sewer (5 - 6 m Deep)	lin. m.	$ 750.00	80	$ 60,000.00
.5	450 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 600.00	410	$ 246,000.00
.6	450 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 750.00	320	$ 240,000.00
.7	Appurtenances and Incidentals	lump sum			$ 201,600.00
.8	Surface Restoration	lin. m.	$ 50.00	1610	$ 80,500.00
	SUB-TOTAL				**$ 1,548,120.00**
	Contingency (30%)				$ 464,436.00
	TOTAL (2015 Value)				**$ 2,012,556.00**
	Inflation (2% Annually)				$ 2,431,247.00
	TOTAL (2020 Value)				**$ 4,443,803.00**
C.	**SOUTH TRUNK**				
.1	General Requirements	lump sum			$ 218,700.00
.2	525 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 650.00	150	$ 97,500.00
.3	525 mm Dia. Sewer (4 - 5 m Deep)	lin. m.	$ 800.00	650	$ 520,000.00
.4	750 mm Dia. Sewer (3 - 4 m Deep)	lin. m.	$ 1,000.00	250	$ 250,000.00
.5	Appurtenances and Incidentals	lump sum			$ 173,500.00
.6	Surface Restoration	lin. m.	$ 50.00	1050	$ 52,500.00
	SUB-TOTAL				**$ 1,312,200.00**
	Contingency (30%)				$ 393,660.00
	TOTAL (2015 Value)				**$ 1,705,860.00**
	Inflation (2% Annually)				$ 2,060,747.00
	TOTAL (2020 Value)				**$ 3,766,607.00**