MPE Engineering Ltd.

Final Report for:

TOWN OF Westlock

INFRASTRUCTURE ASSESSMENT AND 10-YEAR CAPITAL PLAN – FINAL REPORT

March 2, 2020
Project Number 5454-006-00

Proud of Our Past... Building the Future

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MPE Engineering Ltd.

March 2, 2020 File: N:\5454\006-00\R01

Town of Westlock
10003-106 Street
Westlock, AB T7P 2K3

Attention: Simone Wiley
Chief Administrative Officer

Dear Simone:

Re: Town of Westlock Infrastructure Assessment and 10-Year Capital Plan – Final Report

MPE Engineering Ltd. is pleased to submit our Final Report to the Town of Westlock for the Infrastructure Assessment and 10-Year Capital Plan.

We appreciate the opportunity to provide our services for this project. Should you have any questions or require additional information, please contact the undersigned at (780) 509-4304.

Yours truly,

MPE ENGINEERING LTD.

Mirek Grzeszczuk, P.Tech.(Eng.)
Edmonton Region Manager

PM/lp
Enclosure

CORPORATE AUTHORIZATION

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SCOTT A. KUSALIK
2020-03-02

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The Association of Professional
Engineers and Geoscientists of Alberta

Scott Kusalik, P.Eng.

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Corporate Permit

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1.0 INTRODUCTION

1.1 Overview

The Town of Westlock (Town) requires an assessment of the following infrastructure for the purpose of planning and budgeting for maintenance and upgrades:

- Water Distribution System
- Wastewater Collection and Treatment System
- Storm Drainage System

The Town has authorized MPE Engineering Ltd. (MPE) to perform an inventory of this infrastructure and provide recommendations for upgrades.

1.2 Study Scope

The focus of this assessment is to review the condition and capacity of Town-owned capital assets in the water distribution, wastewater collection and storm drainage systems. MPE will review all information pertaining to all Town water, wastewater and storm drainage assets, determine the condition of the assets, and identify the maintenance needs of the assets.

1.3 Objective

The objective of this assessment is to evaluate the condition of the Town's water distribution, wastewater collection and storm drainage infrastructure, and provide recommendations for upgrades and rehabilitation to the infrastructure including a 10-Year Capital Plan complete with cost estimates.

1.4 Design Standards and Guidelines

The design criteria utilized for the Town is based on design parameters developed in this assessment. MPE prepared this report in accordance with the following standards and guidelines as a minimum:

- *Town of Westlock, Procedures and Design Standards for Development, October 2009*, Town of Westlock
- *Town of Westlock, Wastewater Collection System Master Plan – 2009 Update, November 2009*, ISL Engineering and Land Services
- *Town of Westlock, Water Distribution System Master Plan – 2009 Update, November 2009,* ISL Engineering and Land Services
- *Town of Westlock, Stormwater Master Plan Update – 2017, January 18, 2018,* MPE Engineering Ltd.

1.5 Infrastructure Upgrade Priority

MPE has developed the following priorities for recommended infrastructure upgrades/rehabilitation to aid in development of the 10-Year Capital Plan:

- *Priority 1: Recommended upgrade/rehabilitation remedies an issue which could cause harm to the environment or the public.*
- *Priority 2: Recommended upgrade/rehabilitation remedies an issue that will bring the infrastructure to a good level of service.*
- *Priority 3: Recommended upgrade/rehabilitation is remedied by the Town through ongoing operation and maintenance of the infrastructure*.

A good level of service is one where the asset is in good condition, with no major structural, capacity, or operation and maintenance issues.

2.0 POPULATION ANALYSIS AND PROJECTIONS

Population figures obtained from Statistics Canada show that the population of the Town was 5,101 in the 2016 Federal Census.

The *Town of Westlock, Water Tower and Pumping Station Assessment, April 13, 2017,* completed by MPE Engineering Ltd. (Water Tower Assessment) assumed a 1.1% growth rate for the Town to calculate a 2017 population of 5,157. This growth rate is consistent with the Westlock Regional Water System Business Plan.

Table 2.1 shows the population of the Town recorded in the last four Federal Censuses, and last two Municipal Censuses.

TABLE 2.1: HISTORICAL POPULATION OF THE TOWN OF WESTLOCK

Year	Population	Growth Rate
2001 (Federal Census)	4,819	
2006 (Federal Census)	5,008	3.9%
2008 (Municipal Census)	4,964	-0.9%
2011 (Federal Census)	4,823	-2.8%
2015 (Municipal Census)	5,147	6.7%
2016 (Federal Census)	5,101	-0.9%

For the purpose of this report, MPE will use the 1.1% growth rate for the Town that was used in the Water Tower Assessment, which is consistent with the Westlock Regional Water System Business Plan. A population of 5,101 will also be assumed from the 2016 Federal Census.

For future development, MPE will assume that residential development occurs at a density of 30 persons/hectare. It is also assumed that non-residential development growth will be equivalent to half the residential area developed.

3.0 WATER SUPPLY AND DISTRIBUTION SYSTEM

3.1 General

3.1.1 Per-Capita Demand

The per-capita daily demand is typically determined by dividing the total annual consumption by 365 days. By dividing this rate by the population served, the per capita/per day demand is calculated. This rate is used primarily as the basis for the projection of the total water demand.

To determine the per capita demand, the monthly consumption information for 2016 is shown below:

Month	Monthly Consumption (m^3)
January	53,267
February	51,414
March	54,153
April	54,107
May	67,021
June	66,073
July	64,239
August	61,788
September	57,908
October	56,921
November	55,398
December	54,633
Total Year Usage	**696,922**
Population	5,101
Average Daily Flow (m^3)	1,909
Average Daily Flow (L/c/day)	**374**

The *Town of Westlock, Procedures and Design Standards for Development, October 2009* (Design Standards), the *Town of Westlock Water Distribution System Master Plan,* by ISL in January 2005 (2005 Water Master Plan), and the *Water Distribution System Master Plan, 2009 Update*, completed by ISL Engineering and Land Services in November 2009 (2009 Water Master Plan Update) uses an average day water consumption of 360 L/c/day. The Westlock Regional Water Commission (Commission) prepared a business plan in 2012, which allocates an average day water consumption of 400 L/c/day. As the Commission's Business Plan is a more recent document than the Master Plans or the Town's Design Standards, the average day consumption rate of 400 L/c/day will be used for the residential areas of the Town for this study.

The design criteria used to assess the commercial/institutional and industrial development within the existing water distribution system was taken from the 2005 Water Master Plan. These demands were based on water billing records. The demands are:

- Commercial/Institutional – 3250 L/ha/day
- Industrial – 2000 L/ha/day

The Town's Design Standards, as well as those for other municipalities, does not provide water consumption values for existing systems. The design standard values are developed for future developments, and are conservative because the type of development is not known. As such, MPE has assumed that the commercial/industrial consumption within the existing water distribution system from the 2005 Water Master Plan has not changed. These values are a better representation of actual commercial/industrial water consumption in the Town than the design standard values. MPE recommends the Town confirm these values for the existing system in their next Water Distribution Master Plan.

For future development, MPE used values from the Town's Design Standards for commercial/institutional and industrial water consumption, which are:

- Commercial/Institutional Water Consumption – 6500 L/ha/day
- Industrial Water Consumption – 4000 L/ha/day

The water consumption values used for future development are consistent with those for future development from other municipalities such as Parkland County and Sturgeon County. For other municipalities these values are approximately half of other consumption values. MPE recommends the Town revisit these consumption values in their next Water Distribution Master Plan.

3.1.2 Maximum Day

The maximum day demand is determined by the single day of maximum consumption observed in the distribution system.

The Town's Design Standards and the 2005 and 2009 Water Master Plans use a maximum day demand of 2.0 times the average day demand. The Commission Business Case uses a value of 1.8 times the average day demand. As with the average day demand, MPE will use a maximum day demand of 1.8 times the average day demand, as the Commission's Business Case is a more recent document.

MPE will use the value from the Town's Design Standards for non-residential maximum day demand, which is 1.8 times the average day demand.

3.1.3 Peak Hour

The peak hour demand is the expected maximum demand observed during a short period of the day. The rate is generally established based on experience.

The Town's Design Standards and the 2005 and 2009 Water Master Plans use a peak hour demand of 3.0 times the average day demand. The Commission's Business Case used a value of 2.0 times the maximum day demand. As with the average day demand and maximum day demand, MPE will use a peak hour demand of 2.0 times the maximum day demand, as the Commission's Business Case is a more recent document.

For non-residential development, MPE will use the value from the Town's Design Standards, which is 2.0 times the average day demand.

The peaking factors are consistent with the 1.8 to 2.0 times the average design flow value used in the Alberta Environment and Parks Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems.

3.2 Background Information

Water is supplied to the Town by the Westlock Regional Water Services Commission (WRWSC) through the Town's Water Treatment Plant and Core Area Reservoir.

3.3 Historical Water Usage and Design Criteria

3.3.1 Water Pressure Design Criteria

An analysis will be made for peak hour demand, and watermains will be sized such that there will be a minimum residual pressure of 275 kPa at ground level at any location in the system. A separate analysis will be made for maximum day demand plus fire flow. For this analysis, the residual pressure at any location at the ground level will not be less than 140 kPa. Both values are from the Town's Design Standards.

3.3.2 Fire Flow Requirements

The Town's Design Standards state that fire flow requirements will be in accordance with the latest edition of the Fire Underwriters Survey (FUS).

The 2009 Water Master Plant Update used the following fire flow criteria for various development rate types based on FUS recommendations.

- Single Family Residential: 76 L/s
- Multi-Family Residential: 114 – 227 L/s (227 L/s recommended for large facilities such as schools and hospitals)
- Industrial: 227 L/s
- Commercial: 265 L/s

The *Town of Westlock Water Tower and Pumping Station Assessment, April 13, 2017* (Water Tower Assessment) used a fire flow for Westlock of 230 L/s over a three-hour period.

For the purpose of this study, MPE requires fire flow requirements for all development rate types. We will use fire flow criteria from both the 2009 Water Master Plan Update and Water Tower Assessment for this study:

- Single Family Residential: 76 L/s
- Multi-Family Residential: 114 – 230 L/s (230 L/s recommended for large facilities such as schools and hospitals)
- Industrial: 230 L/s
- Commercial: 230 L/s

3.4 Previous Studies

3.4.1 Previous Recommendations

The 2009 Water Master Plan Update lists recommendations to upgrade the existing system and future development servicing.

The recommendations to upgrade the existing water distribution system are:

- Construct a new 250 mm diameter watermain east from the Pickardville water supply line to the arena complex.
- Construct a new 250 mm diameter watermain from the 200 mm diameter watermain along 103 Street east of 111 Avenue, east to near the east Town boundary. The watermain will then run south to the existing 250 mm diameter watermain south of Highway 18.

The recommendations for future development servicing are:

- Construct future systems with looping.

- Assess the future system in the context of the impacts of the regional system on the Town's distribution system to ensure any upgrading required for the regional system to be facilitated can be undertaken. MPE understands that if the Town's pressure drops too low, the regional system feed will be shut off at the regional system booster station outside of the Town.
- The pressures in the Town system in the 2009 existing system model appear high in the northwest portion of Town, though no pressure issues have been reported in Town. No high pressure issues exist in the 2009 long-term model due to higher demands. In the short-term, as the Town expands to the northwest and northeast into lower elevation areas, pressures should be monitored through hydrant testing. If excessive pressures are noted, additional pressure zones may be required.

3.4.2 Implemented Recommendations

Further to the 2009 Water Master Plan Update and discussions with the Town, MPE understands the Town has completed the following recommended upgrades:

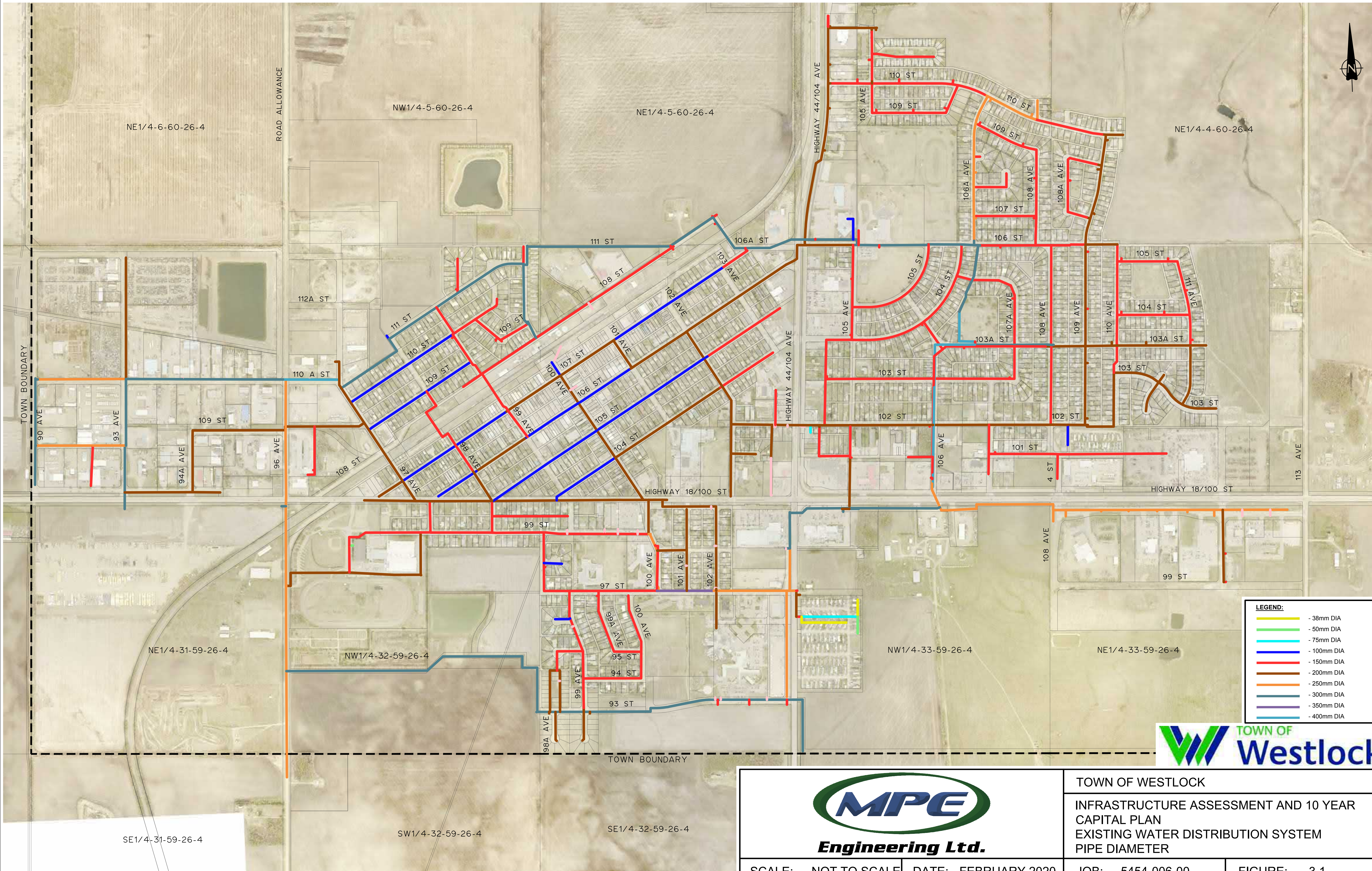
1. The Town constructed a new 200 mm diameter watermain to the arena complex.
2. The Town added some 300 mm diameter watermain looping along the south perimeter of the Town.

3.5 Existing Distribution System

The existing distribution system consists of approximately 48.5 km of 38 mm, 50 mm, 75 mm, 100 mm, 150 mm, 200 mm, 250 mm, 300 mm, 350 mm and 400 mm diameter watermains, and is shown in ***Figure 3.1 (Existing Water Distribution System Pipe Diameter)***. The figure also shows that the diameter of some watermain sections is not known.

The known decade of installation of watermains in the distribution system is shown in ***Figure 3.2 (Existing Water Distribution System Decade Installation)***. As shown, the earliest decade of installation is in the 1960s, and information provided by the Town shows that the earliest installation date is 1961.

The known watermain materials are asbestos cement, ductile iron, and Polyvinyl Chloride (PVC). There are some sections of watermain where the material is not known. The Town is working on their Asset Management Plan, and as this plan matures the material types will be confirmed. For watermains where the material is not known, MPE has assumed the material type based on the installation date. Watermains installed prior to 1978 are assumed to be asbestos cement, and those installed after 1978 are assumed to be PVC.



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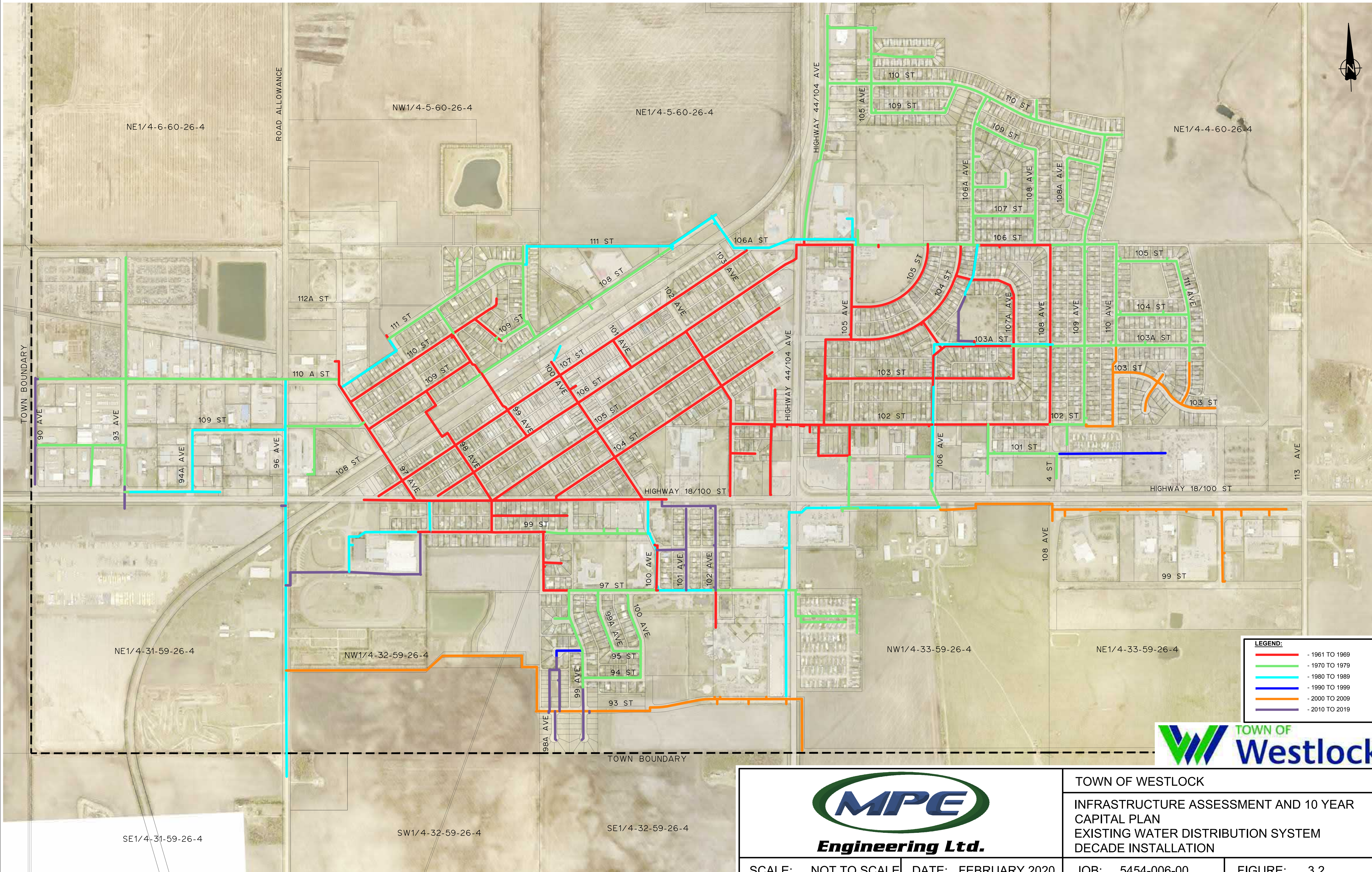
TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
EXISTING WATER DISTRIBUTION SYSTEM
PIPE DIAMETER

SCALE:	NOT TO SCALE	DATE:	FEBRUARY 2020	JOB:	5454-006-00	FIGURE:	3.1



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TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
EXISTING WATER DISTRIBUTION SYSTEM
DECADE INSTALLATION

SCALE: NOT TO SCALE	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 3.2

Figure 3.1 shows that many watermains in the Town are 75 mm, 100 mm and 150 mm in diameter. The *Town of Westlock, Procedures and Design Standards for Development, October 2009* (Design Standards), states that the minimum size of water distribution mains should be:

- 150 mm diameter for short single family cul-de-sacs
- 200 mm diameter for single family residential developments
- 250 mm diameter for multi-family developments
- 300 mm diameter for industrial and commercial developments

As the 75 mm, 100 mm and 150 mm diameter watermains are replaced, MPE recommends that the Town replace them with an increased size based on the requirements from the Design Standards above.

3.6 Watermain Condition

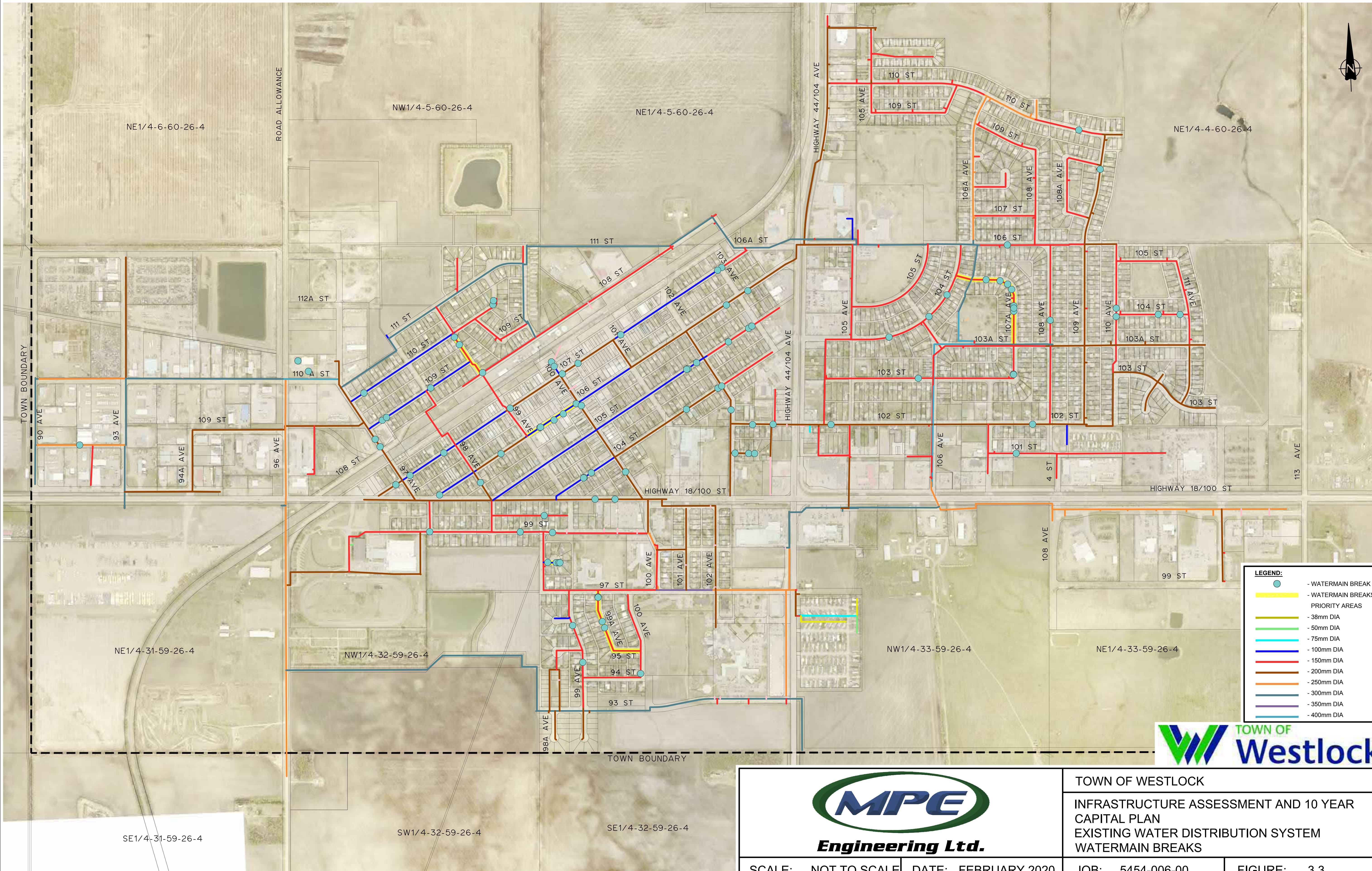
Many communities use a service life of 75 years for watermains. Based on this, watermains installed in the 1960s have approximately 17 years of service remaining. While these mains are nearing the end of their service life, they may survive beyond the 75-year timeframe.

The Town has provided a break history for the watermains since 1997. This is shown in ***Figure 3.3 (Existing Water Distribution System Watermain Breaks)***. The figure shows there are some areas of the Town that have had multiple breaks in the past 10 years. Sections of watermain with two or more breaks in the last 20 years were installed in the 1960s or 1970s, which corresponds to the mains being asbestos cement pipe. The four sections of main with the most breaks in the last 20 years are shown in Figure 3.3, and are:

- 99A Avenue, between 97 Street and 95 Street – 4 breaks in the last 10 years
- 107A Avenue, between 103A Street and 104 Street – 7 breaks in the last 20 years
- 106 Street, between 99 Avenue and 100 Avenue – 5 breaks in the last 10 years
- 98 Street, east of 98A Avenue – 4 breaks in the last 20 years

MPE recommends that the sections of watermain listed above be the first priority for watermain replacement due to condition. Sections of mains with 2 or 3 breaks in the last 10 to 20 years are recommended to be the second priority. These sections are:

- 110 Avenue, between 103A Street and 105 Street – 2 breaks in the last 20 years
- 107A Avenue, between 103 Street and 103A Street – 2 breaks in the last 10 years
- 104 Street, between 106 Avenue and 107A Avenue – 2 breaks in the last 20 years
- 110 Street, between 100A Avenue and 111 Street – 2 breaks in the last 20 years
- 104 Street, between 110 Avenue and 111 Avenue – 2 breaks in the last 20 years
- 109 Street, between 97 Avenue and 98 Avenue – 2 breaks in the last 10 years



TOWN OF WESTLOCK			
INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN EXISTING WATER DISTRIBUTION SYSTEM WATERMAIN BREAKS			
SCALE: NOT TO SCALE	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 3.3

- 97 Avenue, between 108 Street and 109 Street – 2 breaks in the last 10 years
- 107 Street, between 97 Avenue and 98 Avenue – 2 breaks in the last 10 years
- Intersection of 107 Street and 100 Avenue – 3 breaks in the last 20 years
- 105 Street, between 102 Avenue and 103 Avenue – 3 breaks in the last 10 years
- 105 Street, between 101 Avenue and 102 Avenue – 3 breaks in the last 20 years
- 104 Street, between 99 Avenue and 100 Avenue – 3 breaks in the last 10 years
- 104 Street, between 101 Avenue and 102 Avenue – 3 breaks in the last 10 years
- 100 Street, between 100 Avenue and 99 Avenue – 2 breaks in the last 10 years
- Intersection of 103 Avenue and 107 Street – 2 breaks in the last 20 years
- 101 Street, East of 102 Avenue – 3 breaks in the last 20 years
- 99 Avenue, between 110 Street and 108 Street – 3 breaks in the last 20 years

Sections of watermain with 1 break or less in the last 10 years are recommended to be the third and final priority in watermain replacement due to condition.

3.7 Treated Water Storage

The 2009 Water Master Plan Update determined that additional treated water storage was required if the Town replaced the Water Tower. The Town is currently replacing the Water Tower with a 3,000 m^3 treated water reservoir. Using this volume, and the storage at the East Underground Reservoir (2,275 m^3), the total treated water storage available to the Town is 5,275 m^3.

Required water storage is calculated using the requirements of the Westlock Regional Water Services Commission (WRWSC). The WRWSC requires that all communities on the regional supply system provide a minimum storage equal to 2.0 times the daily average demand.

Using the population and growth rate from Section 2.0, the projected population of the Town in 2018 is 5,214. Using this population, the required 2018 treated water storage for the Town is 4,171 m^3. Projecting the population forward 10 years, provides a population of 5,817, and a required treated water storage of 4,654 m^3. Therefore, the Town has sufficient treated water storage for the next 10 years.

4.0 WATER DISTRIBUTION NETWORK ANALYSIS

4.1 General Conditions

MPE reviewed the ability of the water distribution system to meet the pressure and flow requirements of the Town. A review of the existing system using Bentley's WaterCAD Software was completed. Hydrant testing was performed by Town Staff on May 18, 2019.

The Town is currently replacing the existing water tower with the WRPS that will be fed from the Town's Water Treatment Plant (WTP).

4.2 Abbreviations

ADD = Average Day Demand

MDD = Maximum Day Demand

MDD + FF = Maximum Day + Fire Flow

PHD = Peak Hour Demand

WTP = Water Treatment Plant

4.3 Hydraulic Model

The hydraulic analysis of the water distribution system was completed using Bentley WaterCAD computer modeling software. Modeling was completed using the steady state simulation that refers to a state that is unchanging in time. For this analysis, a steady state simulation was deemed sufficient to assess the capacity of the distribution system.

4.4 Design Flows Maximum Day and Peak Hour

Utilizing the design criteria in Section 3.0, the total design flows were calculated and input into the model for each land use type, as summarized in Table 4.1.

TABLE 4.1: WATER DESIGN FLOWS

	Existing	Future	Future + Existing
Average Day Demand (ADD)	2,973 m^3/d	295 m^3/d	3,268 m^3/d
Maximum Day Demand (MDD)	5,351 m^3/d	530 m^3/d	5,881 m^3/d
Maximum Day Demand (MDD)	62 L/s	6 L/s	68 L/s
MDD + Maximum Fire Flow (FF) Standard[1]	292 L/s	236 L/s	298 L/s
Peak Hour Demand (PHD)	107 L/s	11 L/s	118 L/s

[1] Maximum fire flow standard is 265 L/s for commercial land use type.

4.5 Distribution Pumping Capacity

Distribution pumping systems are designed to meet the peak hour demand for the service area for the design period. The distribution system is serviced by either the WTP or the WRPS. The WTP and WRPS can operate concurrently, however, MPE reviewed the operation of these two facilities independently. The WTP distributes water through the distribution system, while, at the same time, filling the treated reservoirs located at the WRPS. The WTP's high lift pumps turn on when there is a call for water to fill the east reservoir. If the WRPS is down, the WTP is available to provide back-up supply to the distribution system. The facility that operates under typical service conditions is the WRPS.

The following table illustrates the pumping capacity at each of the facilities within the Town.

TABLE 4.2: WTP AND WATER RESERVOIR AND PUMPING STATION CAPACITY

	WTP		WRPS	
	High Lift Pump	Fire Pump	Dist. Pump	Fire Pump
Number of Pumps	3	0	3	1
Capacity (L/s)	**60 L/s**	**N.A**	**60 L/s**	**245 L/s**
Pressure (kPa/psi)	480 kPa/69psi	N.A	435 kPa/63 psi	435 kPa/63 psi
Total Capacity* (L/s)	120 L/s	N.A	120 L/s	245 L/s
Total Fire Flow Capacity* (L/s)	120 L/s		365 L/s	

*Assuming one pump remains on standby at all times at each facility.

The capacity of the fire pump at the WRPS in Table 4.2 is the capacity of the pump installed. This capacity is above the fire flow value used in the Water Tower Assessment, which was used to size the pumps at the WRPS.

4.6 Field Calibration Methodology

On May 18, 2019, Town staff performed flow testing on ten fire hydrants within the Town. The hydrants were selected to represent the following criteria:

- Older Pipe (Installed prior to 1980)
- Low Fire Flow Available Compared with Adjacent Areas
- Relative Distance from the WRPS and WTP
- High Fire Flow Required (Institutional, Industrial, and Commercial Land Uses)

The hydrants were first tested for static pressure, and then opened to measure the residual pressure and corresponding flow. These flows were then applied to their locations in the model, the static and residual pressures were compared, and the model was then calibrated to represent actual field conditions. The ***Hydrant Test Locations*** are shown in ***Figure 4.1***. The ***Fire Hydrant Field Tests*** are attached as ***Appendix A***.

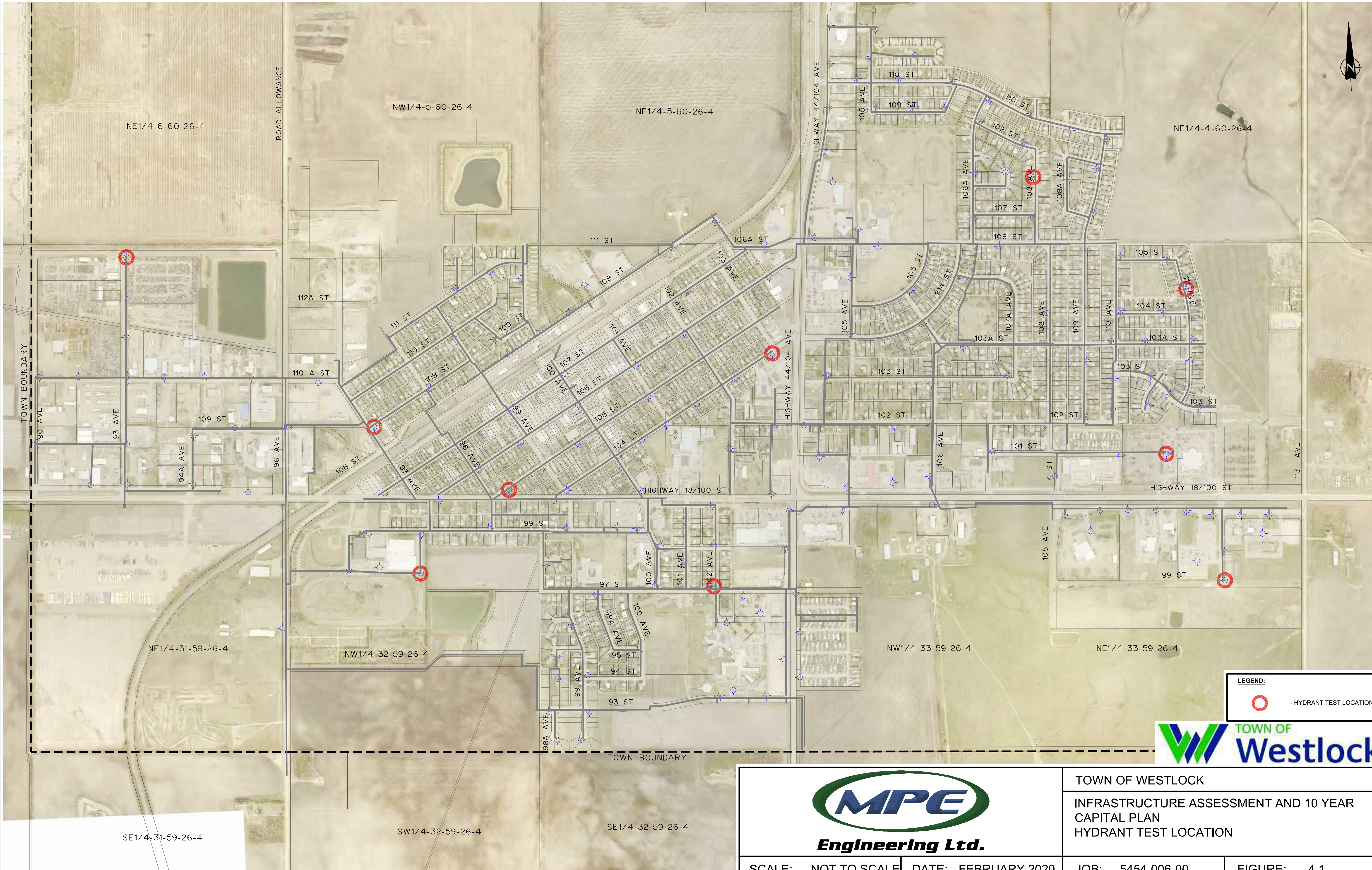
The flow from each fire hydrant field test was input into the hydraulic water model, and the field versus model pressures were compared for the locations shown in Figure 4.1. The desired unilateral flow through an isolated or dead-end main was achieved for 5 of the 10 sites, and partially achieved for one site. Overall, the fire flow testing in the field correlated well with the hydraulic water model. Some field data had lower or questionable accuracy. If the field data with lower or questionable accuracy are removed from the data set, the pressure in the water model is 14 kPa (2 psi) higher than the field tests on average. The data with questionable accuracy for specific locations is discussed further below.

The following is recommended for further investigation by the Town related to the fire flow testing:

- The Town reported that the field test at the hydrant near the hospital was not completed due to concerns about past impacts of fire flow testing to hospital equipment. A fire flow of 70 L/s was input into the model at Hydrant 170 adjacent to the hospital, which reduced the pressure in the model by 21 kPa (3 psi). It is not expected that this pressure drop will affect the service to the hospital. The Town should investigate this issue to determine if it is caused by a partially or completely closed isolation valve located nearby and/or inadequate pipe hydraulic capacity on the hospital site.
- The field test at Test Location #2 along 109 Street indicated 283 kPa (41 psi) lower pressure in the field compared with the model. This suggests a possible restriction in the pipe such as a partially closed upstream valve and/or severe tuberculation in the 100 mm main installed between 1961 and 1969. Regardless of the possibility of tuberculation, the Town should upsize the 100 mm diameter mains to 200 mm in diameter, as recommended in Section 3.5.
- The field test results at Test Location #4 and #8 suggest that the actual pipe in the field may be of larger diameter than is shown in Town records.
  - Location #4: If approximately 400 m of upstream pipe is increased from 150 mm to 200 mm diameter in the model, the model closely matches the field results.
  - Location #8: If approximately 120 m of upstream pipe is increased from 200 mm to 250 mm diameter, the model closely matches the field results.

4.7 Hydraulic Model Analysis

The hydraulic review considers the following scenarios.



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TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
HYDRANT TEST LOCATION

SCALE: NOT TO SCALE	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 4.1

TABLE 4.3: SCENARIOS FOR THE HYDRAULIC MODEL REVIEW

	MDD+FF		PHD	
Supply Source-->	**WTP High Lift Pumps**	**WRPS**	**WTP High Lift Pumps**	**WRPS**
Existing	--	1B	--	1D
Existing + Upgrades	--	2B	--	--
Future - 10 Year	--	3B	--	3D

Important Note: Scenarios supplied by the **WRPS** are "normal operating scenarios". Scenarios supplied by the WTP alone are "emergency operation scenarios", in which the WRPS is offline, which MPE assumed will occur rarely, if ever. MPE will only review scenario(s) 1B, 2B, and 3B, for fire flow capacities, as these scenarios represent "normal operating conditions".

4.8 Maximum Day and Peak Hour Demand Summary

TABLE 4.4: DISTRIBUTION CAPACITY ANALYSIS

			DISTRIBUTION CAPACITY	
Analysis	**System**	**Capacity Required**	**WTP**	**WRPS**
Maximum Day	Existing	62 L/s	120 L/s	120 L/s
Peak Hour Demand	Existing	107 L/s	120 L/s	120 L/s
Maximum Day	Future	68 L/s	120 L/s	120 L/s
Peak Hour Demand	Future	118 L/s	120 L/s	120 L/s

As shown in Table 4.4, both the WTP and the **WRPS** have adequate capacity to supply water for peak hour demand in the future and current system.

4.9 Fire Flow Analysis Summary

TABLE 4.5: FIRE FLOW PUMPING CAPACITY ANALYSIS

			FIRE FLOW CAPACITY	
Analysis	**System**	**Capacity Required**	**WTP**	**WRPS**
Maximum Day + Fire Flow (Residential)	Existing	138 L/s	120 L/s	365 L/s
Maximum Day + Fire Flow (Industrial)	Existing	292 L/s	120 L/s	365 L/s
Maximum Day + Fire Flow (Commercial)	Existing	292 L/s	120 L/s	365 L/s
Maximum Day + Fire Flow (Residential)	Future	144 L/s	120 L/s	365 L/s
Maximum Day + Fire Flow (Industrial)	Future	298 L/s	120 L/s	365 L/s
Maximum Day + Fire Flow (Commercial)	Future	298 L/s	120 L/s	365 L/s

Fire Flow Capacity Requirements for residential, industrial, and commercial are based on Section 3.3.2

As shown in Table 4.5, the WTP does not have adequate pumping capacity to meet maximum day + fire flows for residential, industrial, or commercial properties. These results have been tabulated in red above.

The WRPS has adequate pumping capacity to supply fire flows for the future and current system for residential, industrial, and commercial properties. These results have been tabulated in green above. The fire flow capacity for the WRPS is based on two (2) distribution pumps and the one (1) fire pump operating simultaneously.

The combination of existing pipe sizes, looping locations, and distance from the WRPS limits the Town's ability to meet fire flow requirements within different areas of Westlock.

4.9.1 Fire Flow Analysis – Existing System (Scenario 1B)

As stated in Section 4.8, the WRPS has adequate pumping capacity to meet the demands of maximum day + fire flow, for both residential and commercial analyses.

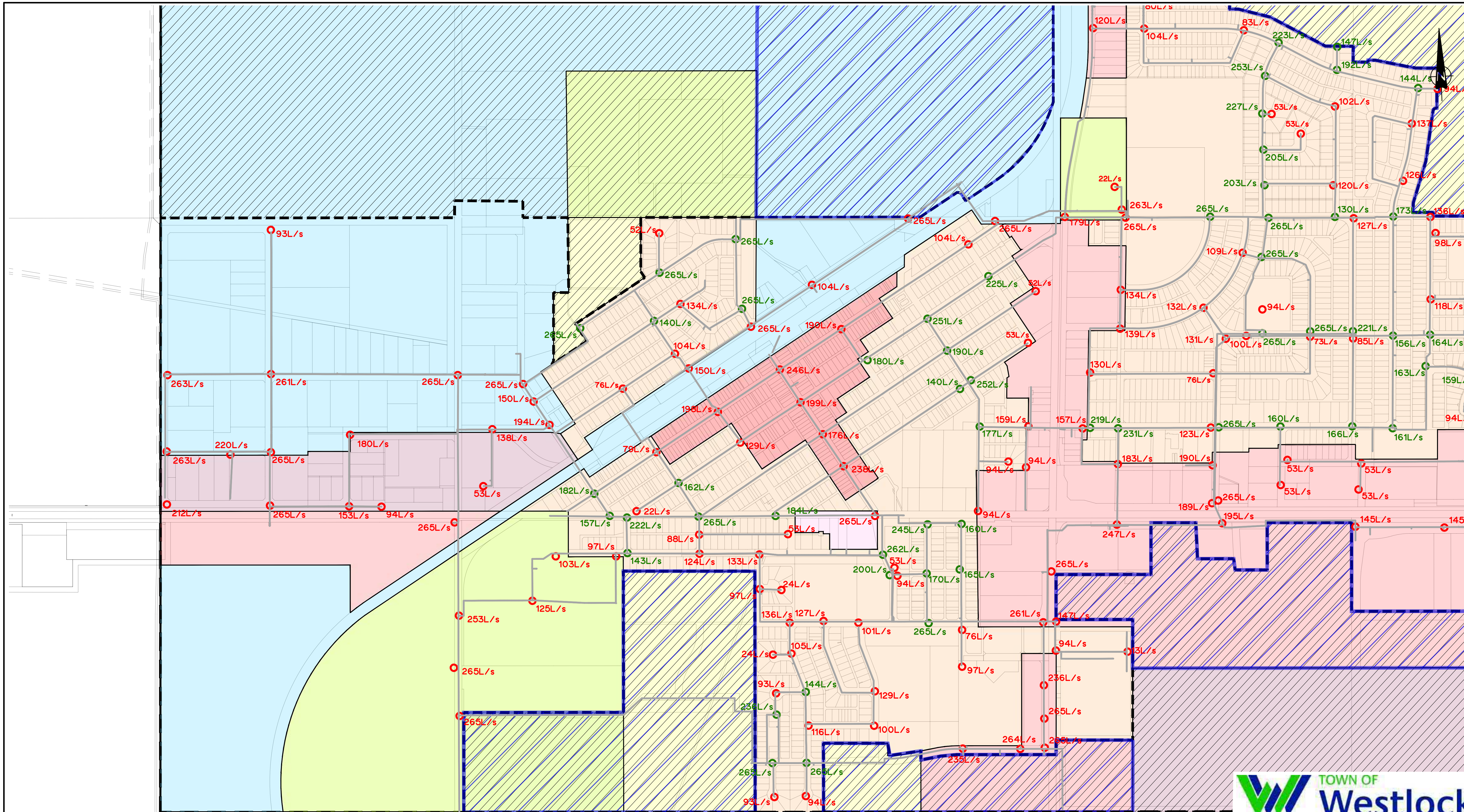
While the WRPS is able to provide adequate pumping capacity, the combination of existing pipe sizes, looping locations, and distance from the WRPS limits the Town's ability to meet fire flow requirements for residential and commercial development. This limited ability results in deficient fire flows in many locations throughout Town.

Figure 4.2 (Available Fire Flow – West) and ***Figure 4.3 (Available Fire Flow – East)*** illustrate the Town's ability to meet fire flow demands for corresponding land uses within different areas of Town.

4.10 Proposed Distribution System Upgrades – Impact(s) to Fire Flow Availability

Four (4) areas within Town were evaluated to assist in prioritizing watermain upgrades to improve fire flow availability. These upgrades are reflected in the 10-Year Capital Plan (Figure 7.1) and are shown in ***Figure 4.4 (Proposed Distribution System Upgrades)***.

1. Commercial Area in the SE (north and south of Highway 18)
2. Central Commercial Area along Highway 44 (between Highway 18 and 106A Street)
3. Downtown
4. Industrial Area on the West Side (between 93 Ave, 110A St, Highway 18 & 97 Ave)



LEGEND:

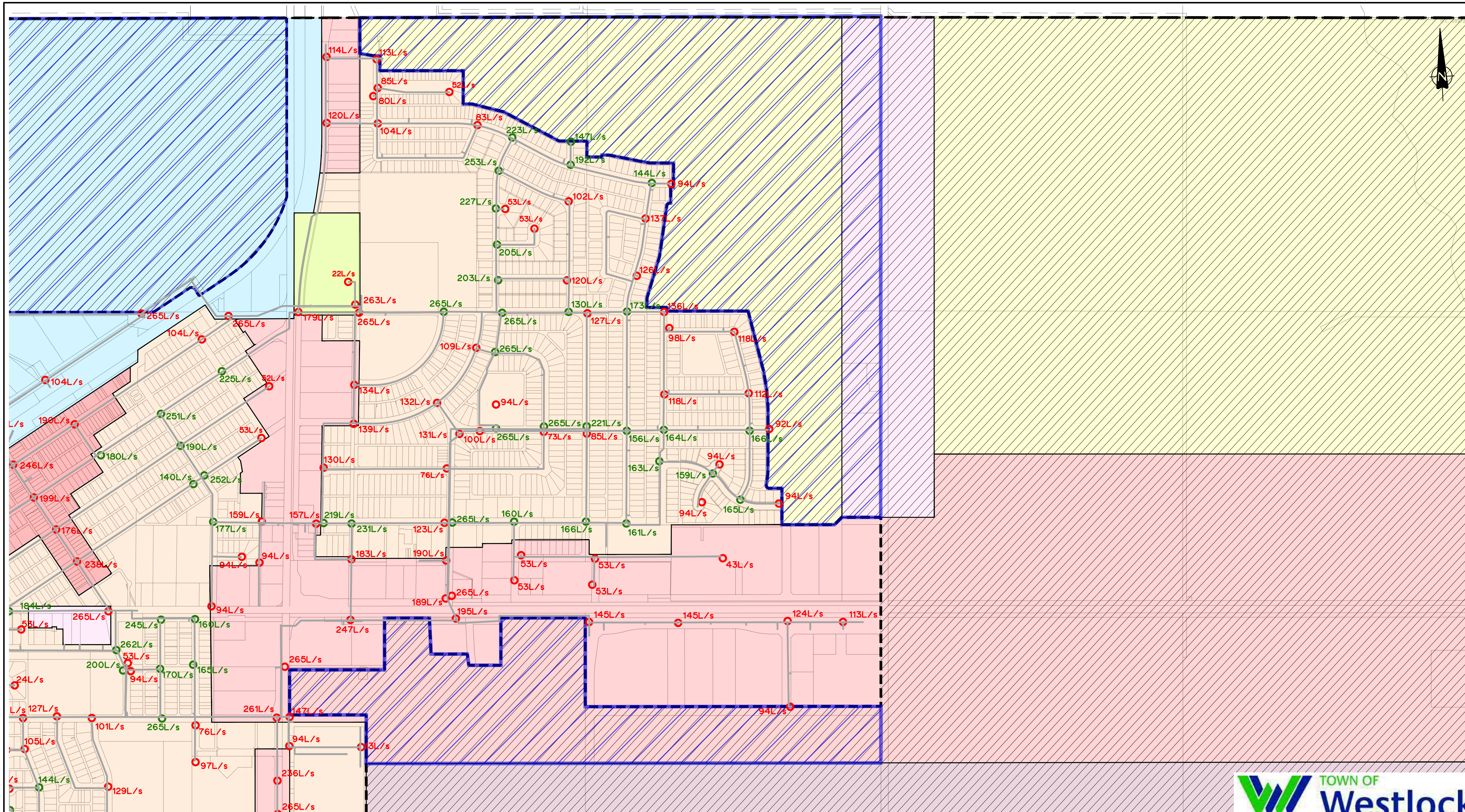
- MN - EX MATURE NEIGHBOURHOODS (A=244.9 Ha)
- IP - EX INDUSTRIAL PARK (A=126.6 Ha)
- BP - EX BUSINESS PARK (A=20.1 Ha)
- DT - EX DOWNTOWN (A=14.6 Ha)
- HC - EX HIGHWAY CORRIDOR (A=105.4 Ha)
- MUC - EX MIXED USE CORRIDOR (A=2.0 Ha)
- RAA - EX REGIONAL AMENITIES AREA (A=53.0 Ha)
- EX DEVELOPMENT BOUNDARY (A=1,395.6 Ha)
- FN - FUTURE NEIGHBOURHOODS (A=287.0 Ha)
- IP - FUTURE INDUSTRIAL PARK (A=171.6 Ha)
- BP - FUTURE BUSINESS PARK (A=120.3 Ha)
- HC - FUTURE HIGHWAY CORRIDOR (A=182.4 Ha)
- MUC - FUTURE MIXED USE CORRIDOR (A=34.8 Ha)
- RAA - FUTURE REGIONAL AMENITIES AREA (A=33.0 Ha)
- EXCEEDS FIRE FLOW REQUIREMENT
- DOES NOT MEET FIRE FLOW REQUIREMENT

MPE Engineering Ltd.

TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
AVAILABLE FIRE FLOW - WEST

SCALE: NOT TO SCALE	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 4.2



LEGEND:

MN - EX MATURE NEIGHBOURHOODS (A=244.9 Ha)	FN - FUTURE NEIGHBOURHOODS (A=287.0 Ha)
IP - EX INDUSTRIAL PARK (A=126.6 Ha)	IP - FUTURE INDUSTRIAL PARK (A=171.6 Ha)
BP - EX BUSINESS PARK (A=20.1 Ha)	BP - FUTURE BUSINESS PARK (A=120.3 Ha)
DT - EX DOWNTOWN (A=14.6 Ha)	HC - FUTURE HIGHWAY CORRIDOR (A=182.4 Ha)
HC - EX HIGHWAY CORRIDOR (A=105.4 Ha)	MUC - FUTURE MIXED USE CORRIDOR (A=34.8 Ha)
MUC - EX MIXED USE CORRIDOR (A=2.0 Ha)	RAA - FUTURE REGIONAL AMENITIES AREA (A=33.0 Ha)
RAA - EX REGIONAL AMENITIES AREA (A=53.0 Ha)	EXCEEDS FIRE FLOW REQUIREMENT
- EX DEVELOPMENT BOUNDARY (A=1,395.6 Ha)	DOES NOT MEET FIRE FLOW REQUIREMENT

MPE Engineering Ltd.

TOWN OF Westlock

TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
AVAILABLE FIRE FLOW - EAST

SCALE: NOT TO SCALE | DATE: FEBRUARY 2020 | JOB: 5454-006-00 | FIGURE: 4.3

FIRE FLOW AVAILABILITY IMPROVEMENTS

AREA / PRIORITY	FIRE FLOW REQUIREMENT	FIRE FLOW AVAILABLE (BEFORE UPGRADES, OR SCENARIO 1B)	FIRE FLOW AVAILABLE (BEFORE UPGRADES, OR SCENARIO 1B)
1	265 L/s	49 TO 144 L/s	76 TO 265 L/s
2	265 L/s	100 TO 158 L/s	161 TO 230 L/s
3	265 L/s	79 TO 246 L/s	130 TO 265 L/s
4	227 L/s	138 TO 235 L/s	155 TO 265 L/s

MPE Engineering Ltd.

TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
PROPOSED DISTRIBUTION SYSTEM UPGRADES

SCALE: NOT TO SCALE | DATE: FEBRUARY 2020 | JOB: 5454-006-00 | FIGURE: 4.4

TABLE 4.6: FIRE FLOW AVAILABILITY IMPROVEMENTS

Area/ Priority	Fire Flow Requirement	Fire Flow Available (Before Upgrades, or Scenario 1B)	Fire Flow Available (After Upgrades, or Scenario 2B)
1	230 L/s	49 to 144 L/s	76 to 265 L/s
2	230 L/s	100 to 158 L/s	161 to 230 L/s
3	230 L/s	79 to 246 L/s	130 to 265 L/s
4	230 L/s	138 to 235 L/s	155 to 265 L/s

Proposed upgrades to the distribution network required to meet the current and future demand, as well as provide improved fire flow, are below. These improved fire flows meet the Town's requirements in some areas, and do not meet the requirements in other areas.

Area 1 Proposed Upgrades

- Construct a new 300 mm diameter watermain in the commercial area in the SE with a length of 770 m.
- Construct a new 300 mm diameter watermain on 102 Street with a length of 130 m.

Area 2 Proposed Upgrades

- Upsize watermain on 105 Avenue, proceeding west to 104 Street to a 200 mm diameter watermain with a length of 800 m. The completion of these upgrades as shown in the 10-Year Capital Plan would occur over various years.

Area 3 Proposed Upgrades

- Upsize watermain along 107 Street from 97 Avenue to 99 Avenue and 101 Avenue to 103 Avenue to a 200 mm diameter watermain. Total length of pipe to be upgraded is 840 m.

Area 4 Proposed Upgrades

- Construct a new 250 mm diameter watermain 20 m in length between 93 Avenue, 110A Street, Highway 18 and 97 Avenue. This upgrade would create a connection between two watermains that currently cross each other without connecting.

4.11 Pressure Zones

The distribution system currently operates under a single pressure zone. Table 4.7 summarizes the static pressure range for the existing and future distribution areas. The pressure is considered supplied by the WRPS to the lowest and highest elevations. The future distribution area considers the extent of the current Town limits.

TABLE 4.7: STATIC PRESSURES WITHIN A SINGLE PRESSURE ZONE

	Lowest Static Pressure	Highest Static Pressure
Existing System Elevation Where Static Pressure Estimated (m)	660	638
Existing Distribution System Pressure (kPa / psi)	377/55	592/86
Future System Elevation Where Static Pressure Estimated (m)	665	633
Future distribution system (extent of current annexation area) (kPa/psi)	328/48	641/93

The highest elevations are generally toward the southeast corner of the Town limits. The lowest elevations are generally toward the northwest corner of the Town limits.

The range of existing and future static pressure is within the bounds of 328 kPa (48 psi) to 641 kPa (93 psi). This pressure range is acceptable for a single pressure zone, provided the minimum distribution pressures are maintained and pressure reducing valves are used in buildings with a supply pressure of over 550 kPa (80 psi) as per building code requirements. Periodic future hydraulic reviews are required to evaluate the system pressure as the system grows to confirm that a single pressure zone is adequate.

5.0 SANITARY SEWER COLLECTION SYSTEM ASSESSMENT

5.1 Overview

MPE reviewed the ability of the sanitary sewer collection system to meet the demands of the Town. This was completed through review of previously completed CCTV reviews from 2008, 2012 and 2018, and previous Master Plans and flow monitoring studies. MPE also reviewed GIS data provided by the Town.

5.2 Existing Collection System Material, Size and Length

The sanitary collection system is comprised of approximately 56.7 km of 150 mm, 200 mm, 250 mm, 300 mm, 350 mm, 375 mm, 400 mm, 450 mm, 525 mm, 600 mm, and 675 mm diameter sanitary sewer mains. This is shown in ***Figure 5.1 (Existing Sanitary Sewer System Pipe Diameter)***. As shown, there are some pipes where the diameter is not known. The known pipe materials are clay tile and PVC.

The known decade of installation of the sanitary sewer mains is shown in ***Figure 5.2 (Existing Sanitary Sewer System Decade Installation)***. As shown, the earliest decade of installation is the 1960s, and information provided by the Town shows that the earliest installation date is 1961. The figure also shows that there are some pipes where the date of installation is not known.

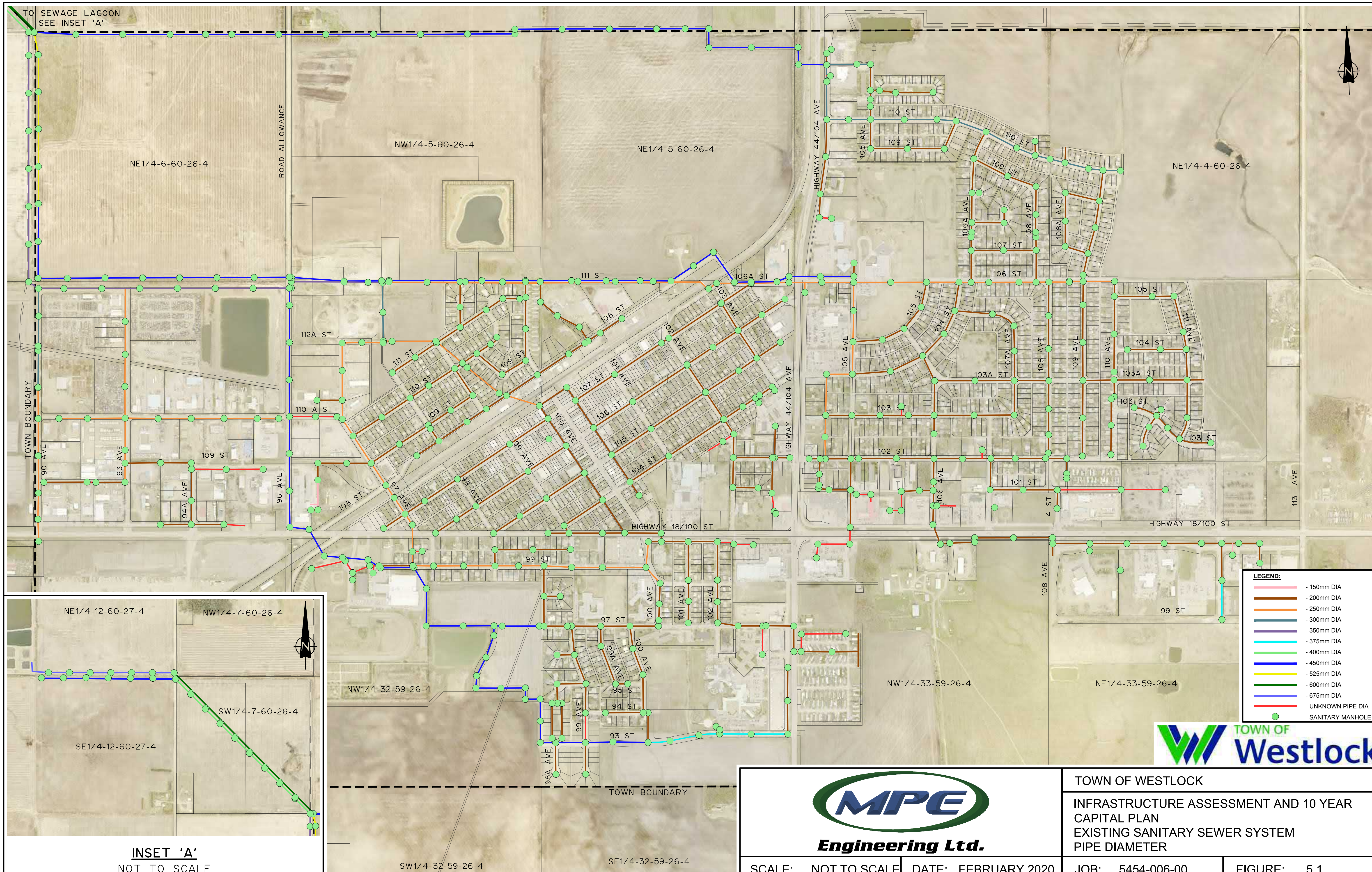
5.3 Sanitary Collection System Model – Existing System

The Town's Design Standards and the *Wastewater Collection System Master Plan, 2009 Update*, completed by ISL Engineering and Land Services in November 2009 (2009 Wastewater Master Plan Update) uses an average day sewage generation of 350 L/c/day. MPE will use this value for the residential sewage generation for the Town.

The design criteria used to assess the commercial/institutional and industrial development within the existing sanitary sewer collection system was taken from the *Wastewater Collection System Master Plan* completed by ISL in January 2005 (2005 Wastewater Master Plan). The sewage generation was based on billing records as follows:

- Commercial/Institutional – 3,250 L/ha/day
- Industrial/Regional Amenities – 2,000 L/ha/day

The Town's Design Standards as well as those for other municipalities do not provide sewage generation values for existing systems. The design standard values are developed for future developments, and are conservative because the type of development is not known. As such, MPE has assumed that the commercial/industrial generation within the existing sanitary sewer system from the 2005 Wastewater

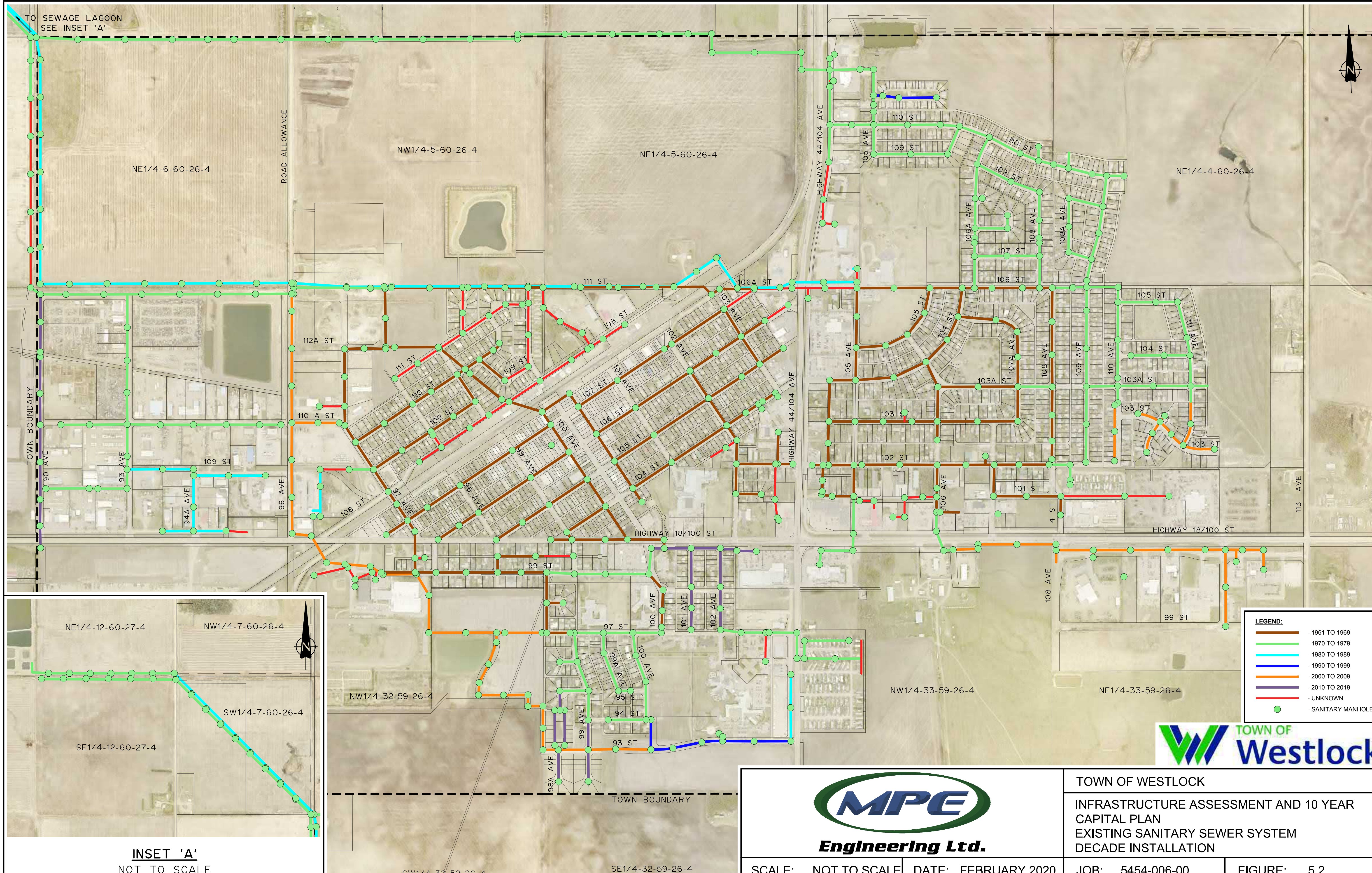


MPE Engineering Ltd.

TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
EXISTING SANITARY SEWER SYSTEM
PIPE DIAMETER

SCALE: NOT TO SCALE	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 5.1



MPE Engineering Ltd.

TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
EXISTING SANITARY SEWER SYSTEM
DECADE INSTALLATION

SCALE: NOT TO SCALE	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 5.2

Master Plan has not changed. These values are based on water billings, and are a better representation of actual sewage generation within the Town than the design standard values. MPE recommends the Town confirm these numbers when the Water Master Plan is updated.

For future development, MPE used values from the Town's Design Standards and the 2009 Wastewater Master Plan Update:

- Commercial/Institutional – 40,000 L/ha/day
- Industrial/Regional Amenities – 20,000 L/ha/day

The sewage generation used for future industrial development is consistent with those values for future industrial development from other municipalities such as the Town Whitecourt, Town of High Prairie, and Strathcona County. The sewage generation rate for future commercial development is twice the values of many other municipalities, but are consistent with the sewage generation recommended in the Alberta Environment and Parks Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems.

Table 5.1 provides a summary of Commercial and Industrial Sewage Generations Rates for other Alberta Municipalities.

TABLE 5.1: SEWAGE GENERATION RATE FOR OTHER ALBERTA MUNICIPALITIES

Municipality	Commercial/Industrial Sewage Generation
City of Fort Saskatchewan	17,280 L/ha/day
City of Lloydminster	17,280 L/ha/day
Parkland County	6,170 L/ha/day plus Inflow and Infiltration
Regional Municipality of Wood Buffalo	17,280 L/ha/day
Strathcona County	18,000 L/ha/day
Sturgeon County	6,170 L/ha/day
Town of High Prairie	18,000 L/ha/day
City of Edmonton	17,200 L/ha/day
Town of Whitecourt	17,000 L/ha/day – Commercial 22,500 L/ha/day – Industrial
Town of Morinville	22,500 L/ha/day – Commercial 16,878 L/ha/day – Industrial

Based on the above sewage generation rates, and discussions with the Town, MPE will use 18,000 L/ha/day for future commercial/institutional developments.

MPE will use an infiltration allowance of 0.20 L/s/ha and a pipe roughness of 0.013 from the Town's Design Standards. Population densities used in the model will be in accordance with the zoning established in the Town's current land use bylaw.

Using the Town's Design Standards, MPE will calculate the peak residential flow using a peak factor of 2.5 or Harmon's Peaking Factor, whichever is larger.

$$Harmon'sPeaking\ Factor = 1 + \frac{14}{(4 + \sqrt{P})}$$

Where P equals the tributary population in thousands.

Commercial, Industrial and Institutional peak flows will be calculated using the value from the Town's Design Standards of 3.0 times the average daily design flow.

The Town's Design Standards state that acceptable velocity of sewage flows will be between 0.61 m/s (minimum) and 3.0 m/s (maximum).

The maximum design pipe capacity will be flowing at 86% of full flow.

5.4 Previous Studies

5.4.1 Previous Recommendations

The 2009 Wastewater Master Plan Update lists recommendations to address existing system needs and future servicing.

The recommendations to address existing system needs are:

1. Conduct sewer flow monitoring on 200 mm diameter sanitary sewer mains at the following locations to determine current system performance during wet weather and the need for upgrades:
   a. On 107 Street east of 103 Avenue, and south on 103 Avenue from 107 Street
   b. On 102 Street between 105 Avenue and 106 Avenue
   c. Northwest from 107 Street and 100 Avenue through the CNR lands
2. Survey the sludge build-up in the wastewater treatment lagoons to determine current lagoon capacities. Compare to design capacities, and if required, remove the sludge to ensure adequate lagoon capacity.
3. Conduct further investigation into lagoon system capacities within the next few years to determine the need for upgrades.

The recommendations related to the Town's future sanitary sewer servicing concept are:

1. Construct a new 450 mm diameter sewer main along 113A Street from 96 Avenue to 90 Avenue, and construct a new 600 mm diameter sanitary sewer main on 90 Avenue from 113A Street to the northwest corner of Town.
2. Construct an extension of the 375 mm diameter sewer main from near the Hospital east across Highway 44 to the new extension of the 375 mm diameter sanitary sewer main from the hospital.
3. Construct a pump station and forcemain from the 200 mm diameter sanitary sewer main at 106 Street south of Highway 44 to the new extension of the 375 mm diameter sanitary sewer main from the hospital.
4. Construct a 250 mm diameter sanitary sewer main along the west Town boundary (90 Avenue) from Highway 18 to 113A Street.
5. Construct a network of new trunk sanitary sewer mains to feed a pump station in the extreme northeast.
6. Construct a new 750 mm diameter trunk sanitary sewer main from Highway 44 to the northwest corner of Town along the north Town boundary.
7. Construct a new 1050 mm diameter outfall line to the lagoons from the northwest corner of Town.

The *Town of Westlock Flow Monitoring Report, Model Calibration and Validation* completed by WSP in July, 2018 (Flow Monitoring Report) lists recommendations to improve the performance of the sanitary sewer system based on flow monitoring in the Town in 2016 and 2017.

The recommendations to improve the sanitary sewer system performance are:

1. Construct the new outfall line running to the lagoons northwest portion of Town to reduce the loading on the trunk sewers in the northwest portion of the Town. This will reduce surcharging in areas upstream of the upgrade.
2. Upgrade and install new sewers along 105 Avenue and 102 Street to reduce the hydraulic loading in the area and accommodate additional development in the East Business Park.
3. Conduct an inflow and infiltration identification program, particularly within older portions of the Town to identify what may be causing the high flows in wet weather. This program could include ditch walks, smoke tests and camera inspections.
4. Investigate additional options such as storage including pumping to empty storage facilities after peak flows subside.

5.4.2 Implemented Recommendations

Further to the 2009 Wastewater Master Plan Update and discussions with the Town, MPE understands that the Town has completed the following upgrades:

- The Town conducted sewer flow monitoring during 2017 and 2018 on 200 mm diameter sanitary sewer mains at the following locations to determine current system performance during wet weather and the need for upgrades:
  a. On 107 Street east of 103 Avenue.
  b. Northwest from 107 Street and 100 Avenue through CNR lands.

For recommendations related to the Town's future sanitary sewer servicing concept, MPE understands the Town has completed the following upgrades:

- The Town constructed a 300 mm and 250 mm diameter sanitary sewer main along the west Town boundary (90 Avenue) from Highway 18 to 113A Street in 2010.

With the Flow Monitoring Report being recently completed, the recommendations for sanitary sewer system improvements will be carried forward into the 10-Year Capital Plan.

5.5 Sanitary Sewer Condition

5.5.1 2008 and 2012 CCTV Inspections

MPE received closed circuit television (CCTV) inspections of sanitary sewer pipes that were completed by Cam-Trac Inspection Services of Morinville, Alberta in 2008 and 2012. MPE reviewed the CCTV images and video for the purpose of evaluating the sanitary sewer condition. Unfortunately, the CCTV inspections did not include any pipe inspection reports which means that no pipe segments were evaluated on the basis of NASSCO's Pipeline Assessment Certification Program (PACP) Standards.

Upon MPE's review of the video inspections, MPE noted that Cam Trac's inspections were either complete, or incomplete due to encrustation and/or blockages in the pipe. MPE compared the sewer inspections provided by Cam-Trac to the sanitary overall plans provided by the Town for the purpose of providing precise locations. Discrepancies in manhole labelling in year(s) 2008, and 2012, compared to the Town's 2018 overall plans were noted. MPE correlated the video inspections by referencing street addresses and/or property addresses where possible, and can conclude with confidence that four (4) areas within Town were reviewed by Cam-Trac. Upon completion of the review, MPE recommends completing a CCTV inspection of the following locations to determine if the condition of the mains that was previously reported in 2008 has changed. While the information concluded from the 2008 reports is valuable, the condition of these mains may have deteriorated in the last 10 years and should be re-investigated. Table 5.2 below shows the areas inspected by Cam-Trac in 2008 and 2012.

TABLE 5.2: 2008 AND 2012 CCTV INSPECTION AREAS

Altador Neighbourhood
105 Avenue From Altador Manor to 109 Street
109 Street From 105 Avenue to 106 Avenue
106 Avenue From 109 Street to 110 Street
110 Street From 105 Avenue to 106 Avenue
Eastglen/Belvedere Neighbourhood
103A Street From 106 Avenue to 108 Avenue
107 Avenue From 104 Street to 103 Street
West of 104 Avenue, North of 100 Street
101 Street From 102 Avenue to Back Alley of Petro Canada
Southview Neighbourhood
Cam-Trac completed CCTV inspections in the Southview neighbourhood in 2008, however, since the completion of the CCTV reports, sanitary mains that were reviewed have either been upgraded or abandoned in the Southview neighbourhood in close proximity to 97 Street and 102 Avenue. MPE was unable to determine the exact location of the pipe segments in Southview due to manhole labelling discrepancies (noted above).

The **Structural and O&M defects** present in the sanitary pipe segments listed above are consistent with the following:

<u>**Structural Defects**</u>

- Longitudinal Cracking
- Sagging of Pipes (Camera under water during inspection)
- Spiral cracking at pipe joint
- Joint Displacements

<u>**Operation and Maintenance (O&M)**</u>

- Presence of roots
- Encrustation build up on pipe walls
- Grease and debris

MPE recommends re-investigating the sanitary pipes in the locations listed above to accurately determine the condition of the sewers. MPE cannot, due to age and discrepancies in manhole labelling, accurately determine the condition of the pipes in these areas. MPE recommends ensuring all future reporting conforms to current NASSCO PACP coding conventions and all software used by the Contractor be PACP compliant. MPE also recommends that all future CCTV inspections adhere to the manhole labelling used on the Town's overall plans.

5.5.2 2016 CCTV Inspection

As part of the preliminary design work for the rehabilitation of the road surface of 104 Street between 110 and 111 Avenues, the Town had Sewer Infrastructure Investigation 360 inspect the sanitary sewer on that block by CCTV to determine the condition. The results of the inspection are shown in ***Table 5.3 on the following page***, and Figure 5.4.

MPE recommends that this section of sanitary sewer main between MH0341 and MH0342 be part of the second priority of sanitary sewer mains for rehabilitation. This rehabilitation is recommended to occur after the Town has rehabilitated the sanitary sewer mains in poor structural condition. MPE also recommends that the section of sanitary sewer main between MH0342 and MH0129 have its' condition monitored as part of the Town's ongoing sanitary sewer operation and maintenance.

5.5.3 2018 CCTV Inspection

5.5.3.1 CCTV Inspection

Cam-Trac Inspection Services Ltd. (Cam-Trac) was selected to provide flushing and camera inspection services to determine the condition of a sample of sanitary sewer mains within the Town. Cam-Trac reviewed and rated each section using the NASSCO PACP system. The location of these mains was determined by the Town, and is shown as ***Figure 5.3 (2018 CCTV Program)***. Cam-Trac flushed all the mains selected, however, some mains were not inspected by CCTV due to high flows and debris. This is also shown in Figure 5.3. Cam-Trac flushed and inspected these mains between August 1 and August 8, 2018.

Cam-Trac reported that the section between MH0045 and MH0300 had high water and heavy debris during flushing. Additional flushing and cleaning was required along this line to complete a proper inspection, but this work was not done due to budget constraints. MPE recommends that the Town complete additional flushing and cleaning along this section so that it can be inspected.

Cam-Trac also reported high flows in the section between MH0193 and MH0067. The camera was under water for 60% to 70% of the inspection of this section during flushing and cleaning. This section was not inspected due to high flows. The high flows correspond to flows being above the capacity of the sanitary sewer main in the Flow Monitoring Report. This confirms the recommendation that the Town upgrade and install new sewers along 105 Avenue and 102 Street to reduce the hydraulic loading in the area and accommodate additional development in the East Business Park.

Town of Westlock
Infrastructure Assessment and 10 Year Capital Plan
Table 5.3 - Sanitary Sewer Main Condition Ratings

Legend:		
	1	Good
	2	Good
	3	Fair
	4	Fair to Poor
	5	Poor

								Structural Condition				O&M Condition				
No.	Date	From	To	Street	Material	Complete	Length (m)	Total Score	Avg.	Peak	Condition Rating	Total Score	Avg.	Peak	Condition Rating	Notes
1	09-Nov-16	MH0341	MH0342	104 Street	Clay Tile	Yes	107.1	14	0.131	4	4	4	0.037	2	2	Multiple Multiple Fractures, Multiple Cracks, Multiple Encrustations
2	09-Nov-16	MH0342	MH0129	104 Street	Clay Tile	Yes	53.1	3	0.056	2	2	9	0.169	3	3	Longitudinal Crack, Circumfrential Crack, Multiple Encrustations
01-33	01-Aug-18	MH0166	MH0198	107A Avenue	Clay Tile	Yes	94.7	8	0.084	2	2	8	0.084	2	2	Multiple Longitudinal Cracks, Multiple Encrustations, Grease
01-38	02-Aug-18	MH0189	MH0346	Alley W. 105 Ave.	Clay Tile	Yes	69.1	2	0.029	2	2	69	0.999	4	4	Longitudinal Crack, Camera Underwater Multiple Times, roots, grease, encrustations
02-33	01-Aug-18	MH0198	MH0343	107A Avenue	Clay Tile	No	87.3	18	0.206	3	3	18	0.206	2	2	Sprial fracture, Sprial Crack, Multiple Longitudinal cracks, service prevents complete inspection
02-33	01-Aug-18	MH0343	MH0198	107A Avenue	Clay Tile	No	0.6	0	0.000	0	1	0	0.000	0	1	Reached service that cannot be passed to complete reverse run
02-38	02-Aug-18	MH0346	MH0193	Alley W. 105 Ave.	Clay Tile	Yes	203.2	13	0.064	4	4	304	1.496	4	4	Multiple Fractures, Camera Underwater Multiple times., Longitudinal Cracks
03-33	01-Aug-18	MH0343	MH0197	107A Avenue	Clay Tile	No	41.2	18	0.437	4	4	12	0.291	2	2	Multiple Fractures, Multiple Longitudinal Cracks, Protruding Service Prevents Complete Inspection
03-33	01-Aug-18	MH0197	MH0343	107A Avenue	Clay Tile	No	32.3	20	0.619	4	4	6	0.186	2	2	Multiple Fractures, Multiple Longitudinal Cracks, reverse run complete
03-38	03-Aug-18	MH0309	MH0582	Alley W. 104 Ave.	Clay Tile	No	74.4	2	0.027	2	2	75	1.008	5	5	Longitudinal Crack, Camera Underwater, Root Ball and Grease prevent complete inspection
03-38	03-Aug-18	MH0582	MH0309	Alley W. 104 Ave.	Clay Tile	No	35.6	6	0.169	4	4	68	1.910	4	4	Multiple Fractures, Root mass reached to complete reverse run
04-33	01-Aug-18	MH0197	MH0195	107A Avenue	Clay Tile	Yes	120.8	42	0.348	4	4	4	0.033	2	2	Multiple Fractures, Multiple Cracks, Multiple Longitudinal Cracks
04-38	03-Aug-18	MH0016	MH0582	Alley W. 104 Ave.	Clay Tile	Yes	49.4	4	0.081	4	4	85	1.721	5	5	Longitudinal Hinge Fracture, Multiple Encrustations from 5% to 40% of pipe diameter, Infiltration - Dripper
05-33	01-Aug-18	MH0177	MH0176	106 Avenue	Clay Tile	No	1.5	0	0.000	0	1	0	0.000	0	1	Camera was underwater at start, would not advance
05-33	01-Aug-18	MH0177	MH0176	106 Avenue	Clay Tile	No	59.4	6	0.101	4	4	4	0.067	2	2	Multiple Fractures, Circumfrential Fracture, reverse run complete
05-38	03-Aug-18	MH0683	MH0595	104 Street	Clay Tile	No	60	4	0.067	4	4	19	0.317	4	4	Multiple fractures, Camera Underwater, Encrustation prevents complete survey
05-38	03-Aug-18	MH0595	MH0683	104 Street	Clay Tile	No	15.2	1	0.066	1	1	6	0.395	4	4	Medium Joint Displacement, Reached encrustation to complete reverse run
06-33	01-Aug-18	MH0176	MH0024	106 Avenue	Clay Tile	Yes	48.2	9	0.187	4	4	9	0.187	3	3	Multiple Fractures, multiple cracks, multiple encrustations
06-38	03-Aug-18	MH0314	MH0685	104 Street	Clay Tile	Yes	49	13	0.265	4	4	16	0.327	4	4	Multiple Fractures, multiple Cracks, Multiple Longitudinal Cracks, Camera Underwater
07-33	01-Aug-18	MH0024	MH0175	106 Avenue	Clay Tile	Yes	28.3	4	0.141	4	4	2	0.071	2	2	Multiple Fractures, Encrustation
07-38	03-Aug-18	MH0685	MH0684	104 Street	Clay Tile	Yes	15.3	4	0.261	4	4	4	0.261	4	4	Multiple Fractures, Camera Underwater
08-33	01-Aug-18	MH0175	MH0174	106 Avenue	Clay Tile	Yes	74	5	0.068	3	3	11	0.149	4	4	Multiple Cracks, Longitudinal Cracks, Camera Underwater
08-38	03-Aug-18	MH0684	MH0683	104 Street	Clay Tile	Yes	49.3	6	0.122	4	4	26	0.527	4	4	Multiple Fractures, Circumfrential Fracture, Multiple Camera Underwater
09-33	02-Aug-18	MH0174	MH0173	102 Street	Clay Tile	Yes	121.3	3	0.025	2	2	48	0.396	2	2	Circumfrentail crack, spiral crack, multiple encrustations
09-38	07-Aug-18	MH0542	MH0543	Alley N. 107 St.	Clay Tile	No	2.5	0	0.000	0	1	4	1.600	4	4	Camera underwater at start of inspection, cannot continue, line requires extra flushing
10-33	02-Aug-18	MH0173	MH0172	102 Street	Clay Tile	No	75.7	0	0.000	0	1	24	0.317	3	3	Multiple encrustations, multiple grease, cannot pass auger sticking out of service
10-33	02-Aug-18	MH0172	MH0173	102 Street	Clay Tile	No	44.9	0	0.000	0	1	34	0.757	4	4	Camera Underwater, multiple grease, reverse run complete
10-38	07-Aug-18	MH0322	MH0786	106 Street	Clay Tile	No	52.1	43	0.825	4	4	30	0.576	4	4	Multiple fractures, longitudinal fractures, camera underwater, protruding service prevents complete inspection
10-38	08-Aug-18	MH0786	MH0322	106 Street	Clay Tile	No	0.1	0	0.000	0	1	0	0.000	0	1	Camera cannot enter line due to offset benching. Also a void between benching and main. No reverse run.
11-33	02-Aug-18	MH0172	MH0192	102 Street	Clay Tile	No	19.8	0	0.000	0	1	4	0.202	4	4	Camera underwater, protruding service prevents complete inspection.
11-33	02-Aug-18	MH0192	MH0172	102 Street	Clay Tile	No	3.5	4	1.143	2	2	0	0.000	0	1	Multiple Spiral Cracks
11-38	07-Aug-18	MH0786	MH0321	106 Street	Clay Tile	No	0.1	5	50.000	5	5	0	0.000	0	1	Broken pipe, void visible, prevents complete survey
11-38	08-Aug-18	MH0321	MH0786	106 Street	Clay Tile	No	0.1	0	0.000	0	1	0	0.000	0	1	No benching in MH0321, camera cannot enter line. No reverse run is possible
12-33	02-Aug-18	MH0192	MH0189	102 Street	Clay Tile	Yes	81.6	0	0.000	0	1	12	0.147	4	4	Camera Underwater Multiple Times
12-38	08-Aug-18	MH0581	MH0310	Alley W. 104 Ave. N. 102 St.	Clay Tile	No	2.3	5	2.174	4	4	2	0.870	2	2	Multiple fractures, medium joint displacement, protruding service prevents complete inspection
12-38	08-Aug-18	MH0310	MH0581	Alley W. 104 Ave. N. 102 St.	Clay Tile	No	70.4	3	0.043	3	3	12	0.170	4	4	Longitudinal fracture, multiple camera underwater, transition from PVC to VCP prevents complete inspection.
13-33	03-Aug-18	MH0018	MH0192	101 St & 105 Ave	Clay Tile	No	59.8	0	0.000	0	1	47	0.786	4	4	Multiple grease deposits, one of which prevents complete survey.
13-33	03-Aug-18	MH0192	MH0018	101 St & 105 Ave	Clay Tile	No	2.5	0	0.000	0	1	4	1.600	4	4	Camera Underwater, unseen obstruction prevents complete survey.
14-33	03-Aug-18	MH0191	MH0190	101 St & 105 Ave	PVC	Yes	34.3	0	0.000	0	1	4	0.117	2	2	Grease, encrustation
15-33	03-Aug-18	MH0190	MH0189	101 St & 105 Ave	PVC	Yes	16.4	0	0.000	0	1	2	0.122	2	2	Grease
16-33	03-Aug-18	MH0599	MH0681	106 Street	Clay Tile	No	74.3	3	0.040	3	3	22	0.296	2	2	Multiple cracks, multiple encrustation, multiple gravel, encrustation prevents complete survey
16-33	03-Aug-18	MH0681	MH0599	106 Street	Clay Tile	No	15.5	0	0.000	0	1	2	0.129	2	2	Encrustation, reverse run complete
17-33	03-Aug-18	MH0681	MH0316	106 Street	Clay Tile	No	57.3	0	0.000	0	1	0	0.000	0	1	Protruding service prevents complete survey
17-33	03-Aug-18	MH0316	MH0681	106 Street	Clay Tile	No	30	0	0.000	0	1	11	0.367	3	3	Multiple roots, protruding service prevents complete survey
18-33	03-Aug-18	MH0317	MH0680	106 Street	Clay Tile	Yes	100.3	6	0.060	2	2	4	0.040	2	2	Multiple longitudinal cracks, spiral crack, multiple encrustations
19-33	03-Aug-18	MH0680	MH0316	106 Street	Clay Tile	No	50.2	3	0.060	2	2	6	0.120	2	2	Circumfrential crack, spiral crack, multiple grease, cannot pass under water structure
19-33	07-Aug-18	MH0316	MH0680	106 Street	Clay Tile	No	48.7	2	0.041	2	2	22	0.452	4	4	Longitudinal crack, multiple camera underwater, multiple grease, reverse run complete
20-33	07-Aug-18	MH0288	MH0291	104 Street	Clay Tile	No	3.7	5	1.351	5	5	0	0.000	0	1	Broken pipe, void visible, survey abandoned, camera cannot proceed
20-33	08-Aug-18	MH0291	MH0288	99 Ave. - 105 St.	Clay Tile	No	27.2	6	0.221	3	3	11	0.404	2	2	Multiple cracks, multiple grease, protruding service prevents complete inspeciton
21-33	07-Aug-18	MH0588	MH0589	99 Street	Clay Tile	Yes	122.9	18	0.146	4	4	21	0.171	4	4	Multiple fractures, multiple circumfrential fractures, longitudinal crack, multiple grease
22-33	07-Aug-18	MH0589	MH0590	99 Street	Clay Tile	No	40.9	4	0.098	2	2	0	0.000	0	1	Circumfrential fracture, longitudinal crack, suvery abandoned at 100% sag
22-33	08-Aug-18	MH0590	MH0589	99 Street	Clay Tile	No	41.4	0	0.000	0	1	2	0.048	2	2	Grease, Protruding service prevents complete inspection, reverse run abandoned
23-33	08-Aug-18	MH0586	MH0318	99 Ave. - 105 St.	PVC	Yes	102.5	0	0.000	0	1	0	0.000	0	1	
24-33	08-Aug-18	MH0318	MH0318A	99 Ave. - 105 St.	PVC	Yes	67	0	0.000	0	1	0	0.000	0	1	
25-33	08-Aug-18	MH0318A	MH0292	99 Ave. - 105 St.	PVC	Yes	79.8	0	0.000	0	1	16	0.201	2	2	Multiple grease, encrustation
26-33	08-Aug-18	MH0321	MH0320	106 Street	Clay Tile	No	28.5	0	0.000	0	1	2	0.070	1	1	Multiple roots, protruding service prevents complete survey
26-33	08-Aug-18	MH0320	MH0321	106 Street	Clay Tile	No	0.1	0	0.000	0	1	0	0.000	0	1	No manhole benching, camera under water in mud.
27-33	08-Aug-18	MH0320	MH0293	106 Street	Clay Tile	No	64.9	0	0.000	0	1	2	0.031	2	2	Encrustation, protruding service prevents complete inspection
27-33	08-Aug-18	MH0293	MH0320	106 Street	Clay Tile	No	82	0	0.000	0	1	2	0.024	2	2	Grease, reverse run complete

TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
2018 CCTV PROGRAM

MPE Engineering Ltd.

SCALE:	NOT TO SCALE	DATE:	FEBRUARY 2020	JOB:	5454-006-00	FIGURE:	5.3

5.5.3.2 CCTV Summary

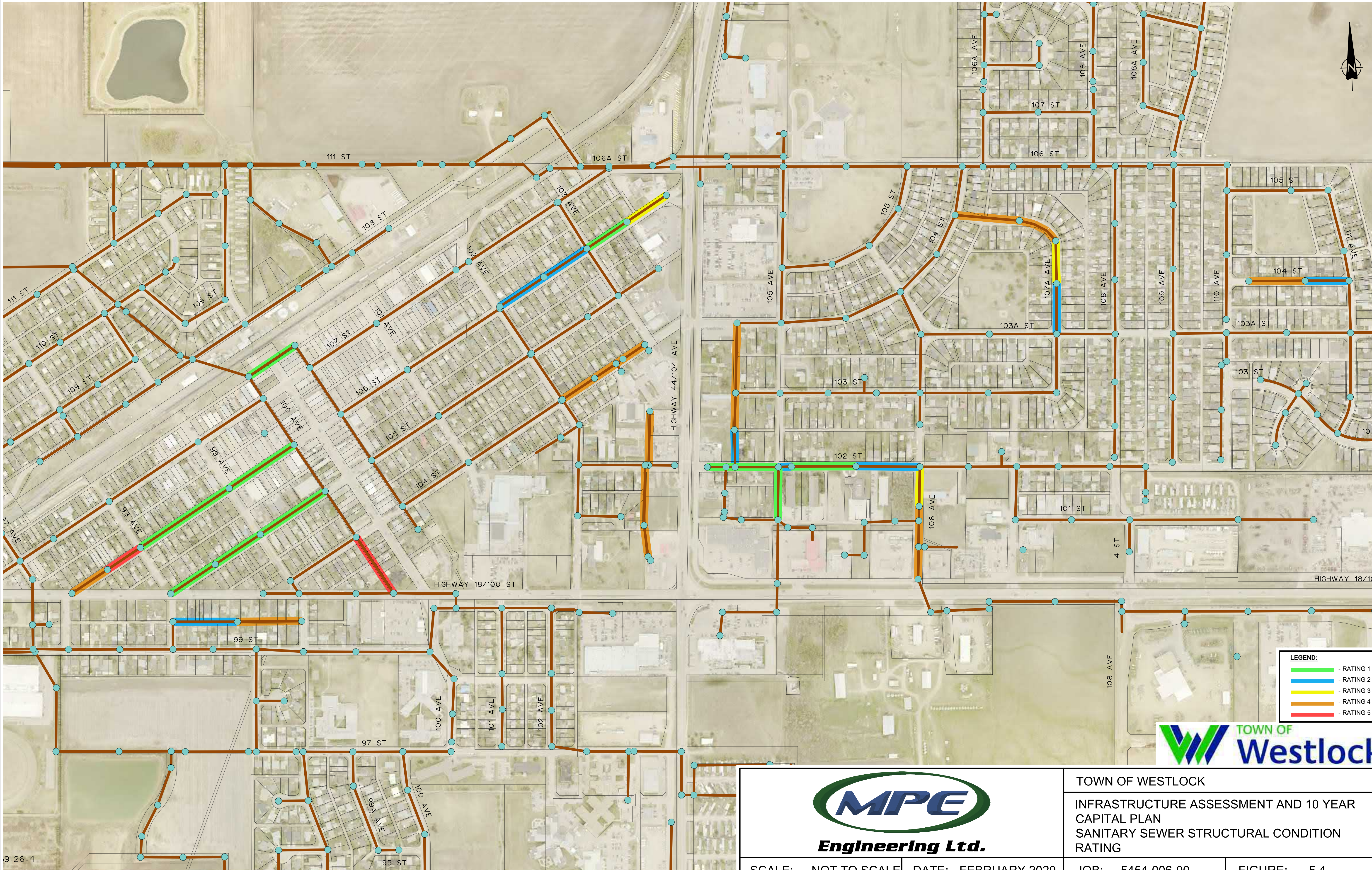
Cam-Trac reviewed and rated each section using the NASSCO PACP System. Each observed defect is assigned a score, and these scores are used to provide structural and O&M condition ratings for each sanitary sewer section. The structural condition of a sanitary sewer main is determined by observed defects such as cracks, fractures, pipe deformation, offset joints, and broken pipe. The O&M condition is determined by observed items such as grease, encrustations, debris, roots, protruding services, and the camera being underwater.

A summary of the scores and condition ratings for each sanitary sewer section are provided in Table 5.3. The structural condition ratings for each section of sanitary sewer main in Table 5.3 are shown in ***Figure 5.4 (Sanitary Sewer Structural Condition Rating)***. A rating of 5 indicates that the sanitary sewer section is in poor structural or O&M condition. A rating of 1 means that the sanitary sewer is in good structural or O&M condition.

While sags are not assigned a score in the PACP program, other rating systems assign scores for these defects. Severe sags occur when the camera is underwater, moderate sags occur when the camera is partially submerged, and light sags occur when the water level is below the camera level. A ***Summary of the Sags Observed*** is provided in Table 5.4 and are shown in ***Figure 5.5***. MPE will consider sections of sanitary sewer main with severe sags to be in poor structural condition, with a rating of 5. MPE will also consider sections of sanitary sewer main with moderate sags to be in moderate structural condition, with a rating of 3.

TABLE 5.4: SAG SUMMARY

From	To	Condition Rating	Sags
MH0309	MH0582	4	Light Sag, Severe Sag
MH0683	MH0595	4	Severe Sag, 3 Moderate Sags
MH0176	MH0024	4	Multiple Light Sags
MH0314	MH0685	4	Severe Sag
MH0024	MH0175	4	Moderate Sag
MH0685	MH0684	4	Severe Sag
MH0175	MH0174	3	Severe Sag
MH0684	MH0683	4	Multiple Severe Sags
MH0174	MH0173	2	Multiple Light Sags
MH0173	MH0172	1	Multiple Light Sags, Severe Sag
MH0322	MH0786	4	Severe Sag



TOWN OF WESTLOCK	
INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN SANITARY SEWER STRUCTURAL CONDITION RATING	

SCALE:	NOT TO SCALE	DATE:	FEBRUARY 2020	JOB:	5454-006-00	FIGURE:	5.4

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TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
SANITARY SEWER SAG SUMMARY

SCALE:	NOT TO SCALE	DATE:	FEBRUARY 2020	JOB:	5454-006-00	FIGURE:	5.5

MH0192	MH0190	1	Multiple Severe Sags
MH0310	MH0581	4	Multiple Severe Sags
MH0681	MH0316	1	Multiple Light Sags
MH0317	MH0680	2	Multiple Light Sags
MH0680	MH0316	2	Severe Sag, Light Sag
MH0588	MH0589	4	Severe Sag, Light Sag
MH0589	MH0590	2	Severe Sag, Multiple Light Sags
MH0320	MH0293	1	Multiple Light Sags

Table 5.3 and Figure 5.3 show the majority of sanitary sewer mains inspected by CCTV have a structural condition rating of 4. There are two sanitary sewer mains with structural condition ratings of 5:

1. MH0786 to MH0321 on 106 Street
2. MH0291 to MH0290 in the alley west of 100 Avenue, between 104 Street and 100 Street

MPE recommends that sanitary sewer mains receiving a structural condition rating of 5 be the first priority for structural condition rehabilitation. MPE understands the Town has rehabilitated the sanitary sewer main between MH0291 and MH0290. MPE also recommends that sanitary sewer mains receiving a structural condition rating of 4 be the second priority for structural condition rehabilitation. As part of the Town's ongoing operation and maintenance program, MPE recommends that sanitary sewer mains receiving a structural condition rating of 3 or less have their condition monitored.

The section of sanitary sewer main between MH0321 and MH0320 on 106 Street has a structural condition rating of 1. Less than half the main was inspected due to a protruding service and no benching in MH0321. MPE recommends that part of the rehabilitation of the sanitary sewer main between MH0786 and MH0321 include the installation of benching in MH0321. Once the benching is installed, it is recommended that the sanitary sewer main between MH0321 and MH0320 be inspected again to determine its' condition.

Table 5.3 also shows the O&M condition ratings for each section of sanitary sewer main inspected. O&M condition items can be remedied through proper operation and maintenance of the sanitary sewer system such as flushing. MPE recommends that the Town develop a flushing program to remedy the sections of sanitary sewer main with O&M condition ratings of 5 and 4. It should be noted that after the inspection, Cam-Trac performed root cutting between MH0582 and MH0309 and grease cutting between MH0018 and MH0192.

5.5.3.3 Manhole Condition

Manholes were not inspected as part of the sanitary sewer CCTV inspection work. MPE recommends the condition of the manholes be evaluated prior to any sanitary sewer rehabilitation. If the manholes are in poor condition, it is recommended that the Town repair or replace them as part of the sanitary sewer rehabilitation.

5.5.3.4 Future Assessment

The CCTV work done by Cam-Trac totals 2.8 km of sanitary sewer main inspected and rated for condition. With a total of approximately 56.7 km of sanitary sewer main, MPE recommends that the Town develop a program as part of the ongoing operation and maintenance to inspect part of the sanitary sewer system every year and assess its' condition. This will help the Town in identifying areas requiring replacement and rehabilitation, and the development of future infrastructure upgrade programs.

MPE also recommends that the Town inspect the sanitary sewer manholes as part of this future assessment to determine their condition.

5.5.3.4 Grease Zones

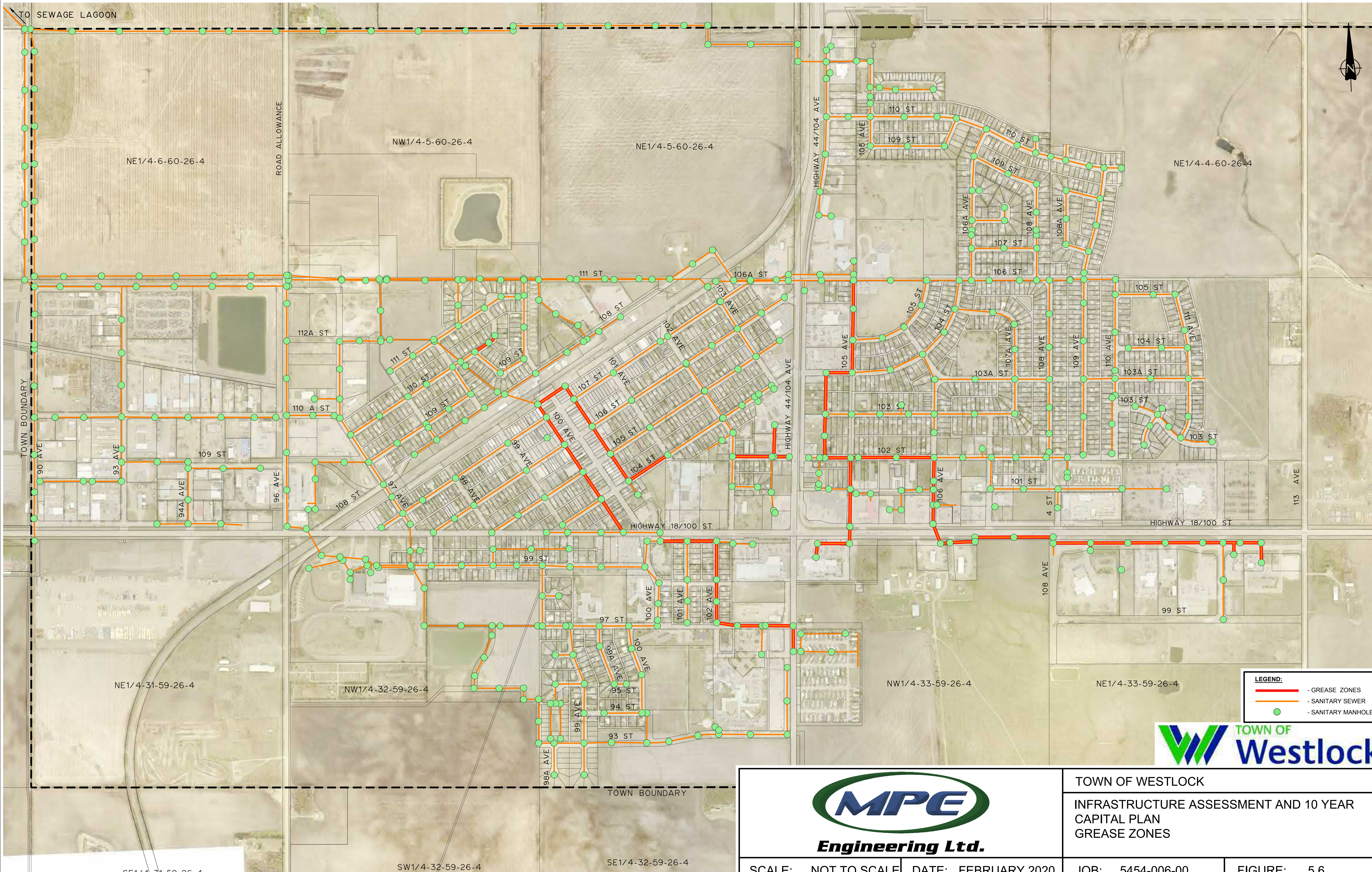
Figure 5.6 (Grease Zones) shows grease zones identified by Town staff. These zones correspond to two areas that were not inspected due to high flow and debris. The zones are in proximity to commercial areas. The trunk mains moving sewage north and west towards the wastewater treatment plant are considered grease zones. To remedy the grease zones, MPE recommends the Town inspect the grease traps in commercial businesses to determine if they are being cleaned by their owners and are working properly.

5.6 Sanitary Lagoon Storage Requirements

The sanitary lagoons are located northwest of the Town, and consist of the following:

- Four anaerobic cells
- Two facultative (anaerobic)cells
- Three storage cells

Using the record drawings available to MPE, the current lagoon system capacities are shown in the table below.



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TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
GREASE ZONES

SCALE: NOT TO SCALE	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 5.6

TABLE 5.5: CURRENT LAGOON SYSTEM CAPACITIES

Anaerobic Cell	Depth (m)	Volume (m^3)	Total Volume (m^3)
1	3.5	4,550	31,100
2	3.5	4,550	
3	3.0	11,000	
4	3.0	11,000	
Facultative Cell	**Depth (m)**	**Volume (m^3)**	**Total Volume (m^3)**
1	1.5	63,700	234,380
2	1.5	170,680	
Storage Cell	**Depth (m)**	**Volume (m^3)**	**Total Volume (m^3)**
1	Varies from 1.8 to 2.47	430,000	1,393,600
2	2.5	333,800	
3	2.5	629,800	

It should be noted that these capacities are based on the design drawings for the lagoons available to MPE, and do not consider sludge accumulation or any other revisions to the system where engineering drawings are not available. These capacities only exist when the system is fully maintained (e.g. solids removed from the lagoons).

Lagoon storage requirements for the Town are dictated by Alberta Environment and Parks based on a standardized set of guidelines for municipalities. Lagoon guidelines are not dependent on effluent water quality, but are instead built around designated retention times for treatment processes occurring in each cell of the lagoon. Based on the average daily flow, the sanitary lagoon storage requirements are presented in Table 5.6. All requirements are based on Alberta Environment and Parks' guidelines.

5.6.1 Anaerobic Cells

In anaerobic cells, much of the solid material present in the waste stream settles out, and microbial action from bacteria presented in the waste stream breaks down organic compounds. The breakdown of organic compounds in an anaerobic cell is a three-stage process that can be susceptible to influent that is acidic or which has highly variable amounts of Biological Oxygen Demand (BOD).

The reduction of BOD present in the waste stream is a vital function within a wastewater lagoon that occurs in the highest intensity in anaerobic cells. Alberta Environment and Parks specifies a contact time within each anaerobic cell of 48 hours, and that each cell maintains a depth of 3 m. The depth of each cell is important to mitigate the amount of oxygen that enters the lagoon through the water surface. Due to the high solids loading rate, anaerobic cells require more frequent maintenance than other cells within the sanitary lagoon system. Each cell is intended to operate independently of the other cell for a period of 48 hours to allow for additional repair and maintenance without negatively impacting the quality or operation of other portions of the lagoon.

5.6.2 Facultative Cells

Despite the reduction of BOD in the anaerobic cells, the constant influx of fresh sewage prevents effluent from reaching the levels required for release into the environment. To reach these levels, two further stages of treatment are required. The first of these stages takes place in the facultative cell. In the facultative cell, both anaerobic and aerobic bacteria act on the sludge in different layers. The Alberta Environment and Parks Standards and Guidelines for Facultative Cells dictate a maximum depth of 1.5 m for these cells, which increases the volume of oxygen that can be absorbed through the water's surface to support the growth of aerobic bacteria.

Alberta Environment and Parks dictates a retention time in the facultative cell of a lagoon system of 60 days. This long retention time allows most of the remaining solids to settle out and significantly reduces the concentration of BOD in the waste stream.

5.6.3 Storage Cell

The final cell in the lagoon is sized to store 12 months of flow at a given time as per Alberta Environment and Parks Standards. This size allows for final finishing of the wastewater effluent to further reduce the environmental loading caused during annual releases. Alberta Environment and Parks identifies that the maximum depth of the storage pond should be 3 m. The anaerobic and facultative cells have treated the water entering the storage cells to a high degree. Because of this, sedimentation of the storage cells is not a concern unless qualitative observations made after lagoon discharge identify sedimentation as an issue. The Town has not identified any such issues at this time.

TABLE 5.6: SEWAGE LAGOON CAPACITY ASSESSMENT

Component	Units	2020	2030
Population	Persons	5,329	5,945
Average Day Flow	m^3/day	1,865	2,081
Anaerobic Cells			
Number of Cells Required	each	2	2
Number of Cells Existing	each	4	4
Retention Required, Each Cell	days	2	2
Retention Available	days	2.4	2.2
Additional Retention Required	**days**	**None**	**None**
Volume Required, Each Cell	m^3	3,730	4,162
Volume Available (Total)	m^3	31,100	31,100
Additional Volume Required	**m^3**	**None**	**None**
Facultative Cells			
Retention Required	days	60	60
Retention Available	days	126	113
Additional Retention Required	**days**	**None**	**None**
Volume Required	m^3	111,900	124,860
Volume Available	m^3	234,380	234,380
Additional Volume Required	**m^3**	**None**	**None**
Storage Cells			
Retention Required	days	365	365
Retention Available	days	747	670
Additional Retention Required	**days**	**None**	**None**
Volume Required	m^3	680,725	759,565
Volume Available	m^3	1,393,600	1,393,600
Additional Volume Required	**m^3**	**None**	**None**

Based on Alberta Environment and Parks' Wastewater Lagoon Standards and Guidelines, the Town does not require any additional anaerobic cells, facultative cells, or storage cells leading into the year 2030. If the average daily flow increases dramatically, MPE recommends reassessing the sewage lagoon capacities.

5.7 Sanitary Lagoon Maintenance

The 2009 Wastewater Master Plan Update recommended the Town survey the sludge build-up in the lagoons to determine the current lagoon capacities. It is MPE's understanding that the Town had not completed this work. MPE recommends that the Town survey the sludge build-up in the lagoons to determine the current capacities.

5.8 Sanitary Sewer Model

5.8.1 Computer Model

The computer model of the Town's Sanitary Sewer System in an XPSWMM model. XPSWMM is a Windows based stormwater and sanitary management modelling software package. The model was created in 2004, and was updated in July 2018 as part of the Flow Monitoring Report. This model update included a calibration using flow and rainfall data collected in 2016 and 2017 for the Flow Monitoring project.

As part of the Infrastructure Assessment, MPE has reviewed and verified the July 2018 update of the model.

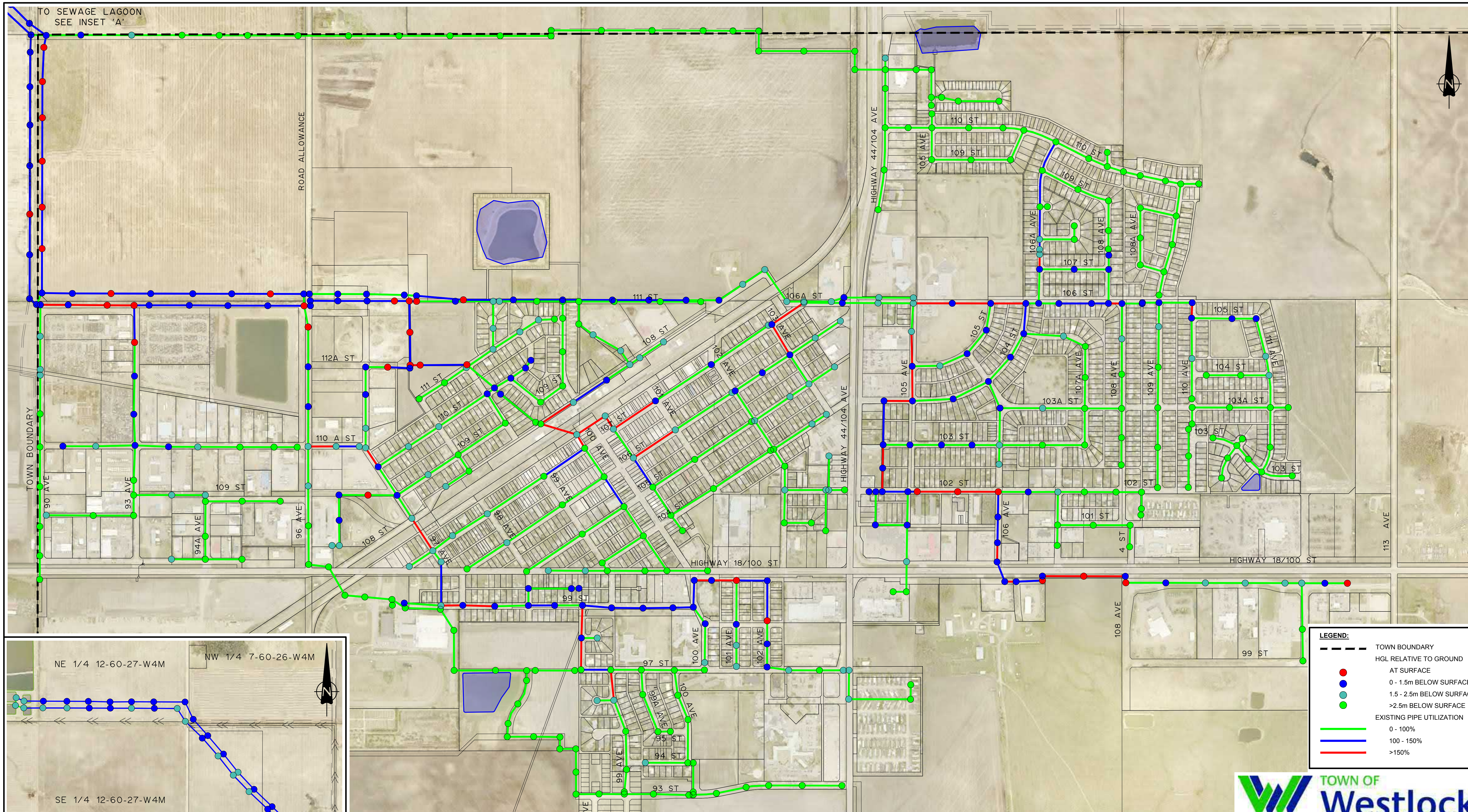
5.8.2 Existing System Assessment

5.8.2.1 Existing System Hydraulics

MPE tested the existing system hydraulics using a historical storm event from July 7-8, 2004. This event was used in the flow monitoring report, as it represents an approximately 1:5-year return period storm. This event is also similar in size to the events used in the calibration process for the Flow Monitoring Report.

The performance of the existing system is shown in ***Figure 5.7 (Existing System Performance)***. The model review and verification found issues with system capacity in the downtown, East Business Park, and the industrial area.

As shown in Figure 5.7, there are situations where the existing pipe utilization is 0 – 100%, but the hydraulic grade line is shown to be at the surface of 0 to 1.5 m below the surface. These are situations where a pipe is backed up due to high levels on infiltration during the peak of a storm event. This infiltration causes upstream manholes at lower elevations to surcharge due to temporary reverse flow. In these cases, the pipe downstream of the upstream manholes that is surcharging is not running beyond 100% flow capacity, it is just backing up.



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TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
EXISTING SYSTEM PERFORMANCE

SCALE:	1:10000	DATE:	FEBRUARY 2020	JOB:	5454-006-00	FIGURE:	5.7

5.8.2.2 Existing System Upgrades

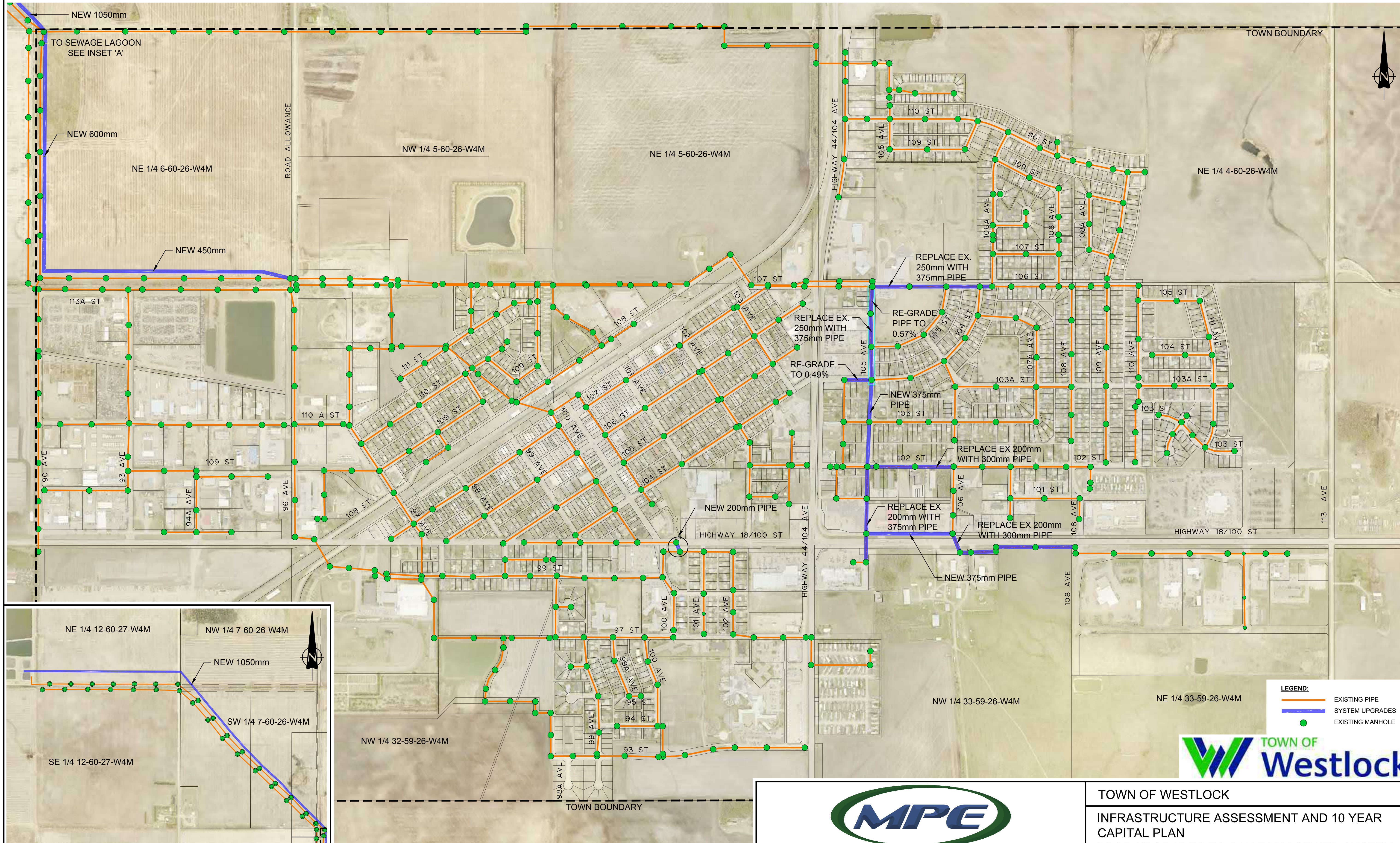
The model review and verification found that downtown Westlock, the East Business Park, and the industrial area have capacity issues. Inflow and infiltration in the industrial area is an issue where the large storm event of August 2016 caused significant inflow and infiltration into the sanitary sewer system. The Flow Monitoring Report showed that other storm events resulted in significant inflow and infiltration contributions in this area.

To solve these issues, MPE developed two scenarios:

- ***Scenario 1*** includes construction of a proposed outfall line from 96 Avenue and 113A Street to the sewage lagoons as shown in ***Figure 5.8***. The Town can construct this trunk main in phases, with the 450 mm and 600 mm diameter pipe installed first, and the remaining 1,050 mm diameter pipe installed afterwards.
- ***Scenario 2*** includes construction of a proposed 450 mm diameter main across the NE ¼ Sec. 5-60-26 W4M, and the same 1,050 mm diameter outfall line shown in Scenario 1. This is provided in ***Figure 5.9***. The Town can also construct this scenario in phases, with the 450 mm pipe installed first, and the remaining 1,050 mm diameter pipe installed afterwards.

With either scenario, the existing system still has capacity issues in the eastern part of the Town, north of Highway 18 on 102 Street and 105 Avenue. To resolve these issues, MPE recommends upgrades to the sewer from 106 Street and 105 Avenue to 102 Street and 106 Avenue. This upgrade includes the following:

- Upgrade the existing sanitary sewer main on 106 Street between 105 Avenue to 106A Avenue from 250 mm to 375 mm.
- Upgrade the existing sanitary sewer main on 105 Avenue between 106 Street and 104 Street from 250 mm diameter to 375 mm diameter.
- Construct a new 375 mm diameter sanitary sewer main along 105 Avenue from 102 Street to 104 Street.
- Re-grade the new sanitary sewer mains along 105 Avenue from 106 Street to 102 Street to 0.57%.
- Re-grade the existing sanitary sewer main on 104 Street from 105 Avenue west to 0.49%.
- Upgrade the sanitary sewer main on 102 Street from 105 Avenue to 106 Avenue from 200 mm diameter to 300 mm diameter.
- Upgrade the sanitary sewer main on 105 Avenue between 102 Street and Highway 18 from 200 mm diameter to 375 mm diameter.
- Install a new 375 mm diameter sanitary sewer main on Highway 18 between 105 Avenue and 106 Avenue.
- Upgrade the sanitary sewer main on Highway 18 between 106 Avenue and 108 Avenue from 200 mm diameter to 300 mm diameter.



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NOT TO SCALE

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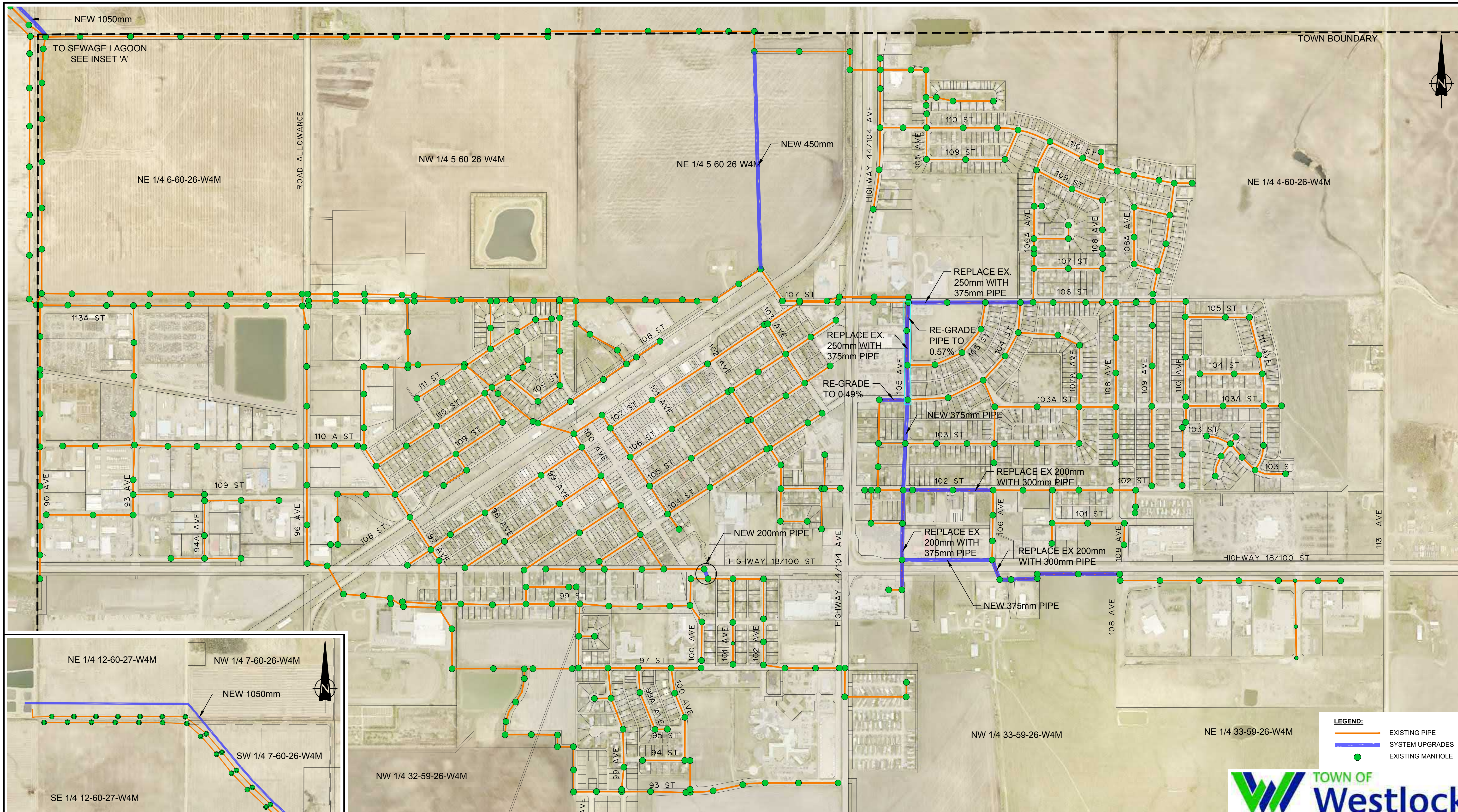
TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
PROP UPGRADES TO SANITARY SEWER SYSTEM
REDUCED BUILD-OUT SCENARIO 1

SCALE: 1:10000	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 5.8



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TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
PROP UPGRADES TO SANITARY SEWER SYSTEM
REDUCED BUILD-OUT SCENARIO 2

SCALE: 1:10000	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 5.9

MPE recommends that the Town confirm the actual manhole elevations and sanitary sewer main upgrades prior to construction of any sanitary sewer upgrades.

The existing 200 mm diameter main that flows east-west from 102 Avenue to 101 Avenue is modelled to surcharge, and be over capacity. This is due to the mains being installed at 0.12% grade, which is less than the minimum 0.4% grade in the Design Standards. To resolve these issues, MPE examined the following options:

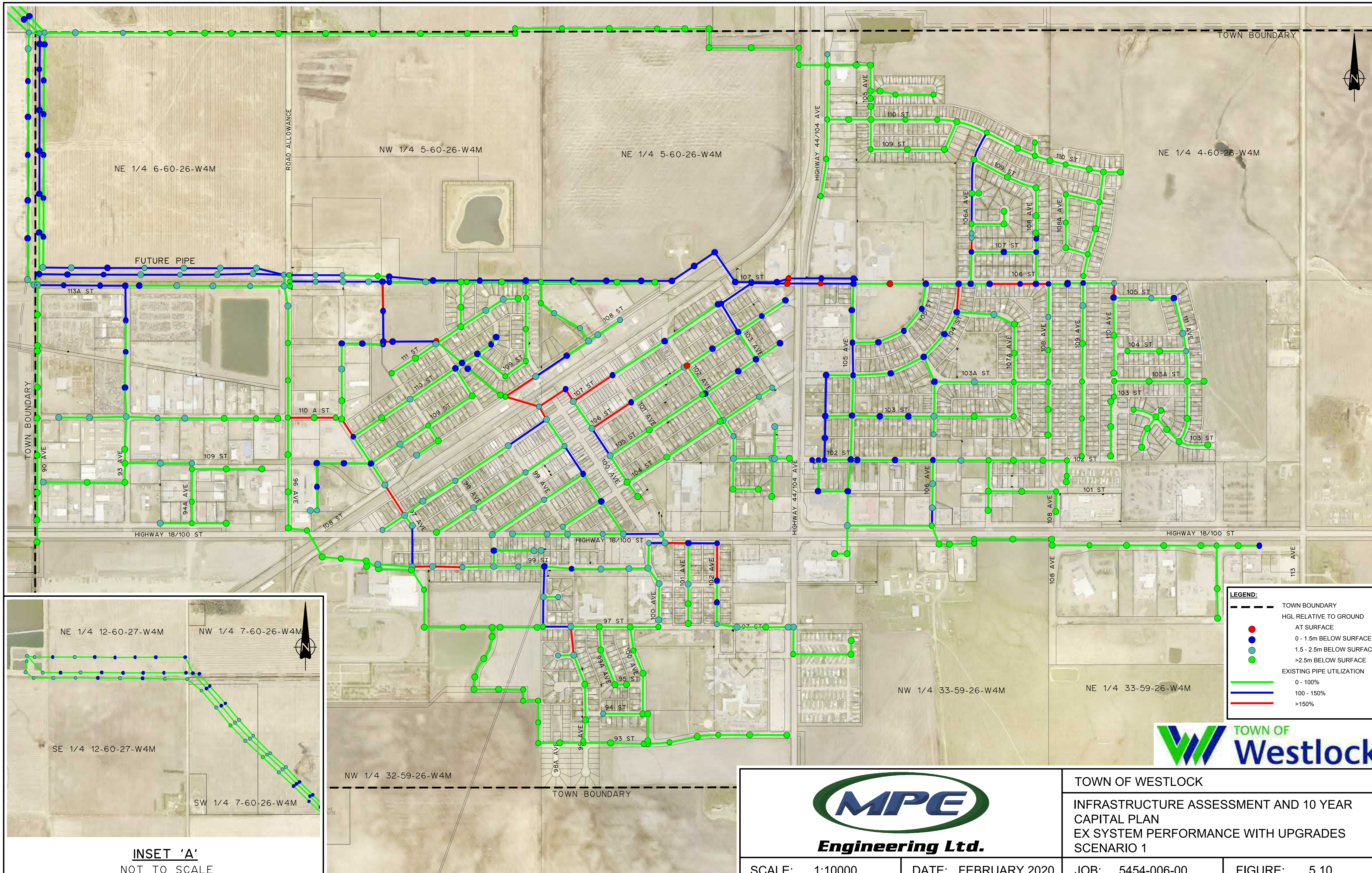
- Construction of a 200 mm diameter sanitary sewer main crossing Highway 18 east of the intersection with 100 Avenue.
- Replacement of the main with 0.12% grade, and the main immediately upstream, by increasing the size or slope of these mains.

When examining these options, the 200 mm diameter sanitary sewer main alleviated some of the manhole surcharging. Increasing the size of the main with 0.12% grade would result in a larger diameter main flowing into a smaller diameter main. This is because the mains downstream are not modeled to surcharge, and do not require upgrading. According to information available to MPE, the mains and road surface in the area were rehabilitated by the Town in 2016. Recommending upgrading of main in an area that was recently rehabilitated will have a negative public perception for the Town amongst neighbourhood residents. As such, it is recommended that the Town conduct flow monitoring along this section to determine the actual flows in the area to confirm the degree of surcharging that occurs in the area.

If the Town proceeds with crossing Highway 18 with a sanitary sewer main, MPE recommends the Town consider a sanitary sewer main larger than 200 mm in diameter to accommodate future growth. The costs associated with this upgrade are mostly due to crossing the Highway, and an increase in pipe size is minimal in comparison.

Upgrades to the existing sanitary sewer system for each scenario are shown in Figures 5.8 and 5.9. The performance of the Town's sanitary sewer system with the proposed upgrades for each scenario is shown in Figures 5.10 and Figure 5.11 on the following pages.

As mentioned above, the sanitary sewer system has issues with inflow and infiltration. The exact causes of the inflow and infiltration are unknown. MPE recommends that the Town undertake an inflow and infiltration program to determine the sources of flow into the sanitary sewer system. This program could include:



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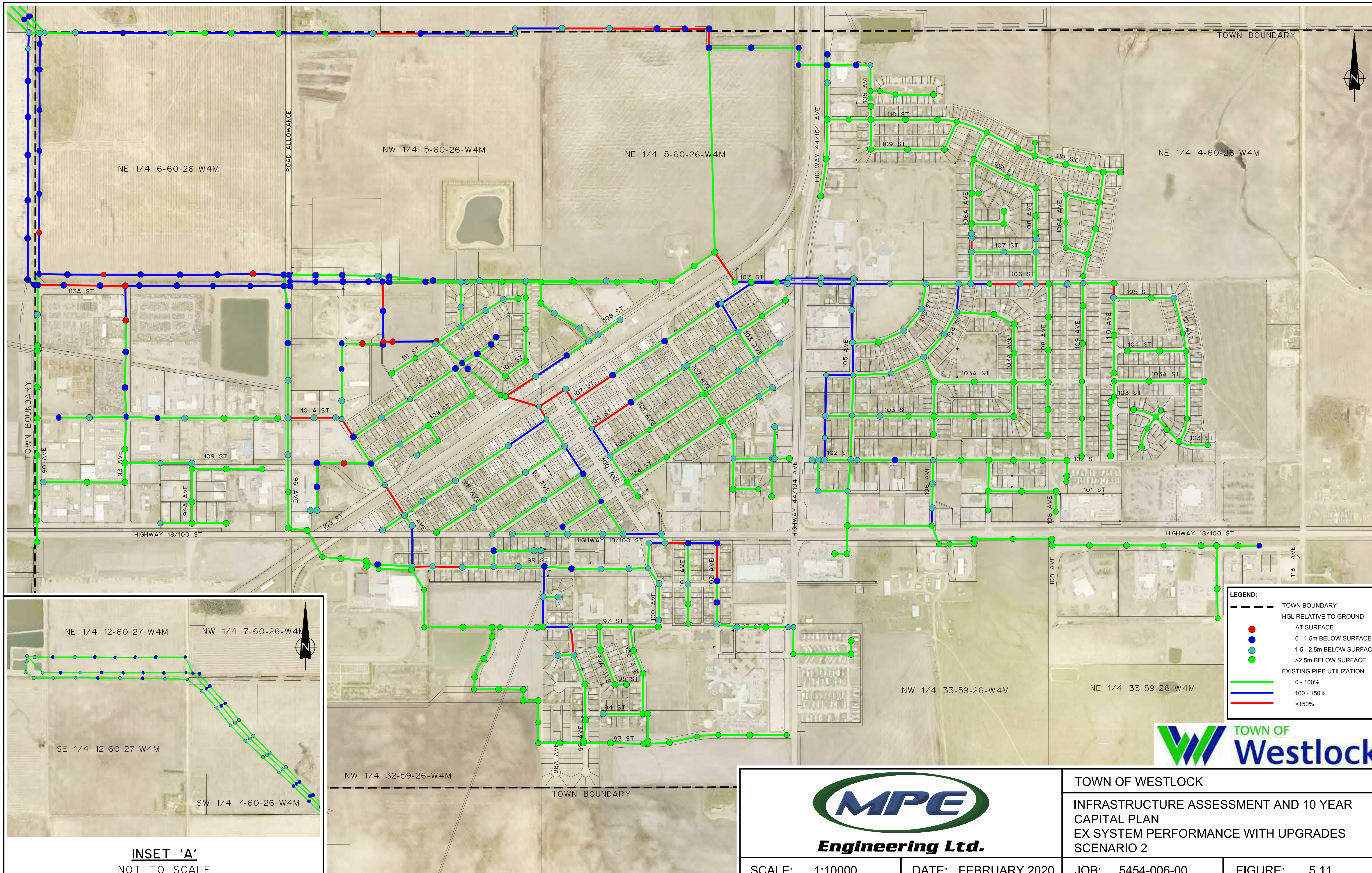
TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
EX SYSTEM PERFORMANCE WITH UPGRADES
SCENARIO 1

SCALE: 1:10000	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 5.10



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TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
EX SYSTEM PERFORMANCE WITH UPGRADES
SCENARIO 2

SCALE: 1:10000	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 5.11

- Walking ditches to look for faulty manholes or ponding locations
- Smoke tests to identify poor or illegal connection
- CCTV inspection of the sanitary sewers

Addressing some of the poor or illegal connections and areas ponding over manhole covers will aid in relieving the sanitary sewer system during storm events.

MPE also recommends that the Town monitor newer neighbourhoods (Aspendale, Altador, Polymanth) to see how the new sanitary sewer mains perform in comparison to the older portions of the system. This monitoring will also provide further model calibration data in the northeast portion of Town.

5.8.2.3 Scenario Analysis

Scenario 1 model results (Figure 5.10) resolves the manhole and pipe surcharging at the west industrial park. This scenario also resolves most of the surcharging along in the eastern portion of the Town, north of Highway 18 on 102 Street and 105 Avenue.

There is a section of sanitary sewer on 106 Street between 106A Avenue and 108 Avenue that has greater than 150% utilization in both scenarios. However, the adjacent manholes are not surcharging. This high utilization is caused by the sanitary sewer main having less grade than the mains upstream or downstream. The Town could resolve this surcharging through re-grading the sanitary sewer on 106 Street. However, this area is not considered a high priority for upgrades due to the manholes not surcharging in the model.

There are other sanitary sewer sections with high utilization that are not surcharging. This is typically a result of a lower pipe slope than adjacent pipe, similar to the 106 Street example above.

Upgrade Scenario 2 resolves the manhole and pipe surcharging along the eastern portion of the Town. This scenario conveys flows to the sanitary trunk along the north limits of the Town, which results in pipe surcharging along this trunk. Some manhole and pipe surcharging from the west industrial park and the existing sanitary trunk in the NE ¼ Sec. 6-60-26 W4M is resolved in this scenario, but not as much as in Scenario 1.

The majority of the recommended upgrades to the existing sanitary sewer system are:

- To improve the capacity issues on the eastern residential portion of Town; and
- To accommodate additional flows from planned developments in the southeast portion of Town (serviced by the sanitary sewer on Highway 18, 102 Street, and 105 Avenue).

Scenario 2 resolves the capacity issues in the eastern portion of Town with less surcharging than Scenario 1. Upgrading Scenario 1 resolves more capacity issues from the west industrial park than Scenario 2. However, there is no development planned in the west industrial park.

MPE recommends that the Town implement Scenario 2 if they can accept the surcharging in the west industrial park. If the Town proceeds with Scenario 2, MPE recommends that the Town proceed with an inflow and infiltration program within the west industrial park to determine the sources of inflow in that area and explore options to reduce it. Inflow and infiltration was a cause of surcharging in the west industrial park during the 2016 storm event.

6.0 STORM DRAINAGE SYSTEM

6.1 Overview

6.1.1 Existing Drainage

The existing drainage in the Town is split into two, roughly in line with 104 Avenue (Highway 44). The east half of the Town drains north and northeast. The west half of the Town drains north and northwest, towards Wabash Creek.

Overland flow from the south is intercepted by the ditch on the south side of Highway 18 and is conveyed to the West Ditch. The West Ditch begins near Highway 18 and 96 Avenue and travels north and west towards Wabash Creek.

6.1.2 Existing Storm Drainage Infrastructure

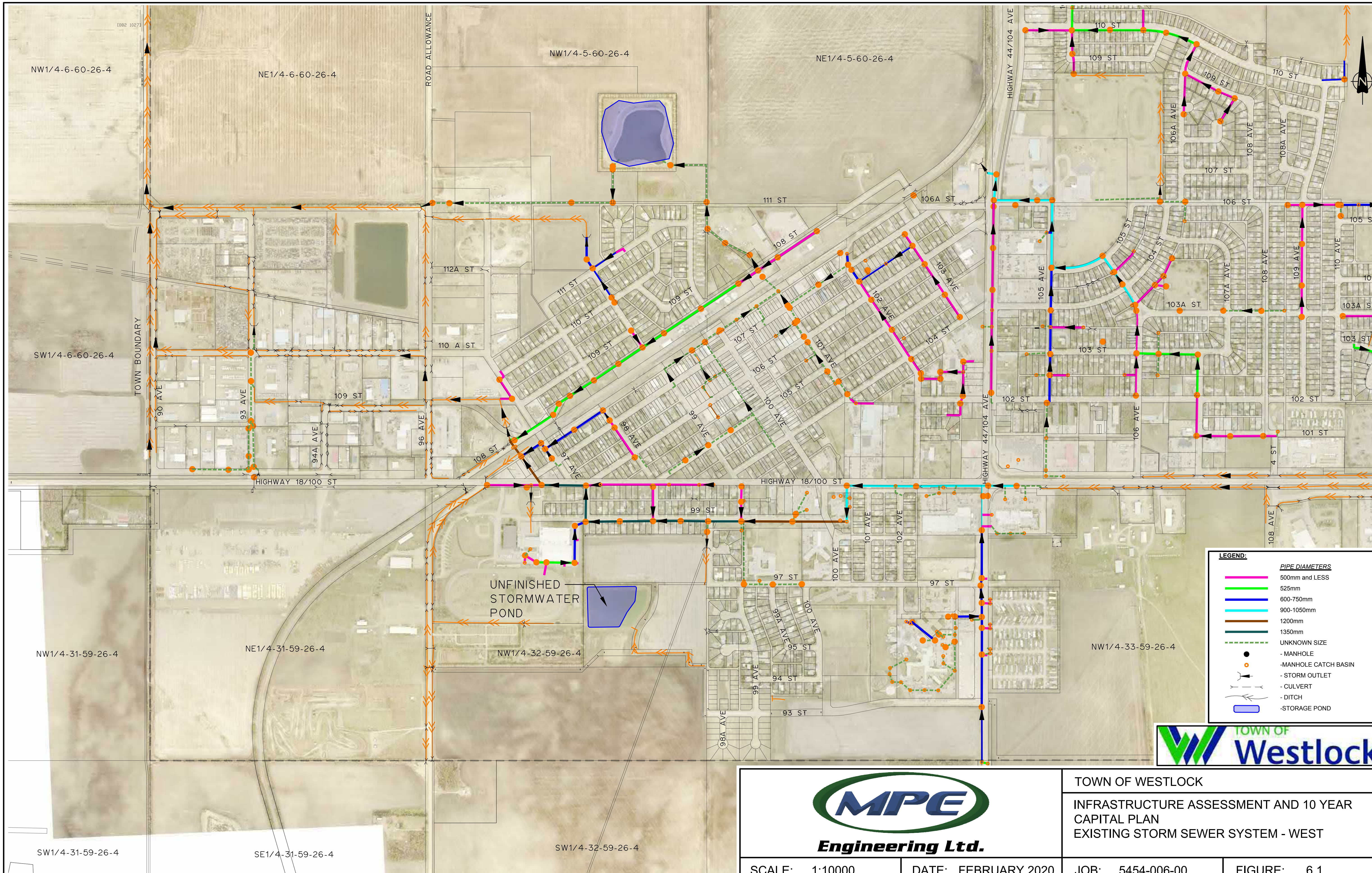
The existing storm drainage system consists of a combination of open ditches, storm sewers and stormwater management facilities. This is shown in ***Figure 6.1 (Existing Storm Sewer System – West)*** and ***Figure 6.2 (Existing Storm Sewer System – East)***. Water generally flows towards the north with the flows split by 104 Avenue. The drainage west of 104 Avenue flows to the northwest and into a stormwater management facility located in the NW ¼ Sec. 5-60-26 W4M. From this facility, the water flows to the northwest and enters Wabash Creek through a series of surface drainage ditches.

Drainage from areas to the south of 100 Street (Highway 18), flows to the northwest through storm sewers and drainage channels. Flows cross the railroad tracks at 108 Street and move north along 96 Avenue through surface drainage channels and drains into Wabash Creek.

Areas east of 100 Avenue drain to the north and northeast. The drainage between 104 Avenue and 108 Avenue moves to the north through the storm sewer system and empties into the highway ditches on 104 Avenue. The areas east of 108 Avenue drain northeast through the storm sewer system and surface drainage channels. The areas south of 100 Street and east of 104 Avenue drain to the north and west.

6.2 Stormwater Master Plan

MPE Engineering Ltd. completed the *Town of Westlock, Stormwater Master Plan Update 2017, January 18, 2018* (2017 Stormwater Master Plan Update). MPE recommends that upgrades to the storm drainage system from the 2017 Stormwater Master Plan Update be carried forward to this report to address capacity issues within the Town's storm drainage system.

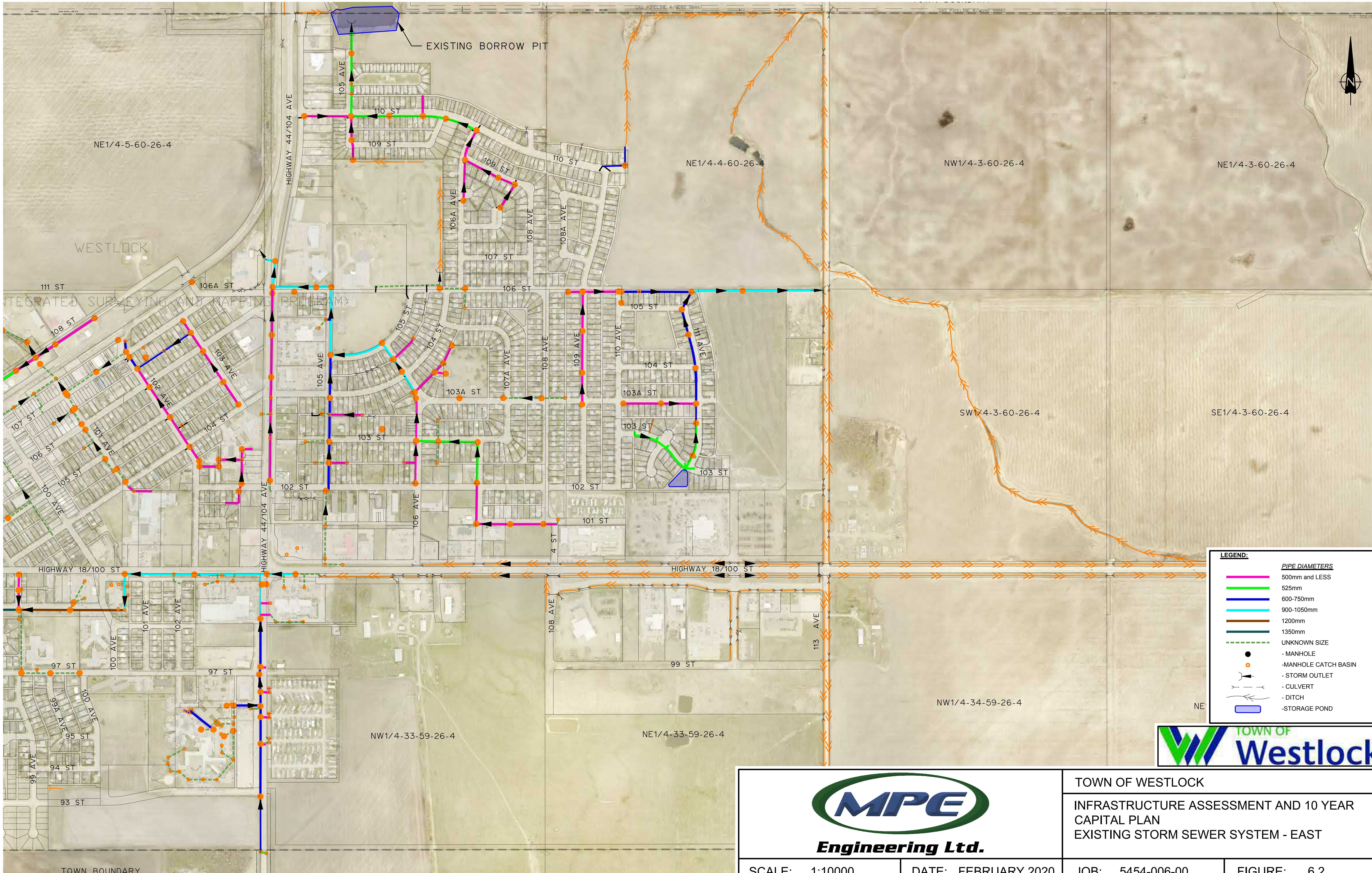


MPE Engineering Ltd.

TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
EXISTING STORM SEWER SYSTEM - WEST

SCALE: 1:10000	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 6.1



MPE Engineering Ltd.

TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
EXISTING STORM SEWER SYSTEM - EAST

SCALE: 1:10000	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 6.2

The Town is currently in the design phase of the following recommended projects from the 2017 Stormwater Master Plan Update:

- **Priority 1 – West Ditch Upgrades and Naturalized Stormwater Pond project**

  The West Ditch that runs along 96 Avenue and west along the 113A Street road allowance is currently undersized to meet both the 1:100 and 1:10-year flood capacities. As part of the ditch upgrades, the Town is widening and deepening the existing channel along 113A Street, west of 96 Avenue and north to the proposed stormwater pond. The Town is constructing this pond in the SW ¼ Sec. 7-60-26 W4M. The pond will provide storage for the 1:100-year storm event, and will provide adequate treatment prior to discharge into Wabash Creek.

- **Priority 2 – 108 Street Outlet Replacement**

  This outlet is damaged and has caused the building of sediment at the beginning of the West Ditch that has nearly blocked the 108 Street Crossing. The Town will install new 1,650 mm diameter pipe and a new flared end structure complete with gabion end treatment to repair the damaged outlet.

The remaining recommended projects from the 2017 Stormwater Master Plan Update include:

- **Priority 3 – Future Development in Westgate and Greenfield Estates**

  The existing storm sewer system along 97 Avenue and 99 Street south of Highway 18 is comprised of three sections of 1,350 mm diameter pipe that is surcharging in the stormwater model during the 1:5-year storm event. MPE is proposing to upsize these three sections to 1,650 mm diameter pipe to provide capacity for the future discharge of the Greenfield Estates pond. MPE anticipates that the Greenfield Estates pond and the Westgate ditch will be constructed in the near future.

- **Priority 4 – Stormwater System Preventative Maintenance**

  MPE is proposing to work with the Town to develop a Stormwater System Preventative Maintenance Plan. These generally include annual maintenance tasks that include:

  - Street sweeping to remove grit from roadways before it enters the system
  - Hydrovaccing catchbasin sumps to remove sediment build-up
  - Adding storm manholes at strategic locations to allow for effective access
  - High pressure flushing of the stormwater pipes to remove deposits within the pipes

  These tasks are typically implemented in a cyclic manner to allow for every portion of the system to be maintained within a certain period, such as four years, meaning that approximately one quarter of the system would be maintained every year.

- **<u>Priority 5 – Pond G</u>**

  Pond G is identified in the 2009 Stormwater Master Plan Update. This pond is located east of Highway 44 and south of Highway 18 and will support a catchment area of approximately 110 hectares. MPE is proposing to construct Pond G with a total storage volume of 43,400 m^3 and a controlled outlet flow rate of 0.44 m^3/s, as per the previous master plan update. The proposed location of the pond is east of the Westlock Inn.

- **<u>Priority 6 – Overland Drainage in Southview</u>**

  It is noted that the neighbourhood of Southview, located south of Highway 18, is frequently subject to overland flooding. This is due to a limited storm sewer system in Southview. The conveyance of stormwater in Southview is primarily overland. MPE is proposing a survey of Southview be completed to determine any grade deficiencies and the feasibility of swales. The primary inflow of stormwater into Southview is through overland flow from the undeveloped land to the south. Once the Westgate ditch is implemented, MPE assumes that these issues may cease naturally due to decreased inflow into the neighbourhood.

- **<u>Priority 7 – General Surcharging During 1:5-Year Storm Events</u>**

  An important finding of the stormwater modeling was that various portions of the piped system are surcharging during the 1:5-year storm event. MPE is proposing the Town approach these upgrades on a case-by-case basis as issues arise or as budget permits. These small upgrades could be implemented with the annual preventative maintenance plan as scheduled upgrades.

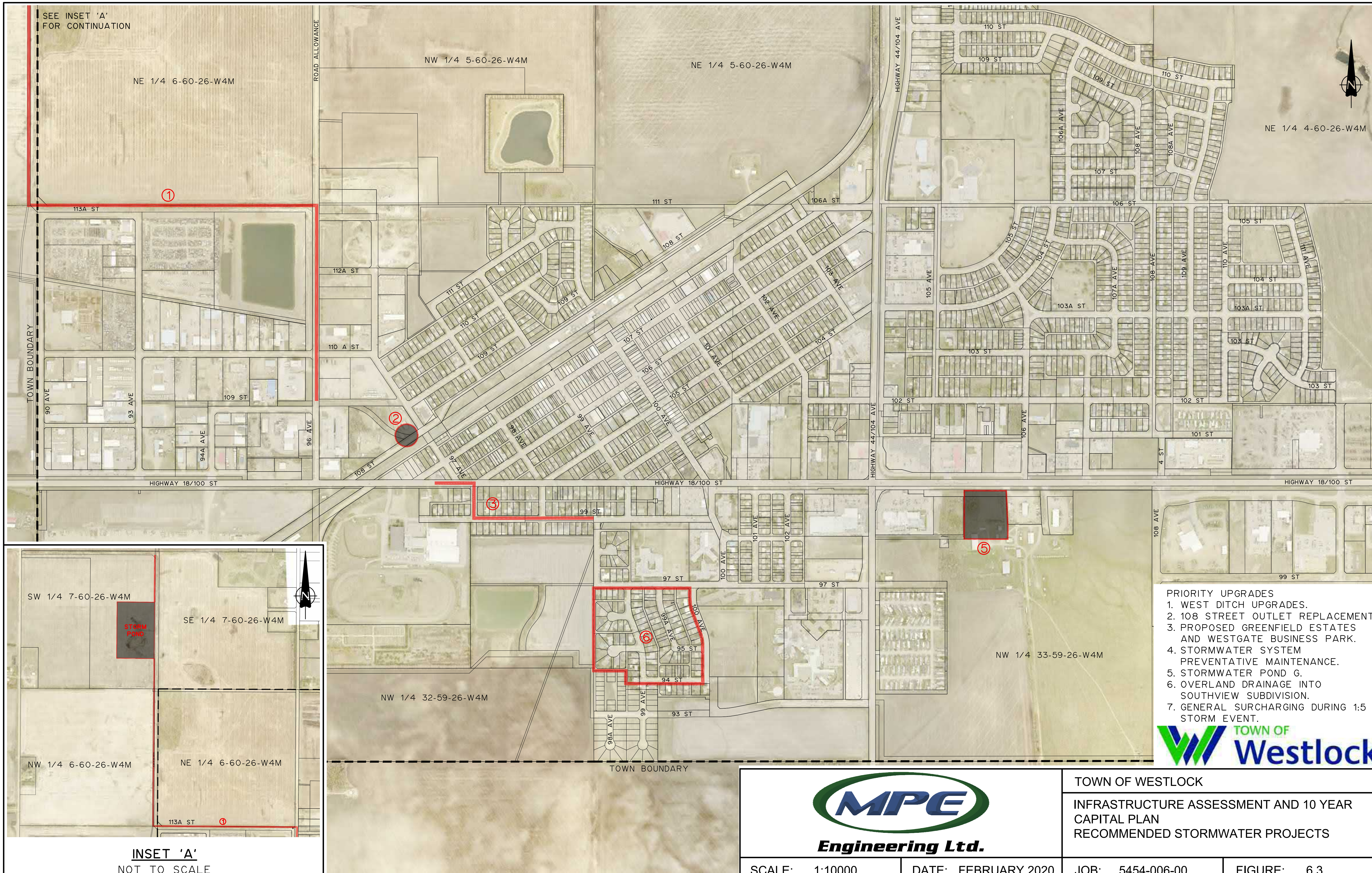
The location of these upgrades is shown in ***Figure 6.3 (Recommended Stormwater Projects)***.

6.3 Condition

There is no information available on the condition of the storm drainage system infrastructure. MPE recommends that the Town develop a program to inspect the storm sewer system by CCTV, and visually inspect all storm drainage channels, stormwater management facilities, culverts, inlets and outfalls to determine their condition. These inspections can be completed prior to recommended water or sanitary sewer main rehabilitation to determine if the storm sewer main is in the same location as the mains requiring rehabilitation.

6.4 Operation and Maintenance of the Storm Sewer System

The Stormwater Master Plan identified a preventative maintenance program the Town could implement as Priority 4. The tasks identified in the priority item are usually implemented in a cyclic manner to allow



PRIORITY UPGRADES
1. WEST DITCH UPGRADES.
2. 108 STREET OUTLET REPLACEMENT.
3. PROPOSED GREENFIELD ESTATES AND WESTGATE BUSINESS PARK.
4. STORMWATER SYSTEM PREVENTATIVE MAINTENANCE.
5. STORMWATER POND G.
6. OVERLAND DRAINAGE INTO SOUTHVIEW SUBDIVISION.
7. GENERAL SURCHARGING DURING 1:5 STORM EVENT.

TOWN OF Westlock

MPE Engineering Ltd.

TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
RECOMMENDED STORMWATER PROJECTS

SCALE: 1:10000	DATE: FEBRUARY 2020	JOB: 5454-006-00	FIGURE: 6.3

for the entire system to be maintained within a certain period. For example, the Town could maintain one quarter of the system every year, resulting in maintenance of the entire system over a four-year period.

Depending on the Town's desired level of service and the overall cleanliness of the system, the estimated cost for the preventative maintenance program ranges between $25,000.00 and $100,000.00 per year. It is expected that the first cycle would cost more, as it will require more effort to bring the system back to a good level of service.

MPE recommends that the Town implement a preventative maintenance program as described above as part of the ongoing operation and maintenance of the storm sewer system. It is also recommended that the Town include the budget for this program as part of their operating budget.

7.0 10-YEAR CAPITAL PROJECTION

MPE prepared a 10-Year Capital Projection that outlines the proposed schedule for project delivery based on municipal needs. The projection outlines high-priority rehabilitation and maintenance required to address immediate issues within the infrastructure systems. MPE recommends the Town re-evaluate existing needs and solidify the remaining items of the 10-year capital projection annually.

Projects identified in the 10-year capital projection are outlined in this section. MPE has calculated approximate costs for these projects as outlined below, and our assumptions in calculating these costs are:

- In areas where storm sewer is present, MPE recommends that the Town inspect the storm sewer main by CCTV prior to construction to determine the condition.
- In areas where the sanitary sewer main was not inspected, it is recommended that the Town inspect these mains by CCTV to determine their condition.
- MPE has included the increase in watermain size recommended in Section 3.5 in areas of sanitary and storm sewer main rehabilitation.
- MPE has assumed roadway repairs for water, sanitary sewer and storm sewer rehabilitation.
  - In areas where only one underground utility is rehabilitated, it is assumed restoration of the trench, and a mill and overlay of the road.
  - In areas where two or more underground utilities are rehabilitated, a complete roadway reconstruction is assumed, as the rehabilitation will involve the removal of the majority of the roadway.

The unit rates used for the rehabilitation recommendations are:

- Watermain, Open-Cut Replacement: $1,065.00/m
- Sanitary Sewer Main, Open-Cut Replacement (200 mm): $1,190.00/m
- Sanitary Sewer Main, Open-Cut Replacement (Upgrade to 300 mm): $1,440.00/m
- Sanitary Sewer Main, Open-Cut Replacement (Upgrade to 375 mm): $1,565.00/m
- Sanitary Sewer Main, Spot Repairs: $2,500.00/m
- Sanitary Sewer Main, Cured-In-Place Pipe (Relining): $315.00/m
- CCTV Inspection of Storm/Sanitary Sewer: $12.50/m
- Roadway, Trench Reconstruction: $750.00/m
- Roadway, Mill and Overlay: $375.00/m
- Roadway, Reconstruction: $2,000.00/m

These unit rates engineering (15%) and contingency (10%).

7.1 Project Master List

MPE prepared a project master list illustrated in ***Table 7.1 on the following page***.

Using the project master list, MPE selected projects with the highest priority first that would create the Town's 10-Year Capital Plan (see Section 7.2). The number of each project corresponds to the number in Table 7.1.

A summary and cost estimate of each project is outlined below.

1. **Sanitary Sewer Trunk – 450 mm Diameter Main Through NE ¼ Sec. 5-60-26 W4M:** First stage of construction for the sanitary sewer trunk. Open-cut installation of 450 mm diameter sanitary sewer main through the NE ¼ Sec. 5-60-26 W4M. Estimated Cost: $525,000.00, Year of Construction: 2020.

2. **Sanitary Sewer Trunk – Northwest Corner of Town to Sewage Lagoons:** Second stage of construction for the sanitary sewer trunk. Open-cut installation of 1,050 mm diameter sanitary sewer main. Due to cost, MPE has extended this project over three years so the Town has enough budget to complete the project. Estimated Cost: $3,750,000.00, Years of Construction: 2020, 2021, 2022, 2023, and 2024.

3. **Water and Sanitary Sewer Main Replacement – 105 Avenue, 106 Street to 104 Street:** Open-cut replacement of water and sanitary sewer main. Increase watermain size to 200 mm in diameter. Increase sanitary sewer size to 375 mm in diameter. Inspect storm sewer main by CCTV to determine condition. Full reconstruction of roadway. Estimated Cost: $1,393,000.00, Year of Construction: 2025, 2026, and 2027.

4. **Sanitary Sewer Main Replacement – 105 Avenue, 104 Street to 102 Street:** Open-cut installation of 375 mm diameter sanitary sewer main. Inspect storm sewer main by CCTV to determine condition. Trench reconstruction and mill and overlay of roadway. Estimated Cost: $744,000.00, Year of Construction: 2027.

5. **Water and Sanitary Sewer Main Replacement – 104 Street, 105 Avenue to 103 Avenue:** Open-cut replacement of water and sanitary sewer main. Increase watermain size to 200 mm in diameter. Full reconstruction of roadway. Estimated Cost: $669,000.00, Year of Construction: 2027 and 2028.

6. **Sanitary Sewer Main Replacement – 106 Street, 105 Avenue to 106A Avenue:** Open-cut replacement of sanitary sewer main, and increase in size to 375 mm in diameter. Trench reconstruction and mill and overlay of roadway. Estimated Cost: $1,022,000.00.

Town of Westlock
Infrastructure and 10 Year Capital Plan
Table 7.1 - Project Master List

					Existing Infrastructure					Water Main		Sanitary Sewer							Storm Sewe	Road			Other	Total Cost
Project Number (From Report)	Year	Description			Water	Sanitary	Storm	Road	Prices DO NOT Include Mobilization	Open-Cut Replacement (Removal, Supply, Install)	Pipe Bursting	Open-Cut Replacement (Removal, Supply, Install)	Open-Cut Replacement (300 mm)	Open-Cut Replacement (375 mm)	Spot Repairs	Cured-In Place Pipe (Re-lining)	CCTV Inspection of Sanitary	CCTV Inspection of Storm	Storm Pipe (Supply and Install)	Trench Reconstruction: 3 m road width	Mill and Overlay 10 meter width	Local Road Reconstruction: 10 m road width	Other Projects	Excludes Engineering and Contingency
										$1,065.00	$750.00	$1,190.00	$1,440.00	$1,565.00	$2,500.00	$315.00	$12.50	$12.50	$1,250.00	$750.00	$375.00	$2,000.00		
									Units	m	m	m	m	m	m	m	m	m	m	m	m	m		
		Street/Avenue	From	To																				
		MPE Hydraulic Assessment (Sanitary)																						
1	2020	Construct a new 450 mm diameter sewer main through NE 1/4 Sec. 5-60-26 W4M																						$525,000.00
2	2020, 2021, 2022, 2023, 2024	Construct a new 1050 mm diameter outfall line to the lagoons from the northwest corner of Town.																						$3,750,000.00
3	2025, 2026, 2027	105 Avenue	106 Street	104 Street	x	x	x	x		300				300				300				300		$1,392,750.00
4	2027	105 Avenue	104 Street	102 Street		x	x	x						275				275		275	275			$743,187.50
5	2027, 2028	104 Street	105 Avenue	103 Avenue	x	x		x		185		85										185		$668,175.00
6		106 Street	105 Avenue	106A Avenue	x	x		x						380						380	380			$1,022,200.00
7		102 Street	106 Avenue	105 Avenue	x	x		x					270							270	270			$692,550.00
8		Highway 18/100 Street Connection						x				30								30	30			$111,000.00
		MPE Condition Assessment																						
9		107A Avenue	104 Street	103A Street	x	x		x		380		380										380		$1,616,900.00
10		99a Avenue	97 Street	95 Street	x	x		x		200							200					200		$615,500.00
11		B/w 105/104 Avenue	104 Street	102 Street	x	x				275		275								275		0		$826,375.00
12		99 Avenue	110 Street	108th Street	x	x	x	x		160								55		160	160			$351,087.50
13		106 Street	98 Avenue	100 Street	x	x		x		140		140										140		$595,700.00
14		106 Street	99 Avenue	100 Avenue	x	x		x		150							150			150	150			$330,375.00
15		West of 100 Avenue (Back Alley)	104 Street	100 Street		x		x							10	130				10	10			$77,200.00
16		98th Street, East of 98A Avenue			x	x		x		55							55			55	55			$121,137.50
17		B/w 104 Ave and 102 Ave	A&W Canada	Esso (North of	x	x	x			100		190						150		480				$694,475.00
18		106 Avenue	102 Street	100 Street	x	x		x		230		230										230		$978,650.00
19		Flushing and Inspection	112A Street	106th Street	x	x		x									800							$10,000.00
20		104 Street	102 Avenue	103 Avenue	x	x		x		200		200										200		$851,000.00
21		99 St Back Alley	98 Avenue	99A Avenue	x	x				250		250								250				$751,250.00
22		Sludge Survey for Lagoons																					10000	$10,000.00
23		Storm - Future Development in Westgate and Greenfield Estates as per 2017 Stormwater Master Plan																						$760,000.00
24		Storm - Pond G as per 2017 Stormwater Master Plan																						$1,340,000.00
		MPE Hydraulic Assessment (Water)																						
25	2024, 2025	Construct a new 300 mm diameter watermain in the commercial area in the SE with a length of 770 meters.								770										770				$1,397,550.00
26	2028	Construct a new 300 mm diameter watermain off of 102 Street								130										130				$235,950.00
27		Upsize watermain from 105 Ave (back alley) to 104 Street, including trench re-construction								210										210				$381,150.00
28	2028, 2029, 2030	Upsize watermain along 107 Street From 97 Ave to 99 Ave and 101 Avenue to 103 Avenue to a 200 mm dia. watermain.								840							840	100		840				$1,536,350.00
29	2028	Construct a new 250 mm diameter watermain 20 meters in length between 93 Ave, 110A St, Highway 18 & 97 Ave.								20										20				$36,300.00
		Water (2009 Water Master Plan)																						
30		Construct a new 250 mm diameter watermain from the 200 mm diameter watermain along 103 Street east of 111 Avenue east to near the east Town boundary. The watermain will then run south to the existing 250 mm diameter watermain south of Highway 18.																						$400,000.00
		Sanitary (WSP Flow Monitoring)																						
31		Conduct an inflow and infiltration identification program, particularly within older portions of the Town to identify what may be causing the high flows in wet weather. This program could include ditch walks, smoke tests and camera inspections.																						$75,000.00
32		Investigate additional options such as storage including pumping to empty storage facilities after peak flows subside.																						$25,000.00
		Sanitary (2009 Wastewater Master Plan)																						
33		Construct an extension of the 375 mm diameter sewer main from near the Hospital east across Highway 44 to the new extension of the 375 mm diameter sanitary sewer main from the hospital.																						$250,000.00
34		Construct a pump station and forcemain from the 200 mm diameter sanitary sewer main at 106 Street south of Highway 44 to the new extension of the 375 mm diameter sanitary sewer main from the hospital.																						$1,800,000.00
35		Construct a network of new trunk sanitary sewer mains to feed a pump station in the extreme northeast.																						$3,500,000.00
36		Construct a new 750 mm diameter trunk sanitary sewer main from Highway 44 to the northwest corner of Town along the north Town boundary.																						$1,930,000.00

7. **Sanitary Sewer Main Replacement – 102 Street, 106 Avenue to 105 Avenue:** Open-cut replacement of sanitary sewer main, and increase in size to 300 mm in diameter. Trench reconstruction and mill and overlay of roadway. Estimated Cost: $693,000.00.
8. **Sanitary Sewer Main Installation – Highway 18/100 Street:** Trenchless installation of 200 mm diameter sanitary sewer main across Highway 18/100 Street east of 100 Avenue intersection. Estimated Cost: $111,000.00.
9. **Water and Sanitary Sewer Main Replacement – 107A Avenue, 104 Street to 103A Street**: Open-cut replacement of the water and sanitary sewer main. Increase watermain size to 200 mm in diameter. Inspect storm sewer main by CCTV to determine condition. Full reconstruction of the roadway. Estimated Cost: $1,617,000.00.
10. **Watermain Replacement – 99A Avenue, 97 Street to 95 Street**: Open-cut replacement of the watermain, and increase in size to 200 mm in diameter. CCTV inspection of the sanitary sewer main to determine condition. Trench reconstruction and mill and overlay of the roadway. Estimated Cost: $616,000.00.
11. **Water and Sanitary Sewer Main Replacement – Alley Between 105 and 104 Avenue, 104 Street to 102 Street:** Open-cut replacement of the water and sanitary sewer main. Increase watermain size to 200 mm diameter. Full reconstruction of the alley. Estimated Cost: $827,000.00.
12. **Watermain Replacement – 99 Avenue, 110 Street to 108 Street**: Open-cut replacement of watermain. Increase watermain size to 200 mm in diameter. Inspect storm sewer main by CCTV to determine condition. Trench reconstruction and mill and overlay of roadway. Estimated Cost: $351,000.00.
13. **Water and Sanitary Sewer Main Replacement – 106 Street, 98 Avenue to 100 Street**: Open-cut replacement of water and sanitary sewer main. Increase watermain size to 200 mm in diameter. Full reconstruction of roadway. Estimated Cost: $596,000.00.
14. **Watermain Replacement – 106 Street, 99 Avenue to 100 Avenue**: Open-cut replacement of watermain. Increase watermain size to 200 mm in diameter. Inspect sanitary sewer main by CCTV to determine condition. Trench reconstruction and mill and overlay of roadway. Estimated Cost: $331,000.00.

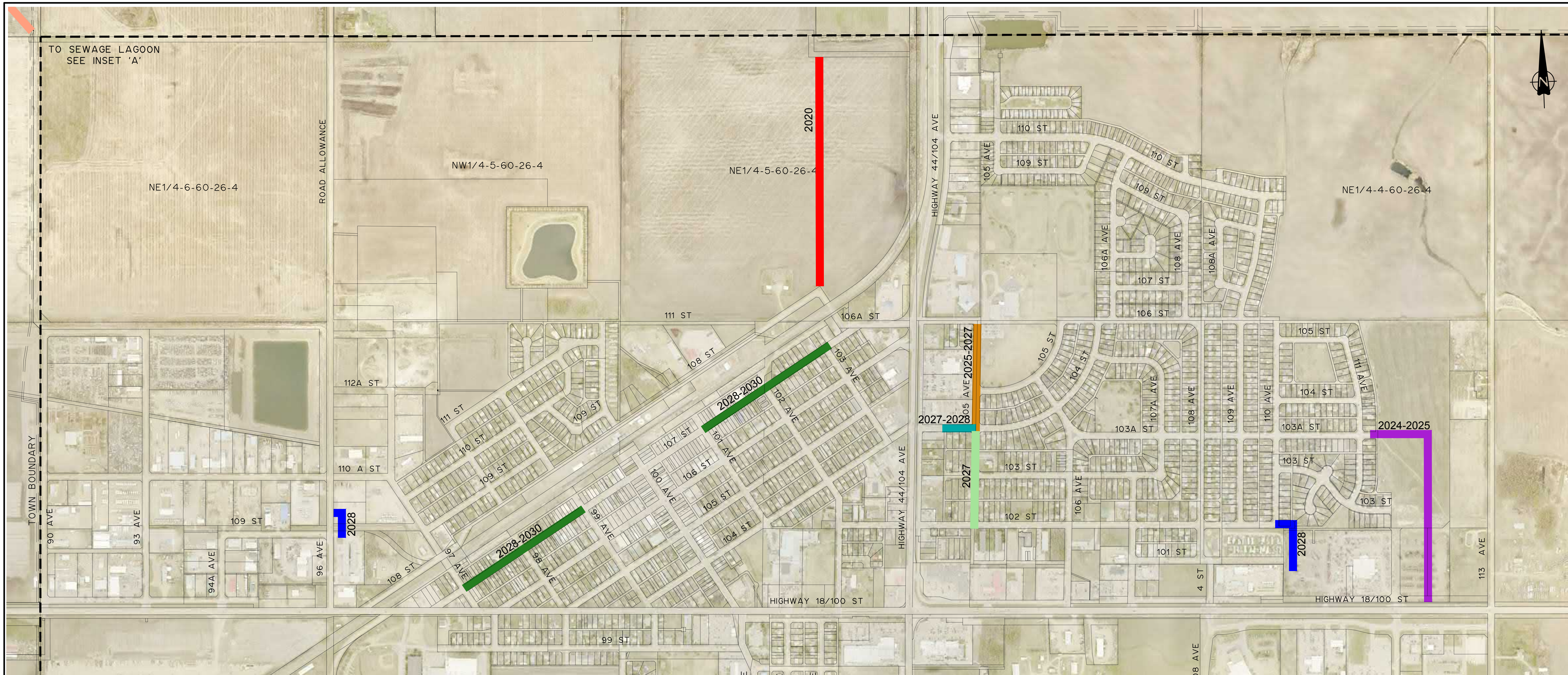
15. **Sanitary Sewer Main Rehabilitation – West of 100 Avenue (Back Alley), 104 Street to 100 Street**: Spot repair and relining of sanitary sewer main. Trench reconstruction. Estimated Cost: $77,000.00.
16. **Watermain Replacement – 98 Street, West of 98A Avenue**: Open-cut replacement of watermain. Increase watermain size to 200 mm in diameter. Inspect sanitary sewer main by CCTV to determine condition. Trench reconstruction and mill and overlay of roadway. Estimated cost: $121,000.00.
17. **Water and Sanitary Sewer Main Replacement – Between 104 Avenue and 102 Avenue, 103 Street to 100 Street**: Open-cut replacement of water and sanitary sewer main. Increase watermain size to 200 mm in diameter. Inspect storm sewer main by CCTV to determine condition. Trench reconstruction and mill and overlay of roadway. Estimated Cost: $695,000.00.
18. **Water and Sanitary Sewer Main Replacement – 106 Avenue, 102 Street to 100 Street**: Open-cut replacement of water and sanitary sewer main. Increase watermain size to 200 mm in diameter. Full reconstruction of roadway. Estimated Cost: $979,000.00.
19. **Flush and Inspect Sanitary Sewer – 112A Street to 106 Street**: Flushing sanitary sewer main noted to have high water and heavy debris. Inspection of sanitary sewer main by CCTV upon completion of flushing. Estimated Cost: $10,000.00.
20. **Water and Sanitary Sewer Main Replacement – 104 Street, 102 Avenue to 103 Avenue**: Open-cut replacement of water and sanitary sewer main. Increase watermain size to 200 mm in diameter. Full reconstruction of roadway. Estimated Cost: $851,000.00.
21. **Water and Sanitary Sewer Main Replacement – 99 Street Back Alley, 98 Avenue to 99A Avenue**: Open-cut replacement of water and sanitary sewer main. Increase watermain size to 200 mm in diameter. Full reconstruction of alley. Estimated Cost: $751,000.00.
22. **Sludge Survey of Wastewater Lagoons**: Sludge Survey of Wastewater Lagoons as recommended in Section 5.7. Estimated Cost: $10,000.00.
23. **Storm Sewer – Future Development in Westgate and Greenfield Estates**: Upsize three sections of 1350 mm diameter pipe to 1650 mm diameter pipe as per Priority 3 in Section 6.2. Estimated Cost: $760,000.00.
24. **Storm Sewer – Pond G**: The pond is located east of Highway 44 and south of Highway 18 and will support a catchment area of approximately 110 hectares. Total storage volume of Pond G is 43,400 m^3 and the controlled outlet flow rate is 0.44 m^3/s. Estimated Cost: $1,340,000.00.

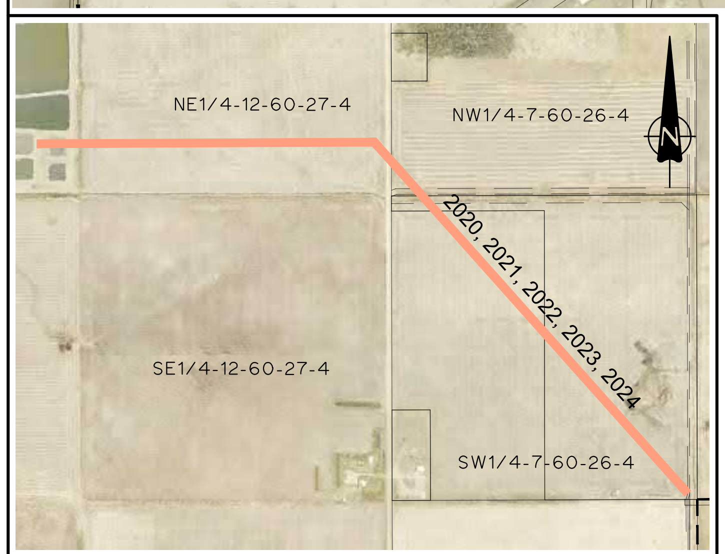
25. **Watermain Addition – Commercial Area in the SE:** Construct a new 300 mm diameter watermain including 3 m trench reconstruction. Estimated Cost: $1,398,000.00, Year of Construction: 2024 and 2025.
26. **Watermain Addition – Commercial Area in the SE:** Construct a new 300 mm diameter watermain off of 102 Street. Estimated Cost: $236,000.00, Year of Construction: 2028.
27. **Watermain Addition – Central Commercial Area along Highway 44:** Construct new 200 mm diameter watermain from 105 Avenue (back alley) to 104 Street including 3 m trench reconstruction. Estimated Cost: $381,000.00.
28. **Watermain Upsizing – Downtown:** Upsize watermain along 107 Street From 97 Avenue to 99 Avenue and 101 Avenue to 103 Avenue to a 200 mm diameter watermain including 3 m trench reconstruction. Estimated Cost: $1,537,000.00, Year of Construction: 2028, 2029, and 2030.
29. **Watermain Addition – Industrial Area on West Side:** Construct a new 250 mm diameter watermain between 93 Avenue, 110A Street, Highway 18 and 97 Avenue including 3 m trench reconstruction. Estimated Cost: $37,000.00, Year of Construction: 2028.

MPE has excluded Projects 6 through 25, and 27 from the 10-Year Capital Plan (see Section 7.2). However, we have included the projects on the master list for the Town's reference (Table 7.1). These projects were identified in previous master plans and studies provided to the Town. Should the Town have more funding available in any given year, MPE recommends re-evaluating the 10-Year Capital Plan and capital projects.

7.2 10-Year Capital Plan

MPE selected projects from the project master list (Table 7.1) to create the 10-Year Capital Plan. Table 7.2 outlines our recommended 10-Year Capital Plan for the Town of Westlock. In developing the Plan, MPE assumed the Town will have a capital budget of $1 million per year to complete the work. We assumed that the first priority to address in the Capital Plan is upgrading the hydraulics of the sanitary sewer and water distribution systems to reduce surcharging and improve fire flows throughout the Town. MPE also assumed that the second priority was to rehabilitate water and sanitary sewer mains that are in poor structural condition. It should be noted that the costs included below do not include engineering and contingency. ***Figure 7.1 (10-Year Capital Plan Summary of Costs)*** illustrates the 10-Year Capital Plan in addition to the capital costs for each year.





INSET 'A'
NOT TO SCALE

LEGEND

	10 YEAR CAPITAL PLAN	TOTAL COST
(red)	2020 UPGRADES	$525,000.00
(salmon)	2020-2024 UPGRADES	$3,750,000.00
(purple)	2024-2025 UPGRADES	$1,398,000.00
(orange)	2025-2027 UPGRADES	$1,393,000.00
(light green)	2027 UPGRADES	$744,000.00
(teal)	2027-2028 UPGRADES	$669,000.00
(dark green)	2028-2030 UPGRADES	$1,537,000.00
	TOTAL (2020 -> 2029)	$10,264,000.00
	REMAINING PROJECTS	$20,138,000.00

MPE Engineering Ltd.

TOWN OF WESTLOCK

INFRASTRUCTURE ASSESSMENT AND 10 YEAR CAPITAL PLAN
SUMMARY OF COSTS

SCALE: NOT TO SCALE | DATE: FEBRUARY 2020 | JOB: 5454-006-00 | FIGURE: 7.1

TABLE 7.2: 10-YEAR CAPITAL PLAN

10-Year Capital Plan	Total Cost	Projects Selected From Master List			
2020	$1,000,000.00	1	2		
2021	$1,000,000.00	2			
2022	$1,000,000.00	2			
2023	$1,000,000.00	2			
2024	$1,000,000.00	2	25		
2025	$1,000,000.00	25	3		
2026	$1,000,000.00	3			
2027	$1,000,000.00	3	4	5	
2028	$1,000,000.00	5	26	29	28
2029	$1,000,000.00	28			
2030	$264,000.00	28			
Total (2020 --> 2029)	**$10,264,000.00**				
Remaining Projects Past 2028	$20,138,000.00				

Prices do not include engineering or contingency, and are net of GST

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

WATER SUPPLY AND DISTRIBUTION SYSTEM

- The Town has completed the following recommended upgrades from the 2009 Water Master Plan Update:
  - The Town constructed a new 200 mm diameter watermain to the arena complex.
  - The Town added some 300 mm diameter watermain looping along the south perimeter of Town.
- The five sections of watermain with the most breaks in the last 20 years are:
  - 99A Avenue, between 97 Street and 95 Street – 8 breaks in the last 10 years
  - 107A Avenue, between 103A Street and 104 Street – 9 breaks in the last 20 years
  - 106 Street, between 99 Avenue and 100 Avenue – 5 breaks in the last 10 years
  - 98 Street, east of 98A Avenue – 5 breaks in the last 20 years
  - 99 Avenue, between 110 Street and 108 Street – 5 breaks in the last 20 years
- The Town has sufficient treated water storage for 10 years.

WATER NETWORK ANALYSIS

- MPE has assumed that the commercial/industrial consumption within the existing water distribution system from the 2005 Water Master Plan has not changed.
- Commercial/Industrial water consumption values used for future development are consistent with those for future development from other municipalities such as Parkland County and Sturgeon County. For other municipalities these values are approximately half of other consumption values.
- On May 18, 2019, Town staff performed flow testing of ten fire hydrants within the Town. Overall, the fire flow testing in the field correlated well with the hydraulic water model. Some field data had lower or questionable accuracy.
- Both the Water Treatment Plant and Water Reservoir and Pumping Station have adequate capacity to supply water for peak hour demand in the future and current system.
- The Water Treatment Plant does not have adequate pumping capacity to meet maximum day + fire flow for residential, industrial or commercial properties.
- The Water Reservoir and Pumping Station has adequate pumping capacity to supply maximum day + fire flow for the future and current systems for residential, industrial and commercial properties.
- The combination of existing pipe sizes, looping locations, and distance from the WRPS limits the Town’s ability to meet fire flow requirements for residential and commercial development. This limited ability results in deficient fire flows in many locations throughout Town.

- The pressure range in the water distribution system is acceptable for a single pressure zone.
- Figure 4.1 and Figure 4.2 illustrate the Town's ability to meet fire flow demands for corresponding land uses within different areas of Town.
- Four (4) areas within Town were evaluated to assist in prioritizing water main upgrades to improve fire flow availability. These upgrades are reflected in the 10-Year Capital Plan (Figure 7.1). The upgrade locations are:
  1. Commercial Area in the SE (north and south of Highway 18)
  2. Central Commercial Area along Highway 44 (between Highway 18 and 106A Street)
  3. Downtown
  4. Industrial Area on the West Side (between 93 Ave, 110A St, Highway 18 & 97 Ave)

SANITARY SEWER COLLECTION SYSTEM

- MPE has assumed that the commercial/industrial generation within the existing sanitary sewer system from the 2005 Wastewater Master Plan has not changed.
- The sewage generation used for future industrial development is consistent with those values for future industrial development from other municipalities such as the Town of Whitecourt, Town of High Prairie, and Strathcona County. The sewage generation rate for future commercial development is twice the values of many other municipalities, but is consistent with the sewage generation recommended in the Alberta Environment Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems.
- With the Flow Monitoring Report being recently completed, the recommendations for sanitary sewer system improvements will be carried forward into the 10-Year Capital Plan.
- MPE cannot accurately determine the condition of the sanitary sewer mains inspected in 2008 and 2012.
- As part of the preliminary design work for the rehabilitation of the road surface of 104 Street between 110 and 111 Avenues, the Town had Sewer Infrastructure Investigation 360 inspect the sanitary sewer on that block by CCTV to determine the condition.
- Sanitary sewer mains between MH0045 and MH0300 had high water and heavy debris during flushing. These mains were not inspected by CCTV.
- The majority of sanitary sewer mains inspected by CCTV in 2018 have a structural condition rating of 4. A rating of 1 indicates the sanitary sewer is in good condition. A rating of 5 means the sanitary sewer is in poor condition.

- There are two sanitary sewer mains with structural condition ratings of 5.
- The CCTV work done by Cam-Trac totals 2.8 km of sanitary sewer main inspected and rated for condition. The total length of sanitary sewer main in the Town is 56.7 km of sanitary sewer main.
- The grease zones identified by Town staff correspond to areas with high flow and debris, in proximity to commercial areas, and act as sewer trunks.
- Based on Alberta Environment's wastewater lagoon standards and guidelines, the Town does not require any additional anaerobic cells, facultative cells, or storage cells leading into the year 2028.
- The model review and verification found that downtown Westlock, the East Business Park, and the industrial area have capacity issues. Inflow and infiltration in the industrial area is an issue where the large storm event of August 2016 caused significant inflow and infiltration into the sanitary sewer system.
- To resolve these issues, MPE developed two upgrade scenarios.
  - Scenario 1 satisfies the manhole surcharging from the west industrial park, but does not resolve all the manhole surcharging along the eastern portion of Town.
  - Scenario 2 satisfies the manhole surcharging along the eastern portion of Town, but does not resolve the pipe and manhole surcharging from the west industrial park.

STORM DRAINAGE SYSTEM

- There is no information available on the condition of the storm drainage system infrastructure.

8.2 Recommendations

WATER SUPPLY AND DISTRIBUTION SYSTEM

- The Town confirm the commercial and industrial water consumption for the existing system in their next Water Distribution Master Plan.
- The Town revisit the future commercial and industrial consumption values in their next water distribution master plan.
- The five sections of watermain with the most breaks in the last 20 years be the first priority for watermain replacement due to condition.
- Sections of mains with 2, 3 or 4 breaks in the last 10 to 20 years are recommended to be the second priority.

WATER NETWORK ANALYSIS

- The Town investigate the following further related to the fire flow testing:
  - The Town reported that the field test at the hydrant near the hospital was not completed due to concerns about past impacts on fire flow testing to hospital equipment. The Town should investigate this issue to determine if it is caused by a partially or completely closed isolation valve located nearby and/or inadequate pipe hydraulic capacity on the hospital site.
  - The field test at Test Location #2 along 109 Street indicated 41 psi lower pressure in the field compared with the model. The Town should upsize the 100 mm diameter mains in the area to 200 mm in diameter, as recommended in Section 3.5.
  - The Town confirm the pipe sizes in the areas of Test Locations #4 and #8, as increasing the pipe size in these areas duplicates the model results.
- The Town upgrade the water distribution network to meet current and future demands as outlined in Section 4.9.
- The Town conduct periodic future hydraulic reviews to evaluate the system pressure as the system grows to confirm that a single pressure zone in adequate.

SANITARY SEWER COLLECTION SYSTEM

- The Town confirm the existing commercial/industrial sewage generation numbers when the Water Master Plan is updated.
- The Town is currently developing a new Sanitary Sewer Master Plan, and the Town should review the future commercial development sewage generation as part of the Master Plan.
- MPE recommends direct replacement of sanitary sewer mains corresponding to any sanitary sewer mains inspected in 2008 and 2012 that have been identified by the public works staff as providing a poor level of service.
- MPE recommends that the section of sanitary sewer main between MH0341 and MH0342 be part of the second priority of sanitary sewer mains for rehabilitation. This rehabilitation is recommended to occur after the Town has rehabilitated the sanitary sewer mains in poor structural condition. MPE also recommends that the section of sanitary sewer main between MH0342 and MH0129 have its' condition monitored as part of the Town's ongoing sanitary sewer operation and maintenance.
- Additional flushing and cleaning of the sanitary sewer mains between MH0045 and MH0300 so that they can be inspected.

- Sanitary sewer mains receiving a structural condition rating of 5 be the first priority for structural condition rehabilitation.
- Sanitary sewer mains receiving a structural condition rating of 4 be the second priority for structural condition rehabilitation.
- The rehabilitation of the sanitary sewer main between MH0786 and MH0321 include the installation of benching in MH0321. Once the benching is installed, the sanitary sewer main between MH0321 and MH0320 be inspected again to determine its' condition.
- The Town develop a flushing program to remedy the sections of sanitary sewer main with O&M condition ratings of 5 and 4.
- The Town develop a program as part of the ongoing operation and maintenance to inspect part of the sanitary sewer system every year to assess its' condition.
- To remedy grease zones, the Town inspect the grease traps in commercial businesses to determine if they are being cleaned and are working properly.
- The Town survey the sludge build-up in the lagoons to determine the current capacities.
- To solve the capacity and inflow and infiltration issues, the Town construct a proposed outfall line from 96 Avenue and 103A Street to the sewage lagoons as shown in Figure 5.7.
- To resolve capacity issues, the Town upgrade the sanitary sewer from 106 Street and 105 Avenue to Highway 18 and 108 Avenue.
- The Town confirm the actual manhole evaluations and sanitary sewer main grades prior to construction of any sanitary sewer upgrades.
- The Town conduct flow monitoring along the section of sanitary sewer main on 100 Street between 101 Avenue and 102 Avenue to determine the actual flows in the area to determine if the modeled surcharging in the area is accurate.
- If the Town proceeds with crossing Highway 18 with a sanitary sewer main, the Town should consider a sanitary sewer main larger than 200 mm in diameter to accommodate future growth.
- The Town undertake an inflow and infiltration program to determine the sources of flow into the sanitary sewer system.
- The Town monitor newer neighbourhoods (Aspendale, Altador, Polymanth) to see how the new sanitary sewer mains perform in comparison to the older portions of the system.

- The Town implement Scenario 2 if they can accept the model results that indicate surcharging in the west industrial park.
- If the Town proceeds with upgrade Scenario 2, the Town should proceed with an inflow and infiltration program within the west industrial park to determine the sources of inflow in that area.

STORM DRAINAGE SYSTEM

- All recommended upgrades to the storm drainage system from the 2017 Stormwater Master Plan be carried forward to this report to address capacity issues within the Town's storm drainage system. These upgrades are:
  - Priority 3 – Future Development in Westgate and Greenfield Estates
  - Priority 4 – Stormwater System Preventative Maintenance
  - Priority 5 – Pond G
  - Priority 6 – Overland Drainage in Southview
  - Priority 7 – General Surcharging During 1:5 Year Storm Events
- The Town develop a program to inspect the storm sewer system by CCTV, and visually inspect all storm drainage channels, stormwater management facilities, culverts, inlets and outfalls to determine their condition.
- The Town implement a preventative maintenance program as described in the Stormwater Master Plan as part of the ongoing operation and maintenance of the storm sewer system. The Town should include the budget for this program as part of their operating budget.

9.0 CLOSURE

MPE Engineering Ltd. has prepared and finalized this Municipal Infrastructure Assessment and 10-Year Capital Projection with input from the Town of Westlock. MPE thanks the Town for the opportunity to provide this Infrastructure Assessment and 10-Year Capital Plan. The Town is encouraged to develop a projection implementation plan to manage priorities to keep infrastructure in good condition and to retain the integrity of the overall system. It is recommend that the Town identify possible government funding sources and programs in budget deliberations to determine which projects are feasible.

The preparation of this project was carried out with assistance from the Government of Canada and the Federation of Canadian Municipalities. Notwithstanding this support, the views expressed are the personal views of the authors, and the Federation of Canadian Municipalities and the Government of Canada accept no responsibility for them.

APPENDIX A

Fire Hydrant Field Tests

Suggested Content for Hydrant Test Form for Water Model Calibration

Hydrant Test Site Number:	# 1
Hydrant Test Site Location Description (Street Names):	113 ave : 99 st.
Is the water main isolated with flow in one direction only?	Yes / No

	Units	Closed Hydrant No. 1 (Farthest from Flowed Hydrant)	Closed Hydrant No. 2 (Closest to Flowed Hydrant)	Hydrant No. 3 to be Flowed	Hydrant No. 4 to be Flowed (if applicable)	Hydrant No. 5 to be Flowed (if applicable)
Fire Hydrant Tag No.	n/a	90	100	102.		
Hydrant Elevation (if available)	m					
Static Pressure (No Hydrant Flow)	kPa	350	400			
Dynamic Pressure (Hydrant Flow)	kPa	250	300			
Hydrant Flowrate	L/s			50.79		

Notes:

← = 160 kpa
≈ 23 psi

1. *A pressure drop of 15 to 20 psi should be obtained between Hydrant No. 1 and No. 2 where possible.*
2. *Minimum system pressure to maintain at least 20 psi during hydrant testing.*

Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15-20 psi (100 - 140 kPa) should be attained.

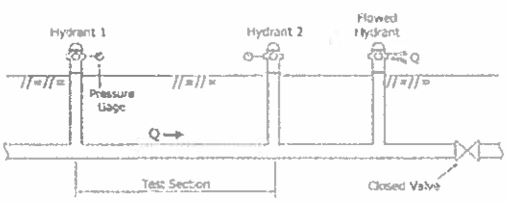


Figure 5.9
The two-gage head loss test

$$h_L = HGL_U - HGL_D \tag{5.5}$$

where HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

$$h_L = C_f(P_U - P_D) + (Z_U - Z_D) \tag{5.6}$$

where P_U = pressure at upstream fire hydrant (psi, kPa)
P_D = pressure at downstream fire hydrant (psi, kPa)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_D = elevation at downstream fire hydrant (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow; therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condi-

Westlock Fire Department
10003 - 106 Street
Westlock, Alberta T7P 2K3
780-349-4444 Phone
780-349-3346 Fax

Water Flow Test Report

LOCATION/HYDRANT #	#1 (Hydrant # 102) 113 avenue & 99 Street			DATE:	18/05/2019
TEST BY:	S. Koflick		TIME OF DAY:	1400hrs	
WATER SUPPLIED BY:	Town of Westlock				
PURPOSE OF TEST:	Water Model Calibration				

DATA

FLOW HYDRANT(S)		A1	A2	A3
	SIZE OPENING:	2.5	2.5	2.5
	COEFFICIENT:	0.9	0.9	0.9
	PITOT READING:	23	0	0
	GPM:	805	0	0

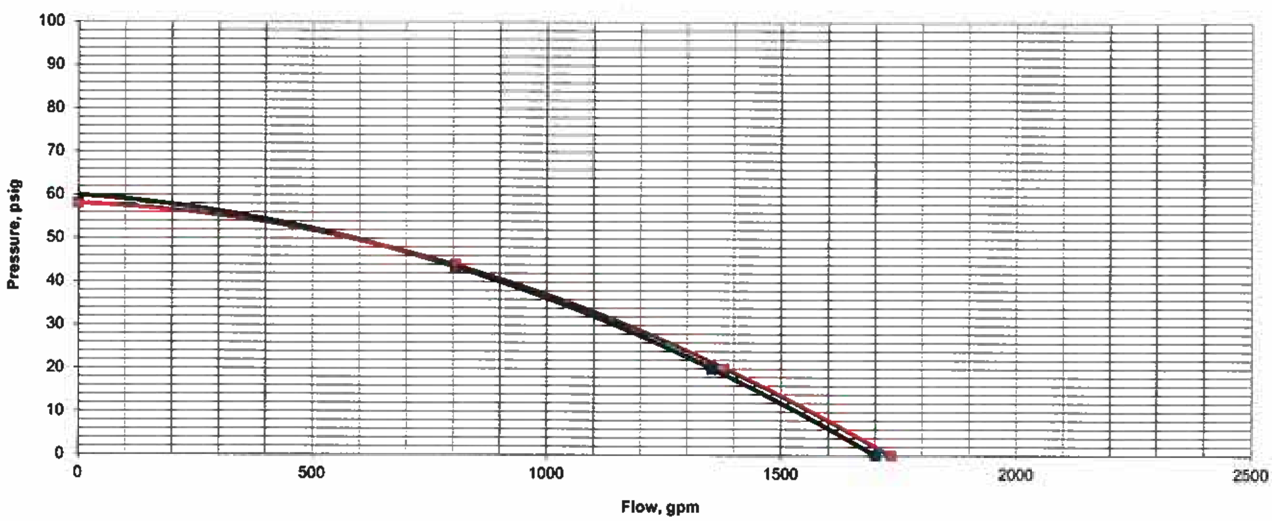
TOTAL FLOW DURING TEST: 805 GPM

TOWER LEVEL @ TIME OF TEST: | TOWER LOW 0

STATIC READING: 58 PSI | RESIDUAL: 43.5 PSI

ADJ. STATIC:	58 PSI		**AJD. RESIDUAL:**	44 PSI
ADJ. FLOW:	**AT 20 PSI RESIDUAL**	1380 GPM	**AT 0 PSI**	1734 GPM

This flow test hyas been conducted in compliance with ***NFPA 291: Recommneded Practice for Fire Flow Testing and Marking of Hydrants***.The Town of Westlock makes no representation or warranties regarding the information contained in this document, including without limitation, whether said information is accurate or complete. Persons using this document do so solely at their own risk, and the Town of Westlock shall have no liability to such person for any loss or damage whatsoever. This document shall not be copied or distributed to any person without the express written consent of the Town of Westlock.



Suggested Content for Hydrant Test Form for Water Model Calibration

Hydrant Test Site Number:	#2
Hydrant Test Site Location Description (Street Names):	97 ave : 109st.
Is the water main isolated with flow in one direction only?	Yes / No

	Units	Closed Hydrant No. 1 (Farthest from Flowed Hydrant)	Closed Hydrant No. 2 (Closest to Flowed Hydrant)	Hydrant No. 3 to be Flowed	Hydrant No. 4 to be Flowed (if applicable)	Hydrant No. 5 to be Flowed (if applicable)
Fire Hydrant Tag No.	n/a		27	26		
Hydrant Elevation (if available)	m					
Static Pressure (No Hydrant Flow)	kPa		500			
Dynamic Pressure (Hydrant Flow)	kPa		200			
Hydrant Flowrate	L/s			54.01		

Notes:

1. *A pressure drop of 15 to 20 psi should be obtained between Hydrant No. 1 and No. 2 where possible.* 180 kpa.
2. *Minimum system pressure to maintain at least 20 psi during hydrant testing.*

Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15 20 psi (100 - 140 kPa) should be attained.

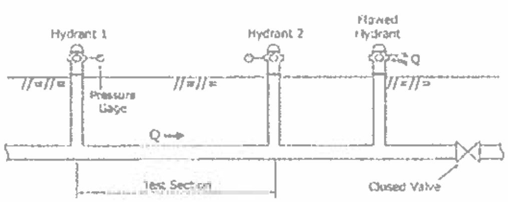


Figure 5.9
The two-gage head loss test

$$h_L = HGL_U - HGL_D \qquad (5.5)$$

where HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

$$h_L = C_f(P_U - P_D) + (Z_U - Z_D) \qquad (5.6)$$

where P_U = pressure at upstream fire hydrant (psi, kPa)
P_D = pressure at downstream fire hydrant (psi, kPa)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_D = elevation at downstream fire-hydrant (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow, therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condi-

Westlock Fire Department

10003 - 106 Street
Westlock, Alberta T7P 2K3
780-349-4444 Phone
780-349-3346 Fax

Water Flow Test Report

LOCATION/HYDRANT #	#2 (Hydrant # 26) 97 ave & 109 st	DATE:	18/05/2019
TEST BY:	S. Koflick	TIME OF DAY:	1000hrs
WATER SUPPLIED BY:	Town of Westlock		
PURPOSE OF TEST:	Water Model Calibration		

DATA

FLOW HYDRANT(S)		A1	A2	A3
	SIZE OPENING:	2.5	2.5	2.5
	COEFFICIENT:	0.9	0.9	0.9
	PITOT READING:	26	0	0
	GPM:	856	0	0

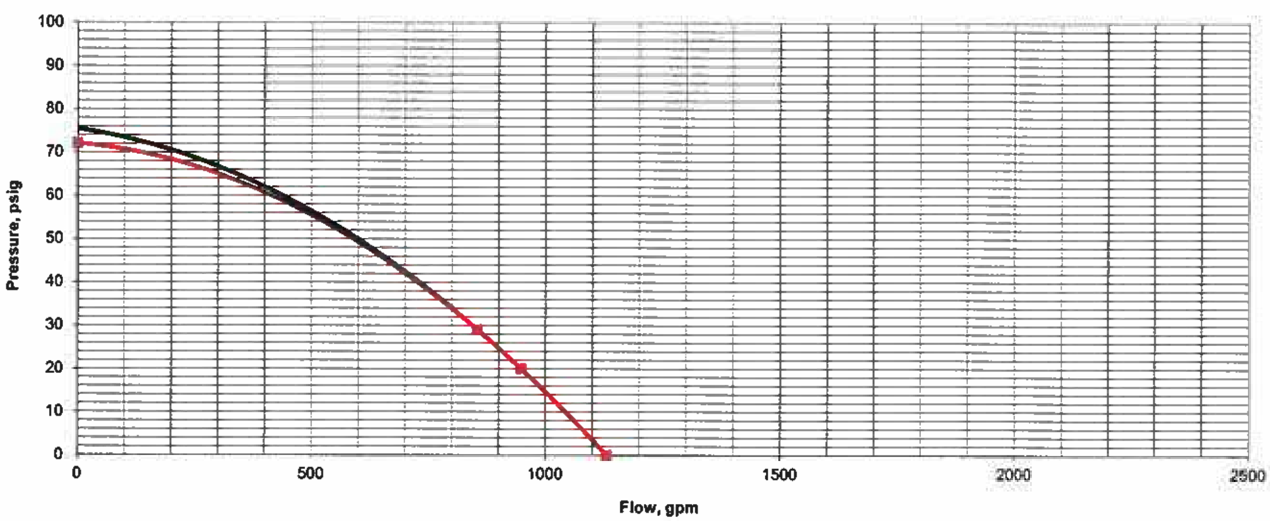
TOTAL FLOW DURING TEST: 856 GPM

TOWER LEVEL @ TIME OF TEST: | TOWER LOW: 0

STATIC READING: 72 PSI | RESIDUAL: 29 PSI

ADJ. STATIC: 72 **PSI** | **AJD. RESIDUAL:** 29 **PSI**

ADJ. FLOW: **AT 20 PSI RESIDUAL** 948 **GPM** | **AT 0 PSI** 1130 **GPM**



Town of Westlock
Infrastructure Assessment
July 16, 2018

Prepared by MPE Engineering Ltd.

Suggested Content for Hydrant Test Form for Water Model Calibration

Hydrant Test Site Number:	3
Hydrant Test Site Location Description (Street Names):	104 st & 103 ave.
Is the water main isolated with flow in one direction only?	Yes / No

	Units	Closed Hydrant No. 1 (Farthest from Flowed Hydrant)	Closed Hydrant No. 2 (Closest to Flowed Hydrant)	Hydrant No. 3 to be Flowed	Hydrant No. 4 to be Flowed (if applicable)	Hydrant No. 5 to be Flowed (if applicable)
Fire Hydrant Tag No.	n/a		#39	#38		
Hydrant Elevation (if available)	m					
Static Pressure (No Hydrant Flow)	kPa		400			
Dynamic Pressure (Hydrant Flow)	kPa		350			
Hydrant Flowrate	L/s			62.46		

240kpa

Notes:

1. *A pressure drop of 15 to 20 psi should be obtained between Hydrant No. 1 and No. 2 where possible.*
2. *Minimum system pressure to maintain at least 20 psi during hydrant testing.*

Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15-20 psi (100 - 140 kPa) should be attained.

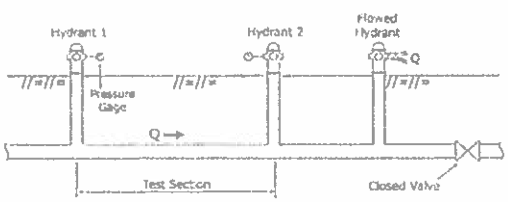


Figure 5.9
The two-gage head loss test

$$h_L = HGL_U - HGL_D \quad (5.5)$$

where HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

$$h_L = C_f(P_U - P_D) + (Z_U - Z_D) \quad (5.6)$$

where P_U = pressure at upstream fire hydrant (psi, kPa)
P_D = pressure at downstream fire hydrant (psi, kPa)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_D = elevation at downstream fire-hydrant (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow; therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condi-

Westlock Fire Department
10003 - 106 Street
Westlock, Alberta T7P 2K3
780-349-4444 Phone
780-349-3346 Fax

Water Flow Test Report

LOCATION/HYDRANT #	#3 (Hydrant # 38) 104 st & 103 ave.	DATE:	18/05/2019
TEST BY:	S. Koflick	TIME OF DAY:	1030hrs
WATER SUPPLIED BY:	Town of Westlock		
PURPOSE OF TEST:	Water Model Calibration		

DATA

FLOW HYDRANT(S)		A1	A2	A3
	SIZE OPENING:	2.5	2.5	2.5
	COEFFICIENT:	0.9	0.9	0.9
	PITOT READING:	34.8	0	0
	GPM:	990	0	0
TOTAL FLOW DURING TEST:		990 GPM		

TOWER LEVEL @ TIME OF TEST:		TOWER LOW	0
STATIC READING:	58 PSI	RESIDUAL:	50.7 PSI
ADJ. STATIC:	**58 PSI**	**AJD. RESIDUAL:**	**51 PSI**
ADJ. FLOW:	**AT 20 PSI RESIDUAL 2468 GPM**	**AT 0 PSI**	**3101 GPM**

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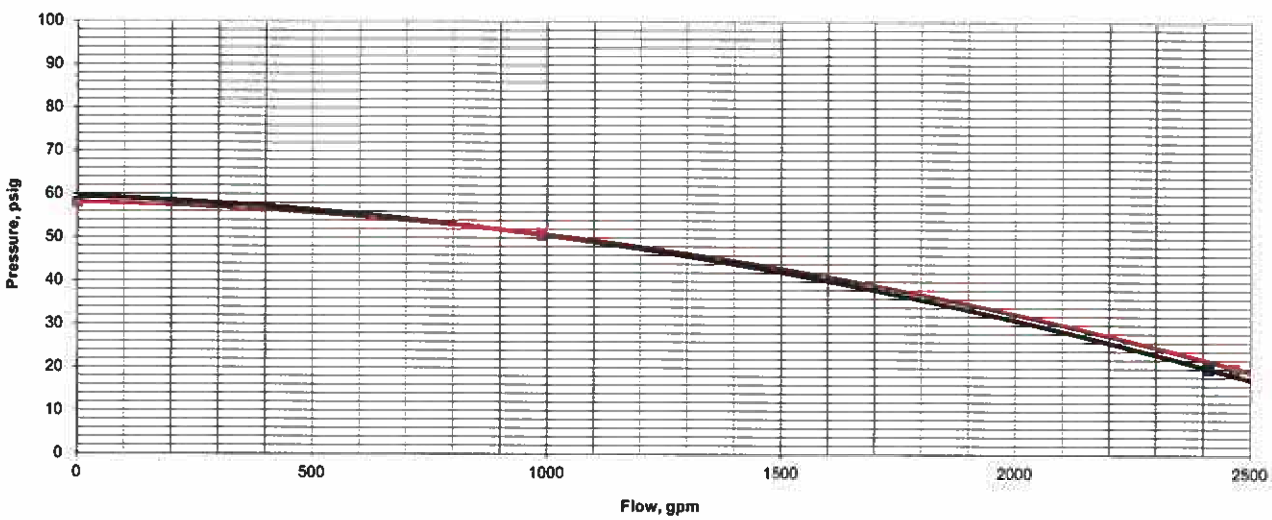

Town of Westlock
Infrastructure Assessment
July 16, 2018

Prepared by MPE Engineering Ltd.

Suggested Content for Hydrant Test Form for Water Model Calibration

Hydrant Test Site Number:	#4
Hydrant Test Site Location Description (Street Names):	100 St (Private Hydrant).
Is the water main isolated with flow in one direction only?	Yes / No

	Units	Closed Hydrant No. 1 (Farthest from Flowed Hydrant)	Closed Hydrant No. 2 (Closest to Flowed Hydrant)	Hydrant No. 3 to be Flowed	Hydrant No. 4 to be Flowed (if applicable)	Hydrant No. 5 to be Flowed (if applicable)
Fire Hydrant Tag No.	n/a		130	129.		
Hydrant Elevation (if available)	m					
Static Pressure (No Hydrant Flow)	kPa		420			
Dynamic Pressure (Hydrant Flow)	kPa		200			
Hydrant Flowrate	L/s			44.16		

120 kPa
= 17.4 psi

Notes:

1. A pressure drop of 15 to 20 psi should be obtained between Hydrant No. 1 and No. 2 where possible.

2. Minimum system pressure to maintain at least 20 psi during hydrant testing.

Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15–20 psi (100 – 140 kPa) should be attained.

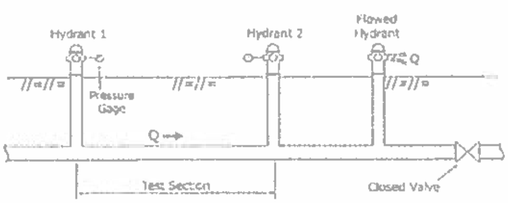

Figure 5.9
The two-gage head loss test

$$h_L = HGL_U - HGL_D \qquad (5.5)$$

where HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

$$h_L = C_f(P_U - P_D) + (Z_U - Z_D) \qquad (5.6)$$

where P_U = pressure at upstream fire hydrant (psi, kPa)
P_D = pressure at downstream fire hydrant (psi, kPa)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_D = elevation at downstream fire hydrant (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow, therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condi-

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Water Flow Test Report

LOCATION/HYDRANT #	#4 (Hydrant #129) Rocky Mountain Equipment Yard		DATE:	18/05/2019
TEST BY:	S. Koflick		TIME OF DAY:	1330hrs
WATER SUPPLIED BY:	Town of Westlock			
PURPOSE OF TEST:	Water Model Calibration			

DATA

FLOW HYDRANT(S)	A1	A2	A3
SIZE OPENING:	2.5	2.5	2.5
COEFFICIENT:	0.9	0.9	0.9
PITOT READING:	17.4	0	0
GPM:	700	0	0

TOTAL FLOW DURING TEST: 700 GPM

TOWER LEVEL @ TIME OF TEST: | TOWER LOW 0

STATIC READING: 60.9 PSI | RESIDUAL: 29 PSI

ADJ. STATIC:	**61 PSI**	**AJD. RESIDUAL:**	**29 PSI**
ADJ. FLOW:	**AT 20 PSI RESIDUAL 800 GPM**	**AT 0 PSI**	**992 GPM**

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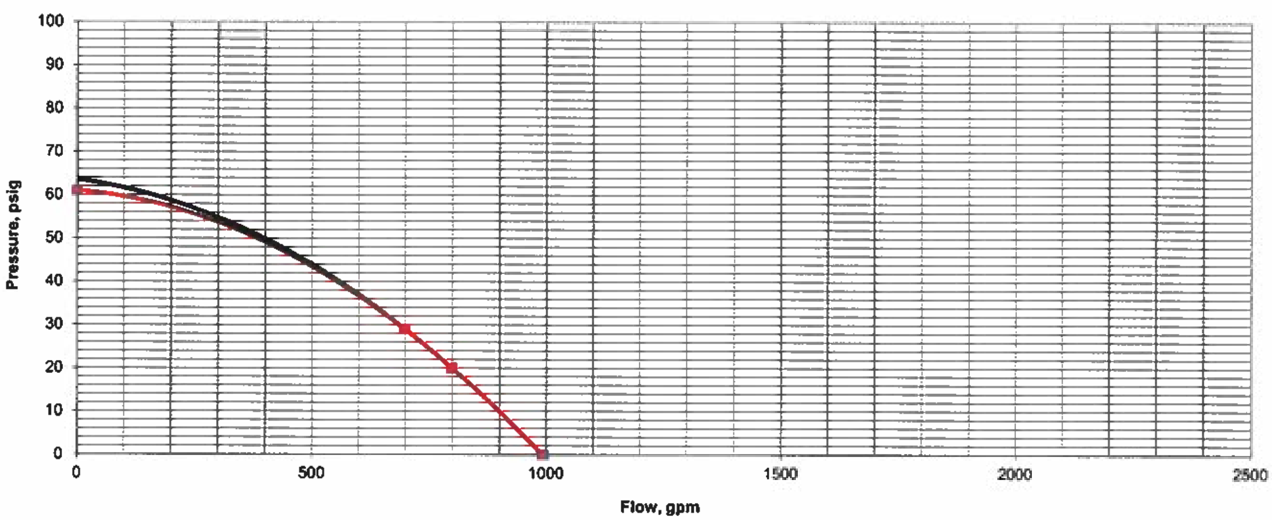

Town of Westlock
Infrastructure Assessment
July 16, 2018

Prepared by MPE Engineering Ltd.

Suggested Content for Hydrant Test Form for Water Model Calibration

Hydrant Test Site Number:	#5
Hydrant Test Site Location Description (Street Names):	105 st & 100 st.
Is the water main isolated with flow in one direction only?	Yes / No

	Units	Closed Hydrant No. 1 (Farthest from Flowed Hydrant)	Closed Hydrant No. 2 (Closest to Flowed Hydrant)	Hydrant No. 3 to be Flowed	Hydrant No. 4 to be Flowed (if applicable)	Hydrant No. 5 to be Flowed (if applicable)
Fire Hydrant Tag No.	n/a	#86	#81	#80		
Hydrant Elevation (if available)	m					
Static Pressure (No Hydrant Flow)	kPa	500	500			
Dynamic Pressure (Hydrant Flow)	kPa	350	300			
Hydrant Flowrate	L/s			44.16		

Notes:

L 200 kPa.

1. *A pressure drop of 15 to 20 psi should be obtained between Hydrant No. 1 and No. 2 where possible.*

= 29 psi

2. *Minimum system pressure to maintain at least 20 psi during hydrant testing.*

Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15–20 psi (100 – 140 kPa) should be attained.

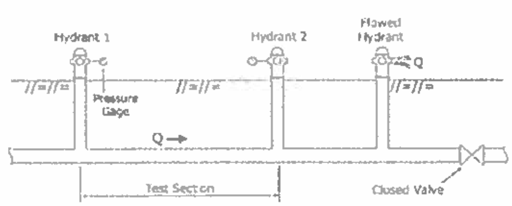

Figure 5.9 The two-gage head loss test

$$h_L = HGL_U - HGL_D \qquad (5.5)$$

where HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

$$h_L = C_f(P_U - P_D) + (Z_U - Z_D) \qquad (5.6)$$

where P_U = pressure at upstream fire hydrant (psi, kPa)
P_D = pressure at downstream fire hydrant (psi, kPa)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_D = elevation at downstream fire-hydrant (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow; therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condi-

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Water Flow Test Report

LOCATION/HYDRANT #	#5 (Hydrant #80) 105 st. & 100 St.		DATE:	18/05/2019
TEST BY:	S. Koflick		TIME OF DAY:	12:00 AM
WATER SUPPLIED BY:	Town of Westlock			
PURPOSE OF TEST:	Water Model Calibration			

DATA

FLOW HYDRANT(S)	A1	A2	A3
SIZE OPENING:	2.5	2.5	2.5
COEFFICIENT:	0.9	0.9	0.9
PITOT READING:	29	0	0
GPM:	904	0	0

TOTAL FLOW DURING TEST: 904 GPM

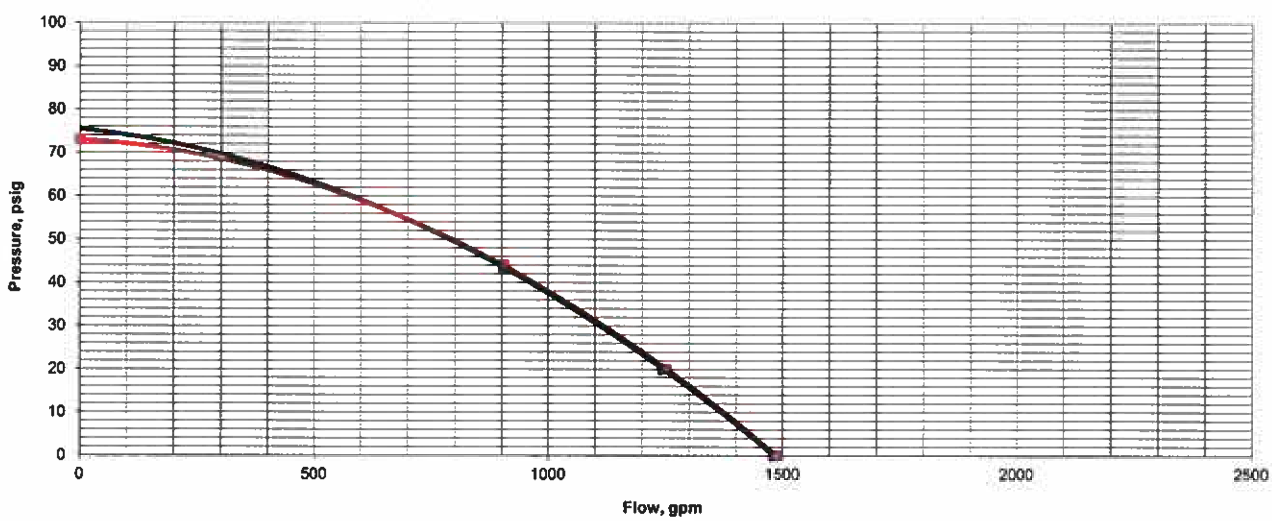
TOWER LEVEL @ TIME OF TEST:

STATIC READING: 72.51 PSI

TOWER LOW RESIDUAL: 0 43.5 PSI

ADJ. STATIC: 73 PSI **AJD. RESIDUAL: 44 PSI**

ADJ. FLOW: AT 20 PSI RESIDUAL 1251 GPM **AT 0 PSI 1488 GPM**



Town of Westlock
Infrastructure Assessment
July 16, 2018

Prepared by MPE Engineering Ltd.

Suggested Content for Hydrant Test Form for Water Model Calibration

Hydrant Test Site Number:	#6
Hydrant Test Site Location Description (Street Names):	100st - Spirit Center SE corner
Is the water main isolated with flow in one direction only?	Yes / No

	Units	Closed Hydrant No. 1 (Farthest from Flowed Hydrant)	Closed Hydrant No. 2 (Closest to Flowed Hydrant)	Hydrant No. 3 to be Flowed	Hydrant No. 4 to be Flowed (if applicable)	Hydrant No. 5 to be Flowed (if applicable)
Fire Hydrant Tag No.	n/a	105	106	104		
Hydrant Elevation (if available)	m					
Static Pressure (No Hydrant Flow)	kPa	450	450			
Dynamic Pressure (Hydrant Flow)	kPa	300	300			
Hydrant Flowrate	L/s			64.98		

Notes:

L 260 kpa

1. *A pressure drop of 15 to 20 psi should be obtained between Hydrant No. 1 and No. 2 where possible.*
2. *Minimum system pressure to maintain at least 20 psi during hydrant testing.*

= 37.7 psi

Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15–20 psi (100 - 140 kPa) should be attained.

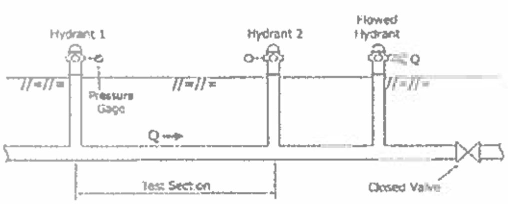


Figure 5.9
The two-gage head loss test

$$h_L = HGL_U - HGL_D \tag{5.5}$$

where HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

$$h_L = C_f(P_U - P_D) + (Z_U - Z_D) \tag{5.6}$$

where P_U = pressure at upstream fire hydrant (psi, kPa)
P_D = pressure at downstream fire hydrant (psi, kPa)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_D = elevation at downstream fire hydrant (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow; therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condi-

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Water Flow Test Report

LOCATION/HYDRANT #: #6 (Hydrant #104) Spirit Center Parking Lot SE corner — DATE: 18/05/2019

TEST BY: S. Koflick — TIME OF DAY: 1015hrs

WATER SUPPLIED BY: Town of Westlock

PURPOSE OF TEST: Water Model Calibration

DATA

FLOW HYDRANT(S)

	A1	A2	A3
SIZE OPENING:	2.5	2.5	2.5
COEFFICIENT:	0.9	0.9	0.9
PITOT READING:	37.7	0	0
GPM:	1030	0	0

TOTAL FLOW DURING TEST: 1030 GPM

TOWER LEVEL @ TIME OF TEST: — TOWER LOW: 0

STATIC READING: 65.2 PSI — RESIDUAL: 43.5 PSI

ADJ. STATIC: 66 **PSI** — **AJD. RESIDUAL:** 44 **PSI**

ADJ. FLOW: **AT 20 PSI RESIDUAL** 1534 **GPM** — **AT 0 PSI** 1865 **GPM**

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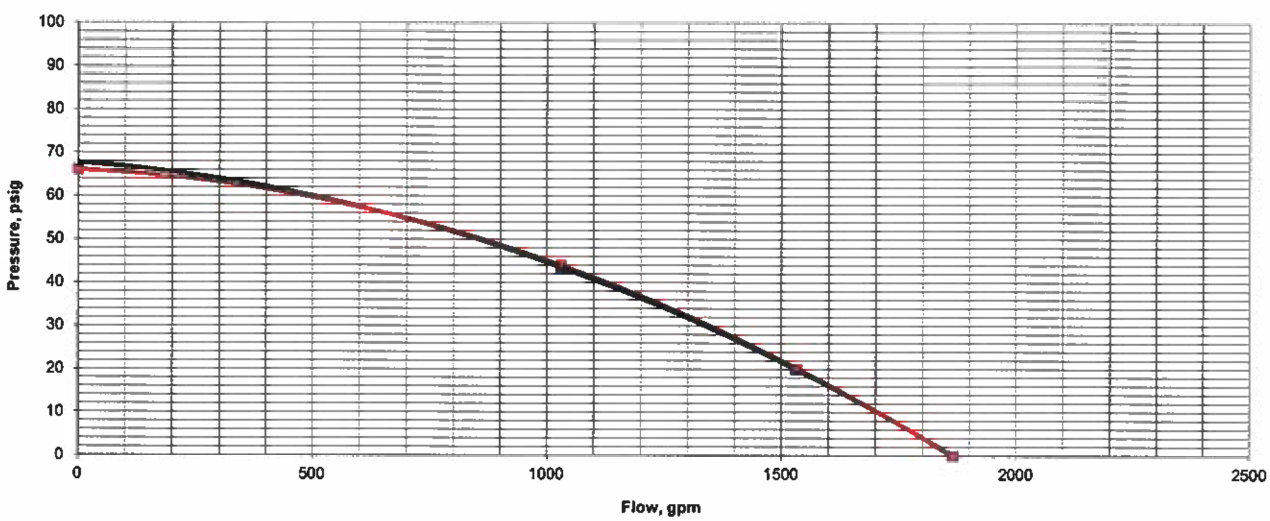

Town of Westlock
Infrastructure Assessment
July 16, 2018

Prepared by MPE Engineering Ltd.

Suggested Content for Hydrant Test Form for Water Model Calibration

Hydrant Test Site Number:	17
Hydrant Test Site Location Description (Street Names):	107 st : 108 ave.
Is the water main isolated with flow in one direction only?	*Yes / No*

	Units	Closed Hydrant No. 1 (Farthest from Flowed Hydrant)	Closed Hydrant No. 2 (Closest to Flowed Hydrant)	Hydrant No. 3 to be Flowed	Hydrant No. 4 to be Flowed (if applicable)	Hydrant No. 5 to be Flowed (if applicable)
Fire Hydrant Tag No.	n/a	127	128	073		
Hydrant Elevation (if available)	m					
Static Pressure (No Hydrant Flow)	kPa	400	500			
Dynamic Pressure (Hydrant Flow)	kPa	380	400			
Hydrant Flowrate	L/s			55.52		

240kpa
= 34.8 psi.

Notes:

1. A pressure drop of 15 to 20 psi should be obtained between Hydrant No. 1 and No. 2 where possible.

2. Minimum system pressure to maintain at least 20 psi during hydrant testing.

Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15-20 psi (100 - 140 kPa) should be attained.

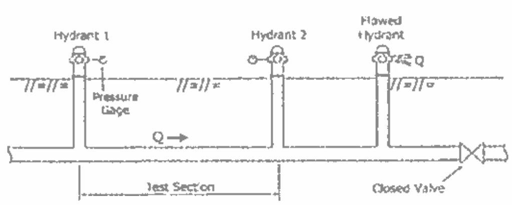

Figure 5.9 The two-gage head loss test

$$h_L = HGL_U - HGL_D \qquad (5.5)$$

where HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

$$h_L = C_f(P_U - P_D) + (Z_U - Z_D) \qquad (5.6)$$

where P_U = pressure at upstream fire hydrant (psi, kPa)
P_D = pressure at downstream fire hydrant (psi, kPa)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_D = elevation at downstream fire-hydrant (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow; therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condi-

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Water Flow Test Report

LOCATION/HYDRANT #	#7 (Hydrant # 073) 107 St & 108 Ave.	DATE:	18/05/2019
TEST BY:	S. Koflick	TIME OF DAY:	1100hrs
WATER SUPPLIED BY:	Town of Westlock		
PURPOSE OF TEST:	Water Model Calibration		

DATA

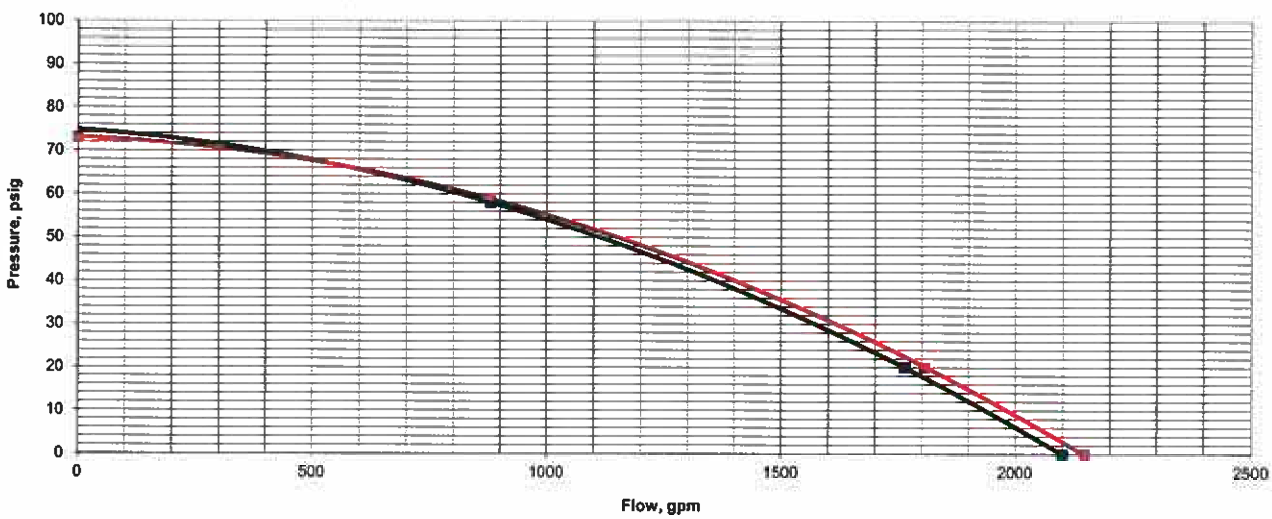
FLOW HYDRANT(S)

	A1	A2	A3
SIZE OPENING:	2.5	2.5	2.5
COEFFICIENT:	0.8	0.9	0.9
PITOT READING:	34.8	0	0
GPM:	880	0	0

TOTAL FLOW DURING TEST: 880 GPM

TOWER LEVEL @ TIME OF TEST:		TOWER LOW	0
STATIC READING:	72.51 PSI	RESIDUAL:	58.01 PSI

ADJ. STATIC:	73 PSI	**AJD. RESIDUAL:**	59 PSI
ADJ. FLOW:	**AT 20 PSI RESIDUAL** 1806 **GPM**	**AT 0 PSI**	2146 **GPM**



Town of Westlock
Infrastructure Assessment
July 16, 2018

Prepared by MPE Engineering Ltd.

Suggested Content for Hydrant Test Form for Water Model Calibration

Hydrant Test Site Number:	E8
Hydrant Test Site Location Description (Street Names):	93 ave : 113A st.
Is the water main isolated with flow in one direction only?	Yes / No

	Units	Closed Hydrant No. 1 (Farthest from Flowed Hydrant)	Closed Hydrant No. 2 (Closest to Flowed Hydrant)	Hydrant No. 3 to be Flowed	Hydrant No. 4 to be Flowed (if applicable)	Hydrant No. 5 to be Flowed (if applicable)
Fire Hydrant Tag No.	n/a	005	007	008		
Hydrant Elevation (if available)	m					
Static Pressure (No Hydrant Flow)	kPa	550	580			
Dynamic Pressure (Hydrant Flow)	kPa	400	500			
Hydrant Flowrate	L/s			82.65 L		

Notes:

1. A pressure drop of 15 to 20 psi should be obtained between Hydrant No. 1 and No. 2 where possible.

2. Minimum system pressure to maintain at least 20 psi during hydrant testing.

420 kpa.
= 60.91 psi.

Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15-20 psi (100 - 140 kPa) should be attained.

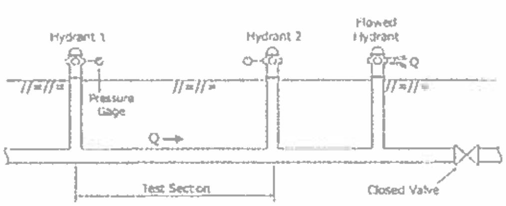


Figure 5.9 The two-gage head loss test

$$h_L = HGL_U - HGL_D \qquad (5.5)$$

where HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

$$h_L = C_f(P_U - P_D) + (Z_U - Z_D) \qquad (5.6)$$

where P_U = pressure at upstream fire hydrant (psi, kPa)
P_D = pressure at downstream fire hydrant (psi, kPa)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_D = elevation at downstream fire hydrant (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow; therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condi-

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Water Flow Test Report

LOCATION/HYDRANT #	#8 (Hydrant # 008) 93 Ave & 113A St			DATE:	18/05/2019
TEST BY:	S. Koflick		TIME OF DAY:	0900hrs	
WATER SUPPLIED BY:	Town of Westlock				
PURPOSE OF TEST:	Water Model Calibration				

DATA

FLOW HYDRANT(S)		A1	A2	A3
	SIZE OPENING:	2.5	2.5	2.5
	COEFFICIENT:	0.9	0.9	0.9
	PITOT READING:	60.91	0	0
	GPM:	1310	0	0

TOTAL FLOW DURING TEST: 1310 GPM

TOWER LEVEL @ TIME OF TEST: TOWER LOW 0

STATIC READING: 84.12 PSI RESIDUAL: 72.5 PSI

ADJ. STATIC: 85 **PSI** **AJD. RESIDUAL:** 73 **PSI**

ADJ. FLOW: **AT 20 PSI RESIDUAL** 3261 **GPM** **AT 0 PSI** 3769 **GPM**

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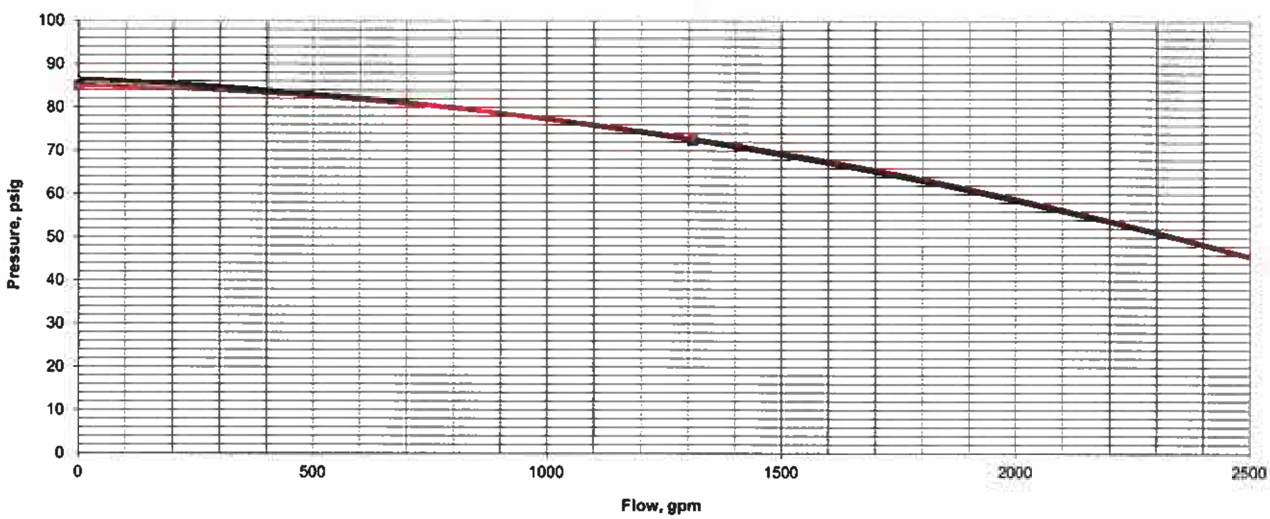

Town of Westlock
Infrastructure Assessment
July 16, 2018

Prepared by MPE Engineering Ltd.

Suggested Content for Hydrant Test Form for Water Model Calibration

Hydrant Test Site Number:	#9
Hydrant Test Site Location Description (Street Names):	102 ave : 99 st.
Is the water main isolated with flow in one direction only?	Yes / No

	Units	Closed Hydrant No. 1 (Farthest from Flowed Hydrant)	Closed Hydrant No. 2 (Closest to Flowed Hydrant)	Hydrant No. 3 to be Flowed	Hydrant No. 4 to be Flowed (if applicable)	Hydrant No. 5 to be Flowed (if applicable)
Fire Hydrant Tag No.	n/a	175.	177	160		
Hydrant Elevation (if available)	m					
Static Pressure (No Hydrant Flow)	kPa	400	470			
Dynamic Pressure (Hydrant Flow)	kPa	350	350			
Hydrant Flowrate	L/s			62.46 L		

= 240 kpa
= 34.80 psi

Notes:

1. *A pressure drop of 15 to 20 psi should be obtained between Hydrant No. 1 and No. 2 where possible.*
2. *Minimum system pressure to maintain at least 20 psi during hydrant testing.*

Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15–20 psi (100 – 140 kPa) should be attained.

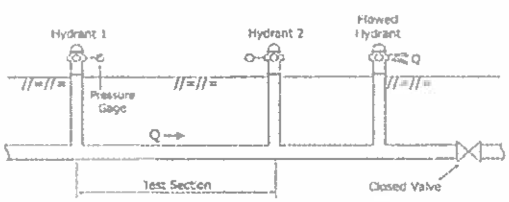

Figure 5.9 The two-gage head loss test

$$h_L = HGL_U - HGL_D \qquad (5.5)$$

where HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

$$h_L = C_f(P_U - P_D) + (Z_U - Z_D) \qquad (5.6)$$

where P_U = pressure at upstream fire hydrant (psi, kPa)
P_D = pressure at downstream fire hydrant (psi, kPa)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_D = elevation at downstream fire hydrant (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow; therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condi-

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Water Flow Test Report

LOCATION/HYDRANT #: #9 (Hydrant #160) 102 Ave & 97 St — DATE: 18/05/2019

TEST BY: S. Koflick — TIME OF DAY: 1430hrs

WATER SUPPLIED BY: Town of Westlock

PURPOSE OF TEST: Water Model Calibration

DATA

FLOW HYDRANT(S)	A1	A2	A3
SIZE OPENING:	2.5	2.5	2.5
COEFFICIENT:	0.9	0.9	0.9
PITOT READING:	34.809	0	0
GPM:	990	0	0

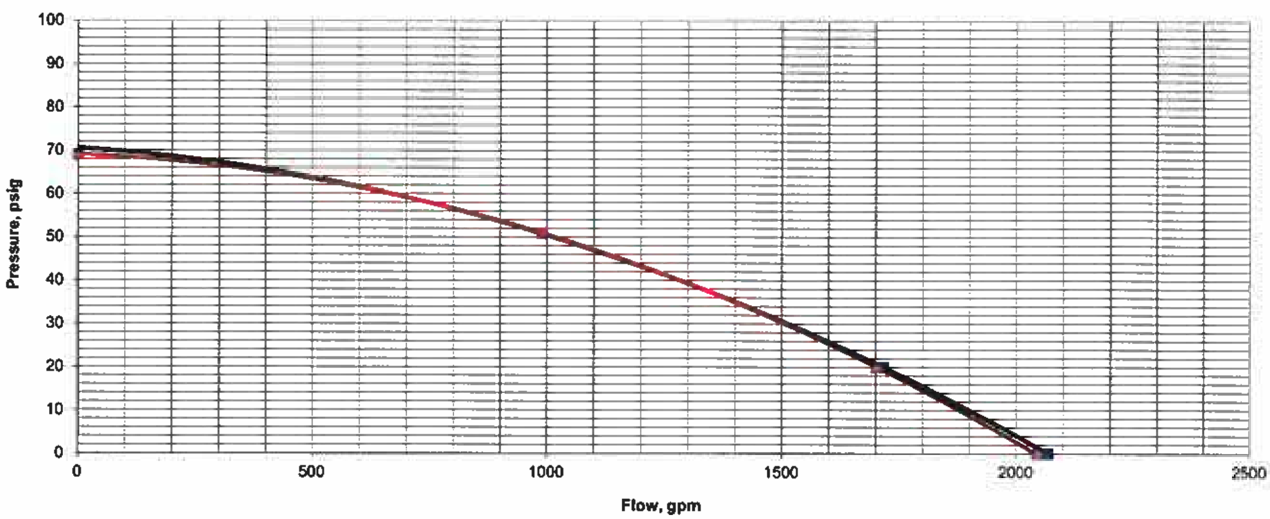
TOTAL FLOW DURING TEST: 990 GPM

TOWER LEVEL @ TIME OF TEST: — TOWER LOW: 0

STATIC READING: 68.167 PSI — RESIDUAL: 50.7632 PSI

ADJ. STATIC: 69 **PSI** — **AJD. RESIDUAL:** 51 **PSI**

ADJ. FLOW: **AT 20 PSI RESIDUAL** 1700 **GPM** — **AT 0 PSI** 2045 **GPM**



Town of Westlock
Infrastructure Assessment
July 16, 2018

Prepared by MPE Engineering Ltd.

Suggested Content for Hydrant Test Form for Water Model Calibration

Hydrant Test Site Number:	11
Hydrant Test Site Location Description (Street Names):	104 st. ; 111 ave.
Is the water main isolated with flow in one direction only?	*Yes* / *No*

	Units	Closed Hydrant No. 1 (Farthest from Flowed Hydrant)	Closed Hydrant No. 2 (Closest to Flowed Hydrant)	Hydrant No. 3 to be Flowed	Hydrant No. 4 to be Flowed (if applicable)	Hydrant No. 5 to be Flowed (if applicable)
Fire Hydrant Tag No.	n/a	088	089	090		
Hydrant Elevation (if available)	m					
Static Pressure (No Hydrant Flow)	kPa	450	450			
Dynamic Pressure (Hydrant Flow)	kPa	350	350			
Hydrant Flowrate	L/s			50.98.		

Notes:

160 kpa
= 23.2 psi

1. A pressure drop of 15 to 20 psi should be obtained between Hydrant No. 1 and No. 2 where possible.

2. Minimum system pressure to maintain at least 20 psi during hydrant testing.

Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15-20 psi (100 - 140 kPa) should be attained.

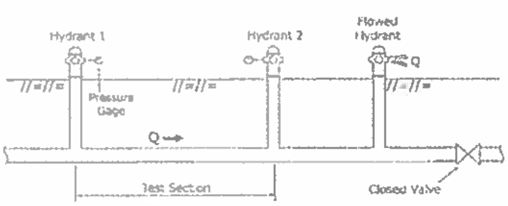


Figure 5.9
The two-gage head loss test

$$h_L = HGL_U - HGL_D \qquad (5.5)$$

where HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

$$h_L = C_f(P_U - P_D) + (Z_U - Z_D) \qquad (5.6)$$

where P_U = pressure at upstream fire hydrant (psi, kPa)
P_D = pressure at downstream fire hydrant (psi, kPa)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_D = elevation at downstream fire hydrant (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow; therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condi-

Westlock Fire Department
10003 - 106 Street
Westlock, Alberta T7P 2K3
780-349-4444 Phone
780-349-3346 Fax

Water Flow Test Report

LOCATION/HYDRANT # #11 (Hydrant #090) 104 street & 111 ave   DATE: 18/05/2019
TEST BY: S. Koflick   TIME OF DAY: 1330hrs
WATER SUPPLIED BY: Town of Westlock
PURPOSE OF TEST: Water Model Calibration

DATA

FLOW HYDRANT(S)

	A1	A2	A3
SIZE OPENING:	2.5	2.5	2.5
COEFFICIENT:	0.9	0.9	0.9
PITOT READING:	23.206	0	0
GPM:	808	0	0

TOTAL FLOW DURING TEST: 808 GPM

TOWER LEVEL @ TIME OF TEST: TOWER LOW 0
STATIC READING: 65.267 PSI   RESIDUAL: 50.7632 PSI

ADJ. STATIC: 66 **PSI**   **AJD. RESIDUAL:** 51 **PSI**

ADJ. FLOW: **AT 20 PSI RESIDUAL** 1480 **GPM**   **AT 0 PSI** 1799 **GPM**