Existing Municipal Servicing Plan Update Report

Town of Edson, Alberta

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Executive Summary

The Town of Edson retained GHD to update the existing 2011 municipal servicing plan (MSP) and to update the 1982 transportation master plan and roadway network. This updated MSP report presents the existing and future conditions and the associated required upgrades for the water supply and distribution system, the wastewater collection system, the stormwater management system and transportation and roadway network system.

The study area for existing and future condition includes areas within the Town's boundary, as well as proposed development areas within the fringe area in Yellowhead County.

Future upgrades in this MSP report focused on two time horizons of 15 and 25 years for the years 2032 and 2042, respectively, while projecting a population increase of 1 percent for the Town of Edson and 0.7 percent for Yellowhead County (fringe area of Town). This population projection was based on the town's 2016 Municipal Development Plan (MDP) as well as the actual historical population increase. Using these population projection rates, the estimated population for the Town in 2032 would be 9,867 and 10,902 in 2042, and 12,157 for Yellowhead County in 2032 and 12,947 in 2042.

The data used in developing and calibrating the models for the water distribution system, sanitary sewers, storm sewers and traffic in this study was gathered through an extensive condition assessment program that included:

1. Hydrant testing of five hydrants on December 11, 2017, 2017 by Fire Protection Inc. These results were used to calibrate the water model and are in Appendix B of this report.
2. Flow monitoring program was performed by SFE Global of four additional locations for the period of August to October 2017. The flow monitoring duration was selected to capture the dry and wet weather conditions. Data from the flow-monitoring program was used to calibrate the sanitary sewer model. This data is available in an ancillary binder accompanying this report.
3. Video Camera and CCTV of roughly 4.5 kms of existing sanitary sewers were conducted during the period of December 2017 to January 2018. This work was performed by Thuro Inc. The results of this CCTV was used to determine the required upgrades of existing sanitary sewers due to poor conditions (major cracks, joint misalignment, etc.). The reports for each of the lengths investigated and their video are included in an ancillary binder accompanying this report.
4. Visual inspection of the existing storm water system that included a field condition assessment of 27 storm manholes, nine culverts ,19 outfalls, and three stormwater facilities completed by GHD's field inspection staff during the week of August 23, 2017. The results of this field assessment was used to determine the required upgrades of existing storm sewer and drainage system. This data is available in Appendix E of this report.
5. Pavement field assessment and a pavement condition index that included a visual evaluation (windshield survey) of all roads within the town and a detailed pavement inspection of selected 53 sites. The results of this pavement assessment was used to determine the required upgrades of the existing roadways. This field assessment was done during the first

week of August 2017 by Shelby Engineering and is available in Tab D in the Transportation & Roadway Network Reports ancillary binder.

6. Intersection turning movement count was performed at ten selected intersections during the morning peak hour (7:30 to 9:30 AM) and the evening peak hour (4:00 to 6:00 PM) during the last week of November 2017 and the first week of December 2017 by WSP. This traffic count was used as an input and to calibrate the transportation model . The traffic count data, synchro reports and signal warrant reports are also in the Transportation & Roadway Network Reports ancillary binder in Tabs A through C respectively, accompanying this report.

Based on the results of the MSP, a summary of the required immediate upgrades and the future upgrades for horizon years 2032 and 2042 for each municipal system is as follows:

Water Supply and Distribution System

The Town's water is supplied by groundwater from 11 active wells. Two additional wells are available to provide fire flows. Five of the active wells supply water to four reservoirs. Water from the other wells is pumped directly into the distribution system. The water from each well is disinfected at the well. There is one other well in the system (Well 3) that has been taken out of service but can be added back after installation of a new pump and disinfection facility. With the addition of two new wells that were planned for 2017, this will potentially bring the total number of supply wells to 16.

Well yields from the existing wells have deteriorated below their pre-approved discharge rates. The total existing discharge of 55.52 liters per second (L/s) from the 11 active wells is less than the Town's maximum daily demand (MDD) of 68 L/s. With the planned addition of two new wells with an estimated 14 L/s increase of flow (7 L/s each); the existing MDD can be met.

Additional new wells providing an average flow of 7 L/s per well are required to meet future demands for the developments within the Town and fringe areas in the County. Water consumption will need to be closely monitored to realistically project when each stage of upgrades will be needed. Future water supply requirements may necessitate a search for alternate water sources as additional groundwater wells may be insufficient.

The existing four water storage and distribution reservoirs include two above ground reservoirs and two smaller below ground reservoirs. The below ground reservoirs are filled by well pumps and supply water to the distribution system by domestic water and fire pumps at each reservoir. The two above ground reservoirs at Grande Prairie Trail maintain the Zone 1 pressure in the distribution network. These reservoirs are filled from the distribution system during low demand and supply water to the distribution system by gravity during normal operation.

The existing total available storage of 6,530 cubic meters (m^3) in the reservoirs is adequate for the current storage requirement of 4,772 m^3, on the existing Provincial standards for fire flow, operation, and emergency storage. Additional 2,000 m^3 storage capacity upgrade will be needed before 2032 to bring the total storage to 8,430 m^3. This volume will satisfy the storage requirement of 7,580 m^3 in 2032 and the requirement of 8,246 m^3 in 2042.

The Town has two pump stations at the two underground reservoirs for water distribution, and an inline booster station at Edson Drive and 13th Avenue. The distribution network operates in two pressure zones, Zone 1 and Zone 2. Zone 1 covers the majority of the system. The booster station at Edson Drive and 13th Avenue draws water from Zone 1 and supplies pressure to Zone 2. The existing distribution system consists of pipes varying in size from 100 millimeters (mm) to 350 mm diameter and pipe materials that include cast iron, ductile iron, asbestos cement, steel, PVC and HDPE.

The required upgrades of the existing water supply and distribution system were recommended as results of two categories, condition assessment and capacity restraints as determined by modelling analysis.

The condition assessment involved a desktop review of the existing water distribution system including the year of installation, type of pipe and pipe breaking database supplied by the Town.

Based on this review, we recommend the replacement of roughly 4.68 km of aging pipe for an estimated cost of roughly $4.27 million.

The capacity of the existing water supply and distribution system was analyzed by modeling the system using WaterCAD V8i software and calibrated using the results of hydrants testing. The results of the model showed that the existing distribution system was found to be generally adequate in supplying the required pressure under the existing peak hour flow except in the northwest area of Town. Two possible improvement options were analyzed to meet the existing peak hour demand requirements. Option 1 involved upgrading the existing booster station. Option 2 involved constructing a new off-line pump station that would draw water from the Grande Prairie Trail reservoirs and supply water and pressure to the northwest part of Town, the future Zone 3 area in the northwest, and ultimately the existing and future Zone 2 areas. Option 2 was recommended due to a lower overall construction cost, as well as lower operating cost. Option 2 also included the abandonment of the existing booster station.

The existing distribution system is not adequate for the combined maximum daily demand (MDD) and Fire conditions, and will require trunk looping and pipe size upgrades. Five deficient locations were identified in this study .In order to address these deficiencies, the following new additions and upgrade of pipes were recommended.

- A proposed new loop of 250 mm diameter at the northwest section of the Town.
- A proposed new 350 mm diameter pipe that creates a loop with the southwest section of the Town.
- A proposed new 350 mm diameter pipe that creates a loop with the east section of the Town
- A proposed upgrade of existing 100 mm, 150 mm and 200 mm pipes at three locations within the Town.
- Option 2 as discussed under the Peak Hourly Demand scenario.

For the future scenarios, additional upgrading to various locations of the existing distribution system will be necessary, as well as extending services to the future development areas of the Town and the County at the urban fringe locations. Two scenarios were analyzed, first one the Town only and second one involved Town and Fringe areas.

Two options for improvements to the distribution system were identified for the MDD and Fire requirements for the 2032 scenario. Option 1 would involve installation of a new fire pump and power generator at the Degas Reservoir, and expansion of reservoir storage to 3,000 m³. Option 2 would involve connecting the Zone 2 network to Zone 1 with a pressure-reducing valve to supply fire flow to the Town's future commercial area in the west and the County's future urban fringe area. In order to increase the pressures to Zone 2A, a new piping loop served by the new booster station and not hydraulically connected to the Prairie Grande Trail Reservoirs to be installed. Option 2 was recommended because it provides more flexible fire flow supply to both the Town and the County fringe area.

This option included, addition of a proposed 1250 m^3 Reservoir and proposed booster station at Degas Site , the addition of another proposed 1300 m^3 Reservoir at Microwave Tower (MT) Site and a proposed new pump at the GPT station.

It should be noted that other option for boosting pressure and fire flow in the area shown as Zone 2A in Figure 3.4 ,would have been to connect this new loop to the future Microwave Tower reservoir recommended in 2032 (Section 3.7.1.1.4). However, when this scenario was modelled, the pressures along 62 Street dropped into a range of 32 psi to 39 psi, which is below the 40 psi requirement, while fire flows requirements were still met. This would decrease the necessary capabilities of the Grande Prairie Trail booster station. This option was not recommended because of deficiency in meeting pressure requirements as the Microwave reservoir installation was not recommended until 2032.

For the Town and the fringe area, a bigger proposed reservoir (4320 m^3) and pump upgrades at Degas Site was recommended as well as an additional new piping at the east and southwest fringe area.

For the 2042, proposed new piping, valves and a new pump was recommended.

The Table below summarizes the existing and future system upgrades and their estimated costs

Stages	Upgrade Work	Cost
Existing System	Upgrades / Install New Pipes	$10,391,150
	Demolish Existing Booster Station	$70,000
	New 250 mm Northwest Loop	$1,866,900
Total for Existing		**$12,328,050**
2032 Town Only	Reservoir at Degas Site	$1,995,000
	Reservoir at Microwave Tower (MT) Site	$1,200,500
	Upgrades / Install New Pipes	$5,859,350
	New Wells and Services	$1,484,000
Total for 2032 Town Only		**$10,538,850**
2032 Town and Fringe	Water System at Degas Site	$3,752,000
	Water System at Microwave Tower (MT) Site	$4,765,530
	Water System From GPT to MT Site	$21,143,220
Total for 2032 Town and Fringe		**$29,660,750**
2042 Town Only	Upgrades / Install New Pipes	$1,417,500
	New Water Services	$151,200
Total for 2042 Town Only		**$1,568,700**

Stages	Upgrade Work	Cost
2042 Town and Fringe	Pump Upgrades	$560,000
	Storage Reservoir	$371,000
	New Water Wells	$252,000
Total for 2042 Town and Fringe		**$1,183,000**

Wastewater Collection System

The existing sanitary system consists of approximately 68 kms of gravity sewer mains ranging in size from 200 mm to 1200 mm diameter .The sewers currently discharge to the town's lagoon until the completion of the construction of the new mechanical wastewater treatment plant.

The required upgrades of the existing wastewater collection system were recommended as results of two categories, condition assessment and capacity restraints due to undersized pipes as determined by modelling analysis.

The condition assessment involved CCTV of roughly 4.5 kms selected at various locations within the Town as a cross section of the whole system.

The CCTV showed poor condition in some locations that included major pipe cracks and misaligned pipe joints. Large percentage of sewer lines that are in poor conditions were aging small diameter vitrified clay tile pipe in the old downtown section .The replacement of roughly 10.5 kms of existing sewers in poor conditions was recommended and summarized in the table below.

The hydraulic capacity of the system was analyzed by modelling the system using XP-SWMM model version 9.14 and calibrated using the flow monitoring data from four locations within the town.

The model included 229 catchments, 756 manholes, one outfall, and 796 pipes using a 5-year rainfall, a 4-hour Chicago design storm and a 24-hour rainfall Huff Design storm.

The Modelled volume and peak flow are within 11 percent and 4.5 percent of the monitored values, respectively. In general, the modelled and monitored wet flows compare favorably. Recommended upgrades of the existing system to overcome shortage hydraulic capacity in some sewer sections included twinning existing undersized pipes, proposing and installing new properly sized sewers. These suggested upgrades are recommended to prevent basements flooding. In order to minimize the risk of basement flooding, our suggested upgrades were limited to sewers that showed surcharging within 2.5 m of the ground level in residential areas in the 5-year 4-hour rainfall event . In addition, recommendation for upgrade included sewer sections that showed surcharging within 1.0 m of the ground level for the 25-year 4-hour rainfall event and have flow 1.2 times (1.2 x) larger than full pipe capacity.

The proposed upgrades due to poor condition pipes and lack of hydraulic capacity and the estimated associated cost are summarized below:

Description	Total Length (m)	Total Cost
Existing System Upgrades (2017)		
Condition Assessment	10,483	$8.23 Million
Capacity Issues	2,548	$2.65 Million
Future Upgrades (2032)		
Condition Assessment	11,284	$9.79 Million

Description	Total Length (m)	Total Cost
Capacity Issues	14,050	$30.98 Million
Future Upgrades (2042)		
Capacity Issues	8,500	$9.22 Million

Stormwater Management System

The current developed area within the Town limits is approximately 550 ha with the majority of the area drainage flowing through gravity storm sewers, with the exception of the Glenwood area at the west part of the town that has open roadway ditches.

The required upgrades of the existing stormwater management system were recommended as results of two categories, condition assessment and capacity restraints due to undersized pipes as determined by modelling analysis.

The condition assessment was based on actual field assessment of 27 storm manholes, nine culverts, 19 outfalls, and three stormwater facilities to evaluate the overall condition of the stormwater infrastructure. Although maintenance issues such as culverts half filled with sediment were identified, there were no major need for immediate upgrade and/ or replacements.

The hydraulic analyze of the existing system was completed by modelling the existing storm sewer and drainage system for 5, 25 and 100-year using both short duration (4-hour) and long duration (24-hour) rainfall events. The modeling results indicated a large amount of surface flooding during the 5-year 4-hour event. A majority of the system pipes experienced a peak flow 1.2 times greater than the pipe capacity. For a long duration (24-hour) rainfall event, the storm system performs significantly better with the lower peak intensity, with only limited pipe sections experiencing a high ground water level (GWL) that is within 1.0 m below the ground level.

For a 25-year, 4-hour rainfall event, the system is highly surcharged with additional surface flooding areas, even worse than the 5-year, 4-hour event.

For the 25-year 24-hour rainfall event, the system would be able to convey the flows with enough capacity. Only a few surcharging pipes were noticed in the model results.

The 100-year, 4-hour event has the worst system performance, with severest system flooding. Under the 100-year, 24-hour rainfall event, most of the sewer pipes had adequate capacity to discharge the inflow, but surface flooding also occurred in a few locations.

The 2011 MSP report suggested a level of service that in the 5-year, 4-hour rainfall event scenario, no storm pipe should surcharge within 1.0 m of the ground level. To achieve this service level, a total of 15,960 meters of storm pipes need to be upgraded, by either replacement, pipe twinning or replacement with a steeper grade. The total estimated length of sewers that required immediate improvement was approximately 19 km with pipe sizes ranging from 300 to 1500 mm diameter. The report also estimated cost for replacement was $29.8 million and the cost for estimated twinning was $24.5 million for total required upgrades of roughly $54.3 million.

A stormwater management plan was developed for the Town based on the future 2032 and 2042 development scenarios, which included build out and servicing of 30 drainage basin areas. The future stormwater system would be an independent, newly built system with stormwater management facilities. Using this design system the discharge rate would not exceed the maximum discharge rates established for each drainage basin area.

The existing storm system improvements and future storm system cost estimates are shown in the table below:

Description	Commercial/Industrial Development	Residential Development	Total
Existing System Upgrades (2017)			
2017			$46.14 million
Future System Upgrades			
2032	$18.73 million	$1.47 million	$20.20 million
2042	$32.62 million	$1.90 million	$34.52 million
Total			**$54.72 million**

Transportation and Roadway Network

The required upgrades of the existing transportation and roadway network system were recommended as results of two categories, pavement condition assessment and corridors capacity restraints as determined by modelling analysis.

The pavement condition assessment was based on visual evaluation (windshield survey) of all roads within the town and a detailed pavement assessment of selected 53 sites using the Pavement Condition Index (PCI) rating. Immediate upgrades of roads with pavement conditions that were found to be “poor”, “very poor” and “serious” (PCI rating 55 or less) were recommended for a total of approximately 12.5 kilometers of linear meters. These estimated immediate rehabilitation cost is estimated to be roughly $15.36 million.

The Corridor and intersection capacity utilized the findings from the 1980 transportation study and the transportation section of the 1982 General Engineering Study as the basis for analysis. The analysis was completed by modelling the roadway network using Synchro Version 9 Software with intersection turning movement count at ten selected intersections during the morning and evening peak hour. This morning and evening peak hour was used in the analysis to determine the need for future upgrades. Additionally, 38 screenline locations were selected for a corridor capacity analysis.

The analysis showed an immediate recommended signalization of the intersection of Highway 16 at 42nd Street is required. The signalization of this intersection should also include a re-design of the intersection, primarily with respect to the re-alignment of the north approach and elimination of the adjacent intersection of 2 Avenue at 42 Street, resulting in an improved concept from an operational and safety perspective. The estimated cost of this upgrade is approximately $400,000.

For the future horizon at 2032, no proposed upgrades due to pavement condition assessment would be required.

The corridor capacity analysis and road network analysis showed a future signalization at Highway 16 at 25th Street intersection would be required as the southbound approach is expected to experience significant delays and reach capacity in the 2032 scenario. The results of the analysis indicate a noticeable improvement with signalizing the intersection, resulting in no capacity concerns, low levels of delay, and acceptable queueing.

In addition, proposed new rural and urban roads, as an expansion of the existing road network, would be required. The total estimated cost for all such upgrades including the proposed new signalization is approximately $62.2 million

For the 2042 time horizon, no upgrades due to condition assessment would be required. However, new expansion of both rural and urban roads were recommended to allow for new developed areas with the Town with an estimated cost $43 million.

Based on the results of the corridor capacity analysis, none of the study corridors are expected to reach capacity by 2042. The corridor expected to experience the highest utilization by 2042 is 13th Avenue west of Edson Drive at only 54 percent. The remaining corridors assessed in the study are not expected to exceed 47 percent utilization by 2042. However, to service the future developable lands as identified in Figure 6.1, GHD has recommended expansions to the existing road network to these developable areas.

The results showed our analysis also included the following two intersections at:

- **4th Avenue at 70th Street:** The southbound through/right-turn movement is expected to reach Level of Service (LOS) ‘F’ by the 2042 scenario. The intersection does not warrant upgrade to all-way stop or traffic signal control and these operating conditions are not expected to be overly concerning given the long-term horizon. As a result, no improvements are recommended for this intersection.
- **13th Avenue at Edson Drive:** The intersection is expected to reach capacity in the eastbound and southbound directions by 2032, as well as the westbound direction by 2037. The all-way stop controlled intersection does not warrant upgrade to traffic signal control. The intersection is currently not experiencing operational issues during the a.m. and p.m. peak hours, and there is uncertainty whether or not the forecasted growth as projected will result in future operating issues. The Town should consider monitoring the intersection in the future. As a result, no improvements are recommended for this intersection.

High-level cost estimates for the proposed road extensions were developed. For the purpose of cost forecasting, it was assumed that proposed roads within the “fringe” area was constructed to a rural cross-section, and proposed roads within the built-up area of the Town will be constructed to an urban cross-section. The summary of all existing and proposed upgrades and their associated estimated cost is list in the table below.

Cost Estimate for Proposed Transportation and Road Network

Horizon Year	Cross-Section	Proposed Linear Distance (estimate)	Cost Rate Estimate (Includes 30 percent Contingency + 10 percent Engineering & Administration)	Total Cost (estimate)
2017	Signalization of Hwy 16 at 25nd Street		$400,000	$400,000
	Road replacement due to poor pavement condition of12.5Kms		$15,360,000	$15,360,000
	2017Total			**$15,76 Million**
2032	2 Lane Urban Cross-Section	10.3 kilometers	$3,900 per meter	$40.2 million

Horizon Year	Cross-Section	Proposed Linear Distance (estimate)	Cost Rate Estimate (Includes 30 percent Contingency + 10 percent Engineering & Administration)	Total Cost (estimate)
	2 Lane Rural Cross-Section	10.3 kilometers	$2,100 per meter	$21.6 million
	Signalization of Hwy 16 at 42nd Street		$400,000	$400,000
	2032 Total	**20.6 kilometers**		**$62.2 million**
2042	2 Lane Urban Cross-Section	5.6 kilometers	$3,900 per meter	$21.8 million
	2 Lane Rural Cross-Section	10.1 kilometers	$2,100 per meter	$21.2 million
	2042 Total	**15.7 kilometers**		**$43 million**

A brief overview has also been provided in Section 6.10 of pertinent Complete Street concepts that can be incorporated into the future planning, design, and reconstruction of existing streets and construction of new streets.

A summary of all recommended upgrades for water supply and distribution, wastewater collection system, stormwater system and transportation and roadway network upgrades for existing, 2032 and 2042 is summarized in the table below.

Table ES.1 Cost Estimate Summary

Description	Water Supply and Distribution System	Wastewater Collection System	Stormwater System	Transportation and Roadway Upgrades
2017 (Immediate Costs)				
2017 (Condition Assessment)	$4.27 Million	$8.23 Million	0	$15.36 Million
2017 (Capacity Analysis & Immediate Improvements)	$12,33 Million	$2.65 Million	$46.14 Million	$0.40 Million
2032 (15 Year Projected Costs)				
2032 (Condition Assessment)	N/A	$9.79 Million	N/A	N/A
2032 (Capacity Analysis & Expansion)	$10.54 Million (Town only) $28.84 Million (Town and Fringe)	$30.98 Million	$20.20 Million	$62.20 Million
2042 (25 Year Projected Costs)				
2042 (Capacity Analysis & Expansion)	$1.57 Million (Town only) $1.18 Million (Town and Fringe)	$9.22 Million	$34.52 Million	$43.00 Million

Table of Contents

Exhibit Index

Table Index

Appendix Index

1. Introduction

1.1 Project Background

The Town of Edson (Town) is looking to update its master servicing plan (MSP) and offsite levy rates (OLRs) to transition the MSP into a blueprint that harmonizes and supports growth management, planning, and sustainable and responsive servicing for communities and businesses of the Town for the next 25-years and beyond. This MSP update was developed from a previous update done in 2011 (2011 MSP Update).

This MSP update will include information from the current (2016) Municipal Development Plan (MDP) and any statutory plans and servicing concepts of new areas that have been approved since the 2011 MSP update. The Town also wishes to expand the scope of study to allow for development of a more comprehensive plan that covers important attributes that are not addressed in the existing MSP. In terms of infrastructure funding, the Town requires a fair and balanced offsite levy rate (OLR) program that will encourage sustainable development while ensuring that the Town receives a fair share of revenue to maintain a high level of service to its residents and businesses.

As stated above, this also includes an update to the OLR. The existing OLRs were developed based on an OLR study done in 2013. Not only do the OLRs need to be updated to reflect the new MSP, but more importantly, the funding mechanism and methodology will need a critical review to ensure development of a fair and equitable levy program to assist the Town in their efforts to maintain high level of service to the community.

1.2 Objectives and Scopes

The scope of this update will include examination of the following infrastructure:

- Water System
- Wastewater System
- Stormwater System
- Transportation and Roadway Network (road) system

1.2.1 Objectives

As stated above, the objectives are to update the MSP and to update the OLR.

1.2.2 Scope

The scopes of this update include:

- The water system
- The wastewater system
- The stormwater system
- The transportation (road) system

The following are key components of the MSP and OLR update studies:

- Hydraulic capacity assessments of existing and future upgraded water, wastewater, and stormwater servicing systems
- Traffic capacity assessments of existing and future upgraded road system
- Model calibration and update of the existing water network, sanitary sewer, and storm sewer models providing training for up to two Town personnel on the use of the selected water network model
- Physical condition assessments of existing infrastructure using a combination of desktop studies and field inspections
- Inclusion of municipal design standards in the MSP where appropriate
- Development of an integrated, prioritized capital expenditure program for infrastructure upgrading and expansion
- Land requirement plan for future road and utility easements
- Updates of offsite levy development areas and their development timelines
- Lands benefitting from existing and future offsite infrastructures and the allocation of infrastructure costs to benefitting parties

1.3 Abbreviations

ADD	Average Daily Demand
MDD	Maximum Daily Demand
PHD	Peak Hour Demand
MDD+FF	Maximum Daily Demand plus Fire Flow
DWF	Dry Weather Flow
WWF	Wet Weather Flow
DEM	Digital Elevation Model
GIS	Geographic Information System
ha	Hectare
HP	Horsepower
HDPE	High-density Polyethylene
CI	Cast Iron
DI	Ductile Iron
AC	Asbestos-cement
PVC	Polyvinyl Chloride
I&I	Inflow and Infiltration
ICI	Industrial/Commercial/Institutional
km	Kilometer
kPa	Kilopascal
Lpcd	Liters per capita per day
L/ha/d	Liters per hectare per day
L/min	Liters per minute
L/s	Liters per second
L/s/ha	Liters per second per hectare
LS	Lift Station
m	Meter
m^3	cubic meters
mm	Millimeter

Psi	Pounds per Square Inch
AEP	Alberta Environment and Parks
OLR	Offsite Levy Rates
ROW	Right-Of-Way
VFD	Variable Frequency Drive

1.4 Information Collection and Review

1.4.1 Information Collection

The following planning documents and study reports were reviewed:

- General Engineering Study (1982 by SAEL)
- Storm Management Plan (July 2005 by UMA-AECOM)
- Edson Urban Fringe Inter-municipal Development Plan (June 2007 by ISL)
- Lagoon Assessment (October 2007 by Earth Tech)
- Municipal Servicing Plan Update (December 2011 by AECOM)
- Municipal Census Stats (2012)
- Offsite Levy Rates – Appendix A, Infrastructure Improvements (May 2013 by AECOM)
- Offsite Levy Rates (July 2013 by Corvus)
- Edson West Sanitary Trunk Main (March 2014 by ISL)
- Town of Edson WWTP Upgrade - Functional Design Brief (November 2014 by AECOM)
- Town of Edson – Municipal Development Plan – bylaw 2172 (January 2016)
- Town of Edson – 1980 Transportation Study

2. Population Projection and Future Development Horizons

2.1 Population Projection Update

The Town's new developments since the 2011 MSP update are shown on Figure 2.1.

Future population growth projections were considered for the Town and the County.

2.1.1 Town of Edson

Future infrastructure will be developed based on future population growth. In the past, the Town's population has shown a gradual growth. However, the population has shown a decreasing trend in the period 2011 to 2016.

The historical populations from 1996 are shown in Table 2.1.

Table 2.1 Historical Populations in the Town

Year	1996	2001	2006	2011	2016
Population	7,399	7,585	8,098	8,475	8,414

Based on these values, the average annual population growth rate for the 20-year period from 1996 to 2016 was 0.65 percent.

To estimate a range of future growth scenarios, population projections in three different scenarios, i.e. a medium growth scenario (equal to the above historic annual growth rate of 0.65 percent), a high-growth scenario (annual growth rate of 1.00 percent), and a low-growth scenario (annual growth rate of 0.50 percent), are shown in Table 2.2.

These population projections were presented in the former study reports. The projections were based on development areas and a population growth rate of one percent recommended by the Town. The projected 2042 population in the Town's MDP (2016) is relatively high as the population projection starts from the 2011 population. Based on the historical population increase, 1.00 percent growth rate will be a relatively conservative growth rate and it is the same growth rate as in Town's MDP (2016).

Table 2.2 Population Projection for the Town

Year	Low Growth 0.50 percent	Medium Growth 0.65 percent	High Growth 1.00 percent	Horizon Years
2016	8,414	8,414	8,414	
2017	8,456	8,469	8,498	
2018	8,498	8,524	8,583	
2019	8,540	8,579	8,669	
2020	8,583	8,635	8,756	
2021	8,626	8,691	8,844	
2022	8,669	8,747	8,932	5
2023	8,712	8,804	9,021	
2024	8,756	8,861	9,111	
2025	8,800	8,919	9,202	
2026	8,844	8,977	9,294	
2027	8,888	9,035	9,387	10
2028	8,932	9,094	9,481	
2029	8,977	9,153	9,576	
2030	9,022	9,212	9,672	
2031	9,067	9,272	9,769	
2032	9,112	9,332	9,867	15
2033	9,158	9,393	9,966	
2034	9,204	9,454	10,066	
2035	9,250	9,515	10,167	
2036	9,296	9,577	10,269	
2037	9,342	9,639	10,372	20
2038	9,389	9,702	10,476	
2039	9,436	9,765	10,581	
2040	9,483	9,828	10,687	
2041	9,530	9,892	10,794	αααααααααααα
2042	9,578	9,956	10,902	25

2.1.2 Yellowhead County

The historical population of the County is shown in Table 2.3.

Table 2.3 Historical Populations in the County

Year	1996	2001	2006	2011	2016
Population	10,092	9,881	10,045	10,469	10,995
	Recent 20 Years Historical Growth Rate				0.43 percent

Based on these values, the average annual population growth rate for the 20-year period from 1996 to 2016 was 0.43 percent.

Based on the County's MDP of September 2013, the population projection rate is approximately 0.43 percent for the future 30 years. To estimate a range of future growth scenarios, population projections in three different scenarios, i.e. a medium scenario (equal to the above historic annual growth rate of 0.43 percent), a high-growth scenario (annual growth rate of 0.70 percent), and a low-growth scenario (annual growth rate of 0.20 percent), are shown in Table 2.4.

Table 2.4 Population Projection for the Yellowhead County

Year	Low-Growth 0.20 percent	Medium Growth 0.43 percent	High – Growth 0.70 percent	Horizon Years
2016	10,995	10,995	10,995	
2017	11,017	11,042	11,064	
2018	11,039	11,089	11,134	
2019	11,061	11,137	11,204	
2020	11,083	11,185	11,275	
2021	11,105	11,233	11,346	
2022	11,127	11,281	11,417	5
2023	11,149	11,330	11,489	
2024	11,171	11,379	11,561	
2025	11,193	11,428	11,634	
2026	11,215	11,477	11,707	
2027	11,237	11,526	11,781	10
2028	11,259	11,576	11,855	
2029	11,282	11,626	11,930	
2030	11,305	11,676	12,005	
2031	11,328	11,726	12,081	
2032	11,351	11,776	12,157	15
2033	11,374	11,827	12,234	
2034	11,397	11,878	12,311	
2035	11,420	11,929	12,389	
2036	11,443	11,980	12,467	
2037	11,466	12,032	12,546	20
2038	11,489	12,084	12,625	
2039	11,512	12,136	12,705	
2040	11,535	12,188	12,785	
2041	11,558	12,240	12,866	
2042	11,581	12,293	12,947	25

Based on the County's MDP, a 0.70 percent growth rate was applied. It is a relatively conservative growth rate.

2.2 Future Development Update

Future development areas in the Town need to be predicted so that the utilities can be planned in different future horizon years.

2.2.1 Future Development Update for the Town of Edson

The Town's MDP (2016) presented the existing land use plan, a plan of the future land supply, and the future land use concept plan. From these plans, GHD produced Figures 2.1 and 2.2, Appendix H, which illustrate the population projections and areas to be developed in the future.

For modelling studies and utility planning for different future horizons, future developments were staged and shown in Table 2.5 and Figure 2.2, which were developed based on the Town's draft staging plan shown in the ancillary binder.

The total increased residential land in different future horizon years were based on the increased population and 40 people per hectare as shown in the 2016 MDP. The total increase of industrial/commercial lands were based on an approximately 1 percent growth rate that is the same as the future population growth rate.

Table 2.5 Future Development Lands in Different Future Horizons in the Town

Future Development Horizon Year	Future Population	Available Residential Land before Development		Available Commercial & Industrial Land before Development		Future Residential Growth Area	Future Commercial & Industrial Growth Land
		Area	No. in Fig 2.2	Area	No. in Fig 2.2		
		ha		ha		ha	ha
2032	9,867						
		7.45	1	9.5	3	7.45	9.5
		34.86	2	21	4	9	10
		20.22	5	22.64	6	11	5
		10.73	8	35.26	7	3	6
		30.26	13	120	9	3	6
		19.92	14	59.9	10	3.05	5
				22.17	11		2
				26.4	12		2.5
	Total (ha)	**123.44**		**316.87**		**36.5**	**46**
2042	10,902						
		0.00	1	0	3	0	0
		25.86	2	11	4	4.5	7
		9.22	5	17.64	6	3	3
		7.73	8	29.26	7	3.5	4.5
		27.26	13	114	9	3	7
		16.87	14	54.9	10	3	5
		44.75	16	20.17	11	3	4
		20.22	17	23.9	12	3	3
		10.50	18	28.7	15	3	2
	Total (ha)	162.41		299.57		26	35.5

It is expected that after 2042, the No. 1 and No. 3 lands will be fully developed and No. 2 and No. 4 through No. 18 will be partially developed. Some of the lands will not be developed before 2042.

2.2.2 Future Development Update for the Fringe Area in Yellowhead County

In the Edson Urban Fringe Inter-municipal Development Plan (June 2007), a framework was provided for the long-term growth and development of lands located within the Edson Fringe Plan Area that included lands in the County and the Town. In the 2011 MSP Update, a projected growth was presented for the Yellowhead Fringe Area. For this study, the industrial/commercial area was assumed to be moderately developed in the future. The total future development area in different horizons will be the same level as the areas in the Town's future commercial/industrial development areas. The fringe area in the County is predicted to be developed as shown in Table 2.6. The water demands and sanitary flow in the County land will be based on the population growth and industrial/commercial land increasing in the areas in the County.

Table 2.6 Future Development Lands in Different Future Horizons in the Fringe Area in the Yellowhead County

Future Development Horizon Year	Future Population	Available Commercial & Industrial Land before Development		Future Commercial & Industrial Growth Land
		Area	Area in Figure 2.2	
		ha		ha
2032	12,157			
		56.1	17	8
		86.4	18	23
		96.5	23	15
	Total (ha)	239	**Red**	46
2042	12,947			
		48.1	17	10
		63.4	18	8
		81.5	23	9
		70.1	19	8.5
	Total (ha)	263.1	**Pink**	**35.5**

3. Water System

3.1 Existing Water System

The following sections provide a summary of the existing water system. The complete system with locations of pump stations, reservoirs, and wells is presented in Figure 3.1. Figures 3.1A, 3.1B, and 3.1C present larger scale views of the water infrastructure to provide a more detailed understanding of the system.

3.1.1 Groundwater

The Town's water system (reference Figures 3.1 and 3.1A through 3.1C) is currently supplied by thirteen existing groundwater wells. Two of these wells (Wells 16 and 21) are not active during daily operations, as described below, effectively leaving eleven existing wells supplying water to the system. A fourteenth well (Well 3), currently has no well pump or disinfection facility. Three of the fourteen wells (Wells 18, 19, and 20) feed water to the existing Degas Reservoir via well pumps in each well, and two existing wells (Wells 26 and 27) supply water to the existing Glenwood Reservoir via well pumps in each well. The pumps in the Degas Pump Station and Glenwood Pump Station pump the water from the reservoirs to the Town's distribution system.

Eight of the fourteen wells are currently able to supply drinking water to the distribution system directly by using well pumps in each well. Disinfection facilities at each well disinfect the water before it reaches the distribution system. However, two of these eight wells supply fire flow only because of high manganese levels (Well 16) and Sulfate Reducing Bacteria/Iron Reducing Bacteria (SRB/IRB) levels (Well 21). It should be noted that these constituents are not toxic (they have issues with taste, odor, color, and staining). During normal daily operations, only six of the fourteen wells supply flow to the distribution system (Wells 2, 12, 17, 22, 24, and 25) directly.

The characteristics and status of the groundwater wells are presented in Table 3.1.

Table 3.1 Well Information

Well No:	Allowed discharge (L/s)	Actual Current Discharge (L/s)	In 2011 model	Abandoned since 2011	New Wells added since 2011	To be installed in 2017	Equipped with VFD	Chlorination	Current status
2	5.68	2.58	Yes				Yes	Yes	In use
3	6.44	0.00	Yes	Pump removed			No		Could be used as standby with installation of a new pump
9				Yes					Abandoned
12	15.15	5.60	Yes				Yes	Yes	In use, but yield is declining. Will eventually be replaced by Well 28
14			Yes	Yes					Abandoned, replaced by Well 24
15			Yes	Yes			Yes		Abandoned, replaced by Well 22
16	7.58	0.00	Yes				No	Yes	Elevated manganese levels, used only in fire flow situations
17 (Degas)	2.80	2.71	Yes				No	Yes	In use
18	9.47	8.88	Yes				Yes	Yes	In use
19	3.13	2.11	Yes				No	Yes	In use
20	2.66	2.51	Yes				No	Yes	In use
21	3.94	0.00			Yes		Yes	Yes	Added since 2011. Elevated SRB/IRB levels, used only in fire flow situations
22	15.15	11.79			Yes		Yes	Yes	In use, replacement for Well 15
23 (Glenwood)				Yes					Abandoned
24	18.94	11.62			Yes		Yes	Yes	In use, replacement for Well 14
25	7.82	3.55			Yes		Yes	Yes	In use
26	2.20	2.20			Yes		Yes	Yes	In use
27	1.99	1.99			Yes		Yes	Yes	In use
28	7.0	0.0				Yes			Future
29	7.0	0.0				Yes			Future
Total	**102.95**	**55.52**							

Without considering future Wells 28 and 29, the total allowed discharge from the existing operating wells in Town is 102.95 L/s. Each well has a pre-approved pumping rate based on its yield. The values from the operating wells add up to the total allowed discharge from the existing Town's Well System. Information provided by the Town indicated that the well yields have deteriorated to below those pre-approved values. Based on recent performance data, it is estimated that the existing wells can only supply a total actual discharge of 55.52 L/s. The estimated flow that each well can supply is identified in Table 3.1. As discussed above, Wells 16 and 21 are not normally used for domestic water supply due to water quality concerns in these wells. During fire events, these wells will operate and can provide an additional combined flow of 11.52 L/s, yielding a total flow from the well system of 67.04 L/s.

Well 3 has not been used for some time. Based on the Town's comments, this well will require installation of a new pump, disinfection facility, and connection piping to resume operation. If the existing booster station at Edson Drive and 13 Avenue were to remain in operation, the Well 3 discharge should connect to the existing distribution system within Pressure Zone 1 to maximize the water distribution efficiency, as pressure Zone 1 provides flows to the majority of Town. The connection point to Zone 1 can be at 49 Street and 18 Avenue or at 48 Street and 18 Avenue. Relatively short pipes will be needed for this connection from Well 3 to the existing distribution system. However, as discussed in Section 3.6.5, it is possible for the existing booster station to be abandoned. If this booster station is abandoned, the Well 3 discharge should connect to the existing distribution at the nearest feasible location. After the planned installation of Wells 28 and 29, Well 3 should be brought back into service before additional wells are installed.

The ground elevation at Well 3 is 937.26 m, and the bottom elevation of the well is 900.36 m. The new pump should be installed at a bottom elevation of 903.96 m.

3.1.2 Water Disinfection

Disinfection of the groundwater from the Wells is required. The groundwater from Wells 16 (elevated manganese levels) and 21 (elevated SRB and IRB levels) is only used to supply fire flows, not for general potable consumption.

As shown in Table 3.1, disinfection is provided in the form of chlorination at all the wells in use, including Wells 16 and 21. Well 3 is not in use at this time, so there is currently no water to disinfect.

3.1.3 Storage Reservoirs and Pressure Zones

The Town's existing system includes four storage reservoirs, and details of the reservoirs are provided in Table 3.2.

Table 3.2 Reservoirs

Name	Location	Description	Storage Volume (m^3)
Reservoir 1 (a)	Grande Prairie Trail (17th Avenue)	Steel reservoir above grade	2,273
Reservoir 1 (b)	Grande Prairie Trail (17th Avenue)	Concrete reservoir above grade	3,410
Degas	Rodeo Road and Highway 16, adjacent to Well 17	Concrete reservoir below grade	147
Glenwood	Wilmore Park Road & 3rd Avenue, adjacent to abandoned Glenwood Well (23)	Concrete reservoir below grade	700
Total Storage			6,530

The existing distribution system has two pressure zones, Zone 1 and Zone 2, as shown on Figure 3.1. Pressure Zone 1 covers most of the Town. The in-line booster station located at Edson Drive and 13 Avenue pumps from Zone 1 and provides pressure to Zone 2.

The two reservoirs at the Grande Prairie Trail site provide pressure directly to pressure Zone 1. The hydraulic grade line for pressure Zone 1 is 966.06 m. The Grande Prairie Trail reservoirs fill with water from the distribution system during times of low demand and supply water to the distribution system by gravity during normal operation.

Water is supplied to the Degas Reservoir via Wells 17-20. There are high levels of methane in the water from these wells, and this reservoir has been designed to strip methane from the wells. Water is supplied to the Glenwood Reservoir by Wells 26 and 27. Water is pumped from the Degas and Glenwood Reservoirs into the distribution system.

3.1.4 Pump Stations

The Degas pump house has three 9 HP pumps, each with a capacity of 14.2 L/s at 30 m of head.

The Glenwood pump house has one distribution pump and one fire pump. The distribution pump capacity is 7.6 L/s at 47 m of head, and the fire pump capacity is 48.6 L/s at 49 m of head. With a volume of 700 m^3 and pumping at a rate of 7.6 L/s + 48.6 L/s = 56.2 L/s, and conservatively neglecting incoming flows from Wells 26 and 27 (4.19 L/s combined), the Glenwood reservoir can provide about 3.5-hours of flow in the event of a fire.

The in-line booster station located at Edson Drive and 13 Avenue has three 10 HP pumps, each with a capacity of 13.3 L/s at 24.4 m head; and three larger 20 HP pumps, each with a capacity of 44.6 L/s at 24.4 m head. These pumps operate in parallel. The booster station was built in 1986 and has not been effectively used in recent years based on the Town's comments.

3.1.5 Distribution System Piping

The Town's distribution system consists of pipes varying in size between 100 mm to 350 mm in nominal diameter. The water distribution system is shown in Figures 3 and 3.1A -3.1C.The pipe materials consist of cast iron (CI), ductile iron (DI), asbestos cement (AC), steel, polyvinyl chloride (PVC), and high-density polyethylene (HDPE). Some of the cast iron pipes included in the 2011 MSP study have since been replaced by PVC pipes.

3.2 Distribution Pipe Assessment

3.2.1 General Pipe Condition from Data

Based on inventory data provided by the Town, some of the water pipes in the system were installed as early as 1931. Some of the pipes have exceeded their expected lifespan, or are nearing the end of their service life. Since 2000, approximately 101 instances of pipe breakages were recorded. Most of these breakages occurred in cast iron pipes. During subsequent repairs following breakages, the general poor condition of the cast iron pipes was confirmed. In general, the older pipes, especially the cast iron pipes, should be inspected in more detail in the near future, and a replacement program should be developed. GHD would suggest to eventually replace these pipes.

Table 3.3 Existing Pipe Upgrades due Poor Condition Assessment

Pipe Size (in mm)	Location	STREET	AVE	On	Pipe Type	Year	Length m	Unit Price $/m	Capital Cost
100	2nd Ave B/w 46th & 47th Street	46 - 47	2	Avenue	C.I.	1963	176	550	96,800
200	50th & 3rd Ave	50	3	Avenue	C.I.	1971	29	575	16,675
150	6930 - 4th Ave	69	4	Avenue	C.I.	1972	183	550	100,650
150	68th - 69th & 4th Ave Service Road	68 - 69	4	Avenue (Service Road)	C.I.	1972	186	550	102,300
150	7228 – 4th Ave (Service Rd.)	72	4	Avenue	C.I.	1972	160	550	88,000
150	Vacant lot beside 7025 4A Avenue	71	4A	Avenue	C.I.	1972	118	550	64,900
150	6615 - 5th Avenue	66	5	Avenue	C.I.	1972	141	550	77,550
150	42nd & 7th Avenue	42	7	Avenue	C.I.	1959	192	550	105,600
150	4214 - 7th Avenue	42	7	Avenue	C.I.	1959	168	550	92,400
100	4736 - 7th Avenue	47	7	Avenue	C.I.	1931	99	550	54,450
150	4310 - 7th Avenue	43	7	Avenue	C.I.	1959	147	550	80,850
100	5004 - 8th Avenue	50	8	Avenue	C.I.	1931	147	550	80,850
200	4416 - 8th Avenue	44	8	Avenue	C.I.	1964	90	575	51,750
100	45 Street and 8B Avenue	45	8B	Avenue	C.I.	1964	62	550	34,100
100	43A & 9th Avenue	43A	9	Avenue	C.I.	1965	177	550	97,350
100	50th & 9th Avenue	50	9	Avenue	C.I.	1931	197	550	108,350
100	50th & 9th Avenue	50	9	Avenue	C.I.		148	550	81,400
150	4326 - 9th Avenue	43	9	Avenue	C.I.	1965	52	550	28,600
150	5911 - 9 Avenue	59	9	Avenue	A.C.	1977	48	550	26,400
100	51st & 10th Avenue	51	10	Avenue	C.I.	1951	84	550	46,200
100	4829 - 10th Avenue	48	10	Avenue	C.I.		152	550	83,600
100	48th & 10th Avenue	48	10	Avenue	C.I.		136	550	74,800
100	49th & 10th Avenue	49	10	Avenue	C.I.		20	550	11,000

Table 3.3 Existing Pipe Upgrades due Poor Condition Assessment

Pipe Size (in mm)	Location	STREET	AVE	On	Pipe Type	Year	Length m	Unit Price $/m	Capital Cost
100	NE of 49th Street & 10th Avenue	49	10	Avenue	C.I.		20	550	11,000
150	NE hydrant 49th Street & 12th Avenue	49	12	Avenue	A.C.	1977	5	550	2,750
200	15th Ave and Edson Drive	Edson Drive	15	Avenue	A.C.	1978	14	575	8,050
100	4819 – 18th Avenue	48	18	Avenue	C.I.	1957	152	550	83,600
150	71st & South Glen Avenue	71	South Glen	Avenue	C.I.	1972	100	550	55,000
150	72nd & Glenwood Drive.	72	Glenwood Dr.	Avenue	C.I.	1972	140	550	77,000
100	43A & 4th Avenue	43A	4	Street	C.I.	1955	70	550	38,500
100	43A & 2nd – 3rd Avenue	43A	2 - 3	Street	C.I.		73	550	40,150
100	43A & 3rd Avenue	43A	3 - 4	Street	C.I.		70	550	38,500
100	714 & 44th Street	44	7	Street	C.I.	1964	111	550	61,050
150	827 - 45th Street	45	8	Street	C.I.	1964	44	550	24,200
150	46th Street and 6 -7th Avenue	46	6 - 7	Street	C.I.	1972	112	550	61,600
200	616 - 50th Street	50	6	Street	C.I.	1931	91	575	52,325
200	50th Street & 9 – 10th Avenue	50	9 - 10	Street	C.I.		98	575	56,350
100	52nd Street & 4th Avenue (North intersection + N-Side Alley)	52	4	Street	C.I.	1931	97	550	53,350
150	52nd Street and 7th Avenue	52	7	Street	A.C.	1974	70	550	38,500
100	53rd & 4th Avenue	53	4	Street	C.I.	1959	92	550	50,600
150	433 - 70th Street	70	4	Street	C.I.	1972	55	550	30,250
150	421 – 70th Street	70	4	Street	C.I.		60	550	33,000
150	South Glen Ave & 71st Street	71	South Glen	Street	C.I.	1972	75	550	41,250

Table 3.3 Existing Pipe Upgrades due Poor Condition Assessment

Pipe Size (in mm)	Location	STREET	AVE	On	Pipe Type	Year	Length m	Unit Price $/m	Capital Cost
150	South Glen-Park Glen & 72nd Street	72	South Glen - Park Glen	Street	C.I.	1972	106	550	58,300
150	72nd & South Glen Avenue	72	South Glen	Street	C.I.		110	550	60,500
						Total Length	4677	Sub-Total	$2,580,400
								Contingency (30 percent)	$774,120
								Engineering and Administration (10 percent)	$258,040
								Capital Cost	**$3,612,560**

3.3 Existing Water Consumption

The Town's water consumption record was used to develop existing baseline flows for residential and commercial users. Table 3.3 shows the Town's water consumption data from 2008 to 2016. Data from 2015 were excluded, as there are errors in the 2015 data.

Table 3.4 Total Historical Water Consumption (including Residential and Commercial)

Year	Total Water Consumption	Total Produced Water	Average Daily Demand (ADD)	Water Consumption (includes Residential and Commercial) per Person
	m^3	m^3	L/s	L/day/Capita
2008	1,032,992	1,259,582	32.76	343.09
2009	962,335	1,186,551	30.52	316.73
2010	993,968	1,272,667	31.52	324.21
2011	1,012,069	1,261,972	32.09	327.17
2012	1,063,623	1,232,824	33.73	344.33
2013	1,057,563	1,275,040	33.54	342.87
2014	1,199,199	1,294,117	38.03	389.35
2016	1,055,574	1,098,071	33.47	343.71
Average	**1,047,165**	**1,235,103**	**33.21**	**341.43**

The population values used in the calculation of per capita water consumption were based on the census populations provided in Table 2.1, and linear interpolations were made for intermediate years. The ADD and Water Consumption per Person values were based on the Total Water Consumption values, not the Total Produced Water values. The Total Produced Water values include water lost as a result of leakage, flushing, and other unmetered flows, which are not occurrences that should be factored into the current ADD.

Additional data regarding residential consumption and commercial consumption are presented in Table 3.4. As there were two months in 2014 where data were unavailable, the water consumption information for 2014 was excluded. It can be seen that the Town's commercial water consumption is somewhat greater than the residential water consumption. The average daily residential water consumption per person shown in Table 3.4 was calculated by using the Town's population for each respective year as described in the paragraph above. It is understood that the Town has some high commercial water consumption users. For clarity purposes, the Table shows only average water consumption values.

Table 3.5 Historical Water Consumption by Connection Type

Year	Total Residential Consumption	Total Commercial Consumption	Average Residential Water Consumption per Person	Average Water Consumption per Commercial/Industrial Area
	m^3	m^3	L/Capita/day	L/ha/day
2008	454,113	502,781	150.83	4,920
2009	469,680	420,982	154.58	4,119
2010	459,917	440,470	150.01	4,310
2011	439,134	490,511	141.96	4,800
2012	490,974	502,289	158.95	4,915
2013	431,160	519,200	139.78	5,080
2016	457,082	496,370	148.83	4,857
Average	**457,437**	**481,800**	**149.28**	**4,714**

Note that the existing commercial/industrial area is approximately 280 ha.

GHD identified that the recorded average residential water consumption values are much lower than the value in the design criteria presented in the 2011 MSP, which was 330 L/capita/day. As seen in Table 3.4, average residential water consumption has historically been closer to 150 L/capita/day.

3.4 Design Criteria

3.4.1 Demands

As shown in Table 3.3, the 2016 ADD (including residential and commercial flows) was 33.47 L/s, and the ADD from 2008 through 2016 was 33.21 L/s. For modeling existing conditions, the ADD was determined by rounding these values up to 34 L/s.

While existing demand is known and presented in units of L/s, future demands are presented in units of L/capita/day, as this makes it straightforward to project future flows based on estimated population growth.

Based on the design standards of similar communities in central Alberta, the average residential daily demands range from 320 L/capita/day to 450 L/capita/day. However, as shown in Table 3.4, residential average daily demand for Edson has historically been closer to 150 L/capita/day. Given this information, it is recommended that 300 L/capita/day be used as the design criterion for projecting future average residential demands in this study. While this value is not within the 320 – 450 L/capita/day range noted above, it is an appropriate conservative value to assume, given the historical data that has been presented. This is similar to consumption rates used in the design standards of other central Alberta communities.

The projected average consumption rate for future commercial users was assumed to be 10,000 L/ha/day. While Table 3.4 shows that historical commercial consumption rates in the Town have been closer to 5,000 L/ha/day, it was determined after gathering input from Town staff that estimating a future commercial water consumption rate of 10,000 L/ha/day would be appropriate. Because future industrial consumption is largely unknown, it is prudent to assume and plan for a more conservative consumption rate. This value is within the typical range of design standard flows for commercial users, and it is the same assumption that was used in the 2011 MSP.

Other design criteria for water demands from the Municipal Servicing Plan Update 2011 were applied to this study and are summarized in the following tables:

Table 3.6 Existing Water Demands

Parameter	Value
Average daily demand (ADD) – Residential and Commercial	34 L/s
Maximum daily peaking factor	2
Maximum daily demand(MDD) – Residential and Commercial	68 L/s (2 times ADD)
Peak hour factor	3
Peak hour demand (PHD)– Residential and Commercial	102 L/s (3 times ADD)

Table 3.7 Future Residential and Commercial Water Demands

Parameter	Value
Average daily demand (ADD) – Residential	300 L/capita/day
Maximum daily peaking factor - Residential	2
Maximum daily demand(MDD) – Residential	600 L/capita/day (2 times ADD)
Peak hour factor – Residential	3
Peak hour demand(PHD)– – Residential	900 L/capita/day (3 times ADD)
Average consumption rate – Commercial	10,000 L/ha/day
Maximum daily peaking factor – Commercial	2
Peak hour factor – Commercial	3

Table 3.8Fire Flow Requirement for the Existing Development Area

Parameter	Value
Single Family Residential	76 L/s
Multiple Family Residential	150 L/s
Hospital	265 L/s
Institutional Areas (i.e., schools)	130 L/s
Industrial and Commercial Areas	265 L/s

Table 3.9 Projected Fire Flow Requirement for Future Development Areas

Parameter	Value
Single Family Residential	100 L/s
Multiple Family Residential	180 L/s
Hospital	300 L/s
Institutional Areas (i.e., schools)	130 L/s
Industrial and Commercial Areas	300 L/s

The projected fire flow requirements presented in Table 3.8 are in accordance with Yellowhead County's standards.

3.4.2 Pressure

Common water system requirements include maintaining a minimum pressure of 280 kPa (40 psi) throughout the distribution system during peak hour demand. A residual pressure of 140 kPa (20 psi) is required above ground level during conditions when there is maximum daily demand plus fire flow. To maintain the integrity of existing pipes in the system, the maximum pressure should not exceed 790 kPa (115 psi).

3.4.3 Velocities

Pipes should typically be sized so velocities do not exceed 3 m/s in order to avoid excessive major head loss in the distribution system due to friction.

3.4.4 Hydrant Tests

Hydrant tests were recently conducted on December 11, 2017 to supplement the previous results that were completed by EPCOR in 2008 in order to properly calibrate our water model . Five locations were tested based on the dual-hydrant test procedure, similar to the 2008 tests described in the 2011 MSP. The hydrant test locations are presented in Figures 3.1A, 3.1B, and 3.1C. The results of the hydrant tests are shown in Appendix B.

3.5 2017 Water Model

The 2017 water model was developed in WaterCAD V8i software and is based upon the water model originally created in 1982 and subsequently updated in 2011 and 2013. For the 2017 model, the geographical data and physical conditions in the 2013 water model were used.

The previously existing models were evaluated and verified for correctness as part of creating the 2017 model. Numerous changes were made to the 2013 model. For ease of reference, the major changes are listed in the subsequent subsections.

3.5.1 Pipe, Pump, and Pressure Reducing Valve (PRV) Model Updates

The pipes, pressure reducing valves (PRVs), and pumps have been updated in the model to reflect real current conditions. Table 3.9 shows the updates that were made to the pipes in the model.

Table 3.10Updated Water Pipes in Water Model in 2017

Item Number	Pipe Replacement Location	Year Pipe was Replaced	New Pipe Material	New Pipe Diameter (mm)
1	42 Street (4 -6 Ave)	2014	PVC	250
2	5 Ave (42-46 St)	2014	PVC	150
3	5 Ave (46-50 St)	2015-2016	PVC	200
4	5 Ave (50-52 St)	2016	PVC	200
5	52 Street (5 -6 Ave)	2016	PVC	200
6	40 Street (41 Street -5 Ave)	2014	PVC	250
7	East of 40 St	2014	PVC	250
8	43 Street – 44 Street (15 - 18 Ave)	2012	PVC	200-300
9	17 Ave (Edson Dr-48 St)	2017	PVC	150
10	16 Ave (48-49 St)	2017	PVC	150-200
11	11 Ave (49-51 St)	2017	PVC	150-200
12	13 Ave (56-61 St)		PVC	200-300
13	Highway 16 (west of 73 St)	2014	HDPE	100

The new Well 21 pump and Well 25 pump were added to the 2017 model. Wells 26 and 27 that supply water to the Glenwood reservoir were added to the model. Wells 22 and 24 have been added in the 2017 model to replace Wells 15 and 14, respectively.

3.5.2 Water Demand Model Update

The existing average daily demand in the model (34 L/s) is considered to be equivalent to the existing average water consumption in the Town. As there is not enough data to determine the peaking factors for maximum daily demand and peak hour demand, a maximum day peaking factor of two and peak hour factor of three were applied to the model, which are the same as those used in the 2011 MSP. With these peaking factors, the existing maximum day flow and existing peak hour flow would be 68 L/s and 102 L/s, respectively. The demands in the 2017 model were updated accordingly.

3.5.3 Model Calibration

Model calibration by nature is a trial and error process of comparing model results with field results or hydrant test results. WaterCAD uses the Hazen-Williams equation to aid in the modeling process. The Hazen-Williams equation relates the flow of water in a pressure pipe with assumed physical properties of the pipe and the pressure loss in the pipe caused by friction. Hazen-Williams coefficients (C-values) are applied to individual pipes in the model, and these C-values are adjusted during model calibration to simulate the field results as closely as possible. Although C-values vary depending on pipe material and pipe condition for different pipes throughout the network, the calibration for the system is typically completed by using the same C-value for pipes of the same material.

C-values in pipe systems normally range from 90 to 140. New pipes and smooth-walled pipes typically have higher C-values, which result in less head loss due to friction. Corrosion inside older pipes typically leads to lower C-values, which result in more head loss due to friction. However, based on the Town's comments, the existing pipe interiors are not heavily corroded. The calibration effort included the use of C-values ranging from 120 to 130 for PVC and HDPE pipes, and from 90 to 120 for AC, Steel, CI, and DI pipes. The results of the five hydrant tests performed in 2017 were used to calibrate the water model. Table 3.10 shows the calibration results.

The water demands and the pumps that were running were not known at the time of the hydrant tests. The hydrant tests were conducted during the day. Tests were not conducted during a peak demand time, so the demands were assumed to be ADD at the time of the hydrant tests. As the hydrant flow would be the dominant flow during a fire event, the difference between the ADD and the actual domestic water demand during the hydrant tests would be insignificant and can be ignored.

Table 3.11 Model Calibration Results

No	Location	Hydrant Flow	Node Name in Model	Hydrant Test		Modeling Result		
							C=130, 105	C=130, 110
				Static Pressure	Residual Pressure at Test Hydrant	Static Pressure	Residual Pressure at Test Hydrant	Residual Pressure at Test Hydrant
		L/s		psi	psi	psi	psi	psi
1	Warehouse Scale Shack	101	J-2470	76	70	83.2	67.3	67.8
2	70 St/4 Ave	75	J-760	56	46	56.2	45	45.5

Table 3.11 Model Calibration Results

No	Location	Hydrant Flow	Node Name in Model	Hydrant Test		Modeling Result		
							C=130, 105	C=130, 110
				Static Pressure	Residual Pressure at Test Hydrant	Static Pressure	Residual Pressure at Test Hydrant	Residual Pressure at Test Hydrant
		L/s		psi	psi	psi	psi	psi
3	48 St/3 Ave	82	J-1710	75	70	75.6	70.4	70.9
4	Edson Dr./49 St	75	J-1176	48	46	51.5	44.2	44.7
5	44 St/18 Ave	84	J2015-5	61	52	63.6	54	54.4

Note: C=130, 110 means C=130 for PVC and HDPE pipes and C=110 for AC, DI. CI, and Steel pipes.

Static pressure was measured at each hydrant prior to releasing flow out of the hydrant. For determining the modeled static pressure at J-1176 in Zone 2, it was assumed that no pumps were running at the booster station, and the booster station bypass pipe in the model was set to open. For model calibration purposes, it was assumed that opening a hydrant would trigger one of the pumps at the Degas pump station and the fire pump at the Glenwood pump station to run for all of the hydrant tests. It was also assumed that opening the hydrant at the Warehouse Scale Shack would trigger the pumps at Wells 16 and 21 to run, and that opening the hydrant at Edson Drive and 49 Street would trigger one of the booster pumps to run.

The calibration results show that the modeled residual pressures using C=130 for PVC and HDPE pipes and C = 110 for AC, DI, CI, and steel pipes are similar to the actual pressure readings recorded during the hydrant tests. These C-values were therefore used in the 2017 model.

3.6 Existing Water System Modeling Study

3.6.1 Existing Peak Hour Demand

The Zone 1 hydraulic grade line is set at 966.06 m by the Grande Prairie Trail reservoirs, so they can supply pressures at or above 280 kPa (40 psi) to water users at or below an elevation of 939.3 m. As a result, water users at elevations above 939.3 m in Zone 1 do not have the minimum pressures required in their water supply. This is the case in the northwest area of Town, specifically areas northwest of 13 Avenue.

The existing peak hour demand scenario was simulated by the 2017 model. This simulation indicated that the existing system can provide adequate flows and pressures to the majority of the Town. The northwest area of the Town where ground elevations are above 939.3 m could expect insufficient water pressures (below 280 kPa, 40 psi). The highest pressure in the system is 94.6 psi on 1 Avenue approximately 200 m west of 27 St. The lowest pressure is 92 kPa (13.3 psi) at 17 Avenue and 63 Street. The modeling results for this scenario are shown in Figure 3.2. This figure shows pressure contours so that the areas of the system that do not meet the minimum pressure requirement of 280 kPa are made apparent. Potential options for addressing the pressure issues in the northwest area of Town are presented in Section 3.6.5.3.

3.6.2 Maximum Daily Demand plus Fire Flow

As pipe sizes of any distribution system are typically determined by fire flow requirements, the existing maximum daily demand plus fire flow scenario was also modeled. The pressure requirement for the MDD plus fire flow scenario is 140 kPa (20 psi), as compared to the requirement of 280 kPa (40 psi) for the PHD scenario.

For this existing scenario, fire demands were assigned to nodes in the model based on the requirements given in Table 3.7. WaterCAD V8i software performs the following analysis when determining the ability of the various nodes to provide the required fire flows:

1. A steady-state simulation using non-fire demands is run at all nodes that are designated as fire flow nodes within the model to ensure that the fire flow constraints that have been set (e.g. minimum residual pressure) can be met without any fire flow demand from any of the nodes. For example, if the residual pressure at one of the nodes is below 140 kPa (20 psi) during this first part of the analysis, the model will say that this node does not meet the fire flow constraints.
2. If the fire flow constraints were met in the initial run, a series of steady-state runs are performed in which the fire flow designated for each individual node is evaluated using a separate analysis. After beginning at an initial maximum pre-set value (set at 360 L/s for this analysis), the software iteratively assigns lesser demands to each node to determine the available fire flow, which is the maximum fire flow that each node can supply without violating the set fire flow constraints.

The modeling results in Figure 3.3 show the areas of the system where fire flow constraints (including flow and residual pressure) are met, as well as areas where fire flow constraints are not met. The red nodes are locations where the required fire flow cannot be fully supplied. Upgrades will need to be performed on the water distribution system so that the requirements for fire flows can be met. This could include some trunk loops and pipe size upgrades, as further discussed in Section 3.6.5.4.

3.6.3 Reservoir Storage Requirement

Based on Alberta Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems (2012), total storage for the system should meet or exceed the following:

Table 3.12 Reservoir Storage Requirements

Parameter	Value
Fire storage	265 L/s at 3h = 2,862 m^3
Equalization storage (25 percent of MDD)	25 percent of 68 L/s over 24h = 1,469 m^3
Emergency storage (15 percent of ADD)	15 percent of 34 L/s over 24 h = 441 m^3
Total	4,772 m^3

The total existing reservoir storage is 6,530 m^3 and therefore fulfills the storage requirement for the Town under existing conditions.

3.6.4 Well Flow Supply Requirement

The eleven existing active wells (excludes Wells 3, 16, and 21 as discussed in Section 3.1.1) have an estimated combined total production capacity of 55.52 L/s and cannot supply enough flows to accommodate the Town's MDD (68 L/s). The Town plans to add two new wells (Wells 28 and 29) in 2017. If each new well can produce 7 L/s of flow, the total Well production capacity will increase to 69.52 L/s, which would meet the Town's MDD (68 L/s) requirement. Wells 16 and 21 are normally used in fire events only. Additionally, Well 3 (allowed discharge of 6.44 L/s) could be put back into use to provide supplemental capacity after the installation of a new pump, disinfection facility, and connecting the well to the distribution system. Model results show that bringing Well 3 back into service does not provide any significant benefits with regard to pressure or fire flow, as the hydraulic grade line is controlled by the Grande Prairie Trail reservoirs. However, one benefit of bringing Well 3 back online, aside from the additional production from the well, is the fact that it would provide added chlorine residual in the northern part of the Town. The majority of the existing wells are in the southern part of the Town.

Level sensors should be installed in each well to monitor water levels at the wells to provide the Town with additional well performance data.

3.6.5 Suggested Distribution System Improvements under Existing Conditions

Based on the results of the water model, the following potential upgrades to the existing distribution system were developed and analyzed as follows:

3.6.5.1 Elimination of the Existing Booster Station

The existing booster station at Edson Drive and 13th Avenue currently provides additional pressure to existing Zone 2 in the Edson Drive area of the system. Modeling was performed to evaluate whether or not the existing booster station is required to meet minimum pressures and fire flow requirements. If the pumps at the booster station are turned off in the model, the lowest pressure in the area served by the booster station is at junction (J-5894) in the model, near the intersection of 53rd Street and 18th Avenue. The modeled pressures at this node were 43.3 psi and 40.9 psi during the base demand and PHD scenarios, respectively, meaning that PHD pressure requirements are met. In our water model, all the requirements in this vicinity were generally met for the MDD + fire flow case as well with the booster station not running except the node at junction (J-1120) near 55th Street and 14A Avenue. The model showed that a flow of 73.4 L/S could be achieved at this particular node while maintaining a pressure above 20 psi. If 76 L/s of flow is provided to this node, the pressure at the node drops to 18.5 psi. While this is below the required 20 psi, it is marginally below the requirement, and the booster station could be abandoned. All existing and future improvement scenarios presented in this document were modelled assuming that the existing booster station is not in use. It is recommended that the Town abandon the existing booster station. The cost for demolishing the booster station is estimated at $70,000 (see Appendix A).

3.6.5.2 Addition of a New 250 mm Northwest Water Distribution Loop

There are many advantages to having loops in a water system. Loops allow for minimal isolation of pipelines during repairs so that systems can remain in service. Loops also eliminate the problem of dead ends in water lines, which need to be flushed periodically by maintenance staff due to stagnation of water and low chlorine residual. Loops also increase firefighting availability by not limiting flows to one water main, and they similarly maintain pressure throughout a system, as water reaches all points within a loop through various paths, which minimizes head loss.

The area of the Town bounded by 17th Avenue, 66th Street, 22nd Avenue, and 63rd Street is not currently being served by the Town's water distribution system. This area is shown as Future Zone 3 in Figure 3.4. It is recommended that a new 250 mm pipe loop be constructed to serve this area. This general upgrade was also recommended in the 2011 MSP. This area would need to be served directly by a booster station as discussed in Section 3.6.5.3, as it is too elevated to be served by gravity from a reservoir in the area. A new reservoir at the Microwave Tower site is a proposed for future upgrade for 2032 (refer to Section 3.7 below) .This proposed reservoir would not be elevated enough to provide adequate pressure to this northwest loop area. The total cost for constructing this proposed loop including contingency (30 percent) and engineering (10 percent)is estimated at $1,866,900 (see Appendix A).

3.6.5.3 Recommendations based on Peak Hourly Demand (PHD) Scenario

As discussed in Section 3.6.1, the system is currently not providing adequate pressure (minimum 280 kPa, 40 psi) to the northwest area of town during the PHD scenario. This area is in pressure Zone 1, served by the Grande Prairie Trail reservoirs, and the elevations in this area are too high relative to the elevation of the reservoirs for adequate pressures to be provided. Two improvement options (see Options 1 and 2 in Figure 3.4) have been considered to address these pressure issues.

Option 1: Upgrade the Existing Booster Station

Option 1 would include using the existing booster station that feeds the pressure Zone 2 area to provide additional pressure to the area northwest of 13th Avenue. A 1.8 km, 350 mm diameter supply pipe was analyzed to connect the existing Zone 2 area to this part of existing Zone 1. This 350 mm pipe would connect to the existing 300 mm pipe at Edson Drive and 48th Street, and then flow westward to the north of 18th Avenue, and tie in to the proposed pipe loop discussed under Option 2 below near 17th Avenue and 63rd Street that is discussed under Option 2 below. The piping upgrades discussed under Option 2 (shown in orange on Figure 3.4) would also be required for Option 1. In the future, the booster station would need to be upgraded and could supply water to future development areas in Zone 2 with ground elevations between 926 m and 960 m. A detailed study would need to be completed to confirm the pipe sizes for future developments.

Option 2: Construct a New Pump Station at Grande Prairie Trail

Pressure Zone 1 is currently fed by gravity via the Grande Prairie Trail Reservoirs. A proposed pump station at the Grande Prairie Trail site could draw water from the reservoirs and supply flows and pressure to the area in Zone 1 northwest of 13th Avenue that is currently not receiving adequate pressure (shown as future Zone 2A in Figure 3.1). The booster station should have four pumps that each provide 100 L/s at 45 m of head, with one pump as a backup. This proposed booster station could also supply flow and increase pressure to the future pressure Zone 3 area (ground elevations of approximately 960 m to 975 m) in the northwest area of Town. Utilizing piping upgrades required for future developments and a new reservoir at the Microwave Tower site in the future (reservoir hydraulic grade line of 980 m, discussed in Section 3.7)), this pump station could also provide pressure to the existing Zone 2 area currently served by the existing booster station. This new booster station could also serve future development areas in the northern part of Zone 2. The addition of a reservoir at the Microwave Tower site and the connection of the new booster station to the Zone 2 area would occur in future proposed upgrades in 2032 as discussed in Section 3.7.

The proposed Grande Prairie Trail booster station would connect to a new piping loop that serves future Zone 2A (see the orange piping in Figure 3.4) to provide adequate pressure to this area. Piping from the new pump station would connect to the new loop near 63rd Street and 17th Avenue. The loop would consist of a new 350 mm diameter pipe along 63rd Street, new 200 mm pipe along 13th Avenue, and new 200 mm pipe from there that connects to the existing piping at the end of the cul-de-sac on 62nd Street. In addition to this loop, a 350 mm branch would be installed along 63rd Street from 13th Avenue to 12th Avenue, and a 200 mm branch would be installed along 61st Street from 13th Avenue to 12th Avenue. The areas that these new branches would serve are not currently receiving adequate pressure.

The 200 mm portion of this new loop was sized to allow for adequate fire flows in this area. The proposed 350 mm diameter pipe was sized to allow for future connections that would serve future development areas, which require a 300 L/s fire flow.

While there are existing water mains on 63rd Street and 13th Avenue, new parallel pipes would be required in these locations to keep this new pressure Zone 2A area separate from Zone 1. Connecting the new Grande Prairie Trail, booster station to the existing piping, as opposed to constructing new parallel pipelines, was considered. The 2011 MSP seemed to recommend connecting the proposed booster station directly to the existing supply/fill line; however, the details of the recommendation were incomplete. The Grande Prairie Trail reservoirs do not have a dedicated fill line. The reservoir levels fill during times of low demand and drop during times of high demand. For the Grande Prairie Trail booster station to provide higher pressures to Zone 2A with a connection to the existing piping, the pumps would have to draw water from the reservoirs and would boost pressures in the common supply/fill lines higher than the hydraulic grade line of the Grande Prairie Trail reservoirs. In this case, the Grande Prairie Trail reservoirs would never be able to fill. To boost pressures to Zone 2A, it would be necessary to have a new piping loop that is served by the new booster station and not hydraulically connected to the Grande Prairie Trail reservoirs.

The advantages and disadvantages of Option 1 and Option 2 are shown in Tables 3.12 and 3.13, respectively.

Table 3.13 Advantages of Option 1 and Option 2

	Advantages of Option 1	Advantages of Option 2
1	The existing booster station can be used to supply flows to existing Zone 2, future Zone 2A, and future Zone 2, without requiring the construction of a new booster station.	Installing a new pump station to supply water to the northwest area would be logistically easier than upgrading the existing booster station, as it would not affect the operation of the existing water system.
2		A power generator could be easily installed in the proposed pump station to provide the backup power for the fire pump.
3	The new 1.8 km long 350 mm diameter supply pipe would also provide new capability to serve projected future developments in this area.	It would be easier to maintain the new off-line pump station than the existing inline booster station that draws water directly from the distribution system.
4		The proposed pump station could be designed to accommodate future expansions.
5		It could supply future Zone 2, future Zone 2A, and future Zone 3.
6		It was recommended in both previous reports, i.e. the General Engineering Study (1982) and Municipal Servicing Plan Update (2011).
7		This Option would be more cost effective than Option 1.

Table 3.14 Disadvantages of Option 1 and Option 2

No.	Disadvantages of Option 1	Disadvantages of Option 2
1	The existing booster station was built more than 30 years ago. The mechanical equipment (e.g. pumps) in a booster station should typically be upgraded after 25 years. Similarly, upgrades of the electrical parts could also be required. Such upgrades are expected to be expensive.	
2	The existing booster station has limited available space, and it would be impractical and/or impossible to add more pumps or install bigger pumps inside the existing booster station. Similarly, there does not appear to be enough space available to add a new backup generator.	
3	The inline booster station affects the pressures in the distribution system during the pumps' start and stop processes. Additionally, the pressures in the distribution system upstream of the booster station could also impact the booster station operation.	

No.	Disadvantages of Option 1	Disadvantages of Option 2
4	This Option would have more hydraulic losses while supplying water to the future Zone 2A compared to Option 2, as the existing booster station is farther from future Zone 2A.	
5	This Option is more expensive than Option 2.	

For cost comparison purposes, it was assumed that the existing booster station would be upgraded to have a new generator and could supply 300 L/s of fire flow in Option 1. A 1.8 km, 350 mm diameter supply pipe would be installed north of 18th Avenue in Option 1 to supply flow to the Town's northwest area. Option 2 would involve constructing a new pump station with a new generator. The proposed piping (shown in orange) in Figure 3.4 would be constructed for both Option 1 and Option 2. Table 3.14 shows the cost estimates for both options.

Table 3.15 Approximate Costs of Option 1 and Option 2

	Cost of Option 1	Cost of Option 2
	$4,600,000	$3,200,000

Cost breakdown details of these two options are shown in Appendix A.

Given the above advantages and disadvantages, Option 2 is recommended at a cost including contingency (30 percent) and engineering (10 percent) of approximately $3.2 million.

3.6.5.4 Recommendations Based on Existing MDD plus Fire Scenario

As shown on Figure 3.3, there are areas of the existing system where the fire flow constraints are not met (see the red nodes in Figure 3.3). Six general areas in the existing system have fire flow deficiencies that have been identified and should be resolved. The deficient locations and associated recommended upgrades for improving fire flow supply (see Figure 3.4) are as follows:

1. 63rd Street and 17th Avenue area (northwest portion of the system)
   - The fire flow issues in this part of the system could be solved by constructing Option 2 as described in Section 3.6.5.3.
2. 8th Avenue and 49th Street area (near the center of the system)
   - This is a localized fire flow issue that could be addressed by upgrading existing pipe sizes in the area as shown on Figure 3.4.
3. 1st Avenue from 42nd Street to 45th Street (southeast portion of the system)
   - This is a localized fire flow issue that could be addressed by upgrading existing pipe sizes in the area as shown on Figure 3.4.
4. 4th Avenue and 70th Street area (southwest portion of the system)

- The fire flow issues in this area could be resolved by upgrading the size of a portion of the existing pipe along 4th Avenue.

5. Southwest portion of the Town – 9th Avenue and 63rd Street to 73rd Street and Highway 16

   - There are generally fire flow issues in the southwest part of town. These issues could be solved by extending the 350 mm pipe along 63rd Street in the northwest part of the system down to the southwest portion of the system. The extension of this pipe has the added benefit of serving future development areas, as discussed further in Section 3.7. There would be an additional 350 mm branch off of this 350 mm pipe that connects to the existing system along Glenwood Drive, creating an additional loop in the system to increase fire flow availability in this area. A 350 mm pipe size was selected to allow for the required 300 L/s fire flow (see Table 3.8 for future fire flow requirements). A 300 L/s flow in a 350 mm pipe would have a velocity of approximately 3.1 m/s, which is generally a maximum velocity target (see Section 3.4.3) for minimizing head loss within a pipe.

     i. An attempt was made in the model to eliminate this new 350 mm pipe. In lieu of the new 350 mm pipe, a new 300 mm line was placed north of the existing 300 mm main along Highway 16. These pipes were connected and looped with the 100 mm line on 4A Avenue west of 72nd Street, but this Option did not resolve the fire flow issues in this area.

6. 1st Avenue from 27th Street to 42nd Street (southeast portion of the system)

   a. This area is currently essentially served by one water supply trunk (see Figure 3.1C). This issue could be resolved by connecting a new 300 mm pipe to the existing system at 42nd Street and 2ndAvenue, and extending the new pipe to the southwest, along Highway 16, and tying in to the existing piping at the 1st Avenue and 27th Street area, thereby creating a loop in the system.

These upgrades should be completed in the same sequence given above. The proposed upgrades would improve the distribution system under existing conditions as Well as under future scenarios.

Table 3.15 shows the upgrade costs by location as per the detailed breakdown provided in Appendix A. Some local deficiencies are shown in the modeling results. The deficiencies are identified in branch pipes or in pipes that are missing loops. Despite the local deficiencies, fire flows can be supplied from the mains nearby. These pipes are deemed acceptable without requiring upgrade. Additionally, some of the fire flow deficiencies are resolved with future looping that is recommended in future upgrades. Other red nodes are on supply lines that are not responsible for providing fire flow.

The estimated cost for upgrades to the existing distribution system to address fire flow requirements, including contingency (30 percent) and engineering (10 percent) are summarized in Table 3.15 below.

Table 3.16 Upgrade Costs for Existing System for Fire Flow Requirements

Location	Cost
63 Street and 17 Avenue area	$3,119,200*
8 Avenue and 49 Street area	$802,550
1 Avenue from 42 to 45 Street	$610,400
4th Avenue and 70th Street area	$593,600
9 Ave and 63 Street to 73 Street and Hwy 16	$3,967,250
1 Avenue and 27 to 42 Street area	$1,298,150
Total	**$10,391,150**

*The upgrade for the 63rd Street and 17th Avenue area is the same as that recommended for the Existing PHD Improvement Scenario as discussed in Section 3.7.5.3.

Typically, 150 mm to 200 mm diameter pipes are required to supply sufficient residential fire flows. Similarly, 300 mm to 350 mm diameter pipes are required to supply sufficient commercial/industrial fire flows. The existing 100 mm diameter CI pipes in the residential areas should be replaced by 150 mm diameter PVC pipes in the near future.

After implementation of the proposed improvements, the available fire flows can be effectively increased in the areas where fire flows are not currently meeting fire requirements. Figure 3.5 shows the MDD plus fire flow modeling results after the improvements are completed.

A cost summary of the suggested upgrades for the existing water system is provided in Table 3.16 below.

Table 3.17 Upgrade Costs for Existing System

Location	Cost
Option 2	$2,228,000
Proposed 520 m of 300 mm diameter watermain at 4th Avenue and 70th Street Area	$424,000
Upgrade Existing 750 m of 100 mm with 150 mm pipe at 8 Avenue and 49 Street Area	$573,250
Upgrade Existing 530 m of 200 mm diameter pipe with 300 mm diameter Watermain & Rehabilitate Road to Existing Conditions at 1 Avenue from 42 Street to 45 Street	$436,000
Proposed 1205 m of 300 mm diameter Watermain & Rehabilitate Road to Existing Conditions at 1 Avenue from 27 Street to 42 Street	$927,250
Proposed 3500 m of 350 mm Watermain at 9 Avenue and 63 Street to 73 Street/HWY 16	$2,833,750
Demolish Existing Booster Station at Edson Drive and 13 Avenue	$50,000
Proposed 1800 m of 250 mm diameter at Northwest Loop	$1,333,500
Sub-Total	$8,805,750
Contingency (30 percent)	$2,641,725
Engineering and Administration (10 percent)	$880,575
Capital Cost	**$12,328,050**

3.7 Future Water Modeling Study

Future scenarios were modelled for this study using projected populations and future developments to determine upgrades that will be required for the Town to provide adequate water production capacity, storage, pressure, and fire flow for existing and future water customers. Separate scenarios were modelled for development into the future; one scenario assumed the Town would serve future growth and development within Town limits only, and the other assumed the Town would serve future development areas within the Town and County fringe areas. For each of these scenarios, 15- and 25 year projections were used to develop model scenarios and system upgrade recommendations for years 2032 and 2042, respectively.

3.7.1 Future Servicing within the Town Limits

This Section provides information on water modeling results that incorporated projected future growth and development within Town limits, as Well as recommended upgrades and associated cost estimates.

3.7.1.1 2032 Study – Town Only

3.7.1.1.1 2032 Average Flow – Town Only

By 2032, the Town's population is projected to increase to 9,867 people, and water services are projected to expand to the areas shown in Table 2.5 and Figure 2.2. At this point, the existing water system should have been improved as recommended in Section 3.6.5. For the future scenarios, projected flow calculations are based on the future design standards shown in Tables 3.6, 3.7, and 3.8. The fire flow requirements in the existing development areas continue to follow the existing standards (Table 3.7), while the fire flows in the future development areas follow the future design standards (Table 3.8). The future flow requirements have been intentionally set at a more conservative level to ensure that the future upgrade plans will not be underestimated. Before the future upgrades are actually implemented, a reality check based on the water consumption records and a hydrant test update would be prudent.

The average residential demand in 2032 is projected to be 9,867 people x 300 L/capita/day = 34.26 L/s. The average non-residential flow is projected to be (280 ha existing non-residential area + 46 ha future addition of non-residential area) x 10,000 L/day/ha = 37.73 L/s. Therefore, the total average flow within the Town would be 34.26 L/s + 37.73 L/s = 71.99 L/s, the maximum daily demand (MDD) would be 2 x 71.99 L/s = 143.98 L/s, and the peak hourly demand (PHD) would be 3 x 71.99 L/s = 215.97 L/s.

3.7.1.1.2 2032 Reservoir Storage – Town Only

For existing conditions, fire storage was calculated as 265 L/s x 3 hours = 2,862 m^3. However, for future scenarios, the required fire flow has been increased to 300 L/s, and fire storage is calculated as 300 L/s x 3 hours = 3,240 m^3. The total reservoir storage requirement in cubic metres (m^3) for the

Town in 2032 = Fire storage (3,240 m^3) + Equalization storage (25 percent of MDD = 3,110 m^3) + Emergency storage (15 percent of ADD = 933 m^3) = 7,283 m^3.

The total existing reservoir storage is 6,530 m^3. Consequentially, if the projections are accurate, approximately 860 m^3 of additional storage will be required by 2032. However, as discussed in the 2042 Town only scenario (Section 3.7.1.2.2), approximately 7,720 m^3 of storage is projected to be required by 2042 (approximately 1,190 m^3 greater than the existing system storage). If a new reservoir is built to accommodate the 2032 storage requirements, it would be prudent to consider constructing a 1,200 m^3 reservoir that would satisfy the projected 2042 requirements as well. This new reservoir should be constructed at the Microwave Tower site, as discussed further in the 2032 Town Only MDD plus fire flow scenario (Section 3.7.1.1.4).

3.7.1.1.3 2032 Well Capacity – Town Only

The eleven existing active wells (excludes Wells 3, 16, and 21 as discussed in Section 3.1.1) have an estimated combined total production capacity of 55.52 L/s. The total well capacity in 2032 is projected to be the summation of the capacity of these eleven wells (55.52 L/s) + Well 3 (6.44 L/s) + Well 28 (7 L/s) + Well 29 (7 L/s) = 75.96 L/s. It was assumed Wells 28 and 29 were installed outside of this MSP effort, and that Well 3 would be rehabilitated (new pump, disinfection facility, and connection piping) prior to 2032 (though a cost for rehabilitating Well 3 is included in the 2032 cost estimate in Appendix A). To provide for the Town's 2032 MDD (143.98 L/s), 10 additional wells would be required, given the projected demands, assuming that the average production out of each new well will be 7 L/s. This would bring the total well production value up to 145.96 L/s.

The actual water demand for the total development area should be monitored to determine the number of wells required for supply. For planning purposes, and to be conservative, it was assumed that four new wells (Wells 30 through 33), which are assumed to provide a combined capacity of 28 L/s, will be added into the system by 2032. For modeling purposes, it was assumed that the additional well capacity of 28 L/s will be added to the system at 63rd Street and 8th Avenue.

3.7.1.1.4 2032 MDD Plus Fire Flow Modeling Study and System Upgrades – Town Only

Given the projected developments and population growth, additional upgrades will be required to the system to provide adequate fire flows in 2032. Assuming that the upgrades recommended in Section 3.6.5 are completed by 2032, pressure issues in the system during PHD would largely be resolved. Aside from storage and production capacity of the system, the driver behind the analysis of future scenarios is ensuring that the system can provide adequate pressure and flow during the MDD plus fire flow scenario. The proposed upgrades that are discussed in this Section are shown in Figure 3.7A.

For modeling the 2032 MDD plus fire flow scenario, it was assumed that the water system will supply water flow to future residential development areas 1, 2, 5, 6, 8, 13, and 14 (see Figure 2.2). The town will need to provide water mains to supply these areas, and developers will need to connect 150 to 200 mm diameter pipes to the Town's water mains and loop them so that adequate fire flow can be supplied to these residential areas. It was also assumed that water flow will be supplied to future commercial and industrial areas 3, 4, 7, 9, 10, 11, 12, and 15 during 2032. Upgrades will need to be made to the system to serve these areas and to supply enough fire flow plus MDD flow.

The addition of development areas 4, 10, and 11 causes fire flow shortfalls in the southwest portion of the Town. These issues could be resolved in one of two ways:

Option 1: Install a new booster station near the Degas Reservoir and construct a new reservoir at the site that is sized to serve fire flow to the southwest portion of the Town. The assumption is made that the existing booster station could not be adequately expanded or upgraded to serve this purpose, and a new booster station would be required. The Existing Degas Reservoir could not be expanded, as it is very small and is designed to strip methane from the water from Wells 17-20.

Option 2: Install a new approximately 5 km long, 500 mm pipe from the Grande Prairie Trail booster station to the Zone 2 area of the southwest part of Town. A pressure reducing valve (PRV) would be required to connect the Zone 2 network to Zone 1. This 500 mm pipeline could get pressure from the Grande Prairie Trail booster station and supply the fire flow to the Town's future commercial areas in the southwest and the County's future fringe area.

Option 1 is recommended, as Option 2 would require extensive pipe upgrades to supply adequate fire flows to the west areas, as discussed in the 2011 MSP. Installing the required 5 km long, 500 mm pipe would be very expensive.

Under Option 1, modeling results showed that approximately 115 L/s would be pumped from the new Degas reservoir in the event of a fire in the southwest Town. Given this, the reservoir should be sized to accommodate 115 L/s for 3 hours = 1,250 m^3. It should be noted that the capacity of the new Degas reservoir was not factored into the reservoir capacity requirements discussed in this document. This is to be conservative, and due to the fact that separate storage for fire flows is required for the southwest part of Town and the rest of Town. Since the new Degas reservoir was sized just to provide fire flows to the southwest part of Town, the new Degas reservoir should not be included when determining overall system storage requirements.

It should also be noted that the additional storage at the Microwave Tower site does not improve upon fire flows in the southwest portion of the Town. This reservoir is too far away, and after construction of this reservoir, the extensive piping upgrades would still be required to provide adequate fire flow to the southwest if the new reservoir and booster station were not constructed at the Degas site.

The new Degas booster station should have two pumps capable of pumping 40 L/s at 30 m of head. These pumps would operate in a lead/lag fashion, and both would turn on in the event of a fire. It should be noted that a fire in the southwest Town causes pressures to drop, and these pumps together end up pumping the 115 L/s discussed above.

The development areas in the northern part of the Town could be served by either: 1) upgrading the Grande Prairie Trail booster station, or 2) constructing a new 1,300 m^3 reservoir at the Microwave Tower site. The new Microwave Tower reservoir would operate similarly to the existing Grande Prairie Trail reservoirs; it would fill during times of low demand, and levels would lower during normal operation. A new reservoir at the Microwave Tower site is recommended for multiple reasons. As discussed in Section 3.7.1.1.2, additional system storage will be required by 2032 regardless, so adding this storage at the Microwave Tower reservoir (ground elevation of 975 m, assumed hydraulic grade line of 980 m) would be prudent. This reservoir could serve all of Zone 2, including the area that is served by the existing booster station at Edson Drive and 13th Avenue, as well as future development areas to the north. In addition to providing new storage, this reservoir would minimize pumping costs as well as costs associated with upgrades to the Grande Prairie Trail booster station. However, a new pump at the Grande Prairie Trail booster station would be required to pump to the Microwave Tower reservoir; this pump should be sized to pump 300 L/s at 15 m head. This provides just enough head to get from the Grande Prairie Trail reservoirs to the Microwave Tower reservoir, and allows Zone 2 to keep up with a fire flow demand of 300 L/s. A pressure reducing valve would be required at Edson Drive and 13th Avenue to separate Zone 1 from Zone 2.

A new 730 m long, 350 mm diameter main would be required from the Grande Prairie Trail booster station to the Microwave Tower reservoir. A new 1620 m long, 350 mm main would be required to serve new development areas north of 18th Avenue. Distribution system piping would tie to these mains as well as the Microwave Tower reservoir.

Other new mains along 40th Street (350 m of 350 mm diameter pipe) and in the east industrial area (1,440 m of 350 mm diameter pipe) would be required to serve new development areas.

The 100 mm diameter CI pipes are too small to provide adequate fire flow. Consequently, the existing 100 mm diameter mains from 8th Avenue between 50th Street and 51st Street to 9th Avenue between 49th Street and 51st Street should be upgraded to 150 mm diameter PVC. The existing 100 mm diameter pipe from 10th Avenue between 48th Street and 51st Street should be upgraded to 150 to 200 mm diameter PVC.

After the water system is upgraded and improved, water to the development areas can be effectively supplied. Figure 3.6A shows the proposed fire flow supply configuration for the 2032 scenario.

The remaining 2032 development areas that have not been discussed will be served by existing mains or the mains discussed above, with loops installed by developers as shown on Figure 3.7A.

3.7.1.1.5 2032 Cost Estimates – Town Only

The upgrades for the 2032 Town only water system is listed in Table 3.17 below and sections to be upgraded are shown on Figure 3.7A.

Table 3.18 Upgrade Costs for 2032 Town Only System

Location	Cost
Proposed 1250 m^3 Reservoir and proposed booster station at Degas Site	$1,425,000
Proposed 1300 m^3 Reservoir at Microwave Tower (MT) Site and new pump at the GPT station	$857,500
Proposed 730 m of 350 mm diameter watermain from GPT to MT Site	$592,500
Proposed 1620 m of 350 mm diameter watermain along the North of 18 Ave	$1,313,000
Proposed 350 m of 350 mm diameter watermain Along 40 st	$296,500

Table 3.18 Upgrade Costs for 2032 Town Only System

Location	Cost
Proposed 1440 m of 350 mm diameter watermain In East Industrial Area	$1,181,000
Proposed 510 m of 150 mm diameter watermain 10 Ave from 48 Street to 51 Street	$396,000
Proposed 350 m of 150 mm diameter watermain 9 Ave from 49 Street to 51 Street	$272,500
Proposed 165 m of 150 mm diameter watermain 8 Ave from 50 Street to 51 Street	$133,750
Proposed 50 new water services 10 new Wells with pumps	$1,060,000
Sub-Total	$7,527,750
Contingency (30 percent)	$2,258,325
Engineering and Administration (10 percent)	$752,775
Capital Cost	**$10,538,850**

3.7.1.2 2042 Study – Town Only

3.7.1.2.1 2042 Average Flow – Town Only

The Town's population is projected to increase to 10,902 people in 2042, an increase of 1,035 people from the 2032 population, and water services are projected to be expanded to the areas shown in Table 2.5 and Figure 2.2. The 2042 Town only residential flows inputted into the model were 10,902 people x 300 L/capita/day = 37.85 L/s. Commercial flow within the Town is projected to increase by 4.11 L/s to 41.84 L/s. The total average flow provided by the Town would be increased to 79.69 L/s. The MDD would be 2 x 79.69 L/s = 159.38 L/s, and the PHD would be 3 x 79.69 L/s = 239.07 L/s.

3.7.1.2.2 2042 Reservoir Storage – Town Only

The total projected reservoir storage requirement for the Town in 2042 = Fire storage (3,240 m^3) + Equalization storage (25 percent of MDD = 3,443 m^3) + Emergency storage (15 percent of ADD = 1,033 m^3) = 7,720 m^3. The total reservoir storage assuming the 1,200 m^3 upgrade recommended in Section 3.7.1.1.2 (2032 Town only case) is completed will be 7,730 m^3, and no additional storage would be required in 2042.

3.7.1.2.3 2042 Well Capacity – Town Only

Assuming all of the wells discussed in the 2032 Town only scenario are installed (Wells 28 and 29, rehabilitation of Well 3, and 10 additional wells), the system total well capacity would be 145.96 L/s. This capacity is insufficient to provide for the Town's 2042 MDD (159.38 L/s). Two additional wells at 7 L/s each would be required.

The MDD is calculated based on the design standards. The actual water demand for the total development area should be monitored to determine the number of wells required for supply. To be conservative, it was assumed that three new wells (Wells 34, 35, and 36) providing an additional combined capacity of 21 L/s will be added to the system by 2042, in addition to the four new wells assumed in 2032 (Wells 30-33). For modeling purposes, it was assumed that the additional well capacity will be added to the system at 63rd Street and 8th Avenue.

3.7.1.2.4 2042 MDD Plus Fire Flow Modeling Study and System Upgrades – Town Only

For modeling the 2042 MDD plus fire flow scenario, it was assumed that the water system will supply water flow to the future residential development areas 16, 17, 18, 19, 20, and additional development within areas 6 and 8, in addition to the development areas served in 2032. Developers will need to loop their water systems by using 150 to 250 mm diameter pipes in residential areas so that enough fire flow can be supplied.

Recommended upgrades for this scenario are shown on Figure 3.9A.

The new Degas reservoir and booster station from the 2032 Town only case does not require upgrades for the 2042 Town only case.

New development areas to the north will continue to be served by the Microwave Tower reservoir, so no upgrades to the Grande Prairie Trail booster station will be required.

For modeling the 2042 scenario, it was assumed that additional water flow will be supplied to future commercial and industrial area 21 in 2042, in addition to the development areas served in 2032. Developers should provide 300 to 350 mm diameter pipe loops in commercial development areas.

The existing 150 mm diameter AC pipe along 6 Avenue between 53 Street and 55 Street should be replaced by 250 mm diameter PVC pipe so that enough fire flow can be provided.

The 100 mm diameter CI pipes along 18 Avenue between 48 Street and 49 Street are too small and should be upgraded to 150 mm diameter PVC pipe. The existing 100 mm diameter pipe from 7th Avenue between 49 Street and 50 Street, and between 51 Street and 52 Street, should be upgraded to 150 to 200 mm diameter PVC for the 2042 Scenario. The 100 mm diameter CI pipes along 49 Street between 2 Avenue and 4th Avenue should also be replaced by 250 mm diameter PVC pipes.

After the water system is upgraded, water to the new development areas will be effectively supplied. Figure 3.8A shows the proposed fire flow supply configuration for the 2042 scenario.

3.7.1.2.5 2042 Cost Estimates – Town Only

The total cost for the proposed 2042 Town only upgrades is listed in Table 3.18 below and sections to be upgraded are shown on Figure 3.9A.

Table 3.19 Upgrade Costs for 2042 Town Only System

Location	Cost
Proposed 350 m of 250 mm diameter Watermain & Rehabilitate Road to Existing Conditions (along 6 Avenue)	$273,000

Table 3.19 Upgrade Costs for 2042 Town Only System

Location	Cost
Proposed 510 m of 150 mm diameter Watermain & Rehabilitate Road to Existing Conditions (along 18 Avenue and 7 Avenue)	$382,500
Proposed 230 m of 250 mm diameter Watermain & Rehabilitate Road to Existing Conditions (along 49 Street)	$179,400
Proposed 72 new Water Services	$108,000
Proposed 150 mm Isolation Valves	$31,500
Proposed 200 mm Isolation Valves	$3,600
Proposed 250 mm Isolation Valves	$22,500
Proposed New Well and Well Pump	$120,000
Sub-Total	$1,120,500
Contingency (30 percent)	$336,150
Engineering and Administration (10 percent)	$112,050
Capital Cost	**$1,568,700**

3.7.2 Future Servicing to the Town and Fringe Areas

This Section provides information on water modeling results that incorporated projected growth and development within the County fringe areas in addition to within Town limits, as well as recommended upgrades and associated cost estimates.

3.7.2.1 2032 Study – Town and Fringe

3.7.2.1.1 2032 Average Flow – Town and Fringe

As discussed in the 2032 Town only case (Section 3.7.1.1.1), the average demand within the Town is projected to be 71.99 L/s in 2032. The assumed development areas along Highway 16 for the fringe areas in 2032 are shown in Figure 2.2. The average flow for non-industrial development areas in the County in 2032 is projected to be 5.29 L/s. Therefore, the total average flow for the Town and the County combined would be 71.99 L/s + 5.29 L/s = 77.28 L/s, the MDD would be 2 x 77.28 L/s = 154.56 L/s, and the PHD would be 3 x 77.28 L/s = 231.84 L/s.

3.7.2.1.2 2032 Reservoir Storage – Town and Fringe

As discussed in the 2032 Town only case in Section 3.7.1.1.2, the required reservoir storage for serving the Town only in 2032 is projected to be 7,283 m^3. However, it was recommended that the total storage in 2032 be brought to 7,720 m^3 to account for the 2042 Town only scenario by constructing a 1,200 m^3 reservoir at the Microwave Tower site. The total reservoir storage requirement for the Town and fringe in 2032 = Fire storage (3,240 m^3) + Equalization storage (25 percent of MDD = 3,338 m^3) + Emergency storage (15 percent of ADD = 1,002 m^3) = 7,580 m^3. The storage recommendations for the Town only case would provide adequate storage for the 2032 Town and fringe case.

3.7.2.1.3 2032 Well Capacity – Town and Fringe

If the upgrades recommended in the 2032 Town only case were made, the total system production would be 145.96 L/s. To meet the Town and fringe MDD of 154.56 L/s, two wells would be required in addition to those recommended in the Town only case, bringing the production capacity of the system up to 159.96 L/s.

3.7.2.1.4 2032 MDD Plus Fire Flow Modeling Study and System Upgrades – Town and Fringe

All of the 2032 Town only system upgrades are required for the 2032 Town and fringe scenario. This Section discusses the upgrades required in addition to the 2032 Town only upgrades to serve the Town and fringe. Proposed upgrades are shown on Figure 3.7B.

New mains would be required to serve the east fringe area. It is assumed that these mains would be installed by developers, but a cost is provided in Appendix A for reference. Because elevation drops to the east fringe area, two pressure reducing valves (PRVs) would be required to transition from pressure Zone 1 to Future Zone 4.

New loops would also be installed to serve development areas in the southwest fringe area. It is again assumed that these loops would be provided by developers, but costs are provided as a reference. Since the elevation increases to the west fringe area, a PRV would be required between the fringe and town boundaries to separate the pressure zones. Upgrades would be required at the new Degas reservoir and pump station to provide adequate pressure and flow to the west fringe. Two pumps operating in a lead/lag fashion would be required that provide 15 L/s at 40 m of head. A fire pump should also be installed that is capable of providing 320 L/s at 40 m of head. These pumps would have connection piping connecting them to the fringe side of the PRV and would serve the fringe and the southwest Town (through the new PRV). The new Degas reservoir would need to be increased in size to provide adequate fire flow storage. The reservoir should be sized to provide 300 L/s for 3-hours, which is a volume of 3,240 m^3. Because separate sources of fire flow storage are required for different parts of the system, the volume of this reservoir should not be factored into the overall storage of the system for meeting storage requirements. It is assumed that the Grande Prairie Trail and Microwave Tower reservoirs would suffice for equalization and emergency storage.

The fire flow situation after the recommended upgrades is shown in Figure 3.6B.

3.7.2.1.5 2032 Cost Estimates – Town and Fringe

The total cost for the proposed 2032 Town and Fringe upgrades is listed in Table 3.19 below and sections to be upgraded are shown on Figure 3.7B.

Table 3.20 Upgrade Costs for 2032 Town and Fringe System

Location	Cost
Proposed 4320 m^3 reservoir and pump upgrades at Degas Site	$2,160,000
Proposed 730 m of 350 mm diameter watermain at East Fringe Area	$3,403,950
Proposed 3270 m of 350 mm diameter watermain at Southwest Fringe	$15,034,800

Table 3.20 Upgrade Costs for 2032 Town and Fringe System

Location	Cost
Sub-Total	$20,598,750
Contingency (30 percent)	$6,179,625
Engineering and Administration (10 percent)	$2,059,875
Capital Cost	**$28,838,250**

3.7.2.2 2042 Study – Town and Fringe

3.7.2.2.1 2042 Average Flow – Town and Fringe

As discussed for the Town only case in Section 3.7.1.2.1, the average flow in the Town in 2042 is projected to be 79.69 L/s. The assumed development areas along Highway 16 for the fringe areas in 2042 are shown in Figure 2.2. The average additional flow for non-industrial development areas in the County in 2042 is projected to be 4.11 L/s in addition to the 5.29 L/s fringe demand in the 2032 scenario (see Section 3.7.2.1.2). Therefore, the total average flow for the Town and County combined would be 79.69 L/s + 5.29 L/s + 4.11 L/s = 89.09 L/s, the MDD for 2042 would be 2 x 89.09 L/s = 178.18 L/s, and the PHD would be 3 x 89.09 L/s = 267.27 L/s.

3.7.2.2.2 2042 Reservoir Storage – Town and Fringe

The total reservoir storage requirement in cubic metres (m^3) for the Town and fringe in 2042 = Fire storage (3,240 m^3) + Equalization storage (25 percent of MDD = 3,849 m^3) + Emergency storage (15 percent of ADD = 1,155 m^3) = 8,250 m^3. The storage under the 2032 Town and fringe case would be 7,720 m^3, so an additional 530 m^3 of storage would be required for the 2042 Town and fringe case. This storage could be provided at either the Grande Prairie Trail or Microwave Tower sites.

3.7.2.2.3 2042 Well Capacity – Town and Fringe

If the upgrades recommended in the 2042 Town only case were made, the total production capacity of the system would be 159.96 L/s. To meet the Town and fringe MDD of 178.18 L/s, three Wells would be required in addition to those recommended in the Town only case, bringing the production capacity of the system up to 180.96 L/s.

3.7.2.2.4 2042 MDD Plus Fire Flow Modeling Study and System Upgrades – Town and Fringe

All of the 2042 Town only system upgrades and 2032 Town and fringe upgrades are required for the 2042 Town and fringe scenario. This Section discusses the upgrades required in addition to the abovementioned upgrades to serve the Town and fringe. Proposed upgrades are shown on Figure 3.8B.

After the 2032 Town and fringe scenario, the southwest fringe expands farther westward. All of the upgrades recommended in the 2032 Town only and 2032 Town and fringe scenarios are required for the 2042 Town and fringe scenario. In addition to those upgrades, the new Degas booster station would need to be upgraded to provide flow and pressure to the new developments in the southwest fringe at higher elevations. The two regular duty Degas pumps at the new booster station would need to be upgraded to provide 30 L/s at 60 m of head, and the fire pump would need to be upgraded to provide 330 L/s at 60 m of head. No upgrades to the new Degas reservoir would be required.

The fire flow situation after the recommended upgrades is shown in Figure 3.8B.

3.7.2.2.5 2042 Cost Estimates – Town and Fringe

The total cost for the proposed 2042 Town and Fringe upgrades is listed in Table 3.20 below and sections to be upgraded are shown on Figure 3.8B.

Table 3.21 Upgrade Costs for 2042 Town and Fringe System

Location	Cost
Pump Upgrades	$400,000
Storage Reservoir	$265,000
New Water Wells	$180,000
Sub-Total	$845,000
Contingency (30 percent)	$253,500
Engineering and Administration (10 percent)	$84,500
Capital Cost	**$1,183,000**

3.7.3 Cost Estimate Summary for Future Upgrades

The cost estimates for the various recommended upgrades for the 2032 and 2042 scenarios are summarized in Table 3.21

Table 3.22 Summary of Water System Cost Estimates for Existing and Future Upgrades

Upgrade Phases	Cost Estimates without Contingency and Engineering	Capital Cost including Contingency and Engineering
Existing System Upgrades	$8,805,750	$12,328,050
Future 2032 Upgrades Town Only	$7,527,750	$10,538,850
Future 2032 Upgrades Town and Fringe	$20,598,750	$28,838,250
Future 2042 Upgrades Town Only	$845,000	$1,120,500
Future 2032 Upgrades Town Only	$7,527,750	$10,538,850

Note: All cost estimates include a contingency allowance of 30 percent, and an engineering and administration allowance of 10 percent.

3.8 Water System Conclusions

It should be noted that one other Option for boosting pressure and fire flow in the area shown as Zone 2A in Figure 3.4 would be to connect this new loop to the future Microwave Tower reservoir recommended in 2032 (Section 3.8.1.1.4). When this scenario was modelled, the pressures along 62 Street dropped into a range of 32 psi to 39 psi, which is below the 40 psi requirement, while fire flows requirements were still met. This would decrease the necessary capabilities of the Grande Prairie Trail booster station. However, because pressure requirements still would not be met, and the Microwave reservoir was not recommended until 2032. Option 2 is still the recommended alternative.

4. Wastewater System

This Section of the report describes the existing wastewater system, presents a sample condition assessment of the linear sewer collection system, assesses capacities of the existing sanitary system performance to identify deficiencies and recommends improvements to the system. The wastewater system assessment also includes future horizon servicing requirements resulting from projected population growth and infrastructure development.

4.1 Existing Wastewater System

Wastewater Collection System

The Town of Edson has 68 km of gravity sewer mains that drain to the Town's wastewater treatment facilities located east of 25th Street and south of the Canadian National Railway right-of-way at the east end of Town. There are no lift stations in the system and all sewer discharges by gravity. The majority of the sanitary system pipes (79 percent) are 300 mm or less in diameter. The largest sanitary sewer is a 1,200 mm diameter (interim size on 1,050 mm diameter sewer trunk main). The system is illustrated in Figure 4.1.

Wastewater Treatment System

The Town's wastewater treatment system currently uses lagoons for treatment. The Town's treatment facility is currently being upgraded to include a mechanical wastewater treatment plant (WWTP). The new plant comprises a conventional activated sludge (CAS) process based on a combined treatment unit (CTU) where the bioreactor is around the secondary clarifier together with a headworks building providing fine screening and grit removal. Treated effluent, both existing and future, is discharged into the McLeod River approximately 2.5 km from the treatment system site.

4.2 Sanitary Sewer Condition Assessment

An understanding of the age and material properties of the existing sanitary sewer mains is essential to assessing the condition of the pipes. The recommendations for the immediate replacement and additional replacement to be made within the next 15-years of mains is dependent on both of these conditions.

4.2.1 Installation History of Sanitary Sewer Mains

Existing sewer data, consisting of pipe material, length, and year of installation for each manhole-to-manhole pipe segment were provided by the Town. Records were provided for 1949 to 2016. 1949 was assumed to be the earliest start date available as most of the sanitary sewer system was installed as the Town developed in the mid-to-late part of the 20th Century. The data was analyzed to produce a plot of the total length of pipe installed in each year of the Town's servicing history. The results of this analysis are presented on in the following exhibit, Exhibit 4.1. A majority of the sanitary system (60 percent) was installed prior to 1970, especially in the 1950s where a significant amount of the sanitary system was installed.

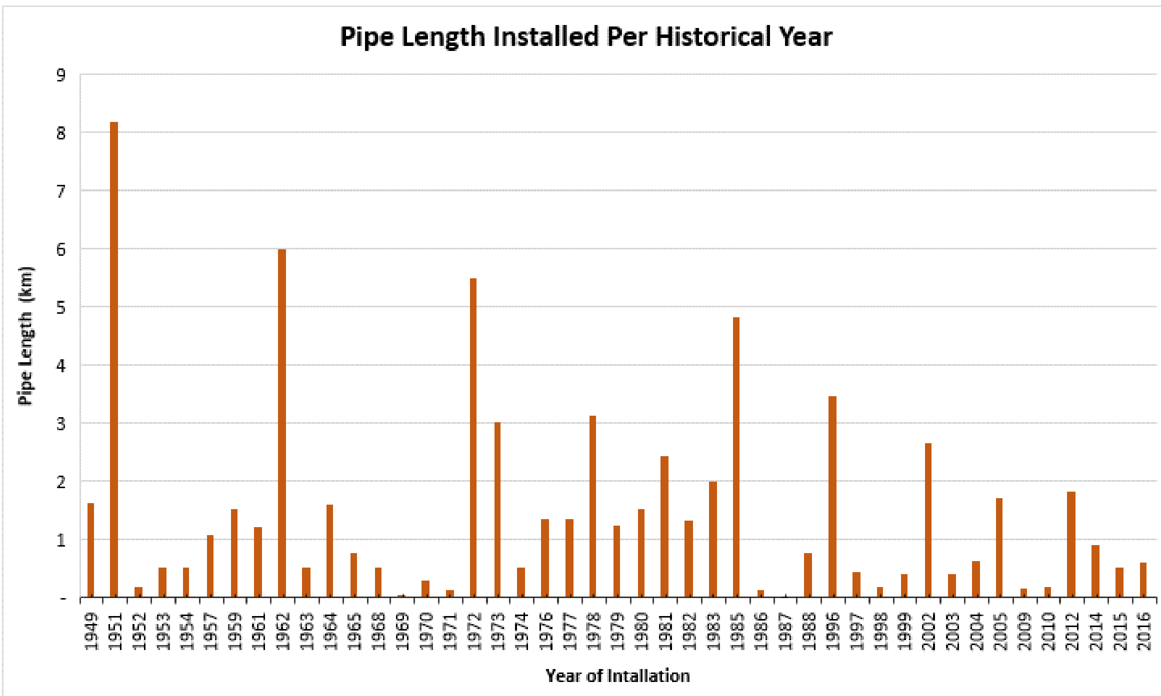


Exhibit 4.1 Pipe Length Installed Per Historical Year

The age of installation of the sanitary sewer mains is important as many portions of the mains would be considered older and be nearing end-of-life service expectations.

4.2.2 Pipe Material (VCT Pipe)

Historically, Vitrified Clay Tile (VCT) was primarily used as the pipe material for local sanitary sewers in the Town. Concrete and PVC pipes are now used primarily. Because of its corrosion-proof properties, VCT was the material of choice for sanitary sewer installation in the 1950s. Based on the Town's records, VCT pipes continued to be installed until around 1981.

If the VCT pipes are properly installed and the soil around the pipe is stable, they can have a long service life. However, if the VCT pipes are not installed properly or if the soil conditions around the pipe change over time, failure of the VCT material will occur. Common problems with VCT pipes include the following:

- Poor soil backfill bedding causing additional stress on the pipe. VCT pipe is very brittle and fractures are a common occurrence.
- Soil drift by the pipe also causes additional stress on the brittle pipe causing cracking and splitting of the pipe wall.
- If a VCT a sewer pipe starts leaking, the surrounding soil enters the pipe with inflow, creating soil backfill voids and uneven loading on the pipe. This can cause the pipe to collapse.
- When installing lateral connections or service connections, holes are broken in the VCT main pipe (main) for the laterals and then any gaps between the main and connections are filled with a sealing material. This sealing material washes away over time allowing groundwater and soil backfill to enter the pipe.
- Poor construction practices sometimes used include restraining lateral service connections with boards, bricks, or rocks to hold them in place while backfilling. As the soil and the rigid do not settle equally, a shear fracture is created in the pipe of the connection.
- Through fractures and cracks, it is common for roots to grow inside the pipe, causing blockage of the sanitary sewer. As roots grow, they also force fractures and further open the cracks.
- Encrustations and scale build-up are common within the pipe at all fractures and cracks as these are areas of infiltration for the pipe. These build-ups can also develop into blockages of the pipe.
- Sags and flattening are more common in VCT pipes as the pipe segments are shorter and joints are more frequent. These sags and flat areas result in buildup of deposits and blockage.

4.2.3 CCTV Inspection

An assessment of the existing sanitary pipe condition was conducted by review of CCTV footage provided by the Town, and by conducting additional CCTV investigations in the locations shown on Figure 4.2. The additional locations selected were based on the following: age of the infrastructure, a good random sampling of the Town's system, and to see if there was a correlation between the age of the pipe and the condition of the pipe as viewed by the CCTV investigation.

The additional CCTV investigation was completed by Thuro Inc. in January 2018. The investigation and video reports provided by Thuro are provided in an ancillary binder accompanying this report. The reports and actual video footage are also provided in electronic formats, of which a thumb drive is included in the binder. A summary Table of the findings of the video inspections has been provided in Appendix C with the Cost Estimates. This summary Table both rates the risk of the sanitary line and adds a recommendation of what corrective action needs to be done with these investigated lines.

The CCTV videos provided by the Town, did not provided adequate details to determine the condition of the sanitary mains. Consequently, some sections of the lines provided by the Town's CCTV records were selected for inspection in the recent CCTV investigation.

Of the 45 lines investigated, 25 appeared to be in need of replacement in the near future. Thirty-two of the pipes, investigated are small diameter (less than 300 mm) pipes; of these 32 pipes, 21 of the pipes are recommended for replacement. The 45 lines were VCT pipe and installed from 1951 to 1976, with the majority of the pipes being in service for more than 50-years.

Various problems from the CCTV video investigation were identified, including: root intrusion, cracks, breaks, and fractures in the lines (especially at service tie-ins and joints). Varying slopes causing undulating and sags in the line. As these lengths of the line were not at minimum grades to ensure gravity flow, sediment would build-up. There are also a number of encrustations and deposits, which if allowed to continue to build, can cause blockage. With 21 of the 32 small diameter pipes investigated appeared to require replacement. Therefore, it is recommended that the Town look at replacing these older smaller diameter lines as soon as possible. It is also recommended that the Town perform an asset management study to develop options to manage this important aging infrastructure.

Of the 45 lines investigated, 11 of them are considered medium risk for integrity failure. For these lines the flaws can worsen over time and therefore should that be monitored for additional progression of deterioration and serviceability every 3-years. It is anticipated that these lines will have to be replaced by 2032 (in 15-years).

The remaining 10 lines are considered low risk for failure so only should be re-evaluated on a regular basis.

4.2.4 Cost Estimate based on Condition Assessment

Expanding from the CCTV Video Inspection, GHD has provided two cost estimates for replacement of the mains deemed to have a high and medium replacement risk with respect to the age of the line. From the sampling of the lines that had a CCTV Investigation performed, the majority of the sanitary mains in need of immediate replacement are the smaller, older VCT pipes (<300 mm). These pipes also have numerous tie-ins for servicing buildings and homes along their length. The majority of the mains were installed prior to 1955 and are VCT pipes. Additional pipes, post 1955, were added to the list of pipes to be replaced immediately, based solely on the evaluation of the findings of the CCTV video investigation. The cost for the Immediate Existing System Upgrades is **$8.23 Million.** It is broken down on a length by length (manhole to manhole) basis and can be reviewed in Appendix C – Wastewater System – Cost Estimates.

GHD has also provided a secondary list of replacements, which should be replaced within the next 15-years, or by 2032. The majority of these pipes are the remaining older pipes, which was installed up to 1970. There are also some even older pipes (1949) in this grouping. However, according to the CCTV Video Inspection, they were still in moderate functioning condition. These and the majority of the pipes are VCT pipes which were installed between 1956 and 1970; these pipes are recommended to be replaced by 2032. The cost for this 2032 System Upgrades is estimated at **$9.79 Million**. These costs are further broken down on a length by length basis in Appendix C – Wastewater System.

Please note that these cost estimates do not include any allowance for pipe replacement due to undersized pipes to avoid surcharge conditions.

Figure 4.3 in Appendix H of this report shows the investigated pipes under different risk level, red ones being at the highest risk, requiring immediate replacement.

4.3 Previous Model in 2011 MSP Update

4.3.1 Summary of Design Flows from Previous Model Calibration

At the time of development of the 2011 MSP update, the Town had only a single flow monitor on the inflow to the wastewater treatment lagoon and did not have a rain gauge that would correspond to the existing flow monitor. Rainfall data was obtained from Environment Canada for the Edson Airport.

Flow monitoring data at the inflow to the lagoon was provided for 2008. The model was verified for the inflow to the lagoon for three selected rainfall events. In general, the modelled and monitored wet weather flow compared quite well for all three events. The average calculated I/I rate for all the rainfall events was approximately 0.11 L/s/ha.

The design criteria based on the calibrated model were as follows:

- Residential Sewage Generation Rate – 375 Lpcd
- Non-residential Sewage Generation Rate – Site Specific or 1,500 L/ha/d
- Effective Area (areas with weeping tile connected) – 5.3 percent
- Effective Area (areas without weeping tile connected) – 0.05 percent

For future development areas, values developed as part of the water distribution analysis were used. These values were similar to values used in other municipalities in Alberta.

Values recommended for the future development areas were as follows:

- Residential Sewage Generation Rate – 330 Lpcd
- Residential Peaking Factor – 3
- Non- residential Sewage Generation Rate – 10,000 L/ha/d
- Non-residential Peaking Factor – 3
- Inflow/Infiltration Allowance – 0.28 L/s/ha

4.3.2 West End Sanitary Trunk Main 2014 Design Flow Rates

In the 2014 ISL study on the Edson West Sanitary Trunk Main, ISL completed a sanitary calculation spreadsheet to compare flows with the 2011 MSP modelling results. ISL noted some inconsistencies with the modelled flows. The sanitary design criteria ISL used in their spreadsheet are as follows:

- Residential Sewage Generation Rate – 375 Lpcd
- Commercial/Industrial Sewage Generation Rate – 13,600 L/ha/d
- Residential I/I Rate – 0.28 L/s

- Commercial/Industrial I/I Rate – 0.05 L/s
- Residential Peaking Factor – 2.6P-0.1 or 1.5 (larger of the two)
- Commercial/Industrial Peaking Factor – 3.0

4.3.3 Review of Design Flow Criteria of Alberta Environment and Parks & Other Municipal Jurisdictions

This MSP update includes a comparative review of the wastewater flow generation criteria used in the Alberta Environment and Parks (AEP) Guidelines and in municipal design standards of several Alberta municipalities.

To supplement the existing single flow monitor at the lagoons and the Environment Canada rain gauge station at the Edson Airport, additional flow monitors were installed at various strategic locations of the wastewater collection system, as well as a rain gauge that corresponds to the existing flow monitor at the lagoon. The rainfall and flow data collected was analyzed. This data was then used in the calibration of the flow generation parameters for this MSP wastewater model update. The design criteria, outlined below, is based on the desktop study.

Development Area in Town of Edson

Residential:

- Residential Sewage Generation Rate – 300 Lpcd
- Sanitary Peaking Factor – 3.0
- Population density – 31.2 pop/ha

Commercial /Industrial:

- Sewage Generation Rates of 1500 L/ha/d or site specific
- Peaking factor 1.5

Existing Infiltration and Inflow (I/I) Allowance:

- Based on the 25-year 24-hour rainfall event (Huff distribution)
- Effective Area (areas with weeping tile connected) – 5.3 percent
- Effective Area (areas without weeping tile connected) – 0.05 percent

Future Development in Fringe Area

For future development in the fringe areas, the design standards are similar to those adopted in other municipalities in Alberta.

Residential:

- Residential Sewage Generation Rate – 330 Lpcd

- Sanitary Peaking Factor of 3 or greater of 1+14/(4+p0.5)
- Development density 40 person/ha
- Inflow/Infiltration Allowance – 0.20 L/s/ha

Commercial/Industrial:

- Non- residential Sewage Generation Rate – 10,000 L/ha/d
- Non-residential Peaking Factor – 3
- Inflow/Infiltration Allowance – 0.05 L/s/ha

All proposed parameters for the sanitary design standard are subject to further calibration and updating as required. New flow monitoring data were collected during the summer of 2017.

4.3.4 Model Development

The following Section provides an overview of the sanitary system model development and calibration. The sanitary system model assesses the capacity of the existing sanitary collection systems and define future needs for sanitary infrastructure based on the growth projections.

The Town provided a previously developed (2011) sanitary system hydraulic model. This model was developed using XP-SWMM version 9.14. The model was reviewed and the following items and issues were identified:

- The model reflected the sanitary collections physical network.
- Node and conduit names were different from the sanitary servicing area map.
- There were discrepancies in some manhole and pipe inverts and diameters.
- The network did not extend into newly developed areas.
- There was no data to support how dry or wet weather flow were calibrated.

The updated sanitary system model will use PCSWMM as the model interface. This model is based on EPA's SWMM 5.0 analytical engine. Although SWMM 5.0 is widely used for the analysis of sanitary, storm, and combined systems, the PCSWMM interface provides superior data management and visualization tools that provide seamless data exchange to GIS for development planning and asset management.

The Town provided an aerial photo and AutoCAD as-built data regarding the sanitary sewer system. Much of the physical network data was carried forward from the previous model. This data was reviewed and, where necessary, additional information was sought from drawings and other records for clarification. The Town's staff also inspected select locations to verify invert elevations and flow directions.

The physical network data was updated based on as-built drawings and supplementary survey data provided by the Town in August 2017. The model extents were compared to existing and new infrastructure and where future growth is expected. The network was extended accordingly to provide a reasonable representation of existing and newly constructed systems.

Flow in the previous model was introduced at load points with no supporting service area data and digital mapping. Recognizing the model objectives, the delineation of catchments was carried out. The resulting catchments provide the basis for flow generation based on the contributing population and service area. Figure 4.1 presents the sanitary sewer catchment areas. The PC- SWMM model updated the 2011 model by changing the following:

- Adding sewers constructed from 2011 to 2017
- Adjusting the population to 2017 data
- The updated model includes 309 catchments, 747 manholes, 777 pipes, and 1 outfall.

4.3.5 Dry Weather Flow (DWF) Model

DWF is primarily generated from domestic use of water and generally estimated from recorded data during periods of no precipitation. DWF is normally comprised of a base flow from constant inputs to the system and a variable or diurnal flow that results from day to day discharges to the system. Based on the comparison of design standards of similar communities across the country, in general, residential wastewater generation rates range from 265 to 450 Lpcd.

Flow monitoring carried out in 2017 reflect actual flow conditions in the sanitary system. The average daily sewage flow for the Town was determined to be approximately 44.8 L/s for 2017. Considering a total population of 8,414 people, the DWF is equivalent to a composite per capita sewage generation rate for residential and non-residential generation, and ground water infiltration and inflow of approximately 460 Lpcd.

The flow data analysis further breaks down the composite generation rate into residential, non-high demand commercial and industrial, and high demand commercial and industrial areas. For existing residential areas, the wastewater generation rate of 265 Lpcd was applied. This equates to an average population density of 26.6 people/ha in residential areas of the Town.

For existing non-residential areas, a rate of 4,500 L/ha/d was used for non-high demand non-residential areas and a rate of 17,000 L/ha/d was used for high demand non-residential areas.

A diurnal flow pattern for DWF was also developed using this flow monitoring data, respectively for both residential and non-residential areas at each flow monitoring location.

4.3.6 Wet Weather Flow (WWF) Model

Houses constructed prior to 2005 are likely to have their weeping tiles connected to the sanitary system. The Town has experienced basement flooding caused by sewer backup in the past in areas suspected to have weeping tiles connections. Newer areas that do not have weeping tiles connected to the sanitary system did not experience flooding. Therefore, it is suspected that wet weather inflow into the sanitary sewer system through weeping tile connections is the main contributor to pipe surcharge and flooding.

The inflow and infiltration simulations during rainfall events vary greatly from the 2011 MSP Upgrade report which used the XP-SWMM model. The SWMM 5.0 model uses the Unit Hydrograph Method for generating rainfall derived infiltration and inflow (RDII). These RDII Hydrographs are used to simulate inflow and infiltration during rainfall events. These parameters are initially derived from the analysis of flow monitoring data characterizing the wet weather response in the sanitary system. The RTK values are then adjusted through the model calibration process using five flow monitoring locations throughout the Town for data from a rain event that occurred in September 2017.

4.3.7 Flow and Rainfall Monitoring Data Analysis

Flow monitoring started in July 2017 and was completed in the fall of 2017 by SFE Global. An ancillary binder by SFE Global accompanies this report with the results of the monitoring. Five flow monitoring stations, as shown in Figure 4.1, Appendix H, were set up as follows:

- #1 Monitor; Glenwood Residential with high ground water infiltration
- #2 Monitor; Residential (Weeping tile)
- #3 Monitor; Light Industrial Area at West End
- #4 Monitor; East End Residential
- #5 Monitor; Existing Town monitor upstream of treatment lagoons

The first four flow monitoring stations are in addition to the existing Town flow monitoring station, #5, at the downstream end of the system where the total volume of the Town's wastewater is discharged to the existing lagoons.

The rain gauge is located at the Wastewater Treatment Plant. Flow data was collected at five-minute intervals and rainfall data in five-minute increments. Table 4.1 provides a description of each monitoring location and summarizes the data. Rainfall collected at the Edson Wastewater Treatment Plant was used for the rain gauge data set used in the wet weather analysis.

The flow data analysis involves the definition of dry weather flow periods, calculation of average dry weather flow and characterizing dry weather flow in terms of diurnal pattern. The diurnal patterns for monitored areas were derived from the monitoring data and used to define a typical 24-hour dry weather flow.

Table 4.1 Flow and Rainfall Monitoring Data Observations

Site	Manhole ID	Pipe Dia. (mm)	Peak DWF (L/s)	Peak WWF (Sept. 2017) (L/s)	Ratio
1 - Glenwood	N593	200	2.6	9.6	2.63
2	N530	375	18.85	61.8	3.28

Table 4.1 Flow and Rainfall Monitoring Data Observations

Site	Manhole ID	Pipe Dia. (mm)	Peak DWF (L/s)	Peak WWF (Sept. 2017) (L/s)	Ratio
3	N423	600	31.57	88.3	2.80
4	N585	250	3.8	18.4	4.81
5 - Lagoon	Lagoon	1,050	56.51	160.8	2.85

The wet weather flow analysis characterizes extraneous flow responses in the sanitary system to a range of wet weather events. The previous dry weather diurnal flow time series that was developed is subtracted from the total measured flow to isolate the wet weather response as an infiltration/inflow (I/I) hydrograph. From this, the volume of wet weather for the event of September 2017 is determined for the contributing area. The ratio of the volume of wet weather divided by the total rainfall volume over the contributing service area was calculated. The higher the ratio, the greater the amount of inflow and infiltration (I/I) rate. Table 4.2 presents a summary of the wet weather flow analysis.

Table 4.2 Wet Weather Flow Analysis Summary

Monitor Station	Contributing Area (ha)	Volume of WWF (m^3)	Volume of Rainfall (m^3)	Ratio
1 - Glenwood	40.5	807	24,484	3.3 percent
2	119.2	2,921	72,116	4.1 percent
3	250	8,103	151,250	5.4 percent
4	76.9	1,645	46,525	3.5 percent
5 - Lagoon	570	11,262	344,850	3.3 percent

4.3.8 Model Calibration

The updated SWMM model was calibrated to the flow monitoring data collected and analyzed. The calibration consists of a two-step process: identification of the dry weather flows and identification of the wet weather flows for the selected rainfall events. The objective of the calibration is to have modelled dry and wet weather flows at each flow monitoring station achieve a reasonable agreement with the flow monitoring data collected.

Overall, dry and wet weather flow calibration provides a reasonable fit between observed flows and modelled flows. The flow at stations 1, 2, 3, and 5 were within 10 percent of measured average dry weather flow. However, the flow at station 4 had a difference greater than 10 percent. The extreme low flow at this station was presumed to have affected the resolution of equipment accuracy.

The September 22, 2017 event was the largest rainfall event that occurred in Edson in 2017. A total of 60.5 mm of rain was recorded over a period of 70 hours. Generally, wet weather calibration provided a good fit (<10 percent) between observed flows and modelled flows for the rainfall event. For all sites, the wet weather flow in the model is considered representative of existing conditions and suitable for undertaking capacity assessments for a design storm event.

Dry and Wet Weather Flow calibration models the diurnal pattern and RTK hydrograph parameters were saved on a USB memory drive which is as provided to the Town on a USB thumb drive as ancillary data for this report.

4.4 Existing System Evaluation

The existing sanitary system was evaluated under dry weather and wet weather conditions. For wet weather flow, both 5 and 25-year short duration (4-hour) and long duration (24-hour) design rainfall events were utilized for evaluation.

Based on the evaluation, GHD concluded that the existing system has sufficient capacity under existing peak dry weather flow conditions in the Town. Hence, capacity constraints were evaluated mainly under wet weather flow conditions. Since the 4-hour rainfall duration poses more problems than the 24-hour duration for both 5-year and 25-year design rainfall events (AECOM 2011), the existing system performance was primarily evaluated for 4-hour duration rainfall events.

Based on information provided by the town, the existing sanitary sewer system experiences flooding in the 5-year storm event at 54 Street around Griffiths Park, and a few other locations in the Town due to exceedance of pipe capacity under surcharging conditions in a 25-year storm event. However, the updated and calibrated model shows less flooding and surcharge problems than the 2011 MSP report.

4.4.1 Existing System Improvements

To solve flooding and capacity problems, the same improvement strategy from the previous report was used. Potential solutions involve upgrading and/or twinning sections of pipe in the problem locations, but replacement is only considered as alternatives for system upgrades. The decision to twin or replace the existing sewer will be based on the condition of the existing pipes, construction method and capital project plan. The improvements are primarily targeting to prevent basements from flooding. Upgrade will be focusing on sewers of surcharging within 2.5 m of the ground level in **residential areas** for the 5-year 4-hour rainfall event for minimizing the risk of basement flooding. In addition, the improvements will also upgrade surcharging sewers within 1.0 m of the ground level for the 25-year 4-hour rainfall event and flow larger than 1.2 x full capacity.

Figure 4.4 in Appendix H, illustrates the location and pipe name included in the proposed upgrades.

The new 350 mm pipe (Alternative 2 in the 2011 MSP report) is no longer required in the Phase 1 improvement as the model showed. However, for future development at the western end of the Town, a 750 mm pipe is still required (Alternative 3 in the 2011 MSP report) and is discussed further in Section 4.6.1, Future Sanitary Servicing Plan.

In addition to the identified improvements, Phase 2 and 3 improvements within the east portion of residential area of the Town as proposed in the 2011 MSP report are still required due to the model uncertainty for this residential area from the limited flow data available.

4.4.2 Improvement Strategies

Any improvement will likely reduce the risk of sanitary sewer backup. However, before proceeding with any improvements, the Town should undertake further flow monitoring and data collection effort to maintain a reliable physical network database. Additional flow monitoring stations are required to quantify I/I values, thus reducing modelling artifacts associated with improperly placing large wastewater flow among a number of junctions. Capacity improvements are driven by rainfall derived I/I. Implementation of a sound I/I program can reduce the cost of improvements based on capacity requirements. To address I/I, the challenge is to identify the sources of I/I and to determine if they can be cost effectively eliminated or controlled. The 2011 MSP report recommended the weeping tile to be disconnected from the sanitary system as the opportunity arises when other repairs or upgrades are being conducted.

Additionally, I/I source identification involves a number of investigative techniques. The Town should continuously make efforts to use following typical techniques:

1. Flow monitoring on a regular basis to identify areas with high I/I;
2. CCTV inspections during or shortly after a wet weather event to identify I/I in service laterals and pipe joints; and,
3. Manhole inspection to provide information on manhole condition and I/I.

Upgrade Costs for Existing System Existing system upgrades only pertain to the current town boundary and do not extend into the Fringe Area of the County. The associated cost estimates with respect to each phase of improvements are as follows:

4.4.3 Existing Upgrade Costs

Existing system upgrades only pertain to the current existing town boundary and do not extend into the Fringe Area of the County. The associated cost estimates with respect to each phase of improvements are as follows:

Table 4.3 Existing System Cost Estimates Due to Pipe Capacity

Description	Cost
Total Improvements	$2.654 million

Cost breakdown details are provided in Appendix C.

4.5 Future System Evaluation

4.5.1 Future Sanitary Servicing Plan

A future sanitary servicing plan was developed to accommodate projected growth outlined in the 2032 and 2042 future development in Appendix H, Figure 4.6. The future servicing plan assumes that the recommended existing upgrades have been completed. For the servicing plan, the population growth within the Town were assumed to be 1 percent and 0.6 percent, respectively. Future land use was obtained from the Town as shown in Figure 2.2, Appendix H. The sanitary sewer design values were adopted from design criteria values developed in the previous current desktop study.

The future sanitary system servicing plan for the development areas assumed piping connections to the existing system in areas that can be serviced by gravity and as Well as where capacity is available. Infill areas were connected to the nearest sanitary lines. The most viable alternatives for expansion for 2032 and 2042 development scenarios are summarized below.

For the 2042, scenario new industrial and residential developments were connected to the existing upgraded sanitary system in Figure 4.6. Preliminary pipe sizing and locations at a planning level were assumed and are consistent provided which is in line with the transportation corridor and other utility plans in this study. However, these assumptions are subject to confirmation of future area structure plans and preliminary designs.

The existing system with completion of the proposed upgrades is adequate for the addition of residential areas to the northeast and northwest portions of the Town.

For the west portion of the Town, a proposed new 750 mm sewer trunk servicing the business, commercial and industrial users in this area will be needed.

A lift station is required for new development areas east of the Town (Sewer Areas 15, 16, and 17). This station should be built prior to the 2032 development scenario, preferably around 2022. This lift station should have a capacity of 28 L/s and should pump flows using a new150 mm diameter, 1,700 m long forcemain to the existing treatment plant.

The new development areas west of the Town (Sewer Area 13) are at a significantly low elevation and will require a lift station as well. A 41 L/s lift station in Area 13, required by 2032 with a 200 mm diameter forcemain that crosses Highway 16 will be required to service this area.

This 750 mm sewer trunk has been designed by AMEC in 2015, and the Town is procuring the land for the utility right of way.

Elevations and slopes for the new sewer sections will depend on future detailed development design and are not included in these estimates. Detailed lengths and diameter estimates for future development for both 2032 and 2042 scenarios are presented in Tables 4.4 through 4.7.

Table 4.4 2032 Future Development for Town of Edson

Development Area	Pipe Type	Pipe Length (m)	Approximate Average Depth (m)	Diameter (mm)
Area 1	Gravity	250	3	200
Area 2	Gravity	250	3	200
Area 8	Gravity	200	4	200
Area 9	Gravity	100	3	200
Area 14	Gravity	300	5	200

Table 4.4 2032 Future Development for Town of Edson

Development Area	Pipe Type	Pipe Length (m)	Approximate Average Depth (m)	Diameter (mm)
Area V-s	Gravity	600	3	250
Area M-g	Gravity	500	3	250
West Areas 11,12,13	Gravity	800	4	675
West Areas 11,12,13	Gravity	1600	4	600
West Area 13	Force Main	1300	3	200
West Area 13	Force Main (HWY)	200	3	200
West Area 13	Lift Station 41 L/s			

Table 4.5 2032 Future Development for Fringe Area (Yellowhead County)

Development Area	Pipe Type	Pipe Length (m)	Approximate Average Depth (m)	Diameter(mm)
Area 15	Gravity	750	4	200
Area 15	Gravity	400	4	300
Area 17	Gravity	800	4	200
East - Area 15,17	Force Main	1700	3	150
East - Area 15,17	Lift Station 28 L/s			
West Areas 22,23	Gravity	3800	4	450
West Areas 18	Gravity (HWY)	200	4	450
West Areas 22	Gravity	1200	4	375

Table 4.6 2042 Future Development for Town of Edson

Development Area	Pipe Type	Pipe Length (m)	Approximate Average Depth (m)	Diameter (mm)
Area 3	Gravity	150	3	200
Area 6	Gravity	100	4	250
Area 7	Gravity	350	4	200
Area 10	Gravity	200	4	200

Table 4.7 2042 Future Development for Fringe Area (Yellowhead County)

Development Area	Pipe Type	Pipe Length (m)	Approximate Average Depth (m)	Diameter (mm)
West Areas 19,20,21,22	Gravity	1600	4	375
West Areas 19,20,21,22	Gravity	1700	4	300
East - Area 16	Force Main	1700	3	200
East - Area 16	Lift Station 43 L/s			
East - Area 16	Gravity	1000	4	300
South - Area 18	Gravity	800	3	300

4.5.2 Estimates for Future Sanitary System

The details of the cost estimates for the future scenarios are as follows:

Table 4.8 Future System Cost Estimates

	Town of Edson	Fringe Area Yellowhead County	Total
2032 (Capacity)	$21.11 million	$9.88 million	$30.99 million
2042 (Capacity)	$1.43 million	$7.79 million	$9.22 million
Total			**$40.21 million**

The cost breakdown details are shown in Appendix C, and is shown on Figure 4.4 and Figure 4.5.

5. Stormwater System

5.1 Description of the Existing Stormwater System

Storm systems in the Town of Edson are separate from the sanitary sewer systems, servicing a total area of approximately 550 ha within the Town limits. The majority of the storm system uses stormwater gravity pipe systems, with the exception of the Glenwood area at the West, which uses an open roadside ditch system. The Town has about 27 km of storm pipe ranging from 300 mm to 1,500 mm in diameter. The majority of the storm sewers pipes are concrete pipe.

Currently, the developed stormwater system of the Town drains into the three creeks (Bench Creek, Wase Creek, Poplar Creek) that run through the Town. The creeks flows in a southeasterly direction, with Wase Creek discharging into Poplar Creek, and Poplar Creek discharging into Bench Creek and ultimately discharges into the McLeod River.

The developed areas of the Town have been delineated into 33 drainage basins; 12 draining to Bench Creek; 14 to Poplar Creek; and 7 to Wase Creek.

5.2 Condition Assessment

The existing stormwater management system within the Town was assessed by performing a field condition inspection during the week of August 23 to 25, 2017. The objectives of the field inspection were to assist in the assessment of the condition of the existing drainage system and clarify any drainage issues to better understand how the overall drainage system operates.

The visual inspection of the existing storm sewer system infrastructure included the following components:

- 27 manholes (8 percent of total manholes);
- 19 outfalls (60 percent of total outfalls);
- 9 culvert locations; and
- 3 Stormwater Management Facilities (SWMF) inlets and the associated SWMF outlets were identified in what appears to be a relatively new subdivision in the extreme northeast corner of the Town. The SWMF is located north of 15th Avenue and west of 41st Street.

In the area, four catch basins, a culvert, an outfall, an unidentified manhole, and an access pit were discovered that does not appear to be shown on the record plans provided by the Town.

The inspection locations were all located north of Highway 16 between 63rd Street on the west and 41st Street on the east.

During the inspection, the presence of sediment, mud and debris to varying degrees in the manholes, culverts and outfalls were consistently observed. A significant number of the outfalls and culverts were noted to contain water, which ranged from 25 percent to 50 percent full at some locations.

The manhole barrels appear to be in decent shape. In seven manholes, the grade rings were misaligned and in a few other manholes the grade rings were broken and/or crumbling.

The pipes north of Outfall 4-064 were approximately 50 percent full of dark dirty water with no evidence of flow. However, Outfall 4-064 was not found and possibly was buried to block the flow.

The stormwater infrastructure that was visually inspected represents only a small fraction of the existing storm systems. In general, the deficiencies identified in the field could be classified as maintenance issues. Overall, the storm system is in a fair condition, but the system requires more operational maintenance on a regular basis.

The records of the field visual inspection are included in Appendix E.

5.3 Design Criteria

Stormwater sewer systems are typically designed to contain a 5-year, 4-hour rainfall event with no surcharging. For the proposed existing system improvements, a level of service such that there is no surcharging within 1.0 m of ground for a 5-year, 4-hour rainfall event was recommended to be adopted by the Town. For this to be an acceptable level of service, surcharging must be localized to an isolated pipe and not have a significant impact on the remaining components of the system.

SWMFs are to be designed to provide sufficient storage for the 100-year, 4-hour Chicago rainfall event while discharging at a predetermined allowable discharge rate.

The allowable discharge rates for Bench, Wase and Poplar Creeks were established in the Town of Edson Stormwater Management Plan completed by UMA Engineering in 2005. A regional analysis was performed which established a flow versus drainage area relationship. This hydrologic relationship was used to determine the estimated 100-year peak flow rate for the three creeks. The established allowable discharge rates are shown in Table 5.1 below.

Table 5.1 Allowable Discharge Rates of Each Drainage Basin

Name of Receiving Watercourse	Allowable Discharge Rate (L/s/ha)
Bench Creek	2.8
Wase Creek	7.2
Poplar Creek	9.0
McLeod River	9.0 assumed the same to Poplar Creek

These allowable discharge rates will be applied to future development areas. The objective will be to control the post-development discharge to the natural pre-development conditions of the respective basins.

For infill developments, the discharge will be going to existing stormwater system, and adding the full flow to the existing stormwater system may not be practical as it could cause potential flooding downstream. Thus the objective would be to restrict the discharge rate. However, infill development areas are typically smaller lots and it may not be practical to achieve a significant amount of on-lot storage in these areas. Therefore, it is recommended to have a realistic allowable discharge rate established on a site specific basis taking into account the size of the development area, what is being proposed for the site, the location of existing storm infrastructure in relation to the site, sewer capacities, existing grading of surrounding roads and properties and flooding history in the area.

5.4 Modeling of the Stormwater System

The stormwater system database was updated with as-built data available as of August, 2017. The updated database was used to construct a model to analyze the hydraulic capacity of the existing stormwater system. The modeling will be used to evaluate:

- Existing system deficiencies;
- Required improvements;
- Impact of future development; and
- A storm servicing concept for the ultimate development scenario.

The stormwater model provided by the Town was created in2011 MSP using XP SWMM version 9.14. PC SWMM is considered as an equivalent software program using the same EPA SWMM 5.0engine for modeling with the advantage of geo-referencing and data management. As such, the existing stormwater model was created in PC SWMM 2017 by converting all data from the XP SWMM model. All data, including both hydrological catchment and infiltration data, and hydraulic data such as physical stormwater pipe networks and manholes were imported.

All data reference the NAD83 3TM117 coordinate system. As a result, the new model has improved geo-referencing and can be projected on online publicly available mapping systems. Subsequent to the completion of 2011 MSP, additional pipes, pipe diameters, lengths and slopes were obtained from the Town in August 2017, and incorporated into the existing model.

The new model included all major stormwater pipes (larger than 200 mm in diameter) and all current developments within the Town limits for which the data was made available. The model does not include individual catch basins and their connections to manholes.

The modelled system consists of:

- 186 stormwater sub-catchments ranging in size from 0.35 ha to 36.55 ha;
- 283 pipes ranging in size from 250 mm to 1,200 mm diameters and having a total length of more than 26,000 m;
- 3 stormwater ponds; and
- 33 outfalls (free discharge).

After updating the model, the following additional models were created for evaluating the existing storm system and the future storm servicing requirements up to the 2042 planning horizon. (Chicago storm event designated as “C” and Huff storm event designated as “H”).

Existing Stormwater Model

- C Models - Using 5, 25, and 100-year 4-hour Chicago storm events
- H Models - Using 5, 25, and 100-year 24-hour Huff storm events
- Improvement Model - 5-year 4-hour Chicago storm events

Future Stormwater Model, 2032

- C Model - Using 100-year 4-hour Chicago storm event
- H Model - Using 100-year 24-hour Huff storm event

Future Stormwater Model, 2042

- C Model - Using 100-year 4-hour Chicago storm event
- H Model - Using 100-year 24-hour Huff storm event

5.4.1 Design Rainfall Events

The rainfall amounts and intensities of design storms for the Town are summarized in Table 5.2. A 4-hour Chicago distribution was adopted to represent a short duration design rainfall event which will result in a high intensity rainfall. A 24-hour Huff distribution was chosen to represent a long duration design rainfall event. The 24-hour Huff distribution results in a maximum rainfall over a long duration but has a much lower intensity than the Chicago distribution. These distributions are typically used in computer modeling of urban drainage systems.

Table 5.2 Design Rainfall Events

Return Period (years)	Duration (hours)	Total Rainfall (mm)	Peak Intensity (mm/hr)
5	4	34	91.4
	24	59.7	8.3
25	4	44.5	125.8
	24	75.5	10.5
100	4	53.3	154.6
	24	88.6	12.3

5.4.2 Horton Infiltration

A set of stormwater sub-catchment parameters was established to generate stormwater runoff. Horton's Equation was used for modeling infiltration in PC SWMM. The equation is based on empirical research observation that infiltration decreases exponentially from an initial maximum rate to some minimum rate over the course of a long rainfall event. Input parameters are summarized in Table 5.3.

Table 5.3 Model Parameter Summary for PC SWMM Model

Parameter	PC SWMM Model
Manning's Coefficient	
Impervious Area Manning's n	n = 0.015
Pervious Area Manning's n	n = 0.25
Depression Storage	
Impervious Depression Storage	3.2 mm
Pervious Depression Storage	6.4 mm

Parameter	PC SWMM Model
Zero detention	25 percent
Infiltration	
Minimum Infiltration Rate	5 mm/hr
Maximum Infiltration Rate	100 mm/hr
Decay Rate	4.14 /hr

5.5 Existing System Evaluation

The existing system was evaluated to assess system performance during rainfall events by examining the following parameters:

- Capacity utilization within the system to identify potential locations where pipe flow exceeds pipe capacity; and
- Hydraulic grade line within the system to identify potential surcharge locations and possible flooding locations.

The magnitude of surcharging in the stormwater system, indicated by the hydraulic grade line, was categorized into three ratings as outlined in Table 5.4.

Table 5.4 Pipe Capacity Utilization Rating

Rating	Ratio of Peak Flow to Pipe Capacity
Green	0 - 1.2
Yellow	1.2 - 2.0
Red	>2.0

When capacity utilization is less than 1.0, the trunk flows are under open channel conditions, which is the most desirable hydraulic condition. For the Town, the capacity of a pipe is considered adequate when the peak flow to pipe capacity ratio is less than or equal to 1.2. Peak flow to pipe capacity ratio in the range of 1.2 to 2.0 may require attention; and those pipes with a ratio greater than 2.0 were considered undersized and require upgrading.

In the situation when downstream trunks are surcharging and causing flow to back-up, the upstream pipe could be surcharged but still shows as a green pipe indicating a capacity utilization of less than the factor of 1.2.

In the SWMM model, nodes are indicative of the water level in the pipe below ground, referred to as the hydraulic grade line (HGL). Nodes in the model represent stormwater manholes. The magnitude of surcharging at manholes was calculated by subtracting the maximum hydraulic grade line (HGL) from the ground elevation at the manhole. The depth of surcharge was divided into levels as outlined in Table 5.5.

HGL of 2.5 m below the ground is indicated by a green node. HGL between 1.0 and 2.5 m is indicated by a blue node. When the HGL reaches to within 1.0 m of the ground surface the node color is indicated as yellow. If the node becomes red, flooding occurs.

In some cases, a red surcharging node may be caused by its shallow bury depth. To determine if the pipe is surcharged or not, the results of pipe capacity and HGL need to be evaluated pipe to pipe.

Table 5.5 Levels of Surcharging in Storm Manhole

Rating	Depth of HGL Below Ground (m)
Red (Flooding)	Above ground surface
Yellow (Major Surcharge)	0 - 1.0
Blue (Minor Surcharge)	1.0 - 2.5
Green (No Surcharge)	> 2.5

The capacity of the existing stormwater system was evaluated in the PC SWMM model by simulating the 5, 25, and 100-year rainfall events. As the return periods increase and the rainfall amount increases, the impact to the system also increases. As illustrated in the results of the simulation, most stormwater sewer systems are not designed to convey runoff from rainfall with return periods larger than the 5-year storm event. Short duration (4-hour) rainfall events have the most impact on the stormwater system due to its greater peak intensities than the long duration (24-hour) rainfall event which results in higher flows.

5.5.1 5-Year Rainfall Event Results

The evaluation results show that for a short duration (4-hour) rainfall event, there is a large amount of surface flooding, as indicated by red nodes in Figure 5.1. The parts of the system not flooding still indicate high surcharge levels and are represented with yellow nodes. A majority of the system pipes are either red or yellow, indicating that the peak flow is greater than 1.2 times the pipe capacity for most of the system. Overall, the existing sewer system does not have adequate capacity for the 5-year 4-hour rainfall event.

For a long duration (24-hour) rainfall event, the system performs significantly better due to its lower peak intensity. As shown in Figure 5.2, many nodes are blue or green and the HGL is still within the diameter of the pipe. A few select locations also indicate a HGL within 1.0 m below ground, as represented by a yellow node. However, these are shallow systems and the pipes have adequate capacity. Along 51st Street, there is one pipe surcharging above the pipe obvert, with the node indicating a HGL within 1.0 m below ground. In general, the system has adequate capacity to convey the 5-year 24-hour rainfall event.

5.5.2 25-Year Rainfall Event Results

The evaluation results show that for a short duration (4-hour) rainfall event, the stormwater system performance during the 25-year rainfall event is slightly worse than during the 5-year, 4-hour rainfall. The system is under higher surcharge conditions and several additional areas experience surface flooding. However, minor systems are not designed for large events. The 25-year 4-hour rainfall event results are demonstrated in Figure 5.3.

For a long duration (24-hour) rainfall event, the system generally has adequate capacity to convey rainfall, as indicated in Figure 5.4 by the green links. However, there are a few yellow nodes where pipes are surcharging, as indicated by the yellow links.

5.5.3 100-Year Rainfall Event Results

As expected, flooding will occur during a 100-year short duration rainfall event. Hydraulic conditions continue to get worse in the 100-year, 4-hour rainfall event, with the majority of the storm system flooding. The 100-year 4-hour rainfall event results are demonstrated in Figure 5.5.

For a long duration (24-hour), the system performs quite Well during this event, with flooding only occurring at a few nodes. The majority of the system continues to have capacity throughout the 100-year, 24-hour rainfall as illustrated in Figure 5.6.

5.6 Existing System Improvement

The 2011 MSP report adopted a level of service such that there would be no surcharging to within 1.0 m of ground level for a 5-year, 4-hour rainfall event. This level of service leads to the recommendation that 18,944 meters of the storm system would need to be upgraded. As there are not many available areas that would effectively provide storage within existing developed areas of the Town, the proposed improvements to the stormwater system would require pipe upgrades. Pipe upgrades were determined for both replacement and twinning options. The decision to twin or replace pipes would be based on the condition of existing pipes which would require that CCTV of the existing stormwater sewer system would need to be done.

5.6.1 Upgrades Required

Table 5.6 summarizes the proposed upgrades required to achieve the recommended level of service. Figure 5.7 shows the locations of the proposed pipe upgrades and Figure 5.8 shows the results of the improved system during a 5-year, 4-hour rainfall event. Nodes that are framed in a red box on Figure 5.8 have a bury depth less than 1.5 m and are not indicative of surcharging. Some surcharging, represented by yellow and red links are still present in the improved system during the 5-year, 4-hour rainfall event. However, these surcharges are localized and do not result in the HGL being within 1.0 m of the ground.

It is proposed that the stormwater sewers Links 1-055 to 1-053, 1-087 to 1-093, and 1-102 to 1-107 indicated on Figure 5.7 are regraded. The recommended level of service was best achieved by increasing the slope of the pipes in the areas. Keeping the existing slopes would result in significantly large pipe diameters, and may not necessarily result in meeting the recommended level of service.

Table 5.6 Proposed Stormwater Improvements

Link Names	Total Length of Improvements (m)	Existing Diameters (mm)	Replacement Diameters (mm)
1-008 – 1-017	517.4	450 - 675	600 - 900
1-018 – 1-077	2084.7	300 – 1,050	525 – 1,200
1-036 – 1-051 (includes regrading)	1661.6	300 – 525	450 – 1,200
1-078 – 1-101 (includes regrading)	2470	300 – 1,050	450 – 1,500
1-108 – 1-112	445.3	300 - 450	525 - 675
1-114 – 1-126	1528.8	300 -1,200	525 - 1,350
2-001 – C40	974.2	450 - 900	600 - 1,200
2-015	92.8	375	450
3-006	92.77	375	450
3-008 - 3-016	821.30	300 - 525	450 - 900
3-017 - 3-022	469.3	450	525 - 750
4-001 - 4-008	588.6	375 - 600	450 - 750
4-011 - 4-014	336.9	300 - 450	450 - 600
4-016 - 4-019	296.9	300 - 375	450 - 750
4-020 - C29	241.2	300	525
4-021 - 4-030	644.6	300 - 450	300 - 900
4-032 - 4-035	306.6	300	450 - 525
4-036 - 4-039_2	326.5	300 - 450	600 - 750
4-040 - 4-046	596.5	250 - 675	450 - 1,200
4-048 - C24	426.5	300 - 900	450 - 1,050
4-054 - 4-060	333.3	300 - 900	450 - 1,050
4-061 - 4-064	357.2	300 - 600	525 - 900
5-001 - 5-004	196.6	300 - 450	525
5-005 - 5-010	402.29	300 - 375	450 - 600
5-013 - 5-015	238.4	300 - 600	450 - 900
5-017 - 5-022	622.49	300 - 450	450 - 900
6-001 - C19	1335.9	375 - 675	450 -675
6-021 - 6-024	253	300 - 375	450 - 525
6-027	56.4	375	450

It should be noted that the stormwater improvements should also take into consideration utilizing streets and open spaces as a major conveyance and storage alternative, respectively. This may delay the requirement of storm pipe system upgrading. Major ponding locations should be identified prior to and after system upgrading. However, we recommended using the major drainage system if possible with some road regrading to provide overland flow routing to the creeks for major storm events. This should be considered as an alternative to pipe upgrading. In addition, the availability of lands such as parks, school sites (dual use concept) along these major drainage routes should be reviewed to determine the possibility of constructing a stormwater storage facility.

Acknowledging the current allowable discharge rates to the creeks, and in the event that the capacity of any of the creeks is exceeded due to additional flow from the future SWMFs, real time control (RTC) or adjusting the outflow rate for some of the SWMFs could be considered as a potential relief option.

5.6.2 Existing System Upgrade Cost Estimate

Revised cost estimates were prepared for the proposed stormwater sewer upgrades; based on the findings of the model, we have determined the upgrading requirements and associated costs for the undersized pipes to be **$46.142** million including contingency and engineering. The breakdown of the above cost estimate is given in Appendix D and as shown on Figure 5.7.

5.7 Future Storm Servicing Plan

A stormwater management plan was developed for the Town based on the 2032 and 2042 development scenarios which includes build out and servicing of the areas as shown in Figure 2.2, and referenced as Future Development Areas.

The future stormwater management plan is not dependent on completion of proposed upgrades to the existing stormwater system. This means the proposed stormwater management plan located in the new development areas will have a new stormwater systems.

Future development areas were delineated into additional stormwater drainage basins, as shown in Figure 5.9. Several of these storm drainage basins are expected to be partially developed in the next 5 to 15 years, and ultimately complete by 2042.

Table 5.7 summarizes the drainage basin areas, the proposed land use, and the impervious ratios for each of the proposed drainage basins.

Table 5.7 Stormwater Drainage Basins

Future 2032

Name	Year of Development	Drainage Basin Area (ha)	Land Use	Percent Imperviousness
A	2022 - 2032	11.4	Residential	50
G	2022 - 2032	53.1	Industrial/ Commercial	90
I	2022 - 2032	60.8	Industrial/ Commercial	90
J	2022 - 2032	91.7	Industrial/ Commercial	90
K	2022 - 2032	53	Industrial/ Commercial	90
N	2022 - 2032	58.1	Industrial/ Commercial	90
U	2022 - 2032	18.1	Industrial/ Commercial	90
V	2022 - 2032	20	Industrial/ Commercial	90

Name	Year of Development	Drainage Basin Area (ha)	Land Use	Percent Imperviousness
V-s	2022 - 2032	28.7	Industrial/ Commercial	90
W	2022 - 2032	26.3	Industrial/ Commercial	90
Y	2022 - 2032	56.1	Industrial/ Commercial	90
M-g	2022 - 2032	32.9	Residential	50
P	2022 - 2032	15.1	Residential	50

Future 2042

Name	Year of Development	Drainage Basin Area (ha)	Land Use	Percent Imperviousness
A	2042	60.1	Industrial/ Commercial	90
B	2042	51.3	Industrial/ Commercial	90
C	2042	63.7	Industrial/ Commercial	90
D	2042	58.6	Industrial/ Commercial	90
E	2042	57.3	Industrial/ Commercial	90
F	2042	75.4	Industrial/ Commercial	90
H	2042	74.1	Industrial/ Commercial	90
L	2042	77.1	Industrial/ Commercial	90
M	2042	33.7	Industrial/ Commercial	90
O	2042	37.1	Industrial/ Commercial	90
Q	2042	146	Industrial/ Commercial	90
X	2042	90.2	Industrial/ Commercial	90
Y	2042	93.5	Industrial/ Commercial	90
P	2042	22	Residential	50
R	2042	17.3	Residential	50
S	2042	39.9	Residential	50
T	2042	29	Residential	50

5.8 Future Stormwater Management System

Based on the allowable discharge rates for each receiving water course (refer to Section 5.3) and the drainage basin areas, the maximum allowable discharge rates for each proposed SWMF was determined based on the allowable discharge rate detailed in Table 5.1 for the Bench, Wase, and Poplar Creeks established in the Stormwater Management Plan (UMA, 2005).

Table 5.8 summarizes the maximum allowable discharge rates for each basin area.

Table 5.8 Stormwater Management Facilities Allowable Discharge Rate

Name	Drainage Basin Area (ha)	Receiving Watercourse	Maximum Discharge Rate (L/s)
2032			

Name	Drainage Basin Area (ha)	Receiving Watercourse	Maximum Discharge Rate (L/s)
A	11.4	Bench Creek	31.92
G	53.1	Bench Creek	148.68
I	60.8	Bench Creek	170.24
J	91.7	Bench Creek	256.76
K	53	Bench Creek	148.4
N	58.1	Bench Creek	162.68
U	18.1	Poplar Creek	162.9
V	20	Bench Creek	56
V-s	28.7	Bench Creek	80.36
W	26.3	Bench Creek	73.64
Y	56.1	Poplar Creek	157.08
M-g	32.9	Bench Creek	92.12
P	15.1	Bench Creek	42.28
2042			
A	60.1	Bench Creek	168.28
B	51.3	Bench Creek	143.64
C	63.7	Bench Creek	178.36
D	58.6	Bench Creek	164.08
E	57.3	Bench Creek	160.44
F	75.4	Bench Creek	211.12
H	74.1	Bench Creek	207.48
L	77.1	Bench Creek	215.88
M	33.7	Bench Creek	94.36
O	37.1	Bench Creek	103.88
Q	146	Bench Creek	408.8
X	90.2	Bench Creek	252.56
Y	93.5	Bench Creek	261.8
P	22	Bench Creek	61.6
R	17.3	Poplar Creek	155.7
S	39.9	Poplar Creek	359.1
T	29	Poplar Creek	261

5.9 Proposed Stormwater Management Facilities

SWMF is proposed to be designed as wet facilities to allow for settling of sediments from the runoff enhance the water quality before being released. The drainage system was assessed using the PC SWMM model.

As previously indicated, the SWMF's were proposed to be wet facilities simulated as having 2.5 m of dead storage, 2.0 m of live storage, and 0.5 m of freeboard. The SWMFs were modelled as trapezoidal shapes with side slope of 7:1 (H: V) from the high water level to 1.0 m below the normal water level (NWL) and side slopes of 3:1 (H: V) from 1.0 m below the NWL to the bottom of the pond. The plan view configurations of the SWMFs in each drainage basin would be customized in the future based on site-specific developments in each area.

Proposed SWMFs are established on the basis of the model simulation of two governing rainfall events - the 100-year, 4-hour Chicago rainfall event and 100-year, 24-hour Huff rainfall event. The rainfall event which produces the greatest runoff volume would be deemed the critical event that would governs the size of the SWMFs for each basin.

Table 5.9 summarizes the storage requirements of the proposed SWMFs. Basins A, P, and Y will require a lower storage volume for the interim 2022 - 2032 development scenario than for 2042 ultimate development scenario. When designing the interim SWMFs, sufficient land must be made available for the ultimate storage requirement.

Table 5.9 Storage Volumes for Stormwater Drainage Basins

Name	Drainage Basin Area (ha)	Critical Rainfall Event	Storage Volume (m^3)
2032			
A	11.4	100 Year 24 Hour	3,377
G	53.1	100 Year 24 Hour	30,991
I	60.8	100 Year 24 Hour	35,274
J	91.7	100 Year 24 Hour	53,351
K	53	100 Year 24 Hour	30,857
N	58.1	100 Year 24 Hour	33,705
U	18.1	100 Year 24 Hour	6,833
V	20	100 Year 24 Hour	11,586
V-s	28.7	100 Year 24 Hour	16,675
W	26.3	100 Year 24 Hour	15,051
Y	56.1	100 Year 24 Hour	32,158
M-g	32.9	100 Year 24 Hour	9,618
P	15.1	100 Year 24 Hour	4,464
2042			
A	60.1	100 Year 24 Hour	34,822
B	51.3	100 Year 24 Hour	29,774
C	63.7	100 Year 24 Hour	36,934
D	58.6	100 Year 24 Hour	34,242
E	57.3	100 Year 24 Hour	33,263
F	75.4	100 Year 24 Hour	43,885
H	74.1	100 Year 24 Hour	42,886

Name	Drainage Basin Area (ha)	Critical Rainfall Event	Storage Volume (m^3)
L	77.1	100 Year 24 Hour	44,689
M	33.7	100 Year 24 Hour	19,616
O	37.1	100 Year 24 Hour	21,576
Q_West	93.7	100 Year 24 Hour	53,241
Q_East	52.3	100 Year 24 Hour	30,063
X	90.2	100 Year 24 Hour	50,267
Y	93.5	100 Year 24 Hour	53,259
P	22	100 Year 24 Hour	6,668
R	17.3	100 Year 4 Hour	3,050
S	39.9	100 Year 4 Hour	7,065
T	29	100 Year 4 Hour	4,889

The location of SWMFs are very preliminary and conceptual. SWMFs will be located within local depressions or at low ends of the drainage basins. This approach provides a baseline of storm storage requirement. The location, pond connection details and operation strategy will depend on the detailed land development plan and staging. For example, in the fringe area at the west of the Town, the developments along Highway 16 could be possibly viewed as having a regional stormwater system. The proposed SWMFs could perhaps be combined into one or two larger facilities or the individual SWMFs connected in series servicing the entire fringe area.

All the proposed SWMFs for the future development scenarios discharge directly to the creeks. The proposed future SWMFs are located near the receiving bodies and it would be more cost effective and convenient to convey storm runoff to the creeks rather than having SWMFs discharge to the existing storm system which may require additional upgrading to accommodate discharge from future SWMFs.

Figure 5.10 shows the proposed future storm management plan, including proposed SWMF locations and proposed drainage routes.

5.10 Future System Cost Estimates

Cost estimates for the proposed SWMFs and associated outlets for the future scenarios are summarized in Table 5.10. This is the general cost, and the contingency of 30 percent and land cost were excluded.

Table 5.10 Future System Cost Estimates

	Commercial/Industrial Development	Residential Development	Total
2032	$18.728 million	$0.908 million	$19.636 million
2042	$32.623 million	$2.462 million	$35.085 million
Total			**$54.721 million**

The cost breakdown details are shown in Appendix D.

6. Transportation and Roadway Network

6.1 Introduction

6.1.1 Background

Future transportation improvements were presented in the Town's 1980 transportation study, adopted improvements incorporated in the transportation Section of the 1982 General Engineering Study. The Transportation and Roadway Network Conceptual Plan (Figure 6.1) was developed to support the Offsite Levy Bylaw found in the 2016 MDP. Changes to the 2016 MDP may be required based on subsequent Transportation Impact Assessments (TIA) to address development staging and trip demands. A staging plan is to be developed for the future transportation systems, including arterial, collector, and local roads and pedestrian facilities, corridors and linkages.

6.1.2 Retainer and Objective

This update to the 1980 transportation study will be done by undertaking the following:

- Establish baseline traffic conditions for the study area during a.m. and p.m. peak hours;
- Update existing traffic conditions to derive future operating conditions for the study intersections and corridors at future 2032 and 2042 planning horizons (15 and 25-year planning horizons);
- Assess the impact of existing and future traffic volumes at study intersections and corridors, and identify any operational concerns;
- Undertake all-way stop and traffic signal warrants at intersections where operational concerns are identified;
- Recommend intersection and/or corridor improvements for each horizon planning year, as required, to accommodate forecasted traffic volumes based on the results of the all-way stop and traffic signal warrants;
- Analyze the Improved Conditions scenarios under existing and future planning horizons to ensure acceptable operating conditions;
- Update the Transportation and Roadway Network Conceptual Plan accordingly with recommended intersection and/or corridor improvements;
- Develop a Rehabilitation Strategy with pavement rehabilitation recommendations resulting from the road classification assessment; and
- Include consideration and discussion of Complete Streets, what conditions will be reached for a given level of funding, and what budget will be required to reach a given level of condition through the planning horizons.

6.1.3 Study Area

The intersections under review are shown in Figure G-1, Appendix G, and are listed as follows:

1. 4th Avenue at 70th Street;
2. 2nd Avenue at 63rd Street;
3. 10th Avenue at 56th Street;
4. 13th Avenue at Edson Drive;
5. Highway 748 at Edson Drive;
6. 9th Avenue at 50th Street;
7. 4th Avenue at 50th Street;
8. 2nd Avenue at 50th Street;
9. Highway 16 at 42nd Street; and
10. Highway 16 at 25th Street.

In addition, based on a review of the Town's road networks, the following screenline locations were selected for inclusion in the corridor analysis component of the study, these are shown in Figure G-2, Appendix G:

1. 70th Street (north of 4th Avenue)
2. 70th Street (south of 4th Avenue)
3. 4th Avenue (east of 70th Street)
4. 4th Avenue (west of 70th Street)
5. 63rd Street (north of 2nd Avenue)
6. Willmore Park Drive (south of 2nd Avenue)
7. 2nd Avenue (east of 63rd Street)
8. 2nd Avenue (west of 63rd Street)
9. 56th Street (north of 10th Avenue)
10. 56th Street (south of 10th Avenue)
11. 10th Avenue (east of 56th Street)
12. 10th Avenue (west of 56th Street)
13. Edson Drive (north of 13th Avenue)
14. 51st Street (south of 13th Avenue)
15. 12th Avenue (east of Edson Drive)
16. 13th Avenue (west of Edson Drive)
17. Hwy 748 (north of Edson Bypass)
18. Edson Drive (south of Edson Bypass)
19. Hwy 748 (east of Edson Drive)

    Edson Bypass (west of Edson Drive)

20. 50th Street (north of 9th Avenue)
21. 50th Street (south of 9th Avenue)
22. 9th Avenue (east of 50th Street)
23. 9th Avenue (west of 50th Street)
24. 50th Street (north of 4th Avenue)
25. 50th Street (south of 4th Avenue)
26. 4th Avenue (east of 50th Street)
27. 4th Avenue (west of 50th Street)
28. 50th Street (north of 2nd Avenue)
29. 50th Street (south of 2nd Avenue)
30. 2nd Avenue (east of 50th Street)
31. 2nd Avenue (west of 50th Street)
32. 42nd Street (north of Highway 16)
33. 2nd Avenue (south of Highway 16)
34. Highway 16 (east of 42nd Street)
35. Highway 16 (west of 42nd Street)
36. 25th Street (north of Highway 16)
37. Range Road 171 (south of Highway 16)

6.2 Existing Conditions

6.2.1 Existing Road Network

The Town's existing road network is illustrated in Figure 6.1, Appendix H, which also identifies roads classified as arterial, collector, or local. GHD has revised the proposed collector and arterial roads shown in the Town's Figure 6.1 since the Town's 2003 Offsite Levy Study. The proposed roads will be discussed further in this section.

Arterial Roads

The primary consideration for arterial roads is traffic movement with land access being a secondary consideration and uninterrupted flow being desirable except at signals and crosswalks. As per the Town's 1982 General Engineering Study, the preferred range of average daily traffic (ADT) is 10,000 to 30,000 vehicles per day for urban conditions and 1,000 to 12,000 for rural conditions. The Town's existing arterial roads are as follows:

- Highway 16;
- 4th Avenue;
- 2nd Avenue;
- 5th Avenue/Glenwood Drive;
- Edson Drive/51st Street north of 2nd Avenue;
- Bear Lake Road; and
- 25 Street north of Highway 16.

Collector Roads

For collector roads, both traffic movement and land access are considered to be of equal importance, with uninterrupted flow being desirable. As per the Town's 1982 General Engineering Study, the preferred range of average daily traffic (ADT) is 1,000 to 12,000 vehicles per day for urban conditions and 1,000 to 5,000 for rural conditions. The Town's existing collector roads are as follows:

- 72 Street;
- 63 Street;
- 54 Street south of 4th Avenue ;
- 48 Street south of 12 Avenue;
- 46 Street north of 6 Avenue;
- 55 Street from 13 Avenue to 10 Avenue;
- 15 Avenue from 66 Street to 63 Street;
- 13 Avenue/12 Avenue;
- 10 Avenue from 55 Street to 48 Street;

- 6 Avenue; and
- Aspen Drive.

Local Roads

The primary consideration for local roads is land access, with traffic movement a secondary consideration and interrupted flow being acceptable and sometimes desirable. As per the Town's 1982 General Engineering Study, the preferred range of average daily traffic (ADT) is less than 1,000 vehicles per day for urban and rural conditions.

6.2.2 Existing Traffic Volumes

Intersection turning movement counts were conducted at the study area intersections from 7:30 a.m. to 9:30 a.m. and from 4:00 p.m. to 6:00 p.m., in the last week of November 2017 and first week of December 2017. The a.m. and p.m. peak hour intersection traffic volumes are presented in Figure G-3, Appendix G, with the detailed turning movement count sheets provided in the ancillary binder, which accompanies this report, entitled Transportation and Roadway Network Reports.

6.3 Future Conditions

6.3.1 Traffic Volume Growth

As shown in Figure 6.1, future development within the 2032 and 2042 planning horizons (15 and 25-year planning horizons) is expected to be generally concentrated to the periphery of the Town. Therefore, it would be expected that the future traffic growth will generally be more concentrated along the road network in these peripheral areas, and to a lesser extent within the generally built-out central area of the Town.

However, as a conservative measure, the compound annual growth rate of 2 percent as consistent with the overall MSP update, was conservatively applied to all turning movements at all study area intersections during both peak hour periods.

Furthermore, in addition to the corridor growth, GHD has applied additional traffic volumes representing future estimated site trips generated from the following future background developments:

- Techmation Industrial Park located on the west side of 26 Street, north of Highway 16 and west of Highway 748 (25 Street), as per the Traffic Impact Assessment undertaken by WSP dated March 2017; and
- Hillendale Subdivision Phase 2 located in the southwest corner of Highway 748 at 40 Street, as per the Traffic Impact Assessment undertaken by GENIVAR dated September 2009.

The forecasted 2032 and 2042 future peak hour volumes at the study area intersections are presented in Figure G-4 and Figure G-5, in Appendix G, respectively.

6.4 Intersection Capacity Analysis

6.4.1 Methodology

The capacity analysis identifies how effectively the intersections are operating. The analysis contained within this report utilized the Highway Capacity Manual (HCM) procedure within the Synchro Version 9 Software package. The following key traffic characteristics of each study intersection are presented in the following sections.

Volume to Capacity Ratio (v/c Ratio)

The ratio of flow rate to capacity (v/c ratio) is typically referred to as degree of saturation. Sustainable v/c ratios range from zero to 1.0 flow rate equals capacity. v/c ratios greater than 1.0 indicate that demand exceeds capacity.

Level of Service (LOS)

The LOS is a function of the average vehicle control delay and is characterized using the quantitative and qualitative measures shown in Table 6.1:

Table 6.1 Level of Service Measures

LOS	Delay at Signalized Intersections	Delay at Unsignalized Intersections	Description
A	≤10 seconds	≤10 seconds	**Free flow**. Traffic flows at or above the posted speed limit and motorists have complete mobility between lanes. The average spacing between vehicles is about 167 m or 27 car lengths. Motorists have a high level of physical and psychological comfort. The effects of incidents or point breakdowns are easily absorbed. LOS A generally occurs late at night in urban areas and frequently in rural areas.
B	10 to 20 seconds	10 to 15 seconds	**Reasonably free flow**. Speeds are maintained at levels similar to LOS A, maneuverability within the traffic stream is slightly restricted. The lowest average vehicle spacing is about 100 m or 16 car lengths. Motorists still have a high level of physical and psychological comfort.
C	20 to 35 seconds	15 to 25 seconds	**STable flow, at or near free flow**. Ability to maneuver through lanes is noticeably restricted and lane changes require more driver awareness. Minimum vehicle spacing is about 67 m or 11 car lengths. Most experienced drivers are comfortable, roads remain safely below but efficiently close to capacity, and posted speed is maintained. Minor incidents may still have no effect but localized service will have noticeable effects and traffic delays will form behind the incident. This is the target LOS for some urban and most rural highways.
D	35 to 55 seconds	25 to 35 seconds	**Approaching unsTable flow**. Speeds slightly decrease as traffic volume slightly increases. Freedom to maneuver within the traffic stream

Table 6.1 Level of Service Measures

LOS	Delay at Signalized Intersections	Delay at Unsignalized Intersections	Description
			is much more limited and driver comfort levels decrease. Vehicles are spaced about 50m or 8 car lengths. Minor incidents are expected to create delays. Examples are a busy shopping corridor in the middle of a weekday, or a functional urban Highway during commuting hours. It is a common goal for urban Streets during peak hours, as attaining LOS C would require prohibitive cost and societal impact in bypass roads and lane additions.
E	55 to 80 seconds	35 to 50 seconds	**UnsTable flow**, operating at capacity. Flow becomes irregular and speed varies rapidly because there are virtually no usable gaps to maneuver in the traffic stream and speeds rarely reach the posted limit. Vehicle spacing is about 6 car lengths, but speeds are still at or above 80 km/h. Any disruption to traffic flow, such as merging ramp traffic or lane changes, will create a shock wave affecting traffic upstream. Any incident will create serious delays. Drivers' level of comfort become poor. This is a common standard in larger urban areas where some roadway congestion is inevitable.
F	>80 seconds	>50 seconds	**Forced or breakdown flow**. Every vehicle moves in lockstep with the vehicle in front of it, with frequent slowing required. Travel time cannot be predicted, with generally more demand than capacity. A road in a constant traffic jam is at this LOS because LOS is an average or typical service rather than a constant state. For example, a Highway might be at LOS D for the AM peak hour, but have traffic consistent with LOS C some days, LOS E or F other days, and come to a halt once every few weeks.

6.4.1.1 95th percentile Queue Length

The 95th percentile queue length is the queue length that is expected to be exceeded only 5 percent of the time during the a.m. and p.m. peak hours. It is a common measure used in determining an adequate length for auxiliary turning lanes.

6.4.1.2 Identification of Operational Issues

In accordance with typical traffic analysis guidelines for municipalities, the intersection capacity analysis in this report includes identification of conditions at signalized intersections where:

- Volume/capacity (v/c) ratios for through movements or shared through/turning movements increased to 0.85 or above;
- V/c ratios for exclusive movements increased to 1.00 or above; or
- 95th percentile queues for an individual movement are projected to exceed available turning lane storage.

The intersection capacity analysis in this report included identification of conditions at unsignalized intersections where:

- Level of service is LOS "D" or greater; or
- 95th percentile queues for an individual movement are projected to exceed available turning lane storage.

6.4.1.3 Intersection Capacity Analysis Results

The following sections summarize the HCM procedure results for the study intersections during the weekday a.m. and p.m. hours under existing 2017 and future 2032 and 2042 traffic conditions. Detailed calculation sheets are provided in the ancillary binder.

4th Avenue at 70th Street

Table 6.2 Capacity analysis of 4th Avenue at 70th Street

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	PM Peak Hour
Existing 2017	• NBLT: 0.04 (C) <1 veh • SBTR: 0.23 (C) < 1 veh	• NBLT: 0.05 (C) <1 veh • SBTR: 0.17 (C) < 1 veh
Future 2032	• NBLT: 0.08 (C) <1 veh • SBTR: 0.44 (D) 15m	• NBLT: 0.09 (C) <1 veh • SBTR: 0.31 (D) 9m
Future 2042	• NBLT: 0.16 (E) <1 veh • SBTR: 0.74 **(F)** 33m	• NBLT: 0.17 (D) <1 veh • SBTR: 0.51 (E) 18m

Under existing and future conditions, this two-way stop controlled intersection is expected to operate generally well, although the southbound through/right-turn movement is expected to reach LOS 'F' during the a.m. peak hour by 2042, representing forced or breakdown flow. The 95th percentile queue is reported at 33 meters (or five vehicles).

2nd Avenue at 63rd Street

Table 6.3 Capacity analysis of 2 Avenue at 63 Street

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	PM Peak Hour
Existing 2017	• NBTR: 0.02 (A) <1 veh • SBLT: 0.16 (B) <1 veh	• NBTR: 0.04 (B) <1 veh • SBLT: 0.23 (B) <1 veh
Future 2032	• NBTR: 0.02 (B) <1 veh • SBLT: 0.25 (B) 1 veh	• NBTR: 0.07 (B) <1 veh • SBLT: 0.39 (C) 13m
Future 2042	• NBTR: 0.03 (B) <1 veh • SBLT: 0.35 (C) 11m	• NBTR: 0.10 (B) <1 veh • SBLT: 0.58 (D) 25m

Under existing and future conditions, this two-way stop controlled intersection is expected to operate very well, with low levels of delay and nominal queueing.

10th Avenue at 56th Street

Table 6.4 Capacity analysis of 10th Avenue at 56th Street

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	**PM Peak Hour**
Existing 2017	• EBLTR: 0.21 (A) <1 veh • WBLTR: 0.06 (A) <1 veh • NBLTR: 0.18 (A) <1 veh • SBLTR: 0.22 (A) <1 veh	• EBLTR: 0.13 (A) <1 veh • WBLTR: 0.10 (A) <1 veh • NBLTR: 0.31 (A) 9m • SBLTR: 0.13 (A) <1 veh
Future 2032	• EBLTR:0.31 (A) 9m • WBLTR: 0.09 (A) <1 veh • NBLTR: 0.27 (A) 8m • SBLTR:0.32 (B) 10m	• EBLTR: 0.19 (A) <1 veh • WBLTR: 0.14 (A) <1 veh • NBLTR: 0.45 (B) 16m • SBLTR: 0.20 (A) <1 veh
Future 2042	• EBLTR: 0.41 (B) 14m • WBLTR: 0.12 (A) <1 veh • NBLTR: 0.35 (B) 11m • SBLTR: 0.42 (B) 15m	• EBLTR: 0.25 (A) 1 veh • WBLTR: 0.18 (A) <1 veh • NBLTR: 0.57 (B) 26m • SBLTR: 0.25 (A) 1 veh

Under existing and all future conditions, this all-way stop controlled intersection is expected to operate very well, with low levels of delay and nominal queueing.

13th Avenue at Edson Drive

Table 6.5 Capacity analysis of 13th Avenue at Edson Drive

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	**PM Peak Hour**
Existing 2017	• EBLTR: 0.61 (C) 29m • WBLTR: 0.41 (B) 14m • NBLTR: 0.26 (B) 1 veh • SBLTR: 0.60 (C) 28m	• EBLTR:0.23 (A) <1 veh • WBLTR: 0.26 (B) 1 veh • NBLTR: 0.32 (B) 10m • SBLTR: 0.30 (B) 8m
Future 2032	• EBLTR: **1.06 (F)** 103m • WBLTR: 0.75 (D) 41m • NBLTR: 0.49 (C) 17m • SBLTR: **1.04 (F)** 98m	• EBLTR:0.36 (B) 11m • WBLTR: 0.40 (B) 13m • NBLTR: 0.49 (B) 18m • SBLTR: 0.46 (B) 17m
Future 2042	• EBLTR: **1.54 (F)** 186m • WBLTR: **1.14 (F)** 66m • NBLTR: 0.75 (D) 26m • SBLTR: **1.49 (F)** 178m	• EBLTR: 0.52 (C) 20m • WBLTR: 0.58 (C) 25m • NBLTR: 0.70 (C) 39m • SBLTR: 0.65 (C) 33m

Under existing and future conditions, this all-way stop controlled intersection is expected to operate very Well during the p.m. peak hour. However, it is expected to reach capacity in the eastbound and southbound directions by 2032, and the westbound direction by 2042, during the a.m. peak hour. These movements are also expected to reach LOS ‘F’ during the a.m. peak hour, representing forced or breakdown flow. The reported 95th percentile queue for the eastbound movement is expected to reach 186 meters (or 27 vehicles), 66 meters (or nine vehicles) for the westbound movement, and 178 meters (or 25 vehicles) for the southbound movement, in 2042.

Highway 748 at Edson Drive

Table 6.6 Capacity analysis of Highway 748 at Edson Drive

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	PM Peak Hour
Existing 2017	EBLTR: 0.10 (B) <1 veh WBLTR: 0.15 (B) <1 veh NBL: 0.01 (A) <1 veh NBT: 0.00 (A) <1 veh SBL: 0.02 (A) <1 veh SBT: 0.00 (A) <1 veh	EBLTR: 0.17 (B) <1 veh WBLTR: 0.32 (B) 10m NBL: 0.02 (A) <1 veh NBT: 0.00 (A) <1 veh SBL: 0.02 (A) <1 veh SBT: 0.00 (A) <1 veh
Future 2032	EBLTR: 0.14 (B) <1 veh WBLTR: 0.23 (B) <1 veh NBL: 0.01 (A) <1 veh NBT: 0.00 (A) <1 veh SBL: 0.02 (A) <1 veh SBT:0.00 (A) <1 veh	EBLTR: 0.27 (B) 8m WBLTR: 0.53 (C) 22m NBL: 0.03 (A) <1 veh NBT: 0.00 (A) <1 veh SBL: 0.03 (A) <1 veh SBT: 0.00 (A) <1 veh
Future 2042	EBLTR: 0.19 (B) <1 veh WBLTR: 0.32 (B) 9m NBL: 0.01 (A) <1 veh NBT: 0.00 (A) <1 veh SBL: 0.03 (A) <1 veh SBT: 0.00 (A) <1 veh	EBLTR: 0.38 (C) 12m WBLTR: 0.80 (E) 48m NBL: 0.04 (A) <1 veh NBT: 0.00 (A) <1 veh SBL: 0.04 (A) <1 veh SBT: 0.00 (A) <1 veh

Under existing and future conditions, this two-way stop controlled intersection is expected to operate very well, with low levels of delay and nominal queueing.

9th Avenue at 50th Street

Table 6.7 Capacity analysis of 9th Avenue at 50th Street

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	PM Peak Hour
Existing 2017	EBLTR: 0.02 (A) <1 veh WBLTR: 0.04 (A) <1 veh NBLTR: 0.22 (A) <1 veh SBLTR: 0.09 (A) <1 veh	EBLTR: 0.01 (A) <1 veh WBLTR: 0.02 (A) <1 veh NBLTR: 0.19 (A) <1 veh SBLTR: 0.13 (A) <1 veh
Future 2032	EBLTR: 0.03 (A) <1 veh WBLTR: 0.06 (A) <1 veh NBLTR: 0.30 (A) 8m SBLTR:0.12 (A) <1 veh	EBLTR: 0.01 (A) <1 veh WBLTR: 0.03 (A) <1 veh NBLTR: 0.27 (A) 8m SBLTR: 0.18 (A) <1 veh
Future 2042	EBLTR: 0.04 (A) <1 veh WBLTR: 0.07 (A) <1 veh NBLTR: 0.37 (A) < 12m SBLTR: 0.15 (A) <1 veh	EBLTR: 0.01 (A) <1 veh WBLTR: 0.04 (A) <1 veh NBLTR: 0.33 (A) 10m SBLTR: 0.22 (A) <1 veh

Under existing and future conditions, this all-way stop controlled intersection is expected to operate very well, with low levels of delay and nominal queueing.

4th Avenue at 50th Street

Table 6.8 Capacity analysis of 4th Avenue at 50th Street

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	**PM Peak Hour**
Existing 2017	Overall 0.41 (B) WBT: 0.69 (B) 58m WBR: 0.04 (B) <1 veh NBLT: 0.11 (A) 15m SBTR: 0.18 (B) 21m	Overall 0.50 (B) WBT: 0.68 (B) 38m WBR: 0.09 (A) <1 veh NBLT: 0.34 (A) 24m SBTR: 0.14 (A) 9m
Future 2032	Overall 0.54 (B) WBT: 0.75 (B) 85m WBR: 0.05 (A) <1 veh NBLT: 0.18 (B) 27m SBTR: 0.29 (B) 40m	Overall 0.61 (B) WBT: 0.76 (C) 88m WBR: 0.10 (B) 8m NBLT: 0.45 (B) 64m SBTR: 0.18 (B) 20m
Future 2042	Overall 0.65 (B) WBT: 0.81 (B) 117m WBR: 0.05 (A) <1 veh NBLT: 0.26 (C) 34m SBTR: 0.40 (C) 51m	Overall 0.74 (C) WBT: 0.84 (C) 119m WBR: 0.11 (B) 10m NBLT: 0.61 (C) 81m SBTR: 0.24 (B) 29m

Under existing and future conditions, this signalized intersection is expected to operate very well, with substantial reserve capacity and nominal queueing.

2nd Avenue at 50th Street

Table 6.9 Capacity analysis of 2 Avenue at 50 Street

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	**PM Peak Hour**
Existing 2017	Overall 0.29 (B) EBT: 0.52 (B) 24m EBR: 0.01 (A) <1 veh NBTR: 0.05 (A) <1 veh SBLT: 0.13 (A) 11m	Overall 0.36 (B) EBT: 0.54 (B) 34m EBR: 0.01 (A) <1 veh NBTR: 0.14 (A) 10m SBLT: 0.19 (A) 13m
Future 2032	Overall 0.38 (B) EBT: 0.62 (B) 33m EBR: 0.02 (A) <1 veh NBTR: 0.07 (A) <1 veh SBLT: 0.19 (A) 15m	Overall 0.48 (B) EBT: 0.64 (B) 49m EBR: 0.01 (A) <1 veh NBTR: 0.20 (B) 15m SBLT: 0.28 (B) 20m
Future 2042	Overall 0.46 (B) EBT: 0.71 (B) 43m EBR: 0.02 (A) <1 veh NBTR: 0.09 (A) 1 veh SBLT: 0.24 (A) 17m	Overall 0.59 (B) EBT: 0.78 (B) 65m EBR: 0.01 (A) <1 veh NBTR: 0.24 (B) 18m SBLT: 0.36 (B) 25m

Under existing and future conditions, this signalized intersection is expected to operate very well, with substantial reserve capacity and nominal queueing.

Highway 16 at 42nd Street

Table 10 Capacity analysis of Highway 16 at 42nd Street

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	PM Peak Hour
Existing 2017	NBL: 0.09 (B) <1 veh NBT: 0.14 (B) <1 veh SBL: 0.12 (C) <1 veh SBR: 0.08 (B) <1 veh	NBL: 0.10 (B) <1 veh NBT: 0.31 (C) 9m SBL: 0.06 (C) <1 veh SBR: 0.08 (B) <1 veh
Future 2032	NBL: 0.19 (C) <1 veh NBT: 0.45 (C) <1 veh SBL: 0.78 **(F)** <1 veh SBR: 0.28 (B) <1 veh	NBL: 0.20 (C) <1 veh NBT: **1.16 (F)** 17m SBL: 0.33 (B) <1 veh SBR: 0.33 (B) <1 veh
Future 2042	NBL: 0.28 (C) <1 veh NBT: 0.62 (E) <1 veh SBL: **1.45 (F)** 11m SBR: 0.34 (C) <1 veh	NBL: 0.30 (C) <1 veh NBT: **1.55 (F)** 27m SBL: 0.40 (C) <1 veh SBR: 0.40 (C) <1 veh

Under existing conditions this two-way stop controlled intersection is expected to be operating generally well. However, during the a.m. peak hour the southbound left-turn movement is expected to experience significant delays by 2032, and then reach capacity by 2037, and during the p.m. peak hour the northbound through movement is expected to experience significant delays and reach capacity by 2032, both of which representing forced or breakdown flow.

Highway 16 at 25 Street

Table 11 Capacity analysis of Highway 16 at 25 Street

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	PM Peak Hour
Existing 2017	EBL: 0.05 (A) <1 veh WBL: 0.00 (A) <1 veh NBLTR: 0.02 (C) <1 veh SBLTR: 0.22 (B) <1 veh	EBL: 0.08 (A) <1 veh WBL:0.00 (A) <1 veh NBLTR: 0.03 (C) <1 veh SBLTR: 0.20 (B) <1 veh
Future 2032	EBL: 0.11 (E) <1 veh WBL: 0.20 (B) <1 veh NBLTR: 0.00 (A) <1 veh SBLTR: **1.36 (F)** 12m	EBL: 0.24 **(F)** <1 veh WBL: 0.20 (A) <1 veh NBLTR: 0.00 (A) <1 veh SBLTR: **1.27 (F)** 12m
Future 2037	EBL: 0.15 **(F)** <1 veh WBL: 0.23 (B) <1 veh NBLTR: 0.00 (A) <1 veh SBLTR: **1.96 (F)** 17m	EBL: 0.40 **(F)** <1 veh WBL: 0.23 (B) <1 veh NBLTR: 0.00 (B) <1 veh SBLTR: **1.81 (F)** 18m
Future 2042	EBL: 0.32 **(F)** <1 veh WBL: 0.27 (B) <1 veh NBLTR: 0.00 (A) <1 veh SBLTR: **3.05 (F)** 23m	EBL: 0.72 **(F)** <1 veh WBL: 0.27 (B) <1 veh NBLTR: 0.00 (B) <1 veh SBLTR: **2.81 (F)** 25m

Under existing conditions this two-way stop controlled intersection is expected to be operating generally well. However, the southbound approach is expected to experience significant delays and reach capacity by 2032 representing forced or breakdown flow, with the eastbound left-turn movement experiencing significant delays as well.

6.4.2 Summary of Intersection Issues

The following is a summary of the intersection issues identified in the previous sections.

- **4th Avenue at 70th Street:** The southbound through/right-turn movement is expected to reach LOS ‘F’ during the a.m. peak hour by 2042, representing forced or breakdown flow. The 95th percentile queue is reported at 33 meters (or five vehicles).
- **13th Avenue at Edson Drive:** The intersection is expected to reach capacity in the eastbound and southbound directions by 2032, as well as the westbound direction by 2037, during the a.m. peak hour. These movements are also expected to reach LOS ‘F’ during the a.m. peak hour, representing forced or breakdown flow. The reported 95th percentile queue for the eastbound movement is expected to reach 186 meters (or 27 meters), 66 meters (or nine vehicles) for the westbound movement, and 178 meters (or 25 vehicles) for the southbound movement, in 2042.
- **Highway 16 at 42nd Street:** During the a.m. peak hour the southbound left-turn movement is expected to experience significant delays by 2032, and then reach capacity by 2037, and during the p.m. peak hour the northbound through movement is expected to experience significant delays and reach capacity by 2032, both of which representing forced or breakdown flow.
- **Highway 16 at 25th Street:** The southbound approach is expected to experience significant delays and reach capacity by 2032 representing forced or breakdown flow, with the eastbound left-turn movement experiencing significant delays as well.

6.5 All-Way Stop and Traffic Signal Warrants

In response to the operational issues identified above, all-way stop and/or traffic signal warrants were conducted for the intersections reporting issues. The results of the undertaken warrants are reported below. As a conservative measure, forecasted 2042 intersection volumes were utilized in the warrant process.

6.5.1 Transportation Association of Canada (TAC) All-Way Stop Warrant

The all-way stop warrants were completed based on the TAC methodology. All-way stop control may be warranted under one or more of the following conditions:

- Where traffic volumes on intersecting roads are approximately equal and combined pedestrian and vehicular volumes on the minor road average 200 vehicles per hour (vph) for an eight-hour period;
- Where the average delay of minor road vehicular traffic entering an intersection exceeds 30 seconds per vehicle during the peak hour;
- Where traffic signals are not warranted and a collision problem exists as indicated by five or more collisions per year of a type which may be prevented by an all-way stop sign installation;
- As an interim measure prior to installation of traffic signals;
- As an interim measure, for a period of approximately one month prior to switching the stop control from one road to an intersecting road and the subsequent removal of existing stop signs on the first road.

6.5.2 TAC Traffic Signal Warrant

Traffic signal warrants were completed based on the TAC methodology, with completed signal warrant worksheets provided in the ancillary binder. The average hourly volume of future conditions is calculated in a conservative fashion using the sum of a.m. and p.m. peak hour volumes divided by four. This conservative average hourly volume is used for each of the six analyze hours.

6.5.3 Warrant Results

4th Avenue at 70th Street

This intersection is currently a two-way stop controlled intersection, therefore both an all-way stop controlled warrant and a signal warrant were undertaken. The results of the all-way stop warrant are presented in Table 6.12. Based on the results, the volume split between the intersecting roads is considered too excessive for all way stop control to be recommended. Approximately 91 percent of the peak hour volume is travelling along 4th Avenue. Consequently, all-way stop control for this would be very problematic from an operational and safety standpoint.

Based on the traffic signal warrant results, traffic signals are not warranted. Details of the traffic signal warrant are provided in the ancillary binder.

Table 12 4th Avenue at 70th Street Volume Split

Peak Hour	Volume Split	AWS Warranted?
p.m.	91 percent (4 Ave) to 9 percent (70 St)	No. Volume split is too excessive

13 Avenue at Edson Drive

This intersection is currently an all-way stop controlled intersection, therefore a signal warrant was undertaken. Based on the traffic signal warrant, traffic signals are not warranted.

Highway 16 at 42 Street

This intersection is currently a two-way stop controlled intersection. It is not recommended all-way stop control be considered on Highway 16 for obvious operational and safety concerns. Based on the appended traffic signal warrant, traffic signals are not warranted.

Highway 16 at 25 Street

This intersection is currently a two-way stop controlled intersection. It is not recommended all-way stop control be considered on Highway 16 for obvious operational and safety concerns. Based on the appended traffic signal warrant, traffic signals **are warranted**.

6.6 Corridor Capacity Analysis

Corridor capacity analysis was undertaken for the screenline locations listed in Section 6.1.3 to determine existing and future 2042 corridor utilization estimates.

6.6.1 Methodology

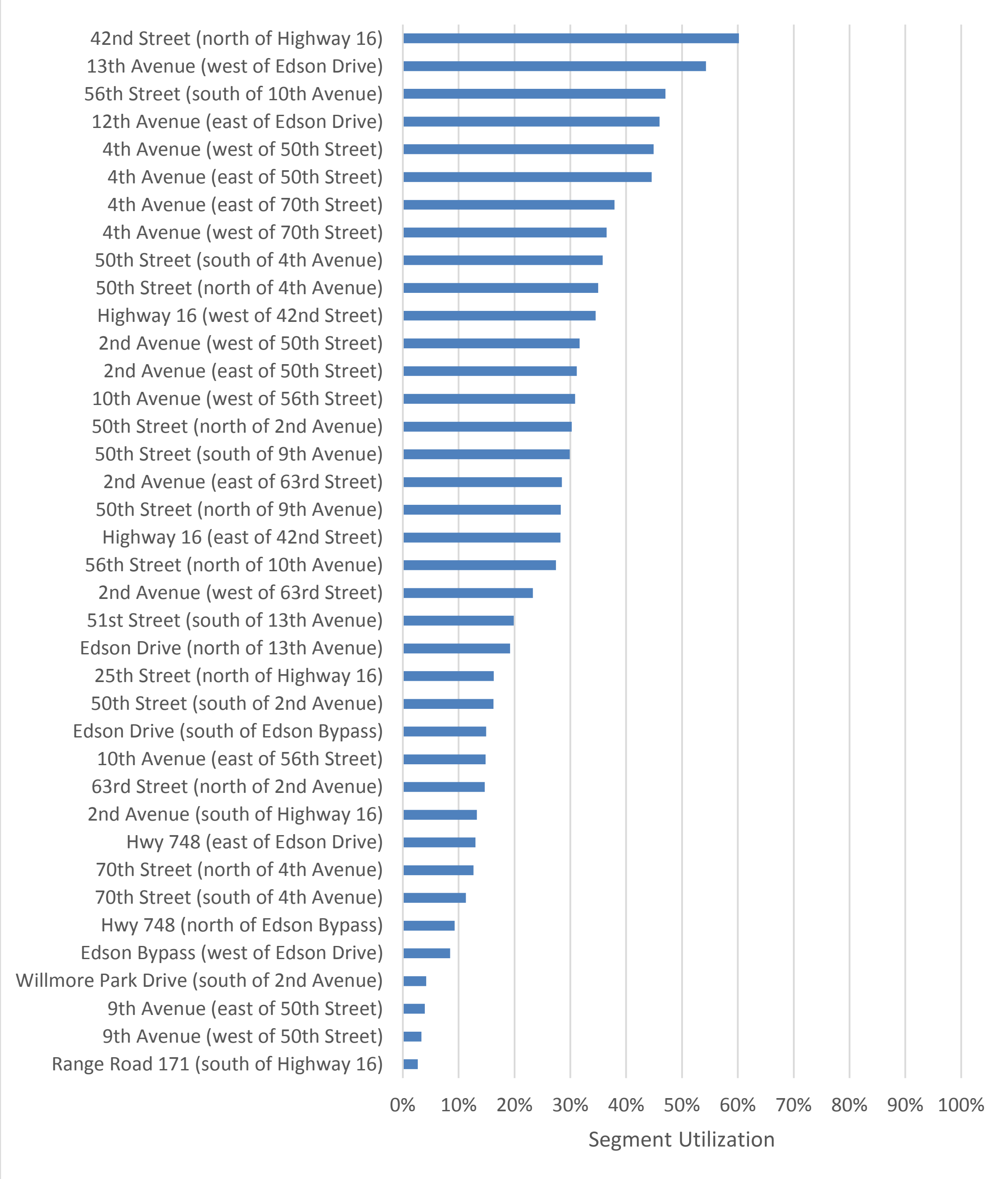
GHD undertook a screenline analysis based on the traffic data collected at the study area intersections. Two-way peak hour traffic data was converted to average daily traffic (ADT), and then compared to the Town's recommended roadway capacity based on the road classification in order to calculate an estimated existing utilization. Future ADT values were then forecasted to the future 2042 horizon year assuming the conservatively applied 2 percent compound annual growth rate. The future ADT values were compared to the Town's recommended roadway capacity based on the road classification to calculate an estimated future utilization estimates.

6.6.2 Results of Corridor Capacity Analysis

A summary of results of the corridor capacity analysis are shown in the Corridor Capacity Analysis Table which can be found in Appendix G. The Town's recommended capacities, which are consistent with those recommended by the TAC are an ADT of 12,000 vehicles for collector roads and 30,000 vehicles for arterial roads.

6.6.3 Summary of Corridor Issues

Based on the results of the corridor capacity analysis, none of the study corridors are expected to reach capacity by 2042. The corridor expected to experience the highest utilization by 2042 is 42 Street north of Highway 16 at 60 percent utilized. The remaining corridors in the analysis are not expected exceed 54 percent by 2042. The 2042 corridor utilization estimates are illustrated in the below graph.



Graph 6.1: 2042 Segment Utilization

6.7 Recommended Capacity Improvements

6.7.1 Recommended Intersection Improvements

Based on the results of the intersection capacity analysis and the completed control warrants, the recommended intersection improvements are presented in the following sections:

4th Avenue at 70th Street

The intersection is expected to reach capacity in the eastbound and southbound directions in the 2032 scenario, as Well as the westbound direction before the 2042 (estimate 2037) scenario, during the a.m. peak hour. These movements are also expected to reach LOS 'F' during the a.m. peak hour, representing forced or breakdown flow. The reported 95th percentile queue is expected to reach 186 meters (or 27 vehicles) for the eastbound movement, 66 meters (or nine vehicles) for the westbound movement, and 178 meters (or 25 vehicles) for the southbound movement, in the 2042 scenario.

The existing all-way stop controlled intersection does not warrant an upgrade to traffic signal control. However, due to future capacity concerns, the Town should consider monitoring the intersection in the future. The intersection is currently not experiencing operational issues during the a.m. and p.m. peak hours, and there is some uncertainty whether or not the forecasted growth as projected will result in the future operating issues.

A sensitivity analysis found that geometric improvements (i.e. auxiliary left and right-turn lanes) are not expected to result in any operational benefit.

Therefore, no improvements are recommended at this intersection at this time. However, it is recommended the Town monitor the intersection for potential future operational issues that may justify signalization at that time.

13 Avenue at Edson Drive

The intersection is expected to reach capacity in the eastbound and southbound directions in the 2032 scenario, as Well as the westbound direction in the 2037 scenario, during the a.m. peak hour. These movements are also expected to reach LOS 'F' during the a.m. peak hour, representing forced or breakdown flow. The reported 95th percentile queue is expected to reach 186 meters (or 27 vehicles) for the eastbound movement, 66 meters (or nine vehicles) for the westbound movement, and 178 meters (or 25 vehicles) for the southbound movement, in the 2042 scenario.

The existing all-way stop controlled intersection does not warrant an upgrade to traffic signal control. However, due to future capacity concerns, the Town should consider monitoring the intersection in the future. The intersection is currently not experiencing operational issues during the a.m. and p.m. peak hours, and there is some uncertainty whether or not the forecasted growth as projected will result in the future operating issues.

A sensitivity analysis found that geometric improvements (i.e. auxiliary left and right-turn lanes) are not expected to result in any operational benefit.

Therefore, no improvements are recommended at this intersection at this time. However, it is recommended the Town monitor the intersection for potential future operational issues that may justify signalization at that time.

Highway 16 at 42nd Street

During the a.m. peak hour, the southbound left-turn movement is expected to experience significant delays in the 2032 scenario, and then reach capacity before the 2042 (estimate 2037) scenario, and during the p.m. peak hour the northbound through movement is expected to experience significant delays and reach capacity in the 2032 scenario, both of which representing forced or breakdown flow. However, the existing two-way stop controlled intersection does not warrant upgrade to traffic signal control.

It is expected to unique configuration of this intersection is contributing to traffic signals not being warranted as per the TAC volume-based methodology, with the results of the operational analysis indicating an upgrade of the intersection control being a reasonable improvement option. Therefore, GHD has undertaken an analysis scenario assessing the operational implications of signalizing the intersection, as shown in Table 6.13.

The results of the analysis indicate a noticeable improvement with signalizing the intersection, resulting in no capacity concerns, low levels of delay, and acceptable queueing. It is therefore recommended signalization be considered as a recommended improvement at this intersection.

It is recommended future signalization of the intersection also include a re-design of the intersection, primarily with respect to the re-alignment of the north approach and elimination of the adjacent intersection of 2 Avenue at 42 Street, resulting in an improved concept from an operational and safety perspective. The discussed concept Option is illustrated in Exhibit 6.8.

Table 6.13 Highway 16 at 42 Street - Signalization Results

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	PM Peak Hour
Future 2042 Unsignalized	NBL: 0.28 (C) <1 veh NBT: 0.62 (E) <1 veh SBL: **1.45 (F)** 11m SBR: 0.34 (C) <1 veh	NBL: 0.30 (C) <1 veh NBT: **1.55 (F)** 27m SBL: 0.40 (C) <1 veh SBR: 0.40 (C) <1 veh
Future 2042 Signalized	WBTR: 0.53 (A) 35m NBL: 0.07 (B) <1 veh NBT: 0.39 (B) 18m SBL: 0.55 (B) 18m SBR: 0.32 (B) 14m	WBTR: 0.70 (B) 40m NBL: 0.08 (A) 1 veh NBT: 0.65 (B) 43m SBL: 0.29 (A) 10m SBR: 0.31 (A) 16m

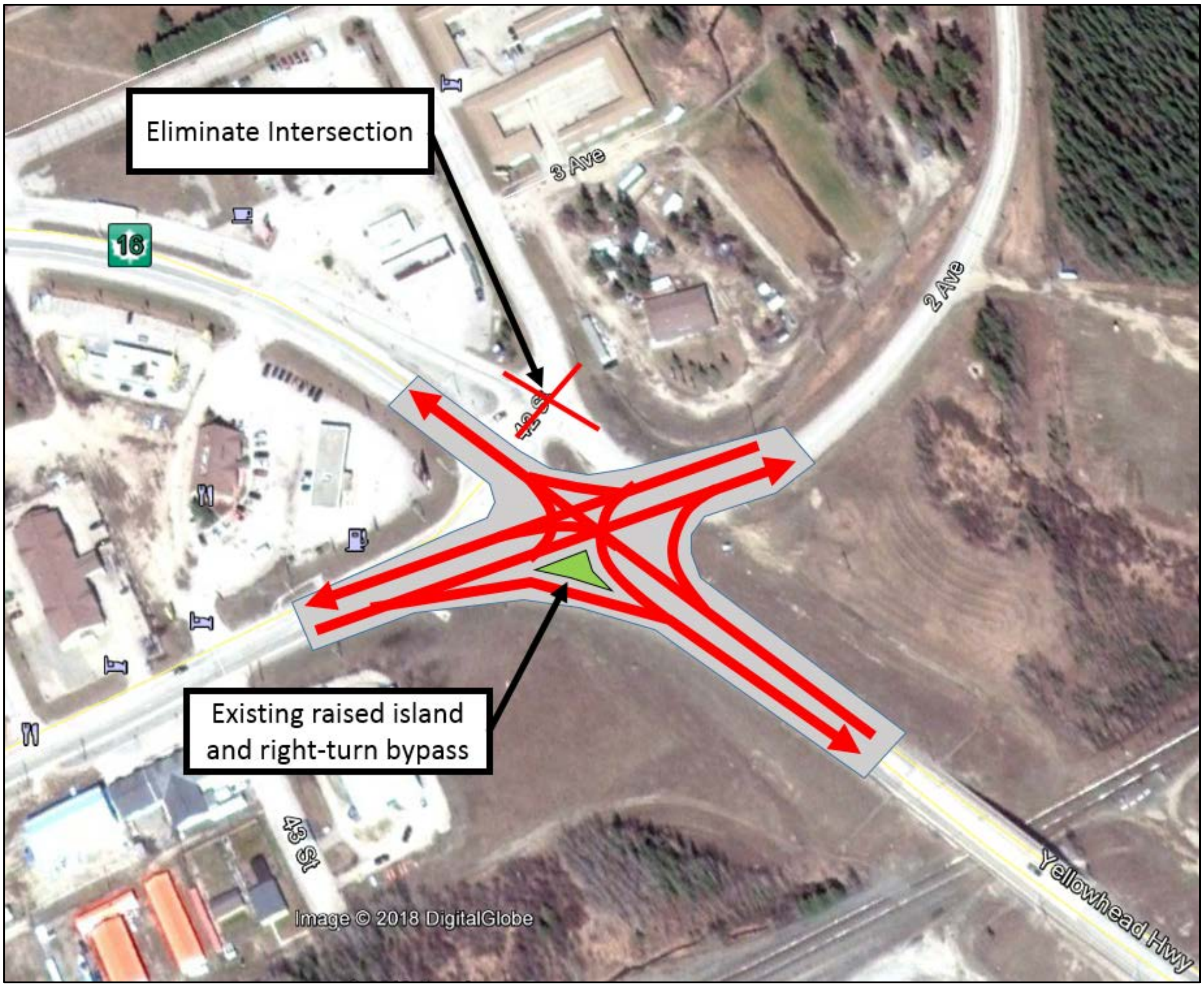


Exhibit 6.1: Proposed Intersection Concept

Highway 16 at 25 Street

The southbound approach is expected to experience significant delays and reach capacity in the 2032 scenario representing forced or breakdown flow, with the eastbound left-turn movement experiencing significant delays as well. The results of the TAC signal warrant indicate a traffic signal is warranted. Therefore, GHD has undertaken an analysis scenario assessing the operational implications of signalizing the intersection, as shown in Table 6.14.

The results of the analysis indicate a noticeable improvement with signalizing the intersection, resulting in no capacity concerns, low levels of delay, and acceptable queueing. It is therefore recommended signalization be considered as a recommended improvement at this intersection.

Table 6.14 Highway 16 at 25th Street - Signalization Results

Traffic Condition	Movement v/c (LOS) 95th percentile Queue	
	AM Peak Hour	**PM Peak Hour**
Future 2042 Unsignalized	EBL: 0.32 **(F)** <1 veh WBL: 0.27 (B) <1 veh NBLTR: 0.00 (A) <1 veh SBLTR: **3.05 (F)** 23m	EBL: 0.72 **(F)** <1 veh WBL: 0.27 (B) <1 veh NBLTR: 0.00 (B) <1 veh SBLTR: **2.81 (F)** 25m
Future 2042 Signalized	EBL: 0.66 (B) 39m EBT: 0.48 (A) 33m EBR: 0.01 (A) <1 veh WBL: 0.01 (A) <1 veh WBT: 0.52 (A) 39m WBR: 0.05 (A) <1 veh NBLTR: 0.03 (B) <1 veh SBLTR: 0.51 (B) 30m	EBL: 0.56 (A) 31m EBT: 0.62 (A) 52m EBR: 0.01 (A) <1 veh WBL: 0.01 (A) <1 veh WBT: 0.39 (A) 28m WBR: 0.07 (A) <1 veh NBLTR: 0.05 (B) <1 veh SBLTR: 0.44 (B) 27m

6.7.2 Recommended Corridor Improvements

Based on the results of the corridor capacity analysis, no corridor improvements to the existing study area road network are recommended.

6.8 Proposed Road Expansion and Improvements

The results of the corridor capacity analysis indicated that corridor improvements are not recommended to the existing study area road network based on the projected growth to the 2042 horizon year. However, to service future developable lands as identified in Figure 6.1, GHD has developed expansion recommendations for the existing road network to service these developable areas.

The revised Figure 6.1 illustrates the existing and proposed collector and arterial road network. Table 6.15 provides a summary of the proposed collector and/or arterial improvements to the existing road network, to be reviewed in conjunction with Figure 6.1.

Table 6.15 Summary of Network Improvements

Horizon Year	Description of Improvements	
2032	Town	• **Zone 1**: 12th Ave extended onto the west side of 63rd St. • **Zone 2**: Road added west of 63st, entering from Zone 1 and into Zone 16 • **Zone 3**: Aspen Dr. extended in the southwest direction into Zone 3 and connecting to 40th St. from the east. Also, new road branches off into Zone 20 • **Zone 4**: Road added east of Range Road 180 • **Zone 5**: Road added west of Edson Dr., entering into Zone 13 from the east • **Zone 6**: Road added west of 72nd St. • **Zone 7**: Road added south of Hwy 16

Table 6.15 Summary of Network Improvements

Horizon Year	Description of Improvements	
		• **Zone 8**: Road added east of Bear Lake Rd. into Zone 20 • **Zone 9**: Road added east of 63rd St., intersecting 54th St. • **Zone 12**: Aspen Dr. extended onto the east side of 25th St. • **Zone 15**: Road added to the east side of 25th St.
	Fringe	• Two roads extend from the western town limit towards the west • Road branches off into south side of Township Rd. 534 in the eastern town limit
2042	Town	• **Zone 16**: 66th St. is extended in the south direction into Zone 16; branching off into Zone 1 and into Zone 2 • **Zone 17**: Road added east of Edson Dr., entering into Zone 19 then into Zone 20 • **Zone 21**: Two separate roads branching off to the east side of Range Rd. 180
	Fringe	• The two roads that will be extended in the 2032 scenario from the western town limit will be further extended in 2042 • In the eastern town limit, an additional road will be attached to the collector road that is to be added in the 2032 scenario

Table 6.16 presents high-level cost estimates for the proposed road extensions. For the purpose of cost forecasting, it has been assumed that proposed roads within the “fringe” area will be constructed utilizing a rural cross-section, and proposed roads within the built-up area of the Town will be constructed utilizing an urban cross-section. This approach is expected to be conservative, as a review of the existing road network within the built-up area of the Town found instances of collector roads with a rural cross-section.

Furthermore, it is important to note the high-level nature of these cost estimates as they solely rely on an estimated per meter cost rate applied to the revised Figure 6.1, and do not consider other factors unique to the areas of development (i.e. required cut and/or fill of the existing landscape).

The cost estimate also includes the proposed signalization of the Highway 16 intersections at Regional Road 171/25 Street and at 42 Street/2 Avenue, as per the results of the intersection capacity analysis.

Table 6.16 Cost Estimate for Proposed Road Network

Horizon Year	Cross-Section	Proposed Linear Distance (estimate)	Cost Rate Estimate (Includes 30 percent Contingency + 10 percent Engineering & Administration)	Total Cost (estimate)
2018	Signalization of Hwy 16 at 42nd Street/2nd Ave		$400,000	$400,000
	2018 Total			**$400,000**
2032	2 Lane Urban Cross-Section	10.3 kilometers	$3,900 per meter	$40.2 million
	2 Lane Rural	10.3 kilometers	$2,100 per meter	$21.6 million

Table 6.16 Cost Estimate for Proposed Road Network

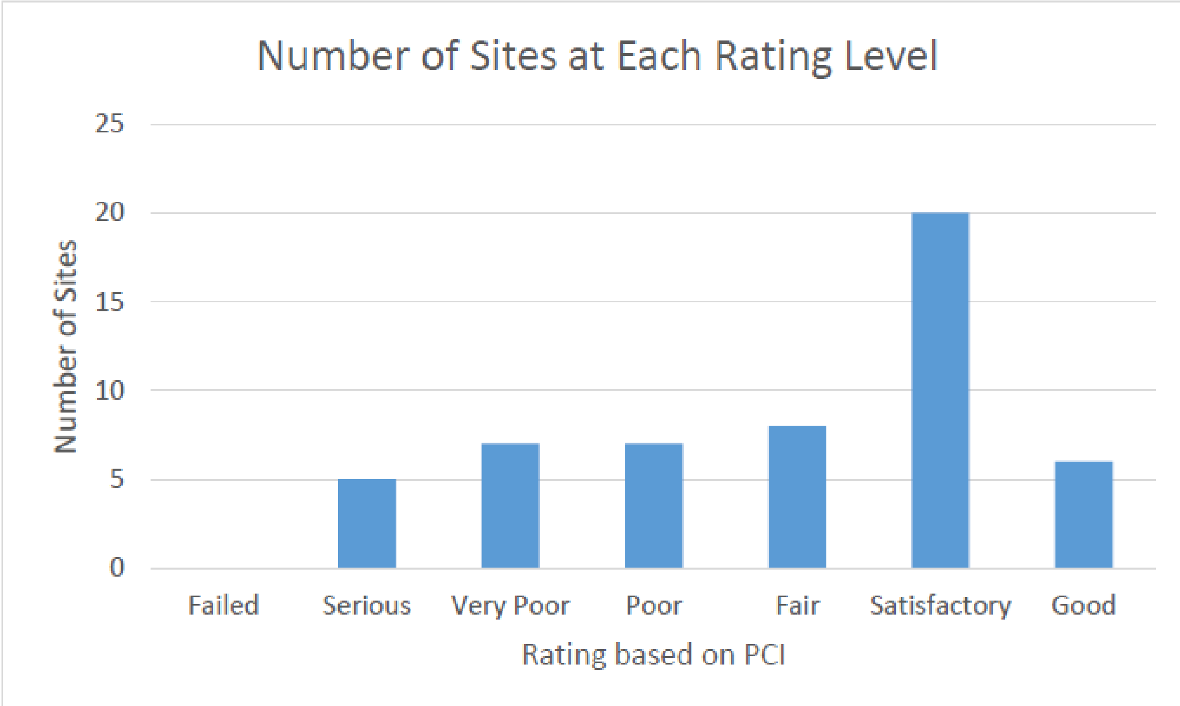
Horizon Year	Cross-Section	Proposed Linear Distance (estimate)	Cost Rate Estimate (Includes 30 percent Contingency + 10 percent Engineering & Administration)	Total Cost (estimate)
	Cross-Section			
	Signalization of Hwy 16 at 42nd Street/2nd Ave		$400,000	$400,000
2032 Total		**20.6 kilometers**		**$62.2 million**
2042	2 Lane Urban Cross-Section	5.6 kilometers	$3,900 per meter	$21.8 million
	2 Lane Rural Cross-Section	10.1 kilometers	$2,100 per meter	$21.2 million
2042 Total		**15.7 kilometers**		**$43 million**

6.9 Pavement Condition Index (PCI) Study

A PCI Study was conducted by Shelby Engineering in August 2017, based on a visual inspection of Streets within the Town of Edson, in support of this MSP update. The full PCI Study report is provided in the ancillary binder labeled Transportation and Roadway Network Reports.

6.9.1 Summary of PCI Study Findings

As shown in Graph 6.2 below, approximately half of all sites were classified as “Good” or “Satisfactory”, with approximately 23 percent as “Very Poor” or “Serious” and the remaining 27 percent as “Fair” or Poor”.



Graph 6.2: Summary of PCI Study Findings

6.9.2 Rehabilitation Strategy

Shelby Engineering provided a prioritization method, presented in Table 6.18**.** as a general means of prioritizing the rehabilitation of identified pavement issues based on road classification and PCI score.

Table 6.17 Prioritization Method

Time of Improvement	Freeway	Arterial	Collector	Local
Adequate	>85	>85	>80	>80
6-10 years	76-85	76-85	71-80	66-80
1-5 years	66-75	56-75	51-70	46-65
Rehabilitate NOW	60-65	50-55	45-50	40-45
Reconstruct NOW	<60	<50	<45	<40

Based on the results of the PCI Study, the prioritization method provided by Shelby Engineering, and consideration for the 2032 and 2042 horizon years, GHD has recommended a Rehabilitation Strategy as presented in the ancillary binder. The Strategy identifies the roads recommended for rehabilitation at each horizon year.

6.9.3 Cost Estimate of Streets Requiring Rehabilitation

The streets found in the Shelby Engineering PCI Report that were identified as being in Failed to Poor condition were deemed to be requiring reconstruction. The cost for these proposed immediate repairs for approximately 12.5 km of road is **$15.36 Million**. Additional road costs for future upgrades based on condition of the road is difficult to determine and is usually done on an annual basis and this rebuilding of these roads would normally be built into the annual maintenance costs for the streets. Additional items in these costs such as landscaping and concrete rehabilitation are not included. A street-by-street breakdown of these cost is included in Appendix F.

6.10 Complete Streets

"Complete Streets aims to increase the attractiveness, convenience and safety of all modes of transportation by creating a new selection of multi-modal streets that emphasize walking, cycling and transit, incorporate elements of green infrastructure and function in the context of surrounding land uses."[1] It involves rebalancing the needs of various road users competing for space within the right-of-way.

The City of Calgary has developed a comprehensive Complete Streets Guide for the development industry and associated stakeholders on how to incorporate Complete Streets concepts into the planning, design, and reconstruction of existing streets and construction of new streets.

The Town of Edson has a significantly lesser population than the City of Calgary and similarly lesser infrastructure needs and capabilities. However, the guidelines from Calgary's Complete Street Guide can be a useful tool to assist making infrastructure decisions within pedestrian and vehicular realms of right-of-way during new road construction and retrofit projects.

The majority of new road construction projects from the existing situation to the 2042 horizon year will occur along the periphery of the urbanized portion of the Town, and therefore will likely be more focused on automobile travel rather than pedestrian and bicycle modes. However, rehabilitation projects in the Town's downtown (i.e. sidewalk and boulevard rehabilitations) may provide opportunities to modify the pedestrian realms of the right-of-way to incorporate Complete Street concepts.

The following provides a brief overview of pertinent Complete Street concepts that can be incorporated into the planning, design, and reconstruction of existing streets and construction of new streets.

6.10.1 Network Design Guidelines

- Establish a block size between 150 to 175 meters in length. Where the block size is exceeded, retrofit large blocks with new streets, alleys, pedestrian, and/or bicycle connections. For existing street networks, do not allow Street closures that would result in larger blocks.
- Improve accessibility within a block by providing alleys, service courts, and other access ways.
- Require multiple street connections between adjacent neighborhoods. This is achieved by having lower order streets that extend beyond the local area (e.g., Primary Collector).
- Provide separate connections over or under Skeletal Roads and geographic barriers (rivers, bluffs, rail lines, etc.) so pedestrians and cyclists have links between neighborhoods without having to travel along intersection ramps and roadways that are not suited to those users.

[1] City of Calgary, "Calgary Transportation Plan." The City of Calgary, Calgary, AB, 2009

- Maintain network quality by accepting growth and expansion of the street network (including development, revitalization, intensifications, or redevelopment) while avoiding increasing the street width or number of travel lanes.
- Provide on-street curbside parking on most streets. Exceptions to this include very narrow streets, streets with bus lanes, high-speed roads or where there is a better use of the space.
- Design all streets below an Arterial classification to 50 km/h or less. These speeds promote safety for vulnerable users. For long straight streets, consider traffic controls, narrower lane widths, and boulevard features to reduce driver comfort at speeds over the posted limit.
- Maintain network function by discouraging:
  - one-way streets;
  - turn prohibitions;
  - full or partial closures (except on bike boulevards, or areas taken over for other public space use);
  - removal of on-street parking (except when replaced by wider sidewalks, an enhanced streetscape, bus lanes, bike lanes, etc. rather than additional vehicle lanes);
  - gated streets/communities;
  - widening of individual streets; and
  - conda version of city streets to limited access facilities.

6.10.2 Pedestrian Design Guidelines

- Separated sidewalks should be a minimum of 1.5 m wide (all classifications).
- Monolithic sidewalks should be a minimum of 2 m wide for improved pedestrian safety and to provide adequate width for snow storage (1.5 m permitted on residential and industrial streets).
- Sidewalks should be provided on both sides of all street classifications (including most residential and industrial areas) with the exception of Skeletal Roads.
- Sidewalks wider than 2 m should be provided along transit routes and connections to transit hubs.
- Sidewalks wider than 2 m should be provided for connections to schools, within activity centers, and near major pedestrian generators (e.g., stadiums).
- If monolithic, sidewalks should be wider than 2 m to provide separation from traffic when:
- Truck volumes are greater than 10 percent of total volume.
- Design speed is greater than 60 km/hr.
- Traffic volume is greater than 20,000 vehicles per day (note: does not apply to industrial streets).
- Sidewalk widths should be determined based on surrounding land uses (higher density requires wider sidewalks).
- Ideally, two directional wheelchair ramps should be installed at all street intersection corners (if corner radii and catch basin locations permit) and as a minimum, at all Arterial, Livable, Primary

Collector, Collector, and Activity Centre Streets should have two wheelchair ramps at each corner.

6.10.3 Bikeway Design Guidelines

- The type of bicycle facility should be determined based on:
  - bicycle network connectivity (as specified in the City of Calgary Pathway and Bikeway Implementation Plan);
  - current and future demand for a route;
  - cycling policies (e.g., Bicycle Policy TP-011);
  - design/posted motor vehicle speed;
  - surrounding land uses;
  - driveway frequency;
  - level of transit service (e.g., frequent BRT vs. infrequent bus); and
  - daily traffic volume and composition.
- Collector streets carrying more than 3,000 vehicles per day shall include dedicated bike lanes.
- Minimum bike lane width is 1.5 m free of obstructions and obstacles (1.2 m may be permitted in retrofit projects where there are constraints).
- Wider on-street facilities (e.g., 1.5 m minimum bike lane + 0.8 m minimum buffer) shall be provided adjacent to a parking lane (door Zone buffer), next to vertical barriers and on a grade (as cyclists may not travel in a straight line while travelling uphill).
- A buffered (e.g., 1.0 m minimum painted or textured buffer) or physically separated (e.g., by a curb or parked vehicles) exclusive facility should be provided when any of the following criteria are met:
  - truck volumes are >10 percent of total volume
  - design speed is >60 km/hr
  - two-way traffic volumes exceed 20,000 vehicles per day
  - the speed differential between cyclists and motor vehicles is too great (e.g., when traveling uphill)
- Minimum width for regional pathways is 3.0 m (uplands) and 4.0 m (river and creek valleys).

6.10.4 New Road Design Standards

GHD has undertaken a review of the Town's current roadway cross-Section drawings for various road classifications, as provided in the ancillary binder. Given the expected future population growth and results of the future conditions capacity analysis, the existing cross-sections utilized by the Town are adequate to accommodate the expected required cross-Section elements (i.e. sidewalks, travel lanes, curb-and-gutter, etc.) in support of new development areas as shown in Figure 6.1.

As previously noted, rehabilitation projects in the Town's downtown (i.e. sidewalk and boulevard rehabilitations) may provide opportunities to modify the pedestrian realms of the right-of-way to incorporate Complete Street concepts. GHD therefore proposed the cross-Section illustration presented in Figure G-3 in Appendix G, applicable to the Town's "downtown" north-south urban roads between 2nd Avenue and 4th Avenue. The cross-Section includes the following elements:

- Curb bump-outs resulting in a decreased crossing distance;
- Tactile walking plates at sidewalk let-downs;
- Upgrading the existing "zebra" style crosswalk to a "ladder" style crosswalk by adding parallel lines for the length of the crosswalk (typically 3 meters wide);
- Dedicated on-street bike lanes (typically 1.5 meters wide);
- Buffer Zone separating bike lanes with on-street parking (typically 0.5-1.0 meters wide; 1.0 meter wide recommended where possible).

The curb bump-outs, tactile walking plates, and "ladder" style crosswalk in combination enhance pedestrian safety by improving detection of the crosswalk by all road users and by reducing the crossing distance required.

Furthermore, the introduction of the dedicated on-street bike lanes and buffer Zone not only enhance safety for cyclists, but are expected to improve overall safety and traffic operations for all road users by resulting in a reduced lane width for the general purpose lanes. Most roads within the "downtown" area of Edson (generally the urban network between 2nd Avenue and 4th Avenue) have noticeably wide pavement widths, and generally are expected to be able to accommodate on-street bike lanes and buffers while maintaining general purpose travel lanes of 3.2 to 3.5 meters in width.

GHD has provided a "downtown" cross-Section in the ancillary binder incorporating these Complete Streets elements.

Due to the presence of existing trees and light standards within the roadside boulevard, widening of the sidewalks thus resulting in a widened boulevard, is not considered feasible and is not recommended. Relocating all boulevard trees and light standards would be required to increase the clear Zone width for pedestrians.

Exhibit 6.2: Proposed "Downtown" Cross-Section

Appendices

Appendix A
Water System - Cost Estimate

Table A.1
Existing Pipe Upgrades Due to Poor Condition Assessment

Municipal Servicing Plan Update
Town of Edson

Pipe Size (mm)	Location	Pipe Type	Year Consutructed	Length (m)	Unit Price ($/m)	Capital Cost
100	2nd Ave b/w 46th & 47th Str	C.I.	1963	176	$ 650	$ 114,400
200	50th & 3rd Ave	C.I.	1971	29	$ 675	$ 19,575
150	6930-4th Ave	C.I.	1972	183	$ 650	$ 118,950
150	68th-69th & 4th Ave service road	C.I.	1972	186	$ 650	$ 120,900
150	7228 - 4 Ave (Service Rd.)	C.I.	1972	160	$ 650	$ 104,000
150	Vacant lot beside 7025 4A Ave	C.I.	1972	118	$ 650	$ 76,700
150	6615-5th Ave	C.I.	1972	141	$ 650	$ 91,650
150	42nd & 7th Ave	C.I.	1959	192	$ 650	$ 124,800
150	4214-7th	C.I.	1959	168	$ 650	$ 109,200
100	4736-7th Ave	C.I.	1931	99	$ 650	$ 64,350
150	4310 - 7th Ave	C.I.	1959	147	$ 650	$ 95,550
100	5004-8th Ave	C.I.	1931	147	$ 650	$ 95,550
200	4416 8th Ave	C.I.	1964	90	$ 675	$ 60,750
100	45 Str and 8B Ave	C.I.	1964	62	$ 650	$ 40,300
100	43A & 9th Ave	C.I.	1965	177	$ 650	$ 115,050
100	50th & 9th Ave	C.I.	1931	197	$ 650	$ 128,050
100	50th & 9th Ave	C.I.		148	$ 650	$ 96,200
150	4326-9th Ave	C.I.	1965	52	$ 650	$ 33,800
150	5911 - 9 Ave	A.C.	1977	48	$ 650	$ 31,200
100	51st & 10th Ave	C.I.	1951	84	$ 650	$ 54,600
100	4829-10th Ave	C.I.		152	$ 650	$ 98,800
100	48th & 10th Ave	C.I.		136	$ 650	$ 88,400
100	49th & 10th Ave	C.I.		20	$ 650	$ 13,000
100	NE of 49 Str & 10 Ave	C.I.		20	$ 650	$ 13,000
150	NE hydrant 49th Str & 12 Ave	A.C.	1977	5	$ 650	$ 3,250
200	15 Ave and Edson Dr.	A.C.	1978	14	$ 675	$ 9,450
100	4819 - 18 Ave	C.I.	1957	152	$ 650	$ 98,800
150	71st & South Glen Ave	C.I.	1972	100	$ 650	$ 65,000
150	72nd & Glenwood Dr.	C.I.	1972	140	$ 650	$ 91,000
100	43A & 4th Ave	C.I.	1955	70	$ 650	$ 45,500
100	43A & 2 - 3 Ave	C.I.		73	$ 650	$ 47,450
100	43A & 3rd Ave	C.I.		70	$ 650	$ 45,500
100	714 & 44th Str	C.I.	1964	111	$ 650	$ 72,150
150	827 - 45th Str	C.I.	1964	44	$ 650	$ 28,600
150	46 Str and 6 -7 Ave	C.I.	1972	112	$ 650	$ 72,800

Table A.1
Existing Pipe Upgrades Due to Poor Condition Assessment

Municipal Servicing Plan Update
Town of Edson

200	616-50th Str	C.I.	1931	91	$ 675	$ 61,425
200	50 Str & 9 - 10 Ave	C.I.		98	$ 675	$ 66,150
100	52 Str & 4 Ave (North intersection + N-Side Alley)	C.I.	1931	97	$ 650	$ 63,050
150	52 St and 7 Ave	A.C.	1974	70	$ 650	$ 45,500
100	53rd & 4th Ave	C.I.	1959	92	$ 650	$ 59,800
150	433-70th Str	C.I.	1972	55	$ 650	$ 35,750
150	421 - 70 St	C.I.		60	$ 650	$ 39,000
150	South Glen Ave & 71st	C.I.	1972	75	$ 650	$ 48,750
150	South Glen-Park Glen & 72nd Str	C.I.	1972	106	$ 650	$ 68,900
150	72nd & South Glen Ave	C.I.		110	$ 650	$ 71,500
			Total Length (m)	4677		
			Sub-Total			$ 3,048,100
			Contingency (30%)			$ 914,430.0
	Engineering & Administration (10%)					$ 304,810.0
			Capital Cost			**$ 4,267,340**

Table A.2
Water System Upgrades (Existing Improvements)
Based on Maximum Daily Demand (MDD) Fire Flow Requirements

Municipal Servicing Plan Update
Town of Edson

Water System at 13 Ave / Edson Dr; 63 Street and 17 Avenue Area					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
1.1	350 mm PVC Watermain	m	650	$800	$520,000
1.2	200 mm PVC Watermain	m	1,090	$700	$763,000
1.3	Connection of Existing Services	each	30	$1,500	$45,000
1.4	Install New Booster Station	lump sum	1	$900,000	$900,000
				Sub-Total Cost	$2,228,000
				Contingency (30%)	$668,400
				Engineering and Administration (10%)	$222,800
				Capital Cost	**$3,119,200**
Water System at 4 Avenue and 70 Street Area					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
2.1	Upgrade Existing 150 mm pipe with 300 mm PVC Watermain & Rehabilitate Road to Existing Conditions (along 4 Avenue)	m	520	$800	$416,000
2.2	300 mm Isolation Valves	each	2	$4,000	$8,000
				Sub-Total Cost	$424,000
				Contingency (30%)	$127,200
				Engineering and Administration (10%)	$42,400
				Capital Cost	**$593,600**
Water System at 8 Avenue and 49 Street Area					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
3.1	Upgrade Existing 100 mm pipe with 150 mm PVC Watermain & Rehabilitate Road to Existing Conditions	m	755	$750	$566,250
3.2	150 mm Isolation Valves	each	2	$3,500	$7,000
				Sub-Total Cost	$573,250
				Contingency (30%)	$171,975
				Engineering and Administration (10%)	$57,325
				Capital Cost	**$802,550**
Water System at 1 Avenue (from 42 Street to 45 Street) Area					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
4.1	Upgrade Existing 200 mm a.c. pipe with 300 mm PVC Watermain & Rehabilitate Road to Existing Conditions	m	530	$800	$424,000
4.2	300 mm Isolation Valves	each	3	$4,000	$12,000
				Sub-Total Cost	$436,000
				Contingency (30%)	$130,800
				Engineering and Administration (10%)	$43,600
				Capital Cost	**$610,400**
Water System at 1 Avenue (from 27 Street to 42 Street) Area					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
5.1	Install new 300 mm PVC Watermain & Rehabilitate Road to Existing Conditions	m	1,205	$750	$903,750
5.2	200 mm Isolation Valves	each	2	$3,750	$7,500
5.3	300 mm Isolation Valves	each	4	$4,000	$16,000
				Sub-Total Cost	$927,250
				Contingency (30%)	$278,175
				Engineering and Administration (10%)	$92,725

Table A.2
Water System Upgrades (Existing Improvements)
Based on Maximum Daily Demand (MDD) Fire Flow Requirements

Municipal Servicing Plan Update
Town of Edson

				Capital Cost	**$1,298,150**
Water System at 9 Avenue and 63 Street to 73 Street and HWY 16					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
6.1	Install new 350 mm PVC Watermain	m	3,500	$800	$2,800,000
6.2	300 mm Isolation Valves	each	1	$4,000	$4,000
6.3	350 mm Isolation Valves	each	7	$4,250	$29,750
				Sub-Total Cost	$2,833,750
				Contingency (30%)	$850,125
				Engineering and Administration (10%)	$283,375
				Capital Cost	**$3,967,250**
Demolish Existing Booster Station					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
7.1	Demolish the Existing Booster Station	lump sum	1	$50,000	$50,000
				Sub-Total Cost	$50,000
				Contingency (30%)	$15,000
				Engineering and Administration (10%)	$5,000
				Capital Cost	**$70,000**
New 250 mm Northwest Loop					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
8.1	250 mm PVC New Loop Watermain	m	1,800	$730	$1,314,000
8.2	250 mm Isolation Valves	each	5	$3,900	$19,500
				Sub-Total Cost	$1,333,500
				Contingency (30%)	$400,050
				Engineering and Administration (10%)	$133,350
				Capital Cost	**$1,866,900**
Water System Existing Improvements Summary					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
1.0	13 Ave / Edson Dr; 63 Street and 17 Avenue Area				$2,228,000
2.0	4 Avenue and 70 Street Area				$424,000
3.0	8 Avenue and 49 Street Area				$573,250
4.0	1 Avenue (from 42 Street to 45 Street) Area				$436,000
5.0	1 Avenue (from 27 Street to 42 Street) Area				$927,250
6.0	9 Avenue and 63 st to 73 st / HWY 16				$2,833,750
7.0	Demolish Existing Booster Station				$50,000
8.0	New 250 mm Northwest Loop				$1,333,500
				Sub-Total Cost	$8,805,750
				Contingency (30%)	$2,641,725
				Engineering and Administration (10%)	$880,575
				Capital Cost	**$12,328,050**

Table A.3
Water System Upgrades - 2032 Town Only

Municipal Servicing Plan Update
Town of Edson

Water System at Degas Site					
Item	Description	Unit	Quantity	Unit Price	Amount
1.1	New Reservoir Storage - 1250 m^3	m^3	1,250	$500	$625,000
1.2	Install New Booster Station	lump sum	1	$800,000	$800,000
				Sub-Total Cost	$1,425,000
				Contingency (30%)	$427,500
				Engineering and Administration (10%)	$142,500
				Capital Cost	**$1,995,000**

Water System at Microwave Tower (MT) Site					
Item	Description	Unit	Quantity	Unit Price	Amount
2.1	New Reservoir Storage - 1300 m^3	m^3	1,300	$500	$650,000
2.2	PRV	each	1	$7,500	$7,500
2.3	New Pump at the GPT Station	each	1	$200,000	$200,000
				Sub-Total Cost	$857,500
				Contingency (30%)	$257,250
				Engineering and Administration (10%)	$85,750
				Capital Cost	**$1,200,500**

Water System From GPT to MT Site					
Item	Description	Unit	Quantity	Unit Price	Amount
3.1	Install 350mm New Main	m	730	$800	$584,000
3.2	350 mm Isolation Valves	each	2	$4,250	$8,500
				Sub-Total Cost	$592,500
				Contingency (30%)	$177,750
				Engineering and Administration (10%)	$59,250
				Capital Cost	**$829,500**

Water System Along the North of 18 Avenue					
Item	Description	Unit	Quantity	Unit Price	Amount
4.1	Install 350mm New Main	m	1,620	$800	$1,296,000
4.2	350 mm Isolation Valves	each	4	$4,250	$17,000
				Sub-Total Cost	$1,313,000
				Contingency (30%)	$393,900
				Engineering and Administration (10%)	$131,300
				Capital Cost	**$1,838,200**

Water System Along 40 Street					
Item	Description	Unit	Quantity	Unit Price	Amount
5.1	Install 350mm New Main	m	350	$800	$280,000
5.2	300 mm Isolation Valves	each	2	$4,000	$8,000
5.3	350 mm Isolation Valves	each	2	$4,250	$8,500
				Sub-Total Cost	$296,500
				Contingency (30%)	$88,950
				Engineering and Administration (10%)	$29,650
				Capital Cost	**$415,100**

Table A.3
Water System Upgrades - 2032 Town Only

Municipal Servicing Plan Update
Town of Edson

Water System in East Industrial Area					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
6.1	Install 350mm New Main	m	1,440	$800	$1,152,000
6.2	300 mm Isolation Valves	each	3	$4,000	$12,000
6.3	350 mm Isolation Valves	each	4	$4,250	$17,000
				Sub-Total Cost	$1,181,000
				Contingency (30%)	$354,300
				Engineering and Administration (10%)	$118,100
				Capital Cost	**$1,653,400**

Water System on 10 Ave from 48 Street to 51 Street					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
7.1	Upgrade the existing water main from 100mm to 150mm	m	510	$750	$382,500
7.2	100 mm Isolation Valves	each	2	$3,250	$6,500
7.3	150 mm Isolation Valves	each	2	$3,500	$7,000
				Sub-Total Cost	$396,000
				Contingency (30%)	$118,800
				Engineering and Administration (10%)	$39,600
				Capital Cost	**$554,400**

Water System on 9 Ave from 49 Street to 51 Street					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
8.1	Upgrade the existing water main from 100mm to 150mm	m	350	$750	$262,500
8.2	100 mm Isolation Valves	each	2	$3,250	$6,500
8.3	150 mm Isolation Valves	each	1	$3,500	$3,500
				Sub-Total Cost	$272,500
				Contingency (30%)	$81,750
				Engineering and Administration (10%)	$27,250
				Capital Cost	**$381,500**

Water System on 8 Avenue from 50 Street to 51 Street					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
9.1	Upgrade the existing water main from 100mm to 150mm	m	165	$750	$123,750
9.2	100 mm Isolation Valves	each	2	$3,250	$6,500
9.3	150 mm Isolation Valves	each	1	$3,500	$3,500
				Sub-Total Cost	$133,750
				Contingency (30%)	$40,125
				Engineering and Administration (10%)	$13,375
				Capital Cost	**$187,250**

Water System - New Wells and Services					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
10.1	New Water Services	each	50	$1,500	$75,000
10.2	Connect Well #3 to the System	each	1	$385,000	$385,000
10.2	New Wells with New Well Pumps	each	10	$60,000	$600,000

Table A.3
Water System Upgrades - 2032 Town Only

Municipal Servicing Plan Update
Town of Edson

			Sub-Total Cost		$1,060,000
			Contingency (30%)		$318,000
		Engineering and Administration (10%)			$106,000
			Capital Cost		**$1,484,000**
Water System 2032 Town Only Summary					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
1.0	Water System at Degas Site				$1,425,000
2.0	Water System at Microwave Tower (MT) Site				$857,500
3.0	Water System From GPT to MT Site				$592,500
4.0	Along the North of 18 Ave				$1,313,000
5.0	Along 40 st				$296,500
6.0	In East Industrial Area				$1,181,000
7.0	10 Ave from 48 st to 51 st				$396,000
8.0	9 Ave from 49 st to 51 st				$272,500
9.0	8 Ave from 50 st to 51 st				$133,750
10.0	New Wells and Services				$1,060,000
			Sub-Total Cost		$7,527,750
			Contingency (30%)		$2,258,325
		Engineering and Administration (10%)			$752,775
			Capital Cost		**$10,538,850**

Table A.4
Water System Upgrades - 2042 Town Only

Municipal Servicing Plan Update
Town of Edson

Water System at Dagas Site					
Item	Description	Unit	Quantity	Unit Price	Amount
1.1	Upgrade Existing 150 mm pipe with 250 mm PVC Watermain & Rehabilitate Road to Existing Conditions (along 6 Avenue)	m	350	$780	$273,000
1.2	Upgrade Existing 100 mm pipe with 150 mm PVC Watermain & Rehabilitate Road to Existing Conditions (along 18 Avenue and 7 Avenue)	m	510	$750	$382,500
1.3	Upgrade Existing 100 mm pipe with 250 mm PVC Watermain & Rehabilitate Road to Existing Conditions (along 49 Street)	m	230	$780	$179,400
1.4	New Water Services	each	72	$1,500	$108,000
1.5	Install 150 mm Isolation Valves	each	9	$3,500	$31,500
1.6	Install 200 mm Isolation Valves	each	1	$3,600	$3,600
1.7	Install 250 mm Isolation Valves	each	6	$3,750	$22,500
1.8	Install New Well and Well Pump	each	2	$60,000	$120,000
				Sub-Total Cost	$1,120,500
				Contingency (30%)	$336,150
				Engineering and Administration (10%)	$112,050
				Capital Cost	**$1,568,700**

Table A.5
Water System Upgrades - 2032 Town and Fringe

Municipal Servicing Plan Update
Town of Edson

Item	Description	Unit	Quantity	Unit Price	Amount
Water System at Degas Site					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
1.1	New Reservoir Storage - 4320 m3	m^3	4,320	$500	$2,160,000
1.2	Pump Upgrades	lump sum	1	$400,000	$400,000
1.3	Two Wells	each	2	$60,000	$120,000
				Sub-Total Cost	$2,680,000
				Contingency (30%)	$804,000
				Engineering and Administration (10%)	$268,000
				Capital Cost	**$3,752,000**
Water System at East Fringe					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
2.1	Install 350mm New Main	m	730	$4,590	$3,350,700
2.2	PRVs	each	2	$7,500	$15,000
2.3	350 mm Isolation Valves	each	9	$4,250	$38,250
				Sub-Total Cost	$3,403,950
				Contingency (30%)	$1,021,185
				Engineering and Administration (10%)	$340,395
				Capital Cost	**$4,765,530**
Water System New Loops of Southwest Fringe					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
3.1	Install 350mm New Main	m	3,270	$4,590	$15,009,300
3.2	350 mm Isolation Valves	each	6	$4,250	$25,500
3.3	PRVs	each	9	$7,500	$67,500
				Sub-Total Cost	$15,102,300
				Contingency (30%)	$4,530,690
				Engineering and Administration (10%)	$1,510,230
				Capital Cost	**$21,143,220**
Water System 2032 Town and Fringe Summary					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
1.0	Water System at Degas Site				$2,680,000
2.0	Water System at Microwave Tower (MT) Site				$3,403,950
3.0	Water System From GPT to MT Site				$15,102,300
				Sub-Total Cost	$21,186,250
				Contingency (30%)	$6,355,875
				Engineering and Administration (10%)	$2,118,625
				Total 2032 Capital Cost	**$29,660,750**

Table A.6

Water System Upgrades - 2042 Town and Fringe

Municipal Servicing Plan Update

Town of Edson

Water System 2042 Town and Fringe Summary					
Item	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**
1.1	Pump Upgrades	lump sum	1	$400,000	$400,000
1.2	Storage Reservoir	m3	530	$500	$265,000
1.3	New Water Wells	each	3	$60,000	$180,000
				Sub-Total Cost	$845,000
				Contingency (30%)	$253,500
				Engineering and Administration (10%)	$84,500
				Capital Cost	**$1,183,000**

Appendix B
Water System - Hydrant Test Results

Fire Protection

Hydrant Flow Test Report

Customer: GHD Limited
Building: 44 Street & 18 Avenue
Contact: Duncan Zhang
Invoice #: EDM0037253
Witness: Wayne Donald

Inspection Date: December 4, 2017
Inspector: S. Hand
Phone/Fax: 780-469-1454
Time: 08:00

General

State purpose of test:
Consumption rate during test:
If pumps affect test, indicate pumps operating:

Flow Hydrants

☒ A1 ☐ A2 ☐ A3 ☐ A4

Size of nozzle:	2.5 INCH				
Pitot reading:	105 PSI				
Discharge Coefficient:	0.7				
GPM:	1337				
Total GPM:					1193

Static/Residual Hydrants

Static B: 61 PSI

Residual B 52 PSI

@ 20 psi residual GPM; or

@ 52 psi residual 1337 GPM

Remarks

Location Map: Show line sizes and distance to next cross connected line. Show valves and hydrant branch size. Indicate north. Show flowing hydrants – label A1,A2,A3,A4. Show location of static and residual – label B.

Hydrant Flow Test Report

Customer: GHD Limited
Building: 48 Street & 3 Avenue
Contact: Duncan Zhang
Invoice #: EDM0037253
Witness: Wayne Donald

Inspection Date: December 4, 2017
Inspector: S. Hand
Phone/Fax: 780-469-1454
Time: 08:00

General

State purpose of test:
Consumption rate during test:
If pumps affect test, indicate pumps operating:

Flow Hydrants

☒ A1 ☐ A2 ☐ A3 ☐ A4

Size of nozzle:	2.5 INCH				
Pitot reading:	100 PSI				
Discharge Coefficient:	0.7				
GPM:	1305				
Total GPM:					1193

Static/Residual Hydrants

Static B: 75 PSI	@ 20 psi residual GPM; or
Residual B 70 PSI	@ 70 psi residual 1305 GPM

Remarks

Location Map: Show line sizes and distance to next cross connected line. Show valves and hydrant branch size. Indicate north. Show flowing hydrants – label A1,A2,A3,A4. Show location of static and residual – label B.

Hydrant Flow Test Report

Customer: GHD Limited
Building: 70 Street & 4 Avenue
Contact: Duncan Zhang
Invoice #: EDM0037253
Witness: Wayne Donald

Inspection Date: December 4, 2017
Inspector: S. Hand
Phone/Fax: 780-469-1454
Time: 08:00

General

State purpose of test:
Consumption rate during test:
If pumps affect test, indicate pumps operating:

Flow Hydrants

☒ A1 ☐ A2 ☐ A3 ☐ A4

Size of nozzle:	2.5 INCH				
Pitot reading:	50 PSI				
Discharge Coefficient:	0.7				
GPM:	1193				
Total GPM:					1193

Static/Residual Hydrants

Static B: 56 PSI	@ 20 psi residual GPM; or
Residual B 46 PSI	@ 46 psi residual 1193 GPM

Remarks

Location Map: Show line sizes and distance to next cross connected line. Show valves and hydrant branch size. Indicate north. Show flowing hydrants – label A1,A2,A3,A4. Show location of static and residual – label B.

Hydrant Flow Test Report

Customer: GHD Limited
Building: Edson Drive & 49 Street
Contact: Duncan Zhang
Invoice #: EDM0037253
Witness: Wayne Donald

Inspection Date: December 4, 2017
Inspector: S. Hand
Phone/Fax: 780-469-1454
Time: 08:00

General

State purpose of test:
Consumption rate during test:
If pumps affect test, indicate pumps operating:

Flow Hydrants

☒ A1 ☐ A2 ☐ A3 ☐ A4

Size of nozzle:	2.5 INCH				
Pitot reading:	50PSI				
Discharge Coefficient:	0.7				
GPM:	1193				
Total GPM:					1193

Static/Residual Hydrants

Static B: 48 PSI	@ 20 psi residual GPM; or
Residual B 46 PSI	@ 49 psi residual 1193 GPM

Remarks

HYDRANT LOCATED ACROSS STREET FROM HOUSE # 1638 EDSON DRIVE

Location Map: Show line sizes and distance to next cross connected line. Show valves and hydrant branch size. Indicate north. Show flowing hydrants – label A1,A2,A3,A4. Show location of static and residual – label B.

Hydrant Flow Test Report

Customer: GHD Limited
Building: Warehouse Scale Shack
Contact: Duncan Zhang
Invoice #: EDM0037253
Witness: Wayne Donald

Inspection Date: December 4, 2017
Inspector: S. Hand
Phone/Fax: 780-469-1454
Time: 08:00

General

State purpose of test:
Consumption rate during test:
If pumps affect test, indicate pumps operating:

Flow Hydrants

☒A1 ☐A2 ☐ A3 ☐A4

Size of nozzle:	2.5 INCH				
Pitot reading:	150 PSI				
Discharge Coefficient:	0.7				
GPM:	1598				
Total GPM:					1193

Static/Residual Hydrants

Static B: 76 PSI

Residual B 70 PSI

@ 20 psi residual GPM; or

@ 70 psi residual 1598 GPM

Remarks

Location Map: Show line sizes and distance to next cross connected line. Show valves and hydrant branch size. Indicate north. Show flowing hydrants – label A1,A2,A3,A4. Show location of static and residual – label B.

Appendix C
Wastewater Systems Cost Estimate & Condition Assessment

Wastewater Systems – Cost Estimates

Table C.1
Cost Estimate for Existing (2017) Wastewater Upgrades System (Based on Condition Assessment)

Municipal Servicing Plan Update
Town of Edson

No.	Line Location	From MH	To MH	Diameter (mm)	Pipe Material	Slope (%)	Length (m)	Year	Unit Price ($/m)	Replacement Cost
1	63 Street – 4 Avenue to South of Highway 16	S55	S75	200	VCT	0.016	77.92	1972	$ 550.00	$42,856.00
2	55 Street - 2 Avenue to 1 Avenue	S100	S91	250	VCT	0.003	114.02	1976	$ 575.00	$65,561.50
3	52 Street – 11 Avenue to 10 Avenue	S355	S337	200	VCT	0.024	95.40	1973	$ 550.00	$52,470.00
4	51 Street - 10th Avenue to 11th Avenue	S356	S357	200	VCT	0.027	108.97	1953	$ 550.00	$59,933.50
5	6 Avenue - 52 Street to 51 Street	S363	S364	200	VCT/PVC	0.014	85.27	2016/1951	$ 550.00	$46,898.50
6	6 Avenue - 52 Street to 51 Street	S364	S365	200	VCT	0.035	85.52	1951	$ 550.00	$47,036.00
7	7 Avenue – 52 Street to 51 Street	S366	S367	200	VCT	0.020	85.66	1951	$ 550.00	$47,113.00
8	7 Avenue – 52 Street to 51 Street	S367	S368	200	VCT	0.038	85.53	1951	$ 550.00	$47,041.50
9	8 Avenue – 52 Street to 51 Street	S369	S370	200	VCT	-	94.49	1951	$ 550.00	$51,969.50
10	8 Avenue – 52 Street to 51 Street	S370	S371	200	VCT	-	89.61	1951	$ 550.00	$49,285.50
11	North of 6 Avenue South Boundary of Sunset Mobile Estates to 6 Avenue	S377	S378	200	PVC	0.009	48.36	1996	$ 550.00	$26,598.00
12	North of 6 Avenue South Boundary of Sunset Mobile Estates to 6 Avenue	S378	S380	375	CONC	0.008	71.64	1973	$ 720.00	$51,580.80
13	North of 6 Avenue South Boundary of Sunset Mobile Estates to 6 Avenue	S380	S358	375	CONC	0.008	76.96	1973	$ 720.00	$55,411.20
14	53 Street - 8th Avenue to 6th Avenue	S381	S382	200	VCT	0.005	105.15	1974	$ 550.00	$57,832.50
15	53 Street - 8th Avenue to 6th Avenue	S382	S361	200	VCT	0.005	117.96	1974	$ 550.00	$64,878.00
16	52 Street - 10th Avenue to 8th Avenue	S355	S383	200	VCT	0.011	60.27	1974	$ 550.00	$33,148.50
17	52 Street - 10th Avenue to 8th Avenue	S383	S372	200	VCT	0.025	69.79	1974	$ 550.00	$38,384.50
18	North of 52 Street - 10th Avenue to 8th Avenue	S372	S384	200	VCT	0.025	65.53	1974	$ 550.00	$36,041.50
19	South of 52 Street - 10th Avenue to 8th Avenue	S384	S385	200	VCT	0.006	57.23	1951	$ 550.00	$31,476.50
20	52 Street - 10th Avenue to 8th Avenue	S385	S369	200	VCT	0.005	15.24	1951	$ 550.00	$8,382.00
21	52 Street - 8th Avenue to 6th Avenue	S369	S386	200	VCT	0.005	27.43	1951	$ 550.00	$15,086.50
22	52 Street - 8th Avenue to 6th Avenue	S366	S363	250	VCT/PVC	0.006	110.96	1951/2016	$ 575.00	$63,802.00
23	51 Street - 10th Avenue to 7th Avenue	S357	S387	200	VCT	0.004	110.94	1953	$ 550.00	$61,017.00
24	51 Street - 10th Avenue to 7th Avenue	S387	S371	200	VCT	0.005	110.93	1951	$ 550.00	$61,011.50
25	51 Street - 10th Avenue to 7th Avenue	S371	S368	200	VCT	0.005	110.95	1951	$ 550.00	$61,022.50
26	6 Avenue – 51 Street to 50 Street	S365	S388	200	VCT	0.035	109.13	1951	$ 550.00	$60,021.50
27	6 Avenue – 51 Street to 50 Street	S388	S389	200	VCT	-	15.24	1951	$ 550.00	$8,382.00
28	6 Avenue – 50 Street to 48 Street	S390	S391	200	VCT	0.021	87.77	1953	$ 550.00	$48,273.50
29	6 Avenue – 50 Street to 48 Street	S391	S392	200	VCT	0.004	85.66	1951	$ 550.00	$47,113.00
30	6 Avenue – 50 Street to 48 Street	S392	S393	200	VCT	0.026	85.66	1951	$ 550.00	$47,113.00
31	6 Avenue – 50 Street to 48 Street	S393	S394	200	VCT	0.026	85.63	1951	$ 550.00	$47,096.50
32	7 Avenue – 51 Street to 50 Street	S368	S395	200	VCT	0.033	109.13	1951	$ 550.00	$60,021.50
33	7 Avenue – 50 Street to 48 Street	S396	S397	200	VCT	0.019	87.83	1951	$ 550.00	$48,306.50
34	7 Avenue – 50 Street to 48 Street	S397	S398	200	VCT	0.007	85.66	1951	$ 550.00	$47,113.00
35	7 Avenue – 50 Street to 48 Street	S398	S399	200	VCT	-	31.96	1951	$ 550.00	$17,578.00
36	7 Avenue – 50 Street to 48 Street	S399	S400	200	VCT	0.011	53.12	1951	$ 550.00	$29,216.00
37	7 Avenue – 50 Street to 48 Street	S400	S401	200	VCT	0.035	85.58	1951	$ 550.00	$47,069.00
38	8 Avenue – 51 Street to 50 Street	S371	S402	200	VCT	0.033	116.14	1951	$ 550.00	$63,877.00
39	8 Avenue – 50 Street to 49 Street	S403	S404	200	VCT	0.023	87.84	1951	$ 550.00	$48,312.00
40	8 Avenue – 50 Street to 49 Street	S404	S405	200	VCT	0.012	85.66	1951	$ 550.00	$47,113.00
41	9 Avenue – 51 Street to 50 Street	S387	S409	200	VCT	0.031	109.13	1951	$ 550.00	$60,021.50
42	9 Avenue – 50 Street to 48 Street	S410	S411	200	VCT	0.020	86.92	1951	$ 550.00	$47,806.00
43	9 Avenue – 50 Street to 48 Street	S411	S412	200	VCT	0.020	86.59	1951	$ 550.00	$47,624.50
44	9 Avenue – 50 Street to 48 Street	S412	S413	200	VCT	0.020	85.65	1951	$ 550.00	$47,107.50
45	9 Avenue – 50 Street to 48 Street	S413	S426	200	VCT	0.020	85.40	1951	$ 550.00	$46,970.00
46	10 Avenue – 51 Street to 50 Street	S357	S414	200	VCT	0.050	50.16	1953	$ 550.00	$27,588.00
47	10 Avenue – 50 Street to 48 Street	S415	S416	200	VCT	0.023	90.85	1951	$ 550.00	$49,967.50
48	10 Avenue – 50 Street to 48 Street	S416	S417	200	VCT	0.006	82.61	1951	$ 550.00	$45,435.50

Table C.1
Cost Estimate for Existing (2017) Wastewater Upgrades System (Based on Condition Assessment)

Municipal Servicing Plan Update
Town of Edson

No.	Line Location	From MH	To MH	Diameter (mm)	Pipe Material	Slope (%)	Length (m)	Year	Unit Price ($/m)	Replacement Cost
49	10 Avenue – 50 Street to 48 Street	S417	S418	200	VCT	0.007	85.43	1951	$ 550.00	$46,986.50
50	10 Avenue – 50 Street to 48 Street	S418	S425	200	VCT	0.028	85.87	1951	$ 550.00	$47,228.50
51	11 Avenue – 51 Street to 50 Street	S356	S420	200	VCT	0.066	53.02	1953	$ 550.00	$29,161.00
52	11 Avenue – 50 Street to 49 Street	S421	S422	200	VCT	0.006	86.80	1951	$ 550.00	$47,740.00
53	11 Avenue – 50 Street to 49 Street	S422	S423	200	VCT	0.006	86.57	1951	$ 550.00	$47,613.50
54	Alley East of 51 Street North of 9th Avenue	--	S409	200	VCT	0.013	70.10	1951	$ 550.00	$38,555.00
55	50 Street - 12th Avenue to 9th Avenue	S424	S421	200	VCT	0.005	52.34	1954	$ 550.00	$28,787.00
56	50 Street - 12th Avenue to 9th Avenue	S421	S415	200	VCT	0.029	110.92	1954	$ 550.00	$61,006.00
57	50 Street - 12th Avenue to 9th Avenue	S415	S410	200	VCT	0.012	110.71	1954	$ 550.00	$60,890.50
58	50 Street - 9th Avenue to 6th Avenue	S410	S403	200	VCT	0.007	111.11	1954	$ 550.00	$61,110.50
59	50 Street - 9th Avenue to 6th Avenue	S403	S396	200	VCT	0.004	110.92	1951	$ 550.00	$61,006.00
60	50 Street - 9th Avenue to 6th Avenue	S396	S390	200	VCT	0.011	110.91	1951	$ 550.00	$61,000.50
61	East of 49 Street 48 Street - 10th Avenue to 8th Avenue	S425	S426	200	VCT	0.008	110.93	1951	$ 550.00	$61,011.50
62	48 Street - 10th Avenue to 8th Avenue	S426	S428	200	VCT	0.012	55.59	1951	$ 550.00	$30,574.50
63	48 Street - 8th Avenue to 6th Avenue	S401	S394	200	VCT	0.025	110.89	1951	$ 550.00	$60,989.50
64	6 Avenue – 48 Street to 46 Street	S394	S431	200	VCT	0.003	33.87	1951	$ 550.00	$18,628.50
65	6 Avenue – 48 Street to 46 Street	S431	S432	200	VCT	0.004	51.22	1951	$ 550.00	$28,171.00
66	6 Avenue – 48 Street to 46 Street	S432	S433	200	VCT	0.014	85.66	1951	$ 550.00	$47,113.00
67	6 Avenue – 48 Street to 46 Street	S433	S434	200	VCT	0.004	171.80	1951	$ 550.00	$94,490.00
68	7 Avenue – 47 Street to 46 Street	S437	S439	200	VCT	0.004	68.24	1951	$ 550.00	$37,532.00
69	7 Avenue – 47 Street to 46 Street	S439	S440	200	VCT	0.004	123.44	1951	$ 550.00	$67,892.00
70	Alley North of 8 Avenue 48 Street - End of 48 Street	S428	S443	200	VCT	0.004	40.54	1951	$ 550.00	$22,297.00
71	47 Street - Alley North of 6 Avenue to 6 Avenue	S437	S433	200	VCT	0.004	64.01	1951	$ 550.00	$35,205.50
72	6 Avenue - 46 Street to 43 Street	S434	S454	200	VCT	0.005	171.90	1951	$ 550.00	$94,545.00
73	Northbound 1 Avenue - 52 Street to 50 Street	S568	S569	200	VCT	0.004	46.33	1951	$ 550.00	$25,481.50
74	1 Avenue - 52 Street to 50 Street	S569	S570	200	VCT	0.004	75.29	1951	$ 550.00	$41,409.50
75	2 Avenue - 52 Street to 50 Street	S571	S572	200	VCT	0.005	86.26	1951	$ 550.00	$47,443.00
76	2 Avenue - 52 Street to 50 Street	S572	S573	200	VCT	0.007	78.94	1951	$ 550.00	$43,417.00
77	2 Avenue - 52 Street to 50 Street	S574	S575	250	VCT	0.003	173.84	1951	$ 540.00	$93,873.60
78	3 Avenue - 52 Street to 50 Street	S577	S578	250	VCT	0.005	85.35	1951	$ 540.00	$46,089.00
79	3 Avenue - 52 Street to 50 Street	S578	S579	250	VCT	0.003	85.94	1951	$ 540.00	$46,407.60
80	3 Avenue - 52 Street to 50 Street	S579	S580	200	VCT	0.010	109.13	1951	$ 550.00	$60,021.50
81	4 Avenue - 54 Street to 52 Street	S583	S584	200	VCT	0.004	104.25	1951	$ 550.00	$57,337.50
82	4 Avenue - 54 Street to 52 Street	S584	S585	200	VCT	0.004	171.83	1951	$ 550.00	$94,506.50
83	4 Avenue - 52 Street to 50 Street	S585	S586	200	VCT	0.014	106.08	1951	$ 550.00	$58,344.00
84	4 Avenue - 52 Street to 50 Street	S586	S587	200	VCT	0.033	65.24	1951	$ 550.00	$35,882.00
85	4 Avenue - 52 Street to 50 Street	S587	S588	200	VCT	0.018	109.13	1951	$ 550.00	$60,021.50
86	5 Avenue - 55 Street to 52 Street	S593	S594	200	VCT	0.004	100.58	1951	$ 550.00	$55,319.00
87	52 Street – 6 Avenue to 4 Avenue	S594	S585	250	VCT	0.011	110.87	1951	$ 540.00	$59,869.80
88	52 Street – 4 Avenue to 1 Avenue	S585	S577	250	VCT	0.003	110.91	1951	$ 540.00	$59,891.40
89	51 Street – 3 Avenue to 2 Avenue	S579	S574	250	VCT	0.003	111.25	1951	$ 540.00	$60,075.00
90	50 Street – 6 Avenue to 4 Avenue	S390	S601	200	VCT	0.026	110.86	1951	$ 550.00	$60,973.00
91	50 Street – 6 Avenue to 4 Avenue	S601	S602	200	VCT	0.026	110.86	1951	$ 550.00	$60,973.00
92	50 Street – 4 Avenue to 1 Avenue	S602	S603	200	VCT	0.038	110.88	1951	$ 550.00	$60,984.00
93	50 Street – 4 Avenue to 1 Avenue	S603	S575	200	VCT	0.018	111.15	1951	$ 550.00	$61,132.50
94	50 Street – 4 Avenue to 1 Avenue	S575	S570	200	VCT	0.004	115.82	1951	$ 550.00	$63,701.00
95	3 Avenue - 50 Street to 48 Street	S603	S614	200	VCT	0.004	56.08	1951	$ 550.00	$30,844.00
96	4 Avenue - 50 Street to 48 Street	S625	S626	200	VCT	0.012	107.30	1951	$ 550.00	$59,015.00
97	4 Avenue - 50 Street to 48 Street	S626	S627	200	VCT	0.007	171.91	1951	$ 550.00	$94,550.50

Table C.1
Cost Estimate for Existing (2017) Wastewater Upgrades System (Based on Condition Assessment)

Municipal Servicing Plan Update
Town of Edson

No.	Line Location	From MH	To MH	Diameter (mm)	Pipe Material	Slope (%)	Length (m)	Year	Unit Price ($/m)	Replacement Cost
98	4 Avenue - 48 Street to 45 Street	S627	S628	200	VCT	0.004	171.83	1951	$ 550.00	$94,506.50
99	4 Avenue - 48 Street to 45 Street	S628	S629	200	VCT	0.004	171.85	1951	$ 550.00	$94,517.50
100	4 Avenue - 48 Street to 45 Street	S629	S630	200	VCT	0.004	8.23	1951	$ 550.00	$4,526.50
101	4 Avenue - 48 Street to 45 Street	S630	S631	200	VCT	0.004	97.54	1951	$ 550.00	$53,647.00
102	Alley East of 50 Street - South of 4 Avenue to 3 Avenue	S643	S614	200	VCT	0.004	50.29	1951	$ 550.00	$27,659.50
103	48 Street – East of 48 Street	S431	S644	200	VCT	0.007	51.43	1951	$ 550.00	$28,286.50
104	48 Street – 6 Avenue to 4 Avenue	S394	S644	250	VCT	0.012	36.38	1951	$ 540.00	$19,645.20
105	48 Street – 6 Avenue to 4 Avenue	S644	S635	250	VCT	0.012	74.06	1951	$ 540.00	$39,992.40
106	48 Street – 6 Avenue to 4 Avenue	S635	S627	250	VCT	0.016	111.54	1951	$ 540.00	$60,231.60
107	48 Street – 4 Avenue to 1 Avenue	S627	S645	250	VCT	0.010	111.15	1951	$ 540.00	$60,021.00
108	48 Street – 4 Avenue to 1 Avenue	S645	S610	250	VCT	0.010	113.26	1951	$ 540.00	$61,160.40
109	47 Street – 5 Avenue to 4 Avenue	S646	S628	200	VCT	-	72.09	1951	$ 550.00	$39,649.50
110	46 Street – 4 Avenue to 2 Avenue	S647	S624	200	VCT	0.042	52.73	1951	$ 550.00	$29,001.50
111	Highway 16 – Between east of 43 Street and west of 40 Street	S656	S657	450	VCT	0.002	121.31	1949	$ 900.00	$109,179.00
112	2 Avenue - 45 Street to 43 Street	S665	S666	375	VCT	0.003	52.06	1954	$ 720.00	$37,483.20
113	2 Avenue - 45 Street to 43 Street	S666	S667	375	VCT	0.003	90.96	1954	$ 720.00	$65,491.20
114	2 Avenue - 4 Avenue to Highway 16	S687	S688	375	VCT	0.015	98.45	1952	$ 720.00	$70,884.00
115	2 Avenue - 4 Avenue to Highway 16	S688	S667	300	VCT	0.015	103.78	1952	$ 600.00	$62,268.00
						0.020	60.89	1953	$ 550.00	$33,489.50
						0.030	60.65	1953	$ 550.00	$33,357.50
					PVC	0.014	12.87	2014	$ 720.00	$9,266.40
									Sub-Total Cost	$5,876,324.80
									Contingency (30%)	$1,762,897.44
									Engineering and Administration (10%)	$587,632.48
									Capital Cost	**$8,226,854.72**

Table C.2
Cost Estimate for Future (2032) Wastewater Upgrades System (Based On Condition Assessment)

Municipal Servicing Plan Update
Town of Edson

No.	Line Locations	From MH	To MH	Diameter (mm)	Pipe Material	Slope (%)	Length (m)	Year	Unit Price ($/m)	Replacement Cost
1	16 Avenue – 49 Street to East of 48 Street	S268	S269	200	VCT	0.021	104.55	1962	$550.00	$57,502.50
2	16 Avenue – 49 Street to East of 48 Street	S269	S270	200	VCT	0.021	57.91	1962	$550.00	$31,850.50
3	16 Avenue – 49 Street to East of 48 Street	S270	S271	200	VCT	0.021	74.07	1962	$550.00	$40,738.50
4	16 Avenue – 49 Street to East of 48 Street	S271	S272	200	VCT	0.021	73.15	1962	$550.00	$40,232.50
5	17 Avenue – Edson Drive to East of 48 Street	S281	S282	200	VCT	0.004	75.73	1951	$550.00	$41,651.50
6	17 Avenue – Edson Drive to East of 48 Street	S282	S283	200	VCT	0.018	60.05	1962	$550.00	$33,027.50
7	17 Avenue – Edson Drive to East of 48 Street	S283	S284	200	VCT	0.006	71.02	1962	$550.00	$39,061.00
8	17 Avenue – Edson Drive to East of 48 Street	S284	S285	200	VCT	0.018	86.26	1962	$550.00	$47,443.00
9	17 Avenue – Edson Drive to East of 48 Street	S285	S286	200	VCT	0.018	86.26	1962	$550.00	$47,443.00
10	17 Avenue – Edson Drive to East of 48 Street	S286	S287	200	VCT	0.020	92.96	1962	$550.00	$51,128.00
11	12 Avenue – 51 Street to 50 Street	S259	S260	200	VCT	0.014	82.91	1962	$550.00	$45,600.50
12	12 Avenue – 51 Street to 50 Street	S260	S261	200	VCT	0.006	91.44	1962	$550.00	$50,292.00
13	12 Avenue – 50 Street to East of 49 Street	S261	S262	200	VCT	0.006	91.44	1962	$550.00	$50,292.00
14	12 Avenue – 50 Street to East of 49 Street	S262	S263	200	VCT	0.006	88.73	1962	$550.00	$48,801.50
15	12 Avenue – 50 Street to East of 49 Street	S263	S264	200	VCT	--	30.48	1962	$550.00	$16,764.00
16	18 Avenue – Southeast of Edson Dr. to East of 48 Street	S296	S297	200	VCT	0.009	71.02	1962	$550.00	$39,061.00
17	18 Avenue – Southeast of Edson Dr. to East of 48 Street	S297	S298	200	VCT	0.036	105.46	1962	$550.00	$58,003.00
18	18 Avenue – Southeast of Edson Dr. to East of 48 Street	S298	S299	200	VCT	0.021	106.68	1962	$550.00	$58,674.00
19	East of 48 Street - Edson Drive to 12 Avenue	S330	S331	200	VCT	0.006	62.89	1972	$550.00	$34,589.50
20	East of 48 Street - Edson Drive to 12 Avenue	S331	S299	200	VCT	0.005	111.34	1972	$550.00	$61,237.00
21	East of 48 Street - Edson Drive to 12 Avenue	S299	S287	200	VCT	0.008	113.57	1962	$550.00	$62,463.50
22	East of 48 Street - Edson Drive to 12 Avenue	S287	S272	200	VCT	0.008	113.17	1962	$550.00	$62,243.50
23	51 Street – 12 Avenue to 11 Avenue	S259	S356	200	VCT	0.026	113.39	1962	$550.00	$62,364.50
24	52 Street – 8 Avenue to 6 Avenue	S386	S366	250	VCT	0.006	60.58	1970	$540.00	$32,713.20
25	8 Avenue – 48 Street to 47 Street	S441	S442	200	VCT	0.018	91.68	1959	$550.00	$50,424.00
26	Alley North of 9 Avenue – 48 Street to 46 Street	S444	S445	200	VCT	0.015	114.91	1959	$550.00	$63,200.50
27	Alley North of 9 Avenue – 48 Street to 46 Street	S445	S446	200	VCT	0.003	52.27	1959	$550.00	$28,748.50
28	Alley North of 9 Avenue – 48 Street to 46 Street	S446	S447	200	VCT	0.004	67.97	1959	$550.00	$37,383.50
29	Alley North of 9 Avenue – 48 Street to 46 Street	S447	S448	200	VCT	0.004	56.39	1959	$550.00	$31,014.50
30	10 Avenue – 48 Street to East of 48 Street	S449	S450	200	VCT	0.018	121.92	1962	$550.00	$67,056.00
31	East of 48 Street - 12 Avenue to 10 Avenue	S272	S451	200	VCT	0.006	77.42	1962	$550.00	$42,581.00
32	East of 48 Street - 12 Avenue to 10 Avenue	S451	S452	200	VCT	0.006	59.29	1962	$550.00	$32,609.50
33	East of 48 Street - 12 Avenue to 10 Avenue	S452	S450	200	VCT	0.006	57.60	1962	$550.00	$31,680.00
34	East of 48 Street - 10 Avenue to 8 Avenue	S450	S445	200	VCT	0.014	115.94	1962	$550.00	$63,767.00
35	East of 48 Street - 10 Avenue to 8 Avenue	S445	S442	200	VCT	0.004	108.90	1959	$550.00	$59,895.00
36	East of 48 Street - 8 Avenue to 6 Avenue	S442	S453	200	VCT	0.004	56.08	1959	$550.00	$30,844.00
37	East of 48 Street - 8 Avenue to 6 Avenue	S453	S438	200	VCT	0.004	77.15	1959	$550.00	$42,432.50
38	East of 48 Street - 8 Avenue to 6 Avenue	S438	S436	200	VCT	0.009	83.52	1959	$550.00	$45,936.00
39	East of 48 Street - 8 Avenue to 6 Avenue	S436	S435	200	VCT	0.008	47.70	1959	$550.00	$26,235.00
40	East of 48 Street - 8 Avenue to 6 Avenue	S435	S431	200	VCT	0.033	12.19	1959	$550.00	$6,704.50
41	6 Avenue – 46 Street to 43 Street	S454	S455	200	VCT	0.004	102.43	1957	$550.00	$56,336.50
42	6 Avenue – 46 Street to 43 Street	S455	S456	200	VCT	0.015	94.06	1957	$550.00	$51,733.00
43	6 Avenue – 46 Street to 43 Street	S456	S457	200	VCT	0.030	98.46	1957	$550.00	$54,153.00
44	7 Avenue – 44 Street to 43 Street	S458	S459	200	VCT	0.004	73.15	1959	$550.00	$40,232.50
45	7 Avenue – 44 Street to 43 Street	S459	S460	200	VCT	0.004	86.26	1959	$550.00	$47,443.00
46	7 Avenue – 44 Street to 43 Street	S460	S461	200	VCT	0.055	86.26	1959	$550.00	$47,443.00

Table C.2
Cost Estimate for Future (2032) Wastewater Upgrades System (Based On Condition Assessment)

Municipal Servicing Plan Update
Town of Edson

No.	Line Locations	From MH	To MH	Diameter (mm)	Pipe Material	Slope (%)	Length (m)	Year	Unit Price ($/m)	Replacement Cost
47	8 Avenue – 45 Street to 43 Street	S463	S464	200	VCT	0.005	99.36	1964	$550.00	$54,648.00
48	8 Avenue – 45 Street to 43 Street	S464	S465	200	VCT	0.004	122.96	1964	$550.00	$67,628.00
49	8 Avenue – 45 Street to 43 Street	S465	S466	200	VCT	0.030	122.83	1964	$550.00	$67,556.50
50	8A Avenue – 45A Avenue to 45 Avenue	S467	S468	200	VCT	0.005	121.01	1964	$550.00	$66,555.50
51	8B Avenue – 45A Street to 45 Street	S469	S470	200	VCT	0.005	63.09	1964	$550.00	$34,699.50
52	9 Avenue – 44 Street to 43A Street	S471	S472	200	VCT	0.018	130.15	1965	$550.00	$71,582.50
53	10 Avenue – 45 Street to 43 Street	S473	S474	200	VCT	0.006	77.72	1965	$550.00	$42,746.00
54	10 Avenue – 45 Street to 43 Street	S474	S475	200	VCT	0.005	94.49	1965	$550.00	$51,969.50
55	10 Avenue – 45 Street to 43 Street	S475	S476	200	VCT	0.014	96.01	1965	$550.00	$52,805.50
56	10 Avenue – 45 Street to 43 Street	S476	S477	200	VCT	0.014	68.58	1965	$550.00	$37,719.00
57	10 Avenue – 45 Street to 43 Street	S477	S478	200	VCT	0.016	43.56	1965	$550.00	$23,958.00
58	North of 9 Avenue - West of 9 Avenue to West of 10 Avenue	S480	S474	200	VCT	--	304.80	1968	$550.00	$167,640.00
59	West of 46 Street - North of 12 Avenue	--	S479	200	VCT	--	94.18	1968	$550.00	$51,799.00
60	West of 46 Street - North of 12 Avenue	S479	S480	200	VCT	--	121.32	1968	$550.00	$66,726.00
61	45 Street - 8B Avenue to 8 Avenue	S470	S468	200	VCT	0.004	44.86	1964	$550.00	$24,673.00
62	45 Street - 8B Avenue to 8 Avenue	S468	S463	200	VCT	0.003	51.21	1964	$550.00	$28,165.50
63	44 Street - 9 Avenue to 8 Avenue	S481	S464	200	VCT	0.013	85.34	1964	$550.00	$46,937.00
64	44 Street - Alley Noth of 7 Avenue to 7 Avenue	S462	S458	200	VCT	0.004	44.20	1964	$550.00	$24,310.00
65	43A Street - 10 Avenue to 9 Avenue	S476	S472	200	VCT	0.010	99.34	1965	$550.00	$54,637.00
66	43 Street - 10 Avenue to 8 Avenue	S478	S482	200	VCT	0.004	83.82	1965	$550.00	$46,101.00
67	43 Street - 10 Avenue to 8 Avenue	S482	S466	200	VCT	0.005	84.12	1965	$550.00	$46,266.00
68	East of 3 Avenue - Between 41 Street and 40 Street	S534	S532	200	VCT	0.006	48.46	1963	$550.00	$26,653.00
69	East of 3 Avenue - Between 41 Street and 40 Street	S532	S533	200	VCT	0.007	17.22	1963	$550.00	$9,471.00
70	4 Avenue - West of 40 Street	S536	S537	200	VCT	0.025	25.04	1963	$550.00	$13,772.00
71	4 Avenue - West of 40 Street	S537	S538	200	VCT	0.010	26.51	1963	$550.00	$14,580.50
72	5 Avenue - 41 Street to East of 41 Street	S539	S540	200	VCT	0.006	53.79	1963	$550.00	$29,584.50
73	5 Avenue - 41 Street to East of 41 Street	S540	S541	200	VCT	0.006	43.89	1963	$550.00	$24,139.50
74	6 Avenue - 43 Street to 42 Street	S457	S527	200	VCT	0.026	110.96	1957	$550.00	$61,028.00
75	6 Avenue - 43 Street to 42 Street	S527	S528	200	VCT	0.014	108.52	1957	$550.00	$59,686.00
76	6 Avenue - 42 Street to 40 Street	S543	S544	200	VCT	0.006	46.63	1962	$550.00	$25,646.50
77	6 Avenue - 42 Street to 40 Street	S544	S545	200	VCT	0.006	102.41	1962	$550.00	$56,325.50
78	6 Avenue - 42 Street to 40 Street	S545	S546	200	VCT	0.006	48.77	1962	$550.00	$26,823.50
79	7 Avenue - 43 Street to 42 Street	S461	S529	200	VCT	0.013	86.26	1959	$550.00	$47,443.00
80	7 Avenue - 43 Street to 42 Street	S529	S530	200	VCT	0.047	86.26	1959	$550.00	$47,443.00
81	8 Avenue - East of 43 Street	S466	S531	200	VCT	0.006	52.63	1964	$550.00	$28,946.50
82	8 Avenue - East of 43 Street	S531	S532	200	VCT	0.023	59.31	1964	$550.00	$32,620.50
83	42 Street - 8 Avenue to 6 Avenue	S532	S530	200	VCT	0.024	112.78	1964	$550.00	$62,029.00
84	42 Street - 8 Avenue to 6 Avenue	S530	S528	300	VCT	0.002	111.56	1959	$600.00	$66,936.00
85	41 Street - Northeast of 6 Avenue	--	S544	200	VCT	--	56.95	1962	$550.00	$31,322.50
86	41 Street - 6 Avenue to 4 Avenue	S544	S542	200	VCT	0.005	61.87	1963	$550.00	$34,028.50
87	41 Street - 6 Avenue to 4 Avenue	S542	S539	200	VCT	0.031	51.43	1963	$550.00	$28,286.50
88	41 Street - 6 Avenue to 4 Avenue	S539	S536	200	VCT	0.017	86.58	1963	$550.00	$47,619.00
89	41 Street - 4 Avenue to South of 4 Avenue	S537	S534	200	VCT	0.019	109.74	1963	$550.00	$60,357.00
90	54 Street - 5 Avenue to 4 Avenue	S598	S581	375	CONC	0.013	62.94	1962	$720.00	$45,316.80
91	54 Street - 4 Avenue to 1 Avenue	S581	S599	375	CONC	0.004	93.05	1962	$720.00	$66,996.00
92	54 Street - 4 Avenue to 1 Avenue	S599	S560	375	CONC	0.002	113.39	1962	$720.00	$81,640.80

Table C.2
Cost Estimate for Future (2032) Wastewater Upgrades System (Based On Condition Assessment)

Municipal Servicing Plan Update
Town of Edson

No.	Line Locations	From MH	To MH	Diameter (mm)	Pipe Material	Slope (%)	Length (m)	Year	Unit Price ($/m)	Replacement Cost
93	54 Street - 4 Avenue to 1 Avenue	S600	S562	375	CONC	0.002	125.65	1985	$720.00	$90,468.00
94	53 Street - 6 Avenue to 4 Avenue	S360	S592	200	VCT	0.005	111.21	1959	$550.00	$61,165.50
95	53 Street - 6 Avenue to 4 Avenue	S592	S584	200	VCT	0.005	110.93	1959	$550.00	$61,011.50
96	2 Avenue – 50 Street to 48 Street	S575	S609	300	VCT	0.002	174.05	1962	$600.00	$104,430.00
97	2 Avenue – 50 Street to 48 Street	S609	S610	300	VCT	0.002	171.84	1962	$600.00	$103,104.00
98	2 Avenue – 48 Street to 45 Street	S610	S611	380	VCT	0.002	171.84	1962	$720.00	$123,724.80
99	2 Avenue – 48 Street to 45 Street	S611	S612	380	VCT	0.002	172.52	1962	$720.00	$124,214.40
100	3 Avenue – 50 Street to 48 Street	S615	S616	200	VCT	0.008	101.50	1964	$550.00	$55,825.00
101	49 Street – 3 Avenue to 1 Avenue	S616	S609	200	VCT	0.015	112.93	1961	$550.00	$62,111.50
102	2 Avenue – 45 Street to 43 Street	S613	S661	380	VCT	0.002	172.52	1962	$720.00	$124,214.40
103	2 Avenue – 45 Street to 43 Street	S661	S662	380	VCT	0.005	38.40	1962	$720.00	$27,648.00
104	2 Avenue – 45 Street to 43 Street	S662	S663	380	VCT	0.005	59.12	1962	$720.00	$42,566.40
105	2 Avenue – 43 Street to East of 42 Street	S663	S664	380	VCT	0.005	74.55	1962	$720.00	$53,676.00
106	2 Avenue – 43 Street to East of 42 Street	S664	S665	380	VCT	0.003	111.00	1962	$720.00	$79,920.00
107	4 Avenue – 43 Street to 42 Street	S669	S687	200	VCT	0.010	172.50	1957	$550.00	$94,875.00
108	Alley North Of 4 Avenue - 45 Street to 43 Street	S670	S671	150	VCT	0.004	124.36	1957	$550.00	$68,398.00
109	Alley North Of 4 Avenue - 45 Street to 43 Street	S671	S672	200	VCT	--	43.53	1957	$550.00	$23,941.50
110	Alley North Of 4 Avenue - 45 Street to 43 Street	S672	S673	200	VCT	0.022	80.77	1957	$550.00	$44,423.50
111	Alley North Of 4 Avenue - 45 Street to 43 Street	S673	S674	200	VCT	0.022	74.98	1957	$550.00	$41,239.00
112	West of 43 Street - 4 Avenue to 2 Avenue	S680	S681	200	VCT	--	45.80	1962	$550.00	$25,190.00
113	West of 43 Street - 4 Avenue to 2 Avenue	S681	S682	200	VCT	--	24.05	1962	$550.00	$13,227.50
114	West of 43 Street - 4 Avenue to 2 Avenue	S682	S662	200	VCT	--	30.24	1962	$550.00	$16,632.00
115	43 Street - 5 Avenue to 4 Avenue	S674	S685	200	VCT	0.015	36.88	1957	$550.00	$20,284.00
116	43 Street - 5 Avenue to 4 Avenue	S685	S669	200	VCT	0.015	38.10	1957	$550.00	$20,955.00
117	43 Street - 4 Avenue to 2 Avenue	S683	S684	200	VCT	0.038	48.56	1962	$550.00	$26,708.00
118	43 Street - 4 Avenue to 2 Avenue	S684	S663	200	VCT	0.026	117.55	1962	$550.00	$64,652.50
119	2 Avenue – Between east of 42 Street and west of 41 Street	S667	S668	300	VCT	0.047	14.40	1962	$600.00	$8,640.00
120	2 Avenue – Between east of 42 Street and west of 41 Street	S668	S656	300	VCT	0.047	26.82	1962	$600.00	$16,092.00
121	North of CNR - 40 Street to Northeast of S716	S721	S722	525	VCT	0.002	90.96	1949	$1,060.00	$96,417.60
122	North of CNR - 40 Street to Northeast of S716	S722	S723	525	VCT	--	152.40	1949	$1,060.00	$161,544.00
123	North of CNR - North of Lot 6 to North of 5 Avenue	S723	S731	525	VCT	--	304.80	1949	$1,060.00	$323,088.00
124	North of CNR - North of Lot 6 to North of 5 Avenue	S731	S732	525	VCT	--	152.40	1949	$1,060.00	$161,544.00
125	North of CNR - North of Lot 6 to North of 5 Avenue	S732	S733	525	VCT	--	162.56	1949	$1,060.00	$172,313.60
126	North of CNR - North of Lot 6 to North of 5 Avenue	S733	S734	525	VCT	--	111.76	1949	$1,060.00	$118,465.60
127	North of CNR - North of Lot 6 to North of 5 Avenue	S734	--	525	VCT	--	126.33	1949	$1,060.00	$133,909.80
									Sub-Total Cost	$6,995,467.40
									Contingency (30%)	$2,098,640.22
									Engineering and Administration (10%)	$699,546.74
									Capital Cost	**$9,793,654.36**

Table C.3
Cost Estimate for Wastewater Upgrades (Based on Hydraulic Capacity)

Municipal Servicing Plan Update
Town of Edson

5 Year 4 Hour - HGL to 1.0 m Below Ground

Link No.	Line Location	Line ID	Original Diameter (mm)	Twining Diameter (mm)	Length (m)	Unit Price ($/m)	Replacement Cost
1-1	54 Street Line and R.O.W. (1 Avenue to 5 Avenue to Park to 6 Avenue)	L509, L508, L767	375	375	257	$720	$185,040.00
1-2	54 Street Line and R.O.W. (2 Avenue to 4 Avenue)	L531, L532	375	375	200	$720	$144,000.00
1-3	54 Street (1 Avenue to 2 Avenue)	L533	375	375	126	$720	$90,720.00
				Total Replacement Length	326	metres	
				Sub-total Cost			$419,760.00
				Contingency (30%)			$125,928.00
				Engineering and Administration (10%)			$41,976.00
				Capital Cost			**$587,664.00**

25 Year 4 Hour - Pipe Peak Flow > 1.2x Pipe Capacity

Link No.	Line Location	Line ID	Original Diameter (mm)	Twining Diameter (mm)	Length (m)	Unit Price ($/m)	Replacement Cost
2-1	4 Avenue (southside 70 Street to 68 Street)	L593	200	200	160	$600	$96,000
2-2	54 Street Line (6 Avenue North in Grffiths Park)	L512	375	375	72	$720	$51,840
2-3	54 Street Line and R.O.W. (2 Avenue to 4 Avenue)	L532	375	450	114	$900	$102,600
2-4	54 Street (1 Avenue to 2 Avenue)	L533	375	525	126	$1,060	$133,560
2-5	10 Avenue (52 Street to 53 Street to 52 Street)	L198, L816	200	200	161	$600	$96,600
2-6	1 Avenue (50 Street to 51 Street)	L412	200	200	175	$600	$105,000
2-7	Sanitary R.O.W. (East of 12 Avenue/48 Street Intersection, North-South partially along 47 Street)	L89, L70, L71, L73, L78, L80, L81, L781, L782, L374	200	200	741	$600	$444,600
2-8	Sanitary R.O.W. near 42 Street between7 Avenue to 6 Avenue)	L269, L268, L267	200	200	225	$600	$135,000
2-9	Sanitary R.O.W. (South of 2 Avenue, East End, between C.N.R and 2 Avenue)	L343	450	450	142	$900	$127,800
				Total Replacement Length	1916	metres	
				Sub-total Cost			$1,293,000.00
				Contingency (30%)			$387,900.00
				Engineering and Administration (10%)			$129,300.00
				Capital Cost			**$1,810,200.00**

5 Year 4 Hour - Miminizing Risk of Basement Flooding

Link No.	Line Location	Line ID	Original Diameter (mm)	Twining Diameter (mm)	Length (m)	Unit Price ($/m)	Replacement Cost
3-1	43 Street (Bend to segment towards 8 Avenue)	L271	200	200	84	$600	$50,400
3-2	48 Street (Edson Drive to 18 Avenue)	L45, L56	200	200	174	$600	$104,400
3-3	Sanitary R.O.W. - Near 41 Street (Cul-de-sac bulb to 2 Avenue)	L323	200	200	48	$600	$28,800
				Total Replacement Length	306	metres	
				Sub-total Cost			$183,600.00
				Contingency (30%)			$55,080.00
				Engineering and Administration (10%)			$18,360.00
				Capital Cost			**$257,040.00**

Table C.4
Cost Estimate for Future Wastewater Upgrades (Based on Future Areas to be Developed)

Municipal Servicing Plan Update
Town of Edson

2032 (15 Year Horizon) Development

Town of Edson

Item	Development Area	Area Classification	Pipe Size (mm)	Pipe Length (m)	Unit Price	Cost
1	Area 1 - Gravity Sewer	Industrial/ Commercial	300	700	$600	$420,000
2	Area 2 - Gravity Sewer	Industrial/ Commercial	200	250	$500	$125,000
3	Area 3a - Gravity Sewer	Industrial/ Commercial	200	200	$500	$100,000
4	South - Area 5					
4a	- Gravity Sewer	Industrial/ Commercial	300	700	$600	$420,000
4b	- Gravity Sewer	Industrial/ Commercial	750	3,200	$1,750	$5,600,000
4c	- Gravity Sewer	Industrial/ Commercial	900	500	$2,150	$1,075,000
5	Area 8, 8a	Residential	200	1,500	$500	$750,000
6	Area 9	Residential	200	1,500	$500	$750,000
7	Area 10	Residential	200	200	$500	$100,000
8	Area V-s	Industrial/ Commercial	300	600	$600	$360,000
9	Area M-g	Residential	250	500	$540	$270,000
10	Combined Areas: 11, 12, 13					
10a	- Gravity Sewer	Industrial/ Commercial	300	500	$600	$300,000
10b	- Gravity Sewer	Industrial/ Commercial	450	800	$900	$720,000
10c	- Gravity Sewer	Industrial/ Commercial	600	1,600	$1,250	$2,000,000
11	Area 13					
11a	- Lift Station	Industrial/ Commercial				$1,500,000
11b	- Forcemain	Industrial/ Commercial	150	1,300	$450	$585,000
			Total Future Sanitary Length	14,050	metres	
			Sub-Total Cost			$15,075,000
			Engineering and Administration (10%)			$4,522,500
			Contingency (30%)			$1,507,500
			Capital Cost			**$21,105,000**

Fringe Area (Yellowhead County)

Item	Development Area	Area Classification	Pipe Size (mm)	Pipe Length (m)	Unit Price	Cost
12	Area 15	Industrial/ Commercial	300	300	$600	$180,000
13	Area 17	Industrial/ Commercial	300	800	$600	$480,000
14	East - Combined Areas: 15,17					
14a	- Lift Station	Industrial/ Commercial				$1,500,000
14b	- Forcemain	Industrial/ Commercial	200	1,700	$550	$935,000
15	West - Areas 18					
15a	- Gravity Sewer	Industrial/ Commercial	375	1,000	$720	$720,000
15b	- Gravity Sewer	Industrial/ Commercial	450	800	$900	$720,000
16	West - Areas 23	Industrial/ Commercial	450	2,800	$900	$2,520,000
			Total Future Sanitary Length	7,400	metres	
			Sub-Total Cost			7,055,000
			Engineering and Administration (10%)			$2,116,500
			Contingency (30%)			$705,500
			Capital Cost			**$9,877,000**
			2032 Development Total Capital Cost			**$30,982,000**

Table C.4
Cost Estimate for Future Wastewater Upgrades (Based on Future Areas to be Developed)

Municipal Servicing Plan Update
Town of Edson

2042 (25 Year Horizon) Development

Town of Edson

Item	Development Area	Area Classification	Pipe Size (mm)	Pipe Length (m)	Unit Price	Cost
Residential Development and Commercial / Industrial Development						
2042 (25 Year Horizon) Areas in Town of Edson						
Item	**Development Area**	**Area Classification**	**Pipe Size (mm)**	**Pipe Length (m)**	**Unit Price**	**Amount**
17	Area 3	Residential	200	500	$500	$250,000
18	Area 6	Residential	250	450	$540	$243,000
19	Area 7	Residential	200	450	$500	$225,000
20	Area 14	Industrial/ Commercial	300	500	$600	$300,000
	Total Future Sanitary Length		Total Future Sanitary Length	1,900	metres	
			Sub-Total Cost			$1,018,000
			Engineering and Administration (10%)			$305,400
			Contingency (30%)			$101,800
			Capital Cost			**$1,425,200**

Fringe Area (Yellowhead County)

Item	Development Area	Area Classification	Pipe Size (mm)	Pipe Length (m)	Unit Price	Cost
17	Area 16					
17a	- Gravity Sewer	Industrial/ Commercial	300	1,000	$600	$600,000
17b	- Lift Station	Industrial/ Commercial				$1,500,000
17c	- Forcemain	Industrial/ Commercial	200	1,700	$550	$935,000
18	West - Area 19	Industrial/ Commercial	300	800	$600	$480,000
19	West - Area 20	Industrial/ Commercial	300	1,500	$600	$900,000
20	West - Combined Areas; 21, 22	Residential	375	1,600	$720	$1,152,000
			Total Future Sanitary Length	6,600	metres	
			Sub-Total Cost			$5,567,000
			Engineering and Administration (10%)			$1,670,100
			Contingency (30%)			$556,700
			Capital Cost			**$7,793,800**
			2042 Development Total Capital Cost			**$9,219,000**

Wastewater Systems – Condition Assessment

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
70 ST – 4A Ave to South of 4 Ave	S5	S10	200	VCT	58.09	1.78%	1972	• Multiple large cracks around STA 1.7 • 4 Sag lengths noted: 4m – 10m in length • Replacement Risk – Medium – Keep Monitoring
Highway 16 -South of Highway 16	S10	S34	200	VCT & PVC	97.10	0.40%	1972	• Multiple medium cracks between STA 42.5 to STA 43.5 • 6 Sag lengths noted: 3m – 14m in length. • 2 changes in material noted VCT to PVC Pipe: STA. 16.6 to STA 31.6 & STA 71.5 to STA 76 • Replacement Risk – Medium – Keep Monitoring
63 ST – 4 Ave to 3 Ave	S55	S75	200	VCT	77.92	1.58%	1972	• Multiple medium to minor cracks in pipe. • Circumferential crack with infiltration observed at STA 38.2. • 1 Sag noted: 5.8m in length. • Multiple deposits and minor encrustation along pipe. • Break identified – portion missing in pipe wall at STA 26.5 • Major Fracture identified in pipe at STA 29.5. • **Replacement Risk – High – Repair Recommended**
55 ST - North of 4 Ave to 1 Ave	S100	S91	250	VCT	114.02	0.34%	1976	• Multiple deposits and minor encrustation along pipe. • 3 Sag lengths noted: 10m – 13m in length. Slope is below recommended minimum of 0.40%. • Multiple Cracks in pipe. • Multiple fractures in pipe. Major fractures at STA 110 and STA 114.

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
								• **Replacement Risk – High – Repair Recommended**
North of 13 Ave – West Boundary to East Boundary of Trailer Park	S140	S141	200	PVC	91.44	0.70%	1973	• Deposits and encrustation along pipe. Large deposits at STA 89.9 to STA 91.6. • 2 PVC Type changes within length of Pipe • Replacement Risk – Medium – **Cleaning Recommended** - Keep Monitoring
Westhaven Drive – 12A Ave to 57 ST	S206	S239	200	Conc.	77.25	1.70%	1979	• Multiple deposits and encrustation along pipe. • Minor joint offset at STA 53.6. • Replacement Risk – Low – Keep Monitoring.
16 Ave – 49 ST to East of 48 ST	S271	S272	200	VCT	73.15	2.10%	1962	• Multiple deposits and encrustation along pipe. • Circumferential cracks at STA 29.3. & STA 61 • Major spiral fracture at STA 42.4 & Additional Fracture at STA 57.5 • Obstruction in pipe (rock) at STA 4.5. • **Replacement Risk – High – Repair Recommended**
17 Ave – Edson Drive to East of 48 ST	S286	S287	200	VCT	92.96	2.00%	1962	• Major Fracture at STA 1.7. • Multiple deposits and encrustation along pipe. • Multiple roots in joints along length of pipe. Major root intrusion at STA 60.0 – STA 71.0. Survey abandoned due to root invasion. • Major encrustation and deposit at STA 36.9. • Infiltration of water into sewer at STA 40.0. • **Replacement Risk – High – Repair Recommended**
East of 48 ST – Edson Drive to 12 Avenue	S287	S272	200	VCT	113.17	0.80%	1962	• 5 (minor) sags identified. 3.0m – 4.4m in length. Grades are uneven. • Multiple minor deposits and encrustations along pipe length.

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
								• Multiple roots intruding in joints (17+). Larger (medium sized) roots at STA 63.7 and STA 66.7. • Multiple cracks in pipe at STA 113.0 and STA 114.4 • Replacement Risk – Medium – Keep Monitoring
52 Street – 12 Ave to 10 Ave	S355	S337	200	VCT	95.40	2.42%	1973	• 1 Sag length noted: 4.6m in length. • Multiple joint offsets noted. Joint offset was sever enough at STA 46.2 to stop survey. Large encrustation formed at this joint offset as well. • **Replacement Risk – Medium to High – Spot Repair at major joint offset recommended.**
7 Ave – 52 Street to 51 ST	S366	S367	200	VCT & PVC	85.66	2.00%	1951	• Material change from PVC to VCT at STA 1.0. • Multiples cracks in pipe. Major crack identified at STA 38.3. • 3 Sag lengths noted: 2.6 – 5.5m in length. • Multiple roots intrusion in pipe ultimately blocking upper half of pipe and stopped survey at STA 75.0. • **Replacement Risk – High – Repair Recommended.**
Alley East of 50 ST – 12 Ave to 9 Ave	S421	S415	200	VCT & PVC	110.92	2.93%	1954	• Material change from PVC to VCT at STA 10.6 • Services lines extending into sewer main at STA. 54.3, 81.5 and 97.1. Survey blocked at 97.1. • Multiple root intrusion in joints (fine roots).

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
								• Fractures/brakes observed in pipe at STA 60.4, 77.6 and 81.5. • **Replacement Risk – Medium to High – Repair Recommended.**
48 ST – 8 Ave to 6 Ave	S401	S394	200	VCT & PVC	110.89	2.46%	1951	• Material change from PVC to VCT at STA 1.8. • Multiple cracks along length of pipe. • Major fractures/breaks identified from STA 1.8 to 7.4. • 6 sag lengths noted from 2.0m to 3.5m long. • Service line extending into sewer main at STA. 85.0, survey blocked. • **Replacement Risk – High – Repair Recommended.**
6 Ave – 48 ST to 46 ST	S394	S431	200	VCT	33.87	0.33%	1951	• 5 sag lengths noted from 1.5m to 7.0m long. Pipe slope is less than recommended minimum of 0.40%. • Pipe broken and joint is offset at STA 32.0. • **Replacement Risk – High – Repair Recommended.**
6 Ave – 48 ST to 46 ST	S431	S432	200	VCT	51.22	0.40%	1951	• 2 sag lengths noted, one 5.5m the second is 28.0m long. The camera was underwater for most of the 28.0m length. • Minor cracks identified along length. • Replacement Risk – Medium – Keep Monitoring
6 Ave – 48 ST to 46 ST	S433	S434	200	VCT	171.80	0.40%	1951	• 1 sag length noted, 2.0m long. • Minor sediment accumulation in bottom of pipe. • Service line extending into sewer main at STA. 62.2.

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
								• Multiple cracks at 7+ locations along pipe. • Multiple fractures between STA 72.9 – 75.0. • Pipe is fractured and broken at STA. 43.7. • **Replacement Risk – High – Repair Recommended.**
East of 48 ST – 8 Ave to 6 Ave	S442	S453	200	VCT & PVC	56.08	0.40%	1959	• Material change from VCT to PVC between STA 1.7 – 7.5. • Joint offset/displaced (medium) at STA 1.7 • Minor roots intrusion in joint at STA 1.7m • 1 sag length noted, 11.0m long. • Replacement Risk – Medium – Keep Monitoring
East of 48 ST – 8 Ave to 6 Ave	S435	S431	200	VCT	12.19	3.30%	1959	• 1 sag length noted, 8.3m long. • Minor depots/encrustation along line. • Replacement Risk – Low.
6 Ave – 46 ST to 43 ST	S434	S454	200	VCT	171.90	0.50%	1951	• 3 service line extending into sewer main along line. • Joint offset/displaced (medium) at STA 12.8. • Multiple cracks at 7+ locations along pipe. • Minor roots intrusion in joint and service ties along pipe. • Minor encrustation and sediment throughout pipe. • Replacement Risk – Medium – Keep Monitoring
10 Ave – 45 ST to 43 ST	S474	S475	200	VCT	94.49	0.50%	1965	• Multiple cracks in line. • Multiple encrustation and sediment deposits in line, until camera not able to pass through.

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
								• Suspected sag in pipe but unknown length because of build-up of encrustation and deposits in line. • **Replacement Risk – High – Cleaning Recommended – Attempt CCTV again.**
43 ST - 10 Ave to 8 Ave	S478	S482	200	VCT	83.82	0.40%	1965	• 3 sag lengths noted, 5.0m to 15.0m long. Camera underwater in 15.0m long sag. • Joint offset/displaced (medium) at STA 27.8. • Multiple encrustation and sediment deposits in line. • Replacement Risk – Medium – Keep Monitoring
6 Ave – 43 ST to 42 ST	S527	S528	200	VCT & PVC	108.52	1.40%	1957	• Material change from VCT to PVC between STA 46.5 – STA 48.1 & STA. 13.4 – STA 15.8 & STA 103.8 – 108.0. • 4 sags lengths noted, 1.5m to 4.7m long. • Major deposit and encrustation with infiltration through crack/break in pipe wall at service tie at STA 2.9. Survey unable to pass. • Multiple service tie-ins with multiple cracks. • Multiple major fractures. Major break in the pipe wall at service tie-in (soil is visible) between STA 17.0 to STA 17.6. • Major joint offset/displaced at STA 103.8. • **Replacement Risk – High – Repair Recommended.**
7 Ave – 43 ST to 42 ST	S529	S530	200	VCT	86.26	4.66%	1959	• Multiple large cracks and fractures at STA 1.7 and STA 84.5. • Large deposits and encrustations at STA 22.1 and 82.5. Surveys unable to pass these encrustations.

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
								• **Replacement Risk – High – Cleaning Recommended – Attempt CCTV again.**
42 ST – Near 8 Ave to 7 Ave	S532	S530	200	VCT	112.78	2.42%	1964	• Roots (fine) in joints along length of pipe. • Encrustations and deposits in pipe. At STA 10.5 survey unable to pass. At STA -8.8 survey unable to pass. • **Replacement Risk – High – Cleaning Recommended – Attempt CCTV again.**
41 ST– 6 Ave to 4 Ave	S539	S536	200	VCT	86.58	1.70%	1963	• 4 sag lengths noted, 5.0m to 15.0m long. • Pipe fracture at service tie-in at STA 29.2. Medium roots entering pipe. • Major spiral crack and some fractures with medium roots in crack at STA 28.1. • Roots (mostly fine in size) intruding at most pipe joints from STA 3.6 to STA 28.0. • **Replacement Risk – High – Repair Recommended.**
41 ST– 4 Ave to South of 4 Ave	S537	S534	200	VCT	109.74	1.94%	1963	• Deposits and encrustation at both ends of pipe Survey unable to pass at STA 150.0 due to severity of deposits and encrustation. • Major pipe offset/displaced at STA 2.7. Next segment is much lower and pipe is half full. • **Replacement Risk – High – Repair Recommended.**
1 Ave – 55 ST to 52 ST	S562	S563	380	VCT	155.16	0.20%	1978	• Multiple encrustation and sediment deposits in line. • Service line extending into sewer main at STA. 118.8. • Replacement Risk – Low.

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
53 ST – 6 Ave to 4 Ave	S360	S592	200	VCT	111.21	0.50%	1959	• 5 sag lengths noted, 2.0m to 7.6m long. • Replacement Risk – Medium – Keep Monitoring
50 ST – 2 Ave to 1 Ave	S575	S570	200	VCT & PVC	115.82	0.35%	1951	• Material change PVC to VCT STA 20.5. • Material change VCT to PVC from STA 34.2 to STA 35.2. • 1 sag length noted, 5.0m long. • Multiple cracks throughout length of pipe. Multiple major cracks and fractures (10+). • Multiple deposits and encrustations throughout pipe (5+). Moderate encrustration at service tie-in at STA 63.6. • Joint offset/displaced at STA 34.2. • Pipe 40%+ full from STA 76 to end. • **Replacement Risk – High – Repair Recommended.**
52 ST – 6 Ave to 4 Ave	S594	S585	250	VCT & PVC	110.87	1.09%	1951	• Material change from PVC to VCT at STA 11.5. • 2 sag lengths noted, 3.0m – 3.8m long. • Multiple cracks identified along pipe length. • Multiple encrustation and sediment (grease) deposits in line. • Service line tie-in appears defective. Large debris/deposit build up inside pipe at STA 82.2. • **Replacement Risk – High – Repair Recommended.**
53 ST – 4 Ave to 1 Ave	S585	S577	250	VCT	110.91	0.25%	1951	• Material change from VCT to Conc. at STA 13.4.

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
								• 1 sag length noted, 6.3m long. • Multiple cracks and fractures along pipe between STA 10.4 and 11.7. Major fracture at STA 8.6. • Major deposits and encrustations are located at 3 areas along the pipe. Camera could not pass through. • Service tie-in from side at STA 108.0. • Service tie-in along top major deposit and encrustation at STA 108.2. • **Replacement Risk – High – Repair Recommended.**
2 Ave – 50 ST to 48 ST	S609	S610	300	VCT	171.84	0.20%	1962	• 1 sag length noted, 5.5m long. • Multiple encrustation and sediment (grease) deposits in line. • Multiple location along pipe of water infiltration (dripping). • Major encrustation in pipe at STA 23.5. Camera unable to pass. • **Replacement Risk – High – Cleaning Recommended – Attempt CCTV again.**
2 Avenue – 45 ST to 46 ST	S612	S612B	380	Conc.	172.52	0.20%	unknown	• Minor encrustation • Replacement Risk – Low.
	S612B	S613B	380	Conc.		0.20%	unknown	• Pipe tie-in defective at STA 32.5. Full of debris. • Pipe wall surface crack or fracture at STA 73.8. • Replacement Risk – Medium – Keep Monitoring
	S613B	S613	380	Conc.		0.20%	unknown	• Chip in joint at STA 11.1. • Major crack in pipe at STA 13.6.

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
								• Deposit and encrustation at pipe tie-in at STA 70.5. • Replacement Risk – Medium – Keep Monitoring.
2 Ave – 45 ST to 44 ST	S613	S661	380	Conc.	172.52	0.20%	unknown	• Multiple minor to medium deposits and encrustations. • Major deposits and encrustations at STA 59.1 and STA 148.7. Survey halted. • **Replacement Risk – High – Cleaning Recommended – Attempt CCTV again.**
48 ST – South of 4 Ave to 3 Ave.	S627A	S645	250	VCT	111.15	0.95%	1951	• Material change from PVC to Conc. STA 2.1. • Multipple deposits and encrustations throughout pipe length. • Multiple sags observed. Pipe grades uneven. • Replacement Risk – Medium – Keep Monitoring
48 ST – 3 Ave to 2 Ave	S645	S610	250	VCT	113.26	0.95%	1951	• Multiple material changes along pipe length: Conc. to VCT for 5.0m STA 54.2-59.2 Conc. to VCT at STA 104.4 VCT to PVC at STA 107.0 PVC to Conc. at STA 108.2 • 8 sag lengths noted, 2.7m to 7.0m long. • Service line extending into sewer main at STA. 66.0. • Multiple (14+) sediment (grease) deposits along length of pipe. • Multiple cracks along pipe. Major cracks at STA 108.4 and STA 109.2. • **Replacement Risk – High – Repair Recommended.**

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
South of 2nd Avenue – Between east of 42 ST and west of 41 ST	S656	S657	450	Conc.	121.31	0.16%	unknown	• Deposit and encrustation at placement holes in each pipe segment. • Multiple longitudinal cracks. • Cracks in and around major patch repair at STA 22.0. • Roots (fine) in joints, cracks and fractures • Water level increases from STA 25.5 onward until camera is submerged at STA 37.1. • Survey abandoned at STA 38.1. Unable to pass debris. • **Replacement Risk – High – Repair Recommended.**
HWY 16 – East of 43 ST to 40 ST	S659	S660	600	VCT	39.47 20.11	0.03% 0.77%	1977	• Minor deposit, grease and encrustation along length of pipe. • Replacement Risk – Low.
Sewer R/W South and East of CNR – 1 Ave and Golf Course Road.	S692	S693	900	Conc.	91.37	0.06%	1985	• Large deposit and encrustation at 0.3m (at MH S692. • Deposits – Settled Gravel at STA 3.4. Survey abandoned unable to pass debris. • **Replacement Risk – High – Cleaning Recommended – Attempt CCTV again.**
Golf Course Road – South of CNR and North of Golf Ave.	S709	S707	900	Conc.	149.14	0.10%	1985	• Medium deposit and encrustation at STA 27.6. • Multiple minor deposits and encrustations along pipe. • Intruding seal material at STA 37.5 (minor). • Replacement Risk – Low.
Along CNR – 38 ST east.	S721	S722	525	Conc.	90.96	0.16%	1949	• Multiple Medium and Fine Roots throughout length of the pipe. • Multiple deposits and encrustations.

Table C.5
Sanitary Sewer CCTV Inspection Summary

Municipal Servicing Plan Update
Town of Edson

Location	From MH	To MH	Line Size (mm)	Pipe Material	Length of Line (m)	Slope	Year Installed	Comments
								• Replacement Risk – Low.
CNR Crossing – West of 25 ST to East of 25 ST	S757	S769	675	Conc.	209.18	0.26%	1949	• Minor encrustation. • Pipe 20% full. • Replacement Risk – Low.
	S758	S769	525	Conc.	211.25	-	1949	• Minor encrustation. • Pipe 20% full. • Replacement Risk – Low.
	S759	S769	675	Conc.				• Pipe 50% full. • Replacement Risk – Low.

Appendix D
Storm Systems – Cost Estimates

Table D.1
Storm System Existing Cost Estimates

Municipal Servicing Plan Update
Town of Edson

Table 1-A

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
1-008	76.97	0.450	0.750	0.780	2.80	$1,750.00	$20,000.00	$154,697.50
1-009	92.36	0.600	0.900	3.570	2.80	$2,150.00	$20,000.00	$218,574.00
1-012	83.11	0.600	0.750	2.060	2.30	$1,750.00	$20,000.00	$165,442.50
1-014	87.20	0.450	0.600	0.500	2.16	$1,250.00		$109,000.00
1-015	65.00	0.675	0.900	2.385	3.20	$2,150.00		$139,750.00
1-016	65.00	0.675	0.900	3.170	2.35	$2,150.00		$139,750.00
1-017	47.50	0.675	0.900	3.170	2.00	$2,150.00		$102,125.00
						Sub-Total		$1,029,339.00
							Contingency (30%)	$308,801.70
							Engineering and Administration (10%)	$102,933.90
							Capital Cost	**$1,441,074.60**

Table 1-B

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
1-018	48.16	0.300	0.525	1.599	3.67	$1,060.00		$51,049.60
1-019	83.82	0.375	0.525	1.300	3.90	$1,060.00		$88,849.20
1-020	54.86	0.375	0.600	1.440	3.86	$1,250.00		$68,575.00
1-021	31.76	0.375	0.600	1.610	3.89	$1,250.00		$39,700.00
1-022	84.28	0.375	0.600	2.883	3.25	$1,250.00		$105,350.00
1-023	85.34	0.300	0.450	1.594	3.39	$900.00		$76,806.00
1-024	85.64	0.300	0.525	3.059	4.03	$1,060.00		$90,778.40
1-025	67.39	0.450	0.675	0.757	3.51	$1,500.00	$20,000.00	$121,085.00
1-026	66.51	0.450	0.750	0.571	3.58	$1,750.00	$20,000.00	$136,392.50
1-027	69.89	0.525	0.900	2.018	3.69	$2,150.00	$20,000.00	$170,263.50
1-028	72.91	0.525	0.900	1.783	3.21	$2,150.00	$20,000.00	$176,756.50
1-029	103.90	0.525	0.900	1.886	3.53	$2,150.00	$20,000.00	$243,385.00
1-031	57.42	0.600	0.900	1.602	3.45	$2,150.00	$20,000.00	$143,453.00
1-035	118.87	0.600	0.900	1.817	4.19	$2,150.00	$20,000.00	$275,570.50
1-058	152.40	0.900	1.200	1.900	3.10	$2,420.00	$35,000.00	$403,808.00
1-059	88.90	0.900	1.200	3.170	4.80	$2,420.00	$35,000.00	$250,138.00
1-060	104.55	1.050	1.350	1.450	4.80	$2,985.00	$35,000.00	$347,081.75
1-061	152.40	1.050	1.500	1.017	4.64	$3,650.00	$50,000.00	$606,260.00
1-062	60.05	1.050	1.500	1.080	3.62	$3,650.00	$50,000.00	$269,182.50
1-068	117.35	0.600	0.750	0.320	3.10	$1,750.00	$20,000.00	$225,362.50
1-069	97.53	0.525	0.900	0.174	4.75	$2,150.00	$20,000.00	$229,689.50
1-074	118.90	0.600	0.900	0.404	4.75	$2,150.00	$20,000.00	$275,635.00

Table D.1
Storm System Existing Cost Estimates

Municipal Servicing Plan Update
Town of Edson

1-075	59.74	0.750	0.900	0.180	2.86	$2,150.00		$128,441.00
1-076	68.32	0.750	0.900	0.270	2.84	$2,150.00		$146,888.00
1-077	33.83	0.750	1.050	0.070	4.41	$2,275.00	$35,000.00	$111,963.25
							Sub-Total	$4,782,463.70
							Contingency (30%)	$1,434,739.11
							Engineering and Administration (10%)	$478,246.37
							Capital Cost	**$6,695,449.18**

Table 1-C

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
1-044	89.92	0.300	0.450	2.875	2.33	$900.00		$80,928.00
1-045	95.27	0.300	0.450	2.470	2.33	$900.00		$85,743.00
1-047	63.00	0.375	0.525	0.170	3.45	$1,060.00		$66,780.00
1-046	47.67	0.450	0.525	0.830	2.66	$1,060.00		$50,530.20
1-048	92.66	0.450	0.750	1.061	3.13	$1,750.00	$20,000.00	$182,155.00
1-049	88.09	0.525	0.900	0.550	3.13	$2,150.00	$20,000.00	$209,393.50
1-050	93.57	0.525	0.900	0.556	3.13	$2,150.00	$20,000.00	$221,175.50
1-051	59.44	0.525	0.900	1.666	3.08	$2,150.00	$20,000.00	$147,796.00
1-052, 1-053 and 1-054 (Link374)	262.13	-	0.900	0.770	4.44	$2,150.00	$80,000.00	$643,579.50
1-055 (Link375)	149.35	-	1.200	0.375	5.97	$2,420.00	$70,000.00	$431,427.00
1-036	85.04	0.300	0.525	1.380	2.24	$1,060.00		$90,142.40
1-037	69.80	0.300	0.525	1.400	2.23	$1,060.00		$73,988.00
1-038	85.25	0.375	0.600	2.540	2.34	$1,250.00		$106,562.50
1-039	97.54	0.450	0.600	1.190	2.41	$1,250.00		$121,925.00
1-040	101.50	0.450	0.600	2.600	2.39	$1,250.00		$126,875.00
1-042	87.17	0.300	0.450	2.120	2.19	$900.00		$78,453.00
1-041	94.17	0.300	0.525	0.850	2.19	$1,060.00		$99,820.20
						Sub-Total		$2,817,273.80
						Contingency (30%)		$845,182.14
			Engineering and Administration (10%)					$281,727.38
						Capital Cost		**$3,944,183.32**

Table 1-D

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
1-078	154.73	0.300	0.450	0.500	2.82	$900.00		$139,257.00
1-079	169.98	0.300	0.600	1.500	5.34	$1,250.00		$212,475.00
1-080	102.11	0.300	0.600	2.977	5.35	$1,250.00		$127,637.50
1-081	111.86	0.375	0.750	2.289	2.39	$1,750.00	$20,000.00	$215,755.00
1-082	86.08	0.300	0.450	3.950	2.24	$900.00		$77,472.00

Table D.1
Storm System Existing Cost Estimates

Municipal Servicing Plan Update
Town of Edson

1-083	87.78	0.300	0.525	2.700	2.19	$1,060.00		$93,046.80
1-084	111.25	0.525	1.050	1.481	2.44	$2,275.00	$20,000.00	$273,093.75
1-085	79.74	0.300	0.450	3.900	2.71	$900.00		$71,766.00
1-086	90.00	0.300	0.450	3.400	2.30	$900.00		$81,000.00
1-088	80.92	0.300	0.450	3.250	1.74	$900.00		$72,828.00
1-089	89.05	0.300	0.450	4.256	2.70	$900.00		$80,145.00
1-087 (Link376)	112.48	0.525	1.050	1.078	2.79	$2,275.00	$20,000.00	$275,892.00
1-090 (Link377)	112.47	0.300	1.200	1.076	3.14	$2,420.00	$35,000.00	$307,177.40
1-093 and 1-094 (Link378)	127.92	0.600	1.200	1.079	3.53	$2,420.00	$35,000.00	$344,566.40
1-095	62.90	0.300	0.375	3.800	2.11	$720.00		$45,288.00
1-096	113.47	0.300	0.375	5.059	3.49	$720.00		$81,698.40
1-097	91.74	0.600	1.200	1.581	3.53	$1,250.00	$35,000.00	$149,675.00
1-098	62.79	0.750	1.350	0.844	1.99	$2,985.00	$35,000.00	$222,428.15
1-099	123.14	0.375	0.525	0.800	2.76	$1,060.00		$130,528.40
1-100	88.39	0.450	0.750	0.300	2.75	$1,750.00	$20,000.00	$174,682.50
1-101	111.56	0.450	0.900	0.296	2.71	$2,150.00	$20,000.00	$259,854.00
1-102 (Link379)	98.11	0.750	1.350	1.640	2.15	$2,985.00	$50,000.00	$342,858.35
1-103, 1-104 and 1-105 (Link380)	229.29	0.900	1.500	1.153	2.15	$3,650.00	$50,000.00	$886,908.50
1-106 and 1-107 (Link381)	156.44	1.050	1.500	1.152	2.26	$3,650.00	$50,000.00	$621,006.00
							Sub-Total	$5,287,039.15
							Contingency (30%)	$1,586,111.75
							Engineering and Administration (10%)	$528,703.92
							Capital Cost	**$7,401,854.81**

Table 1-E

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
1-108	82.30	0.300	0.525	0.377	1.75	$1,060.00		$87,238.00
1-109	81.67	0.300	0.525	1.150	2.84	$1,060.00		$86,570.20
1-110	86.26	0.375	0.525	0.410	2.84	$1,060.00		$91,435.60
1-111	87.17	0.375	0.675	0.350	2.07	$1,500.00	$20,000.00	$150,755.00
1-112	107.90	0.450	0.675	1.250	2.80	$1,500.00	$20,000.00	$181,850.00
						Sub-Total		$597,848.80
						Contingency (30%)		$179,354.64
					Engineering and Administration (10%)			$59,784.88
						Capital Cost		**$836,988.32**

Table 1-F

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
1-114	105.62	0.375	0.525	1.000	1.93	$1,060.00		$111,957.20

Table D.1
Storm System Existing Cost Estimates

Municipal Servicing Plan Update
Town of Edson

1-115	113.88	0.375	0.750	0.600	1.93	$1,750.00	$20,000.00	$219,290.00
1-116	124.35	0.600	0.900	0.520	1.91	$2,150.00	$20,000.00	$287,352.50
1-118	92.96	0.300	0.525	1.689	2.14	$1,060.00		$98,537.60
1-119	26.21	0.450	0.525	3.625	1.81	$1,060.00		$27,782.60
1-120	103.94	0.375	0.600	3.300	1.81	$1,250.00		$129,925.00
1-121	100.58	0.525	0.750	1.034	1.74	$1,750.00	$20,000.00	$196,015.00
1-122	130.45	0.675	0.900	0.424	1.86	$2,150.00		$280,467.50
1-123	126.80	0.375	0.600	0.560	1.98	$1,250.00		$158,500.00
1-124	178.92	0.525	0.900	0.326	2.12	$2,150.00	$20,000.00	$404,678.00
1-125	173.00	0.900	1.200	0.200	2.22	$2,420.00	$35,000.00	$453,660.00
1-126	252.05	1.200	1.350	0.200	2.60	$2,985.00	$50,000.00	$802,369.25
							Sub-Total	$3,170,534.65
							Contingency (30%)	$951,160.40
							Engineering and Administration (10%)	$317,053.47
							Capital Cost	**$4,438,748.51**

Table 1-G

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
2-001	123.05	0.450	0.600	0.690	3.60	$1,250.00		$153,812.50
2-002	124.99	0.450	0.600	1.396	3.62	$1,250.00		$156,237.50
2-003	89.28	0.600	0.675	0.868	2.31	$1,500.00	$20,000.00	$153,920.00
2-004	107.60	0.600	0.675	0.702	2.38	$1,500.00	$20,000.00	$181,400.00
2-005	64.54	0.900	1.050	0.330	3.10	$2,275.00	$35,000.00	$181,828.50
2-006	89.50	0.900	1.050	0.300	3.97	$2,275.00	$35,000.00	$238,612.50
2-007	62.50	0.900	1.200	0.360	4.55	$2,420.00	$35,000.00	$186,250.00
2-008	30.00	0.900	1.200	0.350	5.07	$2,420.00	$35,000.00	$107,600.00
2-009	97.50	0.900	1.050	0.277	5.07	$2,275.00	$35,000.00	$256,812.50
2-010	54.40	0.900	1.050	0.377	3.69	$2,275.00	$35,000.00	$158,760.00
2-011	22.31	0.900	1.200	1.260	2.89	$2,420.00	$35,000.00	$88,990.20
2-012	75.50	0.900	1.200	1.350	2.72	$2,420.00	$35,000.00	$217,710.00
C40 (Downstream from 2-012)	33.00	0.900	1.200	0.600	2.60	$2,420.00	$35,000.00	$114,860.00
							Sub-Total	$2,196,793.70
							Contingency (30%)	$659,038.11
							Engineering and Administration (10%)	$219,679.37
							Capital Cost	**$3,075,511.18**

Table 1-H

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)

Table D.1
Storm System Existing Cost Estimates

Municipal Servicing Plan Update
Town of Edson

2-015	94.00	0.525	0.600	1.350	2.25	$1,250.00		$117,500.00
						Sub-Total		$117,500.00
						Contingency (30%)		$35,250.00
					Engineering and Administration (10%)			$11,750.00
						Capital Cost		**$164,500.00**

Table 1-I

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
3-006	92.77	0.375	0.450	0.800	3.10	$900.00		$83,493.00
						Sub-Total		$83,493.00
						Contingency (30%)		$25,047.90
					Engineering and Administration (10%)			$8,349.30
						Capital Cost		**$116,890.20**

Table 1-J

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
3-008	89.36	0.375	0.525	0.996	2.45	$1,060.00		$94,721.60
3-009	114.60	0.375	0.750	1.001	2.79	$1,750.00	$20,000.00	$220,550.00
3-010	88.39	0.525	0.750	0.900	2.58	$1,750.00	$20,000.00	$174,682.50
3-011	81.00	0.525	0.750	0.900	3.38	$1,750.00	$20,000.00	$161,750.00
3-012	95.46	0.375	0.525	0.995	2.91	$1,060.00	$20,000.00	$121,187.60
3-013	101.19	0.450	0.900	0.860	2.91	$2,150.00	$20,000.00	$237,558.50
3-014	74.98	0.450	0.900	1.280	2.96	$2,150.00	$20,000.00	$181,207.00
3-015	113.84	0.300	0.450	0.720	2.27	$900.00		$102,456.00
3-016	62.48	0.450	0.900	1.793	2.41	$2,150.00	$20,000.00	$154,332.00
						Sub-Total		$1,448,445.20
						Contingency (30%)		$434,533.56
					Engineering and Administration (10%)			$144,844.52
						Capital Cost		**$2,027,823.28**

Table 1-K

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
3-017	86.26	0.450	0.525	1.906	3.28	$1,060.00		$91,435.60
3-018	85.32	0.450	0.525	1.868	3.28	$1,060.00		$90,439.20
3-019	86.87	0.450	0.600	1.530	2.30	$1,250.00		$108,587.50
3-020	86.87	0.450	0.750	1.000	2.72	$1,750.00	$20,000.00	$172,022.50
3-021	99.97	0.450	0.750	1.070	2.79	$1,750.00	$20,000.00	$194,947.50
3-022	23.99	0.450	0.750	1.000	2.33	$1,750.00	$20,000.00	$61,982.50
							Sub-Total	$719,414.80

Table D.1
Storm System Existing Cost Estimates

Municipal Servicing Plan Update
Town of Edson

							Contingency (30%)	$215,824.44
							Engineering and Administration (10%)	$71,941.48
							Capital Cost	**$1,007,180.72**

Table 1-L

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
4-001	76.00	0.375	0.450	0.712	2.36	$900.00		$68,400.00
4-002	72.77	0.375	0.450	1.060	2.52	$900.00		$65,493.00
4-003	62.84	0.375	0.525	1.500	2.65	$1,060.00		$66,610.40
4-004	69.49	0.375	0.525	2.724	2.67	$1,060.00		$73,659.40
4-005	49.99	0.375	0.525	3.231	2.67	$1,060.00		$52,989.40
4-006	80.16	0.525	0.675	0.749	2.69	$1,500.00	$20,000.00	$140,240.00
4-007	87.38	0.600	0.750	1.280	2.46	$1,750.00	$20,000.00	$172,915.00
4-008	89.92	0.600	0.750	1.730	2.53	$1,750.00	$20,000.00	$177,360.00
							Sub-Total	$817,667.20
							Contingency (30%)	$245,300.16
							Engineering and Administration (10%)	$81,766.72
							Capital Cost	**$1,144,734.08**

Table 1-M

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
4-011	47.83	0.300	0.450	0.320	2.43	$900.00		$43,047.00
4-012	100.74	0.375	0.525	0.782	2.53	$1,060.00		$106,784.40
4-013	106.38	0.375	0.525	1.239	3.08	$1,060.00		$112,762.80
4-014	81.99	0.450	0.600	0.922	3.08	$1,250.00		$102,487.50
							Sub-Total	$365,081.70
							Contingency (30%)	$109,524.51
							Engineering and Administration (10%)	$36,508.17
							Capital Cost	**$511,114.38**

Table 1-N

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
4-016	107.90	0.300	0.450	1.168	2.18	$900.00		$97,110.00
4-017	81.99	0.375	0.525	1.769	2.19	$1,060.00		$86,909.40
4-018	75.90	0.375	0.750	1.858	2.02	$1,750.00	$20,000.00	$152,825.00
4-019	31.09	0.375	0.750	1.351	2.05	$1,750.00	$20,000.00	$74,407.50
						Sub-Total		$411,251.90
						Contingency (30%)		$123,375.57
				Engineering and Administration (10%)				$41,125.19

Table D.1
Storm System Existing Cost Estimates

Municipal Servicing Plan Update
Town of Edson

						Capital Cost		**$575,752.66**

Table 1-O

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
4-020	138.68	0.300	0.525	1.642	2.20	$1,060.00		$147,000.80
C29	102.50	0.300	0.450	0.300	2.50	$900.00		$92,250.00
						Sub-Total		$239,250.80
						Contingency (30%)		$71,775.24
			Engineering and Administration (10%)					$23,925.08
						Capital Cost		**$334,951.12**

Table 1-P

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
4-021	57.30	0.300	0.450	0.650	1.80	$900.00		$51,570.00
4-022	42.50	0.300	0.600	0.400	2.21	$1,250.00		$53,125.00
4-023	64.31	0.300	0.450	0.580	2.20	$900.00		$57,879.00
4-024	53.34	0.300	0.600	0.500	2.54	$1,250.00		$66,675.00
4-025	99.82	0.300	0.450	0.580	2.70	$900.00		$89,838.00
4-026	82.90	0.375	0.750	0.300	2.98	$1,750.00	$20,000.00	$165,075.00
4-027	80.00	0.375	0.750	0.450	3.22	$1,750.00	$20,000.00	$160,000.00
4-028	52.38	0.450	0.750	0.420	3.28	$1,750.00	$20,000.00	$111,665.00
4-029	76.74	0.450	0.900	0.430	3.19	$2,150.00	$20,000.00	$184,991.00
4-030	35.36	0.450	0.900	1.300	2.94	$2,150.00	$20,000.00	$96,024.00
						Sub-Total		$1,036,842.00
						Contingency (30%)		$311,052.60
			Engineering and Administration (10%)					$103,684.20
						Capital Cost		**$1,451,578.80**

Table 1-Q

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
4-032	77.72	0.300	0.525	0.978	1.84	$1,060.00		$82,383.20
4-033	79.25	0.300	0.450	4.479	1.86	$900.00		$71,325.00
4-034	111.56	0.300	0.525	1.694	1.89	$1,060.00		$118,253.60
4-035	38.10	0.300	0.450	9.869	2.45	$900.00		$34,290.00
						Sub-Total		$306,251.80
						Contingency (30%)		$91,875.54
			Engineering and Administration (10%)					$30,625.18
						Capital Cost		**$428,752.52**

Table D.1
Storm System Existing Cost Estimates

Municipal Servicing Plan Update
Town of Edson

Table 1-R

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
4-036	87.10	0.300	0.600	0.485	3.20	$1,250.00		$108,875.00
4-037	90.22	0.300	0.600	1.053	3.20	$1,250.00		$112,775.00
4-038	87.78	0.300	0.600	2.256	2.16	$1,250.00		$109,725.00
4-039_1	21.50	0.300	0.600	6.000	1.66	$1,250.00		$26,875.00
4-039_2	39.90	0.450	0.750	1.000	2.00	$1,750.00	$20,000.00	$89,825.00
						Sub-Total		$448,075.00
						Contingency (30%)		$134,422.50
					Engineering and Administration (10%)			$44,807.50
						Capital Cost		**$627,305.00**

Table 1-S

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
4-040	45.68	0.300	0.450	3.260	2.20	$900.00		$41,112.00
4-041	103.02	0.250	0.600	0.400	2.06	$1,250.00		$128,775.00
4-042	50.08	0.450	0.525	3.000	2.02	$1,060.00		$53,084.80
4-043	60.35	0.450	0.750	0.878	2.02	$1,750.00	$20,000.00	$125,612.50
4-044	51.21	0.450	0.750	1.035	1.77	$1,750.00	$20,000.00	$109,617.50
4-045	115.82	0.600	1.050	0.449	1.51	$2,275.00	$35,000.00	$298,490.50
4-046	170.38	0.675	1.200	0.305	1.65	$2,420.00	$35,000.00	$447,319.60
						Sub-Total		$1,204,011.90
						Contingency (30%)		$361,203.57
					Engineering and Administration (10%)			$120,401.19
						Capital Cost		**$1,685,616.66**

Table 1-T

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
4-048	95.45	0.300	0.450	3.100	2.16	$900.00		$85,905.00
4-049	26.52	0.375	0.450	2.000	2.15	$900.00		$23,868.00
4-050	13.44	0.450	0.525	2.976	2.15	$1,060.00		$14,246.40
4-051	84.73	0.450	0.600	2.254	2.10	$1,250.00		$105,912.50
4-052	95.60	0.450	0.675	2.500	2.60	$1,500.00	$20,000.00	$163,400.00
C24 (Downstream from 4-052)	110.80	0.900	1.050	0.260	2.60	$2,275.00	$35,000.00	$287,070.00
						Sub-Total		$680,401.90
						Contingency (30%)		$204,120.57
					Engineering and Administration (10%)			$68,040.19
						Capital Cost		**$952,562.66**

Table D.1
Storm System Existing Cost Estimates

Municipal Servicing Plan Update
Town of Edson

Table 1-U

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
4-054	114.40	0.300	0.450	2.500	1.94	$900.00		$102,960.00
4-055	94.57	0.300	0.450	3.500	1.87	$900.00		$85,113.00
4-060	124.36	0.900	1.050	0.540	1.90	$2,275.00	$35,000.00	$317,919.00
						Sub-Total		$505,992.00
						Contingency (30%)		$151,797.60
				Engineering and Administration (10%)				$50,599.20
						Capital Cost		**$708,388.80**

Table 1-V

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
4-061	98.45	0.300	0.525	2.300	2.18	$1,060.00		$104,357.00
4-063a	127.65	0.600	0.675	1.214	3.70	$1,500.00	$20,000.00	$211,475.00
4-064	131.06	0.600	0.900	0.900	2.00	$2,150.00	$20,000.00	$301,779.00
						Sub-Total		$617,611.00
						Contingency (30%)		$185,283.30
				Engineering and Administration (10%)				$61,761.10
						Capital Cost		**$864,655.40**

Table 1-W

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
5-001	47.55	0.300	0.525	0.379	1.10	$1,060.00		$50,403.00
5-002	42.67	0.300	0.525	0.563	1.34	$1,060.00		$45,230.20
5-003	76.04	0.375	0.525	1.170	1.76	$1,060.00		$80,602.40
5-004	30.32	0.450	0.525	1.850	1.76	$1,060.00		$32,139.20
						Sub-Total		$208,374.80
						Contingency (30%)		$62,512.44
				Engineering and Administration (10%)				$20,837.48
						Capital Cost		**$291,724.72**

Table 1-X

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
5-005	38.71	0.300	0.450	0.400	2.41	$900.00		$34,839.00
5-006	53.57	0.300	0.450	0.560	2.24	$900.00		$48,213.00
5-007	117.65	0.300	0.600	0.700	3.22	$1,250.00		$147,062.50
5-008	81.38	0.375	0.600	1.450	3.51	$1,250.00		$101,725.00
5-009	81.02	0.375	0.600	2.950	2.99	$1,250.00		$101,275.00
5-010	29.96	0.375	0.600	2.270	3.36	$1,250.00		$37,450.00

Table D.1
Storm System Existing Cost Estimates

Municipal Servicing Plan Update
Town of Edson

						Sub-Total		$470,564.50
						Contingency (30%)		$141,169.35
					Engineering and Administration (10%)			$47,056.45
						Capital Cost		**$658,790.30**

Table 1-Y

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
5-013	111.40	0.300	0.450	4.000	1.95	$900.00		$100,260.00
5-014	101.11	0.300	0.600	1.800	1.95	$1,250.00		$126,387.50
5-015	25.91	0.600	0.900	0.500	1.00	$2,150.00	$20,000.00	$75,706.50
							Sub-Total	$302,354.00
							Contingency (30%)	$90,706.20
					Engineering and Administration (10%)			$30,235.40
						Capital Cost		**$423,295.60**

Table 1-Z

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
5-017	112.78	0.300	0.450	1.800	2.58	$900.00		$101,502.00
5-018	110.00	0.300	0.600	2.100	2.64	$1,250.00		$137,500.00
5-019	89.60	0.375	0.900	1.250	2.71	$2,150.00	$20,000.00	$212,640.00
5-020	91.77	0.375	0.750	1.700	2.21	$1,750.00	$20,000.00	$180,597.50
5-021	107.84	0.300	0.450	2.100	2.80	$900.00		$97,056.00
5-022	110.50	0.450	0.750	3.810	2.79	$1,750.00	$20,000.00	$213,375.00
						Sub-Total		$942,670.50
						Contingency (30%)		$282,801.15
					Engineering and Administration (10%)			$94,267.05
						Capital Cost		**$1,319,738.70**

Table 1-AA

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
6-001	72.78	0.375	0.450	2.968	2.70	$900.00		$65,502.00
6-002	115.53	0.375	0.525	1.420	2.75	$1,060.00		$122,461.80
6-003	48.50	0.375	0.525	2.000	2.75	$1,060.00		$51,410.00
6-004	37.00	0.525	0.675	0.649	2.82	$1,500.00	$20,000.00	$75,500.00
6-005	53.00	0.525	0.675	0.800	2.82	$1,500.00	$20,000.00	$99,500.00
6-006	53.00	0.525	0.675	1.377	2.50	$1,500.00	$20,000.00	$99,500.00
6-010	85.45	0.300	0.450	0.351	2.50	$900.00		$76,905.00
6-012	132.45	0.600	0.675	0.640	2.40	$1,500.00	$20,000.00	$218,675.00
6-013	67.05	0.600	0.675	1.060	2.40	$1,500.00	$20,000.00	$120,575.00

Table D.1
Storm System Existing Cost Estimates

Municipal Servicing Plan Update
Town of Edson

C8	83.09	0.600	0.675	0.390	2.90	$1,500.00	$20,000.00	$144,635.00
C15	19.38	0.375	0.675	0.460	2.50	$1,500.00	$20,000.00	$49,070.00
C14	109.70	0.375	0.675	0.100	2.50	$1,500.00	$20,000.00	$184,550.00
C16	91.28	0.675	0.675	0.210	2.50	$1,500.00		$136,920.00
C11	83.79	0.375	0.450	1.580	2.50	$900.00		$75,411.00
C12	146.56	0.375	0.600	0.530	2.50	$1,250.00		$183,205.00
C13	66.41	0.375	0.600	0.390	2.50	$1,250.00		$83,012.50
C18	55.59	0.375	0.450	0.430	2.50	$900.00		$50,031.00
C19	15.35	0.375	0.450	0.440	2.50	$900.00		$13,815.00
						Sub-Total		$1,850,678.30
						Contingency (30%)		$555,203.49
			Engineering and Administration (10%)					$185,067.83
						Capital Cost		**$2,590,949.62**

Table 1-CC

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
6-021	64.78	0.300	0.450	2.300	2.16	$900.00		$58,302.00
6-022	47.27	0.300	0.450	2.200	2.04	$900.00		$42,543.00
6-023	53.44	0.300	0.525	3.200	2.34	$1,060.00		$56,646.40
6-024	87.50	0.375	0.525	4.000	2.47	$1,060.00		$92,750.00
						Sub-Total		$250,241.40
						Contingency (30%)		$75,072.42
			Engineering and Administration (10%)					$25,024.14
						Capital Cost		**$350,337.96**

Table 1-DD

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Unit Cost ($/m)	Manhole Installation (Lump Sum)	Total Cost ($)
6-027	56.39	0.375	0.450	0.750	2.29	$900.00		$50,751.00
						Sub-Total		$50,751.00
						Contingency (30%)		$15,225.30
			Engineering and Administration (10%)					$5,075.10
						Capital Cost		**$71,051.40**
						TOTAL CAPITAL COSTS		**$46,141,504.50**

Table D.2
Cost Estimates for Future (2032) Storm Servicing Plan (Based on Future Areas to be Developed)

Municipal Servicing Plan Update
Town of Edson

Storm Area G

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000.00	1.00	$75,000
2	Clearing and Grubbing	ha	$10,000.00	2.14	$21,400
3	Topsoil Stripping	cu. m.	$6.10	7100.00	$43,310
4	Excavation and Disposal	cu. m.	$18.25	59700.00	$1,089,525
5	Landscaping	ha	$34,400.00	1.00	$34,400
6	Outlet Pipe (525 mm diameter)	m	$1,060.00	100.00	$106,000
7	Control Structure	each	$60,000.00	1.00	$60,000
				Sub-Total	$1,429,635
				30% Contingency	$428,891
				10% Engineering and Overhead	$142,964
				Capital Cost (2017 $)	**$2,001,489**

Storm Area I

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	2.39	$23,900
3	Topsoil Stripping	cu. m.	$6.10	8000	$48,800
4	Excavation and Disposal	cu. m.	$18.25	69200	$1,262,900
5	Landscaping	ha	$34,400	1.00	$34,400
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,611,000
				30% Contingency	$483,300
				10% Engineering and Overhead	$161,100
				Capital Cost (2017 $)	**$2,255,400**

Storm Area J

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	3.42	$34,200
3	Topsoil Stripping	cu. m.	$6.10	11400	$69,540
4	Excavation and Disposal	cu. m.	$18.25	108700	$1,983,775
5	Landscaping	ha	$34,400	1.25	$43,000
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$2,371,515
				30% Contingency	$711,455
				10% Engineering and Overhead	$237,152
				Capital Cost (2017 $)	**$3,320,121**

Storm Area K

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	2.13	$21,300
3	Topsoil Stripping	cu. m.	$6.10	7100	$43,310
4	Excavation and Disposal	cu. m.	$18.25	59500	$1,085,875
5	Landscaping	ha	$34,400	1	$34,400
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,425,885
				30% Contingency	$427,766
				10% Engineering and Overhead	$142,589
				Capital Cost (2017 $)	**$1,996,239**

Storm Area N

Item	Description	Unit	Unit Price	Estimated Quantity	Amount

Table D.2
Cost Estimates for Future (2032) Storm Servicing Plan (Based on Future Areas to be Developed)

Municipal Servicing Plan Update
Town of Edson

1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	2.31	$23,100
3	Topsoil Stripping	cu. m.	$6.10	7700	$46,970
4	Excavation and Disposal	cu. m.	$18.25	66100	$1,206,325
5	Landscaping	ha	$34,400	1	$34,400
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,551,795
				30% Contingency	$465,539
				10% Engineering and Overhead	$155,180
				Capital Cost (2017 $)	**$2,172,513**

Storm Area U

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	0.62	$6,200
3	Topsoil Stripping	cu. m.	$6.10	2100	$12,810
4	Excavation and Disposal	cu. m.	$18.25	10500	$191,625
5	Landscaping	ha	$34,400	0.45	$15,480
6	Outlet Pipe (525 mm diameter)	m	$1,060	50	$53,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$414,115
				30% Contingency	$124,235
				10% Engineering and Overhead	$41,412
				Capital Cost (2017 $)	**$579,761**

Storm Area V

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	0.91	$9,100
3	Topsoil Stripping	cu. m.	$6.10	3,000.00	$18,300
4	Excavation and Disposal	cu. m.	$18.25	18,600.00	$339,450
5	Landscaping	ha	$34,400	0.57	$19,608
6	Outlet Pipe (525 mm diameter)	m	$1,060	50	$53,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$574,458
				30% Contingency	$172,337
				10% Engineering and Overhead	$57,446
				Capital Cost (2017 $)	**$804,241**

Table D.2
Cost Estimates for Future (2032) Storm Servicing Plan (Based on Future Areas to be Developed)

Municipal Servicing Plan Update
Town of Edson

Storm Area V-s

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	1.15	$11,500
3	Topsoil Stripping	cu. m.	$6.10	3800	$23,180
4	Excavation and Disposal	cu. m.	$18.25	33000	$602,250
5	Landscaping	ha	$34,400	0.67	$23,048
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$900,978
				30% Contingency	$270,293
				10% Engineering and Overhead	$90,098
				Capital Cost (2017 $)	**$1,261,369**

Storm Area W

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	1.17	$11,700
3	Topsoil Stripping	cu. m.	$6.10	3900	$23,790
4	Excavation and Disposal	cu. m.	$18.25	26600	$485,450
5	Landscaping	ha	$34,400	0.68	$23,392
6	Outlet Pipe (525 mm diameter)	m	$1,060	50	$53,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$732,332
				30% Contingency	$219,700
				10% Engineering and Overhead	$73,233
				Capital Cost (2017 $)	**$1,025,265**

Storm Area Y

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	3.41	$34,100
3	Topsoil Stripping	cu. m.	$6.10	11300	$68,930
4	Excavation and Disposal	cu. m.	$18.25	108300	$1,976,475
5	Landscaping	ha	$34,400	1.3	$44,720
6	Outlet Pipe (600 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$2,365,225
				30% Contingency	$709,568
				10% Engineering and Overhead	$236,523
				Capital Cost (2017 $)	**$3,311,315**
	Total 2032 Commercial / Industrial Development Capital Cost:				**$18,727,713**

Storm Area (Portion of) P

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	0.6	$6,000
3	Topsoil Stripping	cu. m.	$6.10	2000	$12,200
4	Excavation and Disposal	cu. m.	$18.25	10000	$182,500
5	Landscaping	ha	$34,400	0.43	$14,792
6	Outlet Pipe (525 mm diameter)	m	$1,060	50	$53,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$403,492
				30% Contingency	$121,048
				10% Engineering and Overhead	$40,349
				Capital Cost (2017 $)	**$564,889**

Storm Area M-g

Table D.2

Cost Estimates for Future (2032) Storm Servicing Plan (Based on Future Areas to be Developed)

Municipal Servicing Plan Update

Town of Edson

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	1.32	$13,200
3	Topsoil Stripping	cu. m.	$6.10	4400	$26,840
4	Excavation and Disposal	cu. m.	$18.25	19200	$350,400
5	Landscaping	ha	$34,400	0.5	$17,200
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$648,640
				30% Contingency	$194,592
				10% Engineering and Overhead	$64,864
				Capital Cost (2017 $)	**$908,096**
				Total 2032 Residential Development Capital Cost:	**$1,472,985**
				Total 2032 Development Capital Costs:	**$20,200,698**

Table D.3

Cost Estimates for Future (2042) Storm Servicing Plan (Based on Future Areas to be Developed)

Municipal Servicing Plan Update

Town of Edson

Storm Area A

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	2.37	$23,700
3	Topsoil Stripping	cu. m.	$6.10	7900	$48,190
4	Excavation and Disposal	cu. m.	$18.25	68300	$1,246,475
5	Landscaping	ha	$34,400	1.1	$37,840
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,597,205
				30% Contingency	$479,162
				10% Engineering and Overhead	$159,721
				Capital Cost (2017 $)	**$2,236,087**

Storm Area B

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	2.08	$20,800
3	Topsoil Stripping	cu. m.	$6.10	6900	$42,090
4	Excavation and Disposal	cu. m.	$18.25	57800	$1,054,850
5	Landscaping	ha	$34,400	1	$34,400
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,393,140
				30% Contingency	$417,942
				10% Engineering and Overhead	$139,314
				Capital Cost (2017 $)	**$1,950,396**

Storm Area C

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	2.47	$24,700
3	Topsoil Stripping	cu. m.	$6.10	8200	$50,020
4	Excavation and Disposal	cu. m.	$18.25	72200	$1,317,650
5	Landscaping	ha	$34,400	1.09	$37,496
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,670,866
				30% Contingency	$501,260
				10% Engineering and Overhead	$167,087
				Capital Cost (2017 $)	**$2,339,212**

Storm Area D

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	2.35	$23,500
3	Topsoil Stripping	cu. m.	$6.10	7800	$47,580
4	Excavation and Disposal	cu. m.	$18.25	67500	$1,231,875
5	Landscaping	ha	$34,400	1.05	$36,120
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,580,075
				30% Contingency	$474,023
				10% Engineering and Overhead	$158,008
				Capital Cost (2017 $)	**$2,212,105**

Table D.3
Cost Estimates for Future (2042) Storm Servicing Plan (Based on Future Areas to be Developed)

Municipal Servicing Plan Update
Town of Edson

Storm Area E

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	2.26	$22,600
3	Topsoil Stripping	cu. m.	$6.10	7500	$45,750
4	Excavation and Disposal	cu. m.	$18.25	64400	$1,175,300
5	Landscaping	ha	$34,400	1.03	$35,432
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,520,082
				30% Contingency	$456,025
				10% Engineering and Overhead	$152,008
				Capital Cost (2017 $)	**$2,128,115**

Storm Area F

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	2.83	$28,300
3	Topsoil Stripping	cu. m.	$6.10	9400	$57,340
4	Excavation and Disposal	cu. m.	$18.25	85800	$1,565,850
5	Landscaping	ha	$34,400	1.18	$40,592
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,933,082
				30% Contingency	$579,925
				10% Engineering and Overhead	$193,308
				Capital Cost (2017 $)	**$2,706,315**

Storm Area H

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	2.77	$27,700
3	Topsoil Stripping	cu. m.	$6.10	9200	$56,120
4	Excavation and Disposal	cu. m.	$18.25	83300	$1,520,225
5	Landscaping	ha	$34,400	1.17	$40,248
6	Outlet Pipe (600 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,885,293
				30% Contingency	$565,588
				10% Engineering and Overhead	$188,529
				Capital Cost (2017 $)	**$2,639,410**

Storm Area L

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	2.85	$28,500
3	Topsoil Stripping	cu. m.	$6.10	9500	$57,950
4	Excavation and Disposal	cu. m.	$18.25	86500	$1,578,625
5	Landscaping	ha	$34,400	1.2	$41,280
6	Outlet Pipe (600 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,947,355
				30% Contingency	$584,207
				10% Engineering and Overhead	$194,736
				Capital Cost (2017 $)	**$2,726,297**

Table D.3
Cost Estimates for Future (2042) Storm Servicing Plan (Based on Future Areas to be Developed)

Municipal Servicing Plan Update
Town of Edson

Storm Area M

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	0.85	$8,500
3	Topsoil Stripping	cu. m.	$6.10	2800	$17,080
4	Excavation and Disposal	cu. m.	$18.25	16800	$306,600
5	Landscaping	ha	$34,400	0.5	$17,200
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$590,380
				30% Contingency	$177,114
				10% Engineering and Overhead	$59,038
				Capital Cost (2017 $)	**$826,532**

Storm Area O

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	1.58	$15,800
3	Topsoil Stripping	cu. m.	$6.10	5300	$32,330
4	Excavation and Disposal	cu. m.	$18.25	40200	$733,650
5	Landscaping	ha	$34,400	0.82	$28,208
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$1,050,988
				30% Contingency	$315,296
				10% Engineering and Overhead	$105,099
				Capital Cost (2017 $)	**$1,471,383**

Storm Area Q

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	5.84	$58,400
3	Topsoil Stripping	cu. m.	$6.10	19500	$118,950
4	Excavation and Disposal	cu. m.	$18.25	166608	$3,040,596
5	Landscaping	ha	$34,400	2.31	$79,464
6	Outlet Pipe (525 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$3,538,410
				30% Contingency	$1,061,523
				10% Engineering and Overhead	$353,841
				Capital Cost (2017 $)	**$4,953,774**

Storm Area X

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	3.23	$32,300
3	Topsoil Stripping	cu. m.	$6.10	10700	$65,270
4	Excavation and Disposal	cu. m.	$18.25	101200	$1,846,900
5	Landscaping	ha	$34,400	1.28	$44,032
6	Outlet Pipe (600 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$2,229,502
				30% Contingency	$668,851
				10% Engineering and Overhead	$222,950
				Capital Cost (2017 $)	**$3,121,303**

Table D.3
Cost Estimates for Future (2042) Storm Servicing Plan (Based on Future Areas to be Developed)

Municipal Servicing Plan Update
Town of Edson

Storm Area Y

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	3.41	$34,100
3	Topsoil Stripping	cu. m.	$6.10	11300	$68,930
4	Excavation and Disposal	cu. m.	$18.25	108300	$1,976,475
5	Landscaping	ha	$34,400	1.32	$45,408
6	Outlet Pipe (600 mm diameter)	m	$1,060	100	$106,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$2,365,913
				30% Contingency	$709,774
				10% Engineering and Overhead	$236,591
				Capital Cost (2017 $)	**$3,312,278**
				Total 2042 Commercial / Industrial Development Capital Cost:	**$32,623,207**

Storm Area R

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	0.48	$4,800
3	Topsoil Stripping	cu. m.	$6.10	1600	$9,760
4	Excavation and Disposal	cu. m.	$18.25	6900	$125,925
5	Landscaping	ha	$34,400	0.26	$8,944
6	Outlet Pipe (525 mm diameter)	m	$1,060	50	$53,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$337,429
				30% Contingency	$101,229
				10% Engineering and Overhead	$33,743
				Capital Cost (2017 $)	**$472,401**

Storm Area S

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	0.96	$9,600
3	Topsoil Stripping	cu. m.	$6.10	3200	$19,520
4	Excavation and Disposal	cu. m.	$18.25	18800	$343,100
5	Landscaping	ha	$34,400	0.4	$13,760
6	Outlet Pipe (600 mm diameter)	m	$1,060	50	$53,000
7	Control Structure	each	$60,000	1	$60,000
				Sub-Total	$573,980
				30% Contingency	$172,194
				10% Engineering and Overhead	$57,398
				Capital Cost (2017 $)	**$803,572**

Table D.3
Cost Estimates for Future (2042) Storm Servicing Plan (Based on Future Areas to be Developed)

Municipal Servicing Plan Update
Town of Edson

Storm Area T

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization and Demobilization (5% approx.)	Lump Sum	$75,000	1	$75,000
2	Clearing and Grubbing	ha	$10,000	0.71	$7,100
3	Topsoil Stripping	cu. m.	$6.10	2400	$14,640
4	Excavation and Disposal	cu. m.	$18.25	12200	$222,650
5	Landscaping	ha	$34,400	0.33	$11,352
6	Outlet Pipe (600 mm diameter)	m	$1,060	50	$53,000
7	Control Structure	each	$60,000	1	$60,000
			Sub-Total		$443,742
			30% Contingency		$133,123
	10% Engineering and Overhead				$44,374
			Capital Cost (2017 $)		**$621,239**
	Total 2042 Residential Development Capital Cost:				**$1,897,211**
	Total 2042 Development Capital Costs:				**$34,520,419**

Appendix E
Stormwater Drainage System Inspection

Town of Edson – Stormwater Drainage System Inspection

August 23-25, 2017

By Stephen Briggs

Area 4

Outfall 2-012 • Concrete 900mm. • Water filling half outlet • About ¼ full mud/sand and debris • Grate holding some small debris • Draining South.		
MH ST13 • Manhole in gravel road. • Lots of dirt and mud in manhole. • Iron frame ring and cover ok • Concrete collar rings ok		

MH ST12

- Manhole in grass field.
- Iron frame and cover offset from concrete collar rings
- Concrete collar ok.
- Small-moderate amount of dirt/mud in bottom of manhole.
- Drains to the south.
- Concrete ok

MH ST1

- Iron frame ring and cover ok.
- Bricks used as grade ring on top of concrete collar.
- Large CB lead/pipe (~900mm) from north, with drop.
- Flows to south

MH ST3

- Iron frame and cover ok.
- Bricks used as grade ring on top of concrete collar. Breaking/crumbling.
- Concrete barrel ok.
- 900mm pipes east & west.
- Flowing to west.

<table>
<tr>
<td><u>Culvert North of ST2 crossing 3rd Ave</u>
500mm CSP culvert
South side half full of mud/debris
South side overgrown grass
North side 500mm CSP clear</td>
<td></td>
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<tr>
<td><u>Culvert crossing HWY 16 (4th Ave)</u>
South side half full water. P
CSP
Drain into ditch blocked with dirt.</td>
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<tr>
<td><u>Culvert crossing 5th Ave/63 St turn off</u>
500mm CSP
East side half full dirt and cracked.
West clear, slight grass overgrown.</td>
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</tr>
</table>

MH ST28

- Iron frame and cover ok.
- Concrete collar ok.
- NE & NW CB leads
- Drains to South.
- NE CB collar has wood and rough poured concrete incorporated.
- Water in bottom of NE CB.

Area 16

STM Outfall 1-117

- 750mm flared end.
- Flared end separated from concrete pipe.
- Concrete pipe eroded around, riprap missing/displaced.
- Some dirt and gravel present in Conc pipe.
- Metal straps attaching flared end to pipe detached.

<table>
<tr>
<td><u>STM Outfall 1-117</u></td>
<td></td>
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</tr>
<tr>
<td><u>3250mm CSP culvert under 2nd Ave</u>

Connecting north and south storm ponds.
</td>
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<tr>
<td><u>MH ST264</u>

Iron frame & grate loose from concrete collar. Offset.
Concrete barrel/collars ok
East and west pipes, not visible. Conc base/benching covering pipes.
Flowing to west.
North PVC CB lead (~300mm) flowing into MH.
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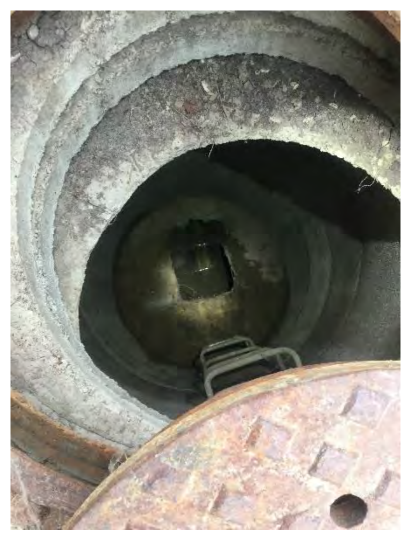
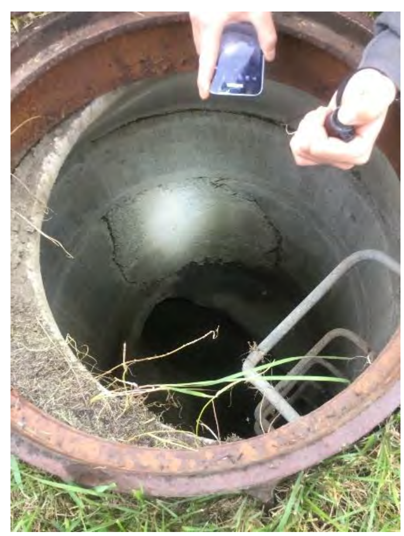
<td></td>
<td></td>
</tr>
</table>

<u>Outfall 3-016</u> • CSP 500mm, Ok. • Flowing SE into storm pond. • Conc CB lead from CB on 2^{nd} Ave east of outfall, ok.		
<u>Outfall 3-022</u> • Overgrown. • Concrete flared end. • Difficult to see/access.		
<u>MH ST254</u> • MH in green field. • Iron frame & cover ok. Loose and offset from collar. • Pipes west and S.E. • Flows to S.E. • MH benched conc. • Concrete patch in side of barrel. • Small dirt/debris in benching.		

Outfall 1-127

- Concrete 1200mm, ok.
- Concrete frame/retaining wall around outfall, ok.
- Include rebar frame/grate, ok.
- Flowing west into pond.

MH ST255

- Iron frame & grate, ok.
- Concrete collar, ok.
- No CB leads visible from top.
- Flowing West to Outfall 1-127.
- Clean
- One ladder ring, with concrete bulge below.

Area 17

Outfall 4-046

- Concrete
- Overgrown
- half full of water
- Flow east into ditch

<u>Culvert north of Outfall 4-046 east side of 47th St</u> • 1200mm CSP • Clean, slightly flattened • Flowing south into ditch		
<u>Culvert East side of 47th St crossing 2nd Ave</u> • ~1200mm CSP • Water pooling in culvert. Half full. • Draining to South. • North side, slightly overgrown. Draining south.		
<u>MH ST294</u> • Manhole in dirt road/drive • Iron frame and grate. • East and south inverts. • Flows to the south, towards outfall. • Barrel and collar ok • Ground water infiltrating into MH barrel. • Pipes and benching half full water • Lots of sediment in pipes/benching		

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<tr><td>

<u>Outfall 4-060 (from MH ST318)</u>

- Concrete 900mm, ok
- ¼ full of mud/debris
- Flowing south into ditch

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<u>Outfall from MH ST294</u>

- Concrete 900mm, ok.
- Almost half full water.
- Lots of mud inside pipe

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<u>Outfall 4-064</u>

- Not found.
- Possibly buried
- Pipes upstream are full of dark/dirty water, possible because outfall is blocked or buried.

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</table>

<u>MH ST324A (MH not shown on plans)</u>

- MH in dirt road/drive
- Iron frame and cover, ok.
- Concrete collar, ok
- North and South pipes ~600mm
- Pipes/benching 1/2+ full of dark/dirty water.
- Not visibly flowing
- Lots of dirt and debris in MH
- NOTE: MH is south of ST324B, and north of Outfall 4-064.

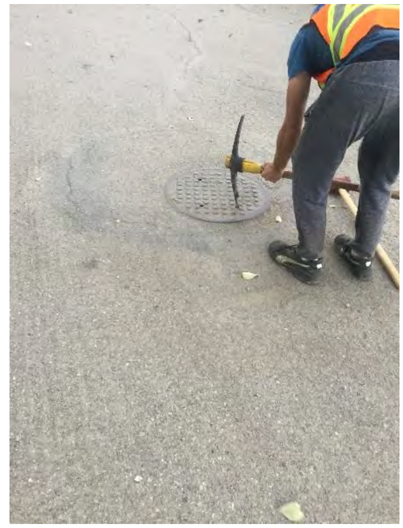
<u>MH ST324B (MH not shown on plans)</u>

- Iron frame and cover, ok.
- Concrete collar slight offset.
- North and South pipes
- Pipes/benching 1/2+ full of dark/dirty water.
- CB lead east to parking lot.
- Not visibly flowing.
- NOTE: MH is south of ST324, and north of ST324A.

<u>MH ST324</u>

- Iron frame and cover(worn smooth)
- North and South Pipes
- 3 CB leads, NE, NW, SW. 300 PVC
- Pipes/benching half full dark/dirty water/debris.
- No visible flow

Area 18

MH ST331 • Iron Frame and cover ok • Concrete collar • North and South pipes. • Pipes/benching ¼ full dark water • CB leads 300mm PVC East and SW. • Some concrete cracking/crumbling inside.		
Outfall 4-065 • Concrete outfall • Some concrete broken off at end • Half full water • Overgrown		
Outfall 5-022 • Concrete 750mm • Flowing NE into ditch/stream • No visible cracks, clean		

Area 14

Outfall 6-024 • 600mm CSP • Clean, small amount of dirt. • Flowing SW into ditch/stream		
Outfall 6-027 • Unable to locate		
Outfall 5-012 & Twin 1000 CSP Culvert Crossing 6 Ave • South Side: • Twin 1000mm CSP • Draining SE into ditch/stream • Lots of riprap and broken concrete in right/east culvert outlet with lots of mud built up behind (1/3 full). • Left/west culvert outlet some riprap and mud built up. • Erosion control filter fabric exposed. • Outfall 5-012: • 400mm CSP, clean. • PVC CB lead draining beside CSP outlet. Partial encase with concrete.		

<u>Outfall 5-012 & Twin 1000 CSP Culvert Crossing 6 Ave</u>

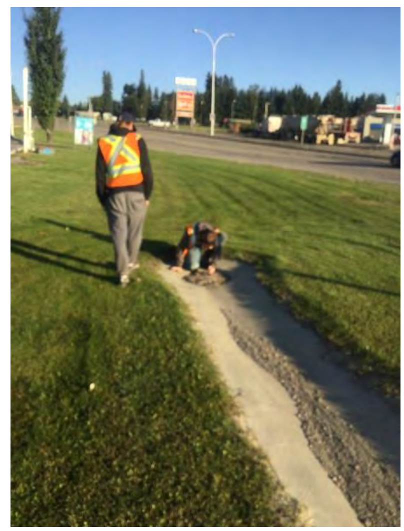
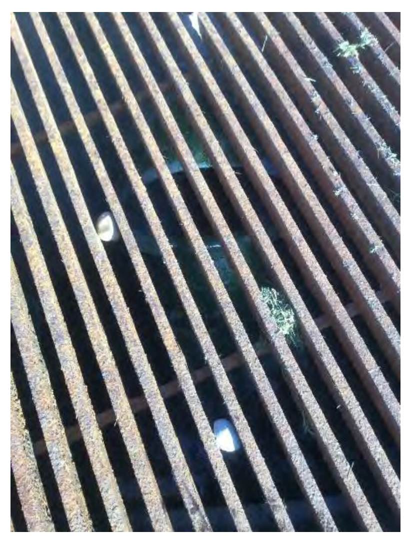
- <u>North Side:</u>
- Draining stream SE under 6th Ave.
- Riprap mostly intact.
- Filter fabric partially exposed.
- Silt fence collapsed.
- Some Riprap rocks and concrete in culvert inlet.

Outfall 5-010 • 500mm CSP outlet. • ¼ submerged, clean. • Flowing NE into ditch/stream		

Area 11

3 barrel Concrete Culvert under 9th Ave • South side fenced in. Unable to access at time. • 3x600mmx1100mm Concrete rectangular culvert barrels. • Gabion bank protection around culvert outlets. • PVC CB lead (~300) draining from 9th Ave. • Silt fences		
3 barrel Concrete Culvert under 9th Ave • North side: • Overgrown • No silt fencing or riprap • Stream appears to be flowing south under the culvert. • Culvert mostly dry. • Some dirt/debris is east-most barrel. • (See site photo IMG_0303 to 0325)		

Area 18

Unknown CB1 (in Shell parking lot, north of 2nd Ave) • Concrete swale drains to CB • Pipe drains west		
Unknown CB2 (At west corner of Hwy 16 and 42nd St) • Concrete swale drains from Hwy 16. • Inlet north • Drains South.		
Unknown CB3 (South side of Hwy 16 (4th Ave) North of gas station) • Large grated top • CSP pipes NW SE		

<table>
<tr>
<td><u>Culvert inlet going to Unknown CB3 (South side of 4th Ave)</u>

600mm CSP
Clean, draining to SE(CB3)
Culvert draining from NE into ditch
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<tr>
<td><u>Unknown CB4 (North side of 2nd Ave east of 43rd St)</u>

Concrete swale draining to CB from 2nd Ave and gas station lot.
NE inlet pipe CSP
Lots of water and debris in CB
Possibly covered outlet pipe. Probably to SE.
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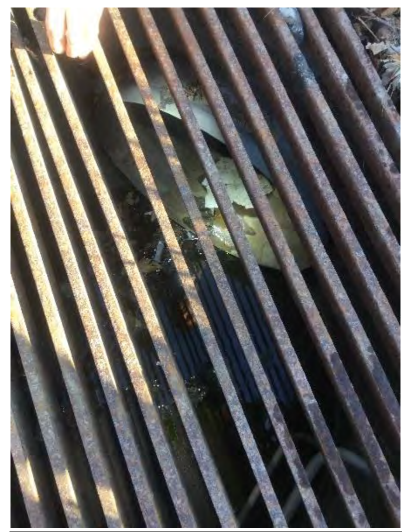
<td></td>
</tr>
<tr>
<td><u>Outfall A (Not marked on plans)</u>

Multiple CSP Outfalls into ditch draining into 1300mm CSP culvert East under Hwy 16.
Some rocks/concrete in 1300mm CSP
2 x 400mm CSP Culvert draining from NW. 1 is clear, other overgrown and partial underwater.
800mm CSP Culvert draining from NW
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<u>Unknown Manholes and Access pit</u> • Dirt pit to pipe separated pipe segments • 2 x CSP ‘manholes”/access to CSP pipes • Unknown sizes • Smaller/east-most CSP pipe has moderate amount of mud inside. • Not safe, no cover.		

Area 14

<u>MH to Outfall 6-027</u> • Iron frame and cover elevated, ok. • Concrete collar offset • Manhole in grass area • North and West 375mm concrete • Concrete CB lead from west. • Flows to west towards outfall • Clean • NOTE: sanitary manhole on C&G that intercepts water before it reaches CB. C&G broken/uneven/grass growing, probable water pooling at sanitary MH.		

Area 13

<table>
<tr>
<td><u>MH ST232</u>
Iron frame and cover ok
Collar/barrel ok
900mm PVC north and east
Flows to east, towards storm pond
Clean</td>
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<tr>
<td><u>Pond Inlet 6-011</u>
Earth heaved above pipe to outfall.
Concrete flared end partially full (~1/4)
Flared end separated from pipe. Eroded around.
Concrete poured on joint between pipe and end, broken, crumbled away.
Some grass/debris in flared end metal grill</td>
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<tr>
<td><u>Pond Inlet 6-006</u>
Earth heaved above pipe to outfall.
Concrete poured on joint between PVC pipe and Conc flared end.
Outfall half full water
Lots of debris in flared end metal grill.</td>
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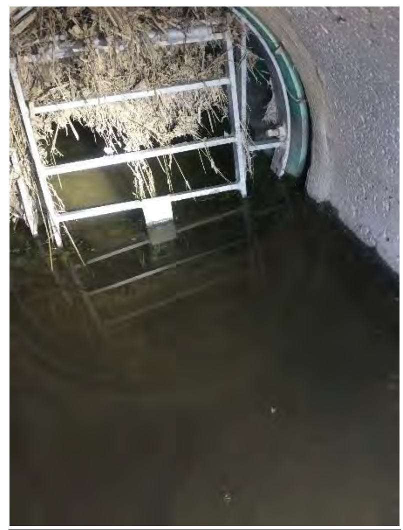
<u>Pond Outlet 6-014</u> • Not visible • Flowing to MH ST235 with flow restrictor (Built east of where shown on plans)		
<u>Pond Inlet 6-013</u> • Earth heaved above pipe to pond. • Flared end half full water • Concrete poured on joint between pipe and flared end • Small amount of debris in flared end metal grill • Looks like PVC pipe is separating from concrete flared end inside (IMG_0528).		
<u>MH ST212</u> • Iron frame and cover ok • Concrete collar ok • 900mm PVC west and South • Flows to the south • CB leads N, NW, S. (~300 PVC) • No 300mm PVC to NE as shown on plans. • South CB barrel good. Sump full of water to 300 PVC		

<u>MH ST235 (Located east of where shown on plans)</u> • Iron frame and cover ok. Elevated. Slight offset. • Concrete collars/barrel ok. Some ground water infiltration. • Pond outlet draining to vertical pipe protruding from bottom of MH. • Flows to 525 PCV east. • (See site photo IMG_0553 to 0567)		

<u>Area 12</u>

<u>MH ST196</u> • Iron frame and cover ok. • Concrete collar includes bricks. Possibly crumbling Conc/bricks or mortar. • Mud in bottom of MH • North and east 300 Concrete pipes • Flows to east. • 2 concrete CB leads NW & SW. • NW CB clogged with lots of dirt and debris.		

Area 9

<table>
<tr>
<td>
<u>Outfall 1-107 (South side of 6 ave, East of 55th St)</u>

Looks like Concrete flared end. Mostly submerged in water.
Pipe is completely full
Probable lots of mud/sediment in pipe.
Overgrown, no riprap/esc visible
No visible flow from outlet
Water in stream/ditch flowing south away from outfall.

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<td>
<u>3250mm CSP culvert under 6th Ave (From SE to NW corner)</u>

About 1/3 full water
No rebar gate/cage
CB Concrete lead from road drains into side
Partial concrete encase on north side

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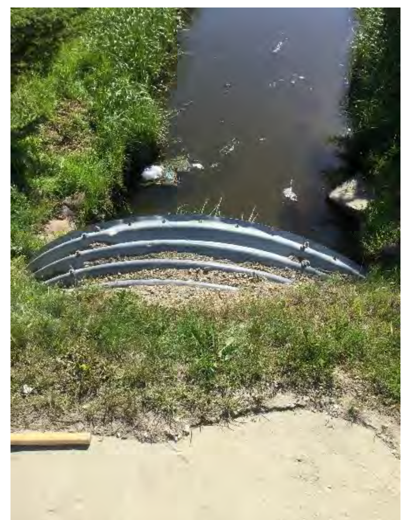
<td></td>
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<td>
<u>MH ST138</u>

Iron frame and cover ok
Concrete collars/barrel ok
Inlet/outlet pipes appear to be completely submerged under water
Lots of plastic bottles and debris floating in MH
No visible flow.

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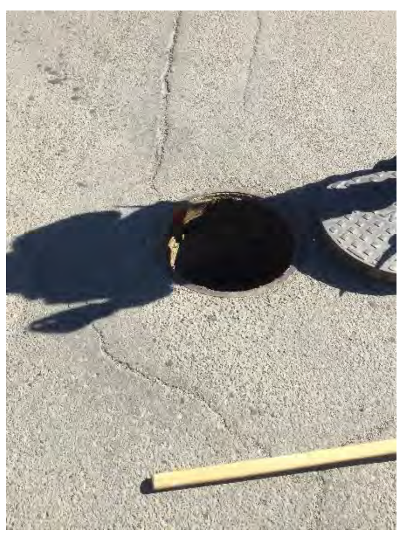
<u>MH ST139</u> • Iron frame and cover ok • Concrete collars ok • East and west pipes mostly full of dark/dirty water • North and south CB leads in above water. • Dirt and debris in MH		
<u>MH ST140</u> • Iron frame and cover ok • Some concrete collars offset • MH mostly clean • Flowing from east to west • Some concrete/limestone (?) overhanging • Some groundwater infiltrating into barrel		
<u>MH ST141</u> • Iron frame and cover ok • Collar has some breaking and loose pieces • 2 pipes Concrete • Flows from east to west. • Small amount of dirt/sediment • Small amount of cracking/broken concrete on/around pipe		

MH ST142

- Iron frame & cover ok
- Concrete collar ok
- PVC pipe from east
- Concrete pipe to west
- Flows to the west.
- NE,NW & S PVC CB leads
- Some dirt/sediment in bottom
- South CB was cleaned out. Some cracking in concrete around grate.

MH ST143

- Iron frame and grate ok. Slight offset
- Concrete collar ok
- North, east and west PVC pipes.
- Flows to west
- Sediment in east PVC
- Only SE PVC CB lead is visible from top
- Some dirt/sediment in MH bottom/sump (?)

Area 9

MH ST116 (CB MH South side of road)

- Circular iron CB frame and grill ok
- Concrete collar and barrel ok
- PVC CB lead north
- Drains to PVC south
- Small pipe draining into CB from east
- Some water and sediment in sump

MH ST117 • Iron frame and cover ok • MH in grassed area off path • Concrete barrel and collar ok • North and west PVC pipes • Draining to west • Clean		
Pond Inlet from MH ST117 • Concrete flared end • Concrete poured over joint between PVC pipe and flared end • Water, mud and debris in outlet. • Possible curved or bend in pipe from MH ST117 (?)		
ST Pond 'A' (East of 62nd St & North of Poplar Place) • Ditch fed pond • 450mm CSP Outlet at SE corner to MH • Pond outlet MH flows from Pond (west) to outlet at ditch (east) • Short PVC pipe segment with what looks like flow restrictor attached to upstream pipe • Outlet into ditch 450mm CSP, lots of organic matter/overgrown.		

Area 15

<table>
<tr>
<td><u>Outfall 1-017</u>

CSP outfall ~900mm
Appears to be cut in half at outlet
Half full of rocks, mud/sediment and filter fabric.
Flowing south into ditch/stream
(See site photo IMG_0754 to 0769)

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<td><u>Outfall 1-062</u>

1070mm Concrete outfall
Flowing south into stream/ditch
Concrete frame/retaining wall around outfall ok
Some small damage/erosion to concrete where pipe meets frame
Some erosion under west side of outfall concrete frame
Concrete slabs broken below outfall
Metal gate/grill mission
Eroded away at toe east corner of concrete frame/retaining wall

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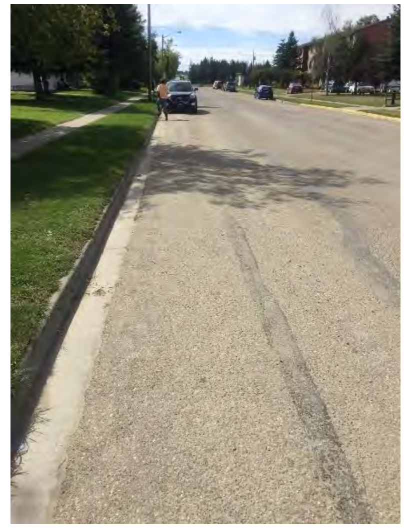
Area 16

MH ST268 • Concrete cracking and broken inside. • Dirt and debris inside • Collar broken/crumbling segments • East and west pipes and benching about 1/3 full dirty water • North CB lead from parking lot • Water not draining/no visible flow. • CB east of MH benching full of dirty water. Not flowing • (See site photo IMG_0802 to 0810) • C&G along south side of 5th Avenue possible sag before CB. Some C&G broken. • (See site photo IMG_0798 to 0801)		

Appendix F
Transportation and Road Network - Cost Estimate

Table F.1
2017 Road Improvements Cost Estimate

Municipal Servicing Plan Update
Town of Edson

Location Description					
Location Description	**Length of Road (m)**	**Width (m)**	**Area to be Replaced (m2)**	**Unit Price**	**Remove & Replace Cost**
1 Avenue - SE of Hwy 16 and R.R. Tracks to 200m East	200	9	1800	$80	$144,000
1 Avenue - (Across from Security Self Storage)	275	10	2750	$80	$220,000
1 Avenue - 25 St to 27 St	220	12	2640	$80	$211,200
1 Avenue - 48 St to 50 St (Main Street)	345	15	5175	$80	$414,000
1 Avenue - 51 Street to 54 Street	500	27	13500	$80	$1,080,000
3 Avenue - 59 St to 63 St	445	10	4450	$80	$356,000
3A Avenue - 59 St to 63 St (Across from Columbia Industries)	390	10	3900	$80	$312,000
4 Avenue - 2nd Ave to 41 St	87	8	696	$80	$55,680
4A Avenue - Lane (between 72 St and 73 St) to End of Road (west of 72 St)	288	7	2016	$80	$161,280
South Glen Avenue - 70 St to 72 St	195	8	1560	$80	$124,800
5A Avenue - Cul-de-sac to 70 St	365	7	2555	$80	$204,400
Glenaire Road - 64 Street to End of Road (West)	330	8	2640	$80	$211,200
5 Avenue - 52 St to 54 Street	335	12	4020	$80	$321,600
6 Avenue - 52 St to Lane (between 52 St and 53 St)	90	15	1350	$80	$108,000
7 Avenue - 49 St to 52 St	518	15	7770	$80	$621,600
8 Avenue - 44 St to 45 St	94	9	846	$80	$67,680
8 Avenue - 47 St to 48 St	164	8	1312	$80	$104,960
8 Avenue - 49 St to 52 St	518	11	5698	$80	$455,840
8 Avenue - 52 St to 53 St	67	11	737	$80	$58,960
9 Avenue - 62A St to 63 St	76	9	684	$80	$54,720
10 Avenue - East end of Ave to 48 St	145	10	1450	$80	$116,000
10 Avenue - 62 St to 62A St	88	10	880	$80	$70,400
11 Avenue - 61 St to 62A St	192	10	1920	$80	$153,600
16 Avenue - 41 St to 42 St	264	10	2640	$80	$211,200
16 Avenue - Cul-de-sac to 49 St	156	12	1872	$80	$149,760
17 Avenue - 48 St to 49 St	165	13	2145	$80	$171,600
18 Avenue - Midway (between 40 St and 41 St) to 42 St	240	14	3360	$80	$268,800
18 Avenue - 48 St to 49 St	165	13	2145	$80	$171,600
41 Street - 4 Ave to 6 Ave	180	9	1620	$80	$129,600
41 Street - 16 Ave to 18 Ave	200	9	1800	$80	$144,000
43 Street - Dirt Backlane (1 Ave alignment) to 2 Ave	110	11	1210	$80	$96,800
46 Street - 2 Ave to 4 Ave	223	11	2453	$80	$196,240
47 Street - 1 Ave to 2 Ave	108	13	1404	$80	$112,320
47 Street - 8 Ave to 9 Ave	223	11	2453	$80	$196,240
48 Street - 1 Ave to 2 Ave	108	13	1404	$80	$112,320
48 Street - 8 Ave to 10 Ave	222	12	2664	$80	$213,120
48 Street - 16 Ave to 51 St	385	12	4620	$80	$369,600
49 Street - 1 Ave to 2 Ave	108	10	1080	$80	$86,400
50 Street - 1 Ave to 2 Ave	108	12	1296	$80	$103,680
52 Street - 6 Ave to 10 Ave	465	11	5115	$80	$409,200
53 Street - 6 Ave to 8 Ave	220	10	2200	$80	$176,000
54 Street - 4 Ave to 5 Ave	104	10	1040	$80	$83,200
55 Street - 1 Ave to 4 Ave	338	11	3718	$80	$297,440
57 Street - 1 Ave to 4 Ave	338	9	3042	$80	$243,360
58 Street - 2 Ave to 4 Ave	218	8	1744	$80	$139,520
59 Street - 1A Ave to 4 Ave	260	10	2600	$80	$208,000
61 Street - Cul-de-sac to Cul-de-sac	284	9	2556	$80	$204,480
62 Street - Cul-de-sac to 10 Ave	128	8	1024	$80	$81,920
62A Street - 9 Ave to 11 Ave	158	10	1580	$80	$126,400
63 Street - 5 Ave to North End of Road	168	8	1344	$80	$107,520
64 Street - 5 Ave to Glenaire Road	200	7	1400	$80	$112,000
65 Street - 5 Ave to Glenaire Road	200	6	1200	$80	$96,000
66 Street - 5 Ave to Glenaire Road	200	7	1400	$80	$112,000
68 Street - 5 Ave to 5A Ave	95	9	855	$80	$68,400
69 Street - 5 Ave to 5A Ave	95	10	950	$80	$76,000
70 Street - 4 Ave to 4A Ave	120	7	840	$80	$67,200
Totals	12,483		137,123		
				Sub-Total Cost	$10,969,840
				Contingency (30%	$3,290,952
				Engineering and Administr	$1,096,984
				Capital Cost	**$15,357,776**

Appendix G
Transportation and Roadway
- Figures & Supporting Data

Site Plans

Figure F.1: Intersection Count Locations

Figure F.2: Screenline Locations

Traffic Volumes

Figure F.3: 2017 Existing Traffic Volumes

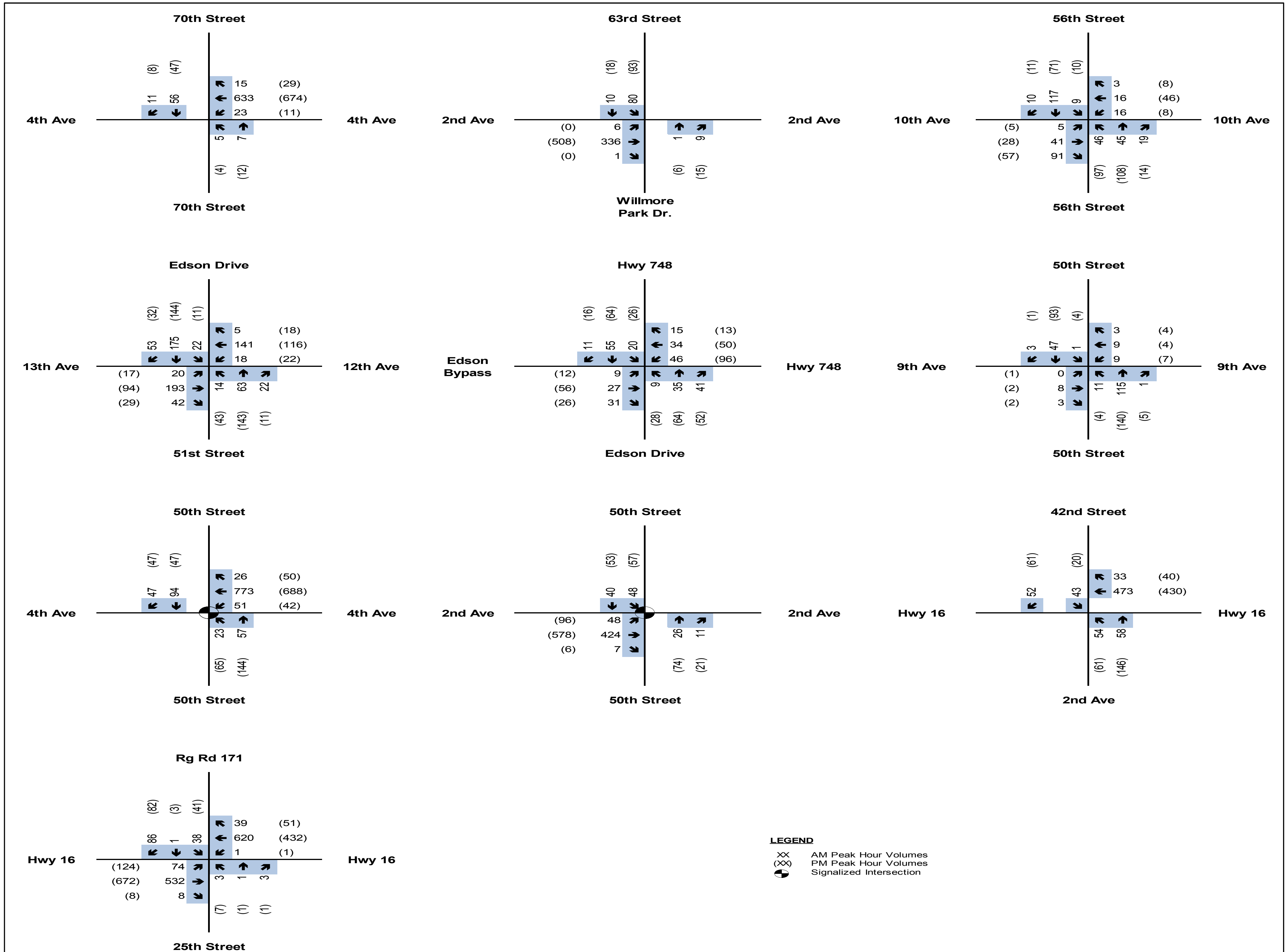


Figure F.4: 2032 Future Traffic Volumes

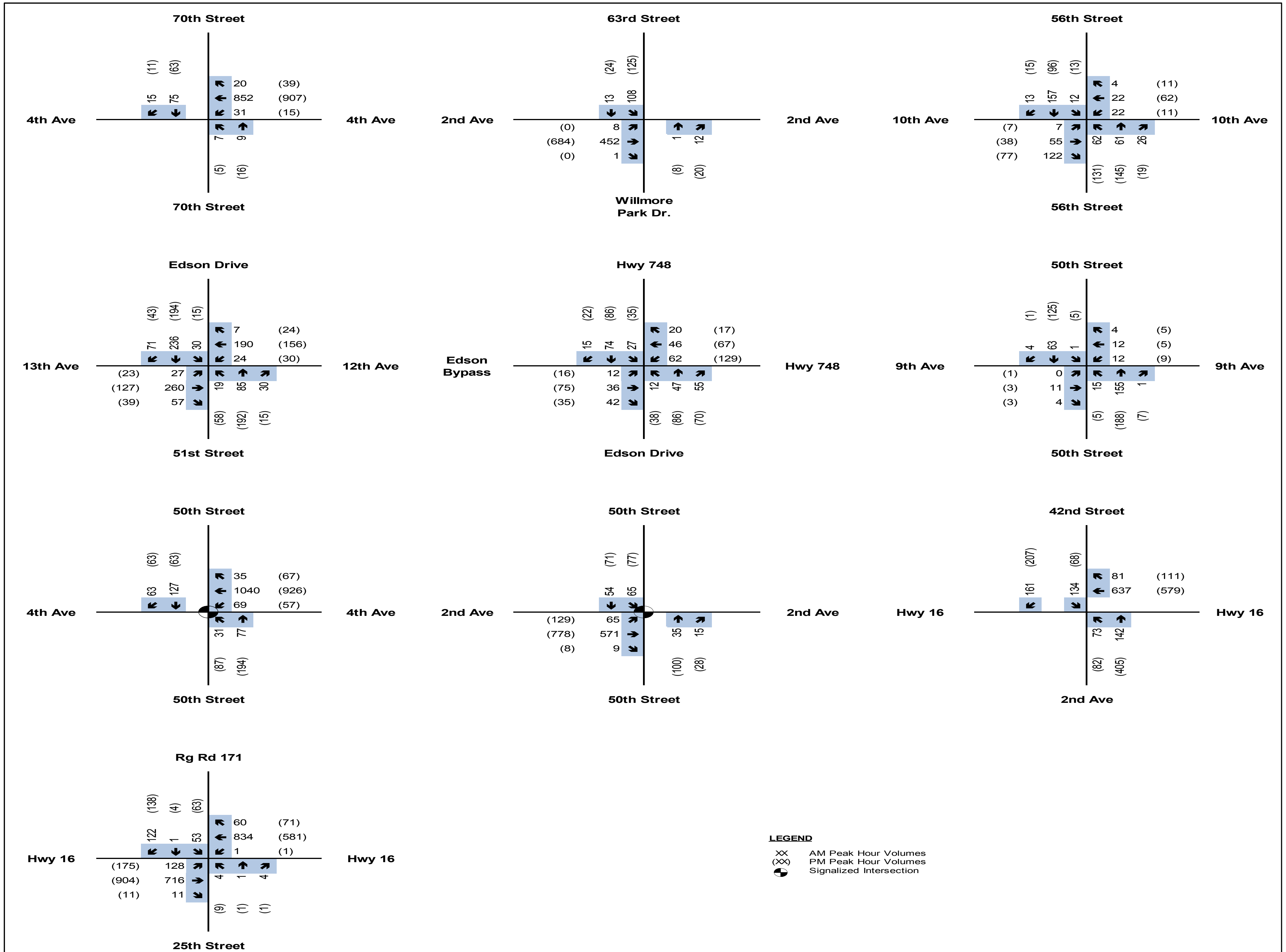
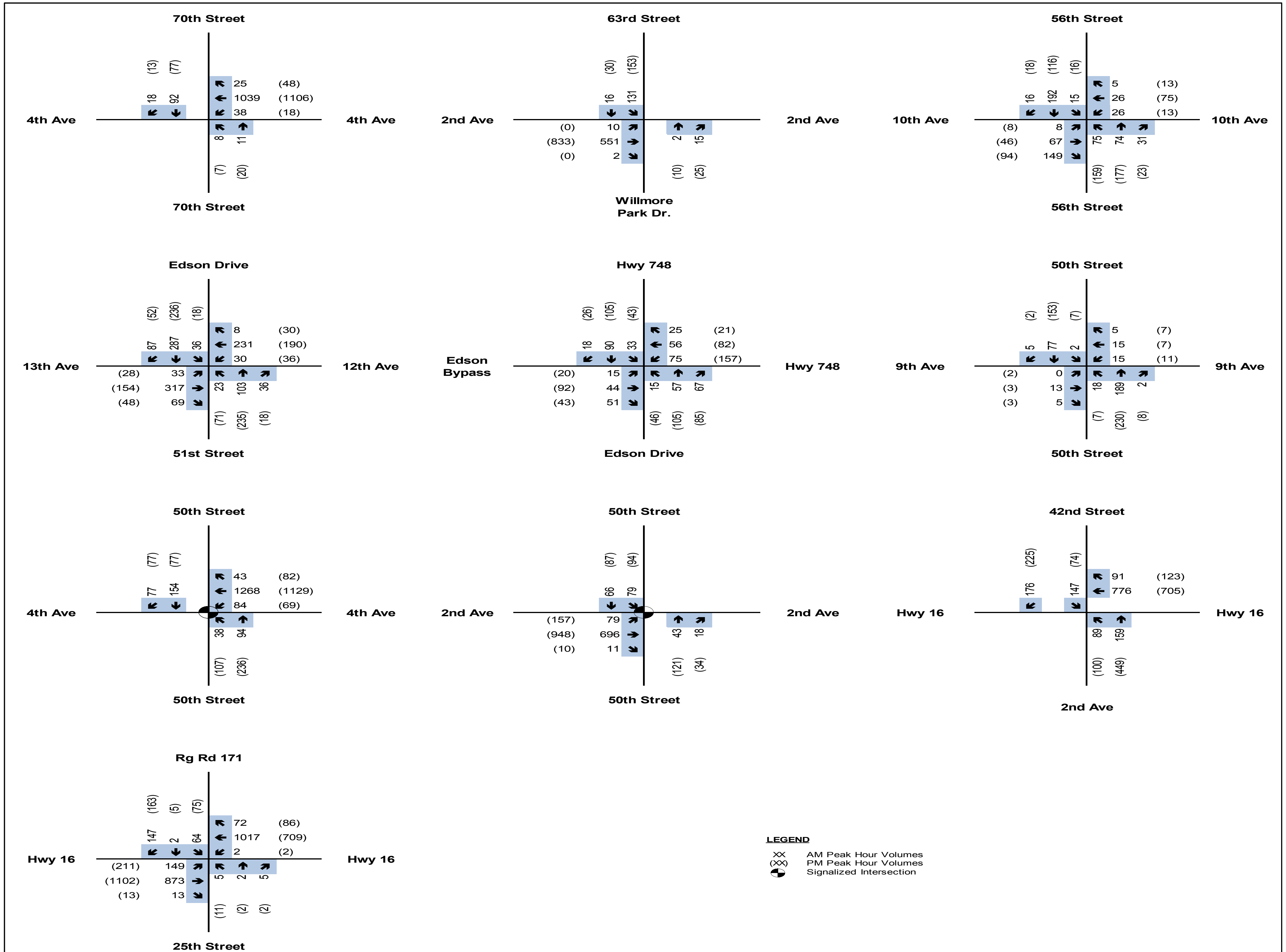


Figure F.5: 2042 Future Traffic Volumes



Transportation Tables

Table G.1
Corridor Capacity Analysis Table

Municipal Servicing Plan Update
Town of Edson

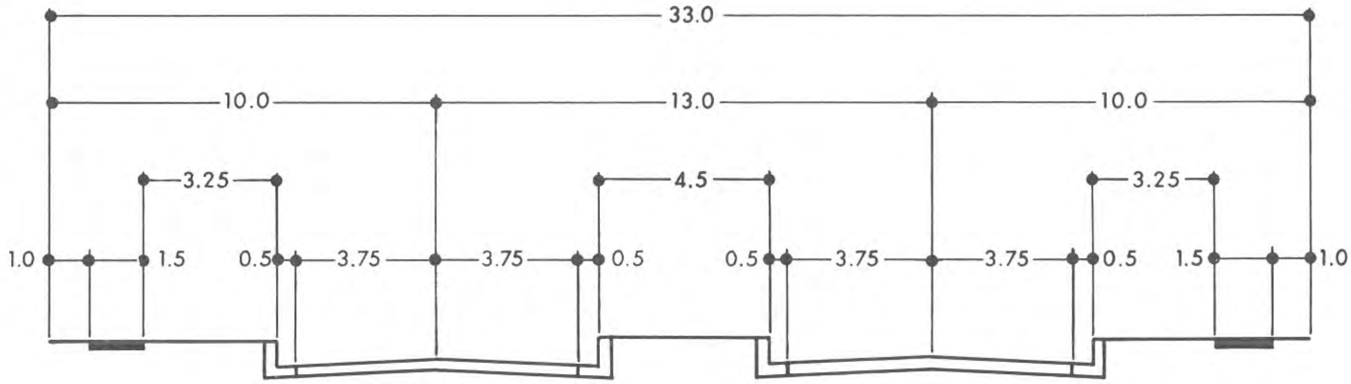
Segment No.	Name of Segment	AM Peak Volume	PM Peak Volume	Daily Capacity (vehicles per day)	2042 ADT	2042 Utilization
1	70th Street (north of 4th Avenue)	146	158	12000	1520	13%
2	70th Street (south of 4th Avenue)	149	122	12000	1355	11%
3	4th Avenue (east of 70th Street)	1102	1172	30000	11370	38%
4	4th Avenue (west of 70th Street)	1065	1126	30000	10955	37%
5	63rd Street (north of 2nd Avenue)	159	193	12000	1760	15%
6	Willmore Park Drive (south of 2nd Avenue)	35	65	12000	500	4%
7	2nd Avenue (east of 63rd Street)	697	1011	30000	8540	28%
8	2nd Avenue (west of 63rd Street)	563	833	30000	6980	23%
9	56th Street (north of 10th Avenue)	310	348	12000	3290	27%
10	56th Street (south of 10th Avenue)	547	582	12000	5645	47%
11	10th Avenue (east of 56th Street)	170	186	12000	1780	15%
12	10th Avenue (west of 56th Street)	341	400	12000	3705	31%
13	Edson Drive (north of 13th Avenue)	554	599	30000	5765	19%
14	51st Street (south of 13th Avenue)	548	644	30000	5960	20%
15	12th Avenue (east of Edson Drive)	658	446	12000	5520	46%
16	13th Avenue (west of Edson Drive)	760	543	12000	6515	54%
17	Hwy 748 (north of Edson Bypass)	238	320	30000	2790	9%
18	Edson Drive (south of Edson Bypass)	355	541	30000	4480	15%
19	Hwy 748 (east of Edson Drive)	300	480	30000	3900	13%
20	Edson Bypass (west of Edson Drive)	199	309	30000	2540	8%
21	50th Street (north of 9th Avenue)	278	401	12000	3395	28%

Table G.1
Corridor Capacity Analysis Table

Municipal Servicing Plan Update
Town of Edson

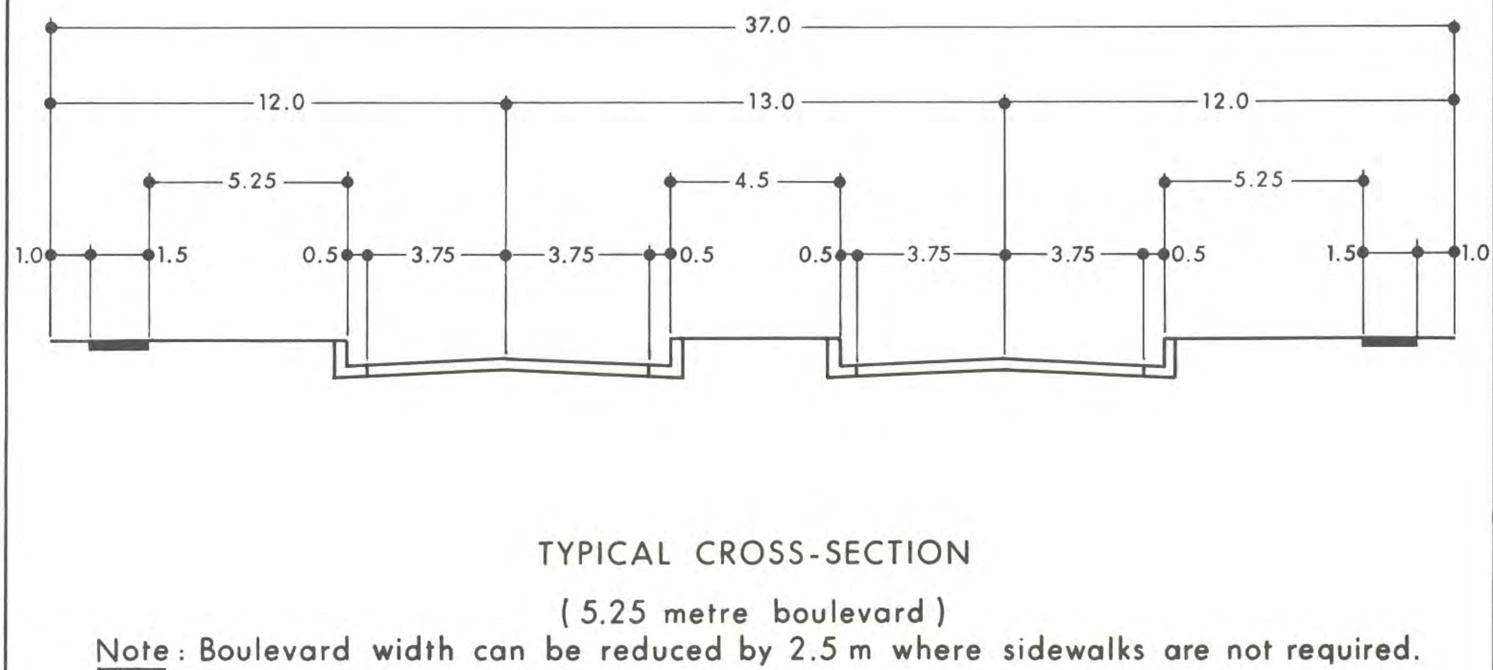
22	50th Street (south of 9th Avenue)	306	412	12000	3590	30%
23	9th Avenue (east of 50th Street)	52	43	12000	475	4%
24	9th Avenue (west of 50th Street)	56	24	12000	400	3%
25	50th Street (north of 4th Avenue)	368	472	12000	4200	35%
26	50th Street (south of 4th Avenue)	370	489	12000	4295	36%
27	4th Avenue (east of 50th Street)	1395	1280	30000	13375	45%
28	4th Avenue (west of 50th Street)	1383	1313	30000	13480	45%
29	50th Street (north of 2nd Avenue)	267	459	12000	3630	30%
30	50th Street (south of 2nd Avenue)	138	252	12000	1950	16%
31	2nd Avenue (east of 50th Street)	793	1076	30000	9345	31%
32	2nd Avenue (west of 50th Street)	786	1115	30000	9505	32%
33	42nd Street (north of Highway 16)	573	871	12000	7220	60%
34	2nd Avenue (south of Highway 16)	248	549	30000	3985	13%
35	Highway 16 (east of 42nd Street)	867	828	30000	8475	28%
36	Highway 16 (west of 42nd Street)	1041	1031	30000	10360	35%
37	25th Street (north of Highway 16)	436	542	30000	4890	16%
38	Range Road 171 (south of Highway 16)	29	35	12000	320	3%

Cross-Sections



TYPICAL CROSS-SECTION

(3.25 metre boulevard)



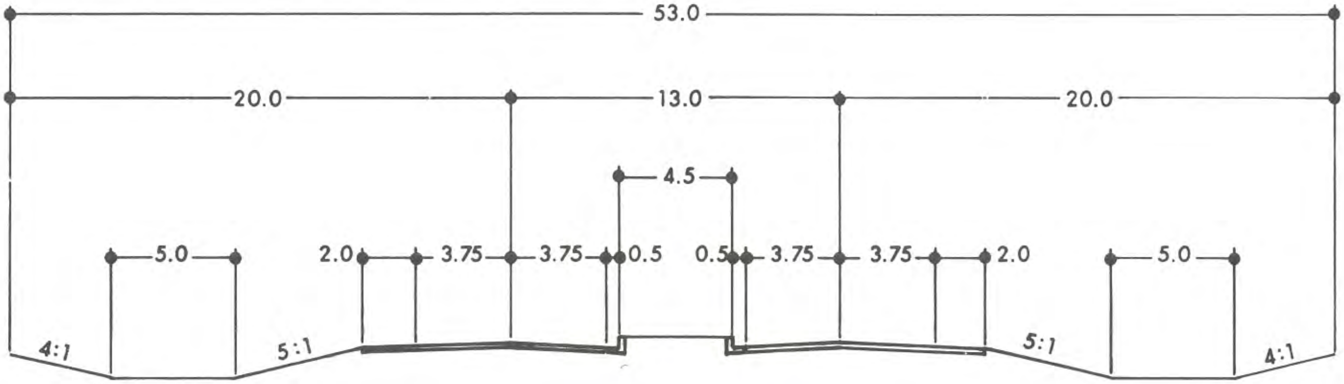
TYPICAL CROSS-SECTION

(5.25 metre boulevard)

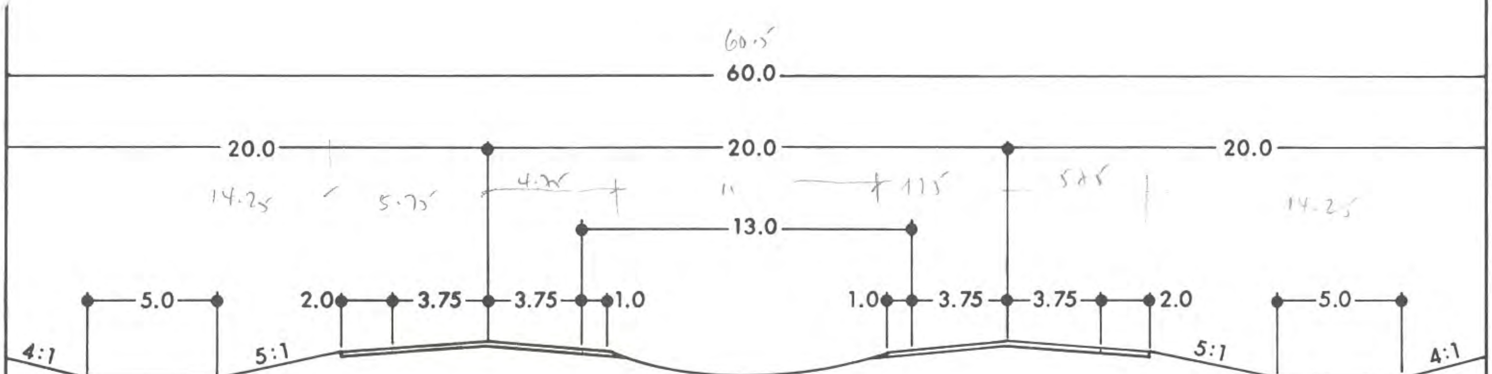
Note : Boulevard width can be reduced by 2.5 m where sidewalks are not required.

Figure 6.1
RECOMMENDED URBAN ARTERIAL ROADWAY STANDARDS

TOWN OF EDSON
General Engineering Study
1982



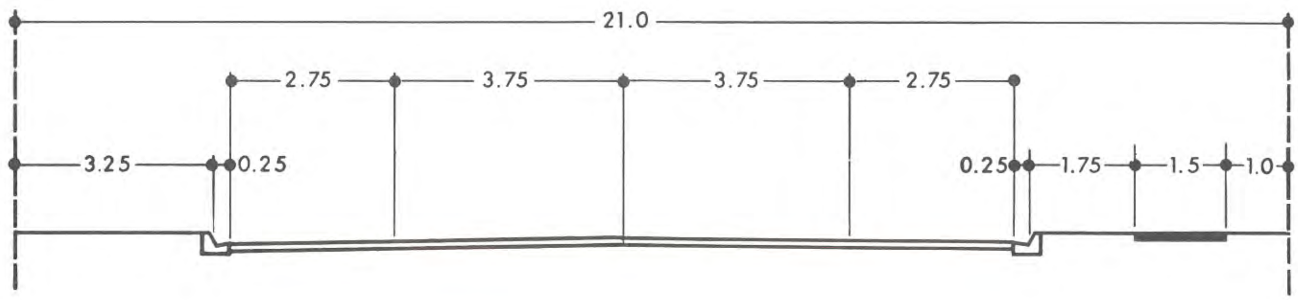
TYPICAL CROSS - SECTION
Raised Median



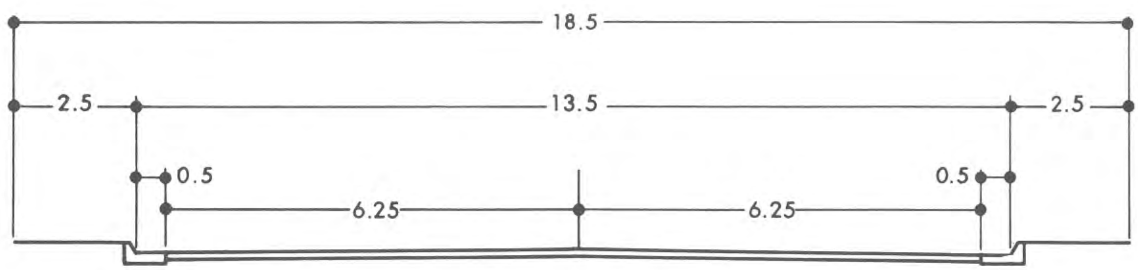
TYPICAL CROSS - SECTION
Narrow Depressed Median

Figure 6.2
RECOMMENDED RURAL ARTERIAL ROADWAY STANDARDS

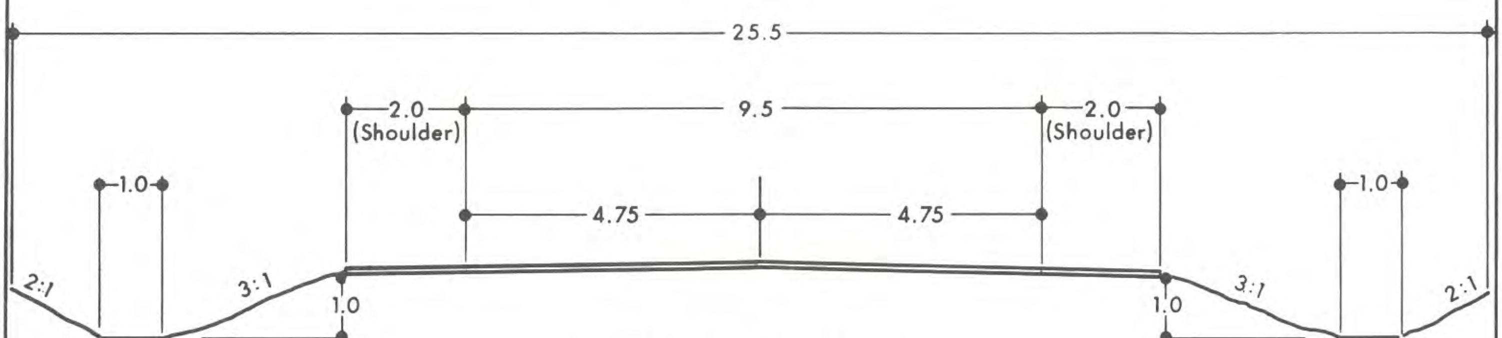
TOWN OF EDSON
General Engineering Study
1982



TYPICAL CROSS-SECTION
Residential Collector

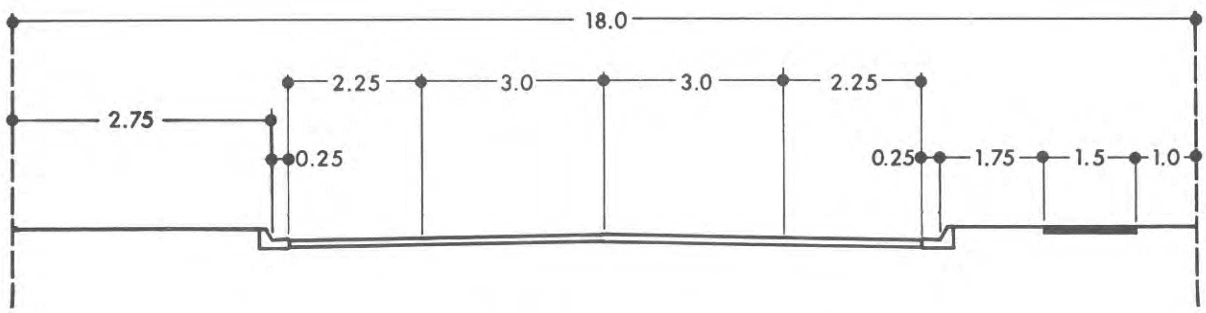


TYPICAL CROSS-SECTION
Urban Industrial Collector

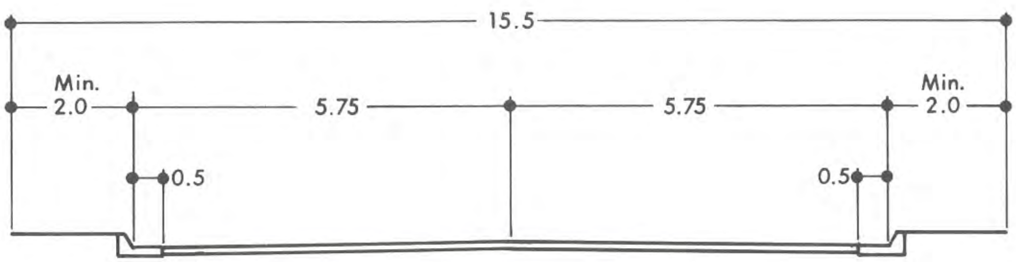


TYPICAL CROSS-SECTION
Rural Industrial Collector
(1 metre ditch)

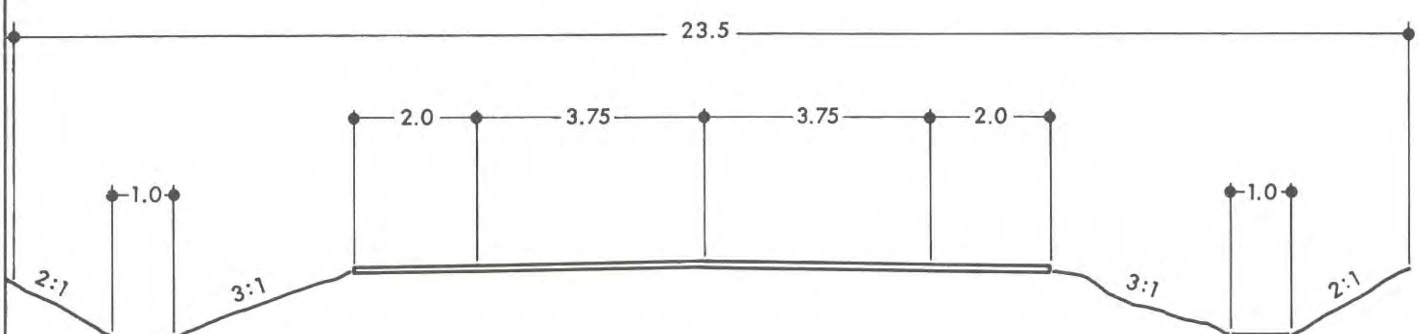
Figure 6.3
RECOMMENDED COLLECTOR ROADWAY STANDARDS



TYPICAL CROSS-SECTION
Residential Local



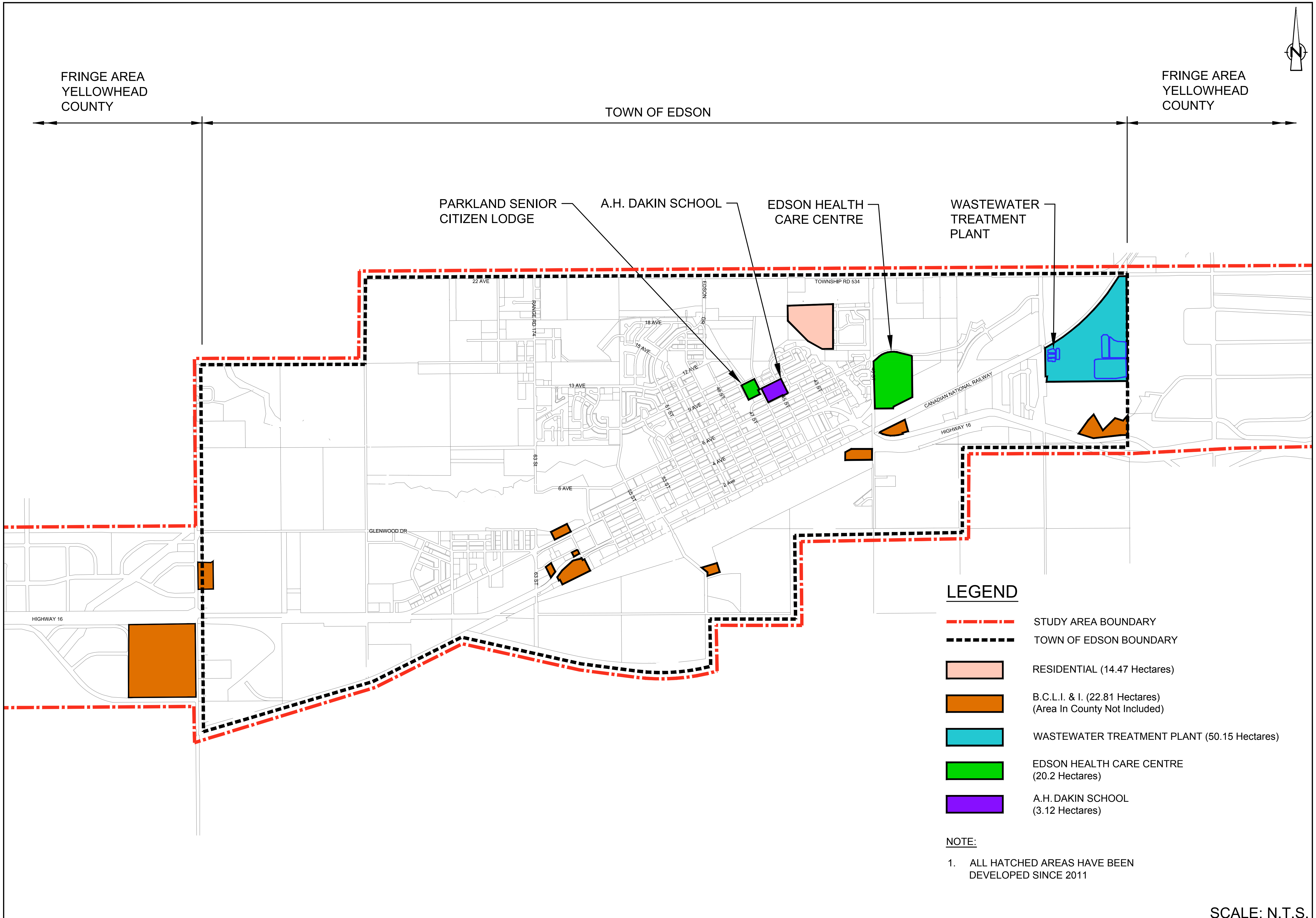
TYPICAL CROSS-SECTION
Urban Industrial Local



TYPICAL CROSS-SECTION
Rural Industrial Local

Figure 6.4
RECOMENDED LOCAL ROADWAY STANDARDS

Appendix H
Figures



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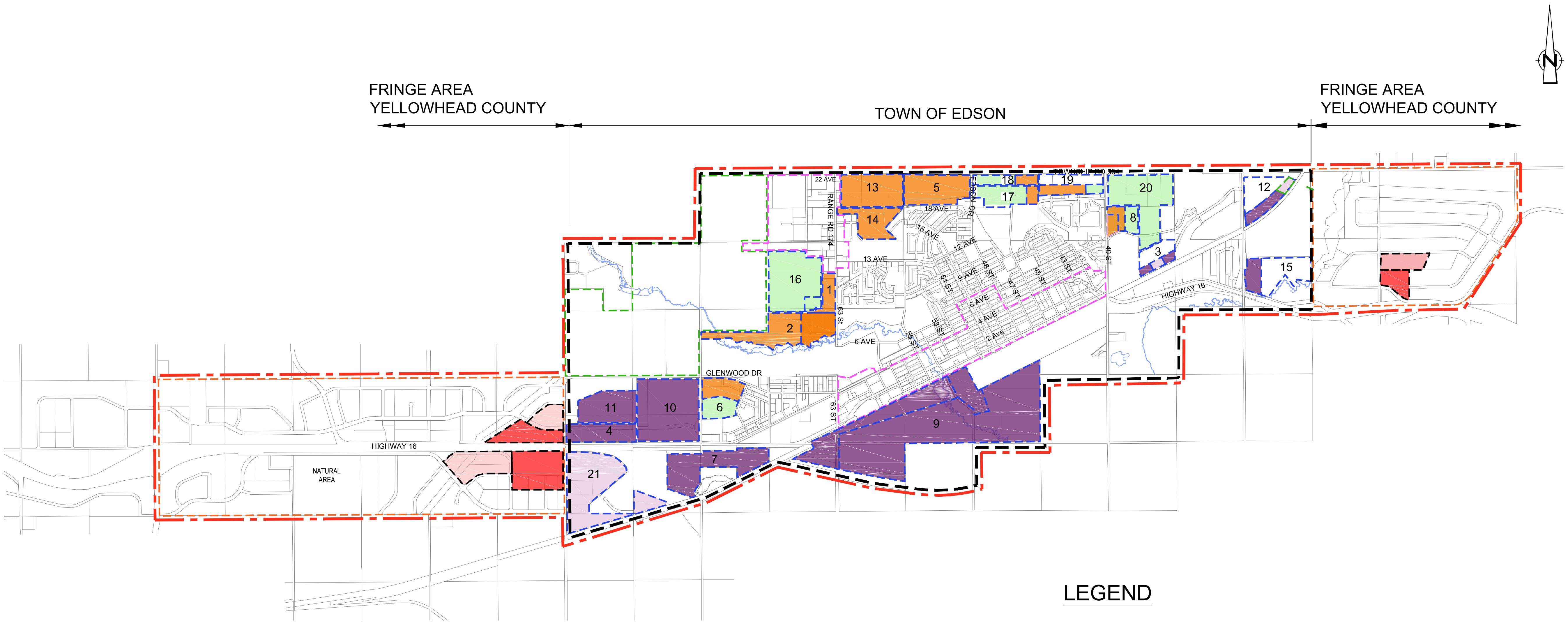
Jul 5, 2018

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

POPULATION PROJECTION & FUTURE DEVELOPMENT HORIZONS
NEW DEVELOPMENT, 2011-2017

FIGURE No. 2.1

CAD File: I:\CAD\11148145 - Town of Edson\11148145-MEMO\11148145-000(MEMO001)GN\11148145-000(MEMO)GN-CA021.dwg



LEGEND

STUDY AREA BOUNDARY

TOWN OF EDSON BOUNDARY

FUTURE AREA STRUCTURE PLAN (ASP)

FUTURE CONCEPT PLAN AREA

FUTURE AREA REDEVELOPMENT PLAN

FRINGE AREA EXPANSION

EDSON INDUSTRIAL/COMMERCIAL SERVICEABLE LANDS

TO 2032

TO 2042

EDSON RESIDENTIAL SERVICEABLE LANDS

TO 2032

TO 2042

FRINGE AREA INDUSTRIAL/COMMERCIAL DEVELOPMENT

TO 2032

TO 2042

ESTIMATED CHRONOLOGICAL ORDER OF DEVELOPMENT IN THE TOWN OF EDSON
(OCTOBER 16, 2017)

No.	Name	Type	Year of Plan	Horizon Year
1	Anderson	Residential	2018	2032
2	Marri	Residential		
3	Hospital	Industrial		
4	Janish (north)	Business Commercial Light Industrial	2019	
5	West Edson Dr.	Residential / Mixed Use	2020	
6	McPhee	Church		
7	Titan	Industrial		
8	Parks South	Residential	2025	
9	South of CN Tracks	Industrial		
10	Gelmici	Business Commercial Light Industrial	2028	
11	Janish (south)	Business Commercial Light Industrial		
12	Wanchulak	Industrial		
13	W of 5	Residential / Mixed Use	2030	
14	Johnston	Residential		
15	Swanberg	Industrial	2032	
16	Marri	Residential	2035	2042
17	N of PCHS	Residential	2040	
18	N of 17	Residential / Mixed Use		
19	HP3	Residential	2045	
20	Parks North	Residential		
21	SW Corner	Business Commercial Light Industrial	2045+	

SCALE: N.T.S.

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

POPULATION PROJECTION & FUTURE DEVELOPMENT HORIZONS
FUTURE DEVELOPMENT AREAS

FIGURE No. 2.2

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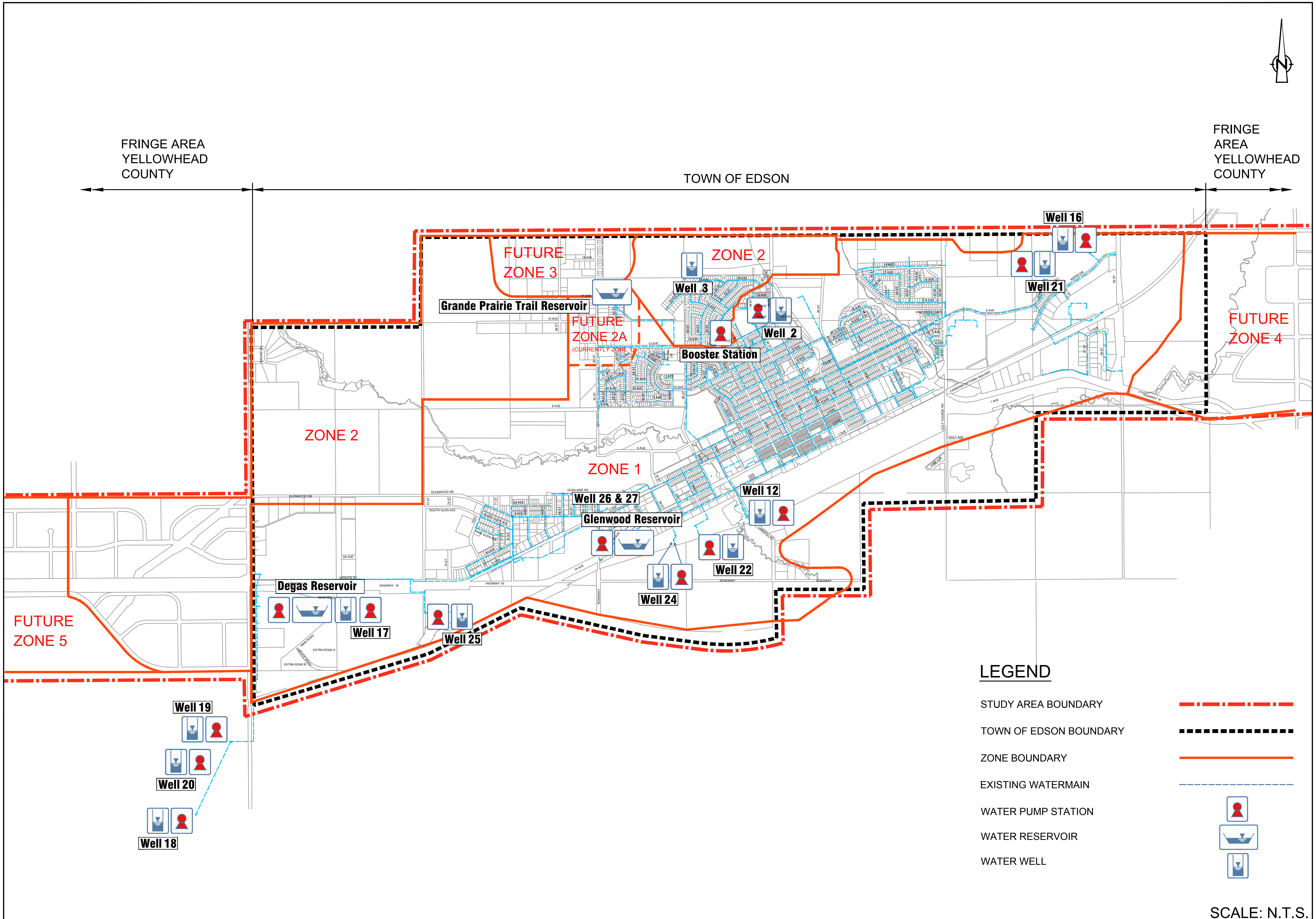

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
EXISTING WATER SYSTEM - OVERALL

FIGURE No. 3.1

11148145-000
Jul 4, 2018

GHD

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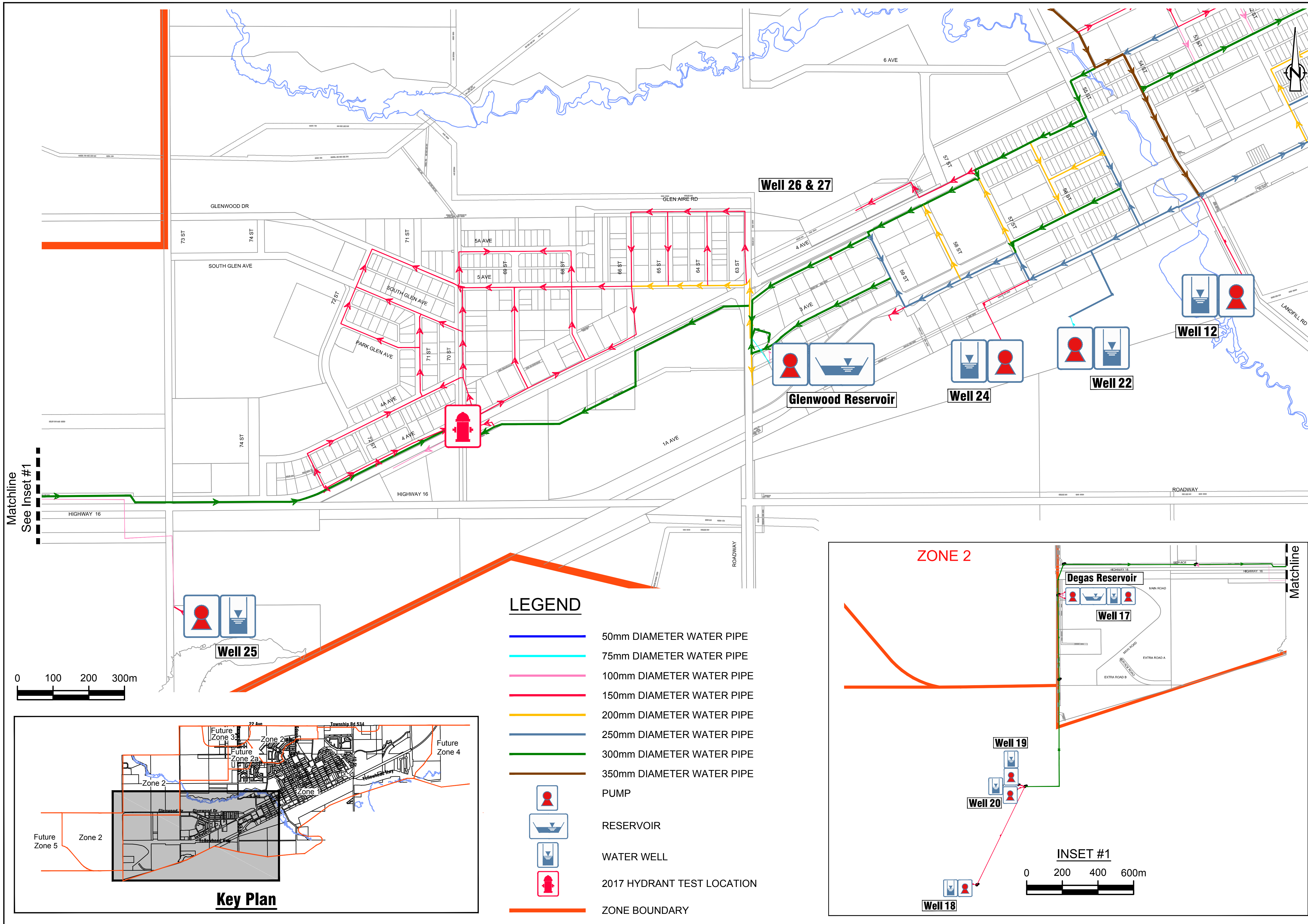

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Jul 6, 2018

FIGURE No. 3.1A

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
EXISTING WATER SYSTEM

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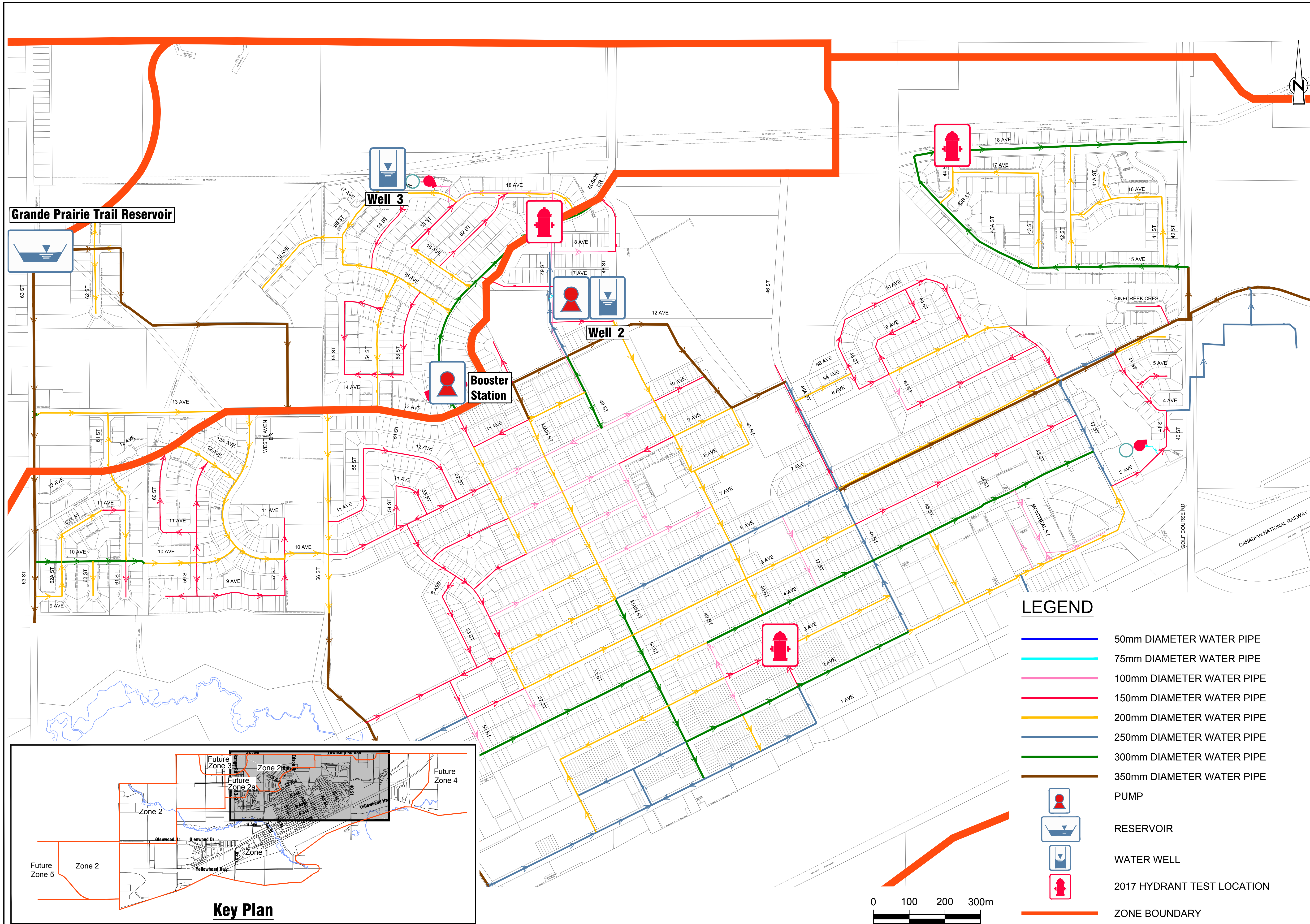

11148145-000

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FIGURE No. 3.1B

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
EXISTING WATER SYSTEM

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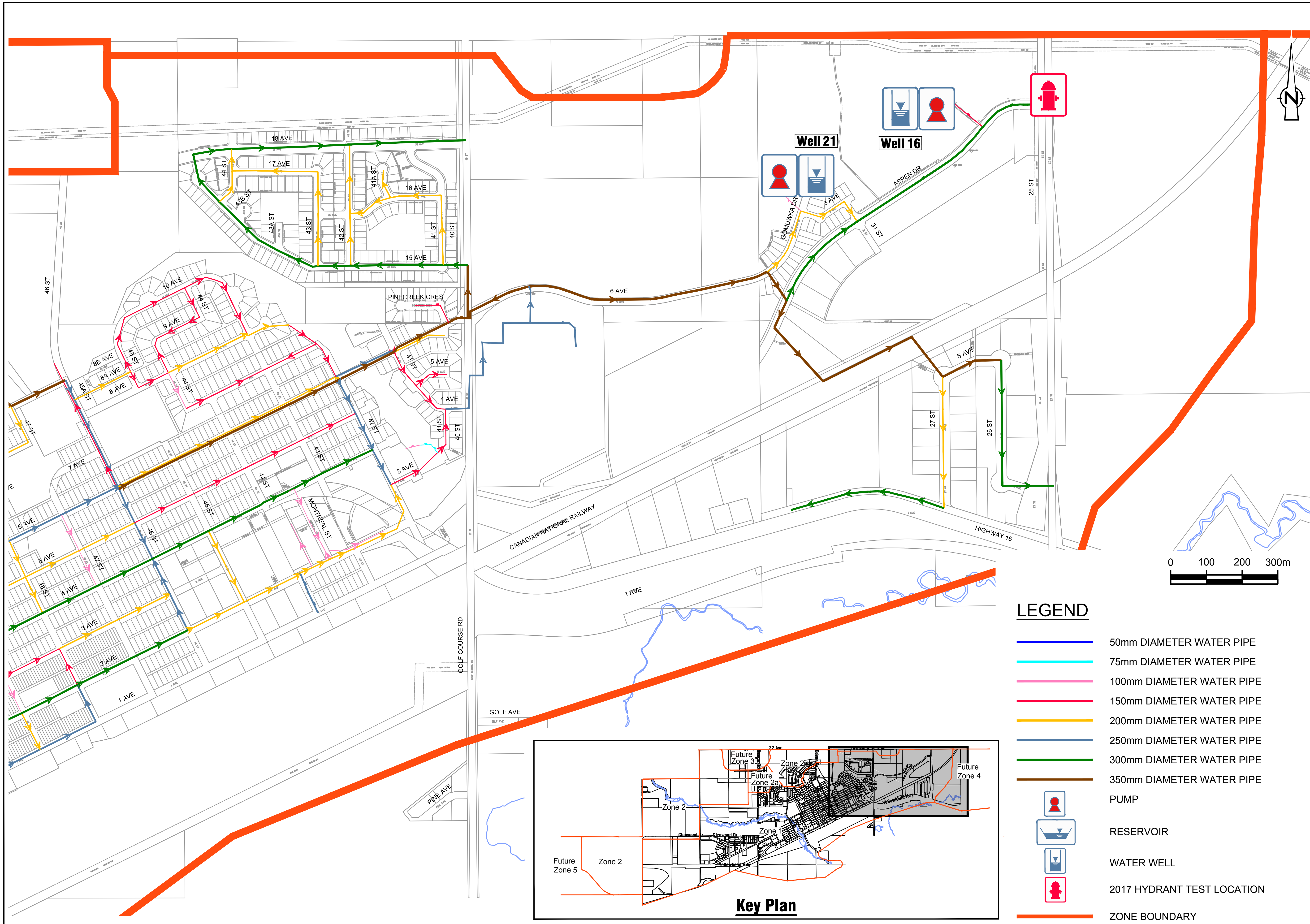

LEGEND

- 50mm DIAMETER WATER PIPE
- 75mm DIAMETER WATER PIPE
- 100mm DIAMETER WATER PIPE
- 150mm DIAMETER WATER PIPE
- 200mm DIAMETER WATER PIPE
- 250mm DIAMETER WATER PIPE
- 300mm DIAMETER WATER PIPE
- 350mm DIAMETER WATER PIPE
- PUMP
- RESERVOIR
- WATER WELL
- 2017 HYDRANT TEST LOCATION
- ZONE BOUNDARY

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
EXISTING WATER SYSTEM

11148145-000
Jul 6, 2018

FIGURE No. 3.1C

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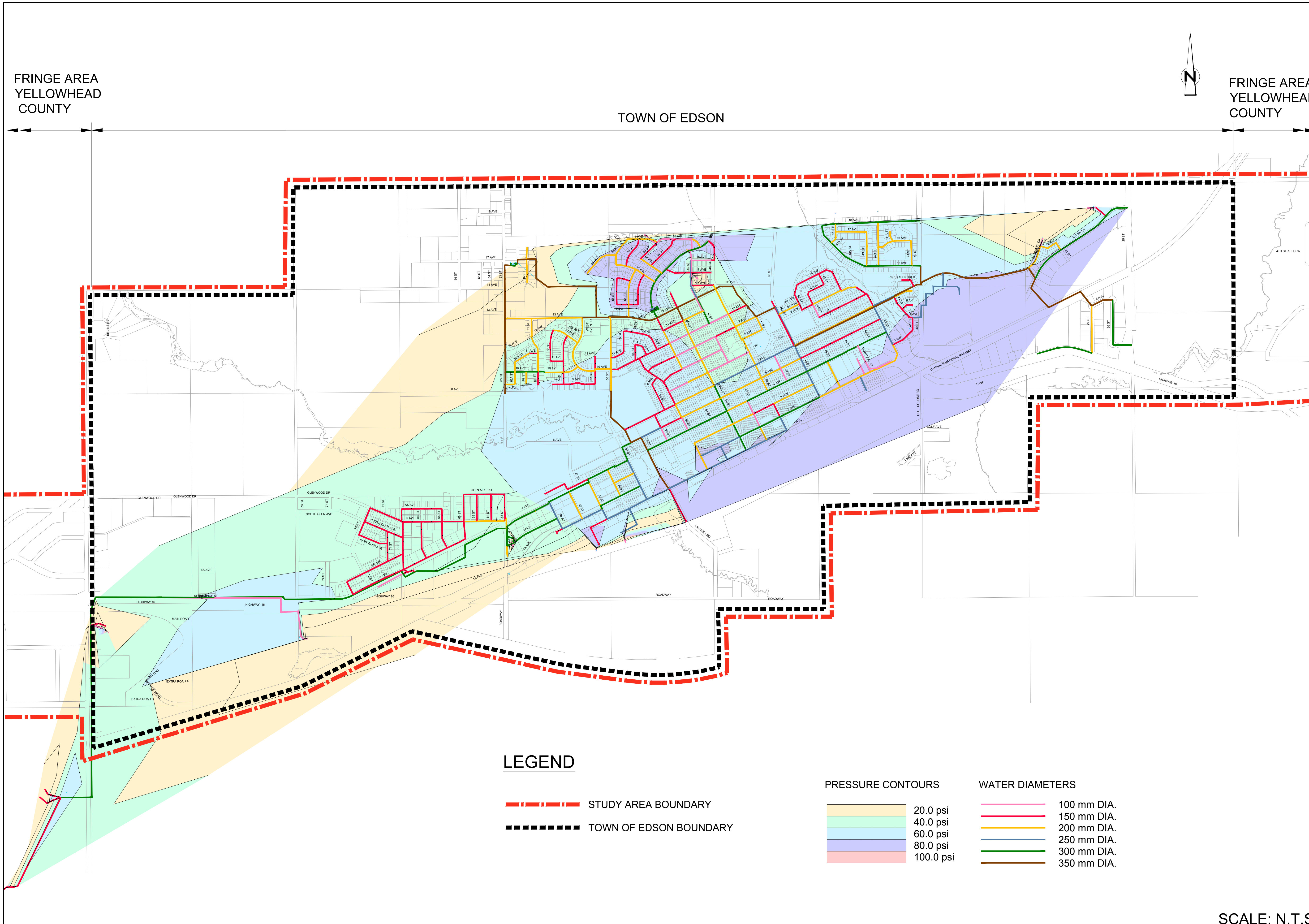

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Jul 4, 2018

FIGURE No. 3.2

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
EXISTING PEAK HOUR PRESSURE DEMAND - PRESSURE CONTOURS

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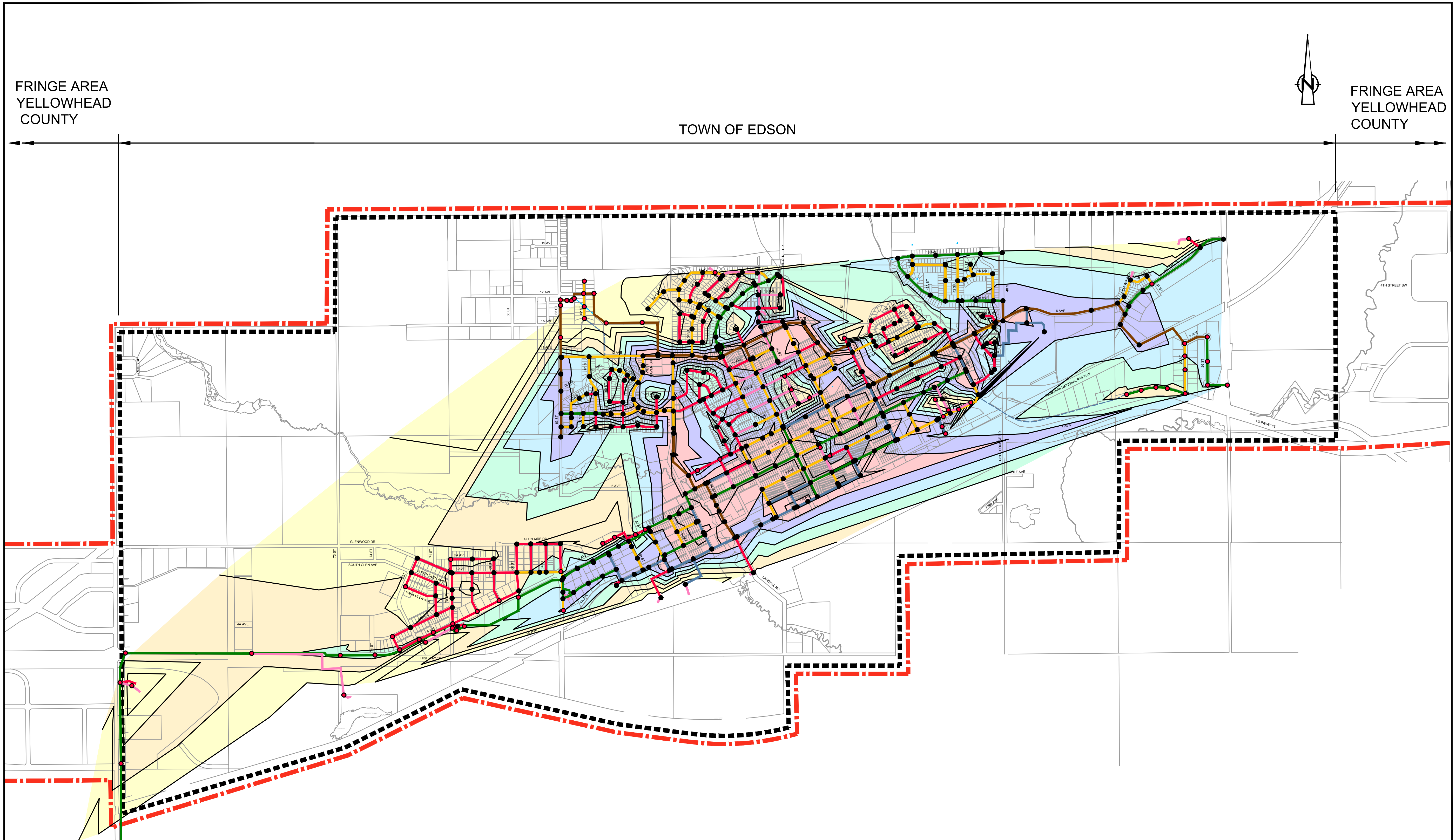

LEGEND

- SATISFIES FIRE FLOW CONSTRAINTS
- DOES NOT SATISFY FIRE FLOW CONSTRAINTS
- STUDY AREA BOUNDARY
- TOWN OF EDSON BOUNDARY

AVAILABLE FIRE FLOW CONTOURS

- 50.0 L/s
- 100.0 L/s
- 150.0 L/s
- 200.0 L/s
- 250.0 L/s
- 300.0 L/s

WATER DIAMETERS

- 100 mm DIA.
- 150 mm DIA.
- 200 mm DIA.
- 250 mm DIA.
- 300 mm DIA.
- 350 mm DIA.
- 500 mm DIA.

SCALE: N.T.S.

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Jul 4, 2018

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
EXISTING SYSTEM FIRE FLOW CONSTRAINTS

FIGURE No. 3.3

GHD

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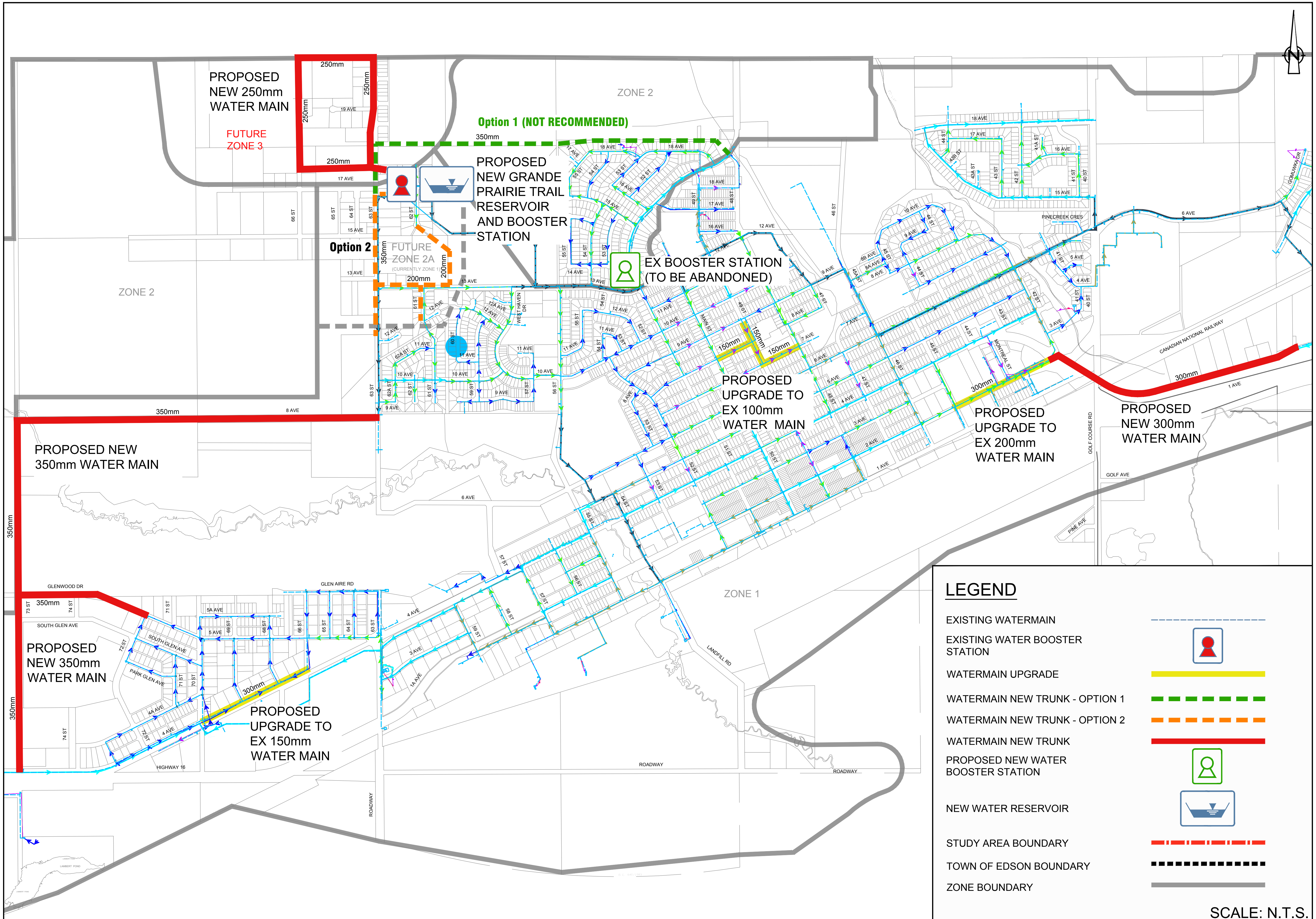

11148145-000
Jul 6, 2018

FIGURE No. 3.4

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
PROPOSED IMPROVEMENTS OF EXISTING WATER SYSTEM

GHD

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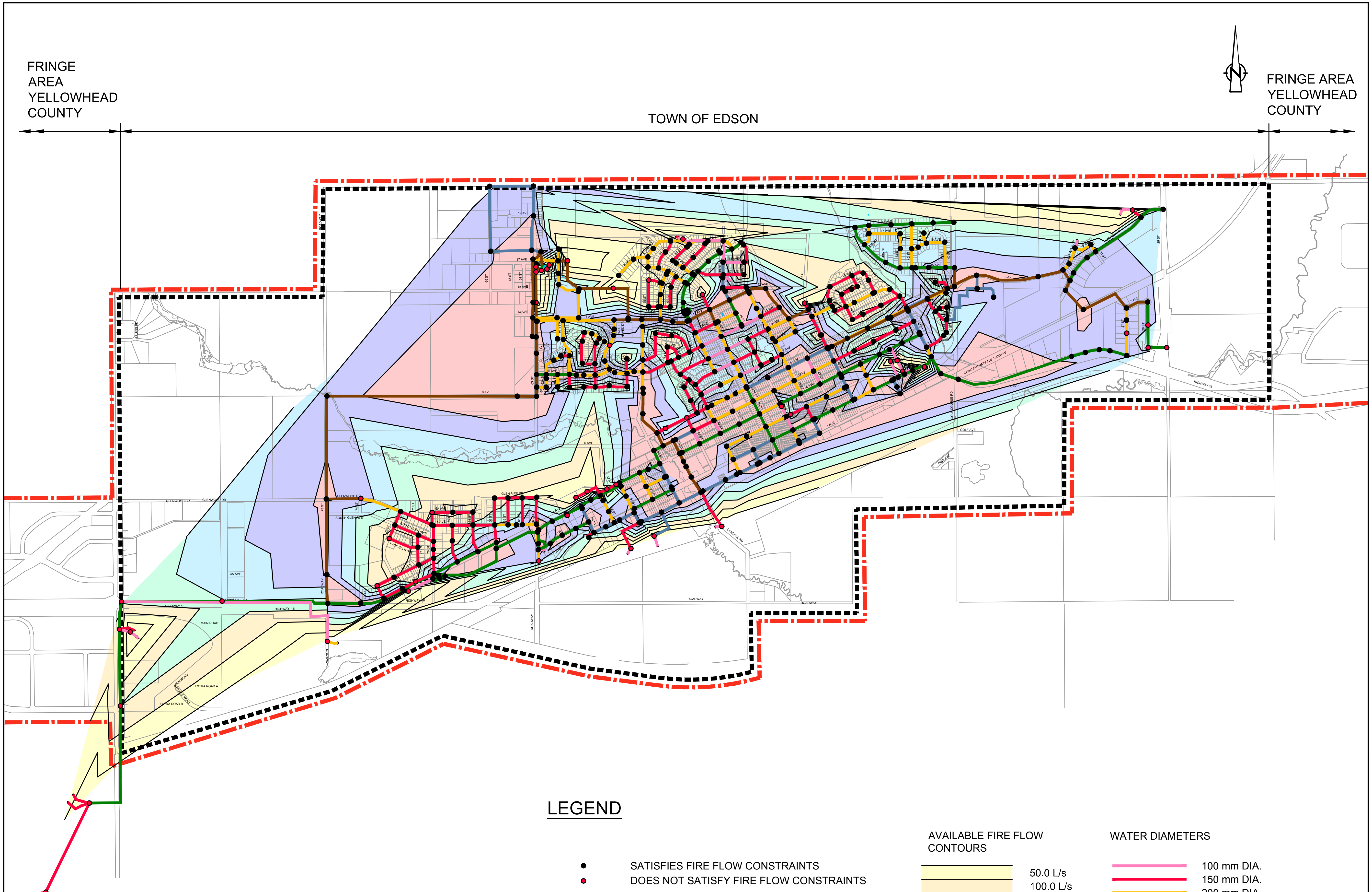

LEGEND

- SATISFIES FIRE FLOW CONSTRAINTS
- DOES NOT SATISFY FIRE FLOW CONSTRAINTS
- STUDY AREA BOUNDARY
- TOWN OF EDSON BOUNDARY

AVAILABLE FIRE FLOW CONTOURS

- 50.0 L/s
- 100.0 L/s
- 150.0 L/s
- 200.0 L/s
- 250.0 L/s
- 300.0 L/s

WATER DIAMETERS

- 100 mm DIA.
- 150 mm DIA.
- 200 mm DIA.
- 250 mm DIA.
- 300 mm DIA.
- 350 mm DIA.
- 500 mm DIA.

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Jul 4, 2018

FIGURE No. 3.5

TOWN OF EDSON
EDSON, ALBERTA
EDSON MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
IMPROVED EXISTING SYSTEM FIRE FLOW CONSTRAINTS

GHD

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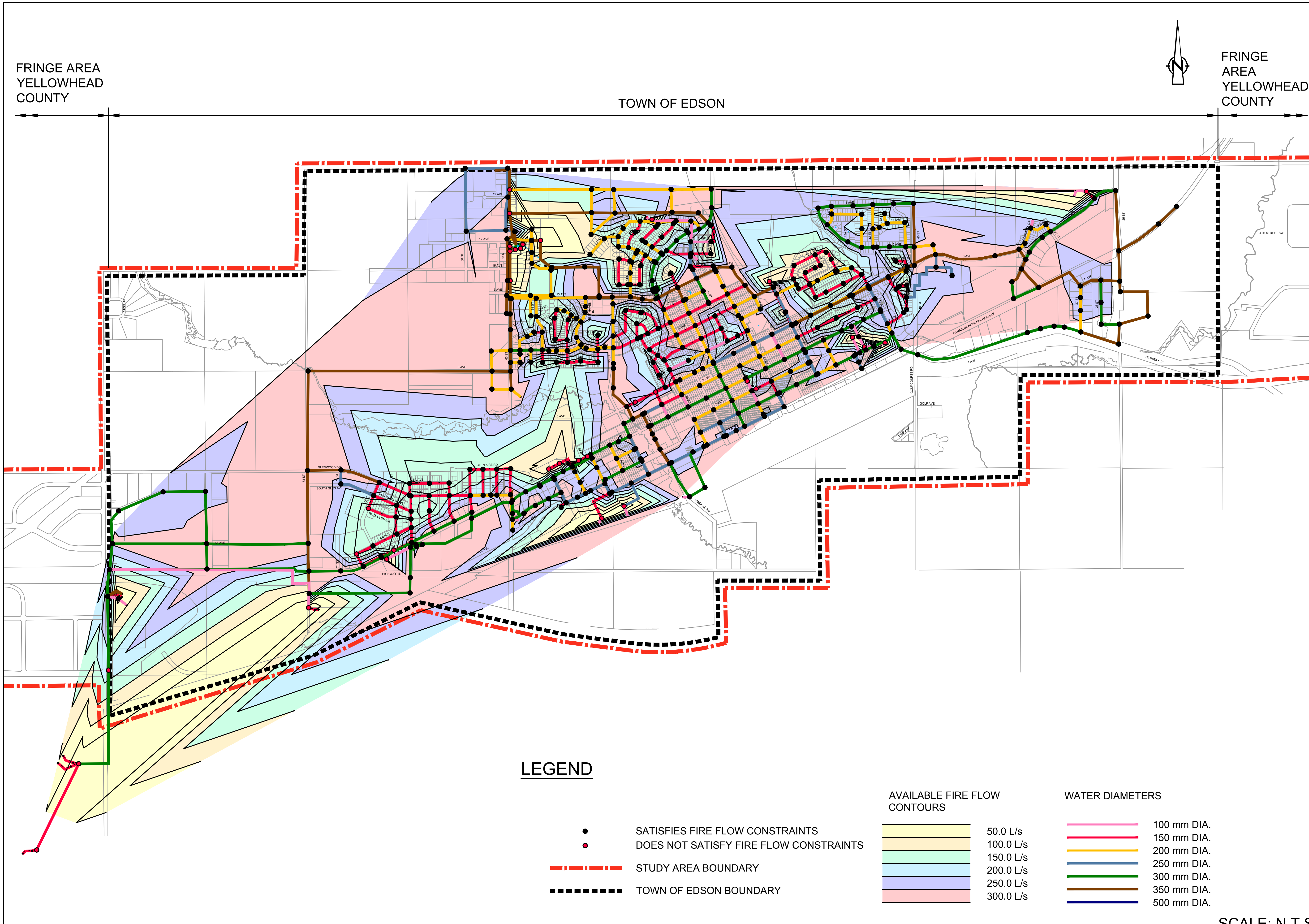

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TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
2032 TOWN ONLY SYSTEM FIRE FLOW CONSTRAINTS

FIGURE No. 3.6A

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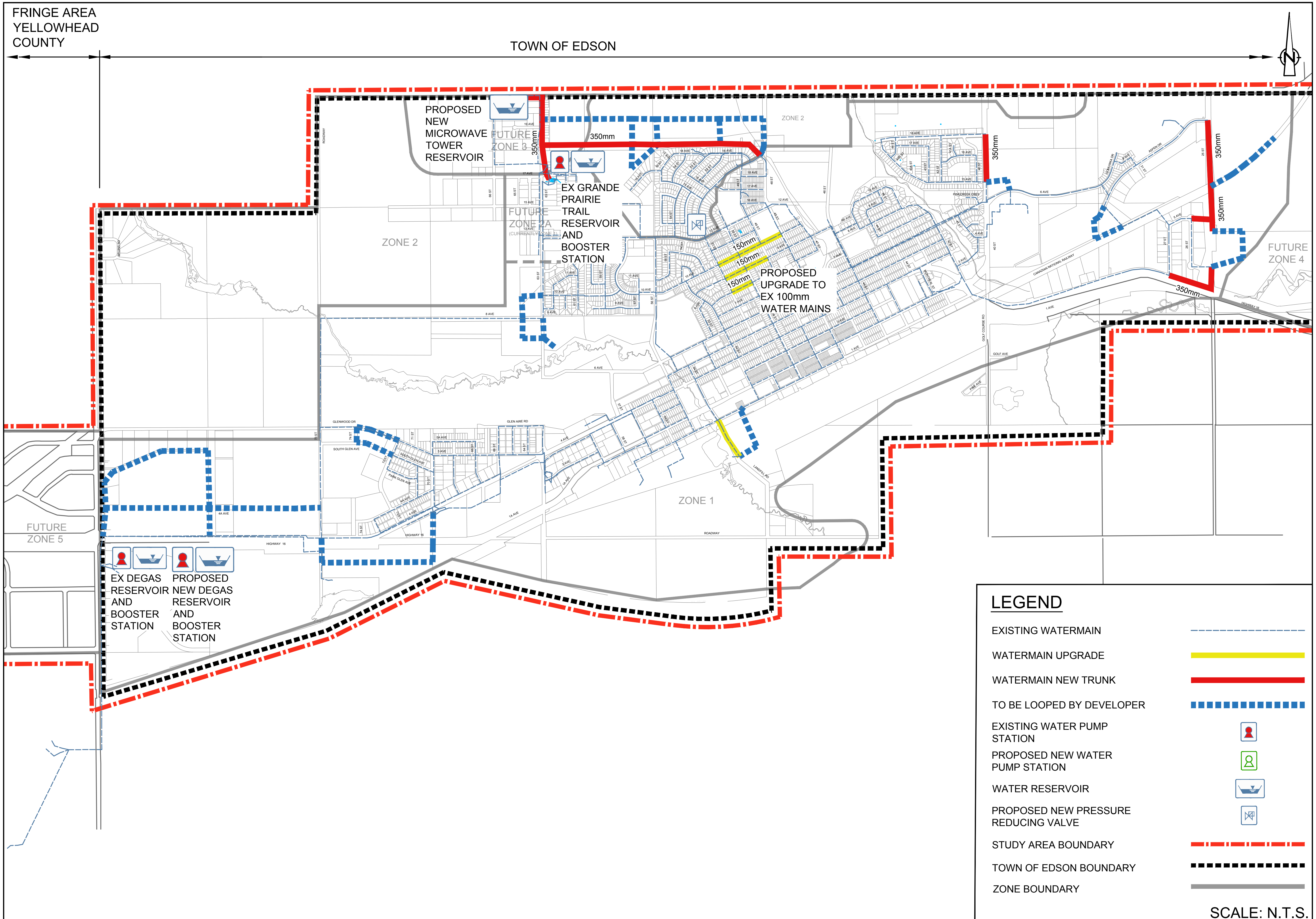

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FIGURE No. 3.7A

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM

WATER DISTRIBUTION IMPROVEMENTS 2032 (TOWN ONLY, 15 YEAR PROJECTION)

GHD

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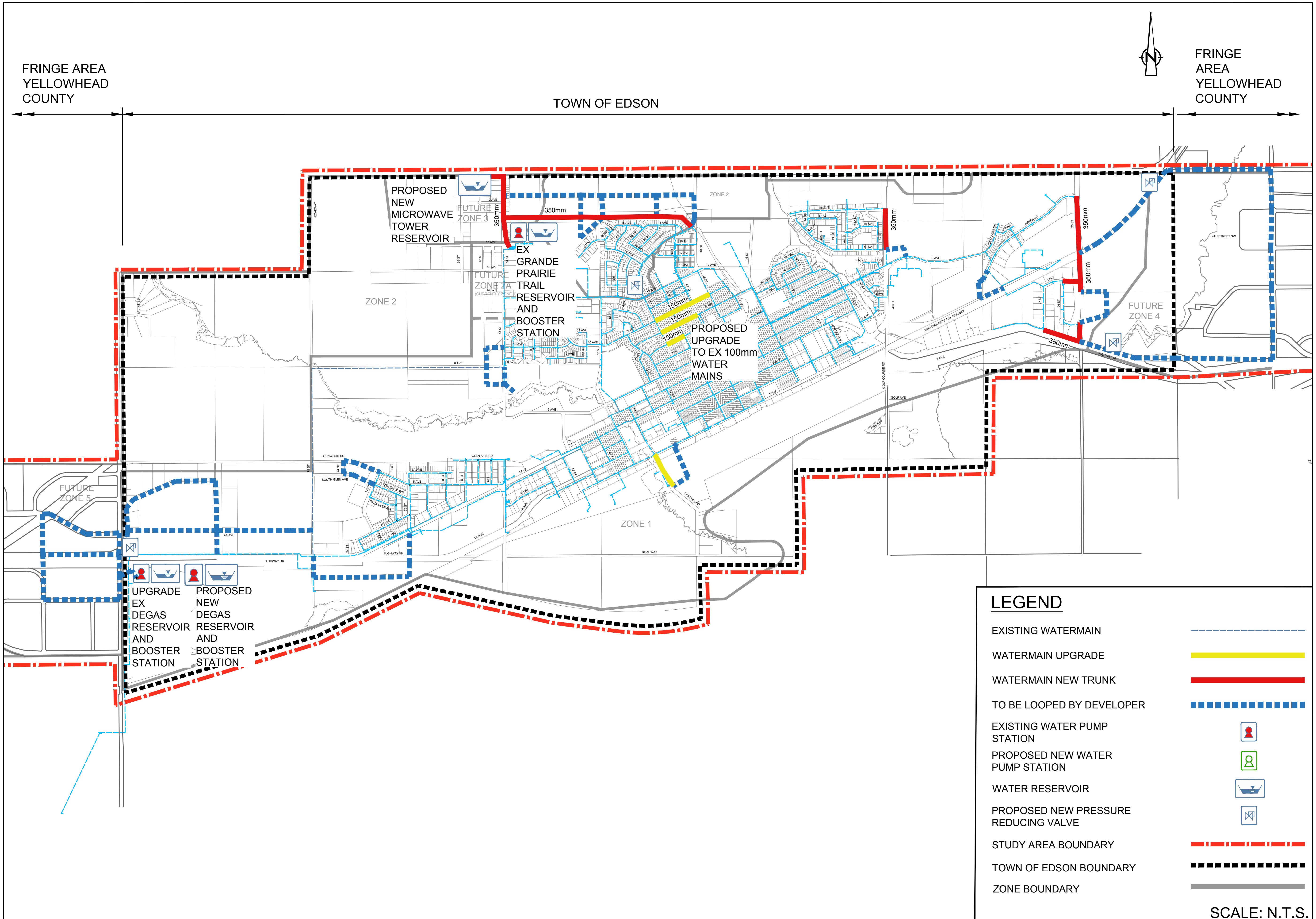

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TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
WATER DISTRIBUTION IMPROVEMENTS 2032 (WITH FRINGE, 15 YEAR PROJECTION) FIGURE No. 3.7B

GHD

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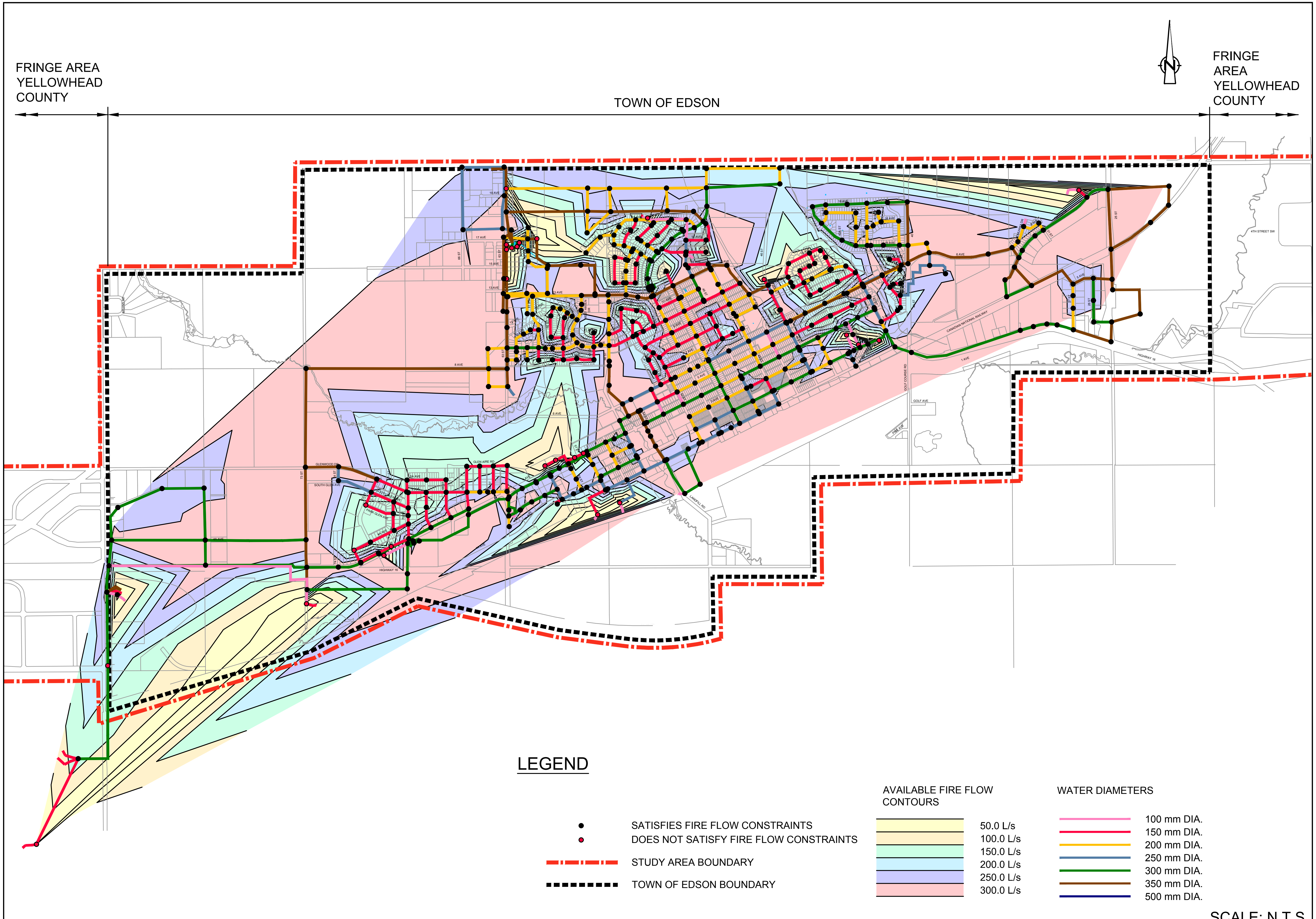

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TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
2042 TOWN ONLY SYSTEM FIRE FLOW CONSTRAINTS

FIGURE No. 3.8A

GHD

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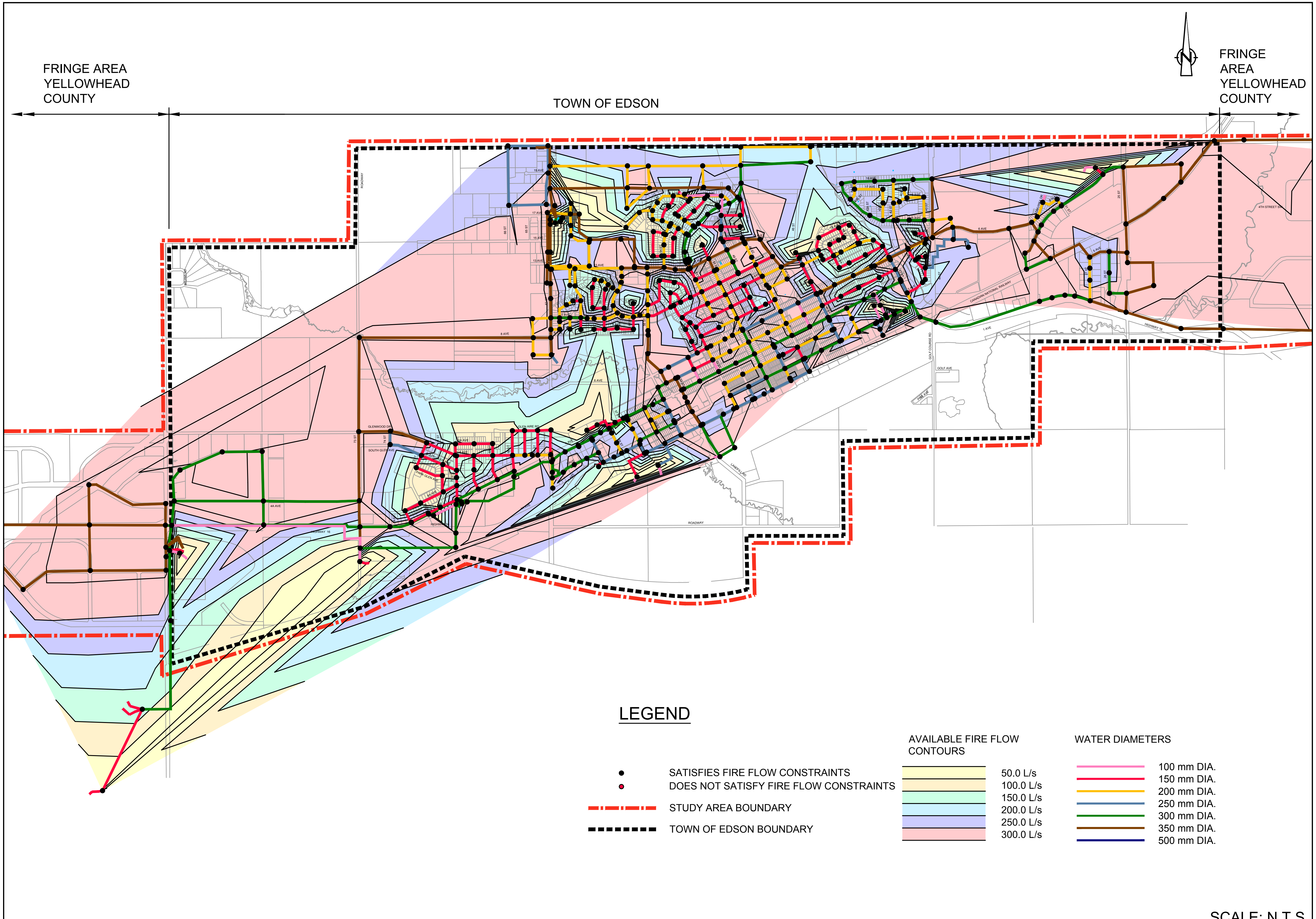

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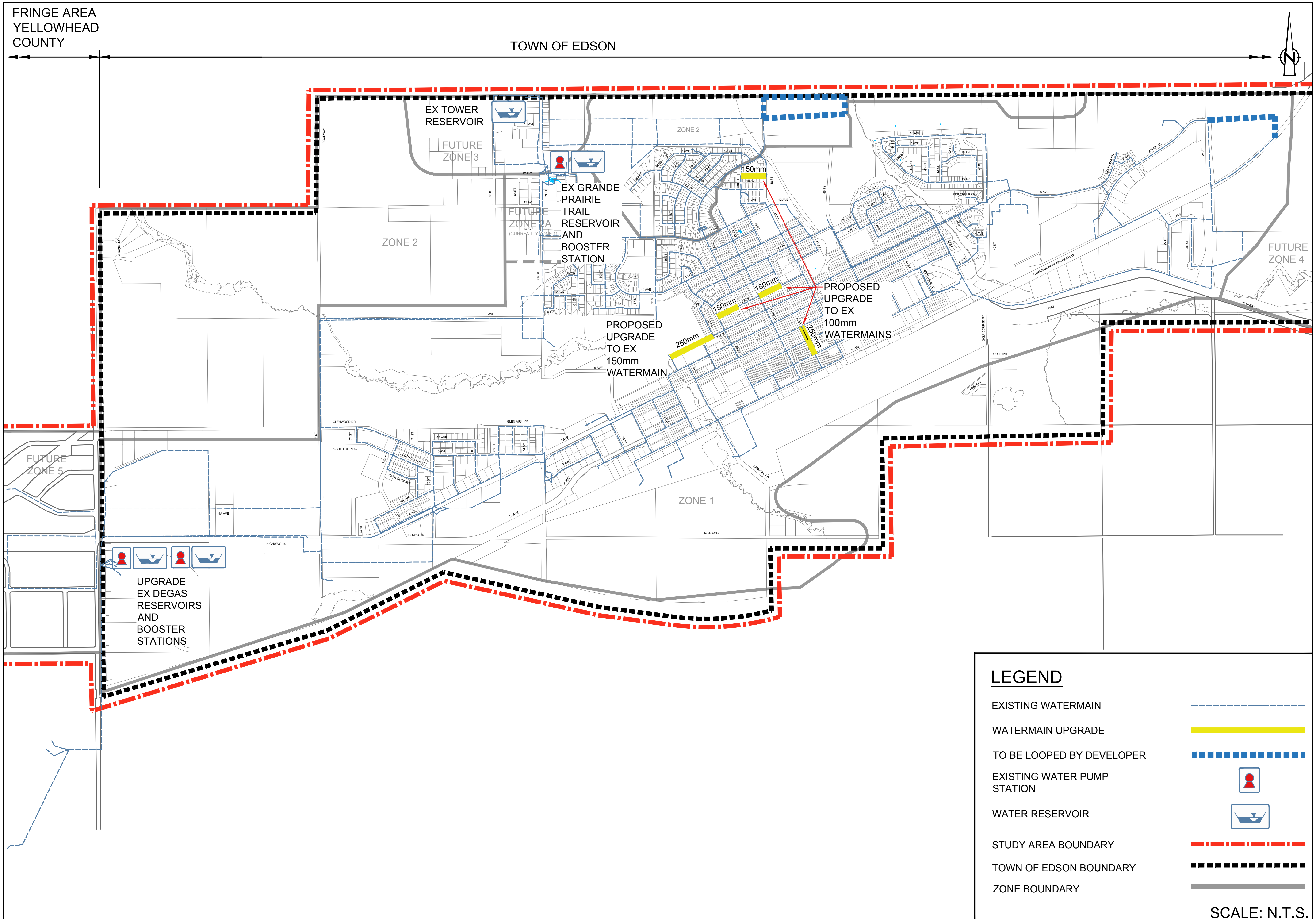
Jul 5, 2018

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
2042 TOWN & FRINGE SYSTEM FIRE FLOW CONSTRAINTS

FIGURE No. 3.8B



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TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
WATER DISTRIBUTION IMPROVEMENTS 2042 (TOWN ONLY, 25 YEAR PROJECTION)

FIGURE No. 3.9A

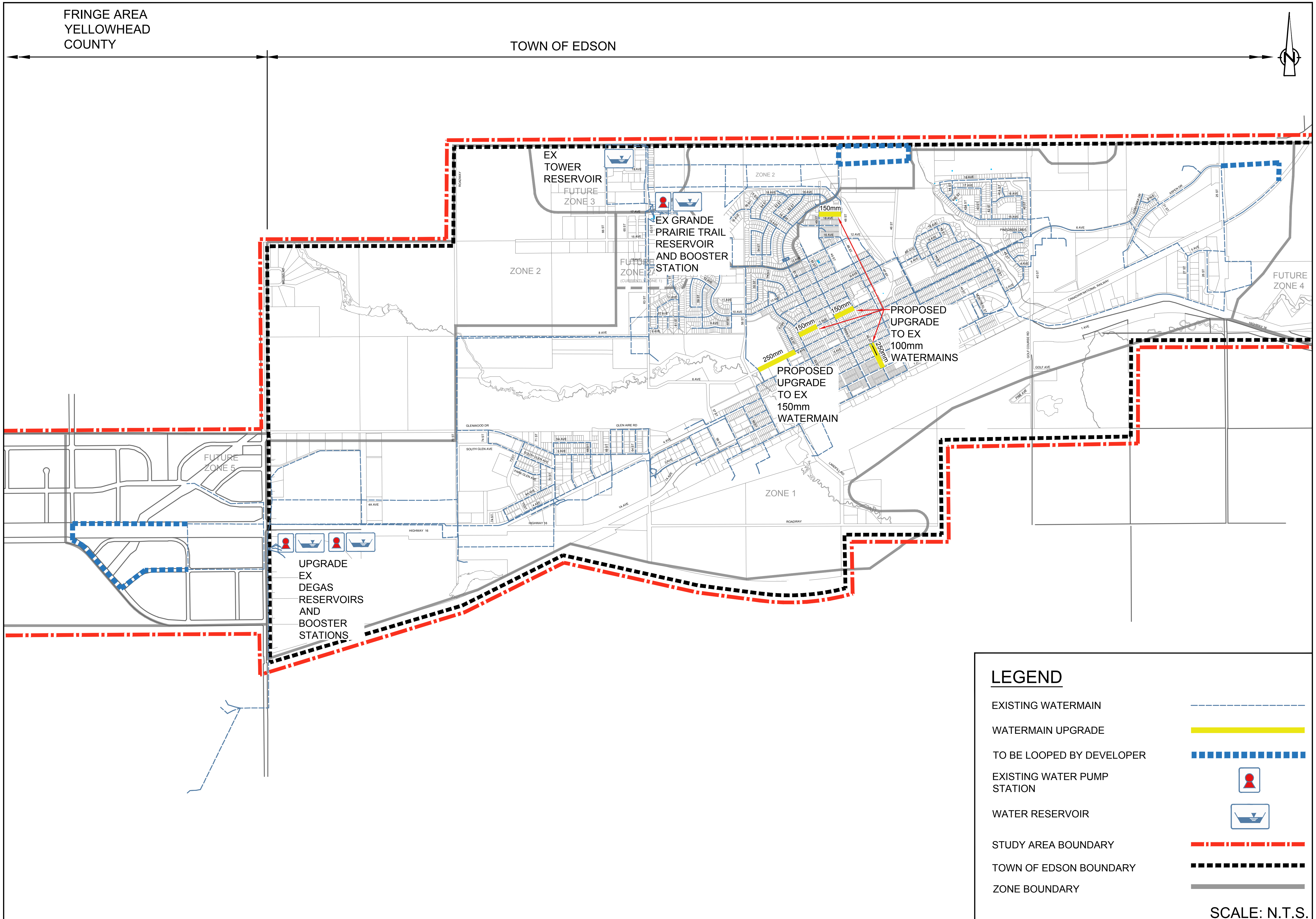

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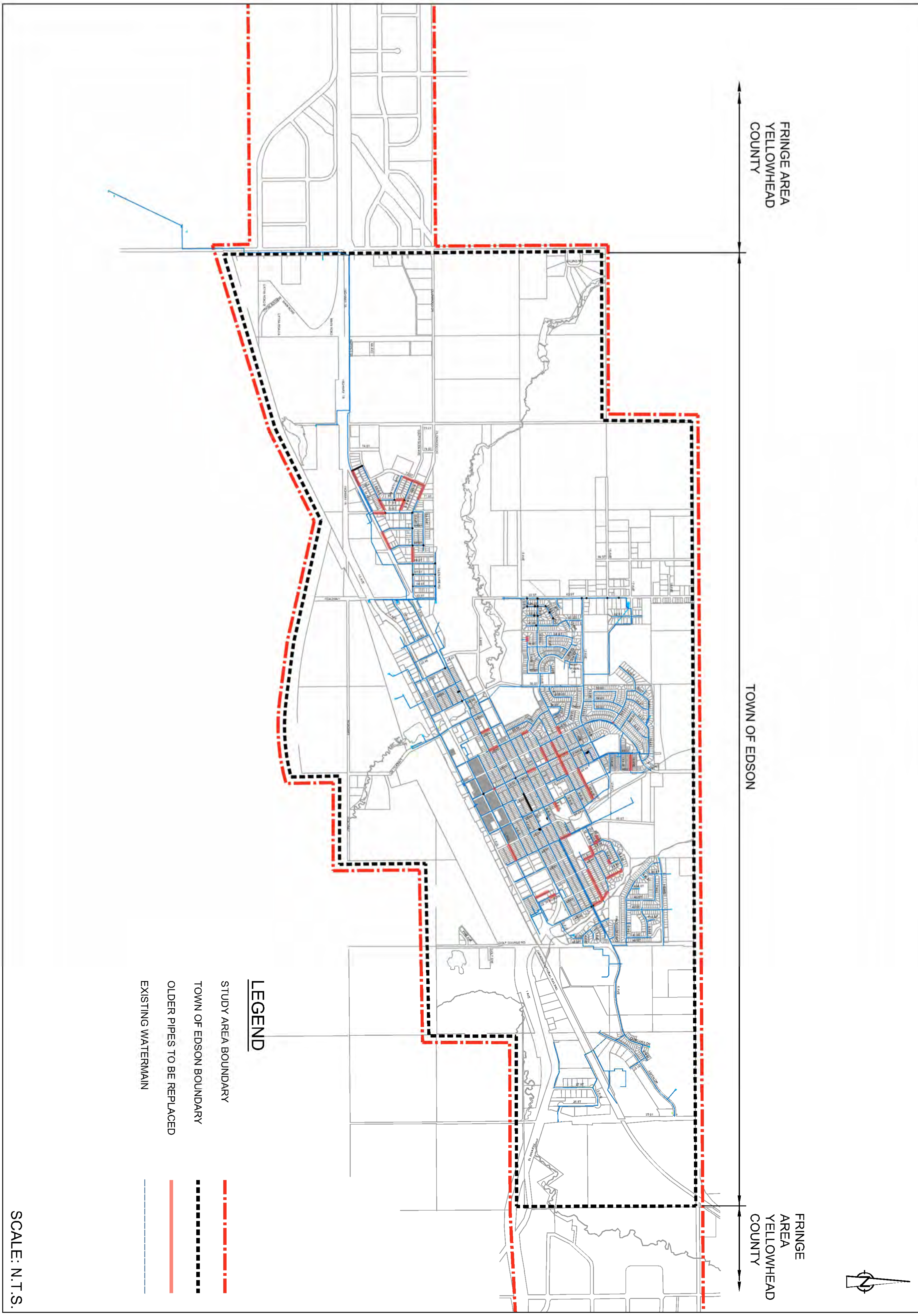
FIGURE No. 3.9B

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE
WATER SYSTEM
WATER DISTRIBUTION IMPROVEMENTS 2042 (TOWN & FRINGE, 25 YEAR PROJECTION)



TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WATER SYSTEM
WATER SYSTEM - OLDER PIPES TO BE REPLACED

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FIGURE No. 3.10

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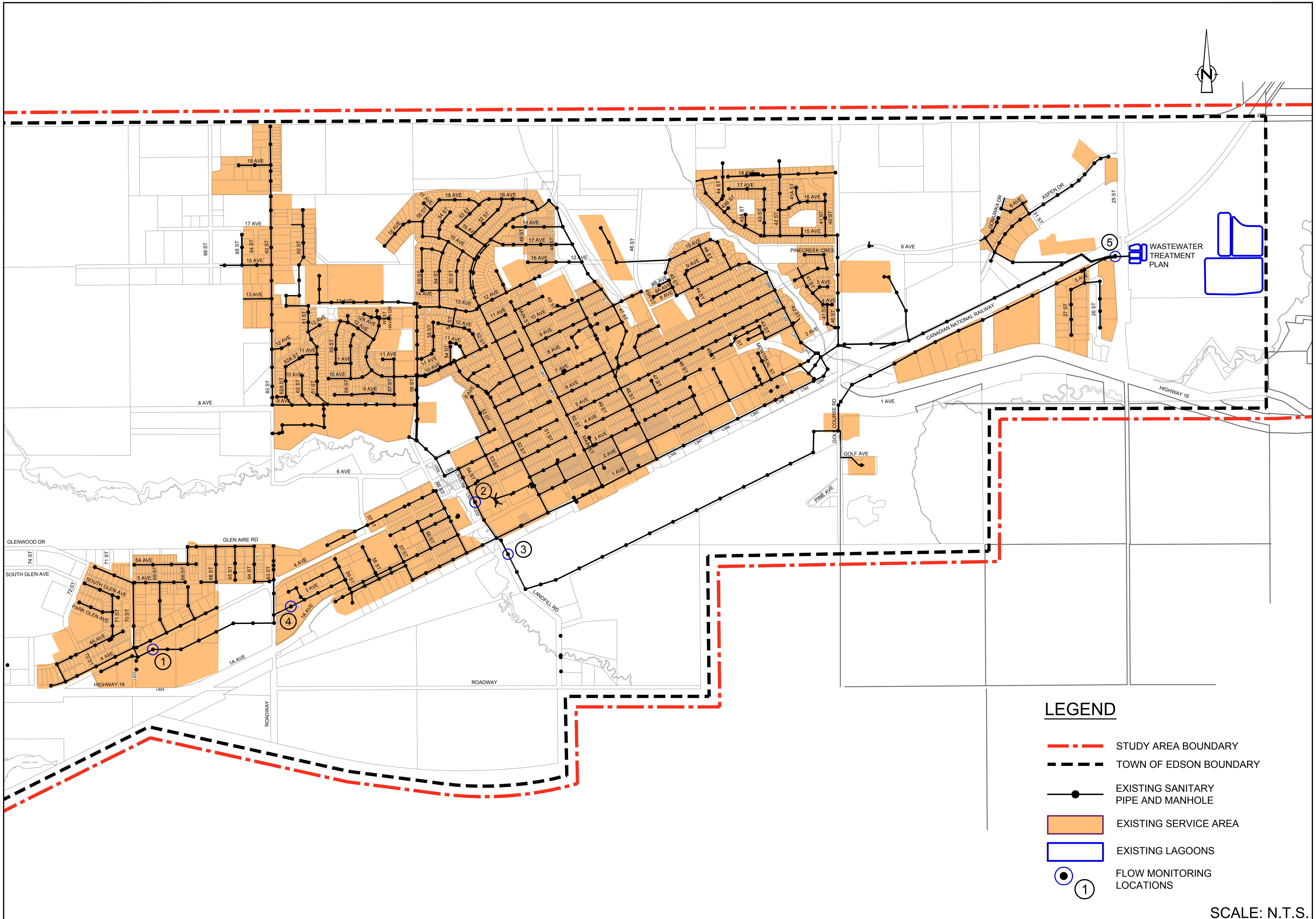

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FIGURE No. 4.1

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WASTEWATER SYSTEM
EXISTING SANITARY SYSTEM

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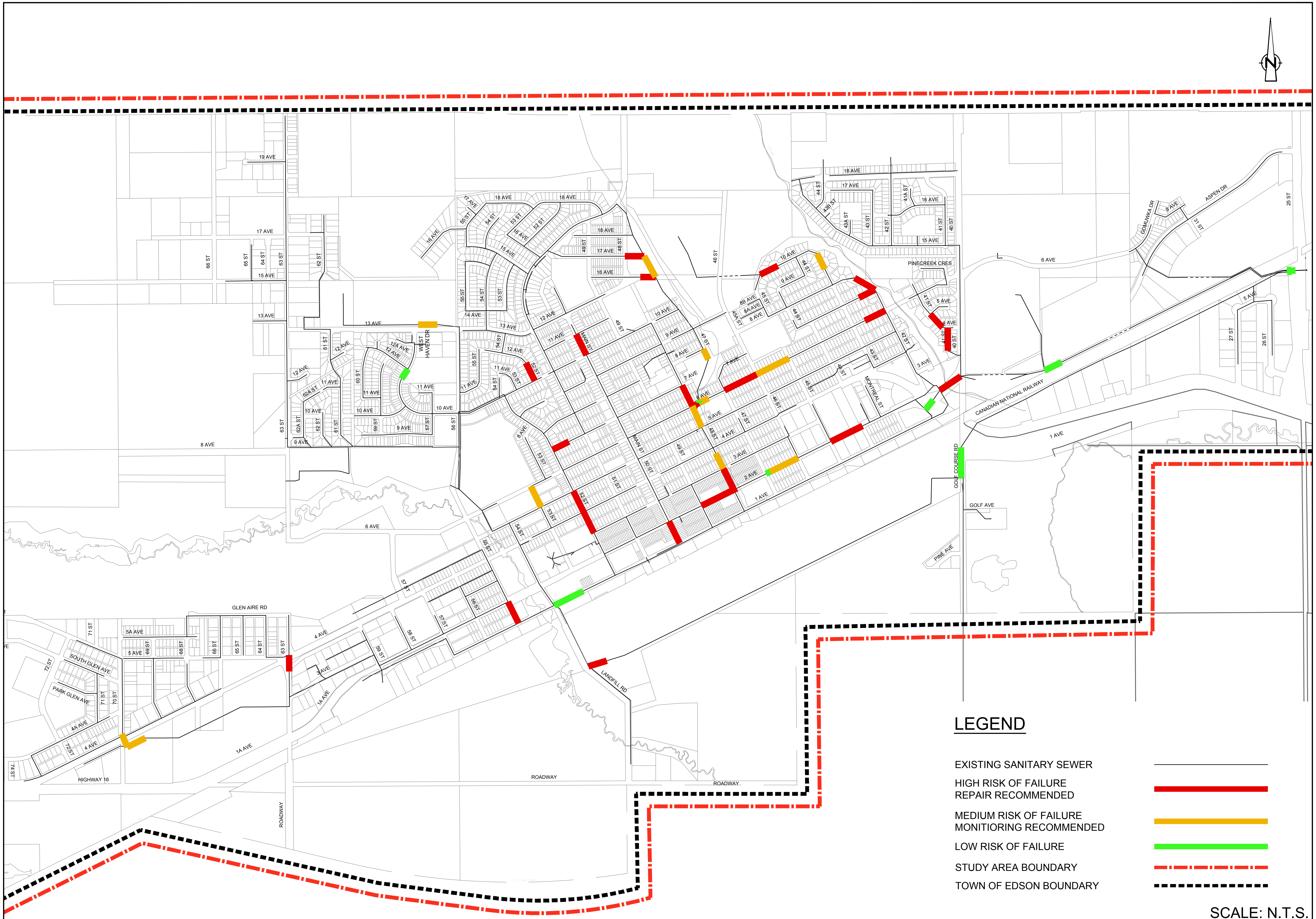

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Jul 5, 2018

FIGURE No. 4.2

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WASTEWATER SYSTEM
CCTV INVESTIGATION (2017)

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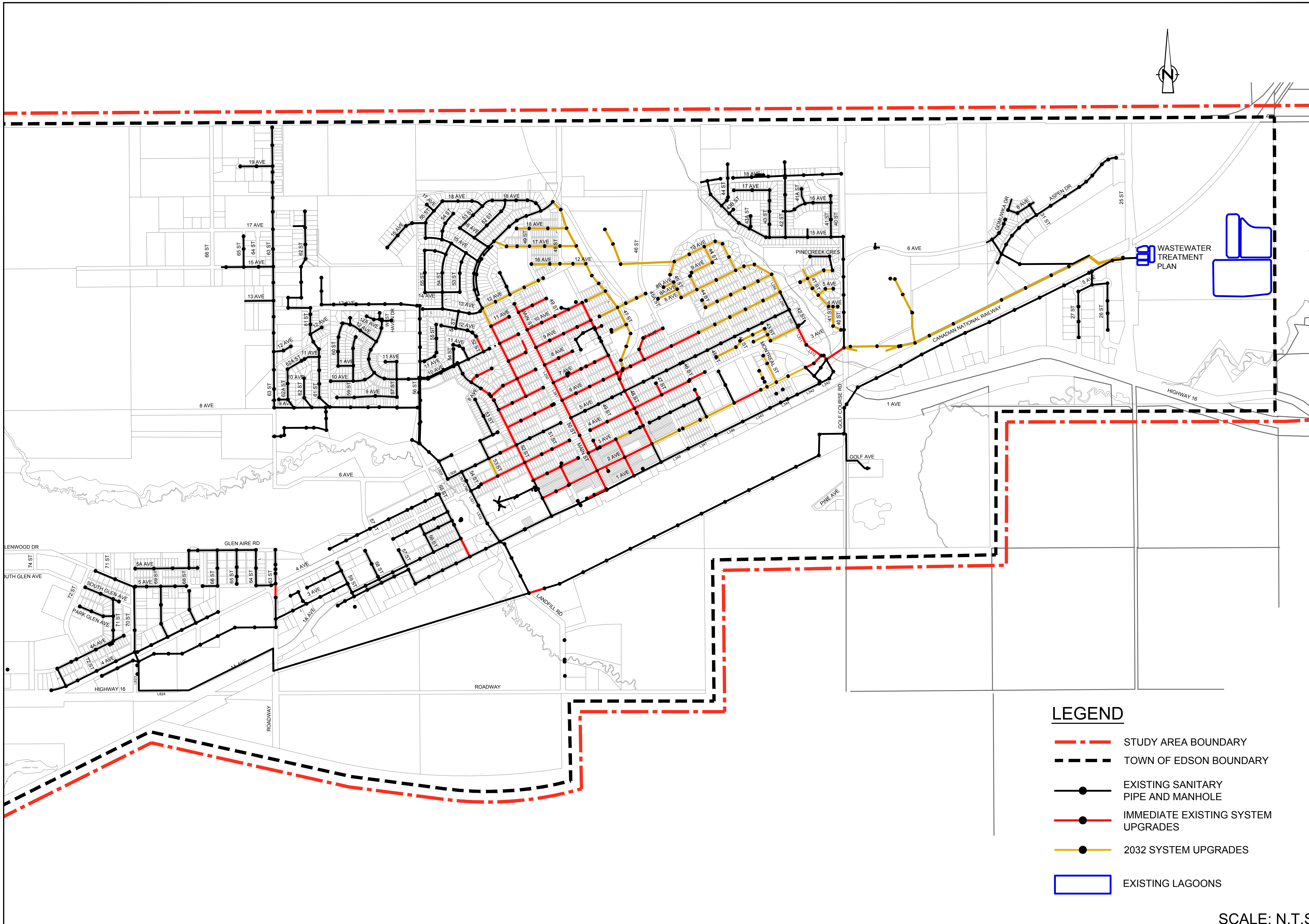

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FIGURE No. 4.3

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WASTEWATER SYSTEM
IMPROVEMENTS BASED ON CONDITION ASSESSMENT

GHD

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TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WASTEWATER SYSTEM
IMPROVEMENTS BASED ON CAPACITY ISSUES

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FIGURE No. 4.4

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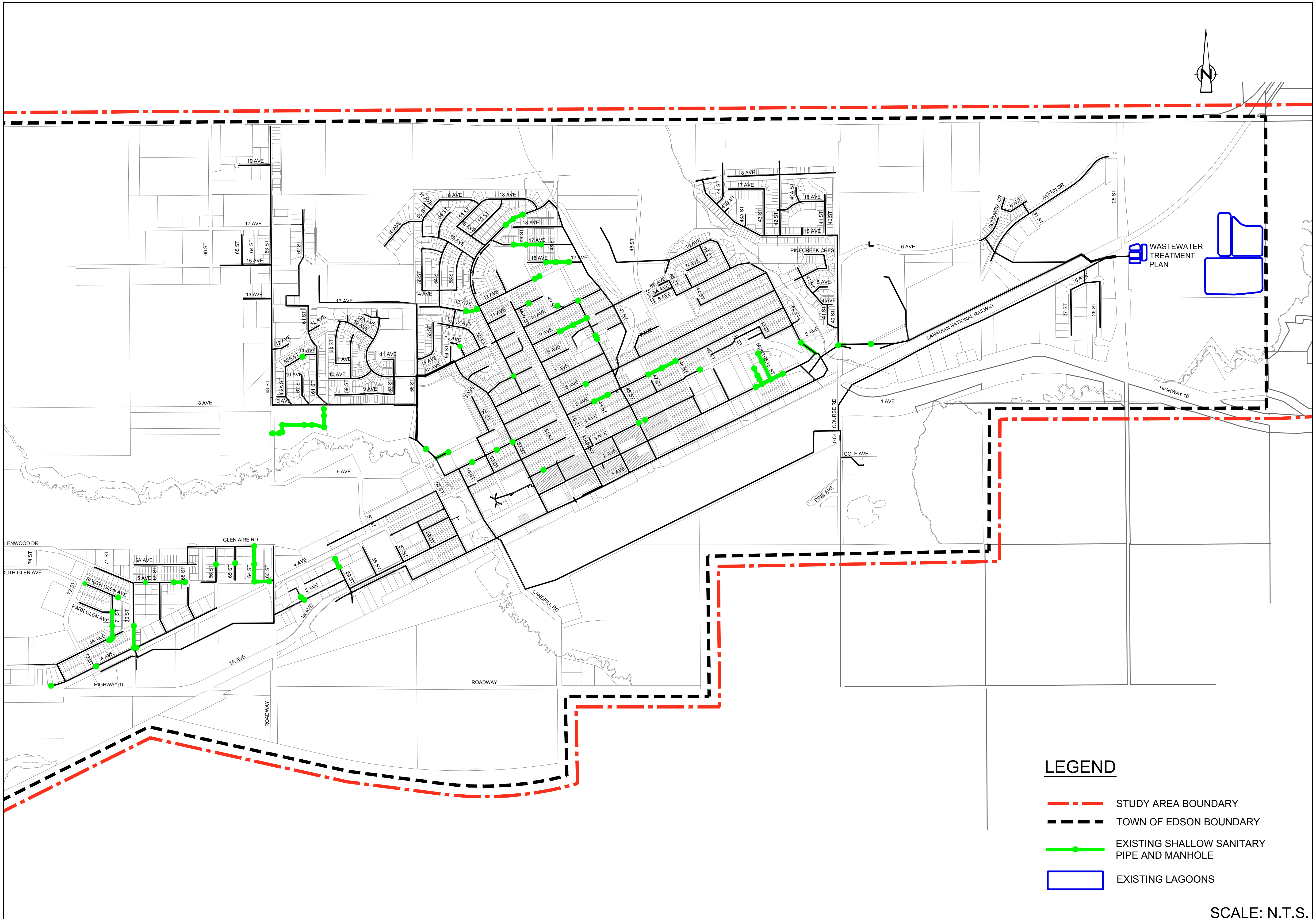

11148145-000

Jul 5, 2018

FIGURE No. 4.5

TOWN OF EDSON
EDSON, ALBERTA
EDSON MUNICIPAL SERVICING PLAN UPDATE

WASTEWATER SYSTEM
EXISTING SANITARY SEWER WITH BURY DEPTH < 2.50m

GHD

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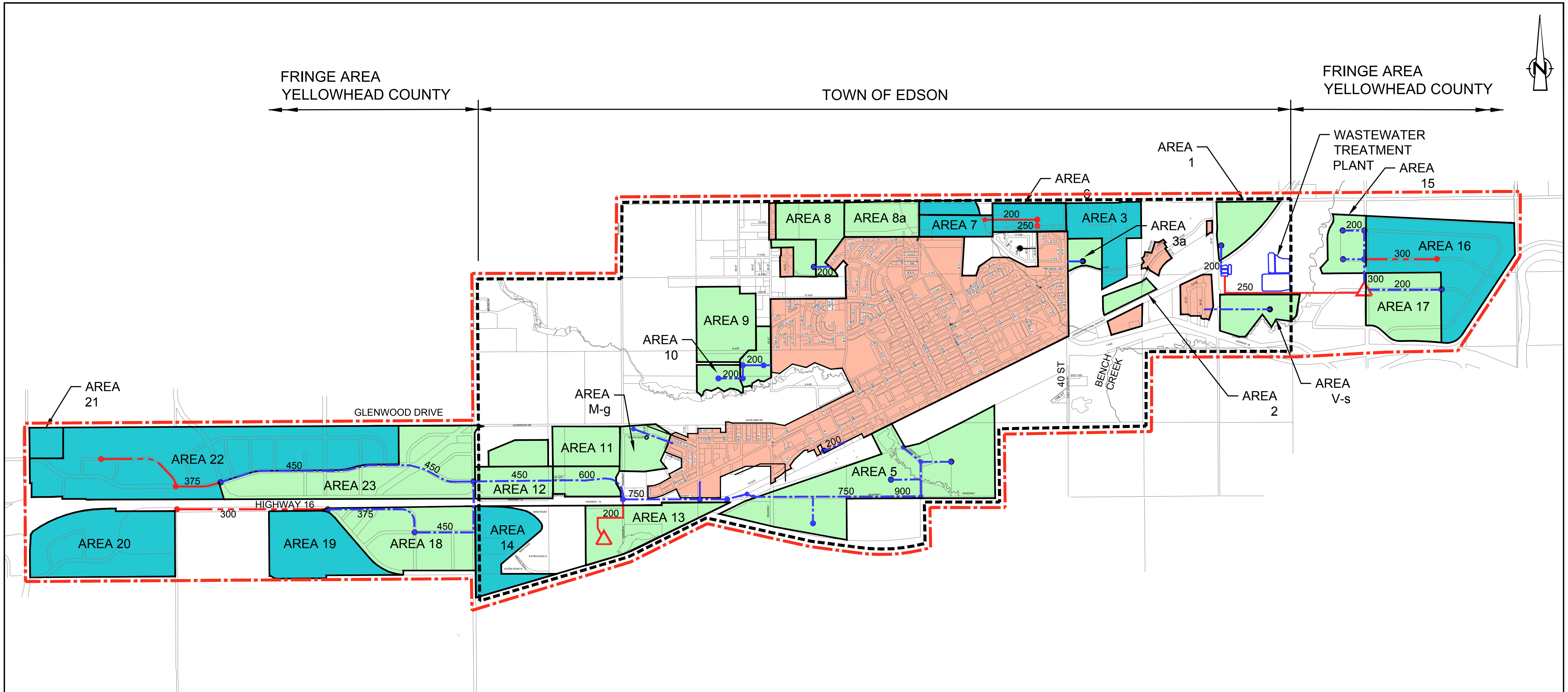

LEGEND

STUDY AREA BOUNDARY	EXISTING SERVICE AREA
TOWN OF EDSON BOUNDARY	2032 PROPOSED DEVELOPMENT
UPGRADED EXISTING SYSTEM	2042 PROPOSED DEVELOPMENT
FUTURE PIPE 2032	
FUTURE PIPE 2042	
FUTURE FORCEMAIN	
FUTURE LIFT STATION	

SCALE: N.T.S.

11148145-000
Jul 5, 2018

FIGURE No. 4.6

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

WASTEWATER SYSTEM
FUTURE DEVELOPMENT AREAS - 2032 & 2042

GHD

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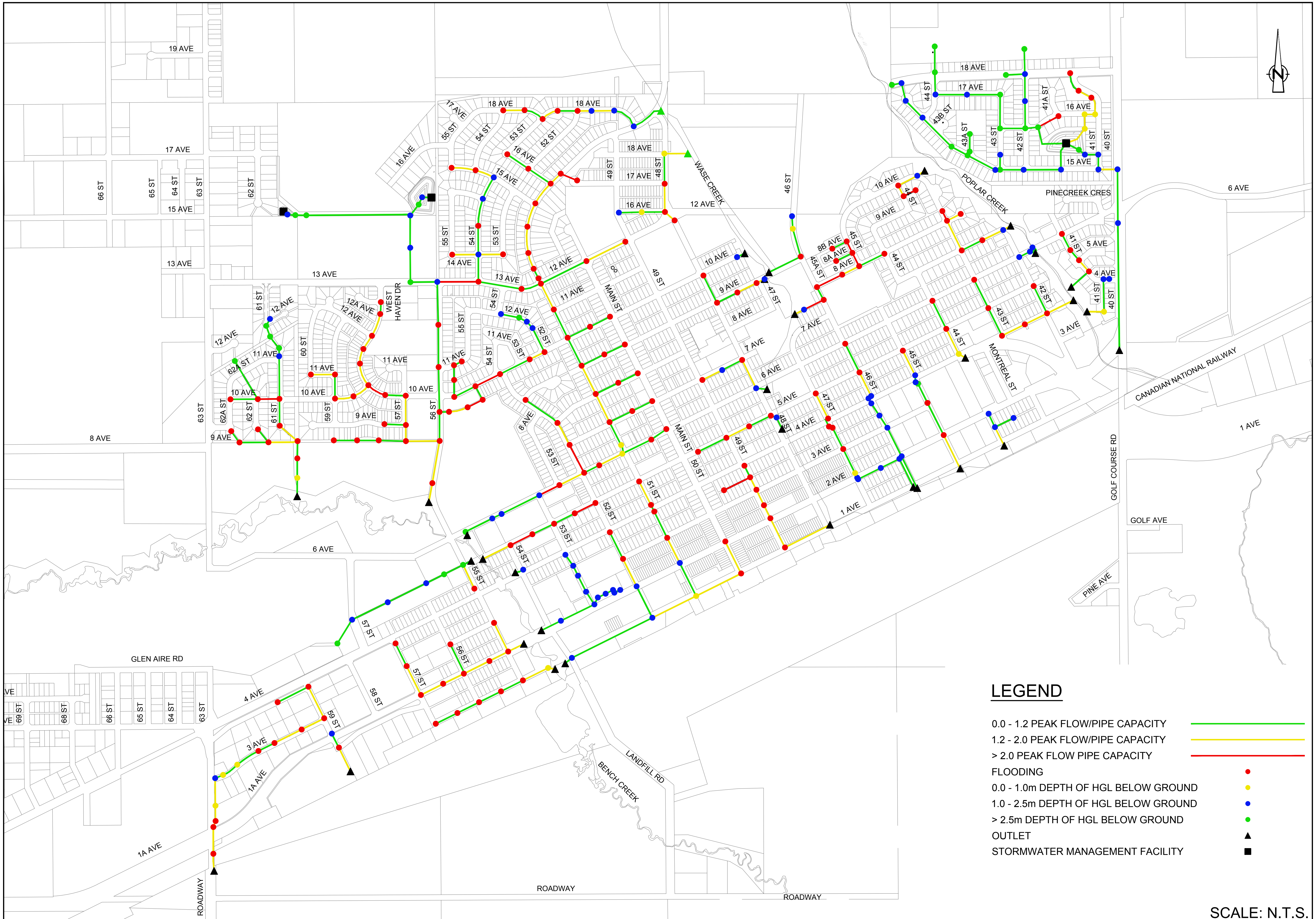

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

STORM SYSTEM
EXISTING SYSTEM - 5 YEAR, 4 HOUR RAINFALL EVENT

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Jul 5, 2018

FIGURE No. 5.1

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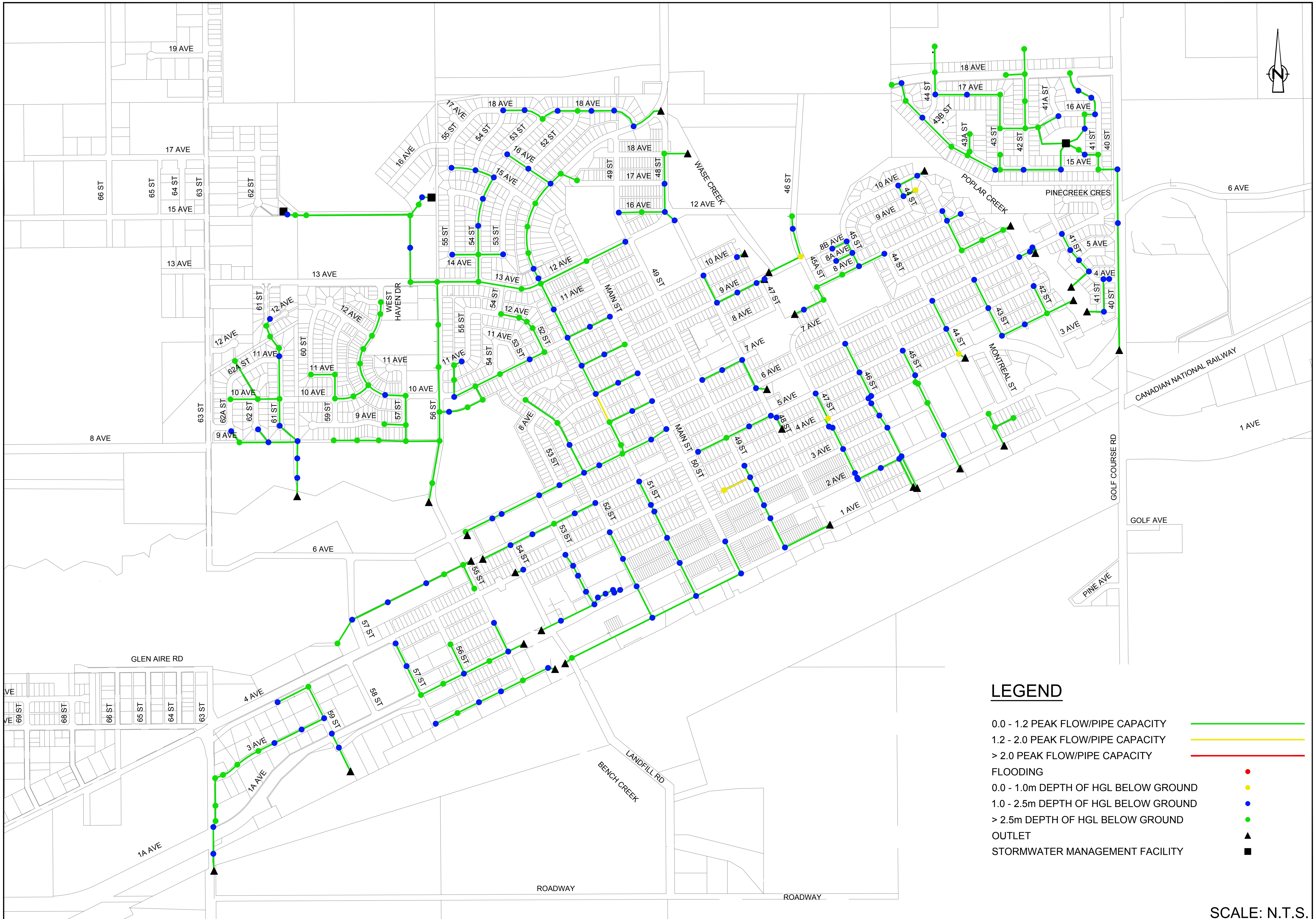

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Jul 5, 2018

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

STORM SYSTEM
EXISTING SYSTEM - 5 YEAR, 24 HOUR RAINFALL EVENT

FIGURE No. 5.2

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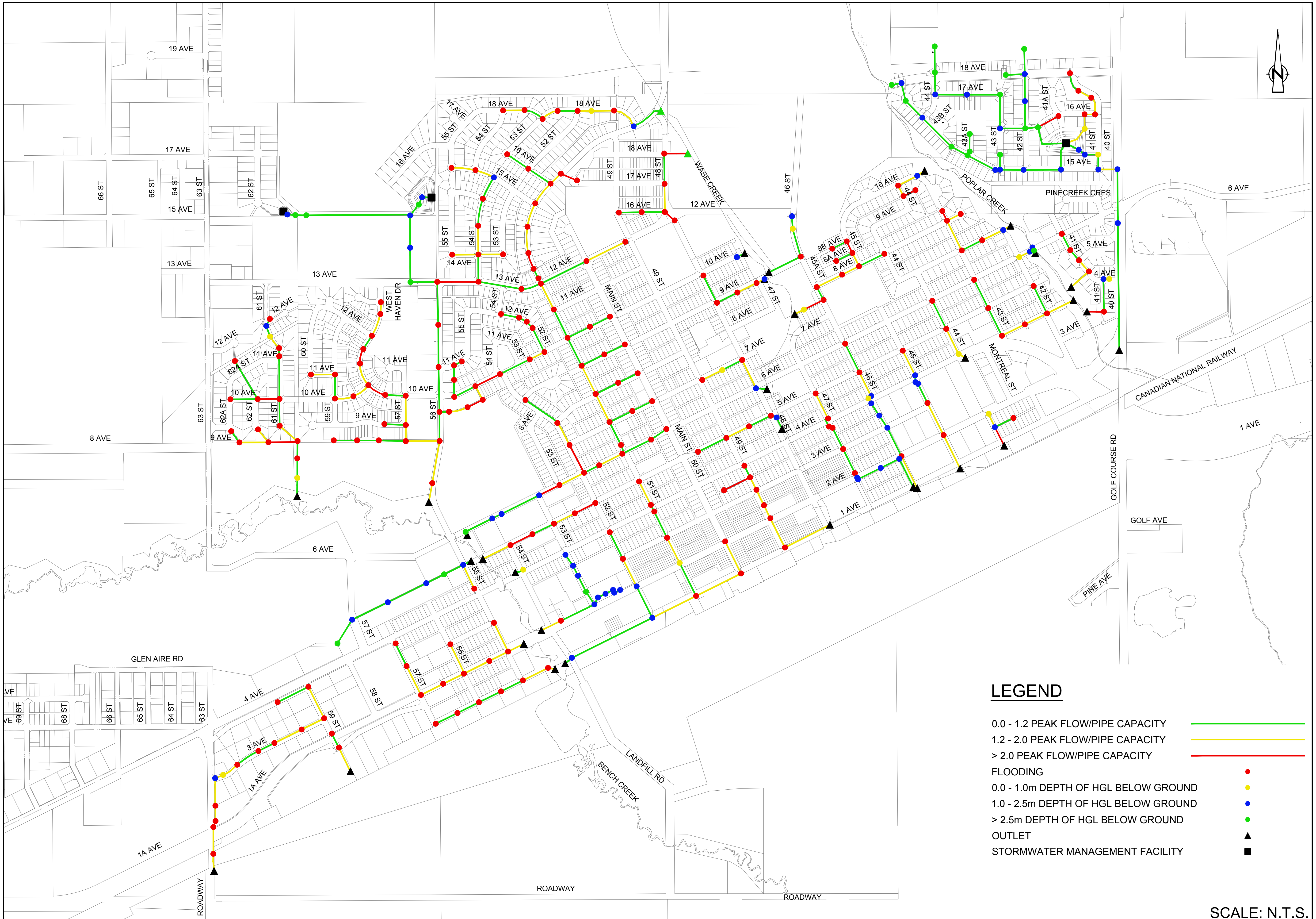

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Jul 5, 2018

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

STORM SYSTEM
EXISTING SYSTEM - 25 YEAR, 4 HOUR RAINFALL EVENT

FIGURE No. 5.3

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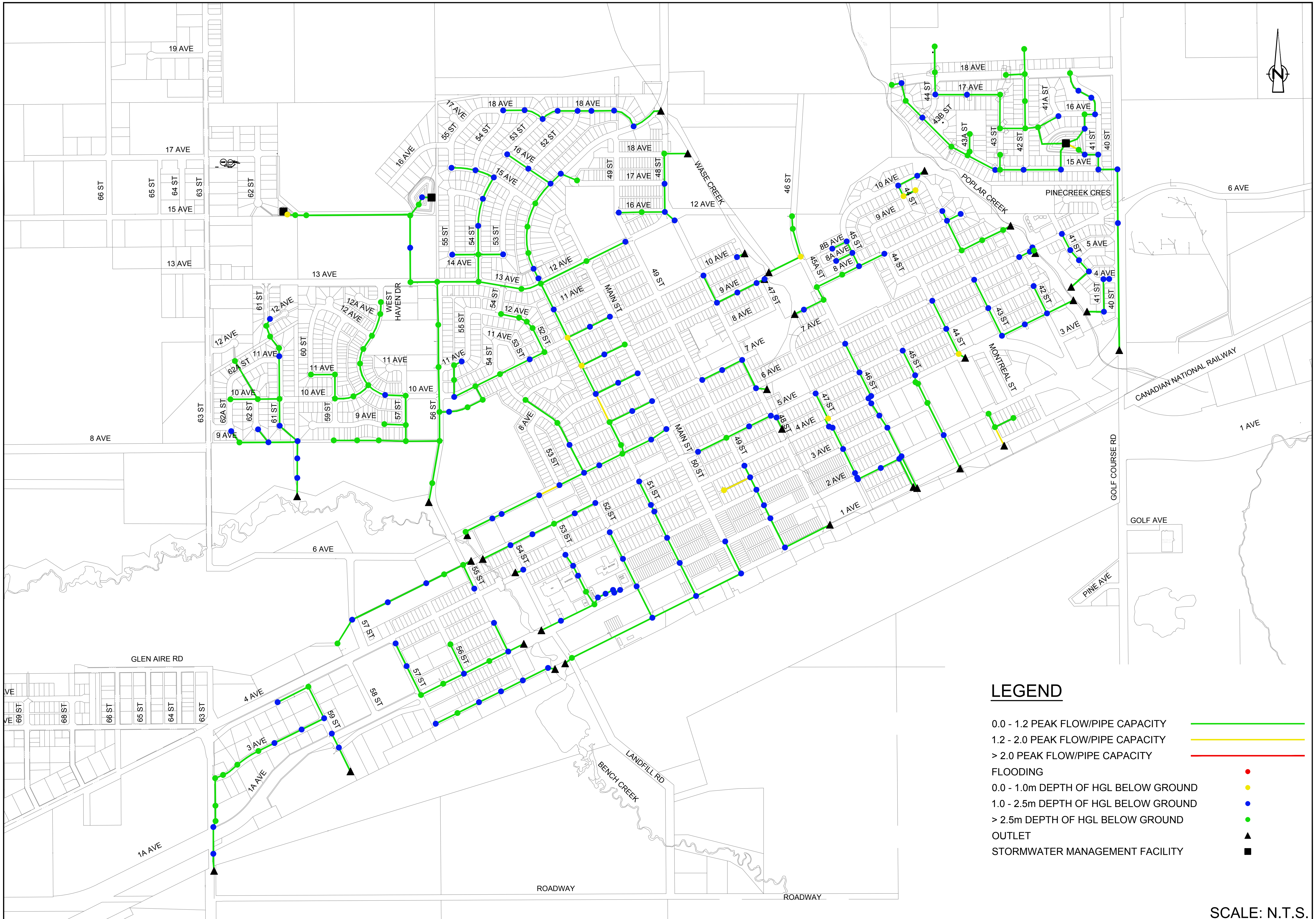

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TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

STORM SYSTEM
EXISTING SYSTEM - 25 YEAR, 24 HOUR RAINFALL EVENT

FIGURE No. 5.4

GHD

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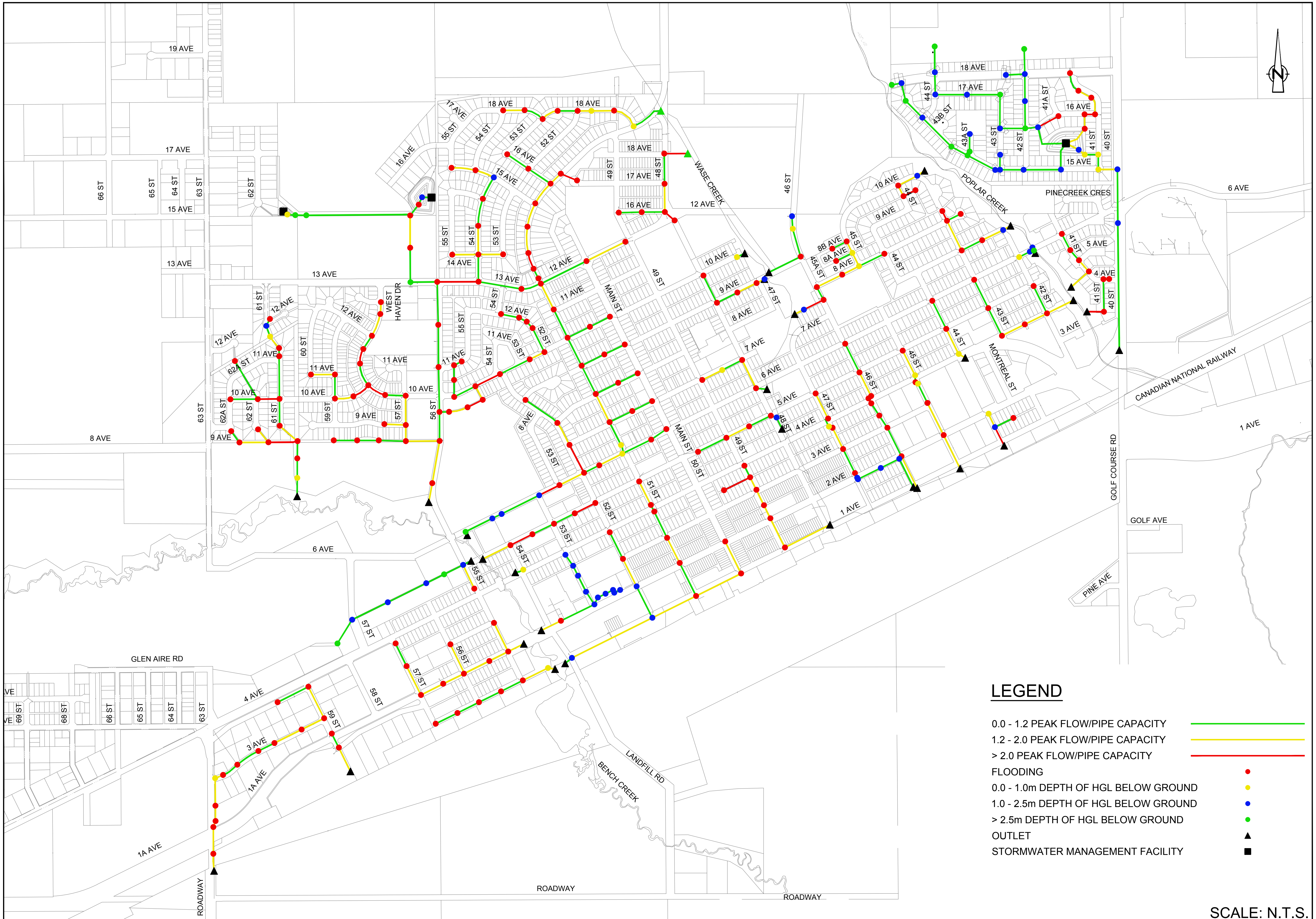

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

STORM SYSTEM
EXISTING SYSTEM - 100 YEAR, 4 HOUR RAINFALL EVENT

FIGURE No. 5.5

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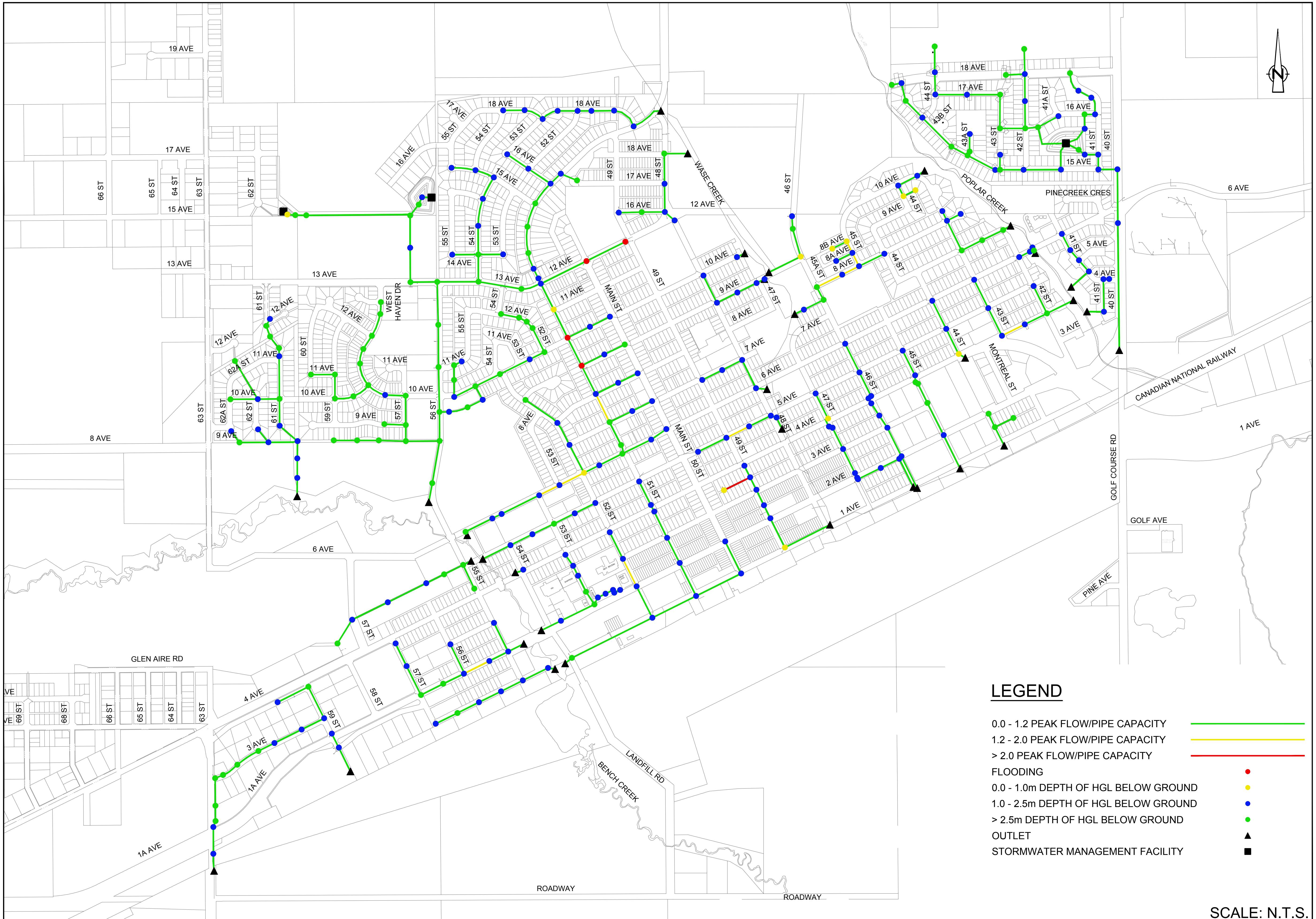

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

STORM SYSTEM
EXISTING SYSTEM - 100 YEAR, 24 HOUR RAINFALL EVENT FIGURE No. 5.6

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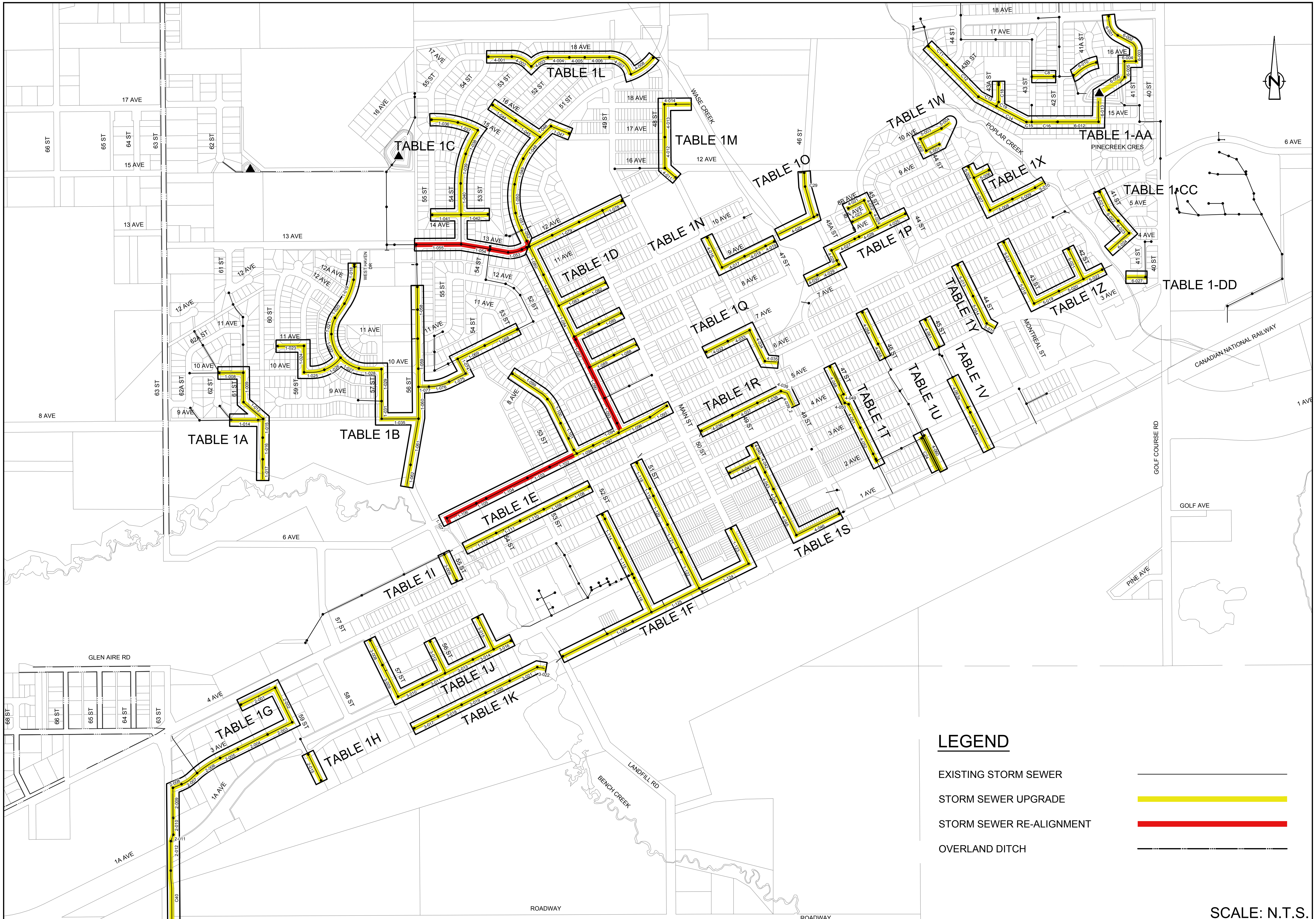

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FIGURE No. 5.7

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

STORM SYSTEM
EXISTING STORMWATER IMPROVEMENTS

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Jul 5, 2018

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

STORM SYSTEM
IMPROVED SYSTEM RESULTS - 5 YEAR, 4 HOUR RAINFALL EVENT

FIGURE No. 5.8

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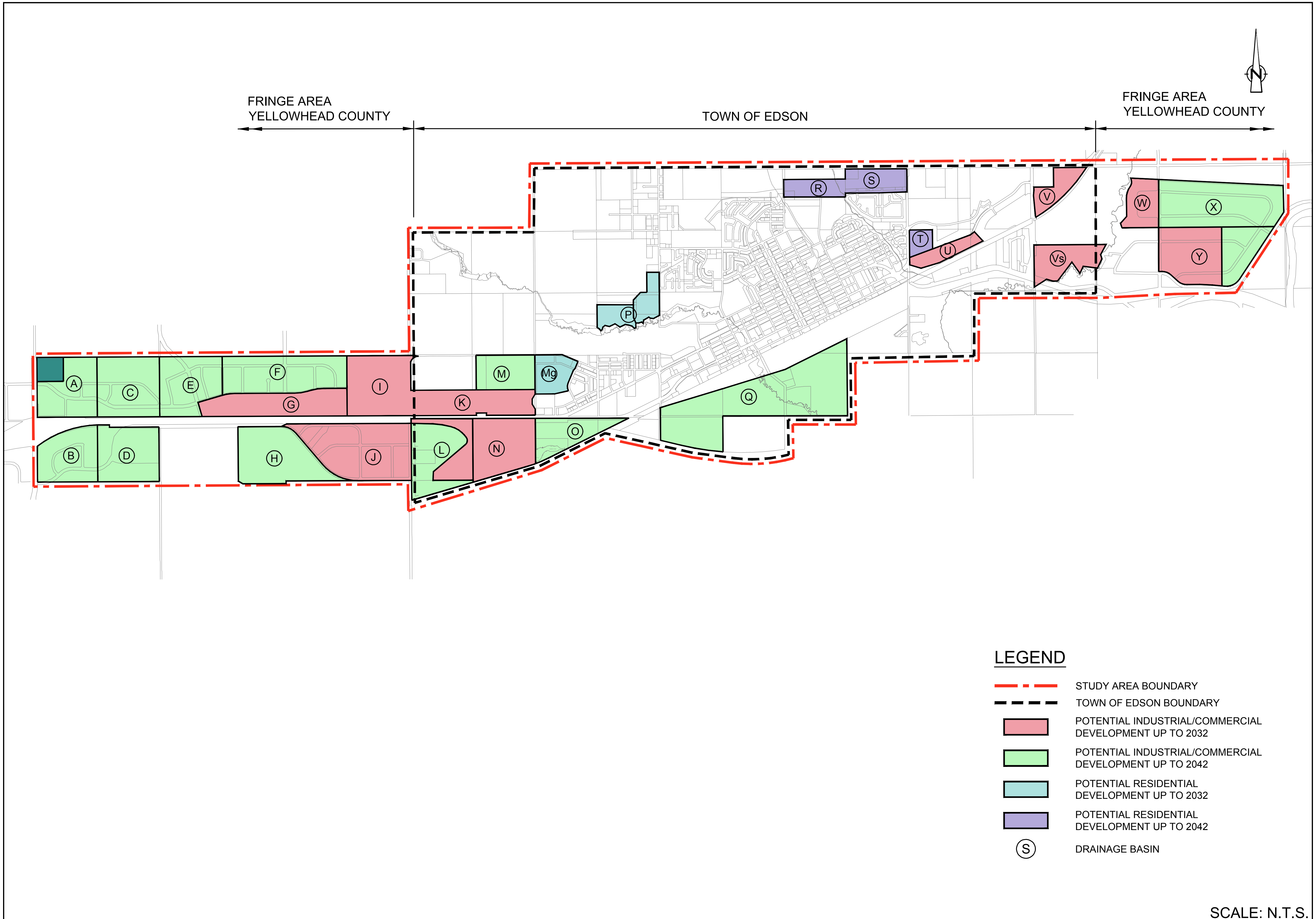

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Jul 5, 2018

FIGURE No. 5.9

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

STORM SYSTEM
FUTURE STORM DRAINAGE BASINS

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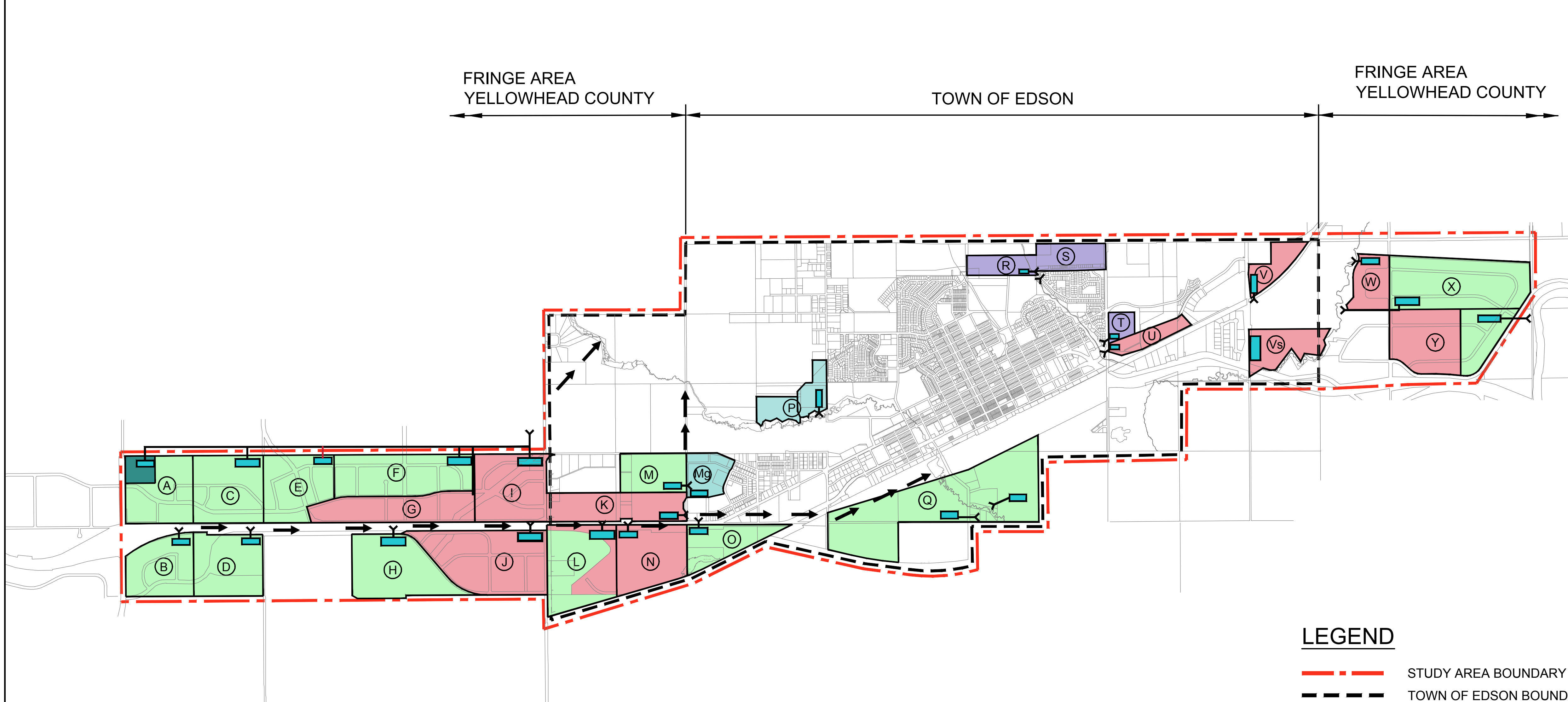

LEGEND

- STUDY AREA BOUNDARY
- TOWN OF EDSON BOUNDARY
- POTENTIAL INDUSTRIAL/COMMERCIAL DEVELOPMENT UP TO 2032
- POTENTIAL INDUSTRIAL/COMMERCIAL DEVELOPMENT UP TO 2042
- POTENTIAL RESIDENTIAL DEVELOPMENT UP TO 2032
- POTENTIAL RESIDENTIAL DEVELOPMENT UP TO 2042
- PROPOSED STORMWATER MANAGEMENT FACILITY
- (S) DRAINAGE BASIN
- OVERLAND DRAINAGE
- OUTLET

DRAINAGE BASIN AREAS

BASIN	AREA (ha)	BASIN	AREA (ha)
A	71.5	N	58.1
B	51.3	O	37.1
C	63.7	P	37.1
D	58.6	Q	146
E	57.3	R	17.3
F	75.4	S	39.9
G	53.1	T	29.0
H	74.1	U	18.1
I	60.8	V	20.0
J	91.7	V-s	28.7
K	53.0	W	26.3
L	77.1	X	90.2
M	33.7	Y	149.6
M-g	32.9		

SCALE: N.T.S.

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

STORM SYSTEM
FUTURE STORM SERVICING PLAN

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Jul 5, 2018

FIGURE No. 5.10

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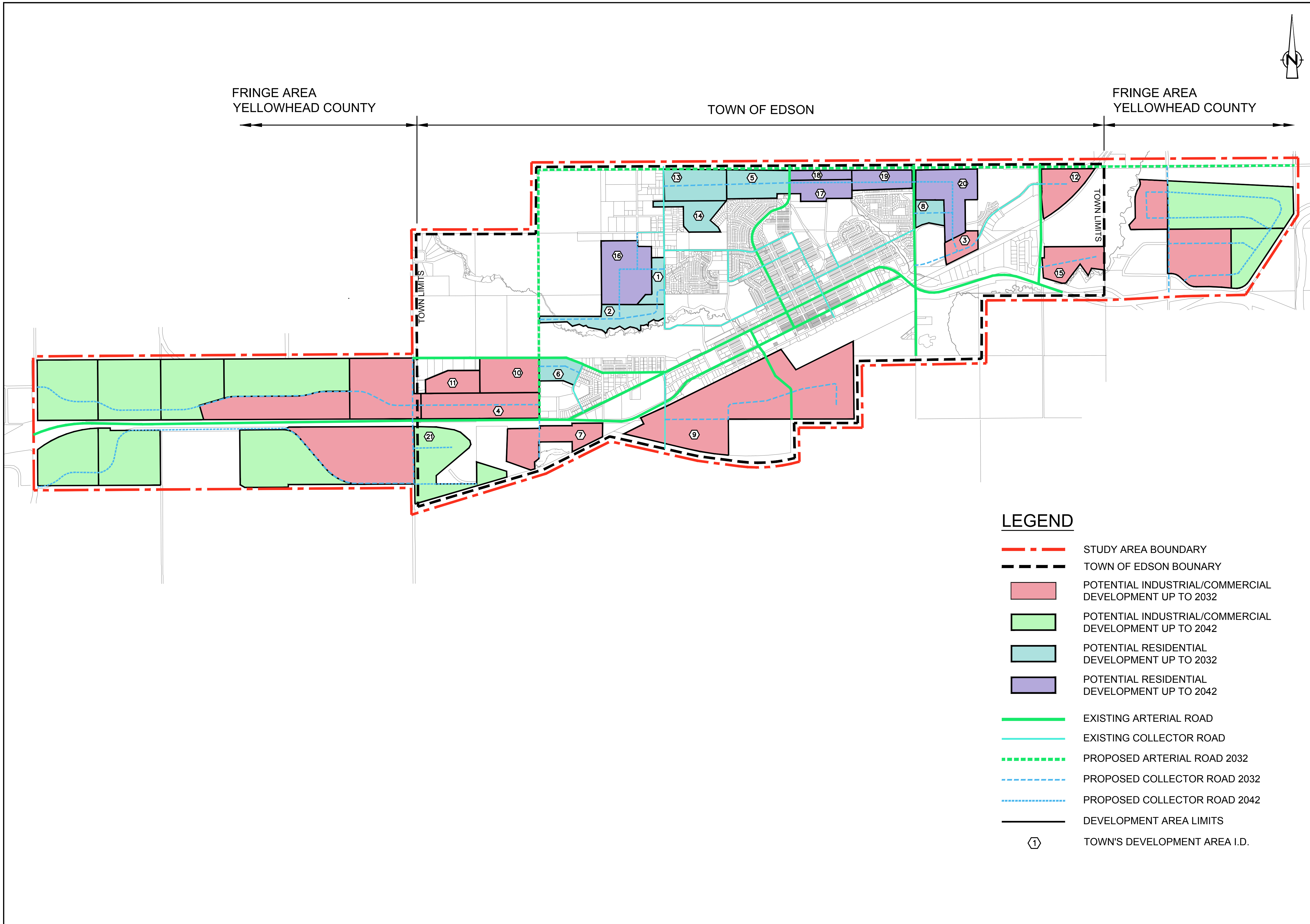

11148145-000

Jul 5, 2018

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

TRANSPORTATION AND ROADWAY NETWORK
CONCEPT MAP UPDATE 2032 & 2042 (15 & 25 YEAR PROJECTION)

FIGURE 6.1

CAD File: I:\CAD\11148145 - Town of Edson\11148145-MEMO\11148145-000(MEMO001)GN\11148145-000(MEMO)GN-CA061.dwg

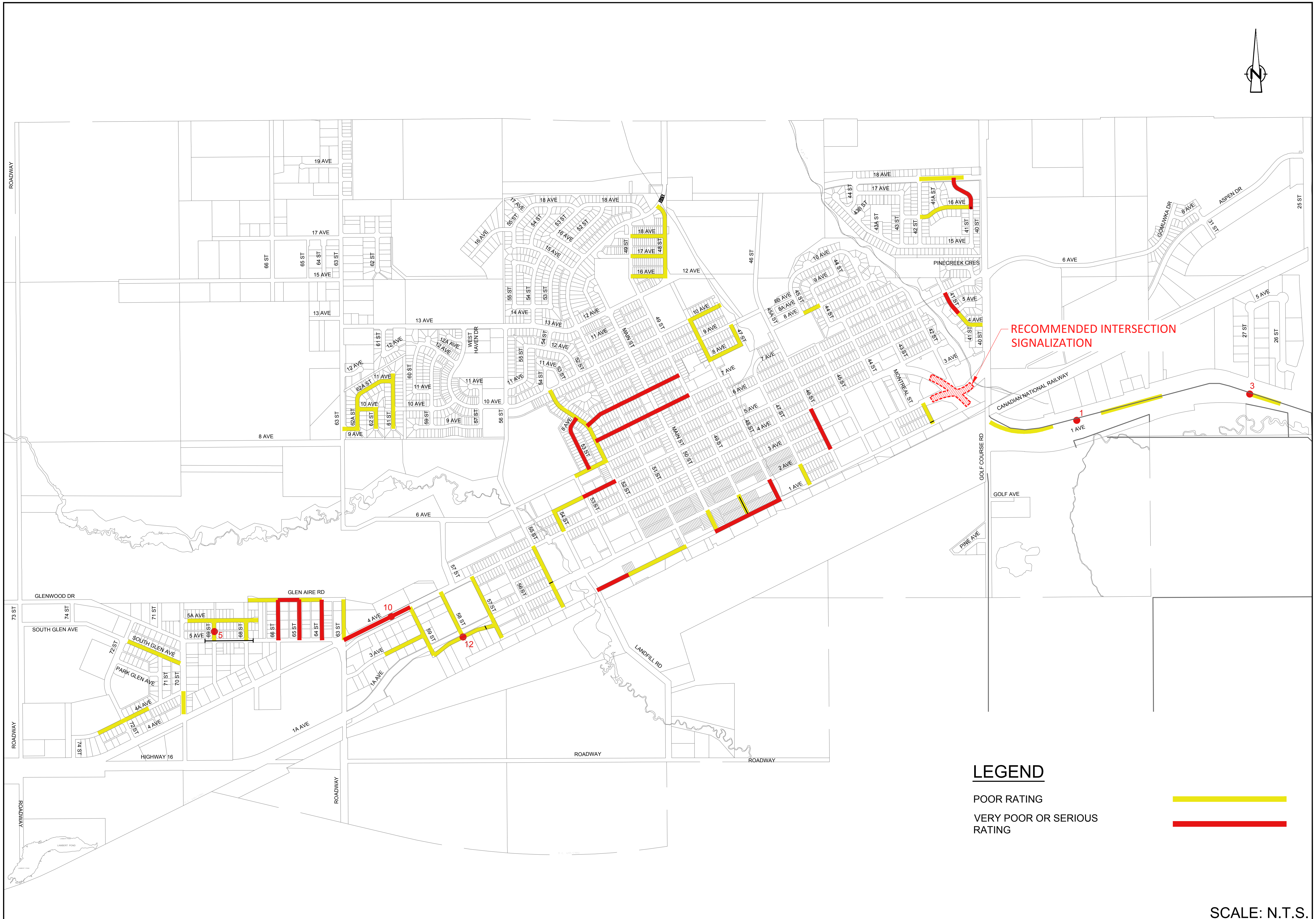

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FIGURE No. 6.2

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

TRANSPORTATION AND ROADWAY NETWORK
EXISTING ROAD REPAIRS REQUIRED (2017)

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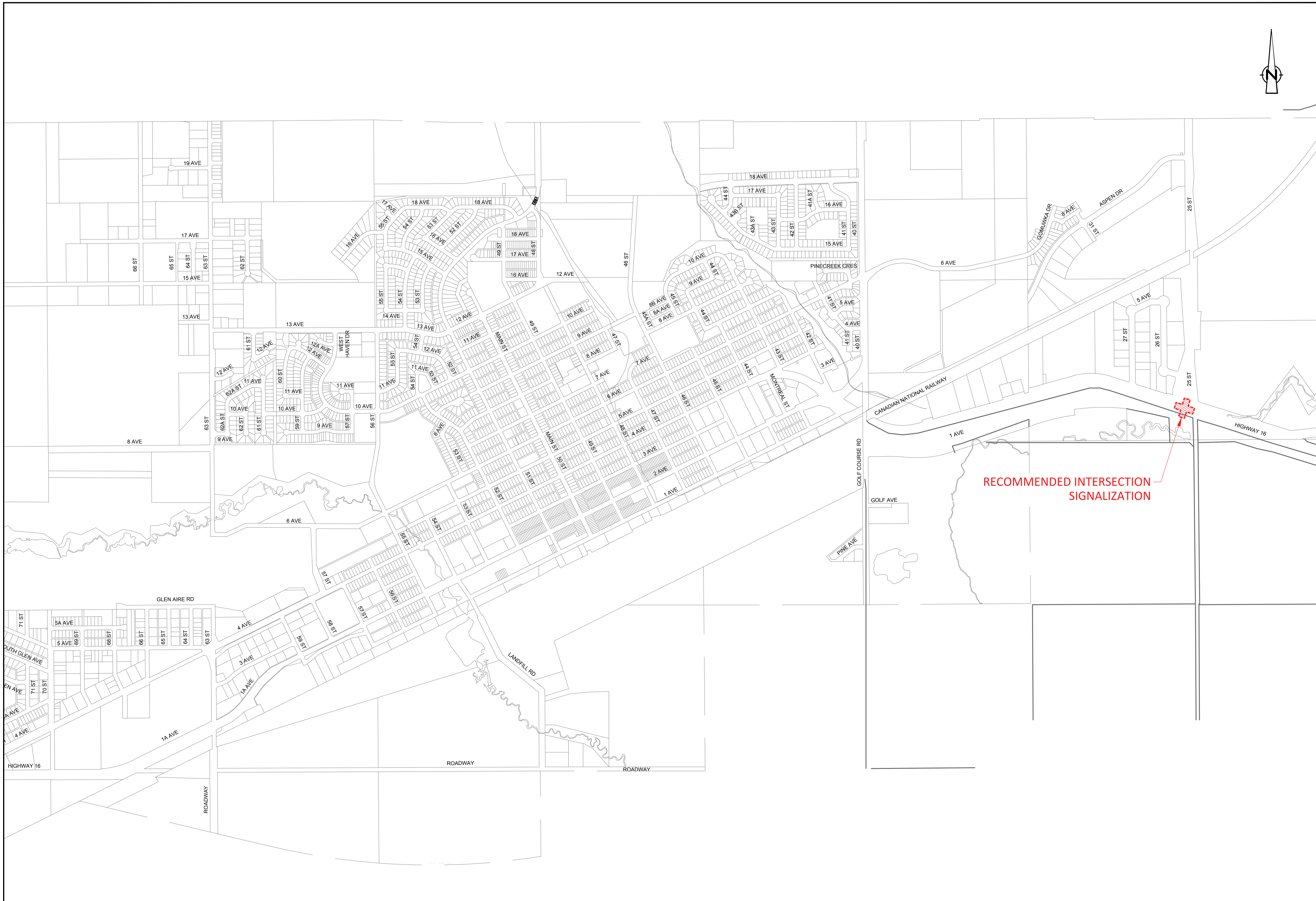

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Jul 5, 2018

TOWN OF EDSON
EDSON, ALBERTA
MUNICIPAL SERVICING PLAN UPDATE

TRANSPORTATION AND ROADWAY NETWORK
PROPOSED TRANSPORTATION UPGRADES (2032)

FIGURE No. 6.3

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about GHD

GHD is one of the world's leading professional services companies operating in the global markets of water, energy and resources, environment, property and buildings, and transportation. We provide engineering, environmental, and construction services to private and public sector clients.

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