Water

Town of Edson

Municipal Servicing Plan Update

Prepared by:

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Project Number:

4193-033-00-4.6.1

Date:

December 2011

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December 21, 2011

Dawit Solomon, MSc., P.Eng.
Director of Engineering
Town of Edson
605 – 50th Street
PO Box 6300
Edson, AB T7E 1T7

Dear Mr. Solomon:

Project No: **4193-033-00-4.6.1**

Regarding: **Municipal Servicing Plan Update**

We are pleased to submit our final report on the Town of Edson Municipal Servicing Plan Update. We have incorporated comments received through review of the draft reports.

If you have any questions or require any additional information please call.

Sincerely,
AECOM Canada Ltd.

Ahtesham

Ahtesham Shirazi, M.Eng., P.Eng.
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Encl.

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Revision Log

Revision #	Revised By	Date	Issue / Revision Description
0	KJS	October 22, 2009	Draft Report of Water Supply and Distribution System
1	MM / KJS	June 25, 2010	Draft Report of Wastewater Collection System and Stormwater Management System
2	KJS	July 29, 2010	Final Report
3	KJS	June 24, 2011	Draft Final Report of Water Supply and Distribution System
4	KJS	September 14, 2011	2nd Draft Final Report of Water Supply and Distribution System
5	KJS	October 7, 2011	Final Report of Water Supply and Distribution System
6	KJS/JJC/DB	December 22, 2011	Final Report

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PERMIT TO PRACTICE
AECOM CANADA LTD.
Signature
Date 11/05/2012
PERMIT NUMBER: P10450
The Association of Professional Engineers, Geologists and Geophysicists of Alberta.

Report Reviewed By:

Ahtesham Shirazi, M.Eng., P.Eng.
Project Manager

Executive Summary

Water Supply and Distribution System

In 2005, a Water Distribution System Analysis was conducted for the Town of Edson. The report detailed the existing and future requirements for the water distribution system within the Town. Subsequently, the Edson Urban Fringe Intermunicipal Development Plan was completed, which provided a framework for development in the Urban Fringe Area within Yellowhead County. AECOM was retained by the Town of Edson to update the 2005 study to include areas within the Urban Fringe Area.

The Town of Edson water distribution system was modeled using WaterCAD version 8i and was updated by adding infrastructure constructed since the completion of the Town of Edson Water Distribution System Analysis (April 2005) and the 2007 water consumption rates. The model was calibrated against hydrant flow test results. It is recommended that a C value of 120 be used for PVC pipes, and 110 be used for all other pipe materials.

Generally, the existing water distribution system cannot provide fire flows to the existing areas. In the northwest area of Town, north of 13 Avenue between 61 and 63 Street, the pressures are below 280 kPa during peak hour demand. For the existing development condition, the reservoirs were evaluated for the Alberta Environment guidelines. Based on this requirement, the existing reservoirs are adequate to provide the required storage volume. The existing system does not have adequate pumping capacity; therefore, it is recommended that a booster station be constructed adjacent to the reservoirs at Grande Prairie Trail with a capacity of 290 L/s at 45 m of head. A 300 mm diameter loop is recommended along Highway 16 to increase the available fire flows in the east area of Town, and several upgrades are recommended to solve local fire flow deficiencies.

The groundwater wells should be able to supply the maximum day demand. Based on the Town of Edson design standards, the maximum day demand is 126 L/s. To meet this demand, all wells should be utilized, and an additional 25 L/s is required. However, the 2007 measured water use in the Town of Edson was approximately 66 L/s for maximum day demand and the existing groundwater wells have sufficient capacity to provide this flow. It is recommended that additional groundwater wells be considered once the measured maximum day demand approaches the allowed design discharge rate of 101.4 L/s. It is recommended that Well No. 3 be brought back into service prior to the installation of additional wells.

For future development, three alternatives were considered. Alternative 1 is based on the Town of Edson design standards, and Alternative 2 is based on the Yellowhead County design standards. Alternative 3 was developed for cost comparison purposes, in which only development within the Town of Edson was considered, based on the Town of Edson water consumption rates. It was determined that the Town of Edson standards should be used for the purpose of the Municipal Servicing Plan; therefore, Alternative 1 was chosen. Details on the future servicing for Alternative 1 are provided below.

Additional groundwater wells will be required to service the 2015 and 2025 development scenarios. Based on an assumed rate of 8.5 L/s per well, 19 additional wells will be required by 2015, and an additional 18 wells will be required by 2025.

In general, 250 mm to 350 mm diameter water mains are required for future water servicing. Seven new pressure reducing valves are recommended, to separate the service area into six pressure zones.

For the 2015 development condition, extra storage capacity will be required. Since the study area is fed through groundwater wells, and is not part of a regional system, it is recommended that the Alberta Environment guidelines be used to determine future storage requirements.

For the 2015 and 2025 development conditions, approximately 9,500 m^3 and 4,100 m^3 of additional storage will be required. It is recommended that this storage be provided at a new reservoir and pumphouse located in the west portion of the study area.

Additional pumping capacity will also be required by the 2015 development condition. It is recommended that the booster station at Grande Prairie Trail be further upgraded to provide 330 L/s at 45 m of head. The additional pumping is recommended to be located at the proposed West Reservoir and Pumphouse (300 L/s at 45 m of head). For the 2025 development condition, the pumping head at the West Reservoir and Pumphouse should be increased to 71.5 m. It was assumed that the future groundwater wells will contribute to the overall pumping requirements.

The total cost of Alternative 1 is $81,074,030, including 10% for engineering and 25% for contingency. The cost estimates for groundwater wells, reservoirs, additional pumping and water mains are summarized in Table ES.1. Costs for Alternative 3 (development only within the Town of Edson) have been included for comparison purposes.

Table ES 1: Water Supply and Distribution System - Cost Estimate Summary

Description	Alternative 1	Alternative 3
Groundwater Well Cost	$3,746,250	$1,518,750
Reservoir Cost	$9,640,430	$6,511,300
Pumping Cost	$3,090,350	$3,049,240
Water Main Costs	$64,030,000	$29,196,000
Pressure Reducing Valve Costs	$567,000	$567,000
Total	**$81,074,030**	**$40,842,290**

To upgrade the existing system, it is recommended that the new booster station at Grande Prairie Trail be constructed first, followed by the 300 mm loop along Highway 16. For the local pipe improvements, if pipe replacement is required due to pipe age or others factors, pipe upgrading should be considered at that time. It should be noted that some of the pipe upgrades indicated can be considered with road upgrades where possible to eliminate or reduce the restoration cost.

Wastewater Collection System

The existing sanitary system consists of approximately 66 km of gravity sewer mains. There are no lift stations present within the Town's system. All the sanitary flow from the Town drains to the existing sewage lagoon located west of 25th Street and south of the Canadian National Railway right of way. The majority of the pipes are 200 mm in diameter, but gradually increase in size closer to the lagoons, becoming as large as 1050 mm. The lagoons are used for treatment rather than storage and currently discharge treated water into the McLeod River, approximately 2.5 km away.

Based on discussions with the Town, all houses constructed prior to 2005 are likely to have weeping tile connected to the sanitary system. The Town has experienced basement flooding and/or sewer backups in the past in areas suspected to have weeping tile connections. Newer areas that do not have weeping tile connections include the East End Subdivision, Skyview and Willishire House. As expected, none of these areas experience flooding in the model.

XP-SWMM version 9.14, an industry accepted modelling software program, was used to develop the detailed model of the existing sanitary sewer system. The model was calibrated in a two step process: identification of the dry weather flows and identification of the wet weather flows for the selected rainfall events (June 6, June 11, and August 21, 2008). The modeled dry weather flow was compared to the monitored dry weather flow. The modelled volume and peak flow compare quite favourably to the monitored volume and peak flow.

The modelled volume and peak flow are within 14% and 7% of the monitored values respectively. For wet weather flows, the model was verified for the inflow to the wastewater lagoon for the three selected rainfall events. In general, the modelled and monitored wet weather flows compare quite favourably. The average calculated I/I rate for all three events was approximately 0.11 L/s/ha.

It is recommended that the Town of Edson continue to collect flow data and verify the model calibration on a yearly basis or when a large rainfall event occurs. A rain gauge with the capability of collecting minute to minute rainfall data is also recommended, as Environment Canada only provides hourly rainfall data.

The existing system was assessed to examine the system performance for various rainfall events and to identify any deficiencies in the system. The existing system was evaluated for the 5 and 25 year short duration (4 hour) and long duration (24 hour) rainfall events.

Currently, the Town of Edson experiences some sanitary sewer line flooding in both the 5 and 25 year events. The majority of the pipes are 200 mm in diameter, which in some cases is too small to handle the Town's potential wet weather flows. The sewer network also tends to back up because there are few lines that experience an increase in diameter as the line runs downstream.

Improvements were divided into 3 Phases, and involve upgrading and/or twinning lengths of pipe in problem areas. The Phase 1 upgrades address all of the surcharging within 1.0 m of the ground level for the 5 year 4 hour rainfall event. The Phase 2 upgrades address all the surcharging within 1.0 m of the ground level for the 25 year 4 hour rainfall event. The Phase 3 upgrades address all the surcharging within 2.5 m of the ground level within residential areas for the 5 year 4 hour event, therefore minimizing the risk of basement flooding. Based on the Town of Edson Lagoon Assessment completed by Earthtech in 2007, the existing lagoons have capacity for 9,500 people. This is sufficient for the existing population of 8,323 people.

The existing system with the proposed upgrades is adequate for the addition of 2015 and 2025 residential areas to the northeast and northwest portions of the Town. For the west portion of the Town a proposed new trunk line servicing the industrial areas in the west of Town will need to be upsized to accommodate the new areas to the west. Table ES.2 summarizes the costs for the existing system improvements, as well as costs for future servicing.

Table ES 2: Wastewater Collection System - Cost Estimate Summary

Description	Total Length (m)	Total Cost ($)
Existing System Upgrades		
-Phase 1	5964	$11,750,080
-Phase 2	5558	$5,164,960
-Phase 3	936	$610,004
2015/2025 System Upgrades[1]	2518	$4,633,905
Lagoon upgrades (Earthtech, 2007)	-	$2,010,000
2015 Development	14,200	$12,295,125
2025 Development	6,800	$4,772,250
Total	35,975	**$36,602,419**

[1]2015/2025 system upgrades are not included in the total as they are included in Phase 1.

It is recommended that Phase 1 improvements are implemented first followed by Phase 2 and Phase 3 improvements. Generally, upgrades can be prioritized from downstream to upstream (east to west) and residential areas have higher priority than non-residential areas.

However, improvements should be completed, where possible, as part of the street improvement program or other proposed underground projects to minimize the excavation and restoration costs as well as disruption.

Stormwater Management System

The existing system was assessed to examine the system performance for various rainfall events and to identify any deficiencies in the system. The existing system was evaluated for the 5, 25, and 100 year short duration (4 hour) and long duration (24 hour) rainfall events. During the 5 year 4 hour event, the existing system experiences a large amount of surface flooding. The parts of the system not flooding have high surcharge levels. Overall, the existing sewer system does not have adequate capacity for the 5 year 4 hour rainfall event. The system performs significantly better during the 5 year 24 hour rainfall event and generally has adequate capacity to convey the 5 year 24 hour rainfall event.

Flooding and surcharging in the system increases during the 25 year and 100 year rainfall events. The 4 hour duration events continue to cause the system to flood and operate under surcharged conditions. The 24 hour duration events generally have capacity to convey the runoff; however, flooding occurs at one location during the 25 year event and at several locations during the 100 year event.

For the proposed existing system improvements, a level of service such that there is not surcharging within 1.0 m of ground for the 5 year 4 hour rainfall event was adopted.

There are not many areas that would effectively provide storage within the existing developed areas of Edson; therefore, the proposed improvements consider pipe upgrades. Once the storm sewer upgrades are implemented, the majority of the system does not have any surcharging during the 5 year 4 hour rainfall. Some surcharging still exists; however, it is localized and does not result in the HGL being within 1.0 m of the ground. Upon redevelopment of the existing developed areas, there may be potential to provide on-lot storage at that time which may eliminate the need for large size conveyance pipes. Considering storage over large size conveyance pipes may also be advantageous to control runoff towards creeks to allowable discharge rates as well as to control the quality of the runoff.

A stormwater management plan was developed for the Town of Edson based on 2015 and 2025 development. The future stormwater management plan is not dependant on the proposed existing system upgrades. The future development areas were delineated into 24 storm drainage basins. Each of the proposed drainage basins will be graded such that the runoff is routed to a stormwater management facility (SWMF). The future SWMFs will be designed to service the critical 100 year rainfall event while discharging at the allowable discharge rate. It is proposed that the SWMFs be designed to be wet facilities to allow for sediments to settle out of the runoff and therefore enhance the water quality before being released. The SWMF locations may change in the preliminary design stage and should drain by gravity to the receiving water body.

The results of the model simulation showed that there were two governing rainfall events for the proposed SWMFs. The 4 hour duration rainfall event is the critical event for SWMFs that have residential development and discharge to Poplar Creek. All other SWMFs are designed for the 100 year 24 hour rainfall.

The total cost for the storm sewer improvements is approximately $24.5 million, and the total cost for construction of the future SMWFs is approximately $41.6 million.

In developing a stormwater management plan for infill developments, physical conditions, infrastructure capacity, increase in percent imperviousness, and the opportunity for retrofitting or rehabilitating stormwater management systems should be considered.

Servicing of infill developments can be achieved through:

- No Control - this is best limited to small individual lots of less than one hectare, as cumulative effects of several infill developments can create problems including flooding.
- Minimum Runoff Capture -this requires the developer to capture all runoff from a lesser rainfall event, such as the 5 to 25-year event, and retain it on-site until it infiltrates, evaporates, or consideration can be given to releasing the runoff after the rainfall event.
- Conveyance - to an existing storm sewer system or construction of new conveyance infrastructure.
- Off-Site Systems - this can involve a stormwater management facility to control the generated runoff at another location downstream of the infill development. The potential locations for OSS can be addressed during preliminary design phase.
- Sustainable Development - sustainable methods such as permeable landscaping and green roofs can significantly reduce the runoff generated by a development.

The proposed improvements to the storm sewer system will be adequate to convey the runoff and meet the recommended service level for the proposed infill developments. The existing storm sewer system is currently surcharging at most locations proposed for infill development. The small lot sizes (less than 1.0 ha) for the infill develop areas would be difficult to provide a significant amount of on-lot storage and cost prohibitive to provide underground storage. Storage should be provided for the 100 year 4 hour rainfall event, with a discharge of 10 L/s/ha for infill development areas.

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1. Introduction

The Town of Edson has retained AECOM to update and consolidate the sanitary, water, and storm servicing studies into a servicing master plan. The objective of this master plan will be to identify the existing system deficiencies, the servicing requirements for future development and to identify the impact of future development on the existing infrastructure.

1.1 Background

The Intermunicipal Development Plan (IDP) was prepared as a joint initiative between the Town of Edson and Yellowhead County. The IDP was developed to address and plan for future growth in the Edson Urban Fringe Area. The Action Plan developed as part of the IDP process, identified the need for the Town to undertake a comprehensive update of the 1982 Municipal Servicing Plan. The servicing plan will identify opportunities to extend the existing water distribution and wastewater collection system to service the developable areas within the Urban Fringe Area.

1.2 Scope of Work

The scope of work includes the following:

Water Supply and Distribution System

- Review of the 2005 Water Distribution System Analysis
- Collection and review of all relevant data, including record drawings of all developments constructed since 2005 and modifications to the water distribution system.
- Update of the existing water network model to reflect the new developments and modifications to the existing water distribution system.
- Assessment of the existing water consumption rates for residential and non-residential areas.
- Calibration and verification of the model based on hydrant flow test data.
- Develop growth scenarios for the ultimate development of the Town of Edson and the Urban Fringe Area.
- Assess the system performance under Peak Hour and Maximum Day plus Fire Flow Demands for both existing and future development.
- Identify system improvements to address system deficiencies.
- Develop cost estimates and an implementation plan.

Wastewater Collection System

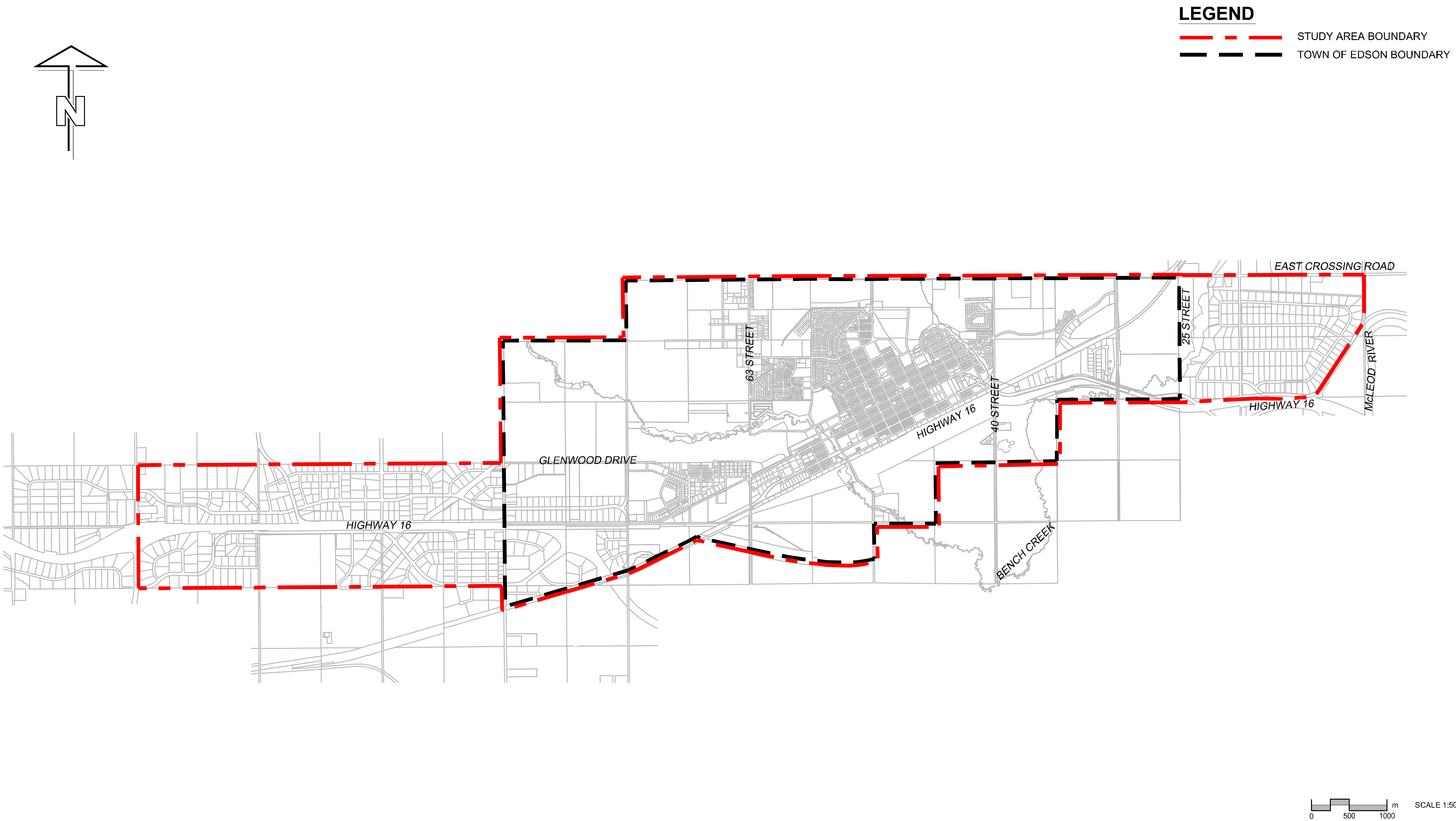
- Collection and review of all relevant data.
- Develop a wastewater system model
- Calibrate the model based on flow monitoring data
- Assess the existing wastewater collection system,
- Develop growth scenarios for the ultimate development of the Town of Edson and the Urban Fringe Area
- Identify existing system deficiencies and associated improvements,
- Identify the available system capacity
- Develop an overall servicing plan for the Town of Edson and the Urban Fringe Area;
- Develop cost estimates and an implementation plan.

Stormwater Management System

- Collection and review of all relevant data, including the July 2005 Stormwater Management Plan, existing as-builts and mapping.
- A field reconnaissance will be conducted, as well as any field survey that may be required.
- Develop growth scenarios for the ultimate development of the Town of Edson and the Urban Fringe Area.
- Develop a model of the existing storm sewer system.
- Assess and evaluate the existing system performance and identify the need for improvements.
- Develop a future storm servicing concept including: allowable discharge rates; location and sizing of stormwater management facilities; and trunk sewer sizing and alignment.
- Develop stormwater management guidelines for infill developments.
- Develop cost estimates and an implementation plan.

1.3 Study Area

The study area is located within the Town boundary and proposed developments areas within Yellowhead County boundary, as shown in Figure 1.1. The Town of Edson is located within Yellowhead County, along Highway 16. The study area includes areas within the Town boundary, as well as proposed development areas within the Urban Fringe Area in Yellowhead County. The study area boundary is indicated in Figure 1.1.



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Town of Edson
Municipal Servicing Plan Update

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Location Plan
Figure 1.1

2. Data Collection and Review

2.1 General

This section outlines the information collected and reviewed for the Town of Edson, including:

- Previous reports, studies and investigations carried out in the Town
- Design drawings, as-built plans, and pipe information for the Town
- Existing and future land use maps.

2.2 Relevant Reports

The following reports pertaining to the Town of Edson water, wastewater and storm servicing have been reviewed and the applicable data incorporated into the study:

- Edson Urban Fringe Intermunicipal Development Plan, Lovatt Planning Consultants Inc., June 2007
- Town of Edson Water Distribution System Analysis draft report, UMA Engineering Ltd., April 2005
- Water Supply for Public Fire Protection, A Guide to Recommended Practice, Public Fire Protection Survey Services, 1999
- Town of Edson General Engineering Study, Stanley Associates Engineering Ltd., 1982

2.3 Population Projections

The projected development for the years 2015 and 2025, as provided by the Town, is shown in Figure 2.1. Table 2.1 summarizes the expected growth in these areas and associated population projections. Development density for Areas 8, 9, 10 and 11 was assumed to be 25 persons/ha, and for the remaining residential development it was assumed to be 40 persons/ha.

For the industrial/commercial area, the growth was projected based on the information provided by the Town of Edson and Yellowhead County. The total estimated non-residential development for the year 2015 and 2025 will be approximately 135 hectares and 116 hectares (gross area) respectively for the Town of Edson, and 265 hectares and 540 hectares (gross area) respectively for Yellowhead County.

Table 2.1: Projected Growth for the Years 2015 and 2025

Future Growth Area	Year 2015			Year 2025		
	Population	Residential Area (ha)	Industrial / Commercial Area (ha)	Population	Residential Area (ha)	Industrial / Commercial Area (ha)
1			20.0			
2			18.1			
3				320	8.0	
4	480	12.0				
5	680	17.0				
6				1000	25.0	
7				766	19.2	
8	293	11.7				
9	300	12.0				
10				250	10.0	
11	500	20.0				

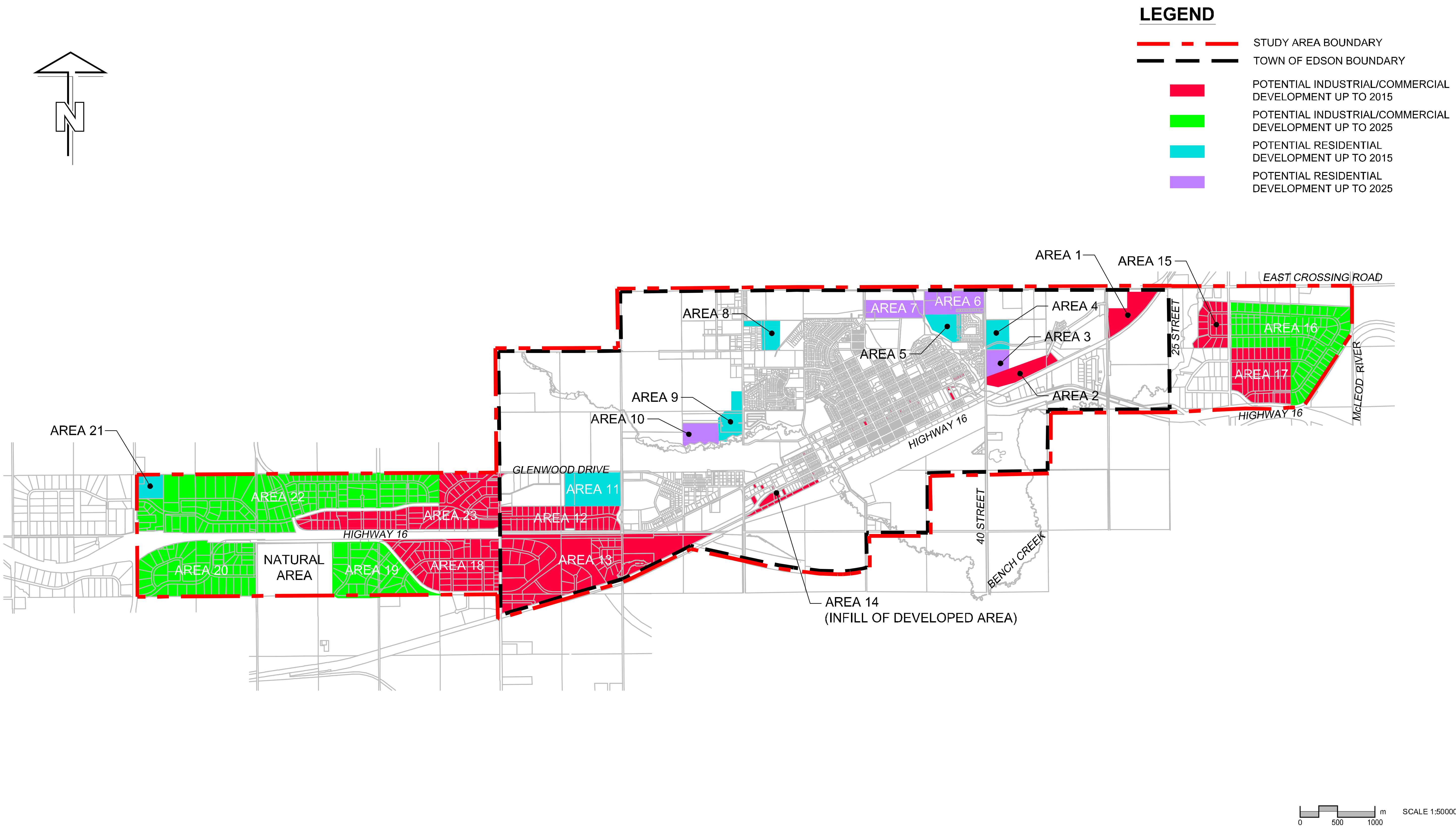
Future Growth Area	Year 2015			Year 2025		
	Population	Residential Area (ha)	Industrial / Commercial Area (ha)	Population	Residential Area (ha)	Industrial / Commercial Area (ha)
12			83.3			
13			116.3			
14			14.0			
15			24.7			
16						122.5
17			56.1			
18			86.4			
19						70.1
20						113.8
21				460	11.5	
22						233.2
23			96.5			
New	2,253	72.7	515.4	2,796	73.7	539.6
Existing	8,323	211	261	10,576	283.7	776.4
Total	10,576	283.7	776.4	13,372	357.4	1,316

The projected growth areas for the year 2015 and 2025, as provided by the Town of Edson, are also shown in Figure 2.1. The growth areas have been numbered for ease of identification and are not meant to indicate the sequence of development.

2.4 Land Use

The land use for the existing and future development areas within the Town the Urban Fringe areas within Yellowhead County are shown in Figure 2.2. The proposed developed areas consist of residential, commercial, industrial and institutional areas.

The anticipated land use for the future development is expected to be residential and commercial / industrial developments. The residential development for the year 2015 and 2025 is proposed in the north and west part of the Town by expanding the existing residential areas. The commercial/industrial development is proposed along Highway 16, both east and west of the Town.



Town of Edson
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Proposed Development
Figure 2.1

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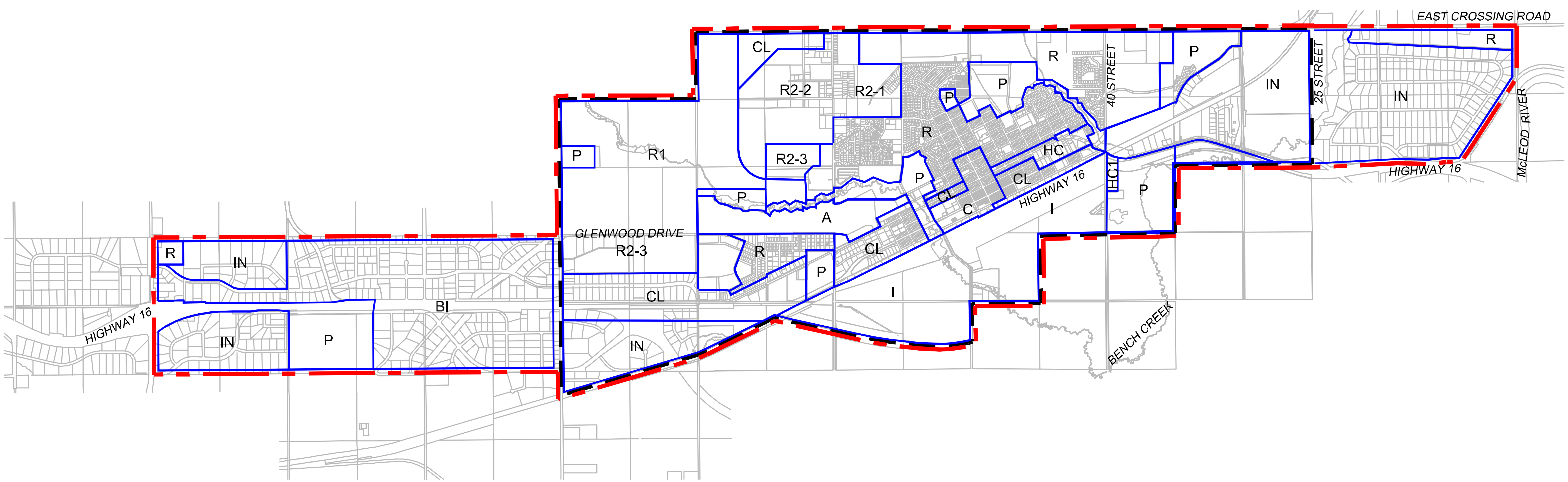
LEGEND

STUDY AREA BOUNDARY
TOWN OF EDSON BOUNDARY

LAND USE CLASSIFICATION

R	RESIDENTIAL - 40 PERSONS/GROSS HECTARE
R1	RESIDENTIAL LOW DENSITY - UNSERVICED
R2-1	RESIDENTIAL LOW DENSITY - 4.94 LOT/HECTARE
R2-2	RESIDENTIAL LOW DENSITY - 2.47 LOT/HECTARE
R2-3	RESIDENTIAL - 25 PERSONS/GROSS HECTARE
P	EXISTING SCHOOLS, PARKS AND PUBLIC OPEN SPACES
I	INDUSTRIAL UNSERVICED
IN	INDUSTRIAL
C	DOWNTOWN COMMERCIAL AREA
CL	COMMERCIAL/LIGHT INDUSTRIAL MIX
HC	HIGHWAY/TOURISM COMMERCIAL
HC1	HIGHWAY/TOURISM COMMERCIAL - UNSERVICED
A	AIRPORT
BI	BUSINESS LIGHT INDUSTRIAL



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Land Use Classification
Figure 2.2

3. Water Supply and Distribution System

3.1 General

This section assesses the capacity of the existing water supply and distribution system, identifies existing system deficiencies and required improvements, identifies impacts of the future development and provides a servicing concept for the years 2015 and 2025.

3.2 Hydrant Testing

Five hydrant flow tests were conducted by EPCOR Water Services on September 17, 2008. The locations of the five hydrant tests are indicated on Figures 3.1 and 3.2. Table 3.1 summarizes the hydrant flow test results; detailed test results are provided in Appendix A.

Table 3.1: Summary of Hydrant Flow Test Results

Test Hydrant	One Port Open		Two Ports Open		Flow through One Port (L/s)	Flow through Two Ports (L/s)	Available Flow at 140 kPa (L/s)
	Flow Hydrant Pressure (kPa)	Residual Hydrant Pressure (kPa)	Flow Hydrant Pressure (kPa)	Residual Hydrant Pressure (kPa)			
1F	379	400	221	338	63	103	179
2F	476	538	310	496	69	118	303
3F	269	290	138	214	55	85	95
4F	476	579	68	510	69	111	194
5F	290	310	193	290	57	97	224

3.3 Design Criteria

<u>Water Consumption Rates</u>

The Town of Edson provided the total water consumption data from 2003 to 2007, which includes the residential and non-residential users. In addition, the Town provided the water consumption data for the top water consumers or high demand users.

Based on the 2007 water consumption data, the total average day demand for the Town of Edson was approximately 33 L/s. Of this demand, approximately 8.5 L/s is attributed to the high demand users. The average demand for the high demand users is approximately 13,600 L/ha/day. For the non-residential areas that were not included as high demand users, if a rate of 1500 L/ha/day is assumed, then the residential average day consumption is approximately 240 L/capita/day. The existing water consumption rates were used to calibrate the model, as well as to aid in determining appropriate standards for the Town of Edson.

For subsequent analyses, design standards were used. For residential areas, it was determined that 330 L/capita/day would be appropriate for average day demand for existing and future areas, with peaking factors of 2.0 and 3.0 for maximum day and peak hour demands. For non-residential areas, an average consumption rate of 10,000 L/d/ha was used for existing and future areas with peaking factors of 2.0 and 3.0 for maximum day and peak hour demands. This average non-residential consumption rate provides flexibility in the design, since it is unknown whether the future development will consist of high demand or low demand users. These water consumption rates were used to evaluate the existing system, as well as the future systems for Alternatives 1 and 3.

Since the system analysis includes areas within Yellowhead County, the design rates for Yellowhead County were also considered. Yellowhead County standards specify a residential water consumption rate of 375 L/capita/day with peaking factors of 2.0 and 4.0 for maximum day and peak hour demands. For non-residential areas, the standards specify a rate of 0.2 L/s/ha with a peaking factor of [10 x (flow rate)^-0.45], to a maximum of 25 and a minimum of 2.5. These water consumption rates were used to evaluate the future systems for Alternative 2.

Alternatives 1 and 3 are described in detail in Section 3.9, and Alternative 2 is described in Appendix C.

Table 3.2 provides a summary of the Town of Edson and Yellowhead County demands.

Table 3.2: Water Consumption Rates

<table>
<tr><th>Demand</th><th>Town of Edson</th><th>Yellowhead County</th></tr>
<tr><td colspan="3">Residential Demands</td></tr>
<tr><td>Average Day Demand (L/capita/day)</td><td>330</td><td>375</td></tr>
<tr><td>Maximum Day Demand (L/capita/day)</td><td>660</td><td>750</td></tr>
<tr><td>Peak Hour Demand (L/capita/day)</td><td>990</td><td>1500</td></tr>
<tr><td colspan="3">Non-Residential Demands (Industrial/Commercial)</td></tr>
<tr><td>Average Day Demand (L/ha/day)</td><td>10,000</td><td>17,280</td></tr>
<tr><td>Maximum Day Peaking Factor</td><td>2.0</td><td rowspan="2">$10*Q^{-0.45}$</td></tr>
<tr><td>Peak Hour Peaking Factor</td><td>3.0</td></tr>
</table>

Using the Yellowhead County standards for non-residential areas, the peaking factor was applied to each development area, and therefore varies for each basin. When applied to the existing areas within the Town, the average peaking factor is approximately 17. This corresponds to a maximum day demand and peak hour demand of approximately 294,000 L/ha/day. For the future development area, the average peaking factor is approximately 7, corresponding to a maximum day demand and peak hour demand of approximately 121,000 L/ha/day. These peaking factors are very large and do not accurately represent the demands of the non-residential areas.

Fire Flows

The existing water distribution system was evaluated for the following fire flow requirements:

- Single Family Residential 76 L/s
- Multiple Family Residential 150 L/s
- Institutional Areas (i.e., Schools) 130 L/s
- Industrial and Commercial Areas 265 L/s

It is recommended that hospitals be evaluated for a fire flow of 265 L/s, consistent with the high value properties. For any new developments, it is recommended that the following fire flow requirements be used:

- Single Family Residential 100 L/s
- Multiple Family Residential 180 L/s
- Institutional Areas (i.e. Schools) 130 L/s
- Industrial and Commercial Areas 300 L/s

These fire flow rates are in accordance with the Yellowhead County standards.

Pressure Requirements

Typically during peak hour demand, a minimum pressure of 280 kPa should be maintained. The Yellowhead County standards indicate that a minimum pressure of 300 kPa be maintained during peak hour demand. The maximum system pressure typically should not exceed 700 kPa.

A minimum residual pressure of 140 kPa is required at ground level during maximum day plus fire flow demand at all locations in the system.

3.4 Existing System Description

Supply System

The Town of Edson is currently being serviced by twelve groundwater wells (well number 2, 3, 9, 12, 14, 15, 16, 17, 18, 19, 20 and Glenwood Well) located within the Town. Well No. 18 and the Glenwood Well discharge into the Degas and Glenwood reservoirs, respectively. Well No. 3 used to supply the reservoirs at 50 Street and 11 Avenue; however, since these reservoirs are not currently in use, Well No. 3 is not operational. Well Nos. 9 and 16 are not used for domestic use; however, they are utilized during fire flow conditions. Well Nos. 19 and 20 are in the process of being brought onto the system and were considered to be active for the existing development scenario. The design discharge rates for each of the groundwater wells are summarized in Table 3.3.

Table 3.3: Groundwater Well Data

Well Number	Ground Elevation (m)	Pump Setting Elevation (m)	Design Discharge (L/s)	
			Allowed	Pumping
2	932.4	897.5	5.7	3.4
3	938.8	905.5	6.5	0
9	905.0	867.8	6.5	0
12	909.2	869.5	15.2	5.6
14	912.7	872.5	19	8.7
15	914.0	863.0	19	4.9
16	914.1	901.7	7.6	0
17	932.0	870.4	2.8	2.7
18	925.0	865.6	9.5	13.6
19	920.0	884.9	3.2	3.2
20	920.0	884.2	2.6	2.6
Glenwood	912.1	872.8	3.8	3.0
TOTAL	**-**	**-**	**101.4**	**47.7**

Based on the allowed discharge rates indicated in Table 3.3, the total allowable discharge rate is approximately 101.4 L/s with all wells in operation and the pumping discharge is approximately 48 L/s. Since Well Nos. 9 and 16 are not currently used to supply maximum day demands, they were not included in the total.

Based on the Town of Edson design standards, the average day demand for the existing system is 63 L/s. Using a peaking factor of 2.0, the maximum day demand is 126 L/s. To meet this demand, all wells should be utilized, and an additional 25 L/s is required. Based on an approximate well discharge of 8.5 L/s, approximately 3 additional wells would be required to supply the design flows for the existing system.

However, as mentioned in Section 3.3, the 2007 measured water use in the Town of Edson was approximately 66 L/s for maximum day demand. The existing groundwater wells have sufficient capacity to provide this flow.

It is recommended that additional groundwater wells be considered once the measured maximum day demand approaches the allowed design discharge rate of 101.4 L/s.

It is recommended that Well No. 3 be brought back into service prior to the installation of additional wells. The flows should be limited to the allowable discharge rate of 6.5 L/s. Prior to re-commissioning Well No. 3, the well should be cleaned and the water quality tested. If the existing license is still valid, no correspondence with Alberta Environment is required. The pump curve for Well No. 3 was not available; therefore, for modelling purposes, a pump providing 6.5 L/s at 75 m of head was utilized for the analysis. If it is determined that the existing pump does not have this capacity, or is in need of repair, a pump capable of providing 6.5 L/s at 75 m of head is an adequate replacement.

Storage Reservoirs

The existing water distribution system presently has four reservoirs, as listed in Table 3.4. The Glenwood and Degas reservoirs are fed by the Glenwood Well and Well No. 18, respectively. The water from these reservoirs is then pumped into the distribution system. The two reservoirs at Grande Prairie Trail fill from the distribution system during low demand periods. During peak demand periods, these reservoirs supply water to the system by gravity. The total storage capacity of all four reservoirs is 6,530 m^3 as indicated in Table 3.4.

Table 3.4: Existing Reservoir Storage

Reservoir	Location	Volume (m^3)
Steel above ground reservoir	Grande Prairie Trail	2,273
Concrete above ground reservoir	Grande Prairie Trail	3,410
Degas Reservoir	Rodeo Road & Highway 16	147
Concrete underground reservoir	Wilmore Park Road & 3 Avenue (Glenwood)	700
Total Storage		**6,530**

As per the 2006 Alberta Environment Standards and Guidelines for Municipal Waterworks, Wastewater & Storm Drainage Systems, the storage volume requirements for the existing development condition include fire storage, equalization storage and emergency storage, and are summarized in Table 3.5.

Table 3.5: Storage Requirements – Existing Development Condition

Description	Existing Required Volume (m^3)
Fire Storage (265 L/s for 3 hours)	2,862
Equalization Storage - 25% of Maximum Day Demand (126 L/s)	2,722
Emergency Storage - 15% of Average Day Demand (63 L/s)	816
Total Required Storage	**6,400**

As indicated in Table 3.5, the total storage required by Alberta Environment is approximately 6,400 m^3. Therefore, the existing reservoirs in the Town are capable of providing adequate storage. Based on the design water consumption rates, the existing reservoir capacity is sufficient for a population increase of approximately 600 people. However, it should be noted that this is highly dependent on the amount of non-residential development that occurs.

Pumphouse Facilities

The Degas and Glenwood reservoirs are filled by Well Nos. 18, 19 and 20 and Glenwood Well, respectively. From these reservoirs the water is pumped into the distribution system. The Degas pumphouse has two identical 9 HP pumps, each capable of providing 14.2 L/s at 30 m of head.

Both pumps run when the water level is 2 m in the Degas reservoir, and one pump shuts off when the water level drops below 1.5 m. In order to simulate the worst case, only one pump was modeled in this study. The Glenwood pumphouse has one distribution pump, capable of providing 7.6 L/s at 47 m of head, and one fire pump, capable of providing 48.6 L/s at 49 m of head.

There is also an in-line booster station located at Edson Drive and 13 Avenue that boosts the pressure into Zone 2. The existing booster pump station has a floor elevation of 929.05 m and is equipped with six pumps. The three smaller 10 HP pumps (P101, P102 and P103) have an individual capacity of delivering 13.3 L/s at a total dynamic head (TDH) of 24.4 m. The three larger 20 HP pumps (P104, P105 and P106) have an individual capacity of delivering 44.6 L/s at TDH of 24.4 m.

The operating philosophy for all the pumps located in the Zone 2 booster station was taken from the report "Town of Edson Contract Documents for Construction of Zone 2 Water Distribution Pumphouse and Valve Chamber, June 1985, UMA Engineering Ltd.". The pumps are set to automatically start and stop depending on flow settings as follows:

- P101 — P101 starts at 5 L/s, stops at 3 L/s
- P101+P102 — P102 starts at 13 L/s, stops at 11 L/s
- P101+P102+P103 — P103 starts at 27 L/s, stops at 25 L/s
- P101+P102+P103+P104 — P104 starts at 40 L/s, stops at 38 L/s
- P101+P102+P103+P104+P105 — P105 starts at 84 L/s, stops at 82 L/s
- P101+P102+P103+P104+P105+P106 — P106 starts at 129 L/s, stops at 127 L/s

The pumps boost the pressure into Zone 2 to 663 kPa. When all six pumps are running in parallel, the pumps are capable of providing approximately 174 L/s at 24.4 m of TDH at the Zone 2 booster station. The typical flows for Zone 2 are included in Table 3.6. During average day demand conditions and maximum day demand conditions, pump P101 will be running; pumps P102 will start during peak hour demand conditions. All six pumps are required to supply the maximum day demand plus fire flow demands and can operate in the event of a power failure.

The flow requirements for the Town of Edson are summarized in Table 3.6.

Table 3.6: Flow Requirements – Existing Development Condition

Demand Scenario	Total Required Flow (L/s)	Zone 1 Required Flow (L/s)	Zone 2 Required Flow (L/s)
Average Day Demand	63	58.3	4.7
Maximum Day Demand	126	116.7	9.3
Peak Hour Demand	189	175	14.0
Fire Flow	265	265	150
Maximum Day Demand plus Fire Flow	391	382	159

As indicated in Table 3.6, the maximum day demand plus fire flow scenario is the critical scenario for the Town of Edson, with a total flow requirement of approximately 391 L/s. Based on the preliminary model results, the above ground reservoirs at Grande Prairie Trail are capable of providing approximately 240 L/s (determined by simulating a fire flow at critical locations). However, the existing system is not capable of providing the required 391 L/s without dropping the pressure in the system below 140 kPa.

For the maximum day demand plus fire flow scenario, the pumping capacity of the existing system is approximately 102 L/s.

This is based on a flow of 39.4 L/s from the wells (excluding Well No. 3, and Well Nos. 18, 19, 20 and the Glenwood Well which discharge into reservoirs), 48.6 L/s from the Glenwood Fire Pump, and 14.2 L/s from the Degas pump. Even though the well pumps may not have a backup supply in case of power failure, they were considered in the maximum day demand plus fire flow scenario. Since the wells are located all around the Town, it is highly unlikely that all wells would lose power. The pumps at the Zone 2 booster station are inline booster pumps were therefore not considered, as they do not provide additional flow, only increase the pressure.

Water Distribution System

The existing distribution system consists of pipe sizes varying from 100 mm to 350 mm in diameter within the Town's residential and industrial/commercial developments. The water distribution system is to provide both domestic water supply and fire protection.

3.5 System Modelling

3.5.1 Existing Model Development

The model utilized in this analysis was originally developed by UMA Engineering Ltd. as part of the Town of Edson Water Distribution System Analysis, April 2005. Updates to the existing model include:

- Addition of all new development and pipe upgrades which have occurred since 2005.
- Addition of groundwater wells 19 and 20.
- Updated demands to reflect 2007 water consumption rates.

The distribution system model for the Town of Edson was developed using WaterCAD version 6.5, developed by Haested Methods Inc., and later upgraded to WaterCAD version 8i (Bentley Systems Ltd.). This model has the capacity to model both steady state and extended period simulations. The program requires physical details of the existing distribution system, such as pipe diameters, lengths, roughness coefficients, water consumption demands, and ground elevations to represent the water distribution system through pipes and junction nodes. The distribution system data was obtained from water distribution system drawings. Ground elevations at nodes were estimated from available topographic maps.

The existing distribution system schematic is shown on Figures 3.1 and 3.2. The demands were estimated by counting the number of lots for single-family residential developments and measuring areas for all other land uses, including multi-family residential developments. Water consumption rates provided by Town of Edson were used to estimate the demands for the existing nodes.

3.5.2 Model Calibration

The Town of Edson distribution pipe material consists of mainly asbestos cement (AC), PVC and cast iron (CI), with some ductile iron (DI) pipes.

The existing system was calibrated by simulating several alternatives for the Hazen-Williams coefficient (C), including the following:

- C=110 for AC, CI, and DI and C=120 for PVC
- C=100 for AC, CI and DI and C=120 for PVC
- C=100 for AC, CI and DI and C=110 for PVC
- C=90 for CI and DI, C=100 for AC, and C=120 for PVC

The system was analyzed for average day demand with fire pumps and the generator set at the Zone 2 booster station in operation, as these were in operation during the entire duration of the hydrant flow tests as confirmed by Town personnel. The average day demands were based on the water consumption data provided by the Town, which is a total of 33 L/s. The average day demands for the industrial areas were also based on existing water consumption and vary from 1,500 L/ha/day to 13,600 L/ha/day.

The measured hydrant test data was analyzed and extrapolated to estimate the available flows at 140 kPa. The flows simulated in the model were then compared with the field hydrant test data for each of the alternatives. The comparison of these measured and simulated results are summarized in Table 3.7.

Table 3.7: Model Calibration

Test Number	Junction Number	Available Flow at 140 kPa (L/s)	Simulated Flow at 140 kPa (L/s)			
			AC=CI=DI=110/ PVC=120	AC=CI=DI=100/ PVC=120	AC=CI=DI=100/ PVC=110	CI=DI=90/ AC=100/PVC=120
1	J-601	179	212	200	197	199
2	J-1685	303	282	263	260	264
3	J-1176	95	80	77	77	77
4	J-2530	194	202	188	184	177
5	J-1360	224	241	226	221	225

Based on the simulation results in Table 3.7, at most locations the hydrant test results generally match closest with the simulated flows based on a roughness coefficient of 110 for asbestos cement, cast iron, and ductile iron, and 120 for PVC pipes. These C values provide a representation of the actual water distribution system, and account for unknown conditions, such as partially closed valves, pump inefficiencies, etc.

For the subsequent analysis, a roughness coefficient of 110 for asbestos cement, cast iron and ductile iron and 120 for PVC pipes was adopted as the simulation results indicated a better match with the hydrant flow test results compared to the other scenarios.

3.6 System Evaluation under Existing Development Conditions

Hydraulic analyses for the following demands were carried out for the Town of Edson water distribution system:

- Peak Hour Demand
- Maximum Day Demand Plus Fire Flow

3.6.1 Peak Hour Demand

The existing distribution system was simulated for the peak hour demand assuming flows from the Grande Prairie Trail reservoirs, the Glenwood distribution pump (supply from the Glenwood Well), the Degas pump (supply from Well Nos. 18, 19 and 20), and all active wells (2, 9A, 12, 14, 15, 16, and 17). As a result, a residual minimum pressure of 90 kPa was simulated at node J-300. The minimum residual pressure is lower than the recommended minimum pressure of 280 kPa (40 psi). A maximum simulated residual pressure of 635 kPa occurred at node J-2514 which is below the recommended maximum of 700 kPa. Hence, the system is not adequate to supply the peak hour demands and some improvements are required. System improvements are further evaluated in Section 3.8.

The pressure contours for the peak hour demand are shown on Figure 3.3.

3.6.2 Maximum Day Demand plus Fire Flows

The distribution system was simulated for the maximum day plus fire flow demands assuming flows from the Grande Prairie Trail reservoirs, the Glenwood fire pump (supply from the Glenwood Well), the Degas pump (supply from Well Nos. 18, 19 and 20) and at all active wells (2, 9A, 12, 14, 15, 16, and 17).

The Glenwood reservoir has a storage capacity of 700 m^3, as indicated in Table 3.4. Therefore, based on the capacity of the Glenwood fire pump (48.6 L/s), if the reservoir is full it could supplement the fire flow for a duration of 4 hours.

Fire flows of 265 L/s were assigned to junctions at or close to hydrants in non-residential areas (e.g. a gas station or shopping center). Similarly, fire flows of 76 L/s and 150 L/s were assigned to junctions in single family and multi-family residential developments, respectively.

Simulation runs were carried out to establish the available fire flow at a minimum recommended pressure of 140 kPa at selected locations within the distribution system. The simulation results shown that the existing system cannot provide the minimum fire flow requirements to the majority of the residential and non-residential areas.

The simulated fire flows for the existing development at selected nodes are summarized in Table 3.8. Although several of the nodes have a residual pressure greater than 140 kPa, the minimum pressure is occurring at another location within the system which limits the available flow.

Table 3.8: Available Fire Flow at Selected Nodes – Existing Development Condition

Node Number	Required Flow (L/s)	Available Flow (L/s)	Minimum Residual Pressure (kPa)	Minimum Zone Pressure (kPa)
J-140	265	188	201	140
J-390	265	223	140	140
J-733	265	104	252	140
J-1630	265	219	337	140
J-2230	76	47	140	171
J-2250	265	214	140	140
J-2510	265	124	142	140

The simulation results are illustrated on Figures 3.4 to 3.6. The simulation results for the existing development condition are included in Appendix B.

3.7 System Deficiencies

The existing water distribution system cannot generally provide adequate fire flows for the commercial and industrial areas, as well as for some of the residential areas. In addition, in the northwest area of Town, north of 16 Avenue, both east and west of 63 Street, the pressures are below 280 kPa during peak hour demand.

3.8 System Improvements

The main deficiencies in the Town of Edson water distribution system were further evaluated for improvement alternatives. The improvements for the existing system will focus on providing higher fire flows to the entire Town.

For the existing system, upgrades were considered only for upgrading the main lines in the water distribution system, where upgrades would be the most cost effective, and provide the greatest benefit.

Some of the local deficiencies in the existing system are resolved in the future scenarios, due to additional looping in the system. Therefore, upgrades are not recommended for these local deficiencies.

The proposed system upgrades, required to provide adequate pressure and fire flows to the existing system, are shown on Figures 3.7 to 3.10. Figure 3.7 includes the pressure contours for peak hour demand, while Figures 3.8 to 3.10 indicate the results of the maximum day demand plus fire flow. The detailed simulation results are included in Appendix B.

To increase the available fire flows and to increase the available pressure during peak hour demand, it is recommended that a booster station be constructed adjacent to the reservoirs at Grande Prairie Trail with a capacity of 290 L/s at 45 m of head. It is recommended that four pumps be installed (includes one backup pump), each capable of providing 100 L/s at 45 m of head. Space should be included for an additional pump in the future. To maintain the pressure requirements, a PRV is required along 63 Street, just south of the booster station. The PRV settings are summarized in Section 3.9.3. Modifications to the 350 mm diameter pipe that runs east/west from the Grande Prairie Trail reservoirs will also be required, such that the areas along 62 Street and 17 Avenue are serviced directly from the proposed booster station and the areas to the south are fed through the proposed PRV.

It is recommended that a 300 mm diameter main be installed along 1 Avenue, between 27 Street and 46 Street to provide looping in the system. Another small loop is recommended along 45 Street, between 4 Avenue and 5 Avenue.

In addition, to service the existing areas along 63 Street and 65 Street, north of 17 Avenue, a 300 mm loop is recommended. With the addition of a booster station adjacent to the reservoirs at Grande Prairie Trail, this area can be serviced directly from the proposed booster station.

In the industrial areas located in the southwest part of Town, there are several nodes that do not satisfy the fire flow requirements. The fire flow requirements for this area will be met with future looping; therefore, upgrades are not recommended at this stage. The 100 mm diameter water main servicing the area west of 70 Street and south of 4 Avenue does not provide adequate fire flows; however, this area is protected by the hydrant connected to the 300 mm diameter water main along 4 Avenue.

3.9 Future Servicing

The future development scenarios for 2015 and 2025 were analyzed assuming all recommended upgrading alternatives have been implemented. For the future water servicing, areas within the Town of Edson and the Urban Fringe Area within Yellowhead County were considered. The growth projections indicated in Table 2.1 were used for this analysis.

Since areas in both the Town of Edson and Yellowhead County are included, both servicing standards were considered in the sizing of the future distribution system; therefore, two alternatives were considered. Alternative 1 is based on the Town of Edson water consumption rates and Alternative 2 is based on the Yellowhead County water consumption rates. It was determined that the Town of Edson standards should be used for the purpose of the Municipal Servicing Plan; therefore, Alternative 1 was chosen. A more detailed description and the analysis for Alternative 2 are available in Appendix C.

In addition, a third alternative was developed for cost comparison purposes, in which only development within the Town of Edson was considered. Alternative 3 was based on the Town of Edson water consumption rates.

For all alternatives, additional pressure zones are proposed to help maintain the system pressure in an acceptable range.

3.9.1 Alternative 1 – Design with Town of Edson Standards

For Alternative 1, the Town of Edson Standards were used to analyze the 2015 and 2025 water distribution systems.

3.9.1.1 Supply System

The future maximum day demand requirements for 2015 and 2025 are 263 L/s and 409 L/s, respectively.

The current allowable discharge rate from the existing groundwater wells is approximately 101 L/s. Therefore, the existing supply system is approximately 162 L/s and 308 L/s short to supply the 2015 and 2025 systems. Based on an approximate well discharge of 8.5 L/s, approximately 19 additional wells will be required by 2015, and another 18 additional wells will be required by 2025.

Since the projected number of wells is based on the design standards, the actual consumption for the service area should be monitored to determine the number of wells required for supply.

As detailed in the General Engineering Study (1982), the long term viability of groundwater wells should be evaluated.

3.9.1.2 Storage Reservoirs

An adequate storage volume for the Town of Edson water distribution system is highly important. With increasing population and new developments, and also considering future potential customers, the existing reservoirs will not be sufficient.

Two options were considered for determining future storage volumes. In Option 1, the Alberta Environment Requirement of 25% of Maximum Day Demand (Equalization Storage) plus 15% of Average Day Demand (Emergency Storage) plus Fire Flow was considered. In this option the equalization storage is assigned to meet the daily demand fluctuation above the supply rates, as the water supply rate is generally lower than the peak water consumption rate. The emergency storage is allocated for the routine disruption of supply for maintenance.

In Option 2, two times Average Day Demand (Supply Interruption) plus fire storage was considered for the storage volume. The supply interruption storage represents the available storage in case of a disruption to the water supply.

Tables 3.9 and 3.10 summarize the 2015 storage requirements for the two options, and Tables 3.11 and 3.12 summarize the 2025 storage requirements for the two options.

During a fire flow scenario, the system will draw water from the closest reservoirs. Similar to the General Engineering Study (1982), it is recommended that the storage requirements for Zones 1, 4, 5 and 6 be considered independently from Zones 2 and 3. The fire flow condition was simulated to determine what portion of the fire storage needed to be allocated to which reservoirs. It was determined that the proposed reservoir west of the Town needs to provide fire storage for 300 L/s for 4 hours and the reservoirs at Grande Prairie Trail need to provide fire storage for 300 L/s for 4 hours. Therefore, the total fire storage requirement for the Town is 600 L/s for a 4 hour duration.

2015 Development Condition

Table 3.9: Option 1 Storage Requirement – Alternative 1 2015 Development Condition

Description	Required Volume (m^3)
Fire Storage (300 L/s and 300 L/s for 4 hours)	8,640
Equalization Storage - 25% of Maximum Day Demand (263 L/s)	5,681
Emergency Storage - 15% of Average Day Demand (132 L/s)	1,711
Total Required Storage	**16,032**

Table 3.10: Option 2 Storage Requirement – Alternative 1 2015 Development Condition

Description	Required Volume (m^3)
Fire Storage (300 L/s and 300 L/s for 4 hours)	8,640
Two times Average Day Demand (132 L/s)	22,810
Total Required Storage	**31,450**

2025 Development Condition

Table 3.11: Option 1 Storage Requirement – Alternative 1 2025 Development Condition

Description	Required Volume (m^3)
Fire Storage (300 L/s and 300 L/s for 4 hours)	8,640
Equalization Storage - 25% of Maximum Day Demand (409 L/s)	8,835
Emergency Storage - 15% of Average Day Demand (205 L/s)	2,657
Total Required Storage	**20,132**

Table 3.12: Option 2 Storage Requirement – Alternative 1 2025 Development Condition

Description	Required Volume (m^3)
Fire Storage (300 L/s and 300 L/s for 4 hours)	8,640
Two times Average Day Demand (205 L/s)	35,424
Total Required Storage	**44,064**

For both Options 1 and 2, upgrades will be required by the 2015 development condition.

The General Engineering Study (1982) recommended that the primary reservoir be constructed in the northwest part of Town, near the Microwave Tower Site. However, the study area for Alternative 1 includes additional development approximately 6.5 km west of the study area considered in 1982. It is recommended that a new reservoir be constructed west of the Town in the Urban Fringe Area, as this is the most cost effective location. Although providing storage by the Microwave Tower Site would eliminate the duplication in fire storage, extensive pipe upgrades to convey the fire flow to the west areas would be required, nearly doubling the capital cost.

3.9.1.3 Pumphouse Facilities

The future pumping requirements for the Town of Edson are based on the projected growth indicated in Section 2.4. Table 3.13 summarizes the future pumping requirements.

Table 3.13: Future Pumping Requirements – Alternative 1

Demand Scenario	Existing (L/s)	Future (L/s)	
		2015	2025
Average Day Demand	63	132	205
Maximum Day Demand	126	263	409
Peak Hour Demand	189	394	614
Fire Flow	265	300	300
Maximum Day Demand Plus Fire Flow	391	563	709

It is important to note that although the fire flow requirement is 300 L/s, the Grande Prairie Trail booster station and West Reservoir and Pumphouse must each be capable of providing this flow. Therefore, the total pumping capacity will exceed the required maximum day demand plus fire flow.

For the 2015 development condition, approximately 450 L/s of pumping capacity is required in the north portion of the study area, and approximately 340 L/s of pumping capacity is required in the west portion of the study area. It is recommended that capacity of the proposed Grande Prairie Trail booster station be increased from 290 L/s to approximately 330 L/s at 45 m of head.

It is recommended that the proposed West Reservoir and Pumphouse be capable of providing approximately 300 L/s at 45 m of head, assuming the Glenwood pumphouse remains in operation. The additional pumping capacity can be provided through the future groundwater wells (162 L/s). However, it should be noted that the locations of the future groundwater wells may affect the pumping requirements at the Grande Prairie Trail booster station and the West Reservoir and Pumphouse.

For the 2025 pumping requirement, approximately 500 L/s of pumping capacity is required in the north portion of the study area, and approximately 415 L/s of pumping capacity is required in the west portion of the study area. The proposed West Reservoir and Pumphouse should be upgraded to provide 300 L/s at 71.5 m of head to provide adequate pressures to the west developments. The groundwater wells can provide the additional pumping capacity (146 L/s).

3.9.1.4 Water Distribution System

The pipe sizes required to service the future development, as well as the schematic pipe layouts, are indicated on Figures 3.11 to 3.13 for 2015, and Figures 3.20 to 3.22 for 2025.

For the 2015 development conditions, the pipe diameters range from 150 mm to 350 mm. To satisfy the fire flow requirements for the 2015 development condition, it is recommended that the pipes along 4 Avenue, from 68 Street to 70 Street, be upgraded from 150 mm to 300 mm in diameter. The pipes required for the 2025 development condition are 250 mm to 350 mm in diameter.

For the 2015 and 2025 development scenarios, all of the fire flow requirements are satisfied, and the pressures during peak hour demand are within an acceptable range (280 kPa to 700 kPa). The nodes at which the fire flow requirements are not met do not provide flow to the hydrants. The 2015 and 2025 peak hour demand results are shown on Figures 3.14 and 3.23, respectively. The 2015 and 2025 maximum day demand plus fire flow results are shown on Figures 3.15 and 3.24, respectively.

Detailed simulation results for the peak hour demand and maximum day demand plus fire flow scenarios for the 2015 and 2025 development conditions are included in Appendix B.

3.9.2 Alternative 3 – Design with Town of Edson Standards – Excluding Yellowhead County Development

For Alternative 3, the Town of Edson Standards were used to analyze the 2015 and 2025 water distribution systems. Alternative 3 excludes future Yellowhead County development.

3.9.2.1 Supply System

The future maximum day demand requirements for 2015 and 2025 are 202 L/s and 220 L/s, respectively. The current allowable discharge rate from the existing groundwater wells is approximately 101 L/s. Therefore, the existing supply system is approximately 101 L/s and 18 L/s short to supply the 2015 and 2025 systems. Based on an approximate well discharge of 8.5 L/s, approximately 12 additional wells will be required by 2015, and 3 additional wells will be required by 2025.

Since the projected number of wells is based on the design standards, the actual consumption for the service area should be monitored to determine the number of wells required for supply.

3.9.2.2 Storage Reservoirs

As mentioned in Section 3.9.1.2, two options were considered for determining future storage volumes. In Option 1, the Alberta Environment Requirement of 25% of Maximum Day Demand (Equalization Storage) plus 15% of Average Day Demand (Emergency Storage) plus Fire Flow was considered. In this option, the equalization storage is assigned to meet the daily demand fluctuation above the supply rates, as the water supply rate is generally lower than the peak water consumption rate. The emergency storage is allocated for the routine disruption of supply for maintenance.

In Option 2, two times Average Day Demand (Supply Interruption) plus fire storage was considered for the storage volume. The supply interruption storage represents the available storage in case of a disruption to the water supply. Tables 3.14 and 3.15 summarize the 2015 storage requirements for the two options, and Tables 3.16 and 3.17 summarize the 2025 storage requirements for the two options.

2015 Development Condition

Table 3.14: Option 1 Storage Requirement – Alternative 3 2015 Development Condition

Description	Required Volume (m^3)
Fire Storage (300 L/s and 300 L/s for 4 hours)	8,640
Equalization Storage - 25% of Maximum Day Demand (202 L/s)	4,363
Emergency Storage - 15% of Average Day Demand (101 L/s)	1,309
Total Required Storage	14,312

Table 3.15: Option 2 Storage Requirement – Alternative 3 2015 Development Condition

Description	Required Volume (m^3)
Fire Storage : (300 L/s and 300 L/s for 4 hours)	8,640
Two times Average Day Demand (101 L/s)	17,453
Total Required Storage	26,093

2025 Development Condition

Table 3.16: Option 1 Storage Requirement – Alternative 3 2025 Development Condition

Description	Required Volume (m^3)
Fire Storage (300 L/s and 300 L/s for 4 hours)	8,640
Equalization Storage - 25% of Maximum Day Demand (220 L/s)	4,752
Emergency Storage - 15% of Average Day Demand (110 L/s)	1,426
Total Required Storage	14,818

Table 3.17: Option 2 Storage Requirement – Alternative 3 2025 Development Condition

Description	Required Volume (m^3)
Fire Storage : (300 L/s and 300 L/s for 4 hours)	8,640
Two times Average Day Demand (110 L/s)	19,008
Total Required Storage	27,648

For Alternative 3, it is recommended storage be provided on the west side of the Town. Similarly to Alternative 1, construction of a new reservoir by the Microwave Tower Site would require extensive pipe upgrades to supply adequate fire flows to the west areas.

Table 3.18 summarizes the water storage requirements for both Alternatives 1 and 3, for Options 1 and 2.

Table 3.18: Water Storage Summary

Development Condition	Alternative 1 (m^3)	Alternative 3 (m^3)
Option 1		
2015	16,032	14,312
2025	20,132	14,818
Option 2		
2015	31,450	26,093
2025	44,064	27,648

It is recommended that the Alberta Environment guidelines, Option 1, be used to determine storage requirements, providing the groundwater wells are capable of supplying the maximum day demand. As development occurs, it is recommended that a sufficient number of groundwater wells be kept in production such that maximum day demand can always be supplied to the system.

3.9.2.3 Pumphouse Facilities

The future pumping requirements for the Town of Edson are based on the projected growth indicated in Section 2.4. Table 3.19 summarizes the future pumping requirements for Alternative 3.

Table 3.19: Future Pumping Requirements – Alternative 3

Demand Scenario	Future (L/s)	
	2015	2025
Average Day Demand	101	110
Maximum Day Demand	202	220
Peak Hour Demand	303	329
Fire Flow*	300	300
Maximum Day Demand Plus Fire Flow	502	520

For the 2015 development condition, approximately 335 L/s of pumping capacity is required in the north portion of the study area, and approximately 350 L/s of pumping capacity is required in the west portion of the study area. It is recommended that capacity of the proposed Grande Prairie Trail booster station be increased from 290 L/s to approximately 300 L/s at 45 m of head. It is recommended that the proposed West Reservoir and Pumphouse be capable of providing approximately 300 L/s at 45 m of head, assuming the Glenwood pumphouse remains in operation. The additional pumping capacity can be provided through the future groundwater wells (101 L/s).

For the 2025 pumping requirement, approximately 375 L/s of pumping capacity is required in the north portion of the study area, and approximately 365 L/s of pumping capacity is required in the west portion of the study area. The proposed Grande Prairie Trail booster station should be further upgraded to provide 321 L/s at 45 m of head. The groundwater wells can provide the additional pumping capacity (18 L/s).

It should be noted that the locations of the future groundwater wells may affect the pumping requirements at the Grande Prairie Trail booster station and the West Reservoir and Pumphouse.

Table 3.20 summarizes the 2015 and 2025 pumping requirements for both Alternatives 1 and 3 for the maximum day demand plus fire flow condition.

Table 3.20: Pumping Requirements Summary

Development Condition	Alternative 1 (L/s)	Alternative 3 (L/s)
2015	563	502
2025	709	520

It is important to note that although the system is designed for a single fire flow of 300 L/s, the Grande Prairie Trail booster station and West Reservoir and Pumphouse must each be capable of providing this flow.

As a fire could occur close to one of these reservoirs/pumphouses, the system may not be able to deliver flow from one location to another. Therefore, the total pumping capacity will exceed the required maximum day demand plus fire flow.

3.9.2.4 Water Distribution System

The pipe sizes required to service the future development, as well as the schematic pipe layouts, are indicated on Figures 3.16 and 3.17 for 2015, and Figures 3.25 and 3.26 for 2025.

As indicated on Figures 3.16 and 3.17, additional water main improvements are required to service the 2015 development for Alternative 3. These improvements provide additional looping, which provides increased fire flows, and range in diameter from 150 mm to 300 mm.

For the 2015 and 2025 development scenarios, all of the fire flow requirements are satisfied. The 2015 and 2025 peak hour demand results for Alternative 3 are shown on Figures 3.18 and 3.27, respectively. The 2015 and 2025 maximum day demand plus fire flow results are shown on Figures 3.19 and 3.28, respectively.

3.9.3 Pressure Zones

For both Alternatives 1 and 3, several pressure zones will be required for the 2015 and 2025 systems due to the large difference in elevations across the study area.

The pressure zones remain the same for the 2015 and 2025 development conditions. For Alternative 1, six pressure zones are required; five pressure zones are required for Alternative 3. These pressure zones are indicated in Figures 3.29 and 3.30 for Alternatives 1 and 3, respectively.

As indicated on Figure 3.29, for Alternative 1, it is recommended that for future servicing the area south of 13 Avenue and west of 56 Street is part of the Zone 1 system, as it is currently. This varies from the General Engineering Study (1982), which recommends that this area be part of Zone 2. Subsequent to the General Engineering Study, the Zone 2 booster station has been constructed to increase the pressure to Zone 2. To include the areas south of 13 Avenue in Zone 2, new water mains would be required to tie the two areas together. As a booster station is proposed at the Grande Prairie Trail reservoirs, the area south of 13 Avenue will receive adequate flows and pressures as part of Zone 1.

To maintain the separate pressure zones, the recommended pressure settings for the existing and proposed pressure reducing valves are summarized in Tables 3.21 to 3.23.

Table 3.21: PRV Settings - Existing With Improvements

PRV	Required Size (mm)	Pressure Setting (kPa)	Maximum Upstream Pressure (kPa)	Maximum Downstream Pressure (kPa)	Required Flow (L/s)
PRV-19	350	200	713.4	200	0 - 315

For Alternative 1, once the east area is brought into a separate pressure zone (Zone 6) it is recommended that the pressure setting of PRV-19 be increased, as indicated in Table 3.22. Similarly for Alternative 3, once Zone 4 is established, it is recommended that the pressure setting of PRV-19 be increased, as indicated in Table 3.23.

Table 3.22: PRV Settings - Alternative 1 (2025)

PRV	Required Size (mm)	Pressure Setting (kPa)	Maximum Upstream Pressure (kPa)	Maximum Downstream Pressure (kPa)	Required Flow Range (L/s)
PRV-17	350	450	695	450	0 - 205
PRV-16	350	450	535	450	0 - 260
PRV-14	350	525	635	525	0 - 370
PRV-19	350	275	605	275	0 - 570
PRV-20	350	450	565	450	0 - 230
PRV-21	350	500	595	550	0 - 150
PRV-22	300	300	695	325	0 - 100

Table 3.23: PRV Settings - Alternative 3 (2025)

PRV	Required Size (mm)	Pressure Setting (kPa)	Maximum Upstream Pressure (kPa)	Maximum Downstream Pressure (kPa)	Required Flow Range (L/s)
PRV-19	350	290	420	290	0-310
PRV-20	350	460	550	460	0-235
PRV-21	300	560	650	560	0-140
PRV-22	300	300	415	325	0 - 80
PRV-56	350	410	580	410	0-120
PRV-57	200	495	665	495	0-75
PRV-59	350	570	740	570	0-160

3.10 Cost Estimates

The costs for the improvements are summarized in Tables 3.24 through 3.28. Costs are based on 2009 dollars, and include a 35% allowance for overhead and engineering, and contingency. Detailed cost breakdowns are provided in Appendix D.

The costs for additional groundwater wells are summarized in Table 3.24, for both Alternatives 1 and 3, based on a unit cost of $75,000/well.

Table 3.24: Cost Estimates – Groundwater Wells

Description	Alternative 1	Alternative 3
Existing Allowable Discharge (L/s)	101	101
Capacity Required-2015 (L/s)	263	202
Deficiency-2015 (L/s)	162	101
Additional Wells Required-2015	19	12
Cost (2015)	**$1,425,000**	**$900,000**
Capacity Required-2025 (L/s)	409	220
Deficiency-2025 (L/s)	146	18
Additional Wells Required-2025	18	3
Cost (2025)	**$1,350,000**	**$225,000**
Sub-Total	**$2,775,000**	**$1,125,000**
Engineering (10%)	$277,500	$112,500
Contingency (25%)	$693,750	$281,250
Total	**$3,746,250**	**$1,518,750**

Since the Town of Edson is fed through groundwater wells, and is not part of a regional system, it is recommended that the Option 1 reservoir upgrades be completed. The reservoir costs for Alternatives 1 and 3 are indicated in Table 3.25. The reservoir costs are based on a unit cost of $525/m^3 for additional storage; details are provided in Tables D.1 and D.2 in Appendix D.

Table 3.25: Cost Estimates – Reservoirs

Description	Alternative 1	Alternative 3
Available Storage (m^3)	6,530	6,530
Additional Storage Required-2015 (m^3)	9,502	8,288
Cost (2015)	**$4,988,550**	**$4,351,200**
Additional Storage Required-2025 (m^3)	4,100	899
Cost (2025)	**$2,152,500**	**$471,975**
Sub-Total	**$7,141,050**	**$4,823,175**
Engineering (10%)	$714,105	$482,318
Contingency (25%)	$1,785,270	$1,205,800
Total	**$9,640,430**	**$6,511,300**

The pumping costs for the existing system improvements, 2015 and 2025 for Alternatives 1 and 3 are indicated in Table 3.26. The pumping costs are based on a unit rate of $4600/HP, which includes the cost of a new building. The costs also include the necessary back up pumps. Details are provided in Tables D.3 through D.5 in Appendix D.

Table 3.26: Cost Estimates – Pumping

Description	Alternative 1	Alternative 3
Existing System Improvements		
Existing Pumping Capacity (L/s)	102	102
Capacity Required-Existing (L/s)	391	391
Deficiency-Existing (L/s)	289	289
Deficiency-Existing (HP)	273	273
Cost (Existing System Improvements)	**$1,257,719**	**$1,257,719**
2015		
Capacity Required-2015 (L/s)	790	685
Available Pumping Capacity (Includes 2015 groundwater wells)	553	492
Deficiency-2015 (L/s)	237	193
Deficiency-2015 (HP)	224	183
Cost (2015)	**$1,031,416**	**$839,930**
2025		
Capacity Required-2025 (L/s)	915	740
Available Pumping Capacity (Includes 2025 groundwater wells)	936	703
Deficiency-2025 (L/s)	0	37
Deficiency-2025 (HP)	0	35
Cost (2025)	**$-**	**$161,023**
Sub-Total	**$2,289,135**	**$2,258,671**
Engineering (10%)	$228,914	$225,867
Contingency (25%)	$572,284	$564,680
Total	**$3,090,350**	**$3,049,240**

For the existing system improvements, it is recommended that a pumphouse be constructed adjacent to the reservoirs at Grande Prairie Trail to boost the pressure from the reservoir.

The total costs for the proposed water main improvements are summarized in Tables 3.27 and 3.28 for Alternatives 1 and 3, respectively. The improvement costs include the pipe cost, as well as the installation and restoration costs.

Table 3.27: Cost Estimates – Water Main Improvements – Alternative 1

Pipe Diameter (mm)	Total Length (m)	Unit Cost ($/m)	Pipe Cost ($)	Restoration Cost ($)	Total Cost ($)
Existing System Improvements					
200	569	333	$189,477	$252,636	$442,113
300	4,195	520	$2,181,400	$1,862,580	$4,043,980
350	298	630	$187,740	$132,312	$320,052
				Sub-Total (Existing with Improvements)	**$4,806,145**
2015					
150	698	300	$209,400		$209,400
200	1,087	333	$361,971		$361,971
250	3,562	425	$1,513,850		$1,513,850
300	16,499	520	$8,579,480	$168,276	$8,944,836
350	27,657	630	$17,423,910		$17,423,910
				Sub-Total (2015)	**$28,453,967**
2025					
200	508	333	$169,164		$169,164
250	7,184	425	$3,058,200		$3,058,200
300	8,105	520	$4,214,600		$4,214,600
350	10,686	630	$6,732,180		$6,732,180
				Sub-Total (2025)	**$14,169,144**
				Sub-Total	**$47,429,256**
				Engineering (10%)	$4,742,926
				Contingency (25%)	$11,587,314
				Total	**$64,030,000**

Table 3.28: Cost Estimates – Water Main Improvements – Alternative 3

Pipe Diameter (mm)	Total Length (m)	Unit Cost ($/m)	Pipe Cost ($)	Restoration Cost ($)	Total Cost ($)
Existing System Improvements					
200	569	333	$189,477	$252,636	$442,113
300	4,195	520	$2,181,400	$1,862,580	$4,043,980
350	298	630	$187,740	$132,312	$320,052
				Sub-Total (Existing with Improvements)	**$4,806,145**
2015					
150	698	300	$209,400		$209,400
200	1,255	333	$417,915	$74,592	$492,507
250	1,683	425	$715,275		$715,275
300	12,199	520	$6,343,480	$168,276	$6,511,756
350	8,946	630	$5,635,980		$5,635,980
				Sub-Total (2015)	**$13,564,918**
2025					
200	508	333	$169,164		$169,164
300	3,354	520	$1,744,080		$1,744,080
350	2,130	630	$1,341,900		$1,341,900
				Sub-Total (2025)	**$3,255,144**
				Sub-Total	**$21,626,207**
				Engineering (10%)	$2,162,621
				Contingency (25%)	$5,406,552
				Total	**$29,196,000**

The cost estimates for the proposed pressure reducing valves are summarized in Table 3.29. The cost estimates include the valves, chambers and installation. For Alternative 1, PRV-13 will be located within the new pumphouse and reservoir located west of Town. The cost for PRV-13 has been included in the pumping cost and has therefore not been included separately in Table 3.29.

Table 3.29: Cost Estimates – Pressure Reducing Valves

Description	Size (mm)		Cost	
	Alternative 1	Alternative 3	Alternative 1	Alternative 3
Existing System Improvements				
PRV-19	350	350	$60,000	$60,000
2015				
PRV-14	350	-	$60,000	-
PRV-20	350	350	$60,000	$60,000
PRV-21	300	300	$60,000	$60,000
PRV-56	-	350	-	$60,000
PRV-57	-	200	-	$60,000
PRV-69	-	350	-	$60,000
		Sub-Total (2015)	**$180,000**	**$300,000**
2025				
PRV-16	350	-	$60,000	-
PRV-17	350	-	$60,000	-
PRV-22	300	300	$60,000	$60,000
		Sub-Total (2025)	**$180,000**	**$60,000**
		Sub-Total	**$420,000**	**$420,000**
		Engineering (10%)	$42,000	$42,000
		Contingency (25%)	$105,000	$105,000
		Total	**$567,000**	**$567,000**

The total costs for Alternative 1 and Alternative 3 are summarized in Table 3.30.

Table 3.30: Cost Estimate Summary

Description	Alternative 1	Alternative 3
Groundwater Well Cost	$3,746,250	$1,518,750
Reservoir Cost	$9,640,430	$6,511,300
Pumping Cost	$3,090,350	$3,049,240
Water Main Costs	$64,030,000	$29,196,000
Pressure Reducing Valve Costs	$567,000	$567,000
Total	**$81,074,030**	**$40,842,290**

Alternative 1 is the recommended alternative. As indicated in Table 3.30, the costs for Alternative 3, servicing the Town of Edson only, are approximately half of the Alternative 1 costs.

3.11 Implementation Plan

Recommendations for the implementation of the improvements can be based on the benefit they provide to the system, either by increasing available pressure or flow. Consideration should also be given to other factors, such as stakeholder acceptance, including public consultation, and traffic disruptions.

For the existing system improvements, it is recommended that a new pumphouse be constructed adjacent to the reservoirs at Grande Prairie Trail to boost the system pressure. This will increase the available fire flows within the Town, and also increase the pressures during peak hour demand, providing a consistent level of service for all areas of the Town. For the existing system, a pumping capacity of 290 L/s at 45 m of head is recommended. Additional space in the booster station should be allowed for as a provision for future upgrades. Pipe modifications and the new pressure reducing valve will be required with the implementation of the booster station.

Secondly, it is recommended that the 300 mm diameter connection be made along Highway 16 to provide additional looping within the system. This will increase the fire flows in the east area of Town.

The remaining upgrades are required to solve localized deficiencies; therefore, cost effectiveness should be considered for the implementation of the upgrades not included above. If pipe replacement is required due to pipe age or others factors, pipe upgrading should be considered at that time. It should be noted that some of the pipe upgrades indicated can be considered with road upgrades where possible to eliminate or reduce the restoration cost.

For reservoir storage, Option 1, the Alberta Environment guidelines are recommended. Based on the design water consumption rates, the existing reservoir capacity is sufficient for an average day demand of approximately 65 L/s; a population increase of approximately 600 people. This corresponds to between 15 ha and 25 ha, depending on the population density. However, since the current water consumption rate in the Town is approximately half of the design water consumption rate, it is recommended that the actual consumption rates be monitored for increases to determine the appropriate timing of a new reservoir.

For the 2015 development condition, the reservoir storage within the Town needs to be expanded. For Alternative 1, it is recommended that a new reservoir and pumphouse be constructed west of Town, in Yellowhead County. For Alternative 3, it is recommended that the new reservoir and pumphouse be constructed on the west side of Town; upgrades could be considered for the Degas Reservoir and Pumphouse. The development of the reservoir and pumping could be staged.

For Alternative 1, it is recommended that the pumping capacity for the proposed Grande Prairie Trail booster station for 2015 be increased to 330 L/s at 45 m of head.

It is recommended that the proposed West Reservoir and Pumphouse be capable of providing approximately 300 L/s at 45 m of head for 2015 development, assuming the Glenwood pumphouse remains in operation. For the 2025 pumping requirement, the proposed West Reservoir and Pumphouse should be further upgraded to provide 300 L/s at 71.5 m of head. Once the locations of the future groundwater wells are determined, the apportioned flows to the Grande Prairie Trail booster station and the West Reservoir and Pumphouse should be re-evaluated.

The future water mains and pressure reducing valves will be required as development occurs. For the existing areas within Yellowhead County, since these areas are already serviced, it is recommended that they be connected to the Town of Edson's water distribution system as adjacent development occurs.

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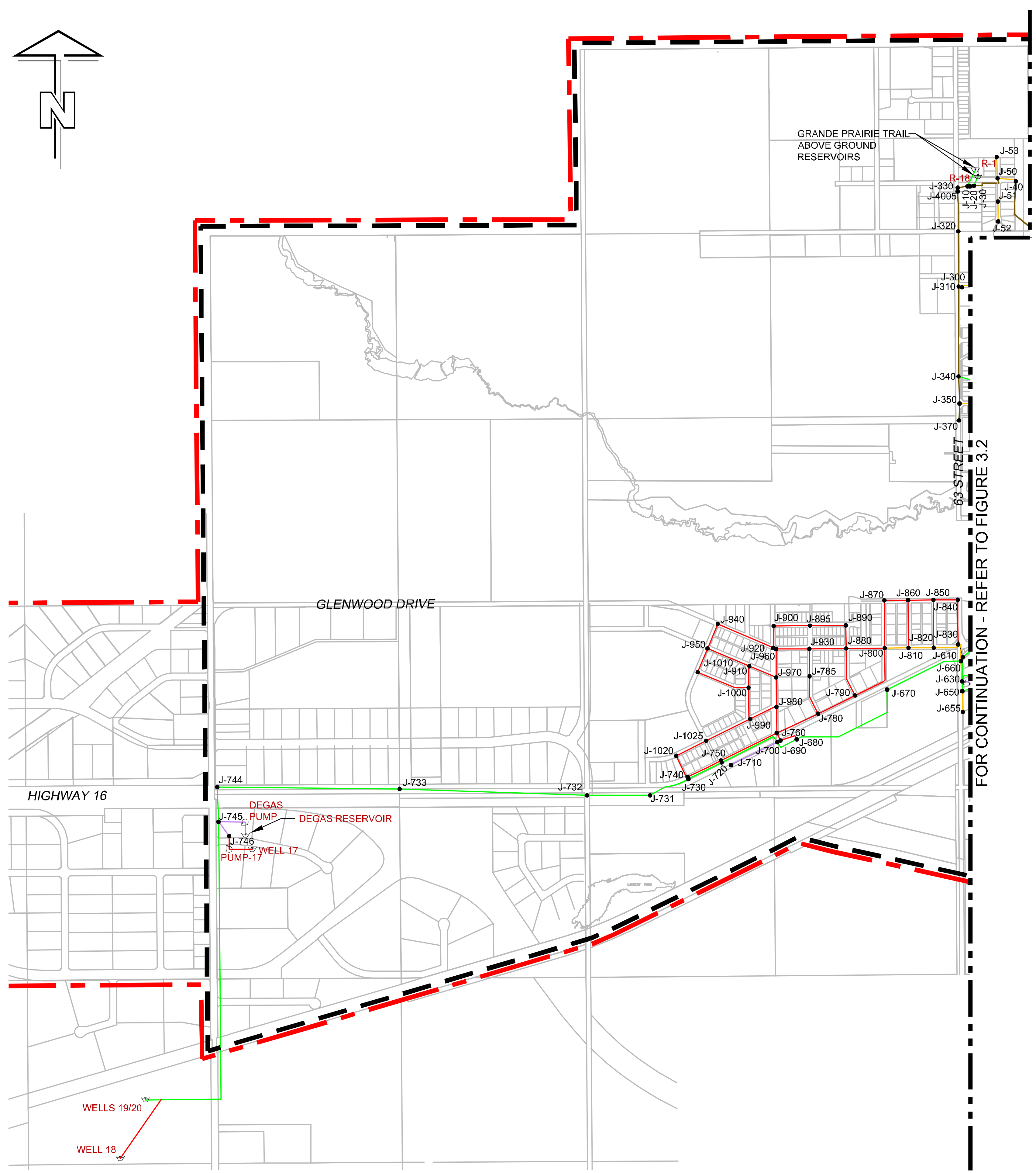

LEGEND

- STUDY AREA BOUNDARY
- TOWN OF EDSON BOUNDARY
- J-1670 SCHEMATIC NODE
- WELL 9A SCHEMATIC RESERVOIR/WELL
- PUMP-9A SCHEMATIC PUMP
- PRV-20 SCHEMATIC PRESSURE REDUCING VALVE
- 2F HYDRANT TESTING LOCATION
- EXISTING 100mm WATERMAIN
- EXISTING 150mm WATERMAIN
- EXISTING 200mm WATERMAIN
- EXISTING 250mm WATERMAIN
- EXISTING 300mm WATERMAIN
- EXISTING 350mm WATERMAIN

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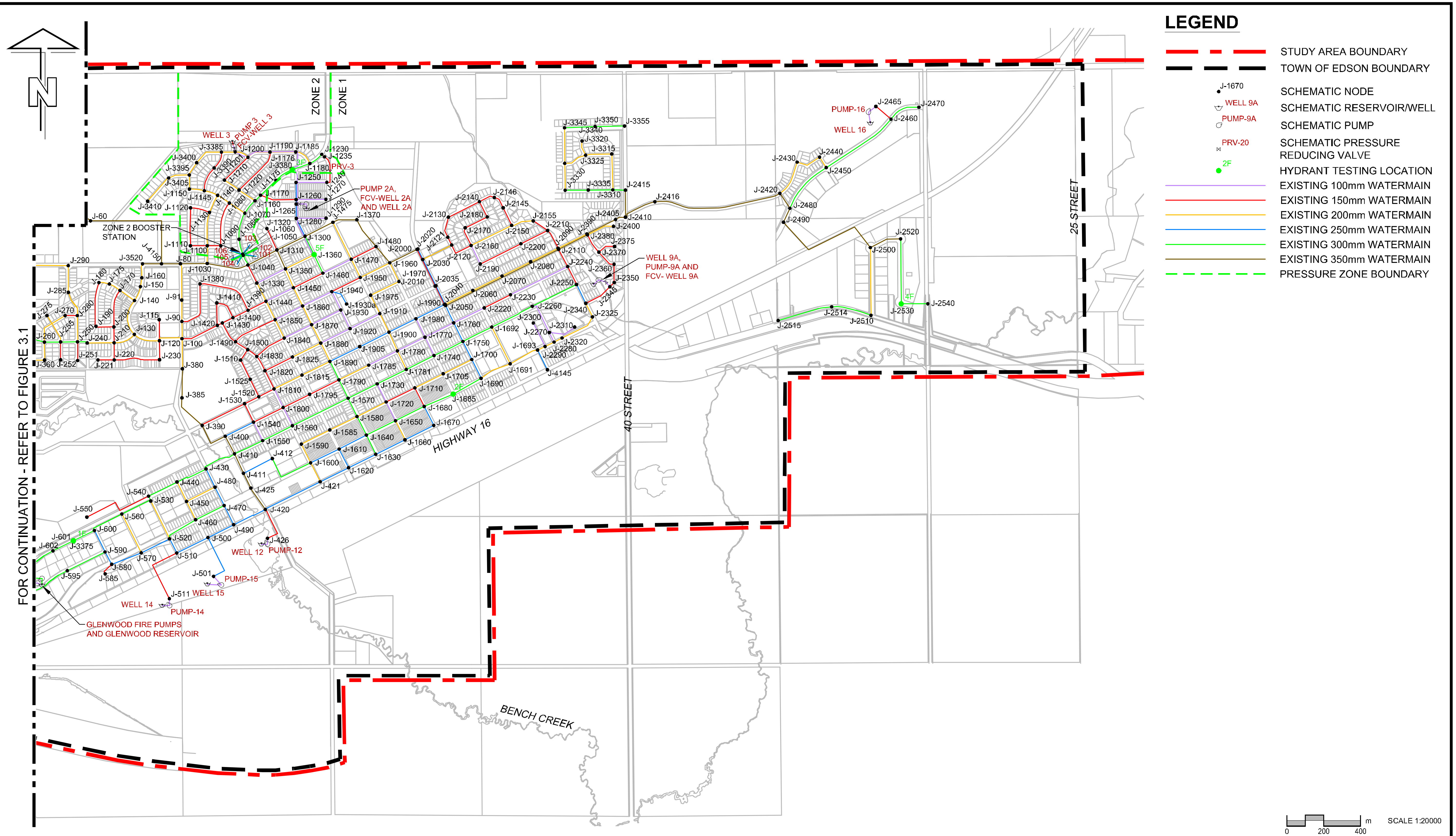
Town of Edson
Municipal Servicing Plan Update

Existing Water Distribution System

Figure 1 of 2

Figure 3.1



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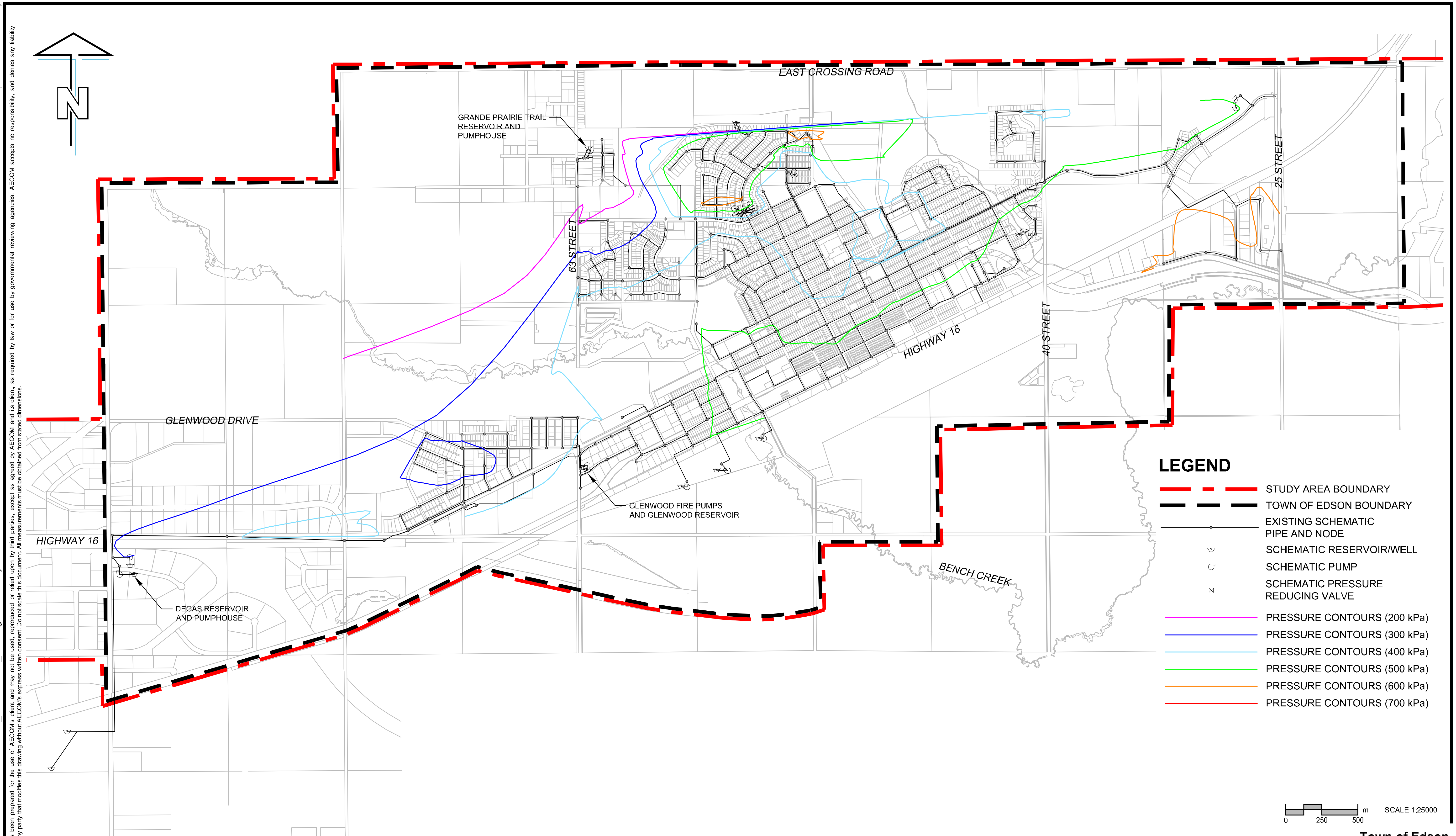
Town of Edson
Municipal Servicing Plan Update

Existing Water Distribution System

Figure 2 of 2

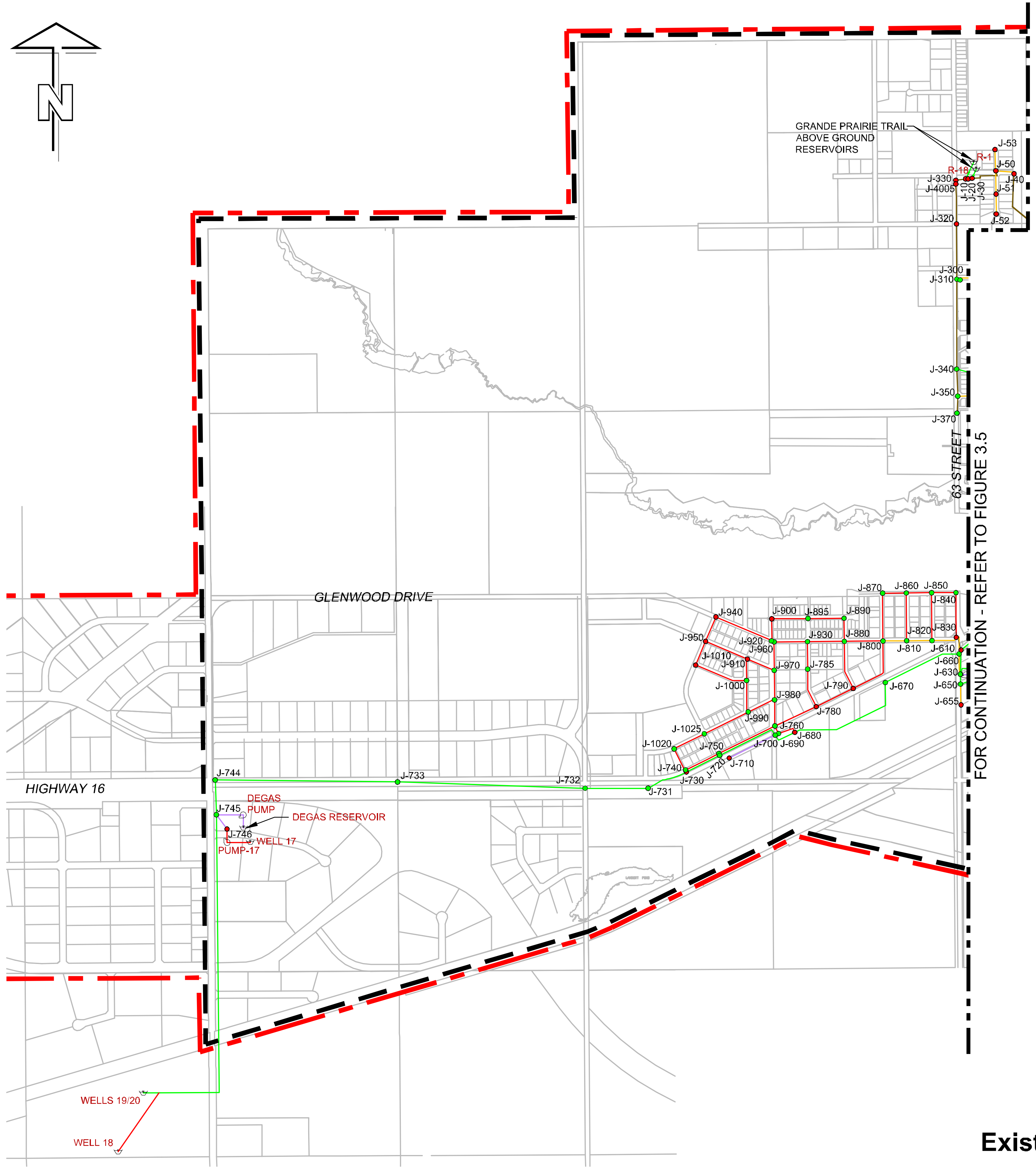
Figure 3.2



Town of Edson
Municipal Servicing Plan Update

Peak Hour Demand Contours
Existing Without Improvements

Figure 3.3



LEGEND

STUDY AREA BOUNDARY
TOWN OF EDSON BOUNDARY
J-1670 FAILED FIRE FLOW REQUIREMENTS
J-731 SATISFIED FIRE FLOW REQUIREMENTS
WELL 9A SCHEMATIC RESERVOIR/WELL
PUMP-9A SCHEMATIC PUMP
PRV-20 SCHEMATIC PRESSURE REDUCING VALVE
EXISTING 100mm WATERMAIN
EXISTING 150mm WATERMAIN
EXISTING 200mm WATERMAIN
EXISTING 250mm WATERMAIN
EXISTING 300mm WATERMAIN
EXISTING 350mm WATERMAIN

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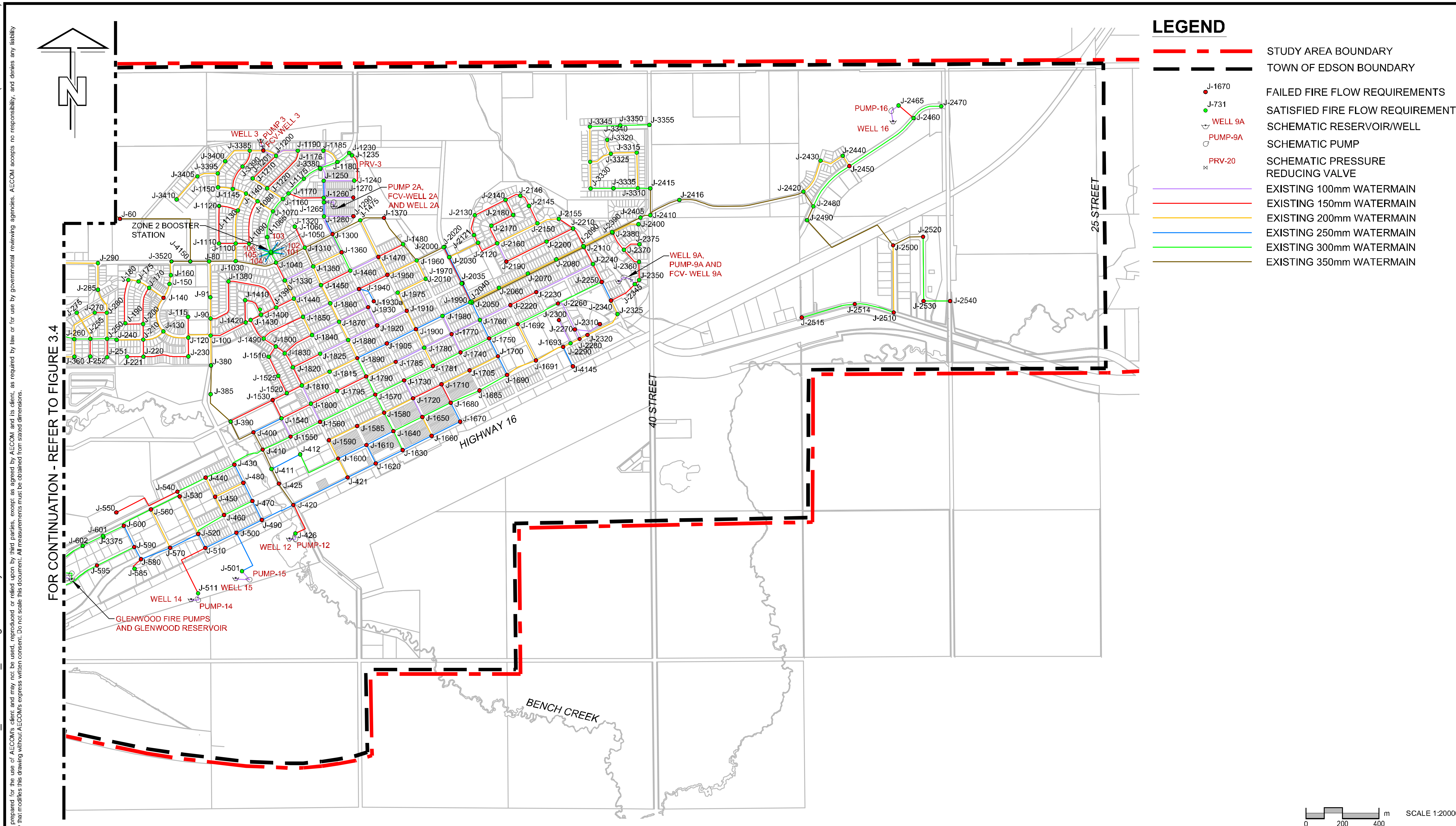
Town of Edson
Municipal Servicing Plan Update

Maximum Day Demand and Fire Flow Existing without Improvements - Figure 1 of 2

Figure 3.4



Town of Edson
Municipal Servicing Plan Update

Maximum Day Demand and Fire Flow Existing without Improvements - Figure 2 of 2

Figure 3.5

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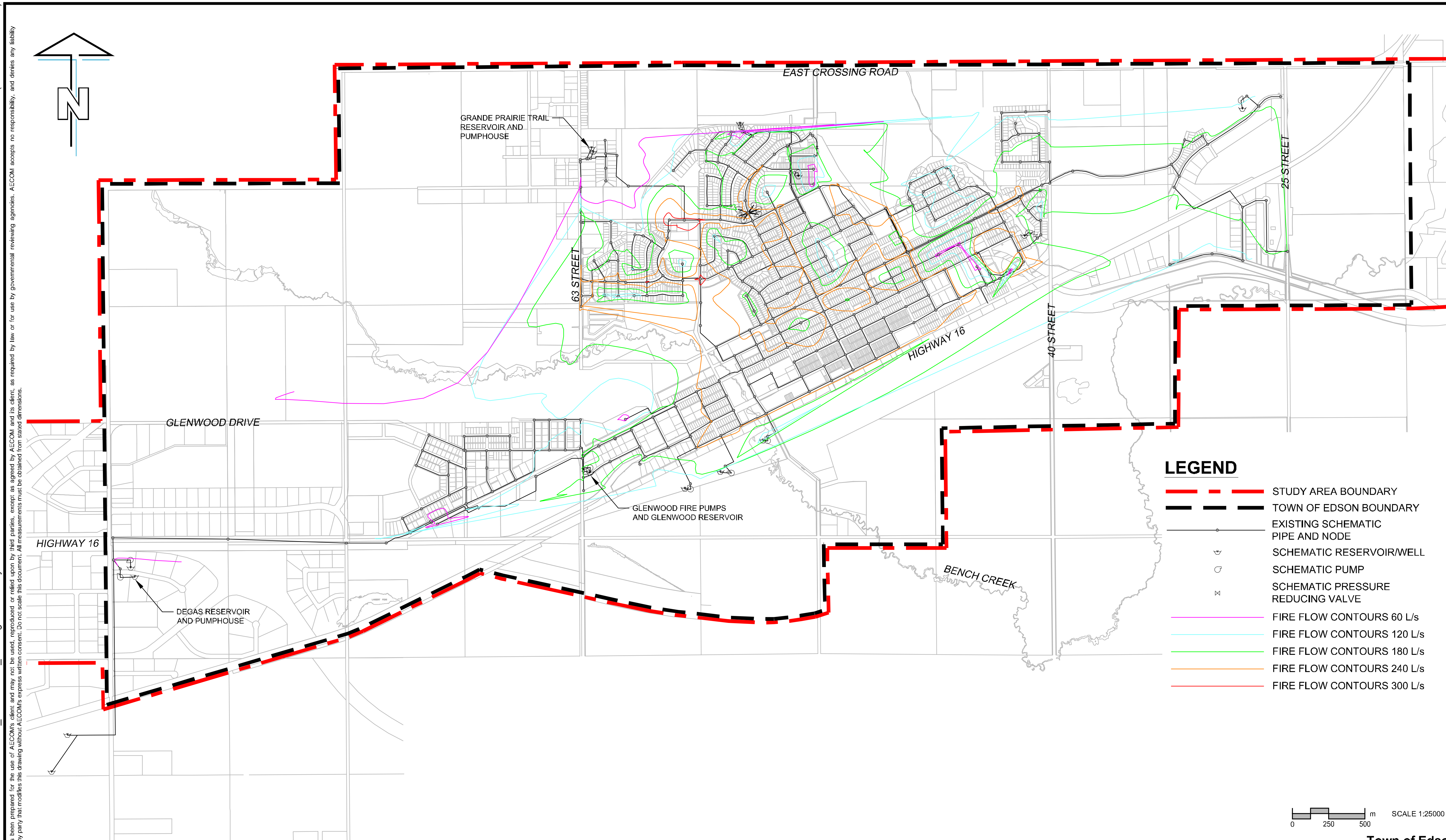

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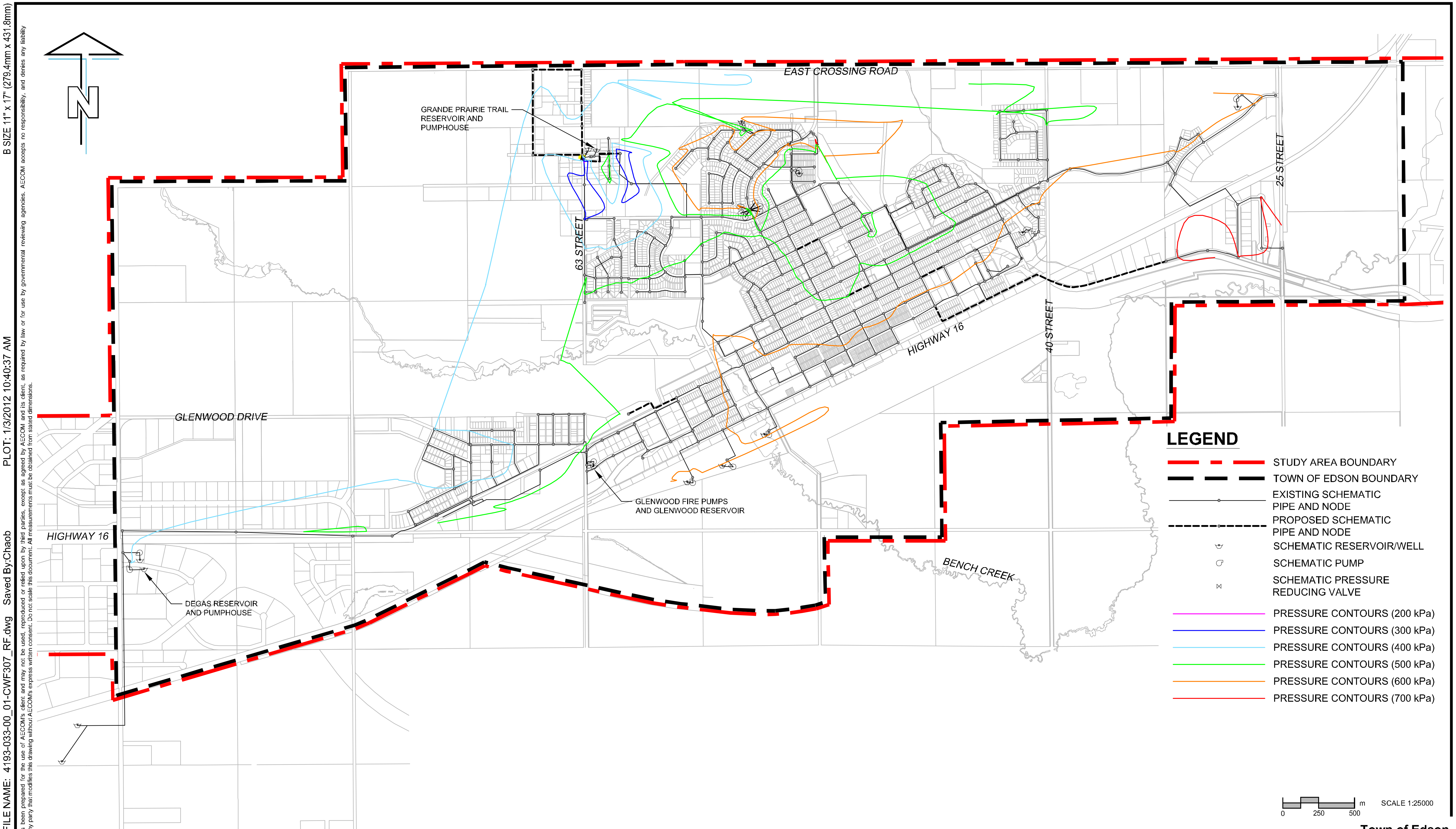
Town of Edson
Municipal Servicing Plan Update

Maximum Day Demand and Fire Flow Contours
Existing Without Improvements

Figure 3.6



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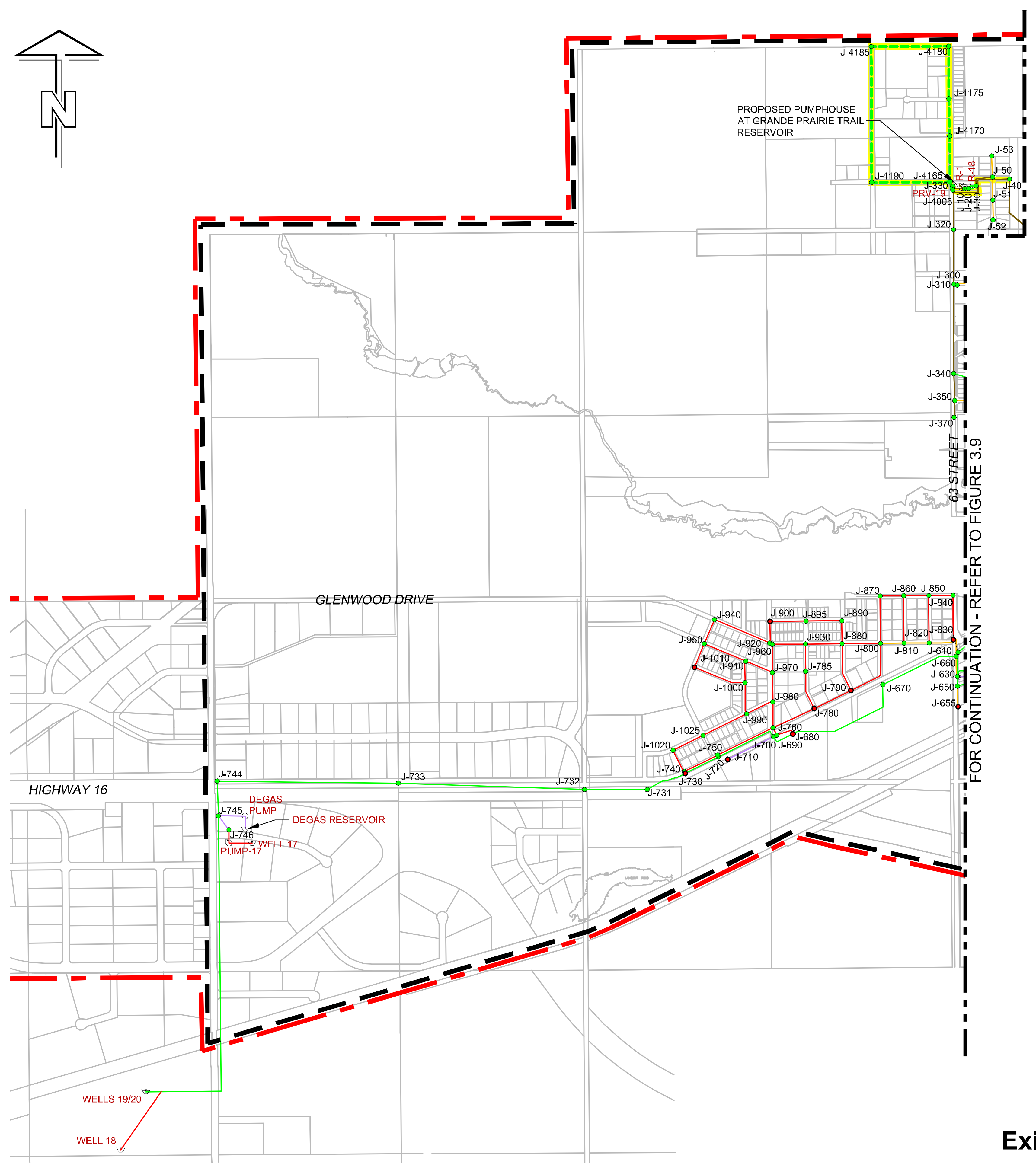
Town of Edson
Municipal Servicing Plan Update

Peak Hour Demand Contours Existing With Improvements

Figure 3.7



LEGEND

- STUDY AREA BOUNDARY
- TOWN OF EDSON BOUNDARY
- J-1670 FAILED FIRE FLOW REQUIREMENTS
- J-731 SATISFIED FIRE FLOW REQUIREMENTS
- WELL 9A SCHEMATIC RESERVOIR/WELL
- PUMP-9A SCHEMATIC PUMP
- PRV-20 SCHEMATIC PRESSURE REDUCING VALVE
- EXISTING 100mm WATERMAIN
- EXISTING 150mm WATERMAIN
- EXISTING 200mm WATERMAIN
- EXISTING 250mm WATERMAIN
- EXISTING 300mm WATERMAIN
- EXISTING 350mm WATERMAIN
- PROPOSED 300mm WATERMAIN
- PROPOSED 350mm WATERMAIN

m 0 200 400 SCALE 1:20000

Town of Edson
Municipal Servicing Plan Update

Maximum Day Demand and Fire Flow Existing With Improvements - Figure 1 of 2

Figure 3.8

AECOM

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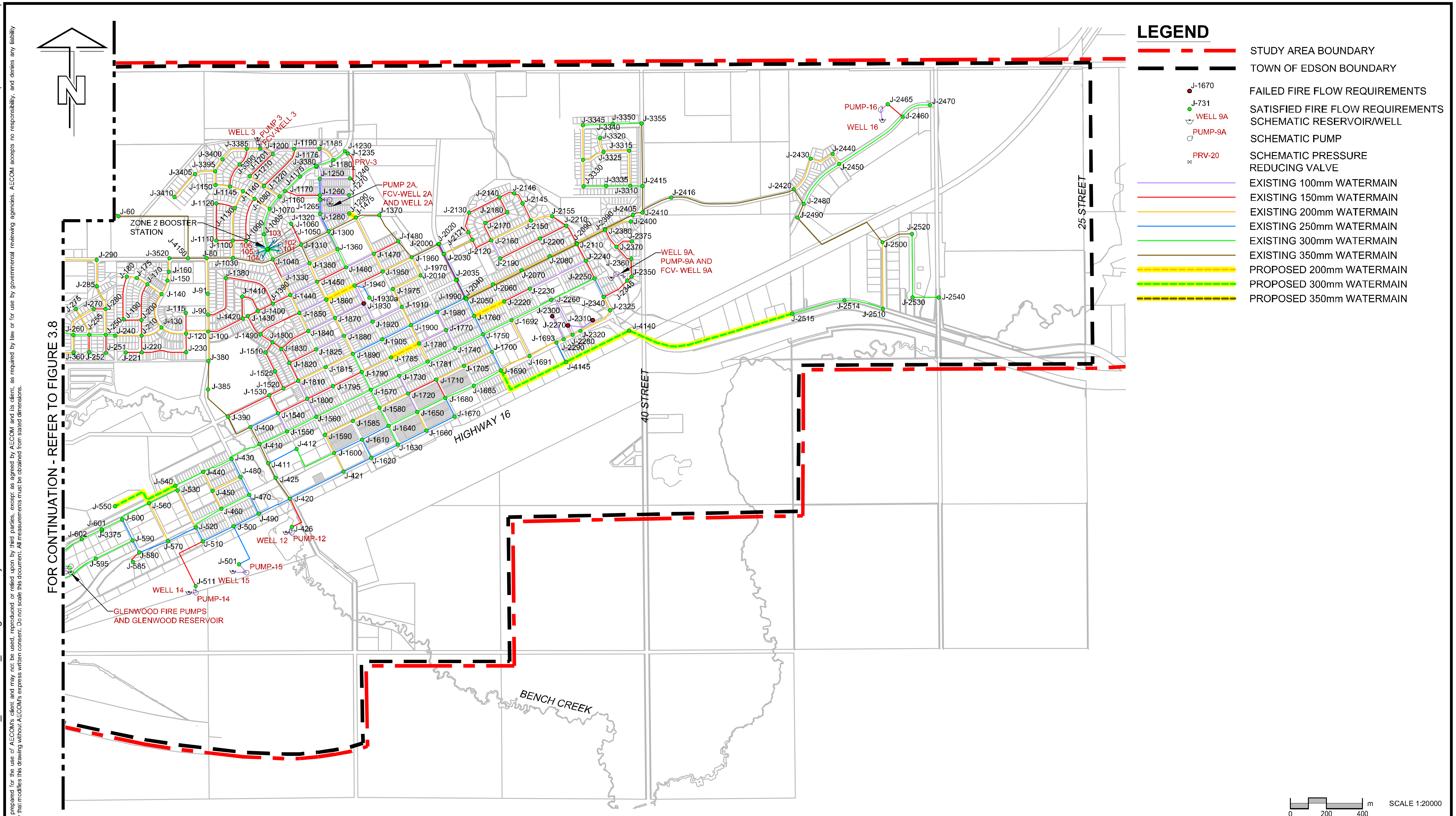

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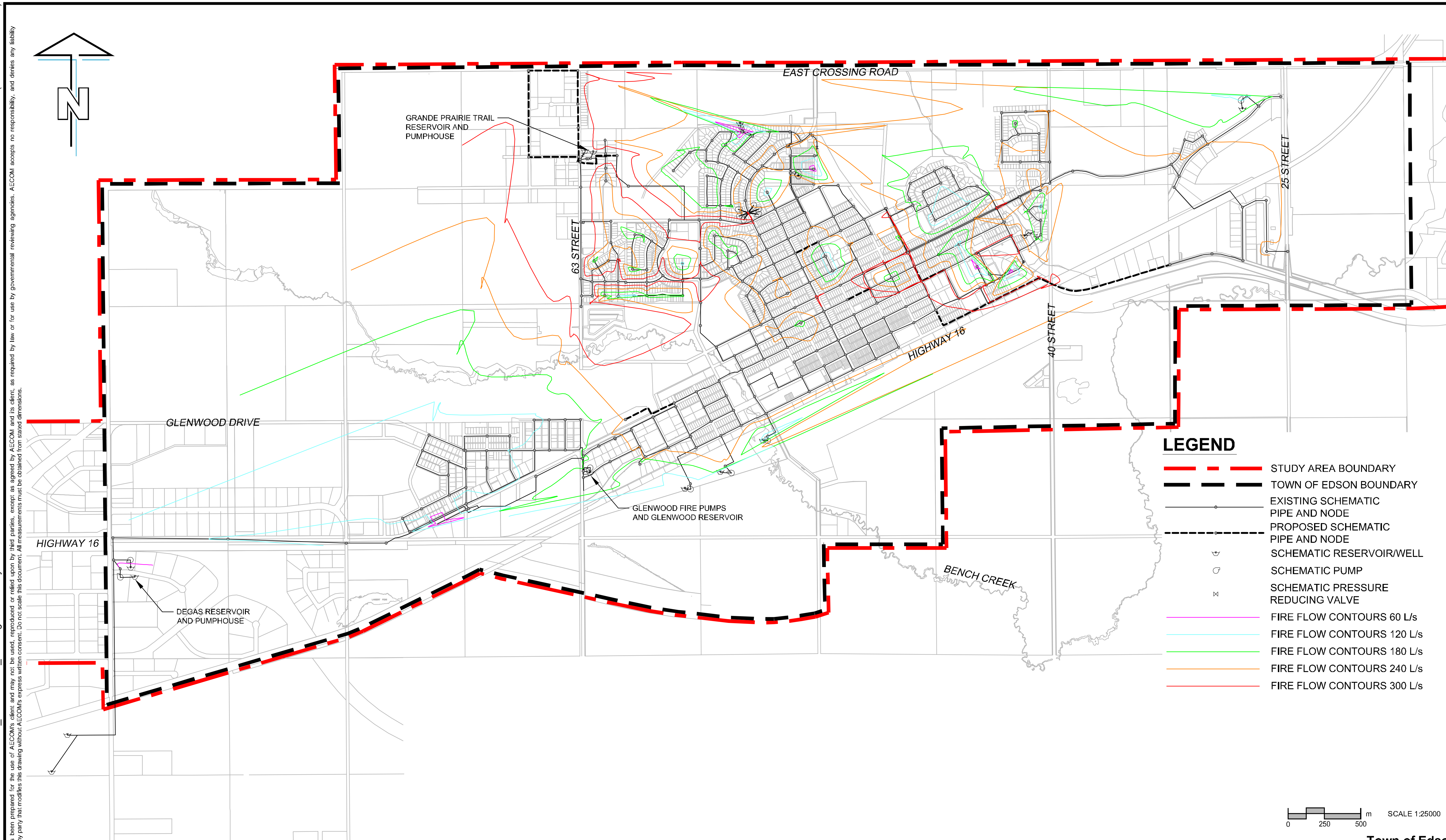
Town of Edson
Municipal Servicing Plan Update

Maximum Day Demand and Fire Flow
Existing With Improvements - Figure 2 of 2

Figure 3.9



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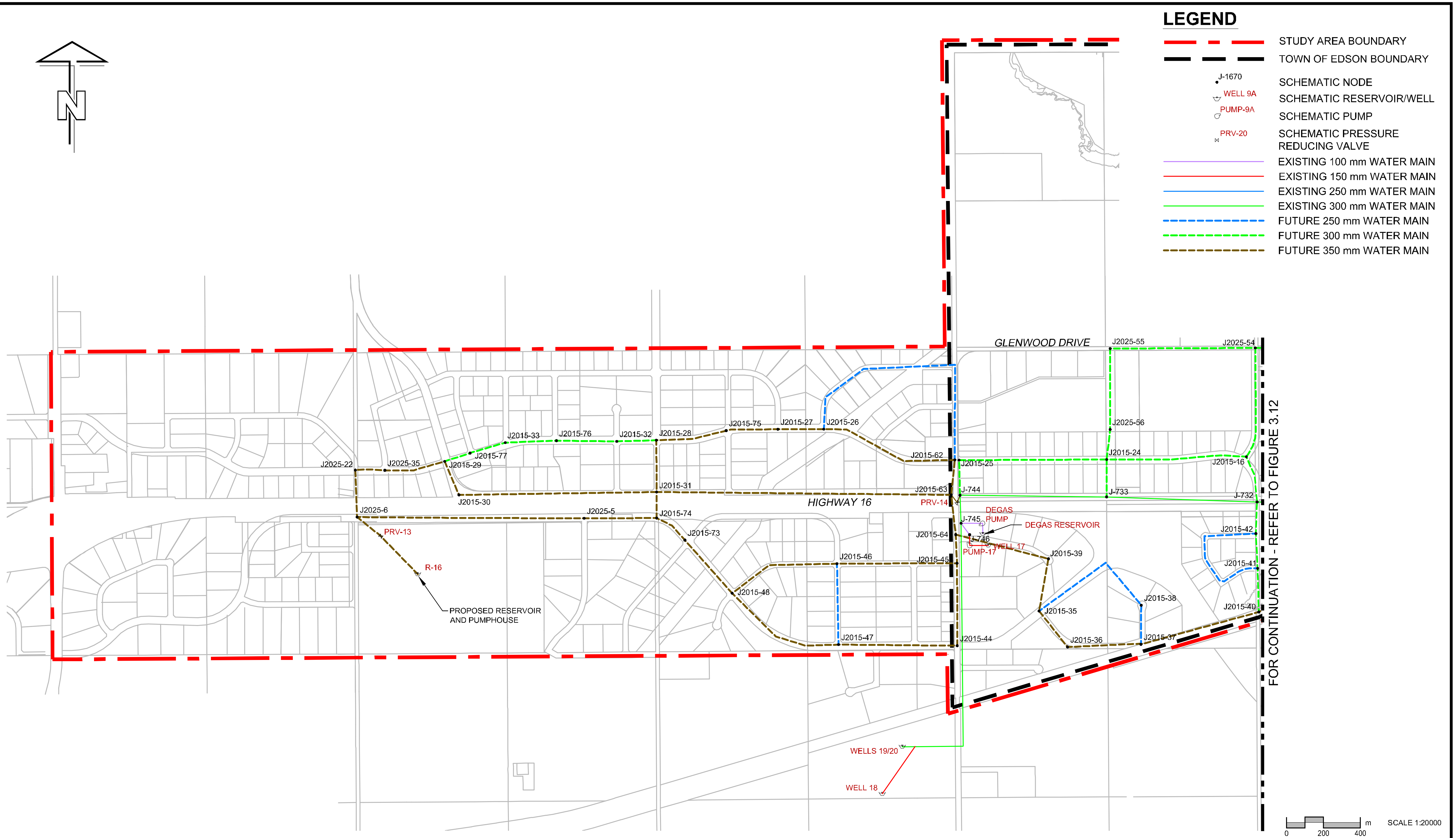
AECOM

Town of Edson
Municipal Servicing Plan Update

Maximum Day Demand and Fire Flow Contours Existing With Improvements

Figure 3.10



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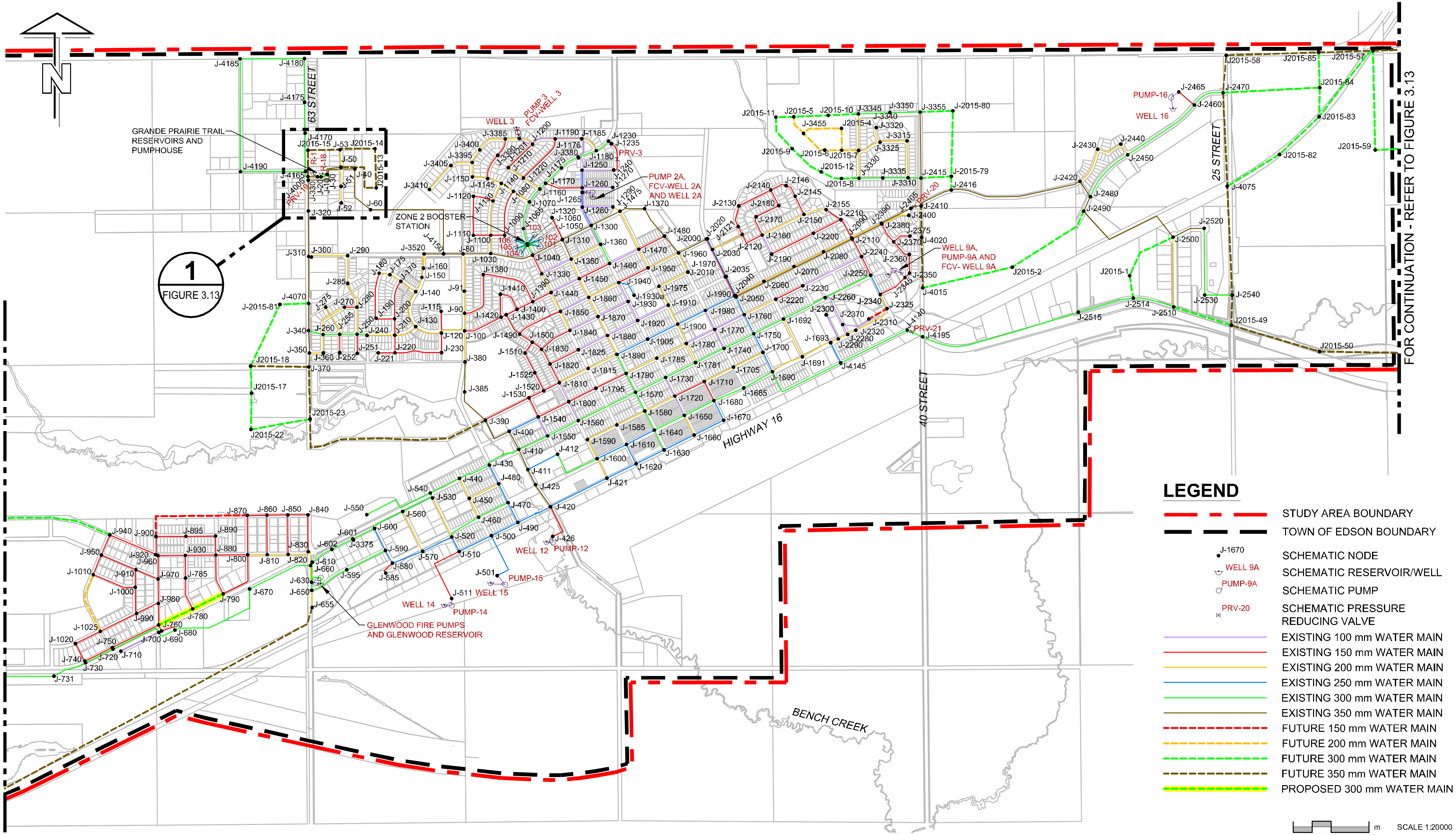
AECOM

Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Alternative 1 - Figure 1 of 3
Figure 3.11



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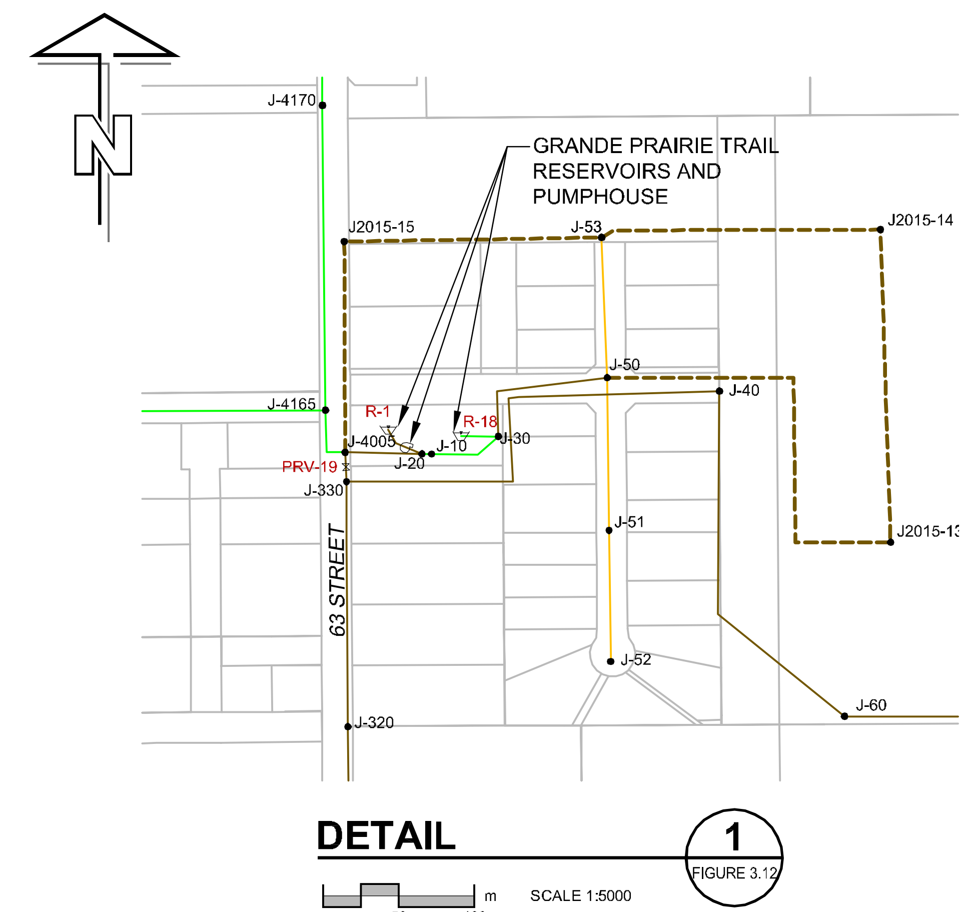
AECOM

Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Alternative 1 - Figure 2 of 3
Figure 3.12

LEGEND

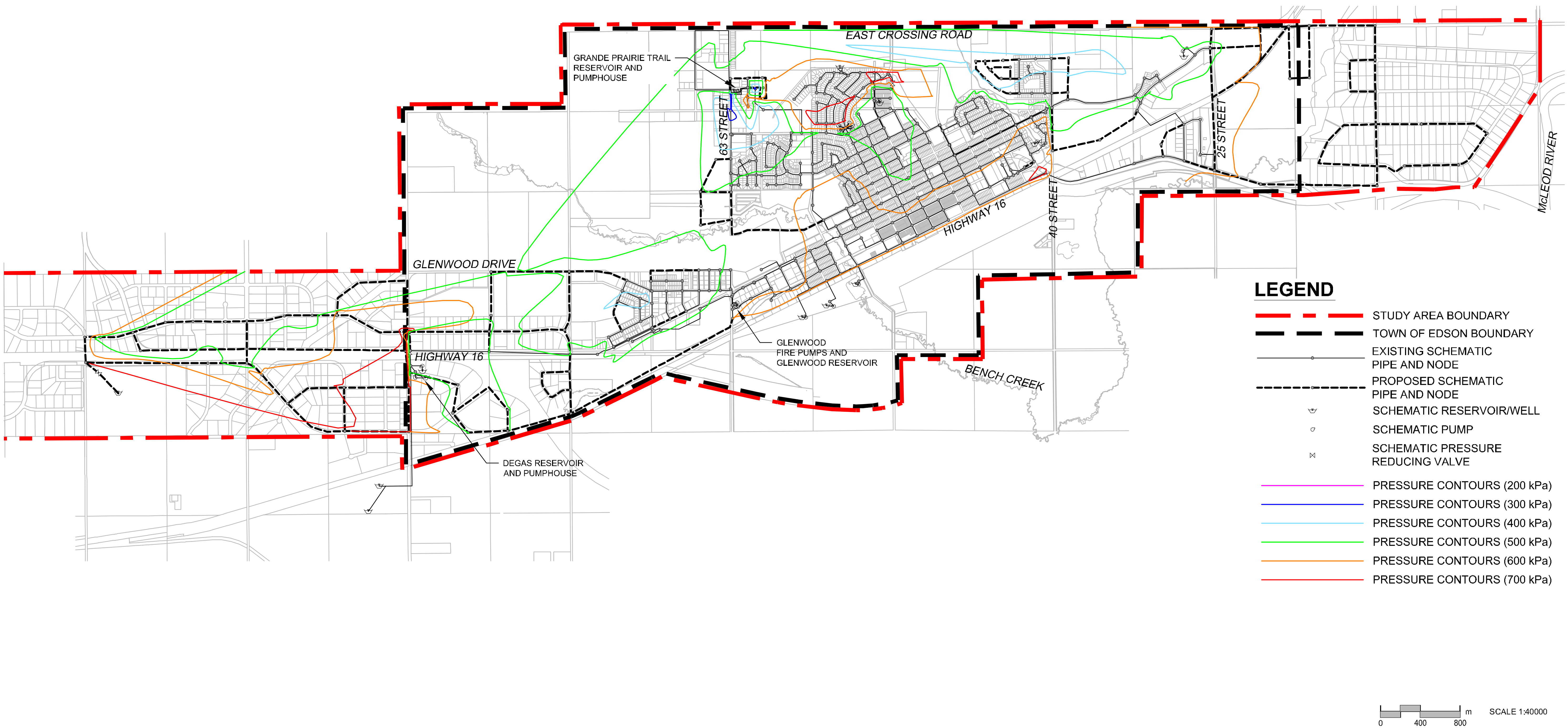
- STUDY AREA BOUNDARY
- TOWN OF EDSON BOUNDARY
- J-1670 — SCHEMATIC NODE
- WELL 9A — SCHEMATIC RESERVOIR/WELL
- PUMP-9A — SCHEMATIC PUMP
- PRV-20 — SCHEMATIC PRESSURE REDUCING VALVE
- EXISTING 200 mm WATER MAIN
- EXISTING 300 mm WATER MAIN
- EXISTING 350 mm WATER MAIN
- FUTURE 300 mm WATER MAIN
- FUTURE 350 mm WATER MAIN



Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Alternative 1 - Figure 3 of 3
Figure 3.13

AECOM



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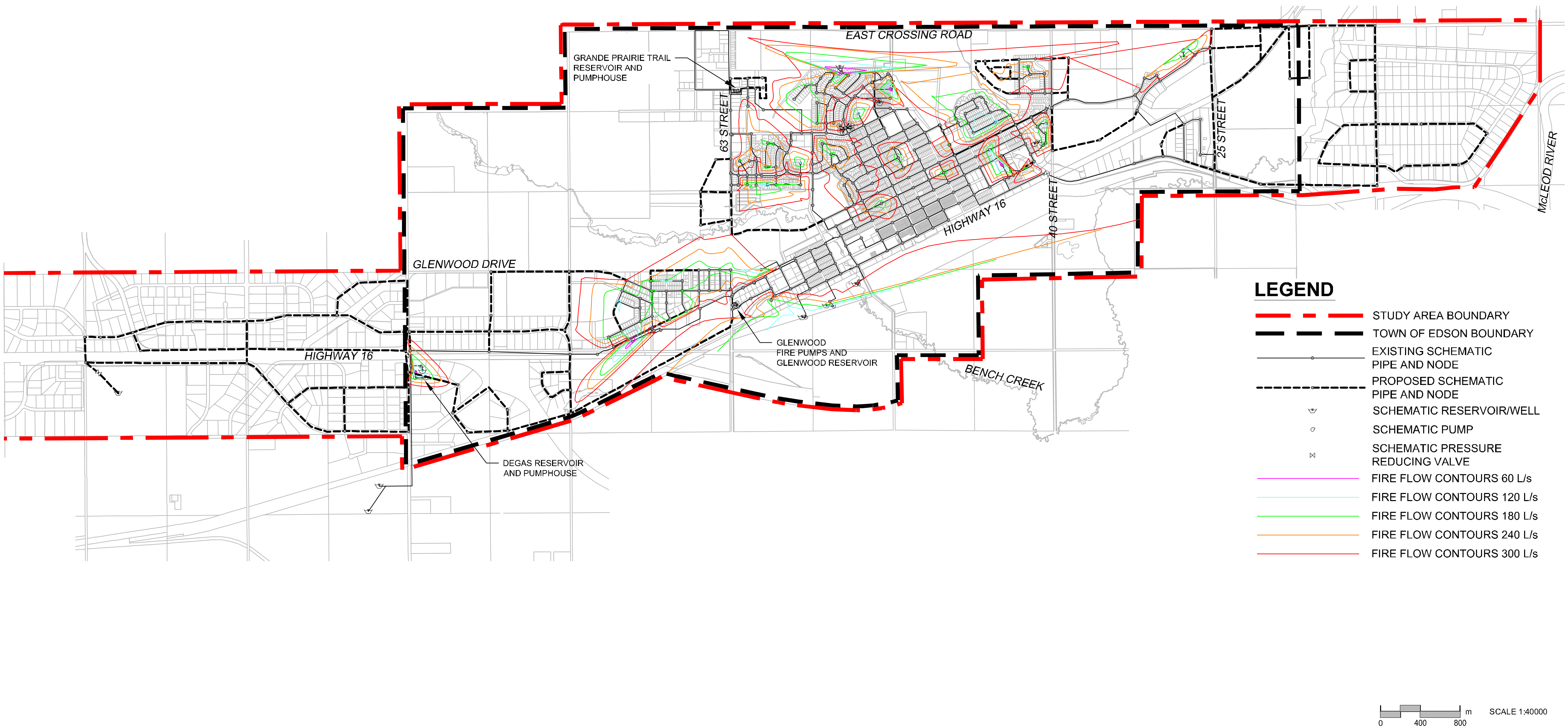
Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Peak Hour Demand - Alternative 1

Figure 3.14



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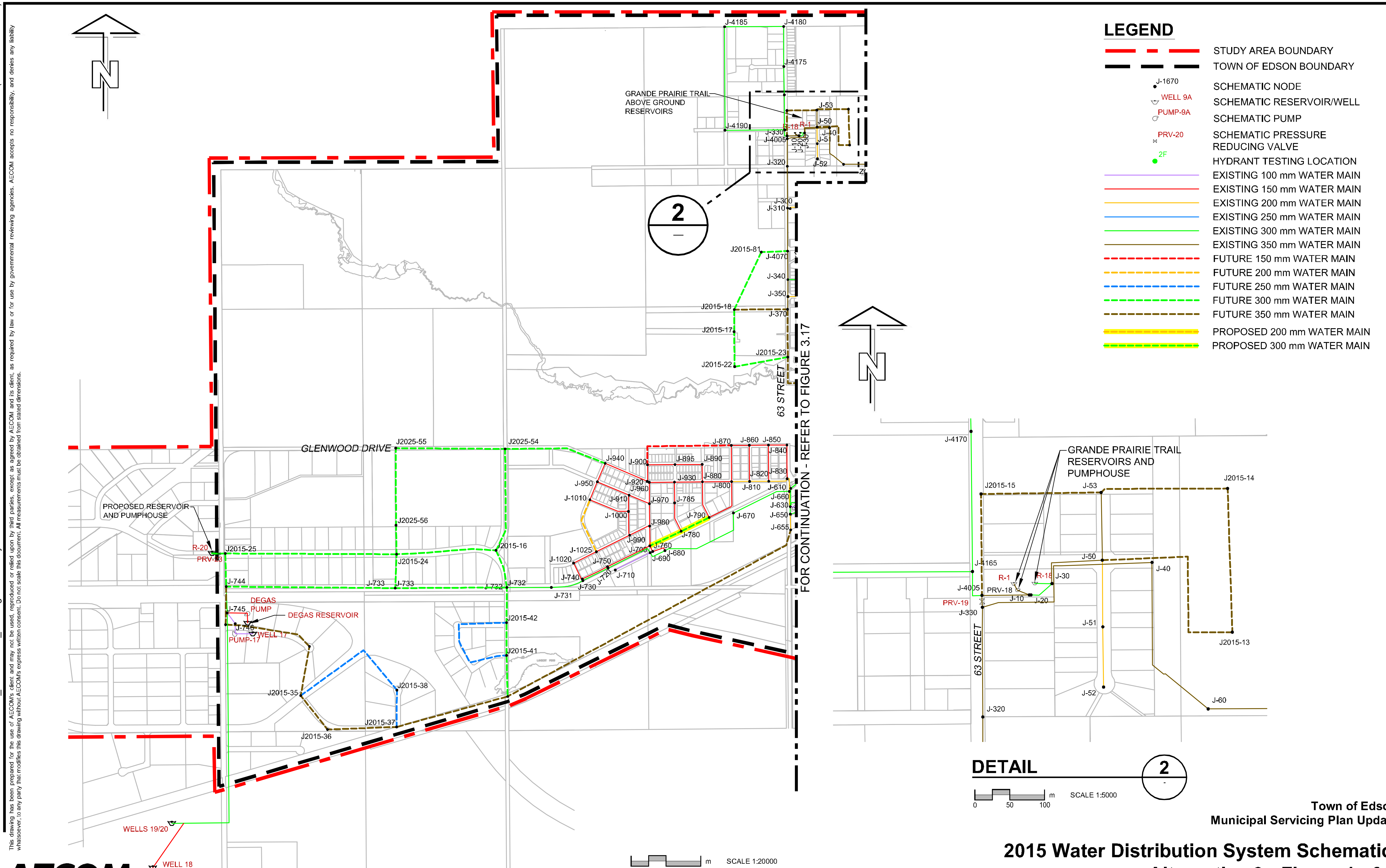
AECOM

Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Maximum Day Demand and Fire Flow - Alternative 1

Figure 3.15



Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Alternative 3 - Figure 1 of 2
Figure 3.16

AECOM

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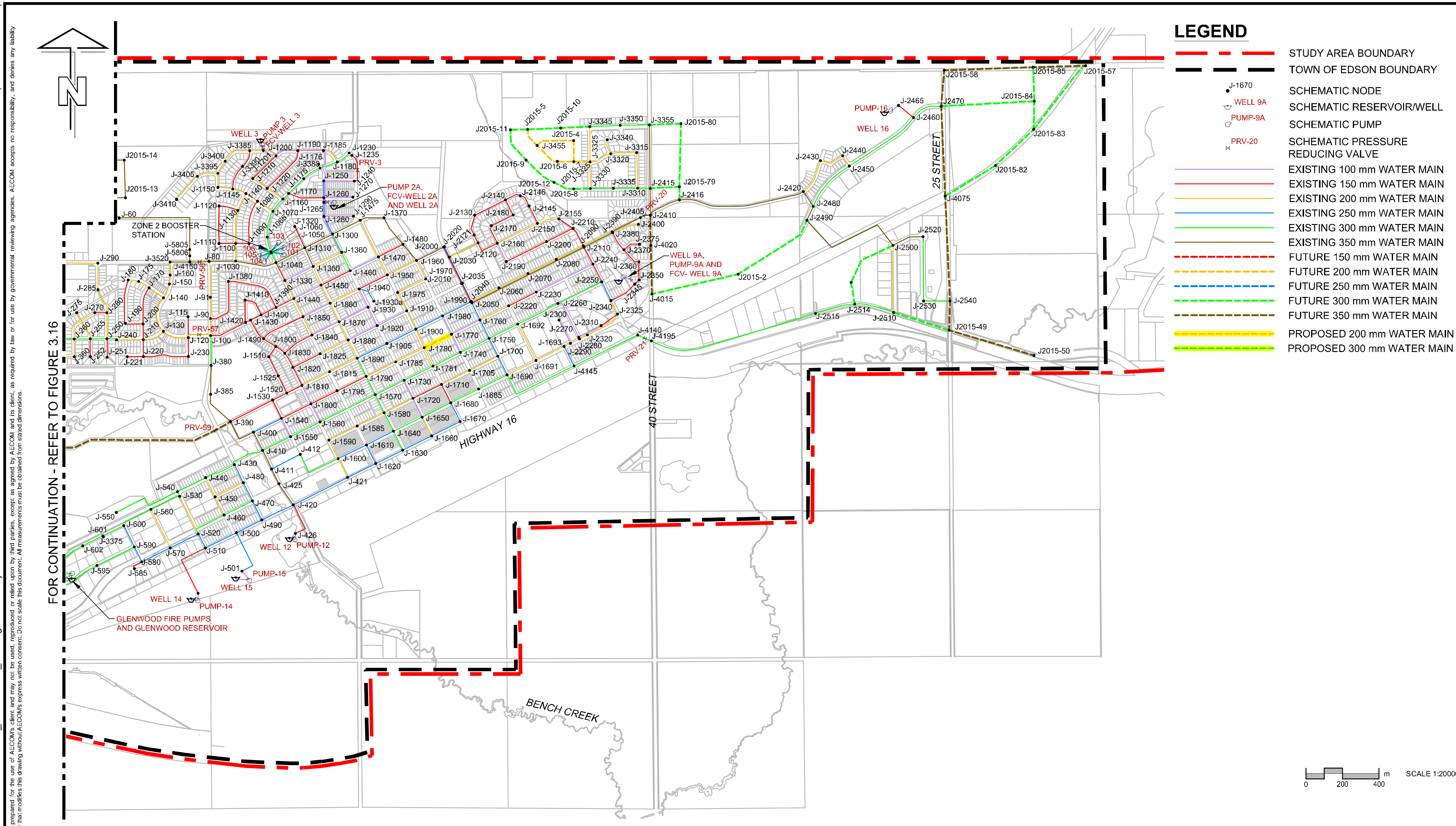
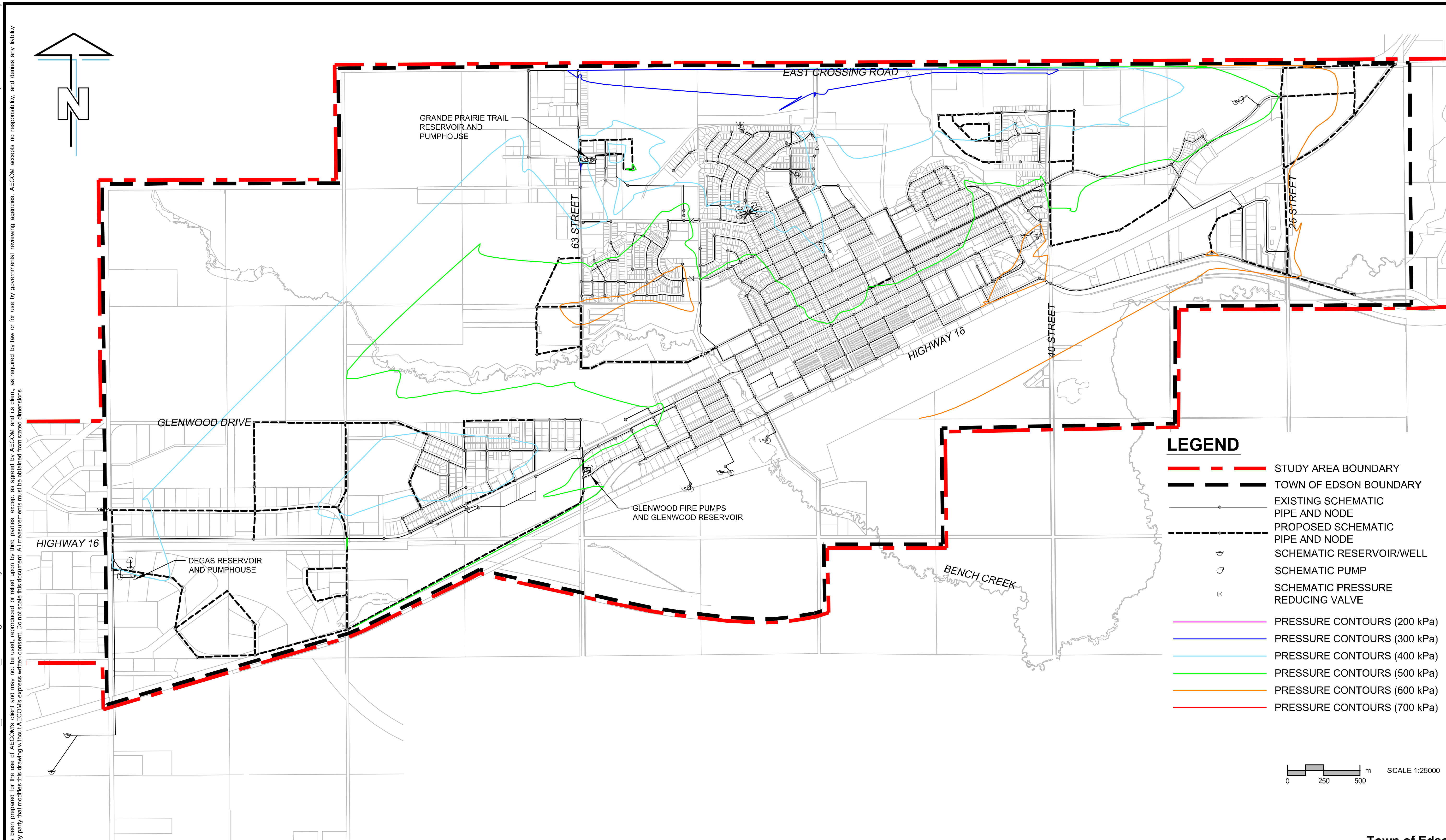


Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Alternative 3 - Figure 2 of 2
Figure 3.17



Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Peak Hour Demand - Alternative 3

Figure 3.18

AECOM

B SIZE 11" x 17" (279.4mm x 431.8mm)
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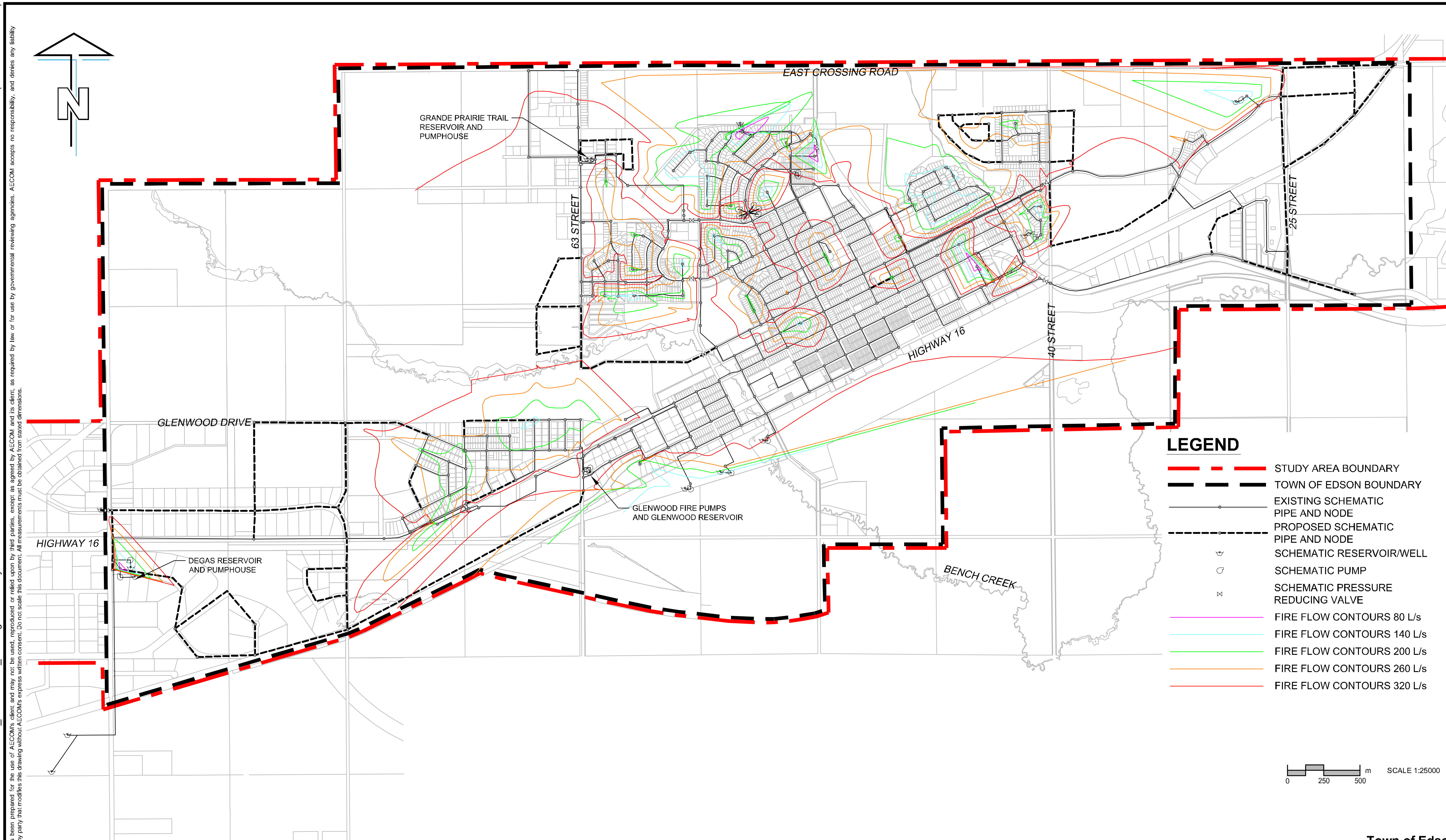

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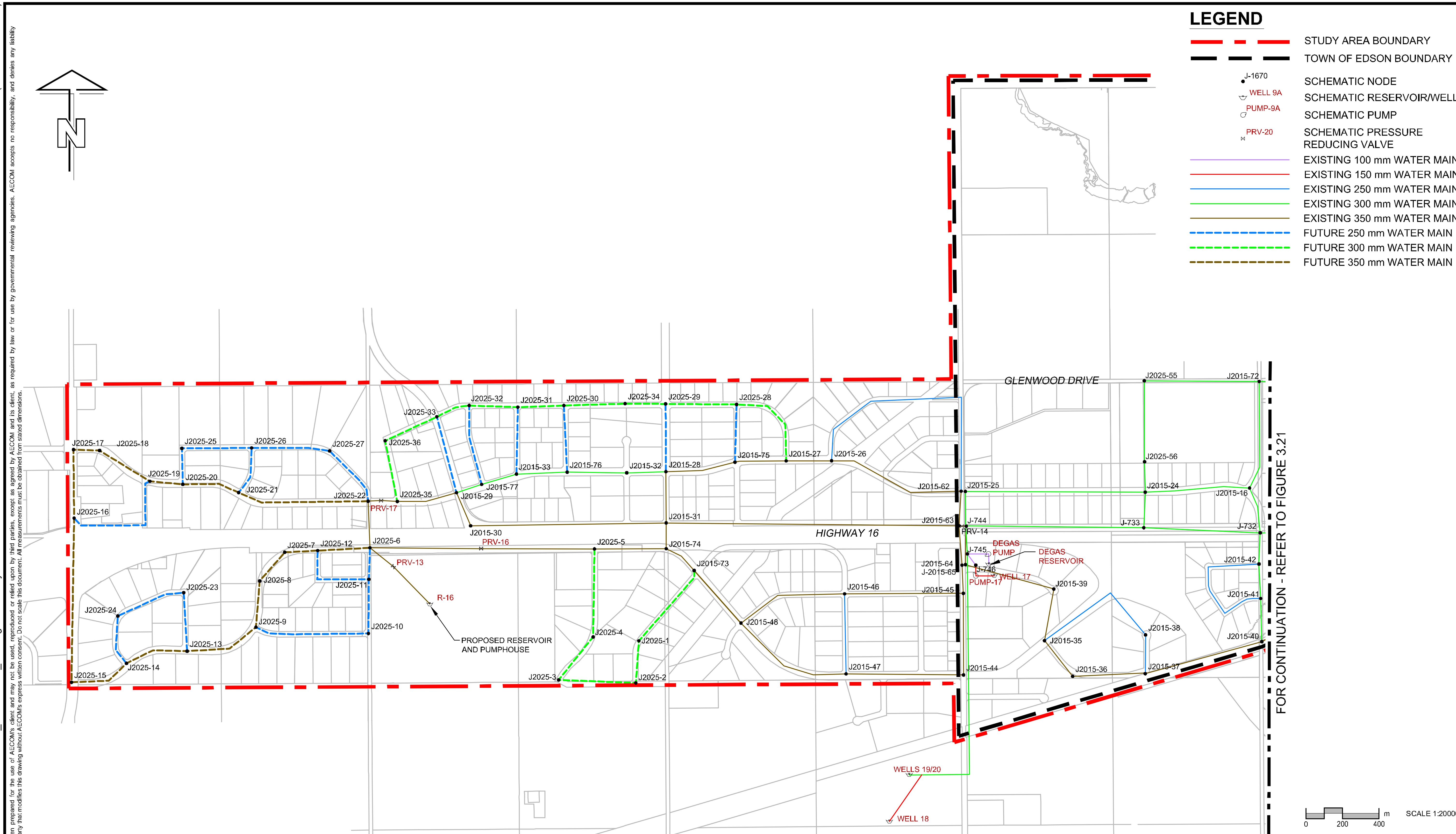
AECOM

Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Maximum Day Demand and Fire Flow - Alternative 3

Figure 3.19



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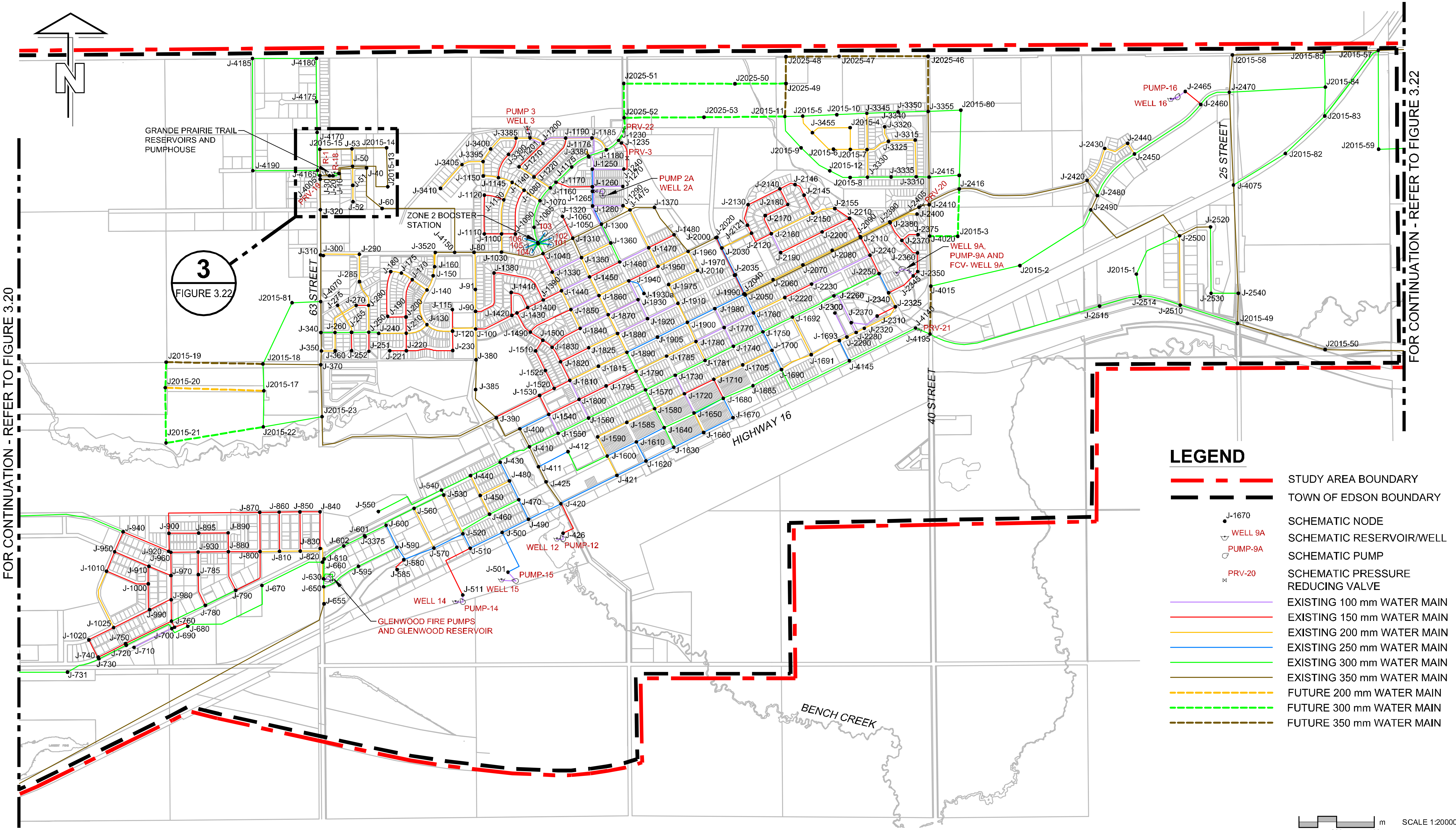
AECOM

Town of Edson
Municipal Servicing Plan Update

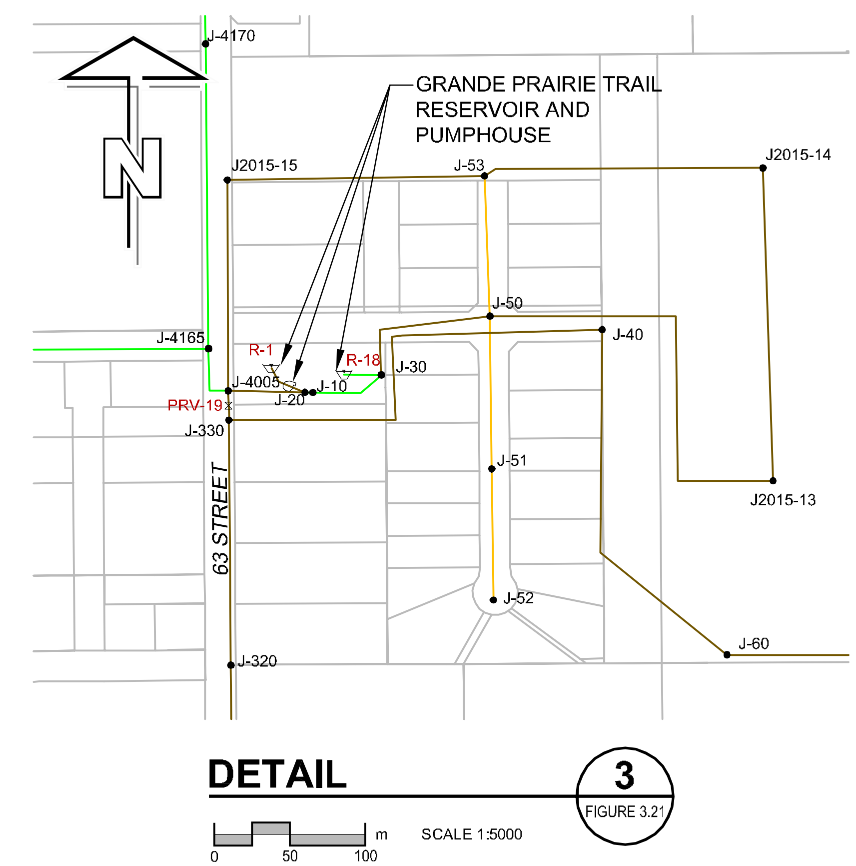
2025 Water Distribution System Schematic
Alternative 1 - Figure 1 of 3
Figure 3.20



Town of Edson
Municipal Servicing Plan Update

2025 Water Distribution System Schematic
Alternative 1 - Figure 2 of 3
Figure 3.21

AECOM



LEGEND

- STUDY AREA BOUNDARY
- TOWN OF EDSON BOUNDARY
- J-1670 SCHEMATIC NODE
- WELL 9A SCHEMATIC RESERVOIR/WELL
- PUMP-9A SCHEMATIC PUMP
- PRV-20 SCHEMATIC PRESSURE REDUCING VALVE
- EXISTING 200mm WATERMAIN
- EXISTING 300mm WATERMAIN
- EXISTING 350mm WATERMAIN
- FUTURE 350mm WATERMAIN

Town of Edson
Municipal Servicing Plan Update

2025 Water Distribution System Schematic
Alternative 1 - Figure 3 of 3
Figure 3.22

AECOM

B SIZE 11" x 17" (279.4mm x 431.8mm)

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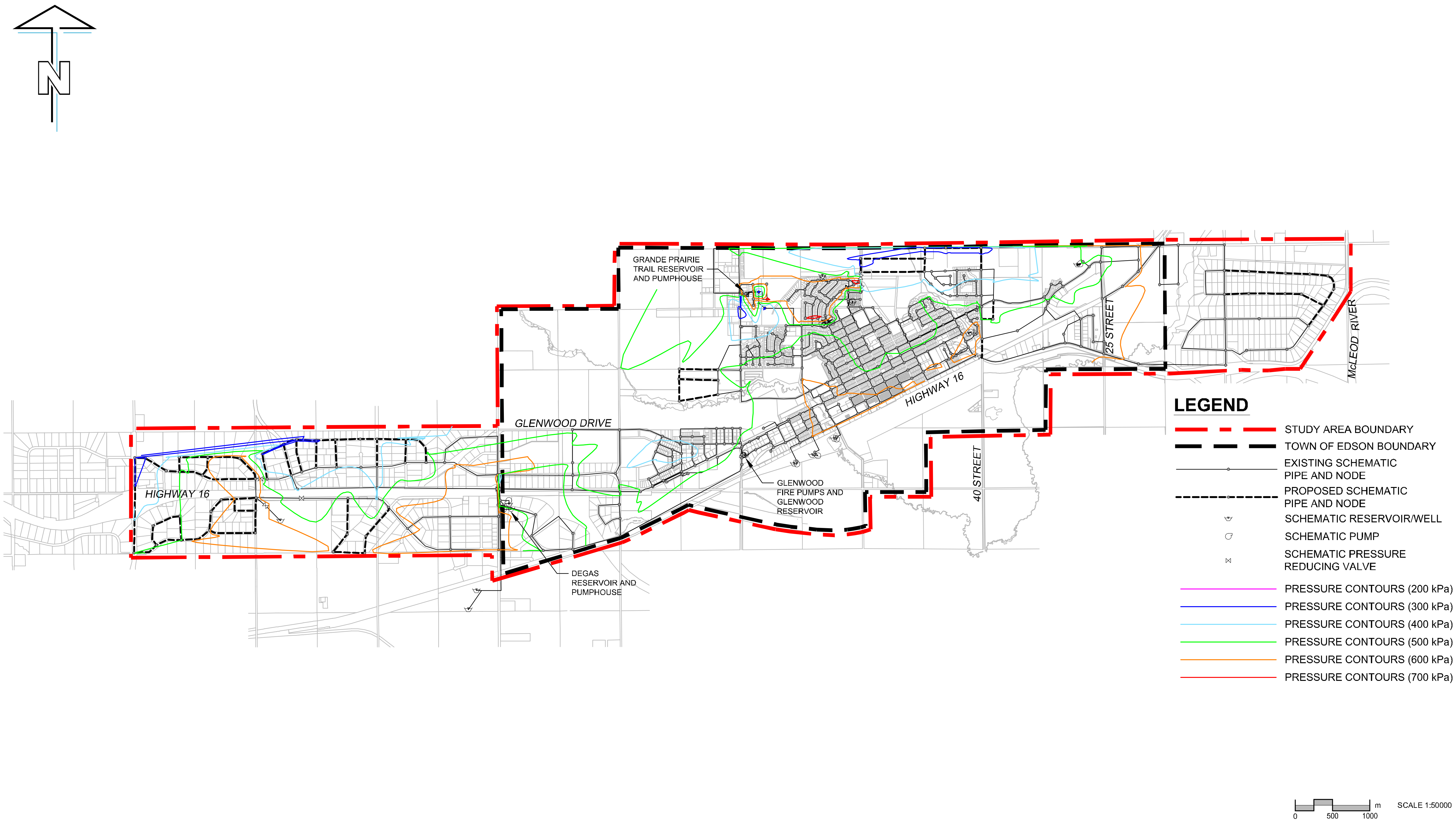

Town of Edson
Municipal Servicing Plan Update

2025 Water Distribution System Schematic
Peak Hour Demand - Alternative 1

Figure 3.23

AECOM

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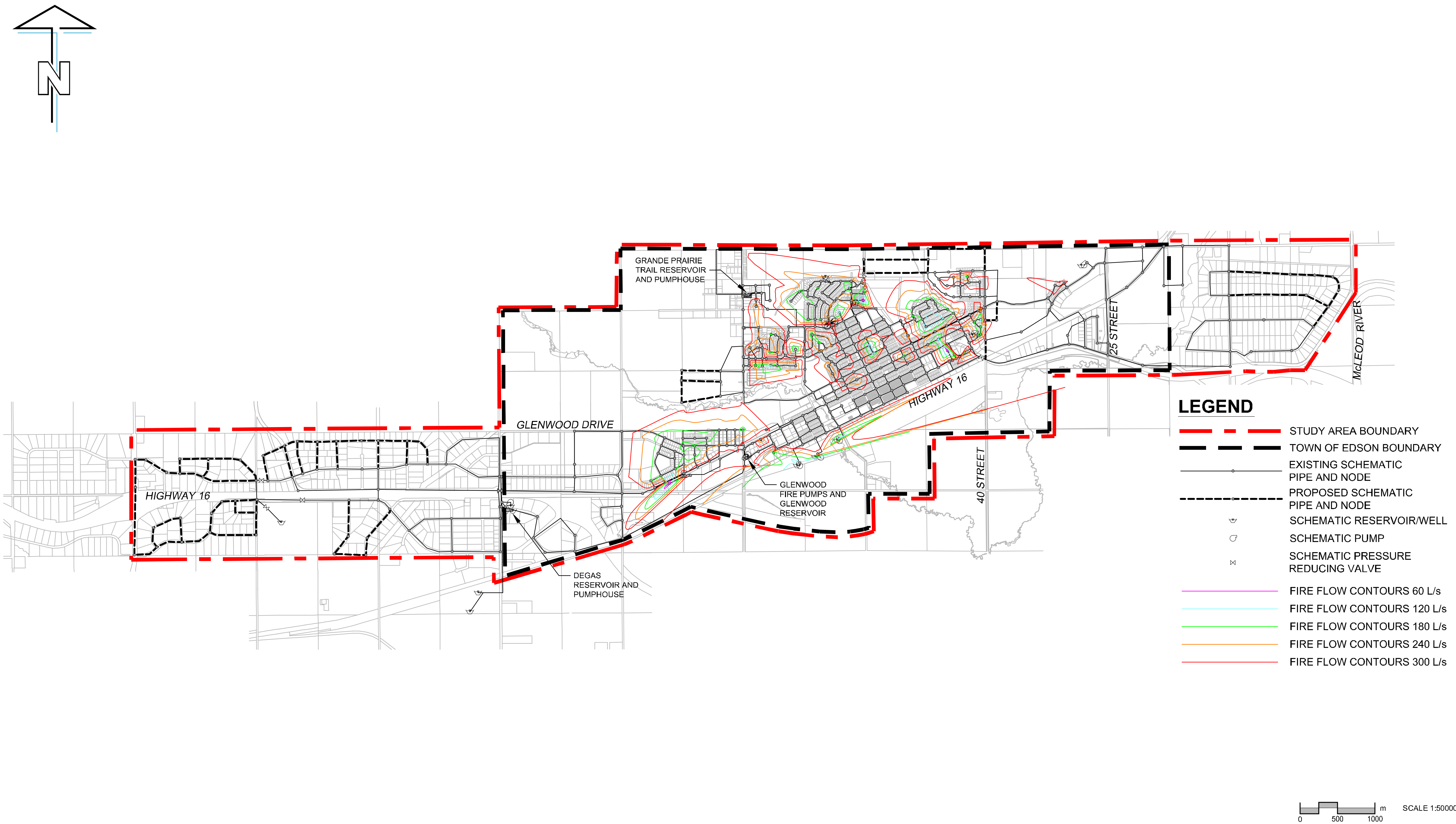

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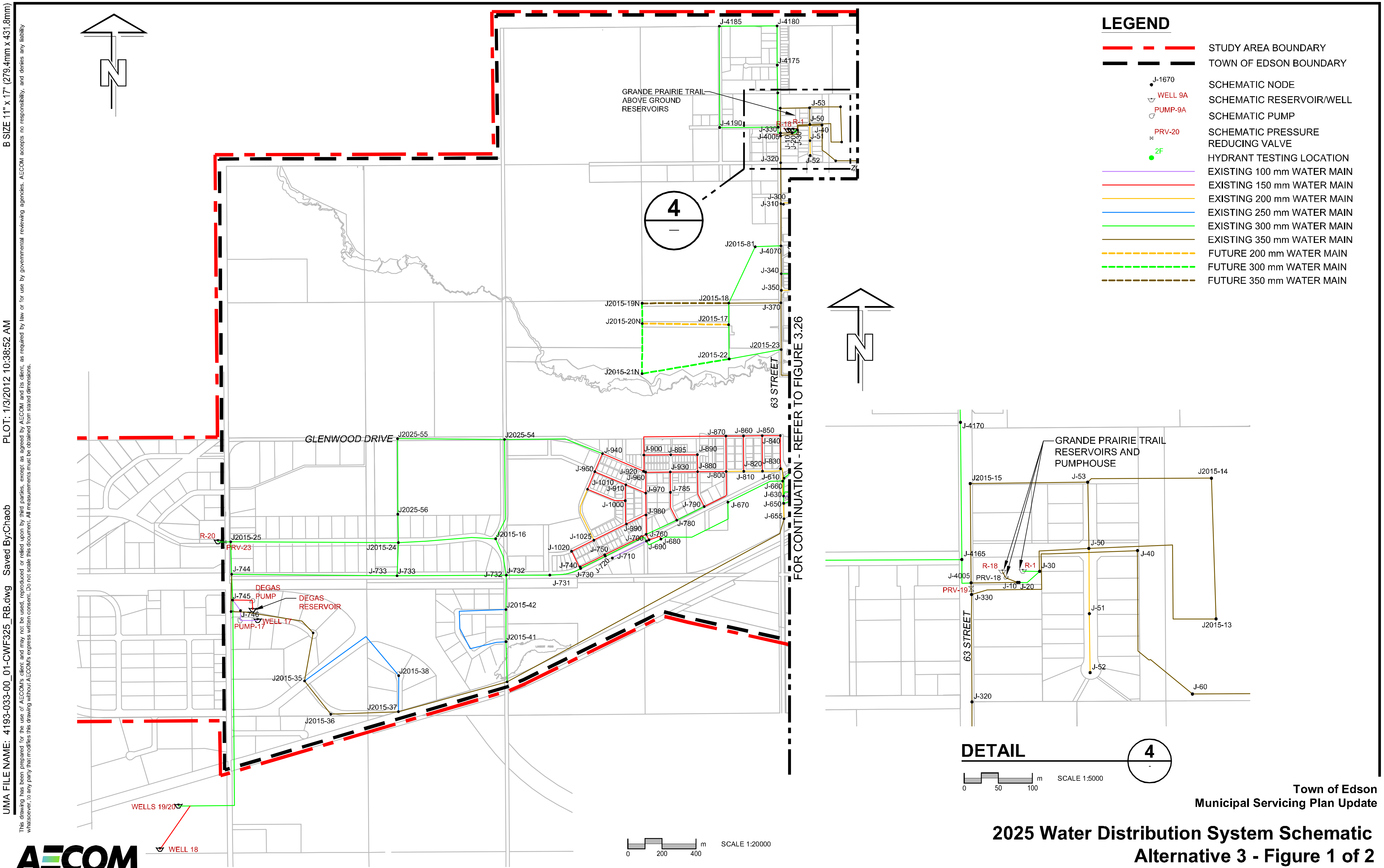
AECOM

Town of Edson
Municipal Servicing Plan Update

2025 Water Distribution System Schematic
Maximum Day Demand and Fire Flow - Alternative 1

Figure 3.24



Town of Edson
Municipal Servicing Plan Update

2025 Water Distribution System Schematic
Alternative 3 - Figure 1 of 2
Figure 3.25

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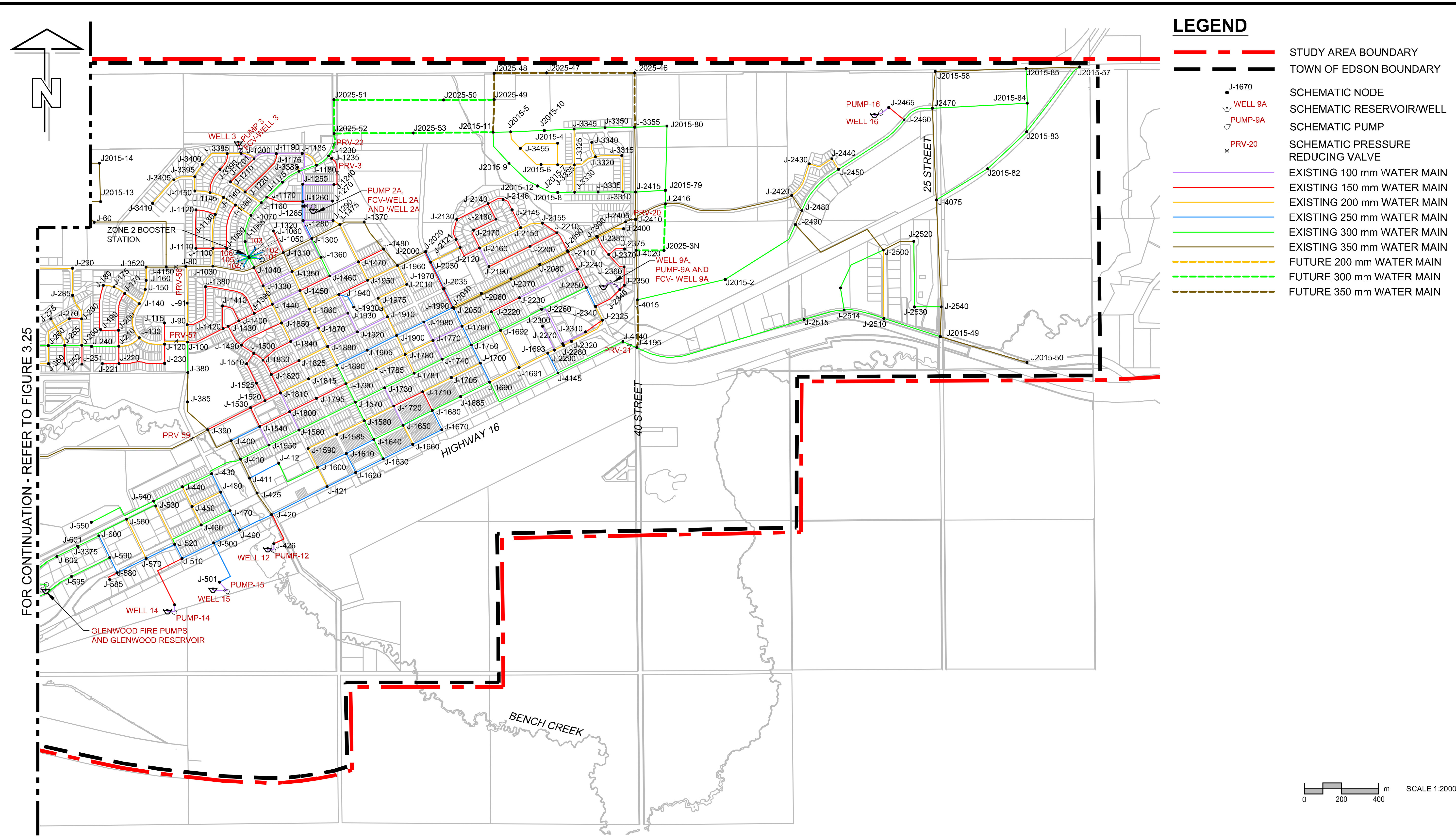

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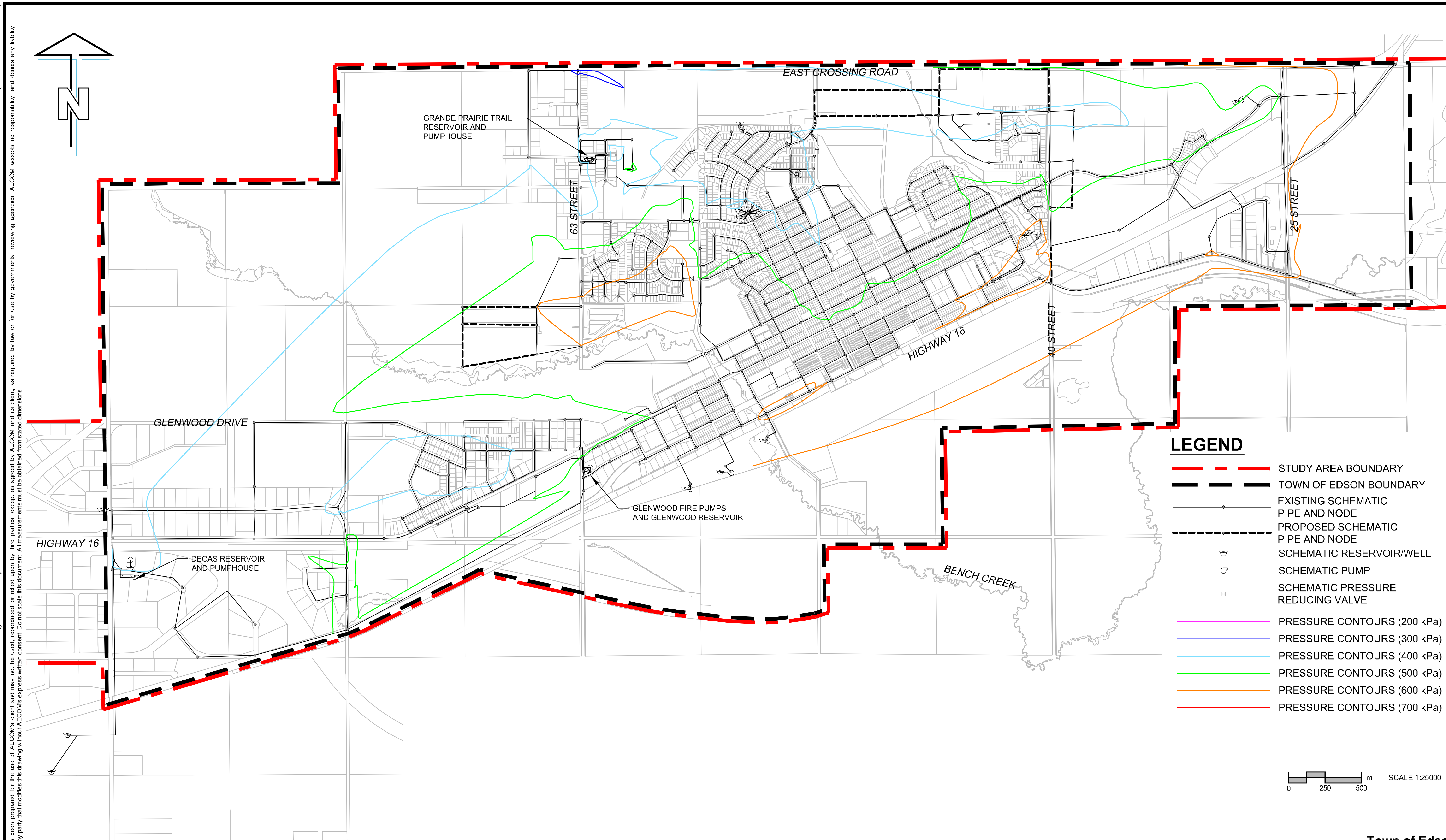
AECOM

Town of Edson
Municipal Servicing Plan Update

2025 Water Distribution System Schematic
Alternative 3 - Figure 2 of 2
Figure 3.26



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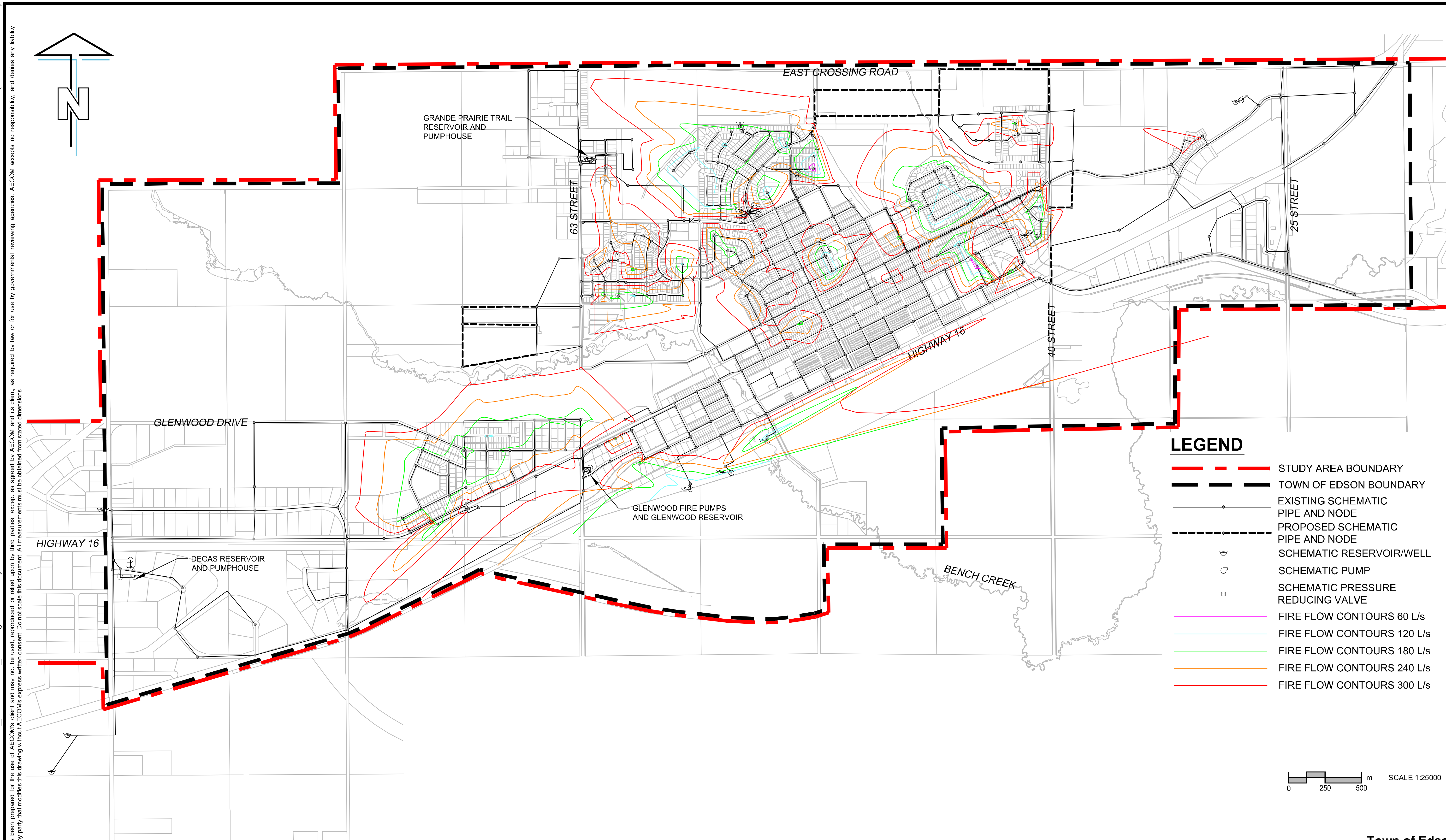
Town of Edson
Municipal Servicing Plan Update

2025 Water Distribution System Schematic
Peak Hour Demand - Alternative 3

Figure 3.27



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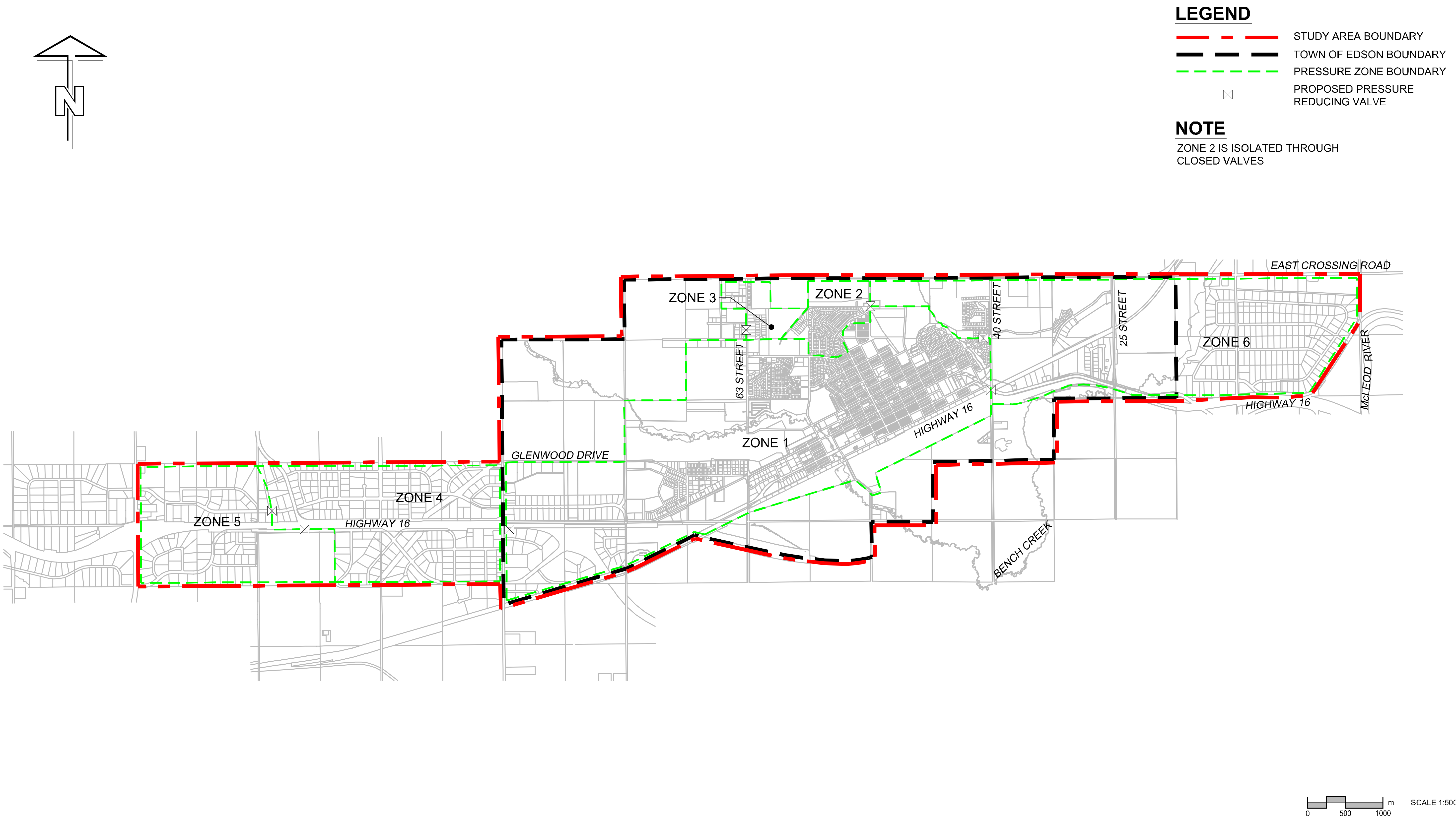
AECOM

Town of Edson
Municipal Servicing Plan Update

2025 Water Distribution System Schematic
Maximum Day Demand and Fire Flow - Alternative 3

Figure 3.28



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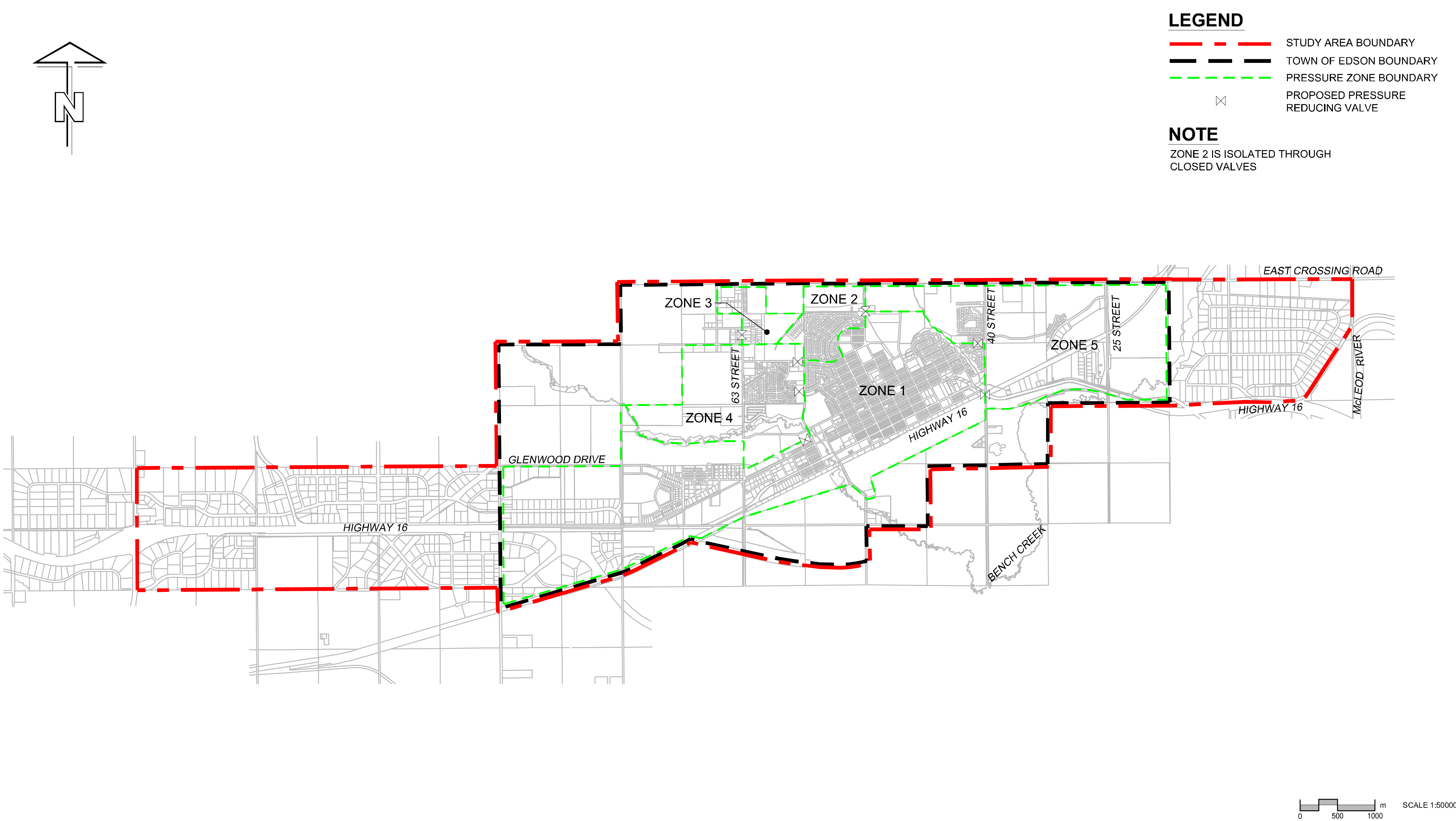
Town of Edson
Municipal Servicing Plan Update

2025 Pressure Zone Boundaries
Alternative 1
Figure 3.29



Town of Edson
Municipal Servicing Plan Update

2025 Pressure Zone Boundaries
Alternative 3
Figure 3.30

ISS/REV:
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4. Wastewater Collection System

4.1 General

The purpose of this section is to assess the existing sanitary system performance, identify any deficiencies and associated improvements to the existing system as well as servicing requirements for future development.

4.2 Study Data

The land use and population projections used for the sanitary sewer system assessment are summarized in Section 2.0.

4.3 Existing System Description

The existing sanitary sewer system and service area is shown in Figure 4.1. The sanitary system consists of approximately 66 km of gravity sewer mains. There are no lift stations present within the Town's system. All the sanitary flow from the Town drains to the existing sewage lagoon located west of 25th Street and south of the Canadian National Railway right of way. The majority of the pipes are 200 mm in diameter, but gradually increase in size closer to the lagoons, becoming as large as 1050 mm. The lagoons are used for treatment rather than storage and currently discharge treated water into the McLeod River, approximately 2.5 km away. The physical data for the existing sanitary sewer system is provided in Appendix E.

Based on discussions with the Town, all houses constructed prior to 2005 are likely to have weeping tile connected to the sanitary system. The Town has experienced basement flooding and/or sewer backups in the past in areas suspected to have weeping tile connections. Newer areas that do not have weeping tile connections include the East End Subdivision, Skyview and Willishire House. The East End subdivision spans from 41 Street to 42 Street and from 15 Avenue to 18 Avenue. Both Skyview and Willishire House are located north of 13 Avenue between 62 Street and 56 Street. As expected, none of these areas experience flooding in the model.

4.4 Model Development

XP-SWMM version 9.14, an industry accepted modelling software program, was used to develop the detailed model of the existing sanitary sewer system. The model features the XP-SWMM Runoff Layer, which generates wet weather flows. It also features the XP-SWMM Hydraulics Layer, which simultaneously simulates the dry and wet weather. These two layers allow for the collection of simulated data for both dry and wet weather flows.

Physical data including manhole rim elevation, invert elevation, pipe diameter and slope was obtained from as built drawings and supplementary survey data provided by the Town. A single flow monitor is located at the inflow to the lagoons. The Town of Edson does not currently have a rain gauge that corresponds with the existing flow monitor. Lagoon influent flow data provided by the Town and rainfall data obtained from the Environment Canada website were used to approximate the dry and wet weather flows. This gives a fairly accurate estimate for dry weather flow; however, wet weather flows can vary a great deal depending on the amount of rainfall and the drainage conditions.

The Town of Edson was delineated into sanitary sewer catchment areas that were used in both dry and wet weather flow simulations. These were estimated based on the locations where sewage would enter the system and allows for input of flow into all pipes as well as the examination of localized problem areas.

Figure 4.2 shows the sanitary sewer catchment areas. Residential catchment areas are shown in red and commercial/industrial catchment areas are shown in blue.

4.4.1 Dry Weather Flow Model

Dry weather flow was generated by the model based on the parameters calculated through analysis of the dry weather flow data provided. For existing residential areas the per capita sewage generation of 375 L/c/d rate was applied. Consistent with the water system analysis, for existing non-residential areas a rate of 13,600 L/ha/d was used for high demand areas and a rate of 1500 L/ha/d was used for non-high demand non-residential areas. A density of between 3 and 3.5 people per lot was used in residential areas, depending on whether there was single family or multi-family development. This equates to an average population density of 31.2 people/ha for the Town. The flow data was used to calculate the average per capita daily sewage generation rate of 375 L/c/d. A diurnal flow pattern for dry weather flow was also developed using this flow monitoring data for both residential and non-residential areas as shown in Figure 4.3. Calibration of the dry weather flow model is discussed in Section 4.5.4.

4.4.2 Wet Weather Flow Model

The XP-SWMM Runoff Layer was used to generate the wet weather flow in the model. The wet weather flow into the sanitary system varies significantly with the depth and distribution of rainfall and the type of servicing. In order to simulate the inflow and infiltration process, an effective drainage area was identified for each basin. Only a portion of runoff will enter the sanitary sewer which means only a portion of the basin area is contributing runoff to the sanitary sewer. Therefore, an effective area is used to generate the runoff that will enter the sanitary sewer. The primary calibration parameter for wet weather flow is the effective area. The effective area is adjusted until the volume of runoff and peak flow generated represents the inflow/infiltration shown in the flow monitoring data.

The infiltration parameters used in the model are summarized in Table 4.1.

Table 4.1: Infiltration Parameters

Parameter	Value
Ground Slope	2.0%
Impervious Area – Manning's n	0.015
Pervious Area – Manning's n	0.25
Impervious Depression Storage	3.20 mm
Pervious Depression Storage	6.40 mm
Initial Infiltration Rate	100 mm/hr
Final Infiltration Rate	5 mm/hr
Decay Rate	0.00115 L/s

A residential area percent impervious of 50% and a non-residential percent impervious of 70% were used.

4.5 Model Calibration

This section outlines the calibration of the Town of Edson sanitary sewer model. 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.

4.5.1 Dry Weather Flow Calibration

Flow monitoring data recorded in 2008 was used to verify the model results. Several dry weather days were reviewed and a dry weather flow hydrograph was selected.

The modeled dry weather flow was then compared to the monitored dry weather flow. Table 4.2 summarizes the monitored and modelled volumes and peaks for the Town of Edson dry weather flow. The modelled volume and peak flow compare quite favourably to the monitored volume and peak flow. Appendix F provides the hydrographs for the modeled and monitored dry weather flows. The modelled volume and peak flow are within 14% and 7% of the monitored values respectively.

Table 4.2: Dry Weather Flow Calibration Summary

Event	Volume (m^3)			Peak Flow (m^3/s)		
	Model	Monitor	Model/Monitor	Model	Monitor	Model/Monitor
Dry Weather Flow	3850	3363	1.14	0.058	0.054	1.07

4.5.2 Wet Weather Flow

The wet weather flow was simulated utilizing the runoff block of XP-SWMM. Flow monitoring data at the inflow to the lagoon was provided for 2008. The model was verified for the inflow to the wastewater lagoon for the three selected rainfall events. A summary of the three rainfall events used in the wet weather flow calibration are summarized in Table 4.3.

Table 4.3: Summary of 2008 Rainfall Events

Event	Cumulative Rainfall (mm)	Duration (hours)
June 6, 2008	28	18
August 21, 2008	24	14
June 11, 2008	10	3

June 6, 2008 Event

The June 6 event was the most significant rainfall event that occurred in Edson in 2008. This event was evaluated based on the rainfall data collected by Environment Canada, available at www.weatheroffice.gc.ca. As indicated in Table 4.3, a total of 28 mm of rain was recorded over a period of 18 hours.

Table 4.4 summarizes the monitored and modelled volumes and peaks for the June 6, 2008 rainfall event. The June 6 event was the largest rainfall that occurred in 2008 and was used to calibrate the wet weather flow. The modelled volume and peak flow are within 5% and 4% of the monitored volumes respectively. Comparison of the modelled versus monitored hydrographs are provided in Appendix F.

Table 4.4: June 6, 2008 Event Calibration Summary

Event	Volume (m^3)			Peak Flow (m^3/s)		
	Model	Monitor	Model/Monitor	Model	Monitor	Model/Monitor
Lagoon Inflow	6542	6292	1.046	0.114	0.119	0.960

August 21, 2008

The August 21, 2008 event was also evaluated using data available from Environment Canada to verify the calibration. As indicated in Table 4.3, a total of 24 mm of rain was recorded over a period of 14 hours. Table 4.5 summarizes the monitored and modelled volumes and peaks for the August 21, 2008 rainfall event. The modelled volume and peak flow are within 15% and 3% of the monitored values respectively. Comparison of the modelled versus monitored hydrographs are provided in Appendix F.

Table 4.5: August 21, 2008 Event Calibration Summary

Event	Volume (m^3)			Peak Flow (m^3/s)		
	Model	Monitor	Model/Monitor	Model	Monitor	Model/Monitor
Lagoon Inflow	6231	5429	1.149	0.113	0.117	0.975

June 11, 2008

The June 11, 2008 event was also evaluated based on the rainfall records from the Environment Canada website. A total of 10 mm of rain was recorded over a period of 3 hours. Table 4.6 summarizes the monitored and modelled volumes and peaks for the June 11, 2008 rainfall event. The modelled volume and peak flow are within 13% and 19% of the monitored values respectively. Comparison of the modelled versus monitored hydrographs are provided in Appendix F.

Table 4.6: June 11, 2008 Event Calibration Summary

Event	Volume (m^3)			Peak Flow (m^3/s)		
	Model	Monitor	Model/Monitor	Model	Monitor	Model/Monitor
Lagoon Inflow	4567	5358	0.869	0.112	0.094	1.191

In general, the modelled and monitored wet weather flows compare quite favourably. The average calculated I/I rate for all three events was approximately 0.11 L/s/ha.

4.5.3 Summary of Model Calibration Criteria

The design criteria based on the calibrated model are summarized in Table 4.7.

Table 4.7: Summary of Calibration Criteria

Parameter	Town of Edson Calibration Criteria
Residential Sewage Generation Rate	375 L/c/d
Non-Residential Sewage Generation Rate	Site-specific, otherwise 1500 L/ha/d
Effective Area (areas with weeping tile connected)	5.3%
Effective Area (areas without weeping tile connected)	0.05%

It is recommended that the Town of Edson continue to collect flow data and verify the model calibration on a yearly basis or when a large rainfall event occurs. A rain gauge with the capability of collecting minute to minute rainfall data is also recommended, as Environment Canada only provides hourly rainfall data. If more detailed flow data is desired in addition to the monitor located at the lagoons, several recommended locations have been identified as shown on Figure 4.1.

4.6 Existing System Evaluation

The existing system was assessed to examine the system performance for various rainfall events and to identify any deficiencies in the system. The existing system was evaluated for the 5 and 25 year short duration (4 hour) and long duration (24 hour) rainfall events.

For the short duration event, a 4 hour Chicago distribution was adopted. This distribution results in a high intensity rainfall, which is representative of short duration rainfall events. A 24 hour Huff distribution was chosen for the long duration event. This distribution results in a maximum rainfall intensity which is much lower than the Chicago distribution and is representative of long duration events. These distributions are typically used in computer modelling of urban drainage systems. The rainfall depths for the design events are summarized in Table 4.8. The rainfall hydrograph was applied such that the peak wet weather flow corresponds to the peak dry weather flow.

Table 4.8: Design Rainfall Events

Return Period (years)	Duration (hours)	Total Rainfall (mm)
5	4	34.0
	24	59.7
25	4	44.5
	24	75.5

The existing system was evaluated to assess the system performance with the proposed sewage generation rates by examining the following parameters:

- The capacity utilization within the system to identify potential locations where pipe flow exceeds pipe capacity; and
- The hydraulic grade line within the system to identify potential surcharge locations.

The magnitude of surcharging at manholes was calculated by subtracting the maximum hydraulic grade line (HGL) from the ground elevation and was divided into 3 levels as outlined in Table 4.9.

Table 4.9: Surcharging Levels

Rating	Depth of HGL Below Ground
Green	> 2.5 m
Blue	1 – 2.5 m
Red	0 – 1 m

The capacity utilization in the pipes was calculated by taking the ratio of the peak flow in the pipe to the pipe capacity and was divided into 3 levels as outlined in Table 4.10. Red indicates that the pipes are above capacity and should be upgraded, blue is the cautionary range and green indicates that capacity is available.

Table 4.10: Capacity Utilization Levels

Rating	Peak Flow / Pipe Capacity
Green	0 – 1.2
Blue	1.2 – 2
Red	> 2

Figures 4.4 to 4.8 show the surcharge and capacity utilization levels in the existing system for dry weather flow as well as the various rainfall events. The colour of the nodes or manholes indicates the level of surcharging and the colour of the pipes indicates the capacity utilization.

4.6.1 Dry Weather Flow Results

As seen in Figure 4.4, the existing system is sufficient to handle the dry weather flows in the Town. All of the nodes and links are green according to the legends given in Section 4.6. Any issues regarding the sanitary system are a result of wet weather flows.

4.6.2 5 Year Event Results

Both the 4 and 24 hour durations were run for the 5 year event. As seen in Figures 4.5 and 4.6, the 5 year 4 hour event poses more problems than the 5 year 24 hour event. Many nodes are red, and a small proportion of the links are blue. Major problem areas include the downtown core along 50 Street and 51 Street, 10 Avenue between 52 Street and 56 Street, and the industrial/residential area on the west side of Edson. The area on the west side experiences some out of system flooding during the 4 hour duration. Many nodes are blue for the 24 hour event, however, these pipes are slightly shallow and the hydraulic grade line is still within the diameter of the pipe. There is some surcharging above the top of pipe in the west area of the Town; however these nodes are under 1.0 m below ground and are in a non-residential area. The system has adequate capacity to convey the 5 year 24 hour rainfall event.

4.6.3 25 Year Event Results

Similarly to the 5 year event, both the 4 and 24 hour durations were run for the 25 year event. The results of these events are illustrated in Figures 4.7 and 4.8. Figure 4.7 depicts the 4 hour duration, showing many red nodes and links. Flooding is more widespread in the 25 year events than during the 5 year events. A major bottleneck occurs in the west end where the residential service connects to the rest of the system. The existing system does not have the capacity for the 25 year events.

For comparison purposes, the ratio of the peak wet weather flow to the peak dry weather flow is summarized in Table 4.11 for each design rainfall event. A composite sewage generation rate as well as the calculated I/I rate is also provided.

Table 4.11: Ratio of Peak Wet Weather Flow to Peak Dry Weather Flow for Design Rainfall Events

Design Event	Ratio	Composite Rate (L/p/d)	Calculated I/I Rate (L/s/ha)
Dry Weather Flow	1	623	-
5 Year 4 Hour	7	4401	0.77
5 Year 24 Hour	5	3010	0.49
25 Year 4 Hour	9	5543	1.00
25 Year 24 Hour	6	3685	0.63

4.7 Existing System Improvements

Currently, the Town of Edson experiences some sanitary sewer line flooding in both the 5 and 25 year events. The majority of the pipes are 200 mm in diameter, which in some cases is too small to handle the Town’s potential wet weather flows.

The sewer network also tends to back up because there are few lines that experience an increase in diameter as the line runs downstream. To solve these problems, several improvement scenarios have been developed. Potential solutions involve upgrading and/or twinning lengths of pipe in problem areas. Improvements were divided into 3 Phases. The Phase 1 upgrades address all of the surcharging within 1.0 m of the ground level for the 5 year 4 hour rainfall event. The Phase 2 upgrades address all the surcharging within 1.0 m of the ground level for the 25 year 4 hour rainfall event. The Phase 3 upgrades address all the surcharging within 2.5 m of the ground level within residential areas for the 5 year 4 hour event, therefore minimizing the risk of basement flooding. Both replacement and twinning options are shown as alternatives for system upgrades. The decision to twin or replace will be based on the condition of the existing pipes.

It is important to note that the majority of the sanitary sewer problems experienced in the town are due to wet weather flows. Improvements assume that the weeping tile remains connected to the system; however, it is recommended that weeping tile be disconnected from the sanitary system as the opportunity arises when other repairs or upgrades are being carried out. Roof leaders, catch basins and storm drains connected to the sanitary sewer, deteriorated manhole barrels and manholes located in sags are other sources of infiltration and inflow and should be addressed as part of the Town’s street improvement and maintenance programs. Backflow preventer valves are also a measure the Town can take to reduce the risk of basement flooding.

Based on the Town of Edson Lagoon Assessment completed by Earthtech in 2007, the existing lagoons have capacity for 9,500 people. This is sufficient for the existing population of 8,323 people.

4.7.1 Existing System Improvements – Phase 1

The Phase 1 improvements focussed on eliminating the problem areas which experienced surcharging to within 1.0 m of the ground level or flooding during the 5 year 4 hour event. These areas would be the most likely to experience basement flooding, therefore Phase I addresses the highest risk areas and eliminates flooding to the ground surface. Figure 4.9 illustrates the pipe lengths included in the proposed Phase 1 upgrades. Upgrades are proposed on 1st Avenue, 42nd Street, 49th Street, 51st Street, 52nd Street, 53rd Street, 55th Street and 70th Street. Several alternatives are proposed to alleviate the surcharging in the west industrial area. Alternative 1 involves upgrading the existing pipes while Alternatives 2 and 3 involve providing a new trunk along a new alignment to the existing 1050 mm line. A new line will be required for servicing of future areas to the west as well; however, for existing development, a 375 mm pipe is required (Alternative 2) while for 2025 development a 750 mm pipe is required (Alternative 3). The 750 mm is discussed further in Section 4.8. Phase 1 proposes upgrades to 5,964 m of pipe as outlined in Table 4.12.

Table 4.12: Summary of Phase 1 Existing System Upgrades

Location	Link Names	Node Names	Replacement Diameter (mm)	Twin Diameter (mm)	Length (m)
1st Avenue	L348-L340	N349-N341	900	NA	1073
42nd Street	L266-L312	N266-N314	375	NA	550
49th Street	L373-L370	N372-N350	375	300	471
51st Street	L250-L381	N248-N378	450	375	307
52nd Street	L101-L104	N100-N105	300	250	223
53rd Street	L497	N495-N500	525	NA	81
55/54th Street	L509-L532	N511-N531	525	375	519
55th Street	L822	N207-N793	300	250	10
70th Street	L599-L823	N600-N807	300	250/300	212
West trunk Upgrades (A1)	L594-L567	N600-N563	375	300/375	1414
New West Trunk (A2)*	L824-L8282	N600-N425	375	NA	2518
Upsize New West Trunk (A3)*	L824-L8282	N600-N425	750	NA	2518
Total					**5964**

Note: the twin diameter is not shown when it is the same as the replacement diameter.

4.7.2 Existing System Improvements – Phase 2

Phase 2 Improvements target the areas which experienced surcharging to within 1.0 m of the ground level or flooding during the 25 year 4 hour event. It is important to note that Phase 1 improvements must be made before Phase 2 improvements to achieve the results as modelled. Figure 4.10 illustrates the pipe lengths included in the proposed Phase 2 upgrades. 5,558 m of pipe are upgraded in this phase of improvements. A summary of the specific pipes to be upgraded is shown in Table 4.13.

Table 4.13: Summary of Phase 2 Existing System Upgrades

Location	Link Names	Node Names	Replacement Diameter (mm)	Twin Diameter (mm)	Length (m)
42nd Street	L267-269, L483	N269-N487	300/675	250/525	402
2nd Avenue	L354-L357	N354-N319	525	375	333
47th Street	L88-L374	N62-N373	300	250	855
48th Street	L394-L410	N389-N349	300/450/600	250/375/450	918
50th Street	L247-L246	N247-N239	300	250	237
1st Avenue	L414	N407-N405	525	450	172
52nd Street	L521-L492	N500-N495	375	300	223
Near 55th Street	L810-L511	N798-N512	525	375	242
54th Street -13th Avenue	L759-L766	N24-N115	375	375	295
10th Avenue	L210-L753	N212-N207	300/375	250/300	271
10th Avenue	L817	N800-N204	300	250	24
10th Avenue	L801-L813	N791-N799	300	250	145
10th Avenue	L198, L816	N198-N800	300	NA	137
4th Avenue west	L603-601, L608, L622-L618	N615-N600	300	250	1001
3rd Avenue	L585	N585-N581	300	250	86
64th Street	L639	N650-N649	300	250	103
4A Avenue	L625	N634-N633	300	250	112
Total					**5558**

4.7.3 Existing System Improvements – Phase 3

Phase 3 consists of several pipe upgrades eliminate all the surcharging within 2.5 m of the ground level within residential areas for the 5 year 4 hour event therefore minimizing the risk of basement flooding in all areas. Similar to Phase 2, Phases 1 and 2 must be completed before Phase 3 in order to achieve the desired results. 935 m of pipe are upgraded in this phase of improvements. Figure 4.11 illustrates the areas that require these upgrades. Table 4.14 describe the upgrades to be completed in these areas.

Table 4.14: Summary of Phase 3 Existing System Upgrades

Location	Link Names	Node Names	Replacement Diameter (mm)	Twin Diameter (mm)	Length (m)
42nd Street	L271	N272-N271	300	250	84
43rd Street	L45-L56	N45-N58	300	250	174
47th Street	L99	N56-N100	300	300	110
48th Street	L108-L109	N108-N110	375	300	85
Near 54th Street	L399-L400	N379-N395	300	250	263
4th Ave. (52nd - 53rd Street)	L526	N526-N522	300	250	171
41st Street	L323	N324-N323	300	250	48
Total					**935**

Based on the modeling and on discussions with the Town of Edson, it was not necessary to identify improvements for the 25 year 4 hour event as they would be too extensive to be practical. The majority of the pipes in the town would require upgrading. This would also be extremely expensive.

Improvements assume that the weeping tile remains connected to the system; however, it is recommended that weeping tile be disconnected from the sanitary system as the opportunity arises when other repairs or upgrades are being carried out. Roof leaders, catch basins and storm drains connected to the sanitary sewer, deteriorated manhole barrels and manholes located in sags are other sources of infiltration and inflow and should be addressed as part of the Town's street improvement and maintenance programs. Backflow preventer valves are also a measure the Town can take to reduce the risk of basement flooding. If extensive I/I reduction measures are undertaken, the effectiveness can be measured based on flow monitoring data and the required improvements can be reassessed.

The effect of future development on the upgraded system is analyzed in Section 4.8.

4.7.4 Existing System Improvements Cost Estimates

Costs for the various phases of sewer line improvements are based on the depth, length, and size of the pipe, as well as the type of ground-level rehabilitation required. Built out areas will require roadway restoration while pipes in open areas require grass restoration only. It is recommended that these improvements are integrated into the street improvement program or combined with other pipe improvement projects. The cost summaries for Phases 1-3 are outlined in Tables 4.15 to 4.17. Costs include supply, installation, excavation, manholes and restoration. Detailed cost estimates are available in Appendix G. The total cost for Phases 1, 2 and 3 are estimated to be \$11.8M, \$5.2M and \$0.6M respectively.

For the Phase 1 improvements there were three alternatives presented to alleviate the surcharging in the west part of the Town. A new pipe along a new alignment is recommended as it will be required for future development in 2015 and 2025. As the pipe will cross Highway 16 it is recommended that it be sized for future development, therefore a 750 mm pipe should be installed. The costs for all three alternatives are shown; however, only the cost for the new 750 mm line is included in the total.

Table 4.15: Phase 1 Cost Summary

Location	Link Names	Replacement Diameter (mm)	Twin Diameter (mm)	Length (m)	Replacement Cost ($)	Twinning Cost ($)
1st Avenue	L348-L340	900	NA	1073	$3,288,183	$3,288,183
42nd Street	L266-L295, L302-L312	375	250	550	$403,351	$403,351
49th Street	L373-L370	375	300	471	$351,410	$257,985
51st Street	L250-L381	450	375	307	$294,788	$218,020
52nd Street	L101-L104	300	250	223	$102,621	$78,082
53rd Street	L497	525	NA	81	$81,188	$81,188
55/54th Street	L509-L532	525	375	519	$628,447	$419,228
55th Street	L822	375	NA	10	$5,916	$5,814
70th Street	L599-L823	300	250/NA	212	$115,337	$108,933
Upgrade pipe from west end (Alternative 1)[1]	L594-L567	375	375/300	1415	$1,050,580	$930,789
New pipe from West End (Alternative 2)[1]	L824-L8282	375	375	2518	$2,101,760	$2,101,760
Upsize new pipe for new development (Alternative 3)[2]	L824-L8282			2518	$3,432,522	$3,432,522
			Subtotal	7285	$8,703,763	$8,293,306
			Engineering (10%) & Contingencies (25%)		$3,046,317	$2,902,657
			Total		**$11,750,080**	**$11,195,964**

[1]*Alternatives 1 and 2 are not included in the total cost for Phase 1*

[2]*Alternative 3 is included in the total cost for Phase 1 as it is the recommended alternative*

Table 4.16: Phase 2 Cost Summary

Location	Link Names	Replacement Diameter (mm)	Twin Diameter (mm)	Length (m)	Replacement Cost ($)	Twinning Cost ($)
42nd Street	L269-L483	300/375/675	200/300/450	514	$304,400	$163,257
4th Avenue	L354-L357	525	375	333	$333,150	$257,466
47th Street	L88-L374	300	250	855	$408,815	$326,183
48th Street	L394-L410	300/450/600	250/375/450	918	$887,190	$645,343
50th Street	L247-L246	300	250	237	$102,421	$83,001
1st Avenue	L414	525	450	172	$191,747	$141,875
52nd Street	L521-L492	375	300	223	$158,676	$102,804
Near 55th Street	L810-L511	375/525/450	250/450	242	$251,778	$171,699
54th Street -13th Avenue	L759-L766	300/375	250/375	295	$209,081	$209,081
10th Avenue	L210-L753	300/375	250/300	271	$200,167	$160,921
10th Avenue	L817	300	250	24	$11,040	$8,400
10th Avenue	L801-L813	300	250	145	$90,948	$82,120
10th Avenue	L198-L816	300	250	137	$63,112	$48,820
4th Avenue west	L608-L618	300	250	1001	$474,711	$377,891
3rd Avenue	L585	300	250	86	$39,679	$30,190
64th Street	L639	300	250	103	$47,527	$36,162
Subtotal				5558	$3,825,896	$2,883,564
Engineering (10%) & Contingencies (25%)					$1,339,064	$1,009,247
Total					**$5,164,960**	**$3,892,812**

Table 4.17: Phase 3 Cost Summary

Location	Link Names	Replacement Diameter (mm)	Twin Diameter (mm)	Length (m)	Replacement Cost ($)	Twinning Cost ($)
42nd Street	L271	300	250	84	$38,557	$29,337
43rd Street	L45-L56	300	250	174	$80,132	$60,970
47th Street	L99	300	250	110	$50,614	$50,614
48th Street	L108-L109	375	300	85	$60,606	$39,266
Near 54th Street	L399-L400	300	200	263	$120,797	$91,911
4th Ave. (52nd - 53rd Street)	L526	300	250	171	$78,857	$60,000
41st Street	L323	300	250	48	$22,292	$16,961
Subtotal				935	$451,854	$349,058
Engineering (10%) & Contingencies (25%)					$158,149	$122,170
Total					**$610,004**	**$471,229**

4.8 Future Sanitary Servicing Plan

A sanitary servicing plan was developed for the Town of Edson based on 2015 and 2025 development which includes build out and servicing of the areas shown in Figure 2.1. The future servicing plan assumes that all of the recommended upgrades outlined in Section 4.7 have first been completed. For future development, values developed as part of the Water Distribution system analysis were used. These values are provided in Table 4.18 and are similar to values used in other municipalities in Alberta. Land use was obtained from the plan as shown in Figure 2.1.

Table 4.18: Sanitary Sewer Design Values

Description	Value
Per Capita Sewage Generation Rate	330 L/c/d
Residential Peaking Factor	3
Non-residential Sewage Generation Rate	10,000 L/ha/d
Non-residential Peaking Factor	3
Infiltration and Inflow Allowance	0.28 L/s/ha

Future development areas were connected to the existing system by placing pipes where areas could best be serviced by gravity as well as where capacity is available. The in-fill areas were connected to the sanitary lines nearest to them. The most viable alternatives for expansion for the 2015 and 2025 development stages are summarized below.

4.8.1 2015 and 2025 Development

For 2015 and 2025 development the new industrial and residential development are connected to the existing upgraded sanitary system as seen in Figure 4.12. Preliminary pipe sizing and location is provided, however, these must be confirmed at the area structure plan and neighbourhood design level.

A lift station is required for the new developments east of the Town (Areas 15, 16 and 17), as the areas are at a significantly lower elevation than the existing wastewater lagoon. At the 2015 stage of development, the lift station will have a capacity of 28 L/s, and can pump via a 150 mm force main to the lagoons. To accommodate 2025 flow demands, the lift station will have to be upgraded to pump another 43 L/s for a total of 71 L/s via an additional 200 mm force main. Alternatively, a 250 mm force main can be installed in 2015. Approximately 1700 m of force main is required to connect new areas east of the Town to the existing lagoons. Area 13 is also located in a low spot and will require a lift station. A 41 L/s lift station is required with a 200 mm force main. Crossing of Highway 16 will be required to service this area.

The existing system with the proposed upgrades is adequate for the addition of residential areas to the northeast and northwest portions of the Town. This includes Areas 1, 2, 3, 4, 5, 6, 7, 8, 9, and 10.

For the west portion of the Town, a proposed new trunk line servicing the industrial areas in the west of Town will need to be upsized to accommodate the new areas. These new pipe upgrades are outlined in Table 4.19. Crossing of Highway 16 will be required. The new trunk recommended in Section 4.7.1 can be upsized to accommodate the future flows.

Table 4.19: 2015 and 2025 Development Scenario Pipe Upgrades

Location	Link Names	Node Names	Replacement Diameter (mm)	Twin Diameter (mm)	Length (m)
West Trunk	L824-L828	N807-N425	750	NA	2518

4.8.2 2015 and 2025 Development Cost Estimates

The existing system has adequate capacity to convey the additional flows from the future development areas with the Phase 1 to 3 improvements specified and no additional upgrades are required other than a new trunk in the west. Sizing was estimated and will depend on the actual neighbourhood layout and service areas at the time of development. Generally, the cost for the pipes to within the new development areas will be paid for by the developers including force mains and lift stations. The cost of the west trunk upgrade and realignment is shown in Table 4.20. Detailed cost estimates are available in Appendix G.

Table 4.20: 2015 and 2025 Development Scenario Cost Estimates: Trunk Sewers

Location	Link Names	Diameter (mm)	Length (m)	Cost ($)
West Trunk	L824-L828	750	2518	$3,432,522
Engineering (10%) & Contingencies (25%)				$1,201,383
Total				**$4,633,905**

Tunnelling was assumed under HWY 16

Table 4.21 gives a summary of estimated total cost for 2015 and 2025 development. All developments shown on Figure 4.12 are taken into account for each time frame. It should also be noted that elevations and slopes for the new sewer development are not available at this time, as these will depend on the final design. Detailed cost estimates for anticipated development for both 2015 and 2025 scenarios are available in Appendix F

Table 4.21: 2015 and 2025 Development Scenario Cost Estimates

Development Scenario	Total Length (m)	Cost ($)
2015	14,200	$9,107,500
2025	6,800	$3,535,000
Engineering (10%) & Contingencies (25%)		4,424,875
Total		**$17,067,375**

Based on the pipe lengths and diameters estimated, the flow capacity of the schematic pipe network can be determined. It is important to ensure that the additional pipes will not overload the capacity of the existing pipes. Table 4.22 compares estimated flow capacity in the new pipes with the available flow capacity of downstream pipes.

Table 4.22: 2015 and 2025 Development Scenario Pipe Flow Capacity Estimates

Area	Estimated Flow Rate (m^3/s)	Flow Capacity of Existing Downstream Pipes (m^3/s)	Ratio of Estimated Flow Rate to Downstream Pipe Flow Capacity
1	0.007	1.073	0.01
2	0.008	0.186	0.04
3	0.006	0.061	0.10
4	0.009	0.055	0.16
5	0.013	0.045	0.29
6	0.032	0.033	0.97
7	0.014	0.032	0.44
8	0.005	0.005	1.00
9 and 10	0.012	0.012	1.00
West (includes Areas 11-13, 18-23)	0.294	0.297	0.98
14	0.009	0.046	0.20
East (includes Areas 15-17)	0.071 (assumes 2025 development stage)	N/A (force main discharges directly to lagoons)	N/A

As seen in Table 4.22, all of the ratios are well below or equal to 1; therefore, the existing system is able to accommodate the flows resulting from the 2015 and 2025 development.

The assessment of the lagoons is outside the scope of this project; however, lagoon upgrades to service a population of approximately 15,000 people are detailed in the Town of Edson Lagoon Assessment Report (Earthtech, 2007). A cost of $2 million was estimated. The existing lagoons have capacity of 9,500 people. Therefore the existing lagoons will be sufficient until approximately 2013. The lagoon upgrades would be sufficient to service the estimated 2025 design population of 13,235 people. Treatment options are discussed as part of the Receiving stream Sensitivity Study (AECOM 2011); however, a life cycle cost analysis to compare a lagoon system to a mechanical treatment system should be considered as part of a separate study.

4.9 Cost Estimate Summary and Implementation Plan

The total cost has been calculated for each stage of development and is outlined in Table 4.23. Costs include 10% for engineering and 25% for contingencies. 2015 and 2025 system upgrades are not included in the total as they are included in Phase 1.

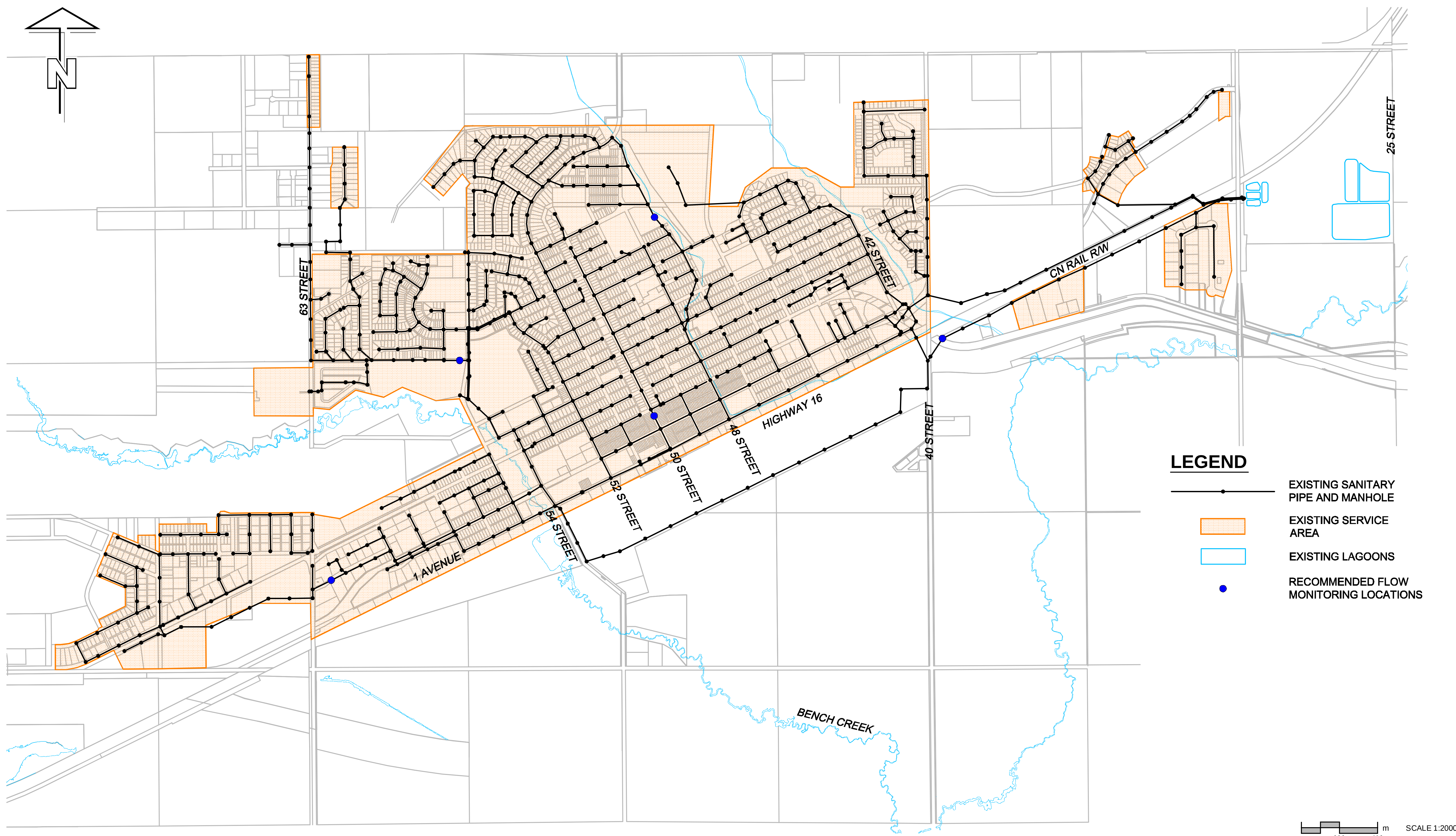
Table 4.23: Cost Estimate Summary

Description	Total Length (m)	Total Cost ($)
Existing System Upgrades		
-Phase 1	5964	$11,750,080
-Phase 2	5558	$5,164,960
-Phase 3	936	$610,004
2015/2025 System Upgrades[1]	2518	$4,633,905
Lagoon upgrades (Earthtech, 2007)	-	$2,010,000
2015 Development	14200	$12,295,125
2025 Development	6800	$4,772,250
Total	35975	**$36,602,419**

[1]2015/2025 system upgrades are not included in the total as they are included in Phase 1.

Implementation Plan

It is recommended that Phase 1 improvements are implemented first followed by Phase 2 and Phase 3 improvements. Generally, upgrades can be prioritized from downstream to upstream (east to west) and residential areas have higher priority than non-residential areas. However, improvements should be completed, where possible, as part of the street improvement program or other proposed underground projects to minimize the excavation and restoration costs as well as disruption.



Town of Edson
Municipal Servicing Plan Update

Existing Sanitary System and Service Area
Figure 4.1

AECOM

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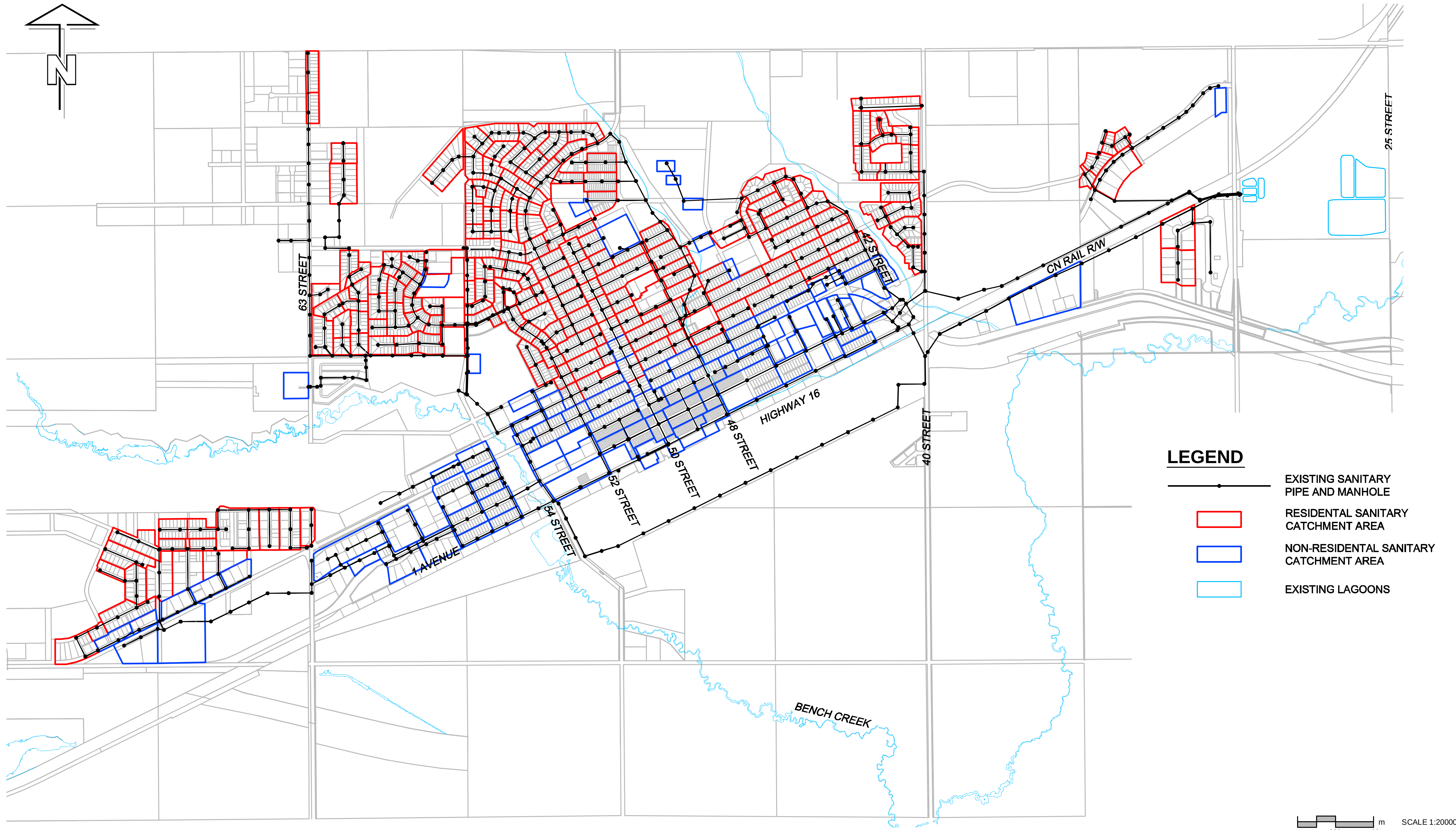

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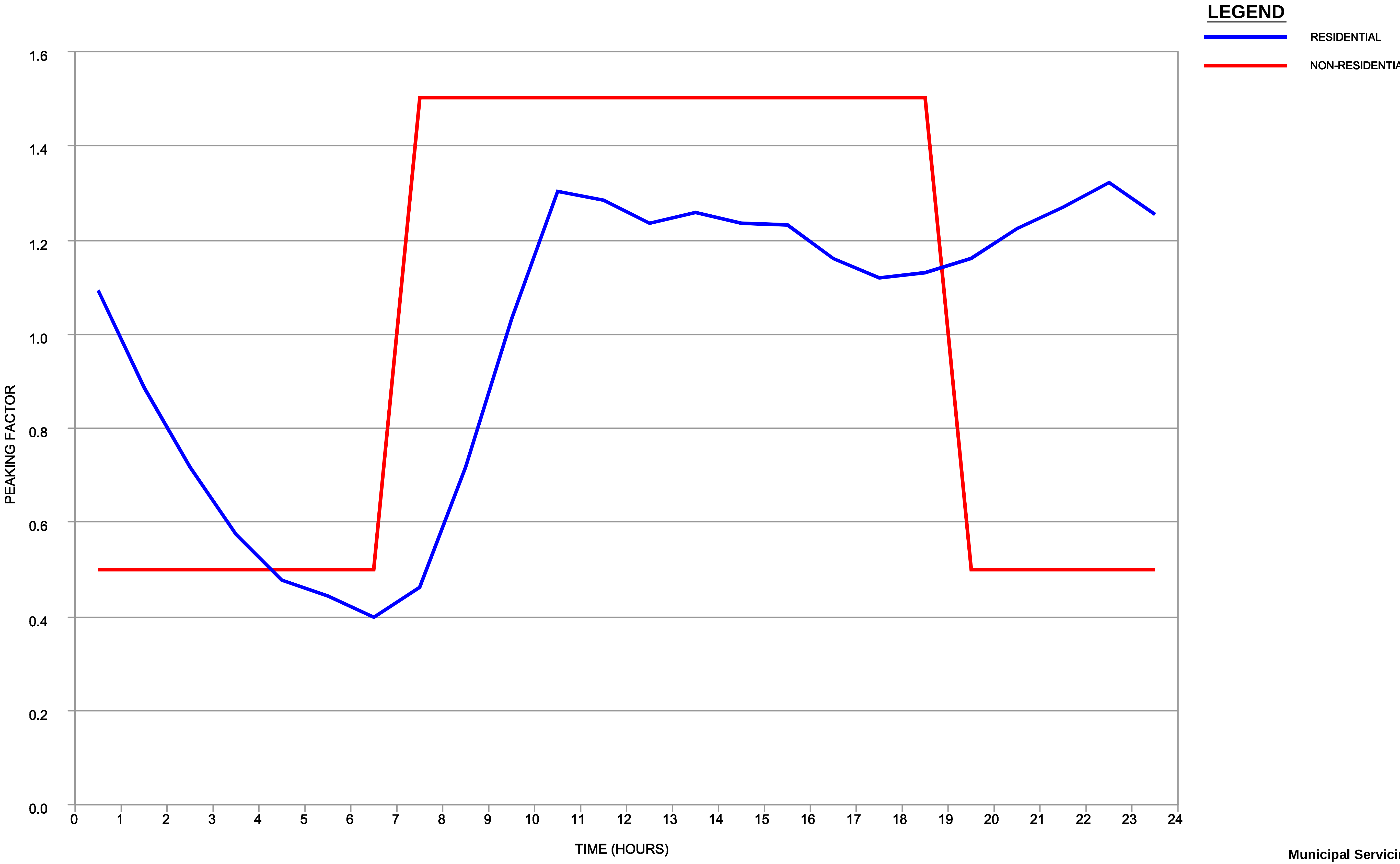
Town of Edson
Municipal Servicing Plan Update

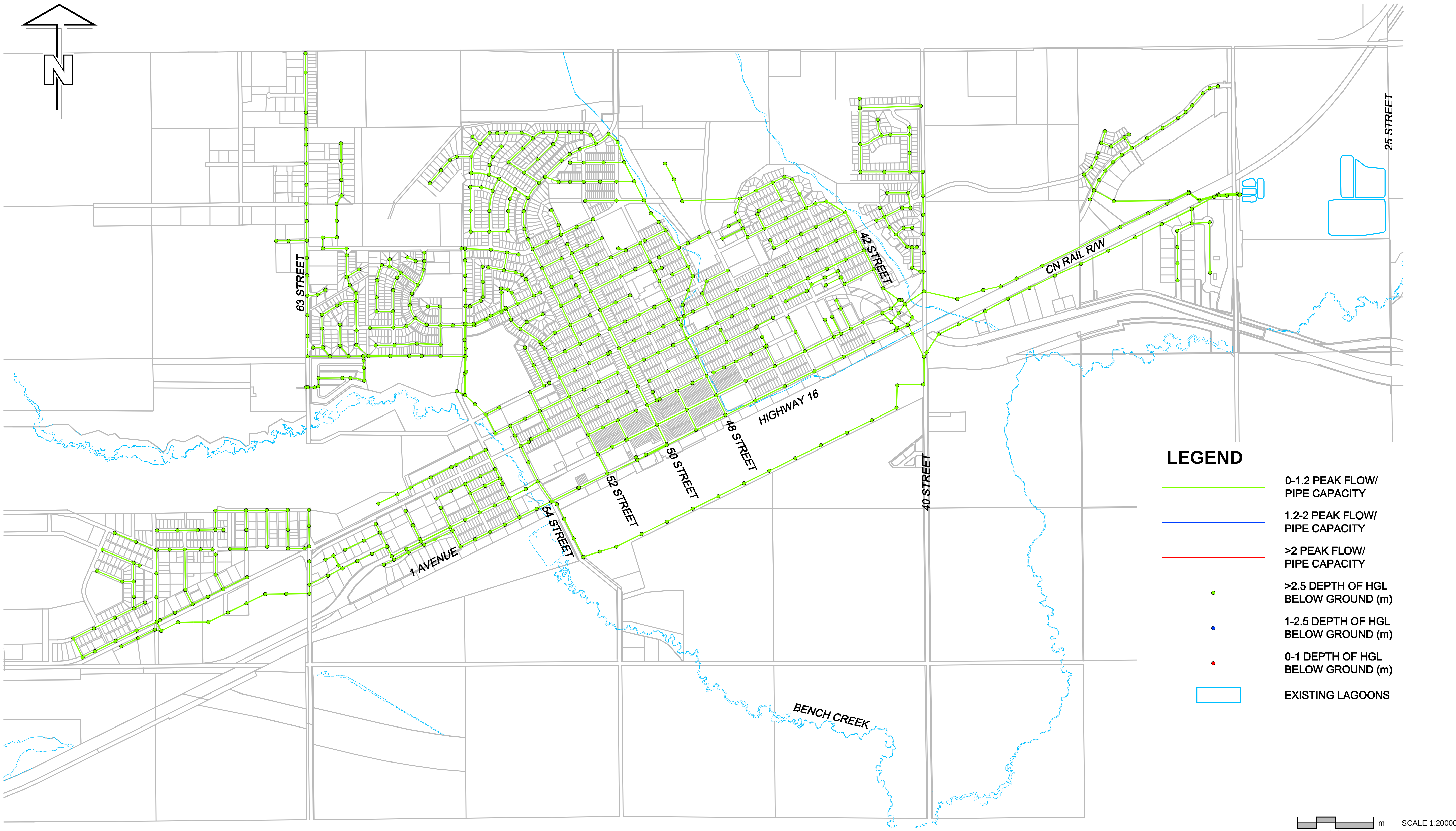
AECOM

Existing Sanitary Catchment Areas
Figure 4.2



Town of Edson
Municipal Servicing Plan Update

Dry Weather Flow
Diurnal Flow Pattern
Figure 4.3



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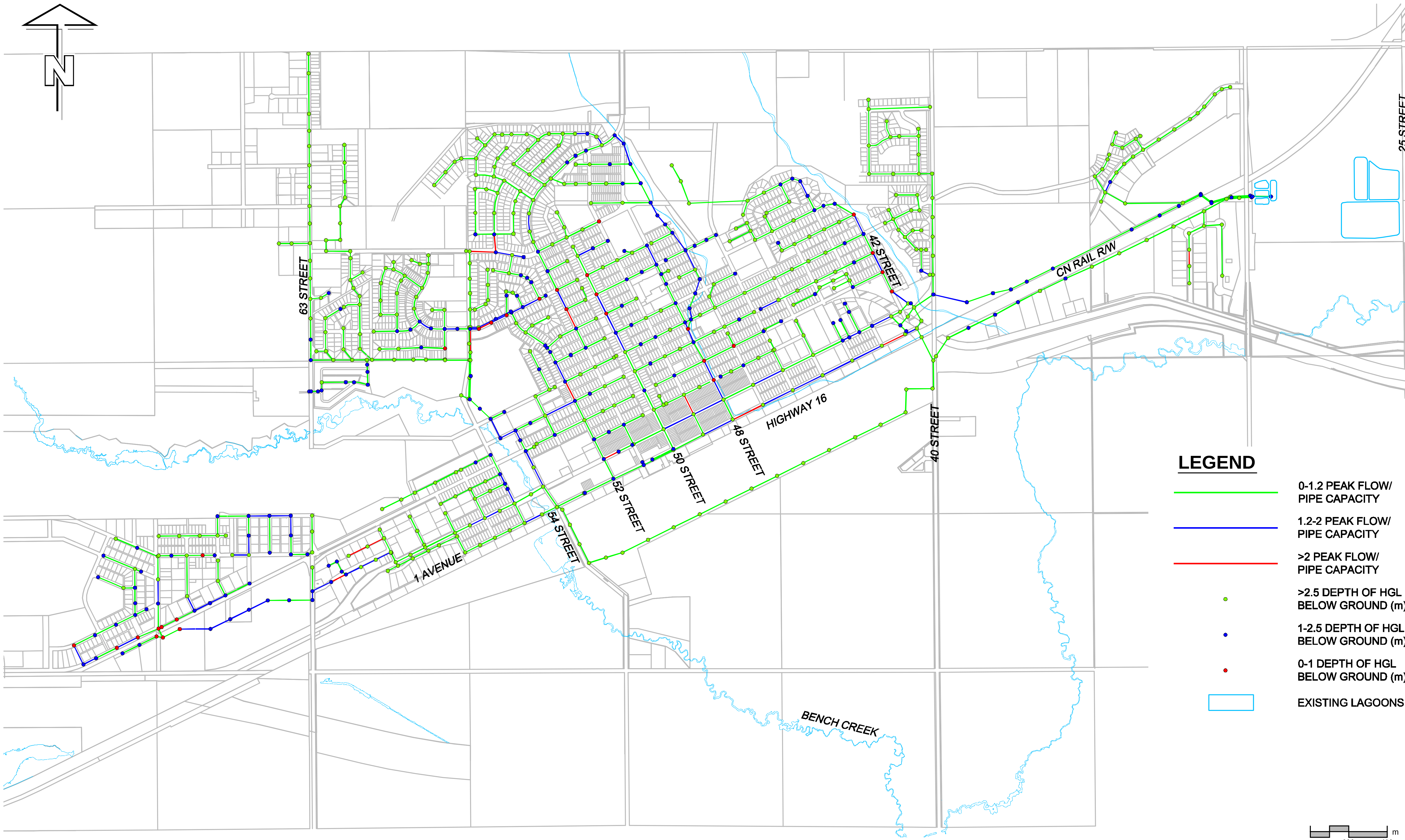
Town of Edson
Municipal Servicing Plan Update

Existing System
Dry Weather Flow Results
Figure 4.4



Town of Edson
Municipal Servicing Plan Update

Existing System
5 Year 4 Hour Event Results
Figure 4.5

AECOM

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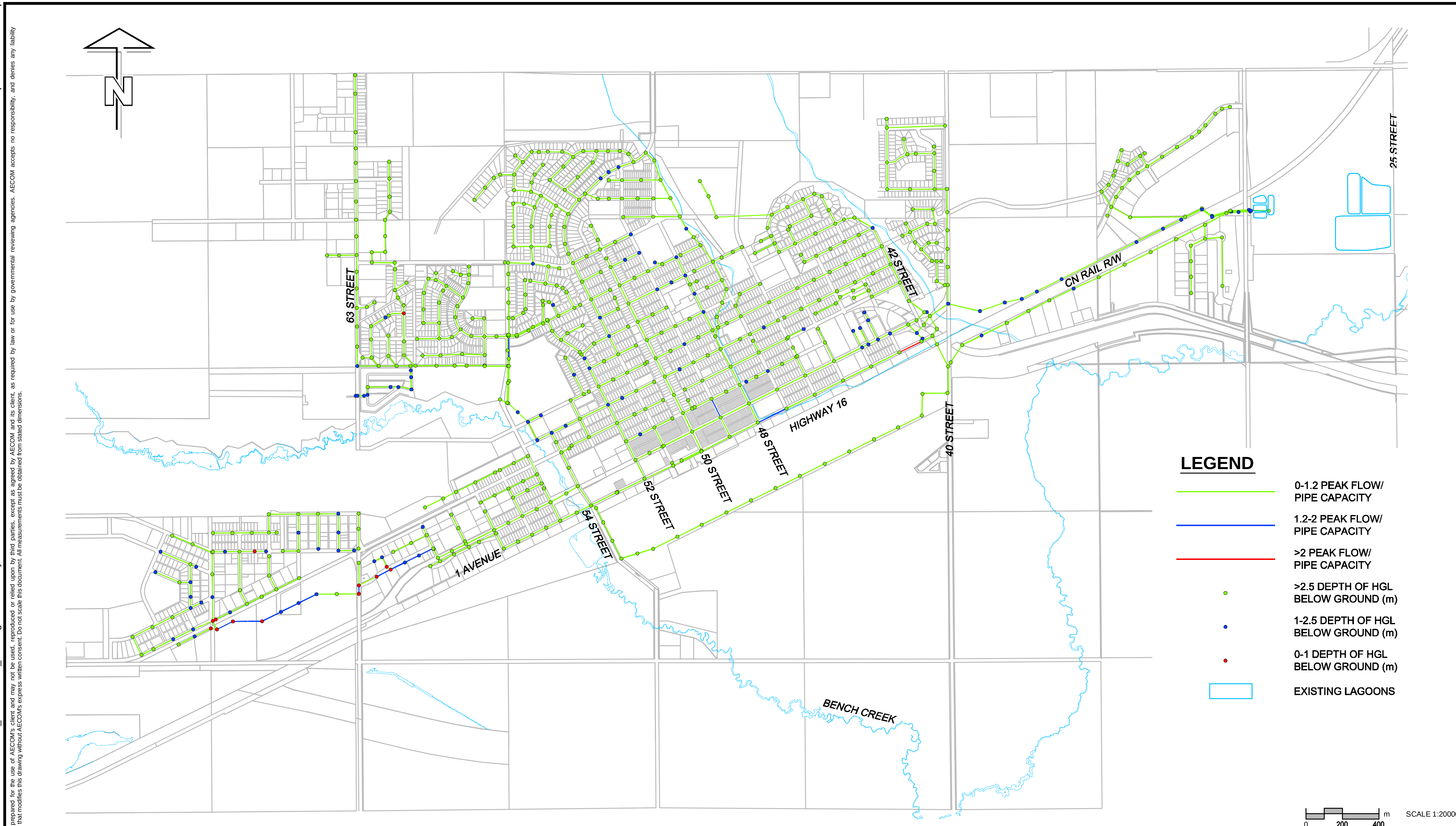
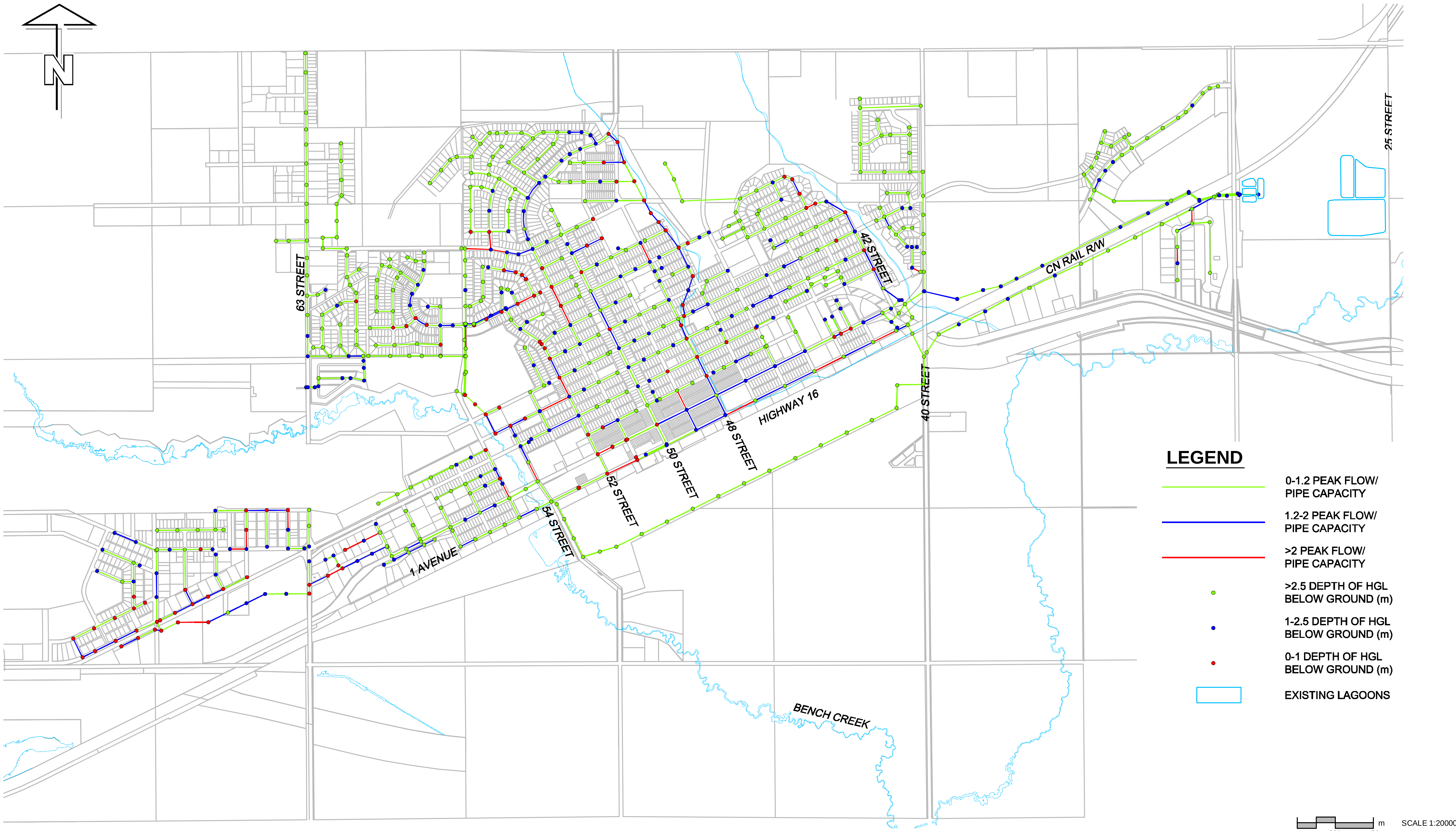


Town of Edson
Municipal Servicing Plan Update

Existing System
5 Year 24 Hour Event Results
Figure 4.6



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Town of Edson
Municipal Servicing Plan Update

Existing System
25 Year 4 Hour Event Results
Figure 4.7

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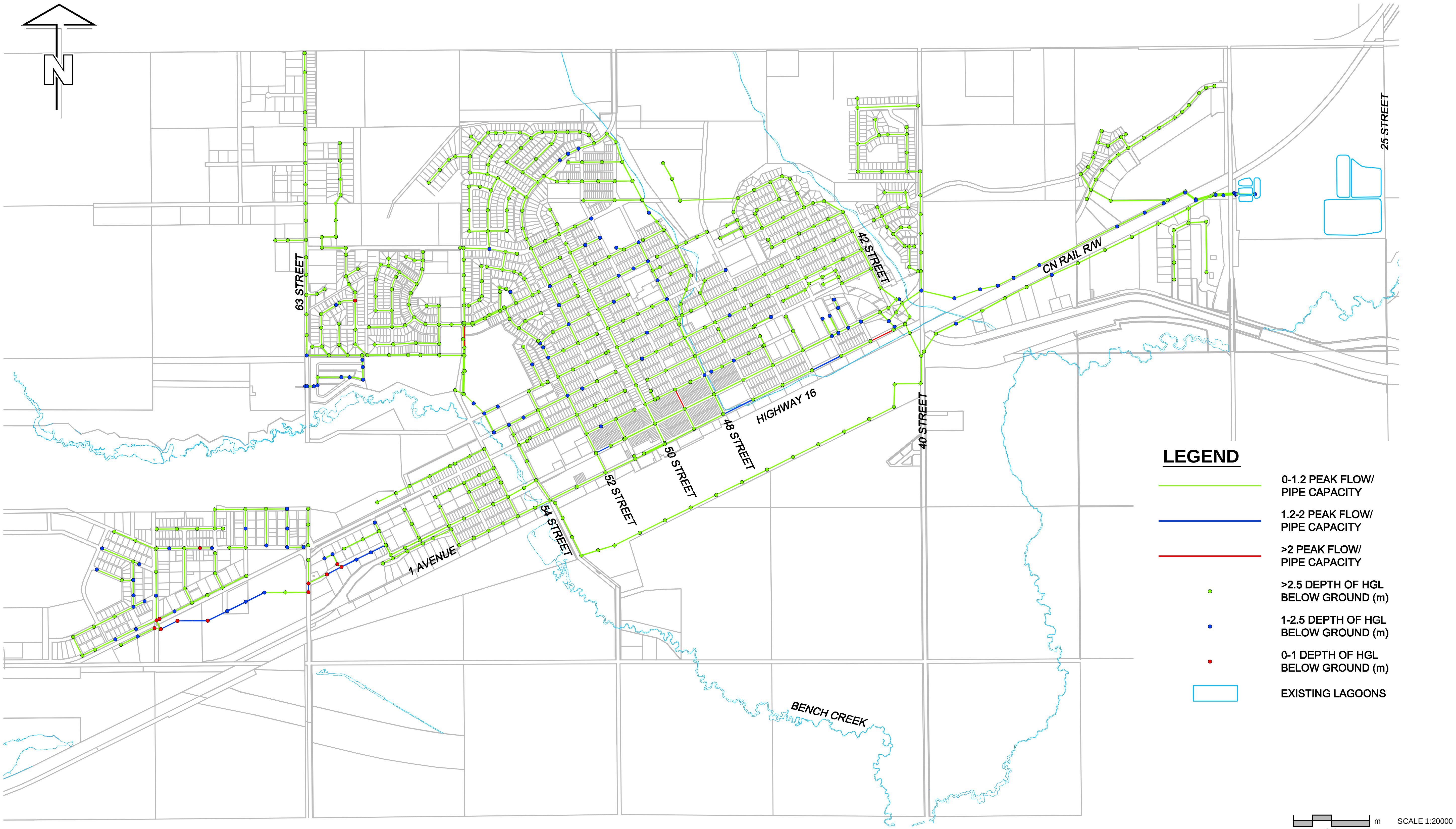

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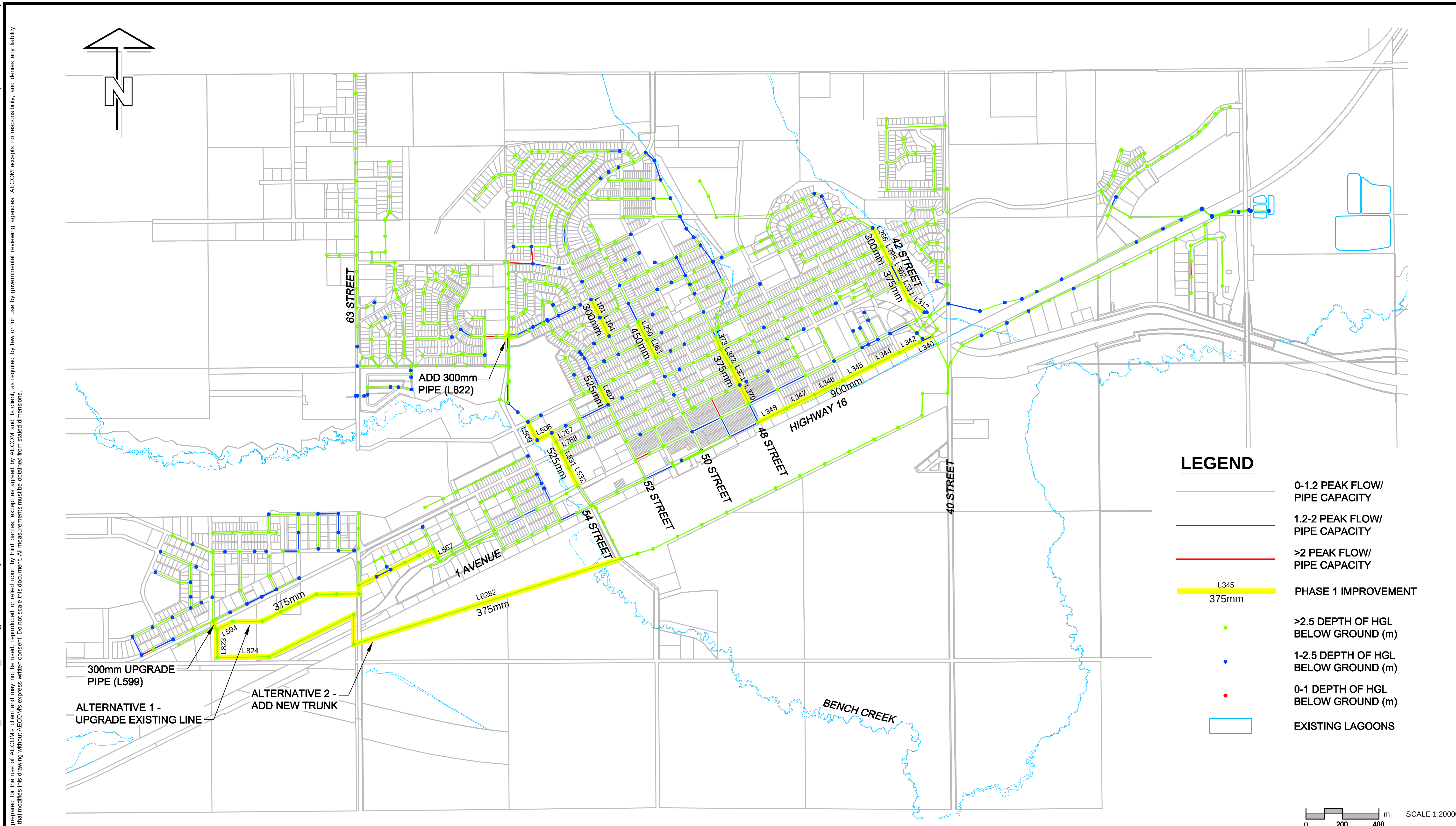
Town of Edson
Municipal Servicing Plan Update

Existing System
25 Year 24 Hour Event Results
Figure 4.8



Town of Edson
Municipal Servicing Plan Update

AECOM

Phase 1 Improvements
Figure 4.9

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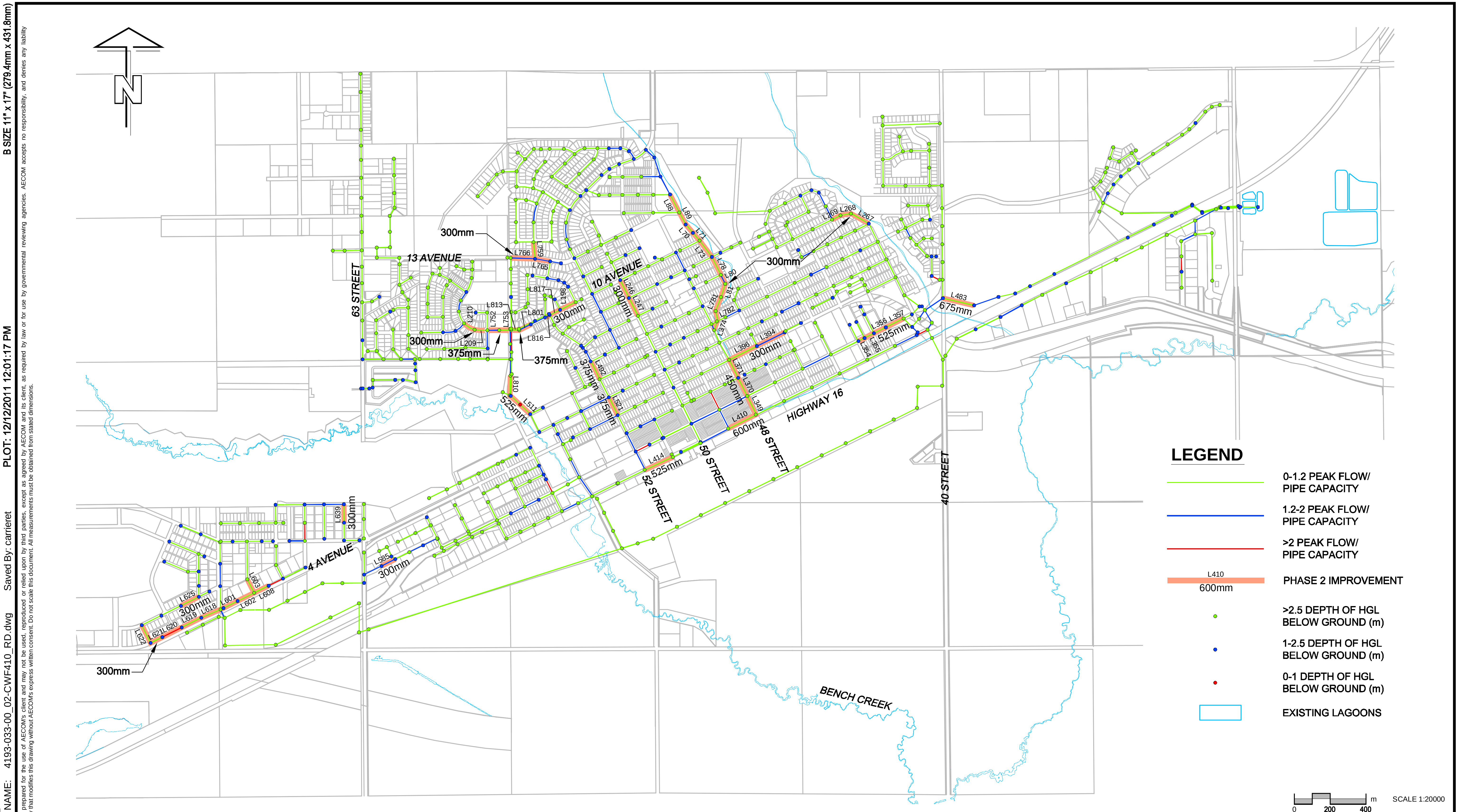

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AECOM

Town of Edson
Municipal Servicing Plan Update

Phase 2 Improvements
Figure 4.10

Town of Edson
Municipal Servicing Plan Update

AECOM

Phase 3 Improvements
Figure 4.11

ISS/REV: B
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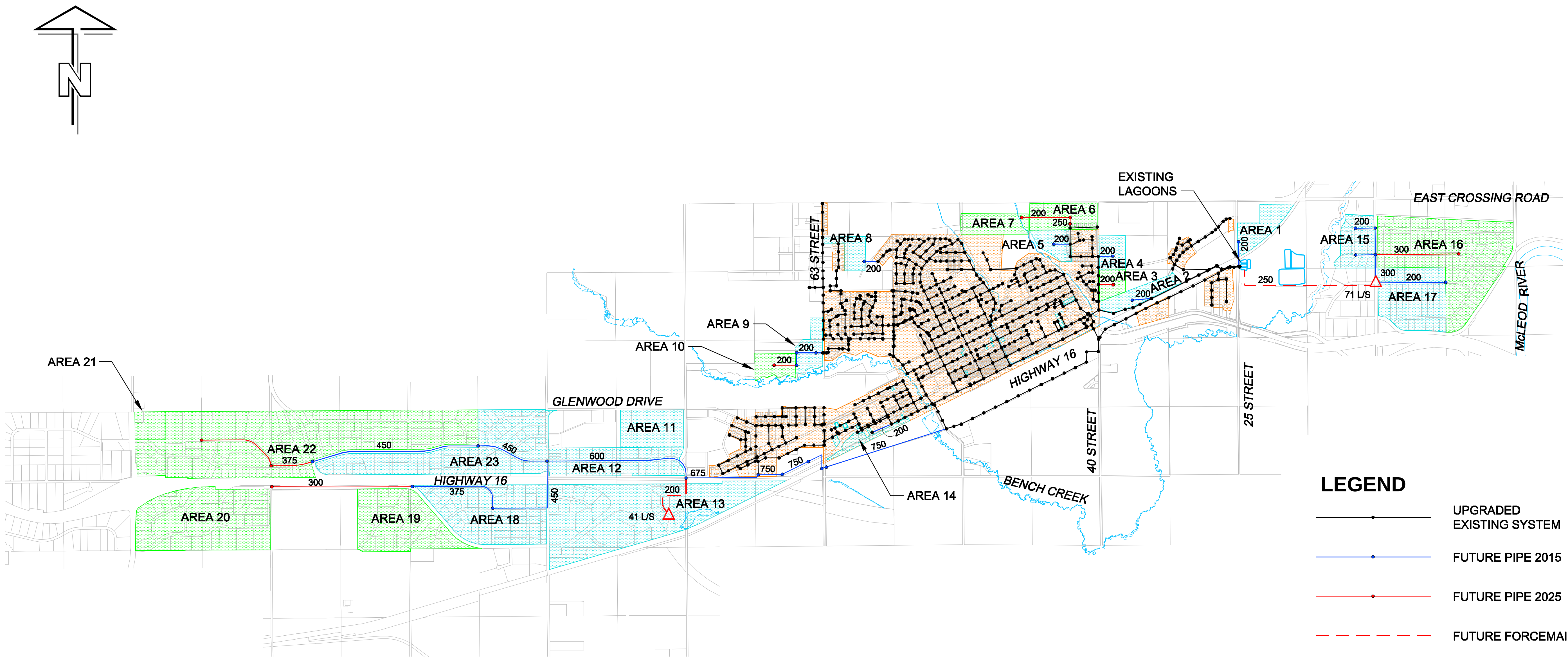

LEGEND

- UPGRADED EXISTING SYSTEM
- FUTURE PIPE 2015
- FUTURE PIPE 2025
- FUTURE FORCEMAIN
- FUTURE LIFT STATION
- EXISTING SERVICE AREA
- 2015 PROPOSED DEVELOPMENT
- 2025 PROPOSED DEVELOPMENT
- EXISTING LAGOONS

0 500 1000 m SCALE 1:50000

Town of Edson
Municipal Servicing Plan Update

2015 and 2025 Sanitary Servicing
Figure 4.12

AECOM

5. Stormwater Management System

5.1 General

This section assesses the capacity of the existing stormwater system, identifies existing system deficiencies and required improvements, identifies impact of future development, and provides a storm servicing concept for the ultimate development. Also included in this section is the development of stormwater management guidelines for infill developments.

5.2 Field Reconnaissance

The objective of the field reconnaissance was to assist in the assessment of the existing drainage system and clarify any drainage issues to better understand how the overall drainage system operates. The field reconnaissance also included an assessment of the undeveloped areas within the Urban Fringe Area.

Prior to the field reconnaissance, the existing information was assessed as closely as possible in order to identify overland drainage routes, existing drainage infrastructure, and any missing data. The field program was used to verify the findings of the data assessment and provide clarification for any points of interest that were not well documented in the collected materials.

The field program was used to clarify the following within the existing system:

- direction of flow through various ditches;
- location of outfalls;
- operation of control structures and stormwater management facilities; and
- overland drainage and flow paths.

Survey was conducted by the Town of Edson, to confirm or determine existing pipe locations, sizing and slope.

The field program of the Urban Fringe Area assisted in identifying:

- the overall drainage;
- the delineation of the drainage basins;
- the identification of drainage problems and additional stormwater issues;
- drainage infrastructure not identified on the record drawings;
- natural storage areas and potential locations for regional stormwater management facilities; and
- the performance of the existing drainage courses.

5.3 Existing System Modelling and Evaluation

The existing system was assessed to examine the system performance for various rainfall events and to identify any deficiencies in the system. The existing system was evaluated for the 5, 25, and 100 year short duration (4 hour) and long duration (24 hour) rainfall events. Undeveloped areas that naturally drain toward the system were included in the analysis in addition to the developed areas. Subsequent to the previous submission, pipe diameters, slopes and additional pond information was obtained and the existing system model was updated to reflect the information provided by the town in August 2011.

For the short duration event, a 4 hour Chicago distribution was adopted. This distribution results in a high intensity rainfall, which is representative of short duration rainfall events. A 24 hour Huff distribution was chosen for the long duration event.

This distribution results in a maximum rainfall intensity which is much lower than the Chicago distribution and is representative of long duration events. These distributions are typically used in computer modelling of urban drainage systems. The rainfall depths and intensities for the Edson design events are summarized in Table 5.1.

Table 5.1: Design Rainfall Events

Return Period	Duration (hours)	Total Rainfall (mm)	Peak Intensity (mm/hr)
5	4	34.0	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

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

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

The magnitude of surcharging in the storm sewer system, indicated by the hydraulic grade line, was categorized into the levels as outlined in Table 5.2.

Table 5.2: Storm Sewer Capacity Utilization Levels

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

The magnitude of surcharging at manholes was calculated by subtracting the maximum hydraulic grade line (HGL) from the ground elevation and was divided into the levels as outlined in Table 5.3.

Table 5.3: Storm Sewer Levels of Surcharging

Rating	Depth of HGL Below Ground (m)
Red	Flooding
Yellow	0-1.0
Blue	1.0-2.5
Green	>2.5

Typically, when the capacity utilization is less than 1.0 the trunk flows under open channel conditions, which is the most desirable flow condition. The capacity of the pipe was considered adequate when the peak flow to pipe capacity ratio is less than or equal to 1.2; indicated by a green pipe. Peak flow to pipe capacity ratio in the range of 1.2 to 2.0 are indicated by yellow pipes and are cautionary, and those pipes with a ratio greater than 2.0 were considered undersized and require upgrading.

A green pipe, indicating a capacity utilization of less than 1.0, could be surcharged in the situation when downstream trunks are surcharging, causing flow to back-up. In addition, as nodes are indicative of the HGL level below ground, a high level of surcharging may be indicated in free-flowing shallow pipes. To determine if the pipe is surcharged or not, the results need to be further evaluated to determine the relation of the maximum HGL depth to the pipe obvert.

Sections 5.3.1 to 5.3.3 summarize the results of the existing system when simulated in the XP-SWMM model with the 5, 25, and 100 year rainfall events. As the return periods increase, and therefore, the rainfall amount increases, the impact to the system will also increase. Most sewer systems are not designed to convey rainfall return periods larger than the 5 year. It is expected that the 4 hour duration rainfall events will have the most impact on the sewer system, as it has a much greater peak intensity than the 24 hour rainfall event, resulting in higher flows.

5.3.1 5 Year Rainfall Event Results

The existing system results for the 5 year 4 hour rainfall event are shown on Figure 5.1. During the 4 hour event, there is a large amount of surface flooding, indicated by red nodes. 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.

Figure 5.2 shows the results for the 5 year 24 hour rainfall event. The system performs significantly better during the 24 hour rainfall event. For the 24 hour event, many nodes are blue, however, these pipes are within 2.5 m of ground and the hydraulic grade line 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.

5.3.2 25 Year Rainfall Event Results

Figure 5.3 shows that storm system performance during the 25 year 4 hour 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 this large of an event.

The 25 year 24 hour rainfall event results are shown on Figure 5.4. Overall, the system generally has adequate capacity to convey the rainfall event, as indicated by the green links. However, there is surface flooding indicated at one node along 51st Street and the pipes downstream are surcharging, as shown by the red and yellow links.

5.3.3 100 Year Rainfall Event Results

As shown in Figure 5.5, conditions continue to get worse in the 100 year 4 hour rainfall event, with the majority of the storm system flooding. However, a storm sewer system is generally not ever designed for such a large event and it would be expected that flooding would occur during a 100 year return period rainfall event.

The system performs quite well during the 24 hour event, with flooding only occurring at a few nodes. As seen in Figure 5.6, the majority of the system continues to have capacity throughout the 100 year 24 hour rainfall.

5.4 Existing System Improvements

The Town of Edson does not have documented Engineering Design Standards for stormwater drainage systems. It is recommended that the Town of Edson consider developing Engineering Design Standards for stormwater drainage systems. Storm sewer systems are typically designed to contain the 5 year 4 hour rainfall event with no surcharging. For the proposed existing system improvements, a level of service such that there is not surcharging within 1.0 m of ground for a 5 year 4 hour rainfall event will be adopted. 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 rest of the system.

Improvements to existing storm sewer systems are typically achieved through implementing storage or by increasing the sewer capacity. There are not many areas that would effectively provide storage within the existing developed areas of Edson; therefore, the proposed improvements consider pipe upgrades. Pipe upgrades were determined for both replacement and twinning options. The decision to twin or replace pipes will be based on the condition of the existing pipes.

Table 5.4 summarizes the proposed upgrades required to achieve the recommended level of service, Figure 5.7 shows the locations of the proposed pipe upgrades. A detailed list of upgrades can be found in Appendix I. Figure 5.8 shows the improved system during the 5 year 4 hour rainfall. Nodes that are shown in yellow on Figure 5.8 are shallow systems and are not indicative of surcharging. Some surcharging, represented by yellow and red links, is still present in the improved system during the 5 year 4 hour rainfall event however, these are localized and do not result in the HGL being within 1.0 m of the ground.

Table 5.4: Proposed Storm Sewer Improvements

Link Names	Total Length of Improvements (m)	Existing Diameters (mm)	Replacement Diameters (mm)	Twinned Diameters (mm)
2-002 – 2-015 2-003 – 2-004 2-009 – 2-010	679.9	450 - 900	525 - 1,050	375- 600
1-004 – 1-008 1-014 – 1-017	517.6	450 - 675	525 - 750	375 - 450
1-018 – 1-062 1-069 – 1-077	1,621.5	300 - 1,050	450 – 1,350	375 - 900
3-008 – 3-016	821.3	375 - 525	450 - 900	375 - 900
3-017 – 3-022	469.3	450	525 - 675	375 - 525
4-001 – 4-008	588.6	375 - 600	450 - 675	375 - 450
1-036 – 1-042 1-044 – 1-090 1-093 – 1-107 (includes realignment)	4,131.5	300 – 1,050	450 -1,350	375 – 1,350
1-108 – 1-112	445.3	300 - 450	525 - 675	375 - 600
1-114 – 1-116	343.9	375 -600	525 - 900	375 -675
4-011 – 4-014	336.9	300 - 450	450 - 600	375 - 450
1-118 – 1-124 1-125	932.9	300 - 900	525 -1200	375 - 1050
4-016 – 4-019	296.9	300 -375	450 - 600	375 - 525
4-020	138.7	300	450	375
4-032 – 4-035	306.6	300	450 - 525	375 - 450
4-036 – 4-039	306.6	300	525 - 600	450 - 525
4-041 – 4-046	550.9	250 - 675	525 – 1,050	375 - 900
4-022 – 4-024 4-025 – 4-030	423.3	300 - 450	600 -750	450 - 675
4-048 – 4-053	440.1	300 - 750	450 - 900	375 - 525
4-054 – 4-055	209.0	300	450	375
5-001 – 5-004	196.6	300 - 450	450 - 525	375
4-061	357.2	300 - 600	525 - 900	375 - 675

Link Names	Total Length of Improvements (m)	Existing Diameters (mm)	Replacement Diameters (mm)	Twinned Diameters (mm)
4-063a – 4-064				
5-005 – 5-010	402.3	300 - 375	375 - 525	375 - 450
5-013 – 5-015	238.4	300 - 600	450 - 900	375 - 675
5-017 – 5-022	514.6	300 - 450	450 -750	375 - 675
6-001 – 6-006 6-010	465.3	200 - 525	450 - 675	375 - 525
6-021 – 6-024	253.0	300 - 375	450 - 525	375 - 450

It should be noted that the storm sewer system with Links 1-036 to 1-107, realignment is proposed. The recommended level of service was best achieved by increasing the slope of the pipes in the areas indicated on Figure 5.7. Keeping the existing slopes resulted in significantly large pipe diameters, which even then, did not necessarily result in meeting the recommended level of service.

5.5 Future Storm Servicing Plan

A stormwater management plan was developed for the Town of Edson based on 2015 and 2025 development which includes build out and servicing of the areas shown in Figure 2.1. The future stormwater management plan is not dependant on the existing system upgrades in Section 5.4.

The future development areas were delineated into storm drainage basins, shown in Figure 5.9. Several of these storm basins are expected to be partially developed by 2015 with the remaining to be developed by 2025. The 2015 development areas include the 2015 areas. Table 5.5 summarizes the basin areas, the proposed land use and impervious ratios for each of the proposed drainage basins.

Table 5.5: Stormwater Drainage Basins

Basin	Year of Development	Drainage Basin Area (ha)	Land Use	Percent Imperviousness
A	2015	11.4	Residential	50
A	2025	60.1	Industrial/ Commercial	90
B	2025	51.3	Industrial/ Commercial	90
C	2025	63.7	Industrial/ Commercial	90
D	2025	58.6	Industrial/ Commercial	90
E	2025	57.3	Industrial/ Commercial	90
F	2025	75.4	Industrial/ Commercial	90
G	2025	53.1	Industrial/ Commercial	90
H	2025	74.1	Industrial/ Commercial	90
I	2025	60.8	Industrial/ Commercial	90
J	2025	91.7	Industrial/ Commercial	90
K	2025	53	Industrial/ Commercial	90
L	2025	77.1	Industrial/ Commercial	90
M	2025	33.7	Residential	50
N	2025	58.1	Industrial/ Commercial	90
O	2025	37.1	Industrial/ Commercial	90
P	2015	15.1	Residential	50
P	2025	22	Residential	50
R	2025	17.3	Residential	50
S	2015	13	Residential	50
S	2025	39.9	Residential	50

Basin	Year of Development	Drainage Basin Area (ha)	Land Use	Percent Imperviousness
T	2015	12.5	Residential	50
T	2025	29	Residential	50
U	2025	18.1	Industrial/ Commercial	90
V	2025	20	Industrial/ Commercial	90
W	2025	26.3	Industrial/ Commercial	90
X	2025	90.2	Industrial/ Commercial	90
Y	2015	56.1	Industrial/ Commercial	90
Y	2025	93.5	Industrial/ Commercial	90

Each of the proposed drainage basins will be graded such that the runoff is routed to a stormwater management facility (SWMF). Proposed stormwater management facilities were located within local depressions, or at the low-end of the basin. SWMF locations are conceptual and can change within the basin as required. The future SWMFs will be designed to service the critical 100 year rainfall event while discharging at the allowable discharge rate. Figure 5.10 shows the proposed stormwater management plan, including proposed SWMF locations and proposed drainage routes.

5.5.1 Allowable Discharge Rate

Allowable discharge rates for the 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 for the creeks. Table 5.6 summarizes the allowable discharge rates for Bench, Wase and Poplar Creek.

Table 5.6: Receiving Watercourse Allowable Discharge Rates

Creek	Allowable Discharge Rate (L/s/ha)
Bench	2.8
Wase	7.2
Poplar	9.0

Based on the allowable discharge rates and the drainage basin area for each basin, the allowable rate of discharge for each proposed stormwater management facility was determined. Table 5.7 summarizes the maximum rates based on the basin areas.

Table 5.7: Stormwater Management Facilities Allowable Discharge Rates

Basin	Year of Development	Drainage Basin Area (ha)	Receiving Watercourse	Maximum Discharge Rate (m^3/s)
A	2015	11.4	Bench Creek	0.03
A	2025	60.1	Bench Creek	0.17
B	2025	51.3	Bench Creek	0.14
C	2025	63.7	Bench Creek	0.18
D	2025	58.6	Bench Creek	0.16
E	2025	57.3	Bench Creek	0.16
F	2025	75.4	Bench Creek	0.21
G	2025	53.1	Bench Creek	0.15
H	2025	74.1	Bench Creek	0.21

Basin	Year of Development	Drainage Basin Area (ha)	Receiving Watercourse	Maximum Discharge Rate (m^3/s)
I	2025	60.8	Bench Creek	0.17
J	2025	91.7	Bench Creek	0.26
K	2025	53	Bench Creek	0.15
L	2025	77.1	Bench Creek	0.22
M	2025	33.7	Bench Creek	0.09
N	2025	58.1	Bench Creek	0.16
O	2025	37.1	Bench Creek	0.1
P	2015	15.1	Bench Creek	0.04
P	2025	22	Bench Creek	0.06
R	2025	17.3	Poplar Creek	0.16
S	2015	13	Poplar Creek	0.12
S	2025	39.9	Poplar Creek	0.36
T	2015	12.5	Poplar Creek	0.11
T	2025	29	Poplar Creek	0.26
U	2025	18.1	Poplar Creek	0.16
V	2025	20	Bench Creek	0.06
W	2025	26.3	Bench Creek	0.07
X	2025	90.2	Bench Creek	0.25
Y	2015	56.1	Bench Creek	0.16
Y	2025	93.5	Bench Creek	0.26

It is proposed that the SWMFs be designed to be wet facilities to allow for sediments to settle out of the runoff and therefore enhance the water quality before being released. Water quality enhancement is generally achieved with deep permanent storage in wet facilities by slowing down the runoff and thus inducing the settlement of particles. Alberta Environment requires that a minimum of 85% of sediments with a particle size of 75 µm or greater be removed from the runoff.

The drainage system was assessed using XP-SWMM, an industry accepted stormwater management model, for the 100 year rainfall event with durations of 4 hours and 24 hours. As the SWMF's are proposed to be wet facilities, they were 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 having a trapezoidal shape with side slopes 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 configuration of the SWMFs can be addressed in detail during the preliminary design phase.

The results of the model simulation showed that there were two governing rainfall events for the proposed SWMFs. The hydrologic and hydraulic modelling results for both the 100 year 4 hour and 100 year 24 hour are provided in Appendix G and Appendix H. Table 5.8 summarizes the storage requirements of the proposed SWMFs. Basins A, P, S, T, and Y have a lower required storage volume for 2015 development than for 2025 development. 2025 development will result in critical storage volume compared to 2015, and will ultimately be used to size the SWMFs. Interim SWMFs can provide storage for 2015 development, however, sufficient land must be available for ultimate storage.

Table 5.8: Storage Volumes

Basin	Year of Development	Drainage Basin Area (ha)	Critical Rainfall Event	Storage Volume (m^3)
A	2015	11.4	100 year 24 hour	3,400
A	2025	60.1	100 year 24 hour	34,300
B	2025	51.3	100 year 24 hour	29,500
C	2025	63.7	100 year 24 hour	36,000
D	2025	58.6	100 year 24 hour	33,900
E	2025	57.3	100 year 24 hour	32,500
F	2025	75.4	100 year 24 hour	42,100
G	2025	53.1	100 year 24 hour	30,400
H	2025	74.1	100 year 24 hour	41,000
I	2025	60.8	100 year 24 hour	34,700
J	2025	91.7	100 year 24 hour	52,000
K	2025	53	100 year 24 hour	30,300
L	2025	77.1	100 year 24 hour	42,400
M	2025	33.7	100 year 24 hour	10,000
N	2025	58.1	100 year 24 hour	33,300
O	2025	37.1	100 year 24 hour	21,400
P	2015	15.1	100 year 24 hour	4,400
P	2025	22	100 year 24 hour	6,400
R	2025	17.3	100 year 4 hour	3,000
S	2015	13	100 year 4 hour	2,500
S	2025	39.9	100 year 4 hour	6,900
T	2015	12.5	100 year 4 hour	2,300
T	2025	29	100 year 4 hour	4,800
U	2025	18.1	100 year 24 hour	6,700
V	2025	20	100 year 24 hour	10,900
W	2025	26.3	100 year 24 hour	14,900
X	2025	90.2	100 year 24 hour	48,900
Y	2015	56.1	100 year 24 hour	31,400
Y	2025	93.5	100 year 24 hour	52,000

The 4 hour duration rainfall event is the critical event for SWMFs that have residential development and discharge to Poplar Creek. These basins have a lower percent imperviousness, representing a basin with larger pervious area. A higher intensity rainfall event, such as the 4 hour duration, produces larger runoff amounts over pervious surfaces since the soil infiltration capacities are exceeded quickly due to the high intensity, and therefore generate a greater amount of runoff than during a lower intensity rainfall where the soil has the capacity to infiltrate the rainfall. Since the basins with lower percent imperiousness have more runoff being generated by the pervious surfaces, the higher intensity rainfall event governs.

5.6 Cost Estimates

Cost estimates were prepared for the storm sewer upgrades and proposed stormwater management facilities. The cost estimates for the storm sewer upgrades were prepared based on the following assumptions:

- costs are in 2009 dollars;
- open cut pipe installation includes excavation and backfill;

- manholes are assumed to be located every 150 m;
- costs do not include any crossings; and
- costs include 25% for contingency and 10% for engineering.

Table 5.9 summarizes the cost estimates for the sewer upgrades. Detailed cost estimates are located in Table I-1 of Appendix I.

Table 5.9: Storm Sewer Improvements Cost Estimates

System	Replacement Cost ($)	Twinning Cost ($)
2-002 – 2-015 **2-003 – 2-004** **2-009 – 2-010**	1,430,000	866,000
1-004 – 1-008 **1-014 – 1-017**	738,000	573,000
1-018 – 1-062 **1-069 – 1-077**	4,396,000	2,987,000
3-008 – 3-016	1,465,000	1,303,000
3-017 – 3-022	640,000	555,000
4-001 – 4-008	804,000	592,000
1-036 – 1-042 **1-044 – 1-090** **1-093 – 1-107** **(realignment)**	8,950,000	8,261,000
1-108 – 1-112	611,000	557,000
1-114 – 1-116	646,000	443,000
4-011 – 4-014	447,000	351,000
1-118 – 1-124 **1-125**	1,839,000	1,515,000
4-016 – 4-019	393,000	324,000
4-020	181,000	134,000
4-032 – 4-035	404,000	360,000
4-036 – 4-039	410,000	401,000
4-041 – 4-046	1,225,000	1,086,000
4-022 – 4-024 **4-025 – 4-030**	639,000	581,000
4-048 – 4-053	752,000	532,000
4-054 – 4-055	272,000	202,000
5-001 – 5-004	258,000	189,000
4-061 **4-063a – 4-064**	693,000	458,000
5-005 – 5-010	532,000	439,000
5-013 – 5-015	350,000	279,000
5-017 – 5-022	754,000	661,000
6-001 – 6-006 **6-010**	620,000	550,000
6-021 – 6-024	333,000	261,000
TOTAL	**29,782,000**	**24,460,000**

From Table 5.9, it can be seen that the cost of twinning is slightly less than cost of replacement.

The cost estimates for the stormwater management facilities were prepared based on the following assumptions:

- costs are in 2009 dollars;
- costs include mobilization and demobilization, topsoil stripping, excavation and disposal, landscaping, and an outlet structure;
- wet facility construction allowing for 2.5 m of dead storage, 2.0 m of live storage, and 0.5 m for freeboard;
- costs include 25% for contingency and 10% for engineering; and
- land cost for stormwater management facilities was not included.

Table 5.10 summarizes the cost estimates for the proposed stormwater management facilities and associated outlets for the ultimate 2025 stormwater storage requirements. Detailed cost estimates are located in Table I-2 in Appendix I.

Table 5.10: Stormwater Management Facility Cost Estimates

Stormwater Management Facility	Total Cost ($)
A	2,102,000
B	1,834,000
C	2,200,000
D	2,081,000
E	2,001,000
F	2,555,000
G	1,882,000
H	2,491,000
I	2,124,000
J	3,130,000
K	1,878,000
L	2,572,000
M	755,000
N	2,046,000
O	1,375,000
P	526,000
R	418,000
S	744,000
T	567,000
U	545,000
V	762,000
W	978,000
X	2,941,000
Y	3,118,000
TOTAL	**41,625,000**

5.7 Flood Mapping

Flood mapping for the watercourses through the Town of Edson is not available. Existing reports were requested, however, it appears that no flood mapping studies have been completed for this area.

The Alberta Environment Flood Risk Map Information & Benchmark Retrieval System is a site that contains flood risk map information throughout Alberta. The information available is based on detailed studies produced under the Canada-Alberta Flood Damage Reduction Program.

The site was accessed on November 6, 2009 and no information was presented for the Edson area. A flood mapping study can be conducted in order to determine the expected extent of flooding during a particular design rainfall event. The data could be used to determine where development should and should not occur.

5.8 Infill Development Guidelines

There are several areas within the existing townsite of Edson that are currently proposed for infill development. These areas are shown on Figure 5.9. This development is not addressed in the proposed upgrades presented in this study. Management of the stormwater runoff generated by the infill developments can present challenges, such as meeting the recommended discharge rates, land availability for storage, and the impact to existing systems. Infill development can result in land uses that are more intensive than previous uses and have higher levels of imperviousness, runoff rates, sediment and erosion. Often, areas surrounding the new infill development were built before stormwater controls were required and are already experiencing stormwater management problems, such as in Edson.

In developing a stormwater management plan for infill developments, the following should be considered:

- physical conditions;
- infrastructure capacity;
- increase in percent imperviousness; and
- the opportunity for retrofitting or rehabilitating stormwater management systems.

Standards typically implemented in other municipalities were investigated. Below are several considerations given to stormwater management for infill developments:

No Control
This approach is not often accepted by most municipalities. It is best limited to small, individual lots, as cumulative effects of several infill developments can create problems including flooding. Stormwater treatment, such as oil/grit separators should still be considered for this alternative.

Minimum Runoff Capture
This requires the developer to capture all runoff from a lesser rainfall event and retain it on-site until it infiltrates or evaporates. Consideration can also be given to releasing the captured runoff after the rainfall event, when the downstream system has capacity.

Storage of stormwater runoff on-site of an infill development should consider rooftop, parking lot and superpipe storage rather than surface stormwater management facilities. These storage alternatives limit the land availability required for a surface stormwater management facility.

Conveyance
Conveyance to an existing storm sewer system or construction of new conveyance infrastructure is a possible solution to infill developments. Existing sewer system capacities need to be considered as to not cause flooding.

Off-Site Systems (OSS)
This can involve a stormwater management facility to control the generated runoff at another location downstream of the infill development.

Several infill developments would need to be considered for OSS to be a viable alternative to on-site stormwater management. This can be implemented in combination with minimum runoff capture and conveyance/ end-of-pipe controls. Opportunities to combine some of the SWMFs can be investigated during preliminary and detailed design to reduce O&M costs.

Sustainable Development
Developing the infill development lots in such a way to reduce the stormwater runoff generated. Sustainable methods such as permeable landscaping and green roofs can significantly reduce the runoff generated by a development. Runoff that is generated can be considered for reuse, such as for irrigation purposes on-site.

Edson Assessment
In Edson, if the proposed infill areas are developed with the land use as shown in Figure 2.2, the improved storm sewer system will be adequate to convey the runoff and meet the recommended service level. If the lots are developed with a higher level of impermeable surface than predicted, the excess runoff generated may not be accommodated by the proposed improved system.

The existing storm sewer system is currently surcharging at most locations proposed for infill development. Adding the full flow from the infill lot to the storm sewer system is not practical and would cause additional flooding. However, the infill development areas are small lots (less than 1.0 ha) and it would be difficult to provide a significant amount of on-lot storage and cost prohibitive to provide underground storage. It is therefore recommended that the small infill lots provide storage resulting from an allowable discharge rate of 10 L/s/ha and release the controlled flow to the storm sewer system. The resulting storage volume will be small enough to provide on-lot via parking lot storage and rooftop storage.

Table 5.11 was created to provide a general indication of the storage volume required for infill lots for various design rainfall events and land uses. Volumes were calculated for runoff coefficients between 0.10 and 1.00 for the 5, 25, and 100 year rainfall events with durations of 4 hours and 24 hours, with a constant allowable discharge rate of 10 L/s/ha. Soil infiltration capacities and topography is not factored into the calculations, Table 5.11 provides an approximate storage volume per hectare.

Table 5.11: Per Hectare Storage Volumes with Outflow of 10 L/s/ha (m3/ha)

	5 year		25 year		100 year	
Runoff Coefficient	**4h**	**24h**	**4h**	**24h**	**4h**	**24h**
0.10	0	0	0	0	0	0
0.15	0	0	0	0	8	0
0.20	0	0	17	0	35	0
0.25	13	0	39	0	61	0
0.30	30	0	62	0	88	0
0.35	47	0	84	0	115	0
0.40	64	0	106	0	141	0
0.45	81	0	128	0	168	0
0.50	98	0	151	0	195	11
0.55	115	0	173	0	221	55
0.60	132	0	195	21	248	100
0.65	149	0	217	59	274	144
0.70	166	0	240	97	301	188
0.75	183	16	262	134	328	233
0.80	200	46	284	172	354	277
0.85	217	75	306	210	381	321
0.90	234	105	329	248	408	365
0.95	251	135	351	285	434	410
1.00	268	165	373	323	461	454

It is recommended that similar to the new developments, infill developments should provide storage for the 100 year rainfall event. As shown in Table 5.11, the 4 hour rainfall events are the governing rainfall events. Storage should be provided for the 100 year 4 hour rainfall event, with a discharge of 10 L/s/ha for infill development areas.

Town of Edson
Municipal Servicing Plan Update

Existing System 5 Year 4 Hour Rainfall Event Results
Figure 5.1

AECOM

B SIZE 11" x 17" (279.4mm x 431.8mm)
PLOT: 10/31/2011 10:11:37 AM Saved By:Chaob
UMA FILE NAME: 4193-033-00_03-CWF501_RD.dwg
ISS/REV: D

B SIZE 11" x 17" (279.4mm x 431.8mm)

PLOT: 10/31/2011 10:11:45 AM

Saved By:Chaob

UMA FILE NAME: 4193-033-00_03-CWF502_RD.dwg

ISS/REV:

AECOM

Town of Edson
Municipal Servicing Plan Update

Existing System 5 Year 24 Hour Rainfall Event Results

Figure 5.2

B SIZE 11" x 17" (279.4mm x 431.8mm)

PLOT: 10/31/2011 10:11:52 AM

Saved By:Chaob

UMA FILE NAME: 4193-033-00_03-CWF503_RD.dwg

ISS/REV:

AECOM

Town of Edson
Municipal Servicing Plan Update

Existing System 25 Year 4 Hour Rainfall Event Results

Figure 5.3

B SIZE 11" x 17" (279.4mm x 431.8mm)

PLOT: 10/31/2011 10:12:00 AM Saved By:Chaob

UMA FILE NAME: 4193-033-00_03-CWF504_DX.dwg

ISS/REV:

AECOM

Town of Edson
Municipal Servicing Plan Update

Existing System 25 Year 24 Hour Rainfall Event Results

Figure 5.4

Town of Edson
Municipal Servicing Plan Update

Existing System 100 Year 4 Hour Rainfall Event Results

Figure 5.5

AECOM

ISS/REV:
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B SIZE 11" x 17" (279.4mm x 431.8mm)

PLOT: 10/31/2011 10:12:16 AM   Saved By:Chaob

UMA FILE NAME: 4193-033-00_03-CWF506_RD.dwg

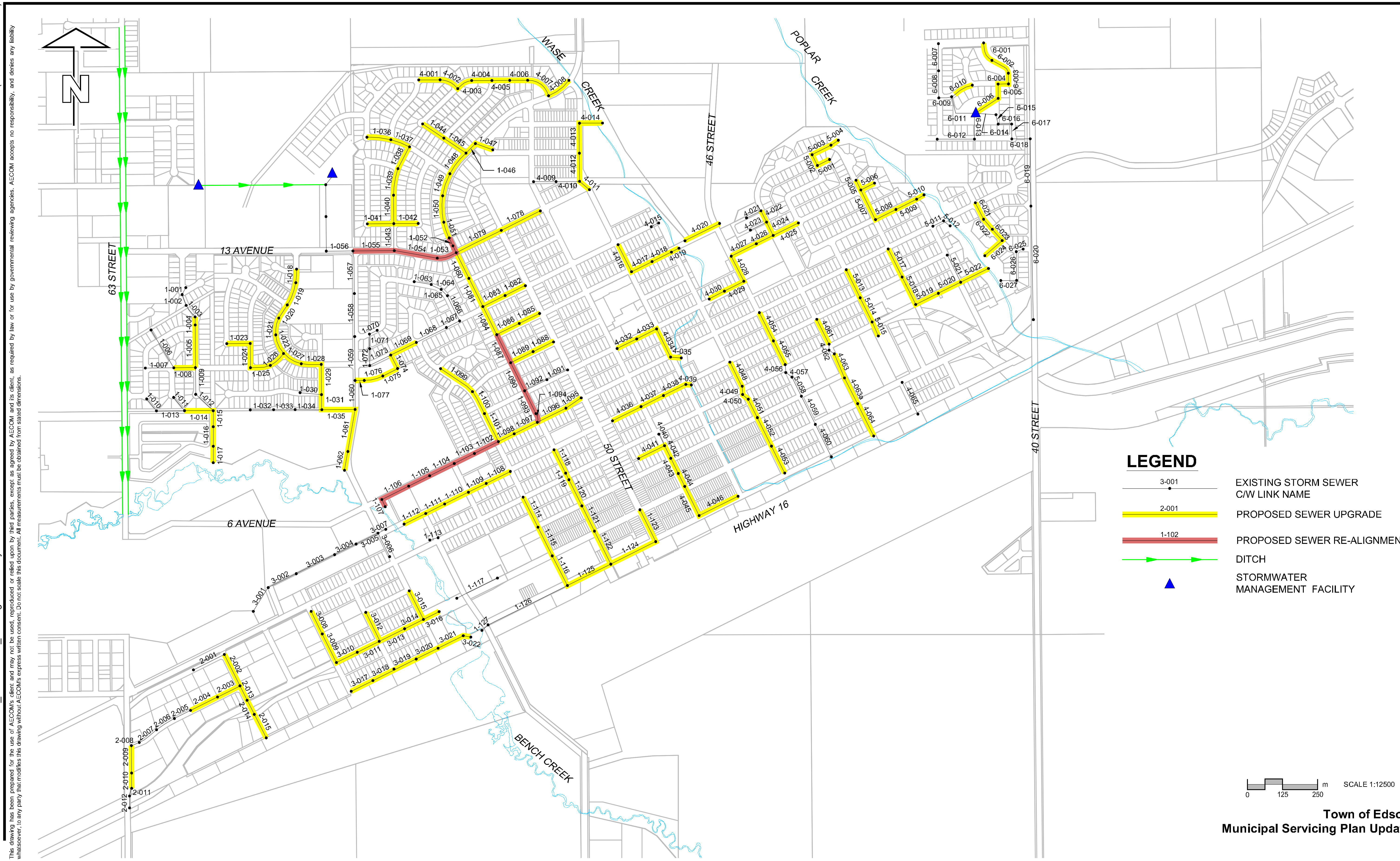
ISS/REV:

AECOM

Town of Edson
Municipal Servicing Plan Update

Existing System 100 Year 24 Hour Rainfall Event Results

Figure 5.6



B SIZE 11" x 17" (279.4mm x 431.8mm)

PLOT: 12/15/2011 10:58:33 AM Saved By:Chaob

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UMA FILE NAME: 4193-033-00_03-CWF507_RE.dwg

AECOM

Proposed Improvements
Figure 5.7

B SIZE 11" x 17" (279.4mm x 431.8mm)

PLOT: 10/31/2011 10:12:32 AM Saved By:Chaob

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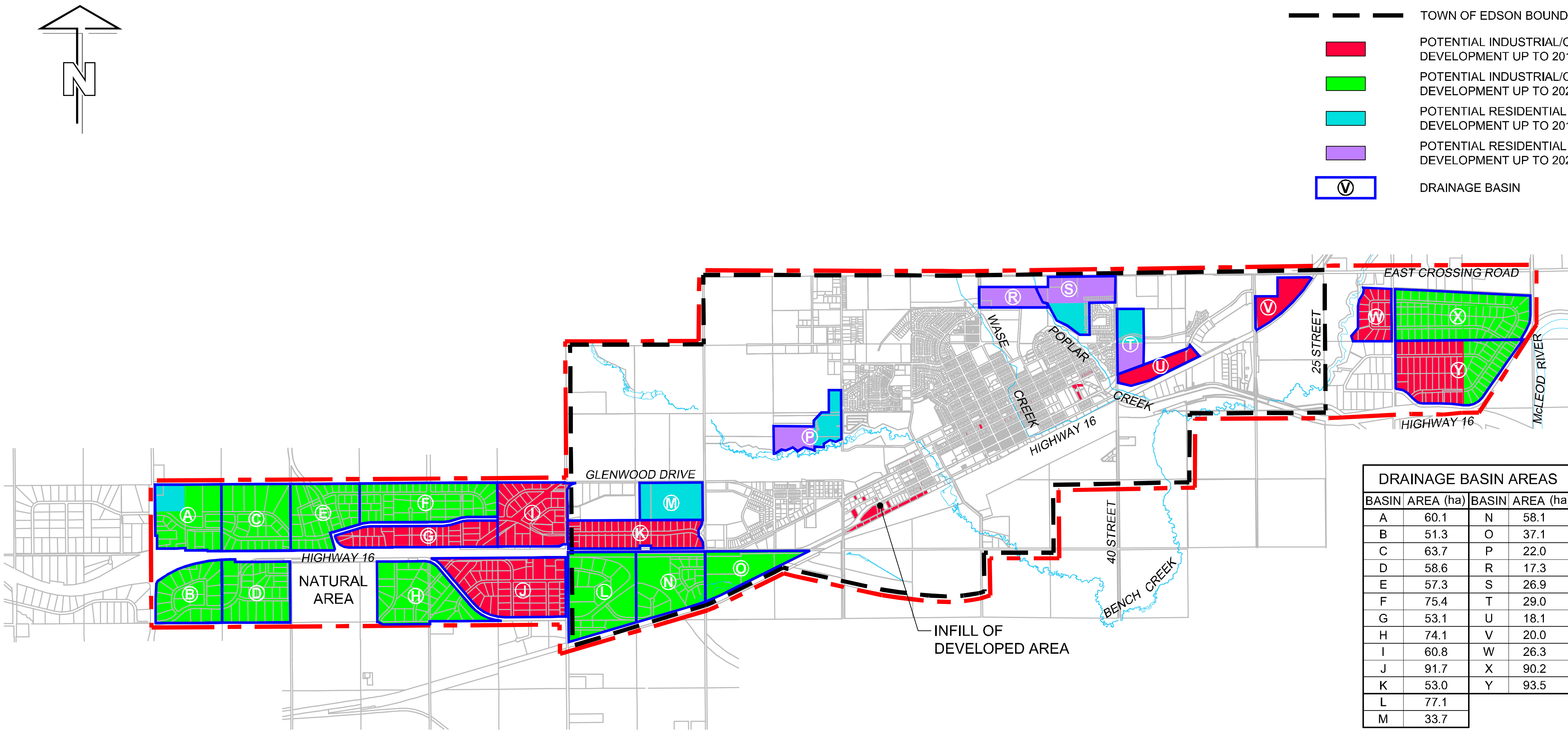
UMA FILE NAME: 4193-033-00_03-CWF508_RD.dwg

Town of Edson
Municipal Servicing Plan Update

Improved System 5 Year 4 Hour Rainfall Event Results

Figure 5.8



LEGEND

- STUDY AREA BOUNDARY
- TOWN OF EDSON BOUNDARY
- POTENTIAL INDUSTRIAL/COMMERCIAL DEVELOPMENT UP TO 2015
- POTENTIAL INDUSTRIAL/COMMERCIAL DEVELOPMENT UP TO 2025
- POTENTIAL RESIDENTIAL DEVELOPMENT UP TO 2015
- POTENTIAL RESIDENTIAL DEVELOPMENT UP TO 2025
- V DRAINAGE BASIN

DRAINAGE BASIN AREAS

BASIN	AREA (ha)	BASIN	AREA (ha)
A	60.1	N	58.1
B	51.3	O	37.1
C	63.7	P	22.0
D	58.6	R	17.3
E	57.3	S	26.9
F	75.4	T	29.0
G	53.1	U	18.1
H	74.1	V	20.0
I	60.8	W	26.3
J	91.7	X	90.2
K	53.0	Y	93.5
L	77.1		
M	33.7		

0 500 1000 m SCALE 1:50000

Town of Edson
Municipal Servicing Plan Update

Future Stormwater Drainage Basins

Figure 5.9

AECOM

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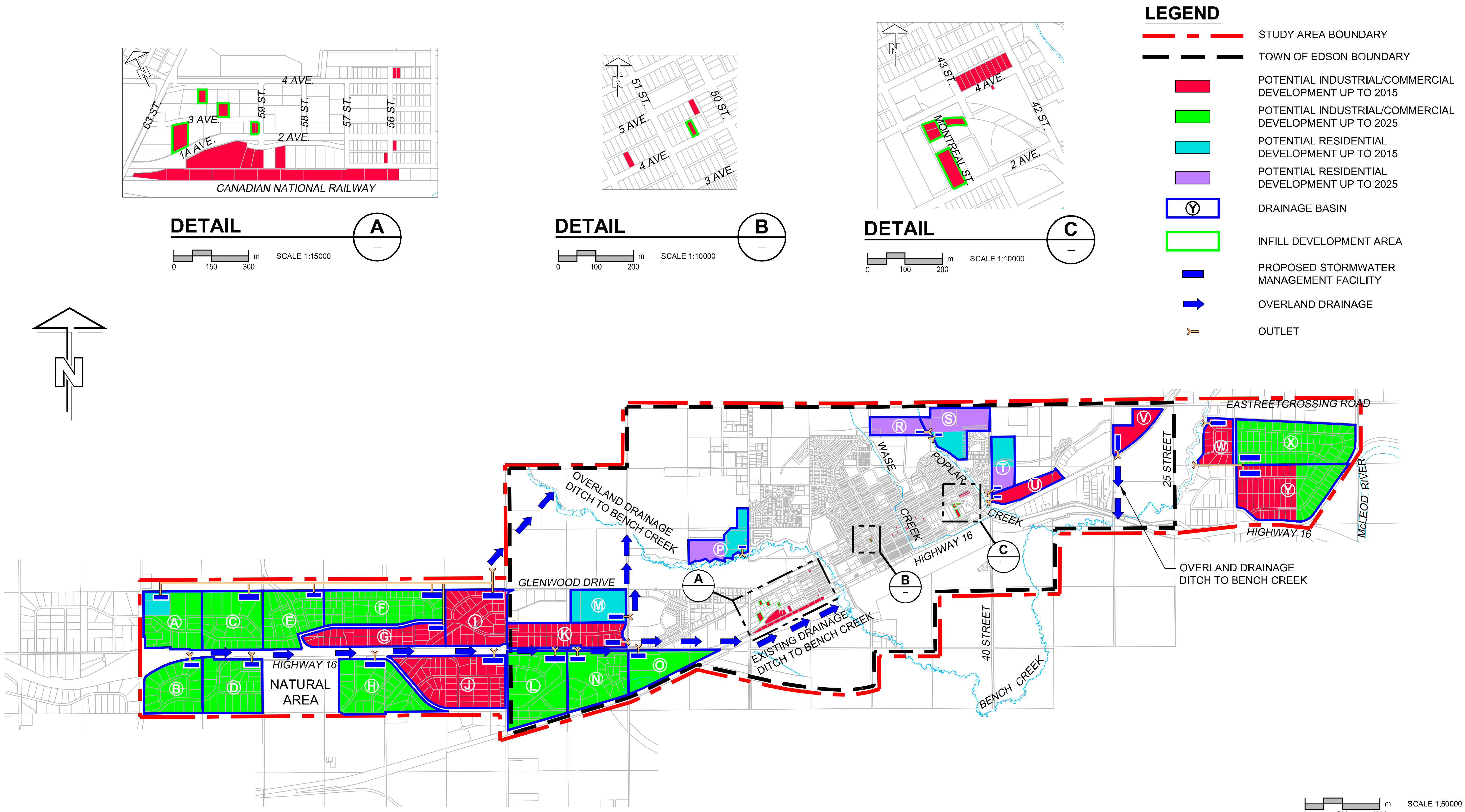

B SIZE 11" x 17" (279.4mm x 431.8mm)

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AECOM

Town of Edson
Municipal Servicing Plan Update

Future Storm Servicing Plan
Figure 5.10

6. Conclusions and Recommendations

6.1 Water Supply and Distribution System

- The Town of Edson water distribution model was updated by adding infrastructure constructed since the completion of the Town of Edson Water Distribution System Analysis (UMA, April 2005) and updating the demands to reflect the 2007 water consumption rates.
- The model was calibrated against hydrant flow test results. It is recommended that a C value of 120 be used for PVC pipes, and 110 be used for all other pipe materials.
- Generally, the existing water distribution system cannot provide fire flows to the existing areas.
- In the northwest area of Town, north of 13 Avenue and between 61 and 63 Street, the pressures are below 280 kPa during peak hour demand.
- To meet the maximum day demand (126 L/s), based on the Town of Edson design standards, all groundwater wells should be utilized, and an additional 25 L/s is required. However, the 2007 measured water use in the Town of Edson was approximately 66 L/s for maximum day demand. The existing groundwater wells have sufficient capacity to provide this flow. It is recommended that additional groundwater wells be considered once the measured maximum day demand approaches the allowed design discharge rate of 101.4 L/s.
- It is recommended that Well No. 3 be brought back into service prior to the installation of additional wells. To provide flows directly to the water distribution system, a pump capable of providing 6.5 L/s at 75 m of head is required.
- For the existing development condition, the reservoirs were evaluated for the Alberta Environment storage guidelines. Based on this requirement, the existing reservoirs (6,530 m^3) are adequate to provide the required storage volume (6,400 m^3).
- The existing system does not have adequate pumping capacity; therefore, it is recommended that a booster station be constructed adjacent to the reservoirs at Grande Prairie Trail, with a pumping capacity of 290 L/s at 45 m of head.
- Pipe upgrades (200 mm to 350 mm in diameter) are recommended to increase the available fire flows and to provide servicing to the areas north of 17 Avenue, between 63 and 66 Street.
- For future development, three alternatives were considered. Alternative 1 is based on the Town of Edson design standards, and Alternative 2 is based on the Yellowhead County design standards. Due to the large peaking factor and higher consumption rate for non-residential areas required by the Yellowhead County standards, the cost of implementing Alternative 2 is approximately double that of Alternative 1. It was determined that the Town of Edson standards should be used for the purpose of the Municipal Servicing Plan; therefore, Alternative 1 was chosen. Alternative 3 was developed for cost comparison purposes, in which only development within the Town of Edson was considered, based on the Town of Edson standards.
- The future maximum day demand requirements for 2015 and 2025 are 263 L/s and 409 L/s, respectively. The current allowable discharge rate from the existing groundwater wells is approximately 101 L/s. Therefore, based on an approximate well discharge of 8.5 L/s, approximately 19 additional wells will be required by 2015, and another 18 additional wells will be required by 2025. Since the projected number of wells is based on the design standards, the actual consumption for the service area should be monitored to determine the number of wells required for supply.
- Since the Town of Edson is fed through groundwater wells, and is not part of a regional system, it is recommended that the Alberta Environment guidelines be used to determine future storage requirements. For 2015 and 2025 (Alternative 1), an additional 9,500 m^3 and 4,100 m^3 of storage capacity are required, respectively. It is recommended that the additional storage capacity be provided at a new reservoir located in the west part of Town.
- For Alternative 1, the 2015 and 2025 pumping requirements are 790 L/s and 915 L/s, respectively. It is recommended that the booster station at Grande Prairie Trail be further upgraded to provide 330 L/s at 45 m of head. The proposed West Reservoir and Pumphouse is recommended to provide 300 L/s at 45 m of head.

- For the 2025 development condition, the pumping head at the West Reservoir and Pumphouse should be increased to 71.5 m. It was assumed that the future groundwater wells will contribute to the overall pumping requirements.
- For Alternative 1, water mains required for future water servicing are generally 250 mm to 350 mm in diameter.
- Six pressure zones are required for Alternative 1, and are recommended to be separated by pressure reducing valves.
- The total cost of Alternative 1 is $81,074,030, including 10% for engineering and 25% for contingency. The costs for Alternative 3 ($40,842,290) have been included for comparison purposes. The cost estimates for groundwater wells, reservoirs, additional pumping, water mains, and pressure reducing valves are summarized in Table 6.1.

Table 6.1: Water Supply and Distribution System - Cost Estimate Summary

Description	Alternative 1	Alternative 3
Groundwater Well Cost	$3,746,250	$1,518,750
Reservoir Cost	$9,640,430	$6,511,300
Pumping Cost	$3,090,350	$3,049,240
Water Main Costs	$64,030,000	$29,196,000
Pressure Reducing Valve Costs	$567,000	$567,000
Total	**$81,074,030**	**$40,842,290**

- The cost for a booster station at the Grande Prairie Trail reservoirs for the existing system is estimated to be $1,240,000, which is included in the total pumping costs shown in Table 6.1.
- To upgrade the existing system, it is recommended that the new booster station at Grande Prairie Trail be constructed first, followed by the 300 mm loop along Highway 16. The local pipe improvements can be completed once replacement is required due to pipe age.

6.2 Wastewater Collection System

- It is recommended that the Town of Edson continue to collect flow data and verify the model calibration on a yearly basis or when a large rainfall event occurs. A rain gauge with the capability of collecting minute to minute rainfall data is also recommended, as Environment Canada only provides hourly rainfall data
- The existing system is sufficient to handle the dry weather flows in the Town. All of the nodes and links are green according to the legends given in Section 4.6. Any issues regarding the sanitary system are a result of wet weather flows.
- Both the 4 and 24 hour durations were run for the 5 year event. For the 5 year 4 hour event, many nodes are surcharged within 1.0 m of the ground level. Major problem areas include the downtown core along 50 Street and 51 Street, 10 Avenue between 52 Street and 56 Street, and the industrial/residential area on the west side of Edson. The area on the west side experiences some out of system flooding for the 4 hour duration. The system does not have adequate capacity to convey the 5 year 4 hour event.
- For the 5 Year 24 hour event, there is some surcharging above the top of pipe in the west area of the town; however these nodes are under 1.0 m below ground and are in a non-residential area. The system has adequate capacity to convey the 5 year 24 hour rainfall event.
- Similarly to the 5 year event, for the 25 year event, the 4 hour duration event is more severe than the 4 hour duration, with many areas flooding within 1.0 m of the ground. Flooding is more widespread in the 25 year 4 hour event than the 5 year 4 hour event. A major bottleneck occurs in the west end where the residential service connects to the rest of the system. The existing system does not have adequate capacity for the 25 year events.

- It is important to note that the majority of the sanitary sewer problems experienced in the town are due to wet weather flows. As such, it is recommended that weeping tile be disconnected from the sanitary system whenever possible. Roof leaders, catch basins and storm drains connected to the sanitary sewer, deteriorated manhole barrels and manholes located in sags are other sources of infiltration and inflow and should be addressed as part of the Town's street improvement and maintenance programs. Backflow preventer valves are also a measure the Town can take to reduce the risk of basement flooding.
- Three phases of improvements are recommended to decrease the risk of basement flooding.
- It is recommended that Phase 1 improvements are implemented first followed by Phase 2 and Phase 3 improvements. Generally, upgrades can be prioritized from downstream to upstream (east to west). However, they should be completed, where possible, as part of the street improvement program or other proposed underground projects to minimize the excavation and restoration costs as well as disruption.
- For the Phase 1 improvements there were three alternatives presented to alleviate the surcharging in the west part of the Town. A new pipe along a new alignment is recommended as it will be required for future development in 2015 and 2025. As the pipe will cross Highway 16 it is recommended that it be sized for future development, therefore a 750 mm pipe should be installed.
- The total cost for Phases 1, 2 and 3 are estimated to be $11.8M, $5.2M and $0.6M respectively.
- The existing system with the proposed upgrades is adequate for the addition of 2015 and 2025 residential areas to the northeast and northwest portions of the Town. For the west portion of the Town a proposed new trunk line servicing the industrial areas in the west of Town will need to be upsized to accommodate the new areas to the west. The estimated cost is $4.6M and is included in the Phase 1 estimate.

6.3 Stormwater Management System

- The existing system was assessed to examine the system performance for the 5, 25, and 100 year short duration (4 hour) and long duration (24 hour) rainfall events.
- During the 5 year 4 hour event, the existing system experiences a large amount of surface flooding. The parts of the system not flooding have high surcharge levels. Overall, the existing sewer system does not have adequate capacity for the 5 year 4 hour rainfall event.
- The system performs significantly better during the 5 year 24 hour rainfall event. In general, the system has adequate capacity to convey the 5 year 24 hour rainfall event.
- Flooding and surcharging in the system increases during the 25 year and 100 year rainfall events. The 4 hour duration events continue to cause the system to flood and operate under surcharged conditions. The 24 hour duration events generally have capacity to convey the runoff; however, flooding occurs at one location during the 25 year event and at several locations during the 100 year event.
- The Town of Edson does not have documented Engineering Design Standards for stormwater drainage systems. The Town of Edson could consider developing Engineering Design Standards for stormwater drainage systems. For the proposed existing system improvements, a level of service such that there is not surcharging within 1.0 m of ground for a 5 year 4 hour rainfall event will be adopted.
- There are not many areas that would effectively provide storage within the existing developed areas of Edson, therefore, the proposed improvements consider pipe upgrades.
- Pipe upgrades were determined for both replacement and twinning options. The decision to twin or replace pipes will be based on the condition of the existing pipes.
- Once the storm sewer upgrades are implemented, the majority of the system does not have any surcharging during the 5 year 4 hour rainfall. Some surcharging still exists; however, it is localized and does not result in the HGL being within 1.0 m of the ground.
- A stormwater management plan was developed for the Town of Edson based on 2015 and 2025 development. The future stormwater management plan is not dependant on the existing system upgrades in Section 5.4.
- The future development areas were delineated into 24 storm drainage basins. Each of the proposed drainage basins will be graded such that the runoff is routed to a stormwater management facility.

- The future SWMFs will be designed to service the 100 year rainfall event while discharging at the allowable discharge rate.
- Allowable discharge rates for the Bench, Wase, and Poplar Creeks were established in the Town of Edson Stormwater Management Plan, completed by UMA Engineering in 2005.
- It is proposed that the SWMF's be designed to be wet facilities to allow for sediments to settle out of the runoff and therefore enhance the water quality before being released. Alberta Environment requires that a minimum of 85% of sediments with a particle size of 75 µm or greater be removed from the runoff.
- The results of the model simulation showed that there were two governing rainfall events for the proposed SWMFs. The 4 hour duration rainfall event is the critical event for SWMFs that have residential development and discharge to Poplar Creek. All other SWMFs are designed for the 100 year 24 hour rainfall.
- The total cost for the storm sewer improvements is approximately $24.5 million.
- The total cost for construction of the future SMWFs is approximately $41.6 million.
- Flood mapping for the watercourses through the Town of Edson is not available. A flood mapping study can be conducted to determine the extent of flooding during the design rainfall events, and thus determine where development should and should not occur.
- In developing a stormwater management plan for infill developments physical conditions, infrastructure capacity, increase in percent imperviousness, and the opportunity for retrofitting or rehabilitating stormwater management systems should be considered.
- Servicing of infill developments can be achieved through:
    - No Control - this is best limited to small, individual lots, as cumulative effects of several infill developments can create problems including flooding.
    - Minimum Runoff Capture -this requires the developer to capture all runoff from a lesser rainfall event and retain it on-site until it infiltrates, evaporates, or consideration can be given to releasing the runoff after the rainfall event.
    - Conveyance - to an existing storm sewer system or construction of new conveyance infrastructure.
    - Off-Site Systems - this can involve a stormwater management facility to control the generated runoff at another location downstream of the infill development.
    - Sustainable Development - sustainable methods such as permeable landscaping and green roofs can significantly reduce the runoff generated by a development.
- The proposed improvements to the storm sewer system will be adequate to convey the runoff and meet the recommended service level for the proposed infill developments.
- The existing storm sewer system is currently surcharging at most locations proposed for infill development.
- The small lot sizes (less than 1.0 ha) for the infill develop areas would be difficult to provide a significant amount of on-lot storage and cost prohibitive to provide underground storage.
- Storage should be provided for the 100 year 4 hour rainfall event, with a discharge of 10 L/s/ha for infill development areas.

Appendix A

Hydrant Test Results

EPCOR Water Services Inc

IVARA - Production
Epcor Water Services Inc (EWSI)

Work Order / Task Id 038046-1

HYDRANT FLOW TEST

Test Date & Time: 9/23/2008 1:00:00 PM

Location: EDSON AB
Cadastral No.:
Residential
Main Size:
Neighborhood: 59 ST - 4 AVE
Stage:
Project:
For: UMA Engineering **Attn.:** Christin **Ph. No:** 486-7060 **Fax. No.:** 486-7070
Diffuser Used: POLLARD

Test with one 2.5 inch nozzle flowing

Coefficient:	0.9	**Static Pressure (PSI):**	65
Static Hydrant:	**Private No.:** 1	**Residual Pressure (PSI):**	58
Flow Test Hydrant No.:	**Private No.:**	**Pitot Gauge Reading(1) - Pressure (psi):**	55
Flow Rate (Litres / sec):	63.36	**(U.S. gpm):**	1,004.28
Flow Rate@20psi (Litres / sec):	173.06	**(U.S. gpm):**	2,743.07

Test with two 2.5 inch nozzle flowing

Coefficient:	0.9	**Static Pressure (PSI):**	65
Static Hydrant:	**Private No.:** 1	**Residual Pressure (PSI):**	49
Flow Test Hydrant No.:	**Private No.:**	**Pitot Gauge Reading(1) - Pressure (psi):**	32
		Pitot Gauge Reading(2) - Pressure (psi):	33
Flow Rate (Litres / sec):	102.69	**(U.S. gpm):**	1,627.61
Flow Rate@20psi (Litres / sec):	179.48	**(U.S. gpm):**	2,844.85

Test with three or four 2.5 inch nozzle flowing

Coefficient:	0.9	**Static Pressure (PSI):**	65
Static Hydrant:	**Private No.:** 1	**Residual Pressure (PSI):**	
Flow Test Hydrant No.:	**Private No.:**	**Pitot Gauge Reading(1) - Pressure (psi):**	
Flow Test Hydrant No.:	**Private No.:**	**Pitot Gauge Reading(2) - Pressure (psi):**	
		Pitot Gauge Reading(3) - Pressure (psi):	
		Pitot Gauge Reading(4) - Pressure (psi):	
Flow Rate (Litres / sec):		**(U.S. gpm):**	
Flow Rate@20psi (Litres / sec):		**(U.S. gpm):**	

Remarks:

Test By: ZELMER, RAYMOND **Approved By:** BELITSKY, DEBRAH

DISCLAIMER

EPCOR Water Services Inc

IVARA - Production
Epcor Water Services Inc (EWSI)

Work Order / Task Id 038046-1

HYDRANT FLOW TEST

Test Date & Time: 9/23/2008 2:00:00 PM

Location: EDSON AB
Cadastral No.:

Residential
Main Size:

Neighborhood: 47 ST - 2 AVE
Stage:
Project:

For: UMA Engineering **Attn.:** Christin **Ph. No:** 486-7060 **Fax. No.:** 486-7070

Diffuser Used: POLLARD

Test with one 2.5 inch nozzle flowing

Coefficient:	0.9	**Static Pressure (PSI):**	83
Static Hydrant:	**Private No.:** 2	**Residual Pressure (PSI):**	78
Flow Test Hydrant No.:	**Private No.:**	**Pitot Gauge Reading(1) - Pressure (psi):**	69
Flow Rate (Litres / sec):	69.37	**(U.S. gpm):**	1,099.56
Flow Rate@20psi (Litres / sec):	272.51	**(U.S. gpm):**	4,319.37

Test with two 2.5 inch nozzle flowing

Coefficient:	0.9	**Static Pressure (PSI):**	83
Static Hydrant:	**Private No.:** 2	**Residual Pressure (PSI):**	72
Flow Test Hydrant No.:	**Private No.:**	**Pitot Gauge Reading(1) - Pressure (psi):**	45
		Pitot Gauge Reading(2) - Pressure (psi):	47
Flow Rate (Litres / sec):	117.98	**(U.S. gpm):**	1,870.00
Flow Rate@20psi (Litres / sec):	302.76	**(U.S. gpm):**	4,798.81

Test with three or four 2.5 inch nozzle flowing

Coefficient:	0.9	**Static Pressure (PSI):**	83
Static Hydrant:	**Private No.:** 2	**Residual Pressure (PSI):**	
Flow Test Hydrant No.:	**Private No.:**	**Pitot Gauge Reading(1) - Pressure (psi):**	
Flow Test Hydrant No.:	**Private No.:**	**Pitot Gauge Reading(2) - Pressure (psi):**	
		Pitot Gauge Reading(3) - Pressure (psi):	
		Pitot Gauge Reading(4) - Pressure (psi):	
Flow Rate (Litres / sec):		**(U.S. gpm):**	
Flow Rate@20psi (Litres / sec):		**(U.S. gpm):**	

Remarks:

Test By: ZELMER, RAYMOND **Approved By:** BELITSKY, DEBRAH

DISCLAIMER

The information contained within this Hydrant Flow Test form is reliable as at the test date any may be used for the limited purpose of verifying water flows and pressure at the subject location. Variations due to system configuration changes after the test date, beyond the control of EPCOR Water Services, would affect system performance. The test information may become unreliable as a result of such changes, at any time, without notice. EPCOR Water services makes no further representations or warranties, express or implied, in relation to the test information contained herein.

EPCOR

EPCOR Water Services Inc

IVARA - Production
Epcor Water Services Inc (EWSI)

Work Order / Task Id 038046-1

HYDRANT FLOW TEST

Test Date & Time: 9/23/2008 11:00:00 AM

Location: EDSON AB

Cadastral No.:

Residential

Main Size:

Neighborhood: 49 ST - EDSON DR

Stage:

Project:

For: UMA Engineering Attn.: Christin Ph. No: 486-7060 Fax. No.: 486-7070

Diffuser Used: POLLARD

Test with one 2.5 inch nozzle flowing

Coefficient: 0.9 Static Pressure (PSI): 79

Static Hydrant: Private No.: 3 Residual Pressure (PSI): 42

Flow Test Hydrant No.: Private No.: Pitot Gauge Reading(1) - Pressure (psi): 39

Flow Rate (Litres / sec): 55.23 (U.S. gpm): 875.35

Flow Rate@20psi (Litres / sec): 71.05 (U.S. gpm): 1,126.19

Test with two 2.5 inch nozzle flowing

Coefficient: 0.9 Static Pressure (PSI): 79

Static Hydrant: Private No.: 3 Residual Pressure (PSI): 31

Flow Test Hydrant No.: Private No.: Pitot Gauge Reading(1) - Pressure (psi): 20

Pitot Gauge Reading(2) - Pressure (psi): 20

Flow Rate (Litres / sec): 84.58 (U.S. gpm): 1,340.55

Flow Rate@20psi (Litres / sec): 94.54 (U.S. gpm): 1,498.56

Test with three or four 2.5 inch nozzle flowing

Coefficient: 0.9 Static Pressure (PSI): 79

Static Hydrant: Private No.: 3 Residual Pressure (PSI):

Flow Test Hydrant No.: Private No.: Pitot Gauge Reading(1) - Pressure (psi):

Flow Test Hydrant No.: Private No.: Pitot Gauge Reading(2) - Pressure (psi):

Pitot Gauge Reading(3) - Pressure (psi):

Pitot Gauge Reading(4) - Pressure (psi):

Flow Rate (Litres / sec): (U.S. gpm):

Flow Rate@20psi (Litres / sec): (U.S. gpm):

Remarks:

Test By: ZELMER, RAYMOND Approved By: BELITSKY, DEBRAH

DISCLAIMER

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EPCOR

EPCOR Water Services Inc

IVARA - Production
Epcor Water Services Inc (EWSI)

Work Order / Task Id 038046-1

HYDRANT FLOW TEST

Test Date & Time: 9/23/2008 10:00:00 AM

Location: EDSON AB

Cadastral No.:

Residential

Main Size:

Neighborhood: 26 ST - 2 AVE

Stage:

Project:

For: UMA Engineering **Attn.:** Christin **Ph. No:** 486-7060 **Fax. No.:** 486-7070

Diffuser Used: POLLARD

Test with one 2.5 inch nozzle flowing — **Coefficient:** 0.9 — **Static Pressure (PSI):** 95

Static Hydrant: **Private No.:** 4 — **Residual Pressure (PSI):** 84

Flow Test Hydrant No.: **Private No.:** — **Pitot Gauge Reading(1) - Pressure (psi):** 69

Flow Rate (Litres / sec): 69.37 — **(U.S. gpm):** 1,099.56

Flow Rate@20psi (Litres / sec): 195.60 — **(U.S. gpm):** 3,100.28

Test with two 2.5 inch nozzle flowing — **Coefficient:** 0.9 — **Static Pressure (PSI):** 95

Static Hydrant: **Private No.:** 4 — **Residual Pressure (PSI):** 68

Flow Test Hydrant No.: **Private No.:** — **Pitot Gauge Reading(1) - Pressure (psi):** 40

Pitot Gauge Reading(2) - Pressure (psi): 40

Flow Rate (Litres / sec): 111.58 — **(U.S. gpm):** 1,768.50

Flow Rate@20psi (Litres / sec): 193.72 — **(U.S. gpm):** 3,070.45

Test with three or four 2.5 inch nozzle flowing — **Coefficient:** 0.9 — **Static Pressure (PSI):** 95

Static Hydrant: **Private No.:** 4 — **Residual Pressure (PSI):**

Flow Test Hydrant No.: **Private No.:** — **Pitot Gauge Reading(1) - Pressure (psi):**

Flow Test Hydrant No.: **Private No.:** — **Pitot Gauge Reading(2) - Pressure (psi):**

Pitot Gauge Reading(3) - Pressure (psi):

Pitot Gauge Reading(4) - Pressure (psi):

Flow Rate (Litres / sec): — **(U.S. gpm):**

Flow Rate@20psi (Litres / sec): — **(U.S. gpm):**

Remarks:

Test By: ZELMER, RAYMOND **Approved By:** BELITSKY, DEBRAH

DISCLAIMER

The information contained within this Hydrant Flow Test form is reliable as at the test date any may be used for the limited purpose of verifying water flows and pressure at the subject location. Variations due to system configuration changes after the test date, beyond the control of EPCOR Water Services, would affect system performance. The test information may become unreliable as a result of such changes, at any time, without notice. EPCOR Water services makes no further representations or warranties, express or implied, in relation to the test information contained herein.

EPCOR

EPCOR Water Services Inc

IVARA - Production
Epcor Water Services Inc (EWSI)

Work Order / Task Id 038046-1

HYDRANT FLOW TEST

Test Date & Time: 9/23/2008 12:00:00 PM

Location: EDSON AB
Cadastral No.:
Residential
Main Size:
Neighborhood: 49 ST - 10 AVE
Stage:
Project:
For: UMA Engineering **Attn.:** Christin
Ph. No: 486-7060
Fax. No.: 486-7070
Diffuser Used: POLLARD

Test with one 2.5 inch nozzle flowing

Coefficient:	0.9	**Static Pressure (PSI):**	48
Static Hydrant: **Private No.:**	5	**Residual Pressure (PSI):**	45
Flow Test Hydrant No.: **Private No.:**		**Pitot Gauge Reading(1) - Pressure (psi):**	42
Flow Rate (Litres / sec):	56.89	**(U.S. gpm):**	901.66
Flow Rate@20psi (Litres / sec):	190.03	**(U.S. gpm):**	3,012.07

Test with two 2.5 inch nozzle flowing

Coefficient:	0.9	**Static Pressure (PSI):**	48
Static Hydrant: **Private No.:**	5	**Residual Pressure (PSI):**	42
Flow Test Hydrant No.: **Private No.:**		**Pitot Gauge Reading(1) - Pressure (psi):**	28
		Pitot Gauge Reading(2) - Pressure (psi):	29
Flow Rate (Litres / sec):	97.43	**(U.S. gpm):**	1,544.36
Flow Rate@20psi (Litres / sec):	223.86	**(U.S. gpm):**	3,548.22

Test with three or four 2.5 inch nozzle flowing

Coefficient:	0.9	**Static Pressure (PSI):**	48
Static Hydrant: **Private No.:**	5	**Residual Pressure (PSI):**	
Flow Test Hydrant No.: **Private No.:**		**Pitot Gauge Reading(1) - Pressure (psi):**	
Flow Test Hydrant No.: **Private No.:**		**Pitot Gauge Reading(2) - Pressure (psi):**	
		Pitot Gauge Reading(3) - Pressure (psi):	
		Pitot Gauge Reading(4) - Pressure (psi):	
Flow Rate (Litres / sec):		**(U.S. gpm):**	
Flow Rate@20psi (Litres / sec):		**(U.S. gpm):**	

Remarks:

Test By: ZELMER, RAYMOND **Approved By:** BELITSKY, DEBRAH

DISCLAIMER

Appendix B

Water Network Model Simulation Results (On CD)

Appendix C

Water Distribution System Alternative 2

1. Alternative 2 – Design with Yellowhead County Standards

1.1 Design Criteria

Water Consumption Rates

For Alternative 2, the design rates for Yellowhead County were considered. Yellowhead County standards specify a residential water consumption rate of 375 L/capita/day with peaking factors of 2.0 and 4.0 for maximum day and peak hour demands. For non-residential areas, the standards specify a rate of 0.2 L/s/ha with a peaking factor of [10 x (flow rate)^-0.45], to a maximum of 25 and a minimum of 2.5.

Using the Yellowhead County standards for non-residential areas, the peaking factor was applied to each development area, and therefore varies for each basin. When applied to the existing areas within the Town, the average peaking factor is approximately 17. This corresponds to a maximum day demand and peak hour demand of approximately 294,000 L/ha/day. For the future development area, the average peaking factor is approximately 7, corresponding to a maximum day demand and peak hour demand of approximately 121,000 L/ha/day. These peaking factors are very large and do not accurately represent the demands of the non-residential areas.

Fire Flows

The fire flow rates specified in the Yellowhead County standards are as follows:

- Single Family Residential 100 L/s
- Multiple Family Residential 180 L/s
- Institutional Areas (i.e. Schools) 130 L/s
- Industrial and Commercial Areas 300 L/s

It should be noted that these fire flow rates were used for future development for all alternatives.

Pressure Requirements

Yellowhead County standards indicate that a minimum pressure of 300 kPa be maintained during peak hour demand. The maximum system pressure typically should not exceed 700 kPa.

A minimum residual pressure of 140 kPa is required at ground level during maximum day plus fire flow demand at all locations in the system.

1.2 Future Servicing

The future development scenarios for 2015 and 2025 were analyzed assuming all recommended improvements to the existing system have been implemented. For the future water servicing, areas within the Town of Edson and the Urban Fringe Area within Yellowhead County were considered. Similarly to Alternatives 1 and 3, the growth projections indicated in Table 2.1 were used for this analysis.

1.2.1 Supply System

The future maximum day demand requirements for 2015 and 2025 are 1,272 L/s and 1,853 L/s, respectively. The current allowable discharge rate from the existing groundwater wells is approximately 101 L/s.

Therefore, the existing supply system is approximately 1,171 L/s and 1,752 L/s short to supply the 2015 and 2025 systems. Based on an approximately well discharge of 8.5 L/s, approximately 140 additional wells will be required by 2015, and 70 additional wells will be required by 2025.

Since the projected number of wells is based on the design standards, the actual consumption for the service area should be monitored to determine the number of wells required for supply.

1.2.2 Storage Reservoirs

As mentioned in Section 3.9.1.2, two options were considered for determining future storage volumes. In Option 1, the Alberta Environment Requirement of 25% of Maximum Day Demand (Equalization Storage) plus 15% of Average Day Demand (Emergency Storage) plus Fire Flow was considered. In this option, the equalization storage is assigned to meet the daily demand fluctuation above the supply rates, as the water supply rate is generally lower than the peak water consumption rate. The emergency storage is allocated for the routine disruption of supply for maintenance.

In Option 2, two times Average Day Demand (Supply Interruption) plus fire storage was considered for the storage volume. The supply interruption storage represents the available storage in case of a disruption to the water supply. Tables C.1 and C.2 summarize the 2015 storage requirements for the two options, and Tables C.3 and C.4 summarize the 2025 storage requirements for the two options.

2015 Development Condition

Table C.1: Option 1 Storage Requirement – Alternative 2 2015 Development Condition

Description	Required Volume (m^3)
Fire Storage (300 L/s and 300 L/s for 4 hours)	8,640
Equalization Storage - 25% of Maximum Day Demand (1,272 L/s)	27,475
Emergency Storage - 15% of Average Day Demand (187 L/s)	2,424
Total Required Storage	38,539

Table C.2: Option 2 Storage Requirement – Alternative 2 2015 Development Condition

Description	Required Volume (m^3)
Fire Storage : (300 L/s and 300 L/s for 4 hours)	8,640
Two times Average Day Demand (187 L/s)	32,314
Total Required Storage	40,954

2025 Development Condition

Table C.3: Option 1 Storage Requirement – Alternative 2 2025 Development Condition

Description	Required Volume (m^3)
Fire Storage (300 L/s and 300 L/s for 4 hours)	8,640
Equalization Storage - 25% of Maximum Day Demand (1853 L/s)	40,025
Emergency Storage - 15% of Average Day Demand (291 L/s)	3,772
Total Required Storage	52,437

Table C.4: Option 2 Storage Requirement – Alternative 2 - 2025 Development Condition

Description	Required Volume (m^3)
Fire Storage : (300 L/s and 300 L/s for 4 hours)	8,640
Two times Average Day Demand (291 L/s)	50,285
Total Required Storage	58,925

It is recommended that the Alberta Environment guidelines, Option 1, be used to determine storage requirements, providing the groundwater wells are capable of supplying the maximum day demand. As development occurs, it is recommended that a sufficient number of groundwater wells be kept in production such that maximum day demand can always be supplied to the system.

1.2.3 Pumphouse Facilities

The future pumping requirements for the Town of Edson are based on the projected growth indicated in Section 2.4. Table C.5 summarizes the future pumping requirements for Alternative 2.

Table C.5: Future Pumping Requirements – Alternative 2

Demand Scenario	Future (L/s)	
	2015	2025
Average Day Demand	187	291
Maximum Day Demand	1,272	1,853
Peak Hour Demand	1,327	1,928
Fire Flow*	600	600
Maximum Day Demand Plus Fire Flow	1872	2453

**300 L/s each at Grande Prairie Trail and West Reservoir and Pumphouse.*

It should be noted that although the system is designed for a single fire flow of 300 L/s, both the proposed booster station at Grande Prairie Trail and the proposed West Reservoir and Pumphouse must be capable of providing this flow.

1.2.4 Water Distribution System

The pipe sizes required to service the future development, as well as the schematic pipe layouts, are indicated on Figures C.1 to C.3 for 2015, and Figures C.4 to C.6 for 2025.

By 2015 additional upgrades are required for the existing system, including a 750 mm diameter water main through the Town. The majority of pipes required for the 2025 development condition are either 600 mm or 750 mm in diameter.

1.2.5 Pressure Zones

Six pressure zones will be required for the 2025 system due to the large difference in elevations across the study area. These pressure zones are indicated in Figure 3.30.

To maintain the separate pressure zones, the recommended pressure settings for the existing and proposed pressure reducing valves are summarized in Table C.6.

Table C.6: PRV Settings - Alternative 2 (2025)

PRV	Required Size (mm)	Pressure Setting (kPa)	Maximum Upstream Pressure (kPa)	Maximum Downstream Pressure (kPa)	Required Flow (L/s)
PRV-17	450	600	670	600	0 - 590
PRV-16	450	400	580	400	0 - 1000
PRV-14	400	600	625	600	0 - 900
PRV-19	350	250	560	600	0 - 710
PRV-20	350	350	545	450	0 - 230
PRV-21	500	500	665	500	0 - 500
PRV-22	300	300	700	300	0 - 210

1.3 Cost Estimates

The costs for the improvements are summarized in Tables C.7 through C.12. Costs are based on 2009 dollars, and include a 10% allowance for engineering and 25% for contingency. Detailed cost breakdowns are provided in Appendix D.

The costs for additional groundwater wells are summarized in Table C.7, for both Alternatives 1 and 3, based on a unit cost of $75,000/well.

Table C.7: Cost Estimates – Groundwater Wells

Description	Alternative 2
Existing Allowable Discharge (L/s)	101
Capacity Required-2015 (L/s)	1,272
Deficiency-2015 (L/s)	1,171
Additional Wells Required-2015	140
Cost (2015)	**$10,500,000**
Capacity Required-2025 (L/s)	1,853
Deficiency-2025 (L/s)	581
Additional Wells Required-2025	70
Cost (2025)	**$5,250,000**
Sub-Total	**$15,750,000**
Engineering (10%)	$1,575,000
Contingency (25%)	$3,937,500
Total	**$21,262,500**

The reservoir costs for Alternatives 1 and 2 are indicated in Table C.8. The reservoir costs are based on a unit cost of $525/m^3 for additional storage; details are provided in Tables D.1 and D.2 in Appendix D.

Table C.8: Cost Estimates – Reservoirs

Description	Alternative 2
Available Storage (m^3)	6,530
Additional Storage Required-2015 (m^3)	32,009
Cost (2015)	$16,804,725
Additional Storage Required-2025 (m^3)	13,898
Cost (2025)	$7,296,450
Sub-Total	$24,101,175
Engineering (10%)	$2,410,118
Contingency (25%)	$6,025,300
Total	**$32,536,600**

The 2015 and 2025 pumping costs for Alternative 2 are indicated in Table C.9. The pumping costs are based on a unit rate of $4600/HP, which includes the cost of a new building. The costs also include the necessary back up pumps. Details are provided in Tables D.3 to D.5 in Appendix D.

Table C.9: Cost Estimates – Pumping

Description	Alternative 2
Existing Pumping Capacity (L/s)	100
Additional Pumping Capacity Required-2015 (L/s)	1,772
Cost (2015)	$7,711,687
Additional Pumping Capacity Required -2025 (L/s)	581
Cost (2025)	$2,528,493
Sub-Total	$10,240,181
Engineering (10%)	$1,024,018
Contingency (25%)	$2,560,050
Total	**$13,824,250**

For the existing system improvements, it is recommended that a pumphouse be constructed adjacent to the reservoirs at Grande Prairie Trail to boost the pressure from the reservoir. Approximately 300 L/s would be required for the existing system, corresponding to a cost of $1,240,000, which is included in the pumping costs shown in Table C.9.

The total costs for the proposed water main improvements are summarized in Table C.10. The improvement costs include the pipe cost, as well as the installation and restoration costs.

Table C.10: Cost Estimates – Water Main Improvements – Alternative 2

Pipe Diameter (mm)	Total Length (m)	Unit Cost ($/m)	Pipe Cost ($)	Restoration Cost ($)	Total Cost ($)
Existing System Improvements					
200	569	333	$189,477	$252,636	$442,113
300	4,195	520	$2,181,400	$1,862,580	$4,043,980
350	298	630	$187,740	$132,312	$320,052
				Sub-Total (Existing with Improvements)	**$4,806,145**
2015					
200	757	333	$252,081		$252,081
250	3,435	425	$1,459,875		$1,459,875
300	11,892	520	$6,183,840		$6,183,840
350	12,910	630	$8,133,300		$8,133,300
400	1,281	750	$960,750		$960,750
450	485	850	$412,250		$412,250
500	3,665	1000	$3,665,000		$3,665,000
600	5,625	1500	$8,437,500		$8,437,500
750	11,856	2000	$23,712,000	$1,512,708	$25,224,708
				Sub-Total (2015)	**$54,729,304**
2025					
200	523	333	$174,159		$174,159
250	5,695	425	$2,420,375		$2,420,375
300	11,490	520	$5,974,800		$5,974,800
350	2,670	630	$1,682,100		$1,682,100
400	2,199	750	$1,649,250		$1,649,250
600	4,095	1500	$6,142,500		$6,142,500
				Sub-Total (2025)	**$18,043,184**
				Sub-Total	$77,578,633
				Engineering (10%)	$7,757,863
				Contingency (25%)	$19,394,658
				Total	**$104,732,000**

The cost estimates for the proposed pressure reducing valves are summarized in Table C.11. The cost estimates include the valves, chambers and installation.

Table C.11: Cost Estimates – Pressure Reducing Valves

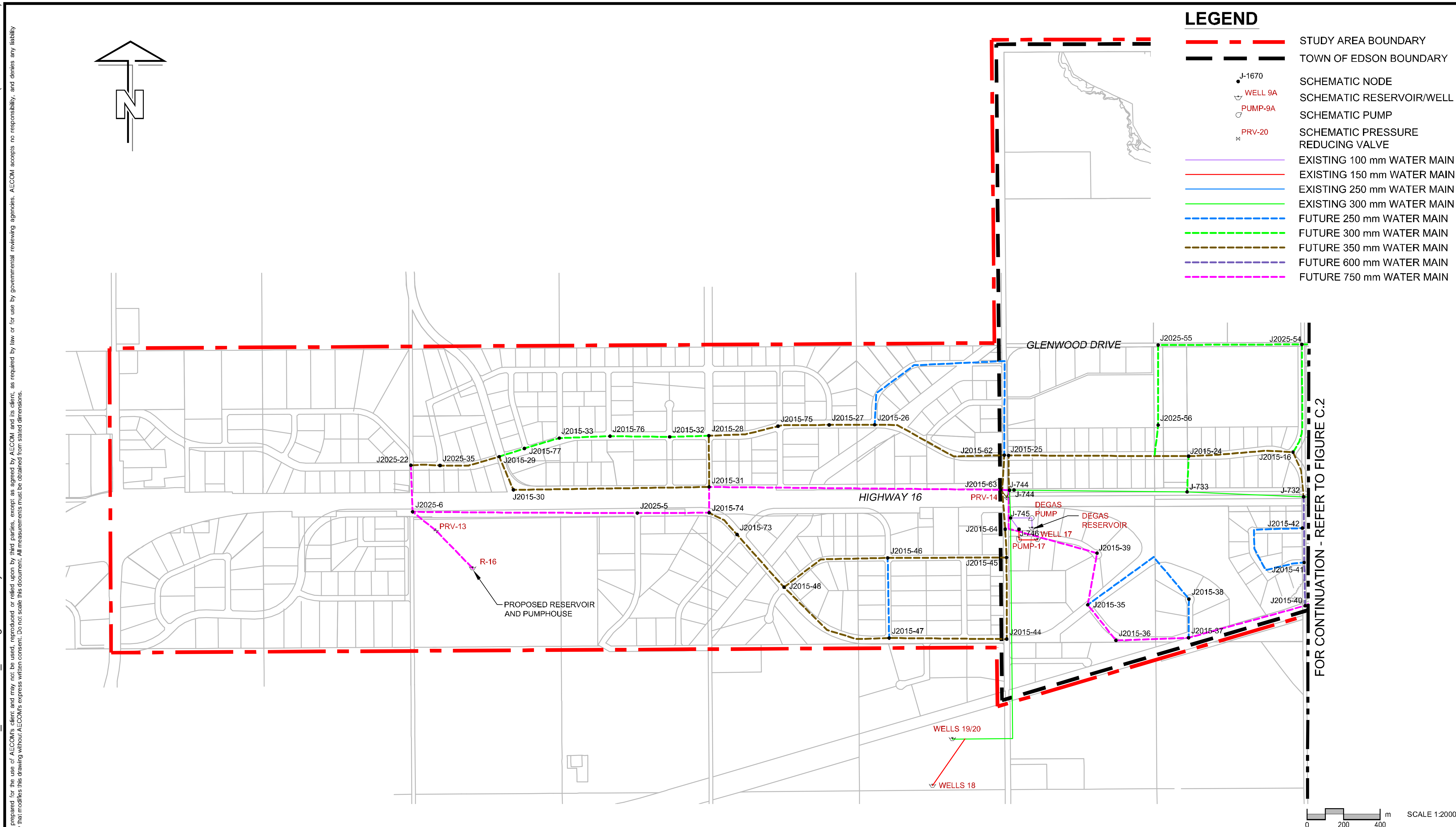
Description	Size (mm)	Cost
Existing System Improvements		
PRV-19	350	$60,000
2015		
PRV-14	750	$110,000
PRV-20	350	$60,000
PRV-21	500	$80,000
	Sub-Total (2015)	$250,000
2025		
PRV-16	350	$110,000
PRV-17	750	$110,000
PRV-22	300	$60,000
	Sub-Total (2025)	$280,000
	Sub-Total	$590,000
	Engineering (10%)	$59,000
	Contingency (25%)	$147,500
	Total	**$796,500**

The total cost for Alternative 2 is summarized in Table C.12. The costs for Alternative 1 have also been included for comparison purposes.

Table C.12: Cost Estimate Summary

Description	Alternative 1	Alternative 2
Groundwater Wells Cost	$3,746,250	$21,262,500
Reservoir Cost	$9,640,430	$32,536,600
Pumping Cost	$3,090,350	$13,824,250
Water Main Costs	$64,030,000	$104,732,000
Pressure Reducing Valve Costs	$567,000	$796,500
Total	**$81,074,030**	**$173,151,850**

Alternative 1 is the recommended alternative. As indicated in Table C.12, the costs for Alternative 1 are approximately half of the Alternative 2 costs.



Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Alternative 2 - Figure 1 of 3
Figure C.1

AECOM

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B SIZE 11" x 17" (279.4mm x 431.8mm)

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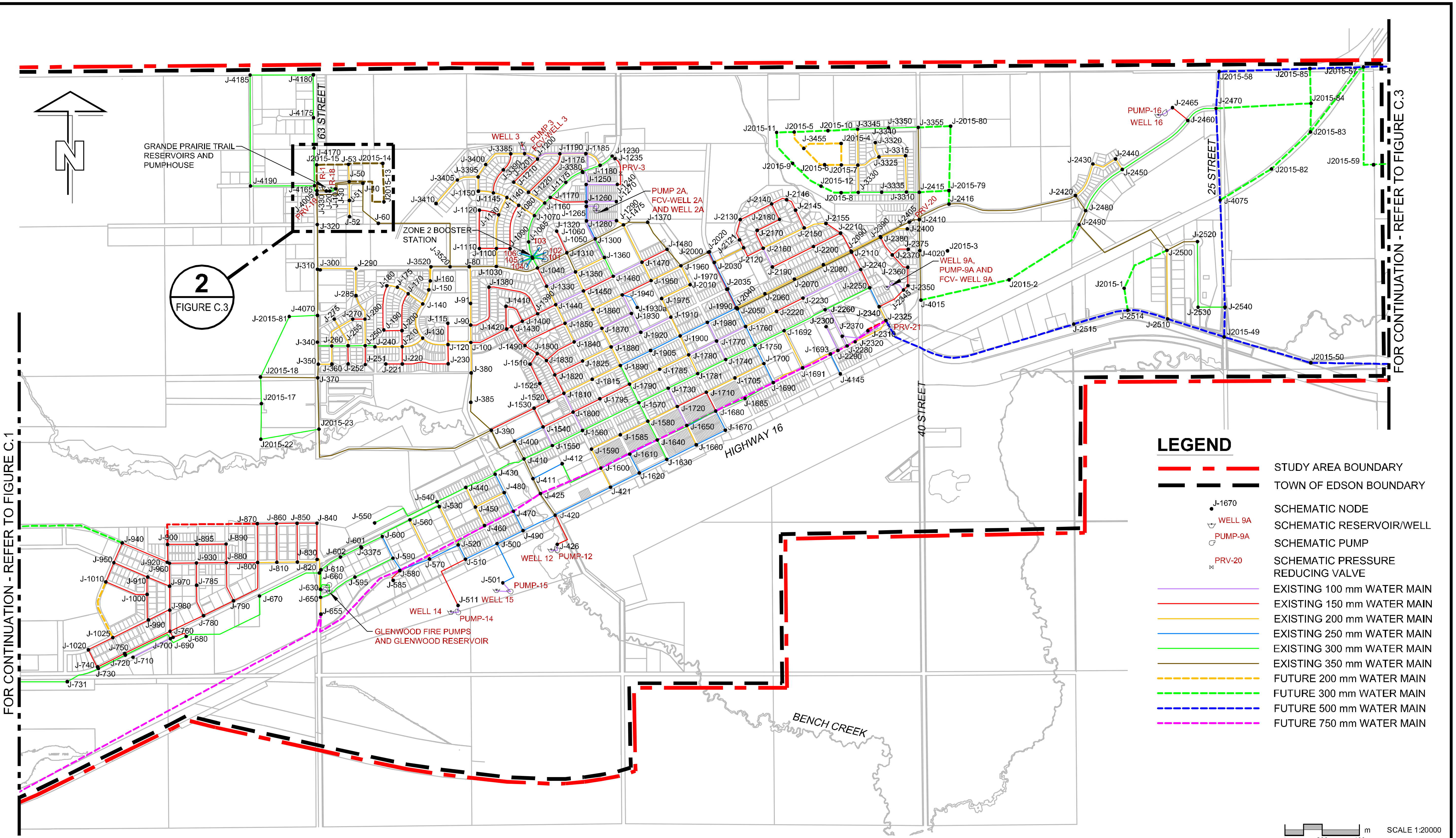

Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Alternative 2 - Figure 2 of 3

Figure C.2

AECOM

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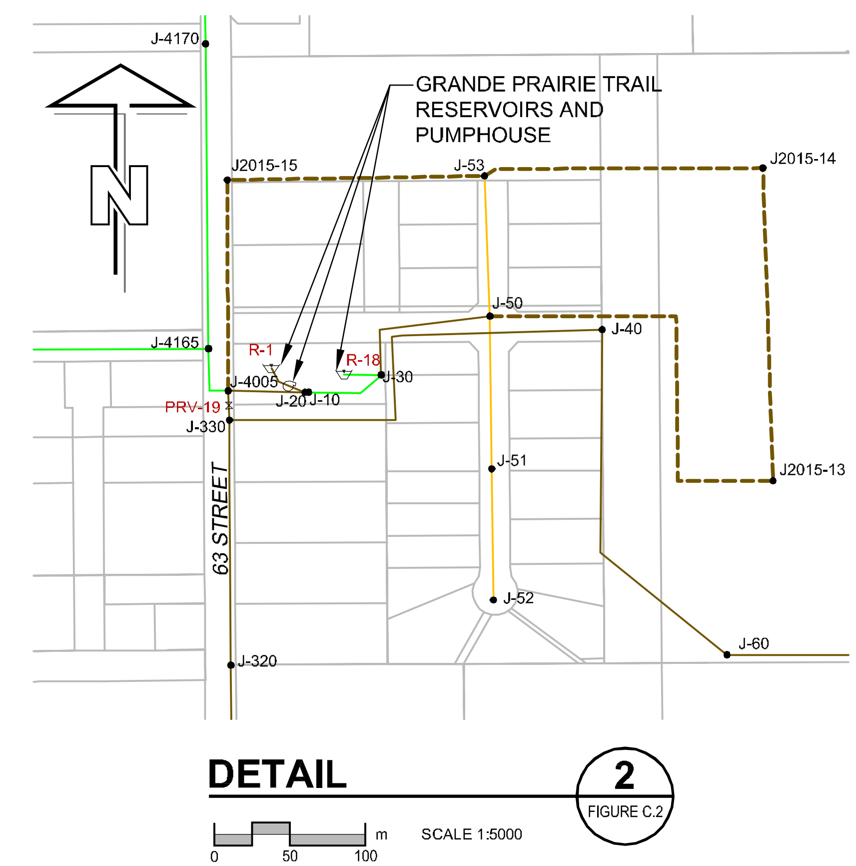
LEGEND

- STUDY AREA BOUNDARY
- TOWN OF EDSON BOUNDARY
- J-1670 — SCHEMATIC NODE
- WELL 9A — SCHEMATIC RESERVOIR/WELL
- PUMP-9A — SCHEMATIC PUMP
- PRV-20 — SCHEMATIC PRESSURE REDUCING VALVE
- EXISTING 200 mm WATER MAIN
- EXISTING 300 mm WATER MAIN
- EXISTING 350 mm WATER MAIN
- FUTURE 300 mm WATER MAIN
- FUTURE 350 mm WATER MAIN
- FUTURE 400 mm WATER MAIN
- FUTURE 500 mm WATER MAIN



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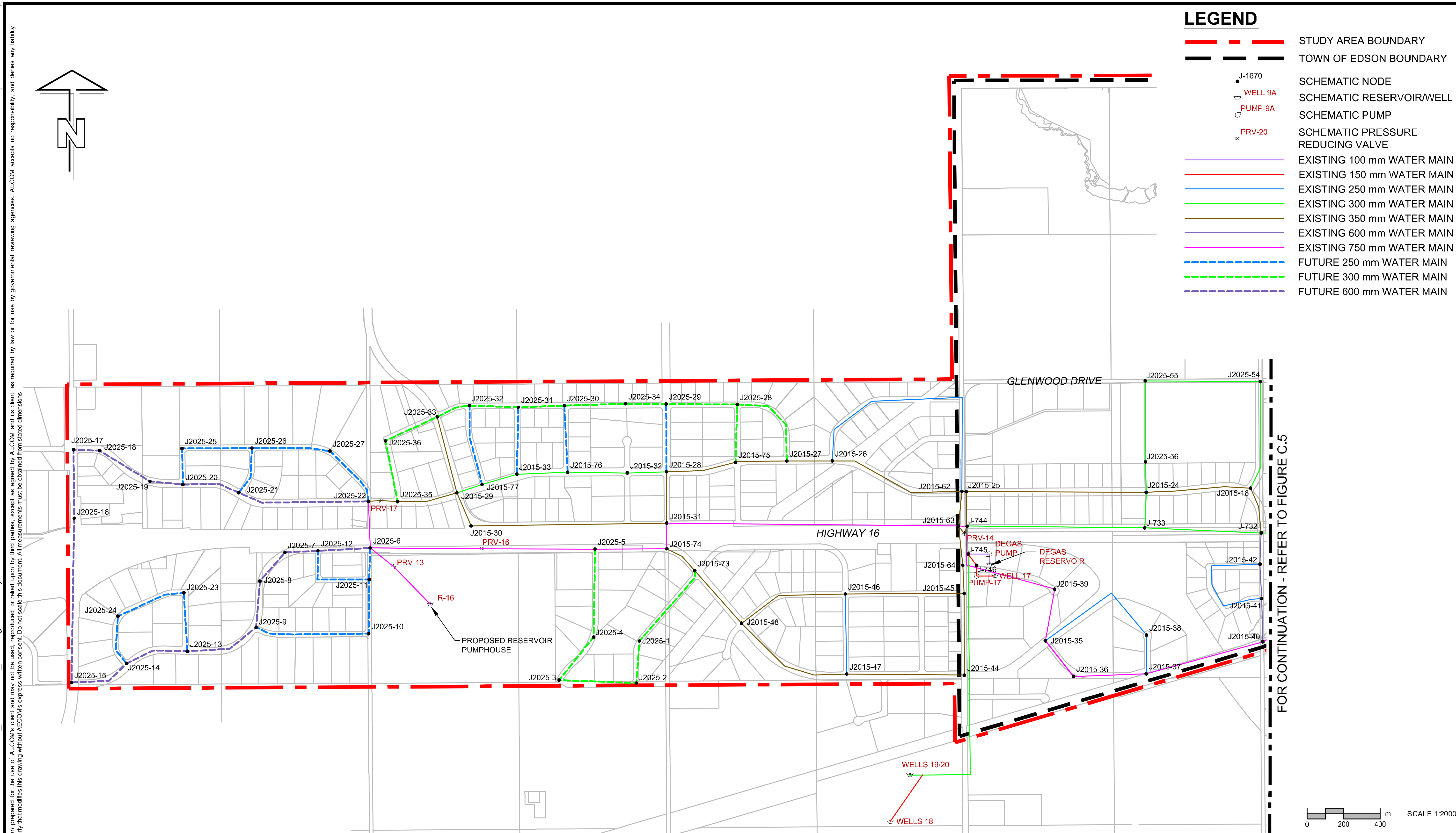
Town of Edson
Municipal Servicing Plan Update

2015 Water Distribution System Schematic
Alternative 2 - Figure 3 of 3
Figure C.3



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B SIZE 11" x 17" (279.4mm x 431.8mm)

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Town of Edson
Municipal Servicing Plan Update

2025 Water Distribution System Schematic
Alternative 2 - Figure 1 of 3
Figure C.4

B SIZE 11" x 17" (279.4mm x 431.8mm)

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UMA FILE NAME: 4193-033-00_01-CWF005_RA.dwg

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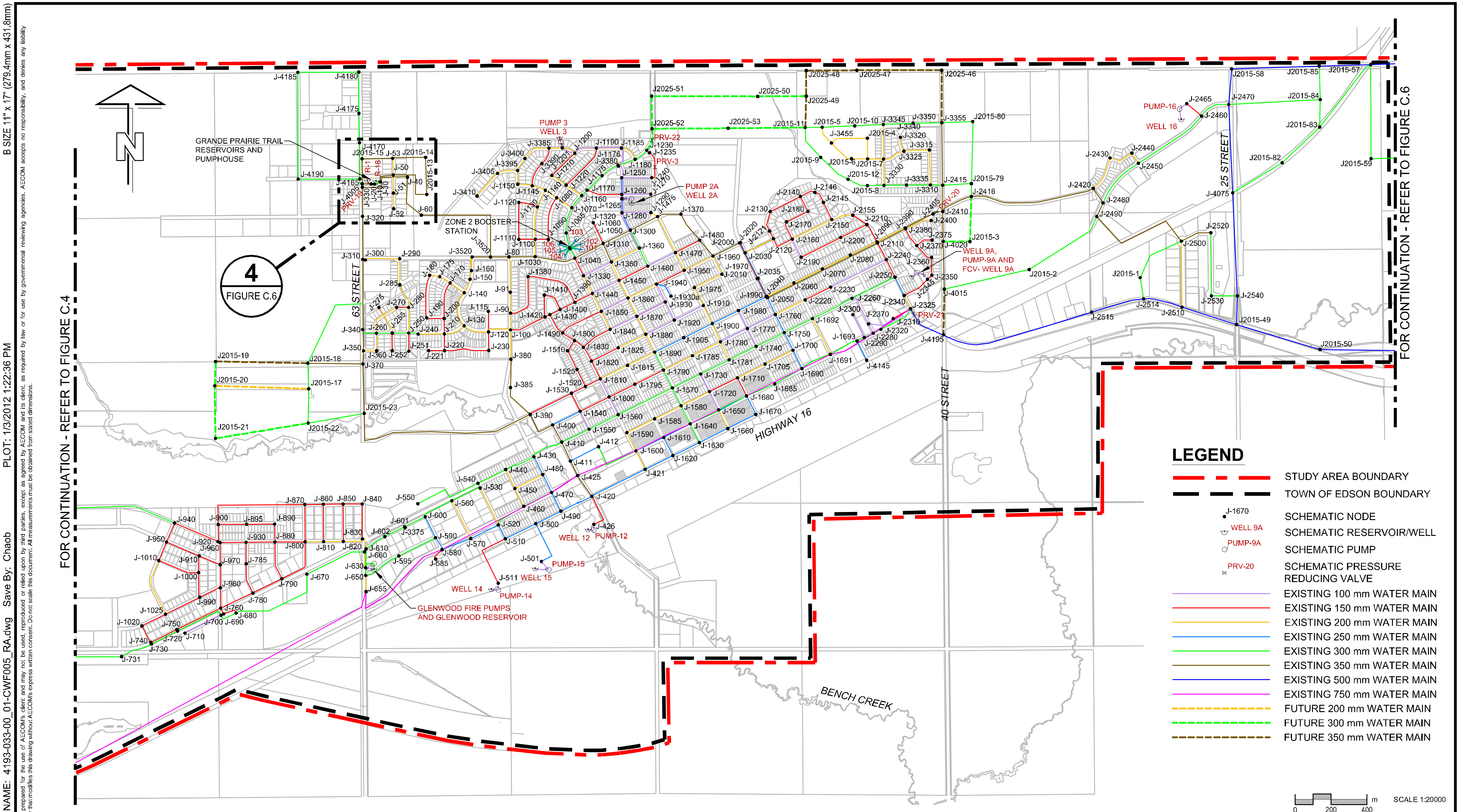


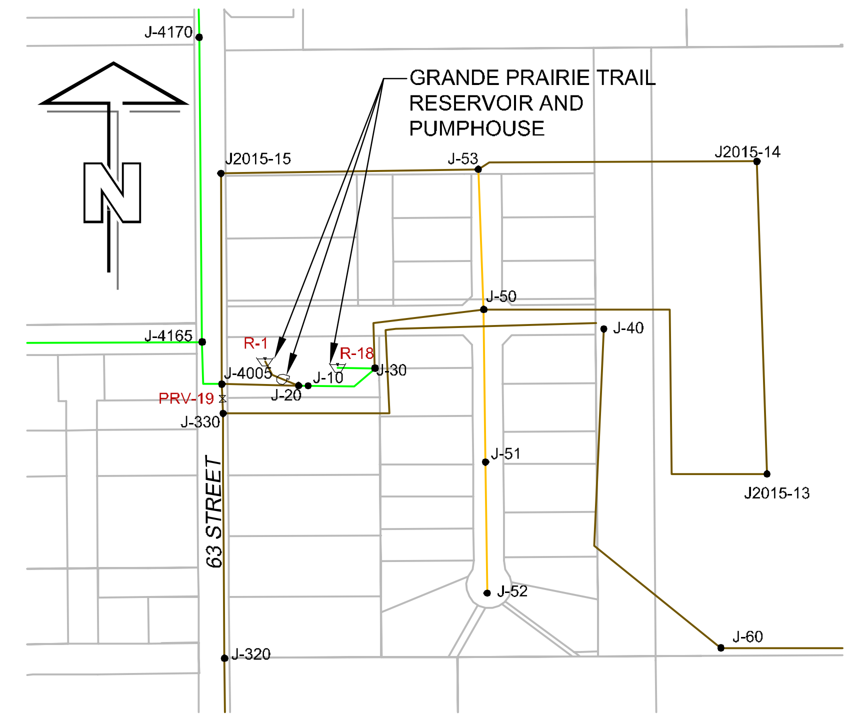
AECOM

Town of Edson
Municipal Servicing Plan Update

2025 Water Distribution System Schematic
Alternative 2 - Figure 2 of 3
Figure C.5

LEGEND

	STUDY AREA BOUNDARY
	TOWN OF EDSON BOUNDARY
J-1670	SCHEMATIC NODE
WELL 9A	SCHEMATIC RESERVOIR/WELL
PUMP-9A	SCHEMATIC PUMP
PRV-20	SCHEMATIC PRESSURE REDUCING VALVE
	EXISTING 200 mm WATER MAIN
	EXISTING 300 mm WATER MAIN
	EXISTING 350 mm WATER MAIN
	EXISTING 400 mm WATER MAIN
	EXISTING 500 mm WATER MAIN
	FUTURE 300 mm WATER MAIN
	FUTURE 400 mm WATER MAIN



DETAIL 4 FIGURE C.5

m 0 50 100 SCALE 1:5000

B SIZE 11" x 17" (279.4mm x 431.8mm)

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AECOM

Town of Edson
Municipal Servicing Plan Update

2025 Water Distribution System Schematic
Alternative 2 - Figure 3 of 3
Figure C.6

Appendix D

Cost Estimates
Water Distribution System

Table D.1: 2015 Water Storage Cost Estimates (Option 1)

Description	Alternative 1	Alternative 2	Alternative 3
Fire Storage (300 L/s for 4 hours) for Proposed West Reservoir	4,320	4,320	4,320
Fire Storage (300 L/s for 4 hours) for Grande Prairie Reservoirs	4,320	4,320	4,320
Equalization Storage - 25% of Maximum Day Demand	5,681	27,475	4,752
Emergency Storage - 15% of Average Day Demand	1,711	2,424	1,426
Total Required Storage	16,032	38,539	14,818
Existing Storage	6,530	6,530	6,530
Storage Deficiency	9,502	32,009	8,288
New Reservoir Cost considering $525/m^3	**$4,988,550**	**$16,804,725**	**$4,351,200**
Engineering (10%)	**$498,855**	**$1,680,473**	**$435,120**
Contingency (25%)	**$1,247,140**	**$4,201,190**	**$1,087,800**
Total	**$6,734,550**	**$22,686,390**	**$5,874,120**

Table D.2: 2025 Water Storage Cost Estimates (Option 1)

Description	Alternative 1	Alternative 2	Alternative 3
Fire Storage (300 L/s for 4 hours) for Proposed West Reservoir	4,320	4,320	4,320
Fire Storage (300 L/s for 4 hours) for Grande Prairie Reservoirs	4,320	4,320	4,320
Equalization Storage - 25% of Maximum Day Demand	8,835	40,025	5,443
Emergency Storage - 15% of Average Day Demand	2,657	3,772	1,633
Total Required Storage	20,132	52,437	15,717
Existing Storage (Including 2015 Upgrades)	16,032	38,539	14,818
Storage Deficiency	4,100	13,898	899
New Reservoir Cost considering $525/m^3	**$2,152,500**	**$7,296,450**	**$471,975**
Engineering (10%)	**$215,250**	**$729,645**	**$47,198**
Contingency (25%)	**$538,130**	**$1,824,120**	**$118,000**
Total	**$2,905,880**	**$9,850,220**	**$637,180**

Table D.3: Existing System Improvements Pumping Cost Estimates

Description	Alternative 1	Alternative 2	Alternative 3
Required Pumping Capacity (L/s)	391	391	391
Existing Pumping Capacity (L/s)	102	102	102
Pumping Deficiency (L/s)	289	289	289
Pumping Deficiency (HP)	273	273	273
New Pumphouse Cost considering $4600/HP	**$1,257,719**	**$1,257,719**	**$1,257,719**
Engineering (10%)	**$125,772**	**$125,772**	**$125,772**
Contingency (25%)	**$314,430**	**$314,430**	**$314,430**
Total	**$1,697,930**	**$1,697,930**	**$1,697,930**

Table D.4: 2015 Pumping Cost Estimates

Description	Alternative 1	Alternative 2	Alternative 3
Required Pumping Capacity (L/s)	790	1,872	685
Existing Pumping Capacity - Including System Improvements and Groundwater Wells (L/s)	553	1,562	492
Pumping Deficiency (L/s)	237	310	193
Pumping Deficiency (HP)	224	293	183
New Pumphouse Cost considering $4600/HP	**$1,031,416**	**$1,349,110**	**$839,930**
Engineering (10%)	**$103,142**	**$134,911**	**$83,993**
Contingency (25%)	**$257,854**	**$337,280**	**$209,990**
Total	**$1,392,420**	**$1,821,310**	**$1,133,920**

Table D.5: 2025 Pumping Cost Estimates

Description	Alternative 1	Alternative 2	Alternative 3
Required Pumping Capacity (L/s)	915	2,453	740
Existing Pumping Capacity - Including 2015 Upgrades and Groundwater Wells (L/s)	936	2,453	703
Pumping Deficiency (L/s)	0	0	37
Pumping Deficiency (HP)	0	0	35
New Pumphouse Cost considering $4600/HP	**$0**	**$0**	**$161,023**
Engineering (10%)	**$0**	**$0**	**$16,102**
Contingency (25%)	**$0**	**$0**	**$40,260**
Total	**$0**	**$0**	**$217,390**

Table D.5: Water Main Cost Estimates - Alternative 1

Pipe Diameter (mm)	Total Length (m)	Unit Cost ($/m)	Pipe Cost ($)	Restoration Cost ($)	Total Cost ($)
Existing System Improvements					
200	569	333	$189,477	$252,636	$442,113
300	4195	520	$2,181,400	$1,862,580	$4,043,980
350	298	630	$187,740	$132,312	$320,052
				Existing System Improvements Subtotal	*$4,806,145*
2015					
150	698	300	$209,400		$209,400
200	1087	333	$361,971		$361,971
250	3562	425	$1,513,850		$1,513,850
300	16878	520	$8,776,560	$168,276	$8,944,836
350	27657	630	$17,423,910		$17,423,910
				2015 Subtotal	*$28,453,967*
2025					
200	508	333	$169,164		$169,164
250	7184	425	$3,053,200		$3,053,200
300	8105	520	$4,214,600		$4,214,600
350	10686	630	$6,732,180		$6,732,180
				2025 Subtotal	*$14,169,144*
				Sub-Total	**$47,429,256**
				Engineering (10%)	**$4,742,926**
				Contingency (25%)	**$11,857,314**
				Total	**$64,030,000**

Table D.6: Water Main Cost Estimates - Alternative 2

Pipe Diameter (mm)	Total Length (m)	Unit Cost ($/m)	Pipe Cost ($)	Restoration Cost ($)	Total Cost ($)
Existing System Improvements					
200	569	333	$189,477	$252,636	$442,113
300	4195	520	$2,181,400	$1,862,580	$4,043,980
350	298	630	$187,740	$132,312	$320,052
				Existing System Improvements Subtotal	*$4,806,145*
2015					
200	757	333	$252,081		$252,081
250	3435	425	$1,459,875		$1,459,875
300	11892	520	$6,183,840		$6,183,840
350	12910	630	$8,133,300		$8,133,300
400	1281	750	$960,750		$960,750
450	485	850	$412,250		$412,250
500	3665	1000	$3,665,000		$3,665,000
600	5625	1500	$8,437,500		$8,437,500
750	11856	2000	$23,712,000	$1,512,708	$25,224,708
				2015 Subtotal	*$54,729,304*
2025					
200	523	333	$174,159		$174,159
250	5695	425	$2,420,375		$2,420,375
300	11490	520	$5,974,800		$5,974,800
350	2670	630	$1,682,100		$1,682,100
400	2199	750	$1,649,250		$1,649,250
600	4095	1500	$6,142,500		$6,142,500
				2025 Subtotal	*$18,043,184*
				Sub-Total	**$77,578,633**
				Engineering (10%)	**$7,757,863**
				Contingency (25%)	**$19,394,658**
				Total	**$104,732,000**

Table D.7: Water Main Cost Estimates - Alternative 3

Pipe Diameter (mm)	Total Length (m)	Unit Cost ($/m)	Pipe Cost ($)	Restoration Cost ($)	Total Cost ($)
Existing System Improvements					
200	569	333	$189,477	$252,636	$442,113
300	4195	520	$2,181,400	$1,862,580	$4,043,980
350	298	630	$187,740	$132,312	$320,052
				Existing System Improvements Subtotal	*$4,806,145*
2015					
150	698	300	$209,400		$209,400
200	1255	333	$417,915	$74,592	$492,507
250	1683	425	$715,275		$715,275
300	12199	520	$6,343,480	$168,276	$6,511,756
350	8946	630	$5,635,980		$5,635,980
				2015 Subtotal	*$13,564,918*
2025					
200	508	333	$169,164		$169,164
300	3354	520	$1,744,080		$1,744,080
350	2130	630	$1,341,900		$1,341,900
				2025 Subtotal	*$3,255,144*
				Sub-Total	**$21,626,207**
				Engineering (10%)	**$2,162,621**
				Contingency (25%)	**$5,406,552**
				Total	**$29,196,000**

Appendix E

Sanitary Sewer System Physical Data

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L1	N13	N12	0.2	91.13	0.406	934.57	934.2	938.5	938.1
L10	N2	N4	0.2	86.571	1.342	933.374	932.212	937.1	937.15
L100	N101	N100	0.2	117.65	3.485	921.58	917.48	924.46	920.13
L101	N100	N102	0.2	111.56	0.394	917.48	917.04	920.13	919.6
L102	N103	N102	0.3	109.73	3.4	920.77	917	923.3	919.6
L103	N104	N103	0.3	80.16	3.9	923.896	920.77	926.4	923.3
L104	N102	N105	0.2	111.53	0.404	917	916.55	919.6	918.9
L105	N106	N105	0.2	116.74	3.33	920.44	916.55	923	918.9
L106	N105	N107	0.2	89.92	3.074	916.55	913.785	918.9	916.5
L107	N107	N108	0.2	94.488	3.08	913.785	910.875	916.5	913.5
L108	N108	N109	0.25	27.55	0.6	910.875	910.71	913.5	913.3
L109	N109	N110	0.25	57.81	0.536	910.71	910.4	913.3	913.006
L11	N1	N2	0.2	53.38	1.5	934.175	933.374	937.9	937.1
L110	N111	N110	0.2	86.26	2.91	912.91	910.4	915.5	913.006
L111	N112	N111	0.2	86.26	3.83	916.21	912.91	918.79	915.5
L112	N105	N112	0.2	111.53	0.287	916.55	916.23	918.9	918.79
L113	N113	N112	0.2	109.73	3.32	919.85	916.21	922.4	918.79
L114	N114	N115	0.2	16.15	8.854	924.82	923.39	927.66	927.66
L117	N118	N117	0.2	24.1	1.079	917.26	917	922.6	922.52
L118	N119	N118	0.2	61.8	0.55	917.6	917.26	922.95	922.6
L119	N120	N119	0.2	23	0.49	917.713	917.6	923	922.95
L120	N121	N120	0.2	78.6	1.99	919.274	917.71	924.5	923
L121	N122	N121	0.2	78.6	2.01	920.85	919.27	924.5	924.5
L127	N127	N128	0.2	65.53	2.5	913.431	911.793	917	915
L128	N129	N128	0.2	18.29	1.4	912.049	911.793	915.5	915
L129	N130	N129	0.2	93.57	1.4	913.359	912.049	916.8	915.5
L130	N128	N131	0.2	175.26	0.48	911.793	910.952	915	913.6
L131	N131	N108	0.25	15.24	0.5	910.952	910.875	913.6	913.5
L136	N136	N135	0.375	3.57	5.042	912.2	912.02	917.7	917.2
L137	N137	N136	0.375	100.02	0.75	912.95	912.2	919.7	917.7
L138	N138	N137	0.375	64	1.094	913.65	912.95	920.2	919.7
L139	N139	N138	0.375	99.4	0.825	914.47	913.65	921.2	920.2
L14	N15	N16	0.2	80.163	0.65	933.48	932.959	937.2	937.4
L140	N140	N139	0.375	103.75	0.8	915.3	914.47	921.97	921.2
L141	N141	N140	0.375	26.174	1.49	917.25	916.86	922.46	921.97
L142	N142	N141	0.2	87.2	1.342	918.67	917.5	922.08	922.46
L143	N143	N142	0.2	70	2.223	920.226	918.67	924.5	922.08
L144	N144	N143	0.2	111	4.738	925.485	920.226	929.75	924.5
L145	N145	N142	0.2	99.52	1.615	920.277	918.67	923.97	922.08
L146	N146	N145	0.2	59.6	2.903	922.007	920.277	925.5	923.97
L148	N148	N147	0.2	92.23	1.204	930.137	929.027	932.6	932.65
L149	N150	N146	0.2	50.28	0.487	922.299	922.054	927.15	925.5
L15	N6	N23	0.2	57.2	2.044	931.279	930.11	934.4	933.4
L150	N151	N145	0.2	114.66	1.747	922.28	920.277	924.8	923.97
L151	N152	N151	0.2	105.9	3.77	926.27	922.28	928.8	924.8
L152	N153	N152	0.2	109.24	3.88	930.51	926.271	938.1	928.8
L153	N154	N153	0.2	102.6	3.48	934.079	930.51	938.7	938.1
L154	N155	N154	0.2	55.56	0.378	934.289	934.079	937.76	938.7
L155	N156	N155	0.2	46.86	0.209	934.414	934.316	937	937.76
L156	N157	N154	0.2	106.7	3.441	937.75	934.079	942.2	938.7
L157	N158	N157	0.2	91.43	4.31	941.69	937.75	946.2	942.2
L158	N159	N158	0.2	91	3.32	944.71	941.69	949.6	946.2
L159	N160	N159	0.2	102.9	2.915	948	945	952.5	949.6
L16	N23	N18	0.2	70.4	3.423	930.11	927.7	933.4	931.36
L160	N161	N160	0.2	91.3	2.673	950.44	948	955.2	952.5
L161	N162	N161	0.2	100.1	2.058	954.5	952.44	959.2	955.2
L162	N163	N162	0.2	111	1.649	956.33	954.5	960.9	959.2
L163	N164	N163	0.2	100.1	1.469	957.8	956.33	962.3	960.9

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L164	N165	N164	0.2	120	1.833	960	957.8	964.88	962.3
L165	N166	N165	0.2	88.2	1.871	961.65	960	965.9	964.88
L166	N167	N166	0.2	100	1.05	962.7	961.65	967.2	965.9
L167	N168	N167	0.2	112.8	1.596	964.5	962.7	970	967.2
L168	N169	N159	0.2	96	0.333	945.03	944.71	949.6	949.6
L169	N170	N169	0.2	64.9	0.231	945.18	945.03	950	949.6
L17	N19	N18	0.2	64.31	2.27	929.19	927.73	932.47	931.36
L170	N171	N140	0.2	57.02	5.349	920.6	917.55	924	921.97
L171	N172	N171	0.2	77.39	4.135	923.8	920.6	927.35	924
L172	N173	N172	0.2	153.54	3.973	929.9	923.8	930.4	927.35
L173	N174	N173	0.2	39.01	4.614	931.7	929.9	935.2	930.4
L174	N175	N174	0.2	20	3.3	932.36	931.7	937.05	935.2
L175	N176	N175	0.2	57	3.019	935.1	933.379	939.4	937.05
L176	N177	N176	0.2	76.7	0.72	935.662	935.11	939.4	939.4
L177	N178	N176	0.2	118.13	3.098	938.76	935.1	942.85	939.4
L178	N179	N178	0.2	35.34	3.424	939.97	938.76	944.5	942.85
L179	N180	N179	0.2	126.29	1.972	942.46	939.97	946.9	944.5
L18	N20	N19	0.2	57.91	2.245	930.49	929.19	933.6	932.47
L180	N181	N180	0.2	56.5	2.726	944	942.46	948.2	946.9
L181	N182	N181	0.2	74.35	0.471	944.35	944	947.3	948.2
L182	N183	N182	0.2	87.34	2.931	946.96	944.35	949.92	947.3
L183	N184	N183	0.2	87.3	2.566	949.25	946.96	952.2	949.92
L184	N185	N184	0.2	50.06	0.899	949.77	949.25	952.88	952.2
L185	N186	N185	0.2	86.75	1.095	950.7	949.75	953.8	952.88
L186	N187	N186	0.2	94.91	1.717	952.38	950.75	955.5	953.8
L19	N20	N21	0.2	127.41	2.512	930.52	927.32	933.6	930.66
L190	N191	N192	0.2	59.8	0.21	923.325	923.2	926.1	925.9
L192	N191	N194	0.2	94.8	3.25	923.325	920.244	926.1	923
L193	N194	N195	0.2	68.58	1.429	919	918.02	923	921.1
L194	N195	N196	0.2	30.48	1.64	918.02	917.52	921.1	920.5
L195	N196	N197	0.2	30.48	1.411	917.52	917.09	920.5	920.3
L196	N197	N198	0.2	96.3	2.461	917.09	914.72	920.3	917.83
L197	N199	N198	0.2	36.88	1.329	915.24	914.75	918.3	917.83
L198	N198	N200	0.2	96.3	0.332	914.72	914.4	917.83	917.54
L199	N201	N200	0.2	95.4	2.369	917.05	914.79	918.45	917.54
L2	N12	N11	0.2	71.53	0.447	934.2	933.88	938.1	937.7
L20	N21	N22	0.2	111.25	2.409	927.29	924.61	930.66	928.01
L200	N202	N201	0.2	30.48	1.542	917.52	917.05	920.58	918.45
L201	N203	N202	0.2	30.48	1.706	918.04	917.52	921.11	920.58
L203	N205	N204	0.2	98.15	2.17	916.23	914.1	919.12	917.2
L205	N206	N207	0.25	39.62	0.49	913.36	913.14	917.9	918.79
L207	N209	N208	0.2	99.79	5.101	920.15	915.06	922.99	920.59
L208	N210	N209	0.2	63.46	1.544	921.13	920.15	923.7	922.99
L209	N211	N208	0.2	69.8	5.1	918.62	915.06	921.84	920.59
L21	N22	N24	0.2	98.76	0.658	924.55	923.9	928.01	927.22
L210	N212	N211	0.2	68.34	2.019	920	918.62	923.76	921.84
L211	N213	N212	0.2	84.67	3.768	923.19	920	926.44	923.76
L212	N214	N213	0.2	60.74	1.284	923.97	923.19	927.87	926.44
L213	N215	N214	0.2	55.02	1.872	925	923.97	928.36	927.87
L214	N216	N215	0.2	83.82	1.181	925.99	925	929.64	928.36
L215	N217	N216	0.2	65.99	1.47	926.96	925.99	930.79	929.64
L216	N218	N217	0.2	35.77	3.187	928.1	926.96	931.52	930.79
L217	N219	N215	0.2	109.3	2.8	928.06	925	932.08	928.36
L218	N220	N219	0.2	67.36	4.572	931.14	928.06	934.48	932.08
L219	N221	N220	0.2	38.25	0.575	931.36	931.14	935.13	934.48
L22	N25	N24	0.2	110	2.327	926.41	923.85	929.98	927.22
L220	N222	N220	0.2	42.52	3.457	932.61	931.14	936.32	934.48
L221	N222	N223	0.2	81.99	1.549	932.61	931.34	936.32	934.49
L223	N224	N225	0.2	85.34	2.777	926.25	923.88	929.7	927.99
L226	N225	N227	0.2	77.11	3.372	923.88	921.28	927.99	924.76

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L227	N227	N228	0.2	67.39	1.098	921.28	920.54	924.76	923.82
L228	N230	N227	0.2	121.9	0.566	921.97	921.28	926.8	924.76
L229	N228	N212	0.2	67.09	0.805	920.54	920	923.82	923.76
L23	N18	N25	0.2	106.38	1.156	927.65	926.42	931.36	929.98
L230	N231	N208	0.2	101.5	0.404	915.47	915.06	918.42	920.59
L231	N232	N231	0.2	122.35	0.695	916.39	915.54	919.98	918.42
L232	N233	N232	0.2	120.06	1.291	918.52	916.97	922.05	919.98
L233	N234	N233	0.2	98.45	1.3	920.39	919.11	923.5	922.05
L234	N235	N236	0.2	52.9	0.5	927.684	927.42	930.5	930.27
L235	N237	N236	0.2	175.439	0.57	928.42	927.42	930.17	930.27
L236	N238	N237	0.2	85.3	0.57	928.906	928.42	930.67	930.17
L237	N236	N239	0.2	119.113	2.93	927.42	923.93	930.27	926.55
L239	N240	N241	0.2	118	0.5	926.67	926.08	929.34	928.8
L24	N23	N27	0.2	99.7	1.414	930.11	928.7	933.4	931.89
L240	N242	N241	0.2	41.4	1	926.494	926.08	928.8	928.8
L241	N241	N243	0.2	86.691	2.78	926.08	923.67	928.8	926.14
L242	N243	N244	0.2	111.33	0.018	923.67	923.65	926.14	926.14
L243	N245	N244	0.2	45.24	2	924.555	923.65	926.5	926.14
L244	N246	N245	0.2	86.26	2	926.28	924.555	928.73	926.5
L245	N246	N247	0.2	175	2	926.28	922.78	928.73	925.3
L246	N239	N247	0.2	111.111	1.17	923.93	922.63	926.55	925.3
L247	N247	N248	0.2	126.036	0.603	922.63	921.87	925.3	924.72
L248	N249	N248	0.2	88.412	2.33	924.08	922.02	926.6	924.72
L249	N250	N249	0.2	86.087	1.15	925.07	924.08	927.66	926.6
L25	N27	N28	0.2	58.22	1.22	928.7	927.99	931.89	931.12
L250	N248	N251	0.2	112.195	0.41	921.87	921.41	924.72	924.58
L251	N252	N251	0.2	87.895	1.9	923.54	921.87	926.09	924.58
L252	N253	N252	0.2	85.915	0.71	924.15	923.54	926.8	926.09
L253	N253	N254	0.2	85.849	1.06	924.15	923.24	926.8	925.92
L254	N254	N255	0.2	86.286	3.5	923.24	920.22	925.92	922.7
L255	N256	N255	0.2	111.399	1.158	921.48	920.19	923.55	922.7
L256	N257	N256	0.2	112.263	1.158	922.78	921.48	925.37	923.55
L257	N258	N257	0.2	40.54	0.4	922.942	922.78	925.6	925.37
L258	N244	N257	0.2	110.406	0.788	923.65	922.78	926.14	925.37
L259	N260	N85	0.2	176	0.5	920.52	919.64	923.2	922.6
L26	N28	N29	0.2	63.7	1.429	927.97	927.06	931.12	930.5
L260	N260	N261	0.2	172.52	0.5	920.52	919.657	923.2	922.5
L261	N261	N262	0.2	103.02	0.4	919.657	919.245	922.5	922.2
L262	N262	N263	0.2	94.79	0.196	919.245	919.06	922.2	922.17
L263	N263	N264	0.2	99.06	2.998	919.06	916.09	922.17	919.73
L264	N264	N265	0.2	111.56	2.599	916.09	913.19	919.73	915.92
L265	N265	N266	0.2	109.12	1.402	913.19	911.66	915.92	914.55
L266	N267	N266	0.2	111.54	0.565	911	910.37	913.04	914.55
L267	N268	N267	0.2	112.78	1.445	912.63	911	916.35	913.04
L268	N269	N268	0.2	59.31	2.361	914.03	912.63	917.43	916.35
L269	N270	N269	0.2	52.88	0.548	914.32	914.03	918.06	917.43
L27	N29	N30	0.2	107.59	1.255	927.04	925.69	930.5	929
L270	N271	N270	0.2	84.12	0.594	914.82	914.32	918.49	918.06
L271	N272	N271	0.2	83.82	0.346	915.11	914.82	918.09	918.49
L272	N273	N272	0.2	43.59	1.606	915.81	915.11	918.48	918.09
L273	N274	N273	0.2	68.58	1.429	916.79	915.81	919.52	918.48
L274	N275	N274	0.2	99.36	1.077	917.86	916.79	920.56	919.52
L275	N276	N275	0.2	130.15	1.16	919.37	917.86	922.52	920.56
L276	N277	N274	0.2	96.01	1.385	918.12	916.79	921.33	919.52
L277	N278	N277	0.2	94.45	0.487	918.58	918.12	922.08	921.33
L278	N279	N278	0.2	77.72	0.669	919.1	918.58	922.04	922.08
L279	N280	N278	0.2	304.8	1.5	923.152	918.58	926	922.08
L280	N281	N280	0.2	121.92	1.5	924.981	923.152	928	926
L281	N282	N281	0.2	94.48	1.5	926.398	924.981	930	928
L283	N284	N283	0.2	85.34	1.348	919.62	918.47	922.77	922.77

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L284	N285	N283	0.2	99.36	0.463	918.93	918.47	922.79	922.77
L285	N286	N285	0.2	51.11	0.411	919.14	918.93	922.5	922.79
L286	N287	N286	0.2	44.81	0.29	919.27	919.14	922.36	922.5
L287	N288	N287	0.2	63.09	0.491	919.58	919.27	922.14	922.36
L288	N289	N286	0.2	121	0.463	919.7	919.14	922.34	922.5
L289	N291	N267	0.2	86.26	3.348	913.888	911	918	913.04
L29	N30	N24	0.2	99.97	1.711	925.64	923.93	929	927.22
L290	N292	N291	0.2	86.26	1.25	914.96	913.882	919	918
L291	N293	N292	0.2	86.26	5.5	919.7	914.96	923	919
L292	N294	N293	0.2	86.26	0.394	920.04	919.7	923.3	923
L293	N295	N294	0.2	73.15	0.397	920.33	920.04	923.6	923.3
L294	N296	N295	0.23	44.196	0.385	920.5	920.33	923.15	923.6
L295	N266	N297	0.2	111.56	1.578	910.37	908.61	914.55	912.27
L296	N298	N297	0.2	109.73	2.005	911.31	909.11	914.4	912.27
L297	N299	N298	0.2	109.73	2.005	913.51	911.31	916.84	914.4
L298	N300	N299	0.2	106.68	1.894	915.53	913.51	919.3	916.84
L299	N301	N300	0.2	109.73	0.665	916.26	915.53	919.52	919.3
L3	N11	N10	0.2	37.52	0.48	933.88	933.7	937.7	937.5
L30	N16	N32	0.2	67.172	1.596	932.959	931.887	937.4	935.956
L300	N302	N301	0.2	109.73	0.602	916.92	916.26	919.79	919.52
L301	N303	N302	0.2	103.63	0.396	917.33	916.92	920.2	919.79
L302	N297	N304	0.2	111.54	0.834	908.61	907.68	912.27	910.5
L303	N305	N304	0.2	172.49	0.997	909.57	907.85	913.05	910.5
L304	N306	N305	0.2	38.1	1.549	910.4	909.81	913.9	913.05
L305	N307	N306	0.2	36.88	6.318	912.92	910.59	916	913.9
L306	N308	N307	0.2	74.98	2.15	914.53	912.92	917.2	916
L307	N309	N308	0.2	80.77	2.154	916.27	914.53	918.9	917.2
L308	N310	N309	0.2	42.11	0.4	916.438	916.27	919	918.9
L309	N311	N310	0.15	124.36	0.401	916.937	916.438	921	919
L31	N33	N32	0.2	73.952	1.402	932.868	931.831	936.589	935.956
L310	N312	N306	0.15	89.31	2.6	912.72	910.4	916	913.9
L311	N304	N313	0.2	111.62	2.464	907.68	904.93	910.5	907.542
L312	N313	N314	0.2	103.77	2.193	904.723	902.448	907.542	905.1
L313	N314	N315	0.45	13.8	3.244	902.448	902	905.1	904.6
L314	N315	N316	0.45	26.82	1.364	902	901.634	904.6	905.88
L315	N317	N316	0.45	56.39	0.19	901.741	901.634	906.008	905.88
L316	N318	N317	0.45	39.47	0.616	901.984	901.741	904.6	906.008
L317	N319	N318	0.45	68.58	1.413	902.953	901.984	907.246	904.6
L318	N319	N320	0.375	62.48	0.555	902.953	902.606	907.246	905.3
L319	N320	N314	0.375	60.96	0.26	902.606	902.448	905.3	905.1
L32	N33	N34	0.2	103.406	1.436	932.868	931.383	936.589	935.26
L320	N316	N321	0.45	121.31	0.304	901.634	901.265	905.88	903.602
L321	N322	N321	0.375	100.3	0.613	901.88	901.265	905.16	903.602
L322	N323	N322	0.2	17.22	2.497	902.31	901.88	905.08	905.16
L323	N324	N323	0.2	48.46	0.536	902.57	902.31	905.4	905.08
L324	N325	N324	0.2	109.73	1.969	904.73	902.57	907.72	905.4
L325	N326	N325	0.2	25.91	1.081	905.01	904.73	907.63	907.72
L326	N327	N325	0.2	25.15	3.777	905.68	904.73	908.42	907.72
L327	N328	N327	0.2	86.87	1.75	907.23	905.68	910.82	908.42
L328	N329	N328	0.2	53.49	0.654	907.58	907.23	911.09	910.82
L329	N330	N329	0.2	43.89	0.57	907.83	907.58	911.25	911.09
L33	N34	N35	0.2	98.323	2.702	931.301	928.644	935.26	933.058
L330	N331	N328	0.2	51.36	3.096	909.55	907.96	912.27	910.82
L331	N332	N331	0.2	61.87	0.533	909.88	909.55	913.84	912.27
L332	N333	N332	0.2	146.63	0.198	910.71	910.42	913.24	913.84
L333	N334	N332	0.2	52.82	1.5	910.672	909.88	915	913.84
L334	N335	N332	0.2	102.41	0.605	910.55	909.93	913.75	913.84
L335	N336	N335	0.2	48.77	0.615	910.85	910.55	913.56	913.75
L336	N337	N336	0.2	80.07	1.5	912.051	910.85	915	913.56
L337	N338	N337	0.2	109.42	1.5	913.692	912.051	917	915

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L338	N340	N316	0.45	79.25	0.14	901.745	901.634	905	905.88
L339	N341	N340	0.45	46.33	0.12	901.801	901.745	905.27	905
L34	N36	N35	0.2	99.081	0.632	929.362	928.736	933.407	933.058
L340	N342	N341	0.45	67.94	0.147	901.9	901.801	905	905.27
L341	N343	N342	0.45	12.95	0.416	901.954	901.9	904.6	905
L342	N318	N343	0.45	42.67	0.07	901.984	901.954	904.6	904.6
L343	N344	N342	0.45	141.73	0.028	901.94	901.9	906	905
L344	N345	N344	0.45	172.52	0.16	902.216	901.94	907.8	906
L345	N346	N345	0.45	172.59	0.102	902.392	902.216	908	907.8
L346	N347	N346	0.45	172.52	0.16	902.668	902.392	908.2	908
L347	N348	N347	0.45	172.52	0.16	902.944	902.668	908.5	908.2
L348	N349	N348	0.375	172.52	0.16	903.22	902.944	908.85	908.5
L349	N350	N349	0.25	115.77	0.976	905.87	904.74	910.51	908.85
L35	N37	N36	0.2	115.214	0.451	929.882	929.362	933.234	933.407
L350	N350	N351	0.375	175	0.2	905.87	905.52	910.51	909.68
L351	N351	N352	0.375	170	0.2	905.52	905.18	909.68	909
L352	N352	N353	0.375	175	0.408	905.18	904.467	909	907.959
L353	N353	N354	0.375	63.636	0.55	904.467	904.117	907.959	906.3
L354	N354	N355	0.375	98.15	0.249	904.117	903.872	906.3	905.8
L355	N355	N356	0.375	74.7	0.201	903.872	903.722	905.8	905.7
L356	N356	N357	0.375	59.7	0.8	903.722	903.245	905.7	905.62
L357	N357	N319	0.375	100.6	0.29	903.245	902.953	905.62	907.246
L358	N358	N356	0.2	117.96	2.158	906.268	903.722	908.4	905.7
L359	N359	N358	0.2	48.16	3.8	908.098	906.268	910.2	908.4
L36	N32	N38	0.2	68.237	1.631	931.831	930.718	935.956	934.788
L360	N360	N355	0.2	100.09	1.225	905.098	903.872	907.2	905.8
L361	N361	N354	0.2	102.4	2	906.165	904.117	908.3	906.3
L362	N362	N353	0.2	113.1	1.91	906.99	904.83	911.1	907.959
L363	N363	N362	0.2	86	4.988	911.28	906.99	915.3	911.1
L364	N364	N352	0.2	113.69	2.1	908.053	905.67	911.4	909
L365	N365	N364	0.2	8.225	1.56	908.181	908.053	911.6	911.4
L366	N366	N365	0.2	52.73	4.1	910.343	908.181	913.1	911.6
L368	N368	N367	0.2	71.018	0.4	909.113	908.829	912	911.35
L369	N369	N368	0.2	90.96	0.268	909.357	909.113	911.81	912
L37	N38	N39	0.2	71.814	3.02	930.718	928.549	934.788	932.925
L370	N370	N350	0.25	112.988	0.947	909.68	908.61	911.96	910.51
L371	N371	N370	0.25	111.932	0.947	910.74	909.68	914.09	911.96
L372	N372	N371	0.25	148.78	1.64	913.18	910.74	917.13	914.09
L373	N373	N372	0.2	97.146	1.23	915.975	914.78	919	917.13
L374	N83	N373	0.2	51.82	1.92	916.97	915.975	920	919
L375	N83	N374	0.2	102	0.5	916.97	916.46	920	920.52
L376	N374	N373	0.2	39.439	1.23	916.46	915.975	920.52	919
L377	N375	N374	0.2	79.508	4.99	920.427	916.46	923.4	920.52
L378	N376	N375	0.2	86.26	2.6	922.67	920.427	925.07	923.4
L379	N376	N377	0.2	85.366	0.41	922.67	922.32	925.07	924.85
L38	N39	N40	0.2	63.604	2.093	928.549	927.218	932.925	930.482
L380	N377	N378	0.2	88.35	2.06	922.32	920.5	924.85	922.98
L381	N251	N378	0.2	194.876	1.093	921.41	919.89	924.58	922.98
L382	N255	N374	0.2	151.626	2.46	920.19	916.46	922.7	920.52
L383	N378	N379	0.2	111.538	2.6	919.89	916.99	922.98	919.83
L384	N380	N379	0.2	103.382	2.07	919.28	917.14	921.6	919.83
L385	N380	N381	0.2	165	0.4	919.28	918.62	921.6	921.47
L386	N381	N382	0.2	87.17	4.274	918.62	914.894	921.47	918.63
L387	N382	N372	0.2	78.64	2.18	914.894	913.18	918.63	917.13
L388	N383	N372	0.2	119.82	1.11	916.34	915.01	919.41	917.13
L389	N384	N383	0.2	60	0.4	916.58	916.34	919.47	919.41
L39	N37	N39	0.2	67.574	1.884	929.882	928.609	933.234	932.925
L390	N385	N384	0.2	86.21	0.841	917.305	916.58	919.8	919.47
L391	N386	N385	0.2	86.21	0.4	917.65	917.305	920.2	919.8
L392	N387	N388	0.2	97.23	3.486	915.932	912.543	917.5	917

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L393	N388	N389	0.2	8.23	0.4	912.543	912.51	917	915.1
L394	N389	N390	0.2	232.5	0.4	912.51	911.58	915.1	914.27
L395	N391	N390	0.2	75.39	1.5	912.711	911.58	916.5	914.27
L396	N390	N371	0.2	172.5	0.4	911.58	910.89	914.27	914.09
L397	N392	N371	0.2	173.134	0.67	912.27	911.11	915.13	914.09
L398	N393	N392	0.2	109.836	1.22	913.61	912.27	916.33	915.13
L399	N379	N394	0.2	111.538	2.6	916.99	914.09	919.83	916.75
L4	N10	N4	0.2	61.98	2.514	933.7	932.142	937.5	937.15
L40	N40	N41	0.2	64	0.634	927.218	926.812	930.482	929.449
L400	N394	N395	0.2	151.064	2.82	914.09	909.83	916.75	912.8
L401	N396	N395	0.2	17.68	0.848	908.24	908.09	911.5	912.8
L402	N397	N396	0.2	50.29	0.4	908.44	908.24	911.7	911.5
L403	N395	N398	0.375	126.8	0.505	908.09	907.45	912.8	910.2
L404	N398	N399	0.3	175.26	0.451	907	906.21	910.2	910.16
L405	N400	N399	0.2	113.69	0	908.12	908.12	911.25	910.16
L406	N401	N400	0.2	101.5	0.778	908.91	908.12	912.08	911.25
L408	N399	N350	0.3	170	0.2	906.21	905.87	910.16	910.51
L409	N399	N403	0.2	116.9	0.813	906.21	905.26	910.16	909.22
L41	N41	N42	0.2	51.768	0.974	926.812	926.308	929.449	929.201
L410	N403	N349	0.45	172.44	0.122	903.52	903.31	909.22	908.85
L411	N404	N403	0.45	174.3	0.161	903.8	903.52	909.22	909.22
L412	N405	N404	0.375	46.63	0.858	904.2	903.8	908.55	909.22
L413	N406	N405	0.2	16.22	1.5	904.443	904.2	908.8	908.55
L414	N407	N405	0.375	171.97	0.169	904.49	904.2	908.49	908.55
L415	N408	N407	0.375	117.72	0.731	905.87	905.01	909.16	908.49
L416	N409	N408	0.2	86.26	0.151	906	905.87	910	909.16
L417	N410	N409	0.2	78.94	0.443	906.35	906	910	910
L418	N411	N408	0.375	113.6	0.669	906.63	905.87	909.68	909.16
L419	N412	N411	0.25	85.95	0.547	907.1	906.63	909.24	909.68
L42	N42	N43	0.2	51.779	0.803	926.308	925.892	929.201	928.7
L420	N412	N413	0.2	172	0.25	907.71	907.28	909.24	910.32
L421	N414	N413	0.2	110	1	909.1	908	911.93	910.32
L422	N413	N415	0.2	111.56	0.645	907.28	906.56	910.32	910.24
L423	N398	N415	0.2	176	0.25	907	906.56	910.2	910.24
L424	N416	N398	0.2	115.82	0.743	907.86	907	910.4	910.2
L425	N417	N416	0.2	75.29	0.4	908.16	907.86	910.7	910.4
L426	N418	N407	0.375	133.12	0.203	904.76	904.49	908.9	908.49
L427	N420	N421	0.375	155.75	0.202	905.47	905.155	908.6	908.3
L428	N421	N422	0.6	31	0.952	905.155	904.86	908.3	909.06
L429	N422	N423	0.6	88.5	0.712	904.86	904.23	909.06	908.98
L43	N44	N43	0.2	86.3	2.45	927.878	925.764	930	928.7
L430	N423	N424	0.6	113.4	0.626	904.23	903.52	908.98	908.49
L431	N424	N425	0.6	107.5	0.633	903.52	902.84	908.49	907.59
L432	N425	N426	0.9	91.37	0.055	902.49	902.44	907.59	907.79
L433	N426	N427	0.9	90.2	0.1	902.44	902.35	907.79	907.91
L434	N427	N428	0.9	148.8	0.229	902.35	902.01	907.91	908.36
L435	N428	N429	0.9	147.95	0.115	902.01	901.84	908.36	908.24
L436	N429	N430	0.9	151	0.093	901.84	901.7	908.24	908.63
L437	N430	N431	0.9	150.5	0.14	901.7	901.49	908.63	908.65
L438	N431	N432	0.9	149.85	0.107	901.49	901.33	908.65	907.93
L439	N432	N433	0.9	150.58	0.12	901.33	901.15	907.93	907.97
L44	N43	N45	0.2	89.82	1.726	925.764	924.214	928.7	927
L440	N433	N434	0.9	150.15	0.067	901.15	901.05	907.97	907.42
L441	N434	N435	0.9	150.7	0.133	901.05	900.85	907.42	906.43
L442	N435	N436	0.9	150.8	0.093	900.85	900.71	906.43	904.53
L443	N436	N437	0.9	147.52	0.129	900.71	900.52	904.53	905.81
L444	N437	N438	0.9	147.85	0.074	900.52	900.41	905.81	905.65
L445	N438	N439	0.9	117.06	0.197	900.41	900.18	905.65	905.31
L446	N439	N440	0.9	132.9	0.075	900.18	900.08	905.31	904.41
L447	N440	N441	0.9	145	0.104	900.08	899.93	904.41	905.08

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L45	N45	N46	0.2	62.9	0.6	924.214	923.837	927	926.5
L450	N443	N444	1.05	117.78	0.195	899.28	899.05	905.31	901.89
L451	N444	N445	1.2	154.8	0.026	899.05	899.01	901.89	902.06
L452	N445	N446	1.05	135.2	0.03	899.01	898.97	902.06	901.97
L453	N446	N447	1.05	128.2	0.062	898.97	898.89	901.97	902.54
L454	N447	N448	1.05	148.02	0.047	898.89	898.82	902.54	901.66
L455	N448	N449	1.05	150.4	0.04	898.82	898.76	901.66	902.32
L456	N449	N450	1.05	158.4	0.063	898.76	898.66	902.32	902.32
L457	N450	N451	1.05	158.1	0.063	898.66	898.56	902.32	903.82
L458	N451	N452	1.05	155.86	0.045	898.56	898.49	903.82	904.94
L46	N47	N44	0.2	61.2	1.608	928.862	927.878	931.2	930
L461	N454	N455	1.05	67.85	0.103	898.36	898.29	901.8	900.93
L462	N455	N456	1.05	62.9	0.064	898.29	898.25	900.93	900.39
L463	N456	N457	1.05	6.13	1.468	898.25	898.16	900.39	900.8
L464	N459	N322	0.375	99.5	1.417	903.29	901.88	907	905.16
L465	N460	N459	0.375	98.7	2.452	905.71	903.29	909.28	907
L466	N461	N460	0.375	102.6	2.144	907.91	905.71	911.62	909.28
L467	N462	N461	0.375	100.3	1.087	909	907.91	913.41	911.62
L468	N463	N462	0.375	127.5	1.153	910	909	914.17	913.41
L469	N473	N463	0.375	98.9	0.475	910.47	910	915.32	914.17
L47	N47	N48	0.2	54.8	0.799	928.862	928.424	931.2	930.6
L470	N474	N473	0.2	100	2.49	912.96	910.47	916	915.32
L471	N475	N474	0.2	95.91	2.49	915.348	912.96	918	916
L472	N476	N475	0.2	10	1.5	915.498	915.348	918.5	918
L473	N477	N475	0.2	100	1.789	917.137	915.348	921	918
L474	N477	N478	0.2	45.96	1.5	917.826	917.137	921	920.8
L475	N479	N478	0.2	84.84	1.5	918.409	917.137	921.5	920.8
L477	N481	N480	0.25	190.62	1.5	918.189	915.33	921.2	918.5
L478	N482	N481	0.2	319	1.5	922.974	918.189	925.7	921.2
L479	N484	N481	0.25	49.3	1.5	918.929	918.189	922	921.2
L48	N48	N49	0.2	120.1	0.804	928.424	927.458	930.6	930.2
L480	N480	N485	0.25	147.32	1.5	915.33	913.12	918.5	917
L481	N485	N486	0.375	125.97	1.5	913.12	911.23	917	916.5
L482	N486	N473	0.375	97.3	0.781	911.23	910.47	916.5	915.32
L483	N321	N487	0.525	177.37	0.081	901.265	901.121	903.602	903.9
L484	N487	N488	0.525	144.31	0.18	901.121	900.861	903.9	903.5
L485	N488	N489	0.575	94.69	0.18	900.861	900.69	903.5	903.3
L486	N489	N490	0.525	90.36	0.18	900.69	900.528	903.3	903.2
L487	N490	N491	0.525	151.2	0.18	900.528	900.255	903.2	902.9
L488	N491	N492	0.525	457.8	0.18	900.255	899.431	902.9	902.1
L489	N492	N493	0.525	162.56	0.18	899.431	899.139	902.1	901.9
L49	N49	N50	0.2	45.7	0.623	927.458	927.174	930.2	930.6
L490	N493	N494	0.525	111.76	0.18	899.139	898.938	901.9	901.6
L492	N110	N495	0.25	111.56	0.699	910.4	909.62	913.006	912.42
L493	N496	N495	0.2	86.26	1.44	910.86	909.62	913.62	912.42
L494	N497	N496	0.2	86.26	3.501	913.88	910.86	916.62	913.62
L495	N498	N497	0.2	109.73	3.5	917.72	913.88	920.5	916.62
L496	N499	N498	0.2	15.24	3.5	918.25	917.72	921.1	920.5
L497	N495	N500	0.25	81.188	0.505	909.62	909.3	912.42	911.63
L498	N501	N500	0.2	86.014	1.43	910.74	909.51	913.15	911.63
L499	N502	N501	0.2	86.224	3.92	914.12	910.74	916.74	913.15
L5	N4	N5	0.2	85.5	0.166	932.142	932	937.15	935.6
L50	N50	N51	0.2	99.6	0.506	927.174	926.67	930.6	929.9
L500	N503	N502	0.2	109.474	3.04	917.448	914.12	919.98	916.74
L501	N500	N504	0.25	172.48	0.046	909.3	909.22	911.63	911.9
L502	N505	N504	0.2	111.56	0.5	909.778	909.22	913.4	911.9
L503	N506	N505	0.2	79.25	0.5	910.174	909.778	913.7	913.4
L504	N507	N506	0.2	30.78	0.5	910.328	910.174	912.9	913.7
L505	N508	N506	0.2	118.26	0.5	910.765	910.174	913.3	913.7
L506	N509	N508	0.2	105.16	0.5	911.291	910.765	914	913.3

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L508	N511	N510	0.375	89.23	0.314	907.35	907.07	909.96	909.71
L509	N512	N511	0.375	111.54	0.323	907.71	907.35	910.3	909.96
L51	N51	N52	0.2	73.2	1.011	926.67	925.93	929.9	930
L510	N513	N512	0.2	79.25	0.505	908.11	907.71	910.43	910.3
L511	N514	N512	0.375	76.2	0.84	908.35	907.71	910.6	910.3
L512	N515	N514	0.375	71.63	0.684	908.84	908.35	911.69	910.6
L517	N519	N515	0.25	122.41	1.544	910.89	909	913.9	911.69
L518	N520	N519	0.25	121.92	1.403	912.6	910.89	916.35	913.9
L519	N521	N520	0.25	40.6	0.739	912.9	912.6	918.79	916.35
L52	N52	N53	0.2	93.88	0.618	925.93	925.35	930	929.41
L521	N500	N522	0.25	111.927	1.09	909.3	907.99	911.63	911.47
L522	N522	N411	0.375	111.51	1.121	907.99	906.63	911.47	909.68
L523	N523	N522	0.2	114.789	1.324	909.51	907.99	914.2	911.47
L524	N524	N523	0.2	66.154	3.25	911.66	909.51	914.3	914.2
L525	N525	N524	0.2	106.286	1.75	913.6	911.74	916.28	914.3
L526	N526	N522	0.2	171.429	0.35	908.6	908	911.3	911.47
L527	N527	N526	0.2	102.857	0.35	908.96	908.6	911.7	911.3
L528	N504	N526	0.2	111.56	0.556	909.22	908.6	911.9	911.3
L53	N53	N54	0.2	57.3	1.361	925.34	924.56	929.41	928.12
L530	N529	N528	0.2	48.77	0.4	906.14	905.94	909.88	909.68
L531	N528	N530	0.375	85.34	0.434	905.94	905.57	909.68	911
L532	N530	N531	0.375	113.69	0.2	905.57	905.34	911	910.8
L533	N531	N421	0.6	115.82	0.155	905.34	905.16	910.8	908.3
L534	N532	N421	0.4	89.002	0.767	905.838	905.155	908.826	908.3
L535	N533	N532	0.375	101.803	0.113	905.953	905.838	909.942	908.826
L536	N534	N533	0.25	114.3	0.35	906.58	905.953	909.68	909.942
L537	N535	N534	0.2	83.82	0.322	906.85	906.58	909.87	909.68
L538	N536	N535	0.2	29.56	1.184	907.2	906.85	909.98	909.87
L539	N537	N536	0.2	55.46	0.216	907.32	907.2	910.84	909.98
L54	N54	N55	0.2	55.78	1.452	924.56	923.75	928.12	926.41
L540	N538	N537	0.2	111.56	0.242	907.59	907.32	910.54	910.84
L541	N539	N538	0.2	87.17	0.62	908.13	907.59	911.04	910.54
L542	N540	N539	0.2	85.34	1.301	909.24	908.13	911.95	911.04
L545	N543	N542	0.2	118.87	1.237	913.13	911.66	916.07	914.47
L546	N544	N543	0.2	118.87	2.002	915.51	913.13	918.69	916.07
L547	N545	N544	0.2	79.86	1.49	916.7	915.51	919.65	918.69
L548	N546	N545	0.2	70.1	0.97	917.38	916.7	920.29	919.65
L549	N549	N537	0.2	87.17	1.641	908.75	907.32	912.08	910.84
L55	N55	N56	0.2	110.5	2.154	922.78	920.4	926.41	923.24
L550	N550	N549	0.2	85.34	1.219	909.79	908.75	913.45	912.08
L551	N550	N551	0.2	56	0.607	909.79	909.45	913.45	913.12
L552	N551	N552	0.2	86.31	1.24	909.45	908.38	913.12	912
L553	N552	N536	0.2	86.31	1.367	908.38	907.2	912	909.98
L554	N551	N553	0.2	80	0.788	909.45	908.82	913.12	912.4
L555	N553	N554	0.2	86.26	0.545	908.82	908.35	912.4	911.4
L556	N554	N534	0.2	86.26	2.052	908.35	906.58	911.4	909.68
L557	N555	N553	0.2	86.26	0.603	909.34	908.82	913	912.4
L558	N556	N555	0.2	86.26	0.754	909.99	909.34	915	913
L559	N557	N556	0.2	98.51	2.507	912.46	909.99	915.7	915
L56	N46	N58	0.2	111.3	0.5	923.837	923.28	926.5	925.9
L560	N558	N556	0.2	172.21	0.568	910.968	909.99	916.15	915
L565	N562	N558	0.2	86.81	0.223	911.161	910.968	915.3	916.15
L566	N563	N562	0.2	86.5	0.29	911.412	911.161	915.5	915.3
L567	N564	N563	0.2	79.57	0.29	911.643	911.412	915.7	915.5
L568	N564	N565	0.25	3.33	0.39	911.643	911.63	915.7	915.8
L569	N565	N566	0.2	80.61	1.507	911.607	910.392	915.8	913.5
L57	N59	N58	0.2	106.68	0.075	923.36	923.28	926.68	925.9
L570	N566	N567	0.3	84.141	0.317	910.392	910.125	913.5	914.4
L571	N567	N568	0.3	91	0.234	910.125	909.912	914.4	913.7
L572	N574	N573	0.3	89.965	0.586	908.858	908.331	914.575	914.29

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L574	N573	N575	0.3	86.258	0.34	908.331	908.038	914.29	913.7
L575	N575	N576	0.3	84.758	0.496	908.038	907.597	913.7	911.094
L576	N576	N577	0.375	86.26	1.316	907.597	906.414	911.094	910.16
L577	N577	N533	0.375	86.26	0.534	906.414	905.953	910.16	909.942
L578	N578	N566	0.2	57.5	0.543	910.704	910.392	913.8	913.5
L579	N579	N564	0.2	89.307	0.355	911.96	911.643	915	915.7
L58	N60	N59	0.2	105	5.638	929.28	923.36	932.2	926.68
L580	N580	N579	0.2	107.6	0.316	912.3	911.96	915.9	915
L581	N581	N580	0.2	86	0.634	912.845	912.3	915.15	915.9
L582	N582	N581	0.2	25.5	1.365	913.293	912.945	915.55	915.15
L583	N583	N582	0.2	58.061	1.516	914.2	913.293	917.4	915.55
L584	N584	N583	0.2	48.727	0.55	914.468	914.2	917.5	917.4
L585	N585	N581	0.25	86.258	0.165	912.987	912.845	915.7	915.15
L587	N587	N586	0.2	46.634	0.36	913.657	913.489	917	916.5
L588	N588	N587	0.2	120.4	2	916.065	913.657	920	917
L589	N589	N588	0.2	112.47	1	917.19	916.065	920.5	920
L59	N61	N60	0.2	71.02	0.887	929.91	929.28	932.64	932.2
L590	N590	N589	0.2	173.43	0.381	917.85	917.19	920.8	920.5
L591	N591	N590	0.2	110.03	0.373	918.26	917.85	921.5	920.8
L592	N592	N591	0.2	114.3	0.42	918.74	918.26	921.8	921.5
L593	N593	N592	0.2	81.08	0.432	919.09	918.74	922	921.8
L594	N594	N593	0.2	97.54	0.4	919.48	919.09	922.2	922
L595	N595	N594	0.15	30.78	1.14	919.831	919.48	922.6	922.2
L596	N596	N595	0.2	98.45	1.402	921.21	919.83	924.2	922.6
L597	N597	N596	0.2	98.3	0.96	922.154	921.21	925.3	924.2
L599	N600	N594	0.2	58.22	1.786	920.52	919.48	923	922.2
L6	N5	N6	0.2	93.5	1.37	932	931.279	935.6	934.4
L60	N58	N62	0.2	114	0.807	923.28	922.36	925.9	925.02
L600	N604	N600	0.2	18.1	5.525	921.52	920.52	922.5	923
L601	N609	N604	0.2	88.39	2.8	924	921.52	925.5	922.5
L602	N607	N609	0.2	103.02	2.5	926.58	924	930	925.5
L603	N610	N607	0.2	85.04	0.8	927.26	926.58	931.5	930
L604	N611	N610	0.2	104.55	3.8	931.23	927.26	934.18	931.5
L605	N612	N611	0.2	80.16	2.844	933.51	931.23	936	934.18
L606	N613	N612	0.2	65.5	1.221	934.31	933.51	937	936
L608	N615	N607	0.2	89.916	0.31	926.86	926.58	929.5	930
L609	N616	N615	0.2	89.92	0.47	927.28	926.86	930	929.5
L61	N63	N62	0.2	93	0.822	923.124	922.36	925.9	925.02
L610	N617	N616	0.2	140.82	0.44	927.9	927.28	931	930
L611	N618	N616	0.2	84.12	2.05	929	927.28	932	930
L612	N619	N618	0.2	106.68	0.47	929.5	929	933	932
L613	N620	N600	0.2	123.75	2.62	923.762	920.52	926.2	923
L614	N621	N620	0.2	124.97	0.37	924.222	923.76	927.7	926.2
L615	N622	N621	0.2	120.4	0.5	924.822	924.22	928.3	927.7
L616	N623	N622	0.2	117.96	0.382	925.27	924.82	927.9	928.3
L617	N624	N623	0.2	119.79	0.37	925.713	925.27	928.5	927.9
L618	N625	N600	0.25	188.87	0.35	921.181	920.52	924	923
L619	N628	N625	0.25	122.53	0.28	921.523	921.18	924	924
L620	N629	N628	0.25	121	0.24	921.81	921.52	925.3	924
L621	N630	N629	0.2	73.46	0.3	922.03	921.81	925.5	925.3
L622	N631	N630	0.2	110.95	0.4	922.474	922.03	926	925.5
L623	N632	N631	0.2	121.31	1.525	924.32	922.47	927.5	926
L624	N633	N632	0.2	121.92	1.25	925.844	924.32	928.5	927.5
L625	N634	N633	0.2	111.86	1.243	927.23	925.84	929.5	928.5
L626	N635	N634	0.2	66.14	0.44	927.521	927.23	930	929.5
L627	N636	N634	0.2	60.96	3.2	929.181	927.23	930.9	929.5
L628	N637	N636	0.2	79.86	3.206	931.74	929.18	933.9	930.9
L629	N638	N637	0.2	57	2.825	933.35	931.74	936.1	933.9
L630	N639	N638	0.2	71.93	0.49	933.702	933.35	936.2	936.1
L631	N640	N637	0.2	96.32	0.41	932.135	931.74	935	933.9

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L632	N641	N640	0.2	36.58	4.893	933.92	932.13	935	935
L633	N642	N640	0.2	346.79	0.41	933.552	932.13	935	935
L634	N643	N586	0.2	123.834	2.506	916.592	913.489	920.15	916.5
L635	N646	N645	0.2	108.2	0.4	918.403	917.97	923.28	921
L636	N647	N645	0.2	26.99	2.823	918.732	917.97	920.5	921
L637	N648	N647	0.2	85.34	3.25	921.732	918.958	923.942	920.5
L638	N649	N648	0.2	98.31	1.173	922.885	921.732	925.3	923.942
L639	N650	N649	0.2	103.32	0.36	923.257	922.885	925.695	925.3
L64	N66	N63	0.2	86.3	1.81	924.686	923.124	927.4	925.9
L642	N653	N652	0.2	101.5	1.44	925.638	924.176	927.6	927.224
L643	N654	N653	0.2	100.58	0.382	926.022	925.638	930.188	927.6
L644	N655	N654	0.2	85.34	0.55	926.491	926.022	930.5	930.188
L645	N656	N652	0.2	161.37	0.41	924.838	924.176	927.59	927.224
L647	N658	N656	0.2	100.28	0.252	925.09	924.838	929.18	927.59
L648	N659	N658	0.2	42.79	1.5	925.732	925.09	930	929.18
L649	N660	N658	0.2	92.96	0.57	925.62	925.09	929.11	929.18
L65	N67	N66	0.2	86.3	1.81	926.249	924.686	929	927.4
L650	N661	N660	0.2	106.68	0.619	926.28	925.62	929.09	929.11
L651	N662	N661	0.2	106.68	1.228	927.59	926.28	930.52	929.09
L652	N652	N680	0.2	110.1	0.403	924.176	923.732	927.224	926.551
L653	N680	N650	0.2	109.73	0.433	923.732	923.257	926.551	925.695
L655	N678	N680	0.2	96.32	1.625	925.297	923.732	927.507	926.551
L656	N687	N688	0.2	10	5	909.274	908.774	912	911.4
L657	N690	N691	0.2	27.31	0.4	905.139	905.03	907.9	907.8
L658	N692	N691	0.2	83.33	1.3	906.113	905.03	908.9	907.8
L659	N688	N692	0.2	54.85	4.85	908.774	906.113	911.4	908.9
L66	N68	N67	0.2	71	0.6	926.674	926.249	929.4	929
L660	N691	N693	0.2	68.49	6.37	905.03	900.667	907.8	903.3
L661	N694	N693	0.2	97	0.4	901.055	900.667	903.7	903.3
L662	N695	N694	0.2	97	0.4	901.443	901.055	904.1	903.7
L663	N696	N695	0.2	97	0.4	901.831	901.443	904.5	904.1
L664	N697	N696	0.2	97	0.4	902.219	901.831	904.9	904.5
L665	N698	N697	0.2	50	0.37	902.404	902.219	905	904.9
L666	N699	N698	0.2	132.65	0.42	902.961	902.404	905.6	905
L667	N700	N699	0.2	45.63	0.71	903.285	902.961	905.9	905.6
L668	N701	N700	0.2	46.02	0.6	903.561	903.285	906.2	905.9
L669	N693	N702	0.2	62.18	0.4	900.667	900.419	903.3	903.1
L67	N69	N68	0.2	60	1.81	927.76	926.674	931	929.4
L670	N702	N703	0.2	60	0.4	900.419	900.179	903.1	902.8
L671	N703	N704	0.2	60	0.4	900.179	899.939	902.8	902.7
L672	N704	N705	0.2	60	0.4	899.939	899.699	902.7	902.4
L673	N705	N706	0.2	60	0.4	899.699	899.459	902.4	902.2
L674	N707	N706	0.675	98.27	0.179	899.635	899.46	902.5	902.2
L675	N708	N707	0.2	60	1.6	900.595	899.635	903.3	902.5
L676	N709	N708	0.2	56	3.4	902.499	900.595	905.2	903.3
L677	N710	N709	0.2	26.4	5	903.819	902.499	906.6	905.2
L678	N711	N706	0.2	53	0.4	899.671	899.459	902.2	902.2
L679	N706	N712	0.625	119.04	0.25	899.459	899.161	902.2	901.9
L68	N69	N49	0.2	75.5	0.4	927.76	927.458	931	930.2
L680	N712	N713	0.675	119.04	0.25	899.161	898.863	901.9	901.9
L681	N713	N714	0.6	103.61	0.18	898.863	898.677	901.9	901.3
L682	N714	N715	0.9	68.6	0.18	898.677	898.553	901.3	901.1
L683	N715	N716	0.9	104.13	0.18	898.553	898.366	901.1	901.2
L684	N716	N717	1.05	102.78	0.18	898.366	898.181	901.2	900.8
L685	N717	N457	1.05	11.76	0.179	898.181	898.16	900.8	900.8
L686	N718	N457	1.05	110.64	0.216	898.399	898.16	901	900.8
L687	N719	N718	0.525	107.76	0.18	898.584	898.39	901.2	901
L688	N720	N719	0.525	69.47	0.18	898.709	898.584	901.3	901.2
L689	N494	N720	0.525	126.97	0.18	898.938	898.709	901.6	901.3
L690	N721	N565	0.2	84.077	1.557	912.916	911.607	917.78	915.8

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L691	N722	N721	0.2	106.713	0.385	913.332	912.921	918	917.78
L692	N723	N722	0.2	110	0.44	913.816	913.332	917.6	918
L693	N724	N721	0.2	50	2.582	915.999	914.708	917.1	917.78
L694	N725	N726	0.25	76	0.267	899.461	899.258	902.421	902.84
L695	N727	N725	0.2	92.4	1.5	900.847	899.461	903.6	902.421
L696	N728	N727	0.2	265.68	1.5	904.832	900.847	907.6	903.6
L697	N729	N730	0.25	89.92	0.588	900.84	900.311	903.5	903.113
L698	N730	N731	0.25	90	0.027	900.311	900.287	903.113	904.151
L699	N731	N732	0.25	85	0.638	900.287	899.745	904.151	903.883
L7	N7	N6	0.2	48.6	0.642	931.591	931.279	934.89	934.4
L70	N70	N71	0.2	64.08	0.468	920.99	920.69	923.54	923.33
L700	N732	N725	0.25	86.5	0.328	899.745	899.461	903.883	902.421
L701	N733	N550	0.2	86.26	2.168	911.66	909.79	914.61	913.45
L702	N734	N733	0.2	86.26	2.122	913.49	911.66	916.53	914.61
L703	N735	N218	0.2	44.91	1.603	928.82	928.1	932.33	931.52
L704	N736	N217	0.2	37.17	1.345	928.33	927.83	931.65	930.79
L705	N478	N480	0.2	120.46	1.5	917.137	915.33	920.8	918.5
L706	N737	N187	0.2	93.18	2.372	954.62	952.41	957.64	955.5
L707	N739	N173	0.2	85.52	0.75	930.541	929.9	935	930.4
L708	N740	N146	0.2	100	3.576	925.583	922.007	929.4	925.5
L709	N147	N740	0.2	72	4.783	929.027	925.583	932.65	929.4
L71	N71	N72	0.2	57.6	0.556	920.69	920.37	923.33	923.6
L710	N35	N50	0.2	95	1.516	928.664	927.224	933.058	930.6
L711	N27	N51	0.2	81.7	2.48	928.7	926.674	931.89	929.9
L712	N741	N742	0.3	10	0.3	918.861	918.831	921.37	921.37
L713	N742	N743	0.3	36.88	0.093	918.831	918.797	921.37	920.437
L714	N743	N744	0.3	19.41	0.333	918.797	918.733	920.437	920.01
L715	N744	N745	0.3	41.47	0.3	918.733	918.608	920.01	921.348
L716	N745	N746	0.3	125.16	0.3	918.608	918.233	921.348	919.5
L717	N746	N747	0.3	43.01	0.3	918.233	918.104	919.5	919.6
L718	N747	N748	0.3	72.45	0.3	918.104	917.886	919.6	919.5
L719	N748	N749	0.3	66.14	0.3	917.886	917.688	919.5	919.1
L72	N73	N72	0.2	121.9	1.8	922.5	920.31	925.8	923.6
L720	N749	N750	0.3	37.59	0.3	917.688	917.575	919.1	919.66
L721	N750	N141	0.3	25.14	0.3	917.575	917.5	919.66	922.46
L728	N726	N454	1.05	150.64	0.016	898.384	898.36	902.84	901.8
L729	N452	N726	1.05	158.34	0.067	898.49	898.384	904.94	902.84
L73	N72	N74	0.2	113.36	0.242	920.31	920.036	923.6	922.8
L730	N441	N756	0.9	24.3	0.412	899.93	899.83	905.08	905.08
L731	N756	N443	0.9	24.3	2.263	899.83	899.28	905.08	905.31
L732	N757	N2	0.2	86.921	1.532	934.706	933.374	938.5	937.1
L733	N757	N758	0.2	77.847	0.65	934.706	934.2	938.5	938
L734	N758	N15	0.2	107.668	0.669	934.2	933.48	938	937.2
L735	N367	N759	0.2	86.258	0.4	908.829	908.484	911.35	911.92
L736	N759	N364	0.2	71.018	0.607	908.484	908.053	911.92	911.4
L737	N760	N225	0.2	98.76	3.362	927.2	923.88	930.74	927.99
L738	N761	N760	0.2	67.06	1.432	928.16	927.2	931.59	930.74
L739	N761	N219	0.2	46.66	0.171	928.16	928.08	931.59	932.08
L74	N75	N74	0.2	52.73	0.4	920.246	920.036	923	922.8
L740	N223	N762	0.2	81.99	2.317	931.34	929.44	934.49	932.87
L741	N762	N224	0.2	81.99	3.891	929.44	926.25	932.87	929.7
L742	N568	N763	0.3	79.248	0.604	909.912	909.433	913.7	913.15
L743	N763	N574	0.3	89.306	0.644	909.433	908.858	913.15	914.575
L744	N764	N765	0.2	83.5	1.565	915.741	914.434	918.85	917.4
L745	N765	N766	0.2	81	3.205	914.434	911.838	917.4	916.2
L746	N766	N558	0.2	16.2	5.154	911.803	910.968	916.2	916.15
L749	N586	N585	0.25	110	0.456	913.489	912.987	916.5	915.7
L75	N76	N75	0.2	67.97	0.4	920.518	920.246	923.3	923
L750	N116	N768	0.2	80.25	2.455	922.72	920.75	925.71	924.15
L752	N208	N769	0.25	66.39	0.467	915.06	914.75	920.59	919.52

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L753	N769	N207	0.25	66.87	2.408	914.75	913.14	919.52	918.79
L756	N115	N116	0.2	76.2	0.8	923.39	922.78	927.66	925.71
L757	N534	N772	0.375	101.8	0.737	906.58	905.83	909.68	908.72
L758	N772	N531	0.4	89	0.494	905.83	905.39	908.72	910.8
L759	N24	N190	0.2	94.18	0.361	923.84	923.5	927.22	926.09
L76	N77	N76	0.2	56.39	0.4	920.744	920.518	923.5	923.3
L764	N192	N55	0.2	114.3	0.367	923.2	922.78	925.9	926.41
L765	N190	N191	0.2	86	0.203	923.5	923.325	926.09	926.1
L766	N190	N115	0.2	114.3	0.096	923.5	923.39	926.09	927.66
L767	N510	N775	0.375	55.78	0.359	907.04	906.84	909.71	909.7
L768	N775	N528	0.375	63.4	1.42	906.84	905.94	909.7	909.68
L77	N78	N74	0.2	114.91	0.317	920.4	920.036	923.5	922.8
L770	N504	N777	0.2	86.26	1.252	909.22	908.14	911.9	910.38
L771	N777	N510	0.2	86.26	1.275	908.14	907.04	910.38	909.71
L772	N283	N778	0.2	124.66	0.409	918.47	917.96	922.77	921.31
L773	N778	N270	0.2	124.66	2.92	917.96	914.32	921.31	918.06
L774	N542	N540	0.2	27.43	6.744	911.09	909.24	914.47	911.95
L775	N415	N410	0.2	45.72	0.459	906.56	906.35	910.24	910
L776	N779	N780	0.2	64.01	0.313	932.47	932.27	934	933
L777	N780	N611	0.2	80.77	0.718	932.27	931.69	933	934.18
L779	N341	N781	0.6	52	2.357	901.801	900.575	905.27	903.7
L78	N74	N79	0.2	111.56	0.403	920.036	919.586	922.8	923.5
L780	N781	N441	0.6	140	0.461	900.575	899.93	903.7	905.08
L781	N82	N782	0.2	83.52	0.85	918.046	917.336	921.8	920.5
L782	N782	N83	0.2	48.77	0.75	917.336	916.97	920.5	920
L783	N240	N783	0.2	90	0.6	926.67	926.13	929.34	928.88
L784	N783	N239	0.2	91.189	2.27	926.13	924.06	928.88	926.55
L785	N204	N784	0.2	80.16	0.449	914.01	913.65	917.2	917
L786	N784	N206	0.2	80.16	0.237	913.65	913.46	917	917.9
L787	N785	N400	0.2	90.09	0.78	908.823	908.12	912	911.25
L788	N786	N646	0.2	83.82	0.44	918.772	918.403	923.5	923.28
L789	N787	N194	0.2	83.82	1.408	920.18	919	923.27	923
L79	N80	N79	0.2	91.68	2.154	921.56	919.586	924.5	923.5
L795	N207	N521	0.2	121.92	0.197	913.14	912.9	918.79	918.79
L796	N768	N789	0.2	80.25	1.271	920.75	919.73	924.15	922.34
L797	N789	N117	0.2	80.25	3.402	919.73	917	922.34	922.52
L8	N8	N7	0.2	47.323	1.456	932.28	931.591	935.79	934.89
L80	N79	N81	0.2	56.17	2.172	919.586	918.366	923.5	921.72
L800	N790	N791	0.2	87.6	0.673	914.03	913.44	917.3	916.8
L801	N791	N792	0.2	100	0.64	913.44	912.8	916.8	917.85
L803	N793	N794	0.375	77.5	0.477	912	911.63	918.75	917.3
L804	N794	N795	0.375	77.5	0.555	911.63	911.2	917.3	915.9
L805	N135	N795	0.375	108.5	0.756	912.02	911.2	917.2	915.9
L807	N797	N796	0.2	27	0.9	909.233	908.99	914	913.05
L808	N796	N515	0.375	7.5	2	908.99	908.84	913.05	911.69
L809	N795	N798	0.375	98.5	1.746	911.2	909.48	915.9	914.2
L81	N81	N82	0.2	78.3	0.409	918.366	918.046	921.72	921.8
L810	N798	N796	0.375	86.5	0.567	909.48	908.99	914.2	913.05
L813	N792	N799	0.2	45.2	0.553	912.8	912.55	917.85	918.73
L814	N799	N793	0.2	12.5	2.16	912.55	912.28	918.73	918.75
L815	N117	N207	0.3	92.6	4.168	917	913.14	922.52	918.79
L816	N200	N800	0.2	40.9	0.72	914.4	914.106	917.54	917.5
L817	N800	N204	0.2	24	0.398	914.106	914.01	917.5	917.2
L818	N800	N790	0.2	6.5	1.162	914.106	914.03	917.5	917.3
L820	N645	N643	0.2	77.52	1.778	917.97	916.592	921	920.15
L821	N457	N802	1.05	10	1.6	898.16	898	900.8	900.8
L83	N84	N83	0.2	86.525	1.156	918.97	917.97	922.36	920
L84	N85	N84	0.2	167.5	0.4	919.64	918.97	922.6	922.36
L85	N86	N85	0.2	64.01	0.4	919.896	919.64	923.6	922.6
L86	N87	N86	0.2	97.54	0.4	920.286	919.896	923.9	923.6
L87	N88	N87	0.2	123.44	0.4	920.78	920.286	923.3	923.9

Name	Upstream Node Name	Downstream Node Name	Diameter (m)	Length (m)	Conduit Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Upstream Ground Elevation (m)	Downstream Ground Elevation (m)
L88	N62	N91	0.2	114.6	0.803	922.36	921.44	925.02	924.2
L89	N91	N70	0.2	75.33	0.597	921.44	920.99	924.2	923.54
L9	N14	N8	0.2	72	2.044	933.752	932.28	936.9	935.79
L90	N89	N91	0.2	73.15	2.119	922.99	921.44	925.7	924.2
L91	N92	N89	0.2	57.9	2.107	924.21	922.99	927	925.7
L92	N93	N55	0.2	82.9	1.436	923.97	922.78	927.5	926.41
L93	N94	N93	0.2	83.2	2.26	925.85	923.97	929.18	927.5
L94	N95	N94	0.2	52.8	0.8	926.27	925.85	929.6	929.18
L95	N96	N95	0.2	86.9	0.8	926.97	926.27	930.3	929.6
L96	N97	N94	0.2	91.63	0.578	926.38	925.85	929.1	929.18
L97	N98	N97	0.2	100.98	2.644	929.05	926.38	930.07	929.1
L98	N99	N56	0.2	120.7	4.681	926.21	920.56	928.73	923.24
L99	N56	N100	0.2	110.03	2.654	920.4	917.48	923.24	920.13

Appendix F

Sanitary Sewer System Model Calibration

Figure F.1 - Dry Weather Flow Calibration

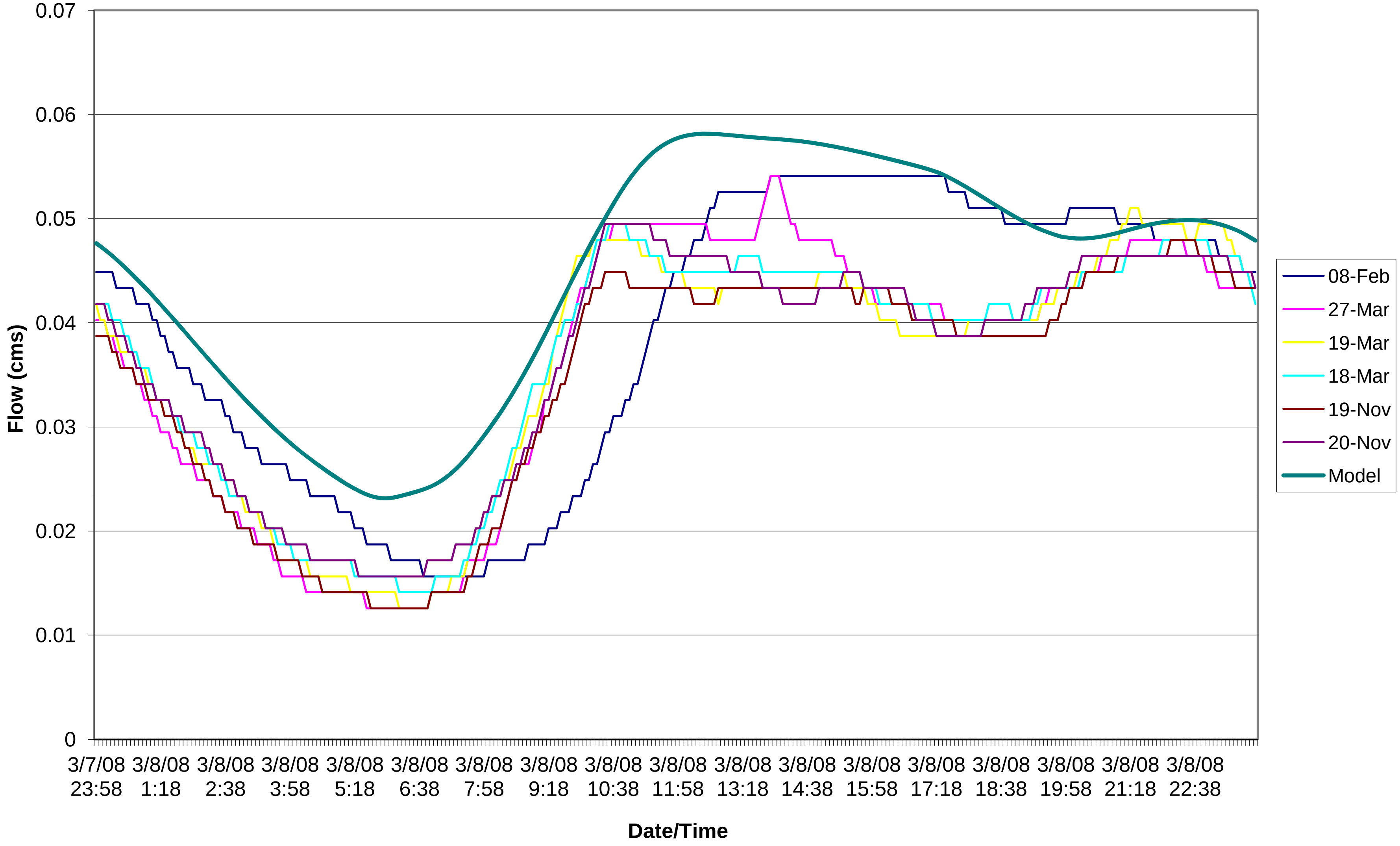
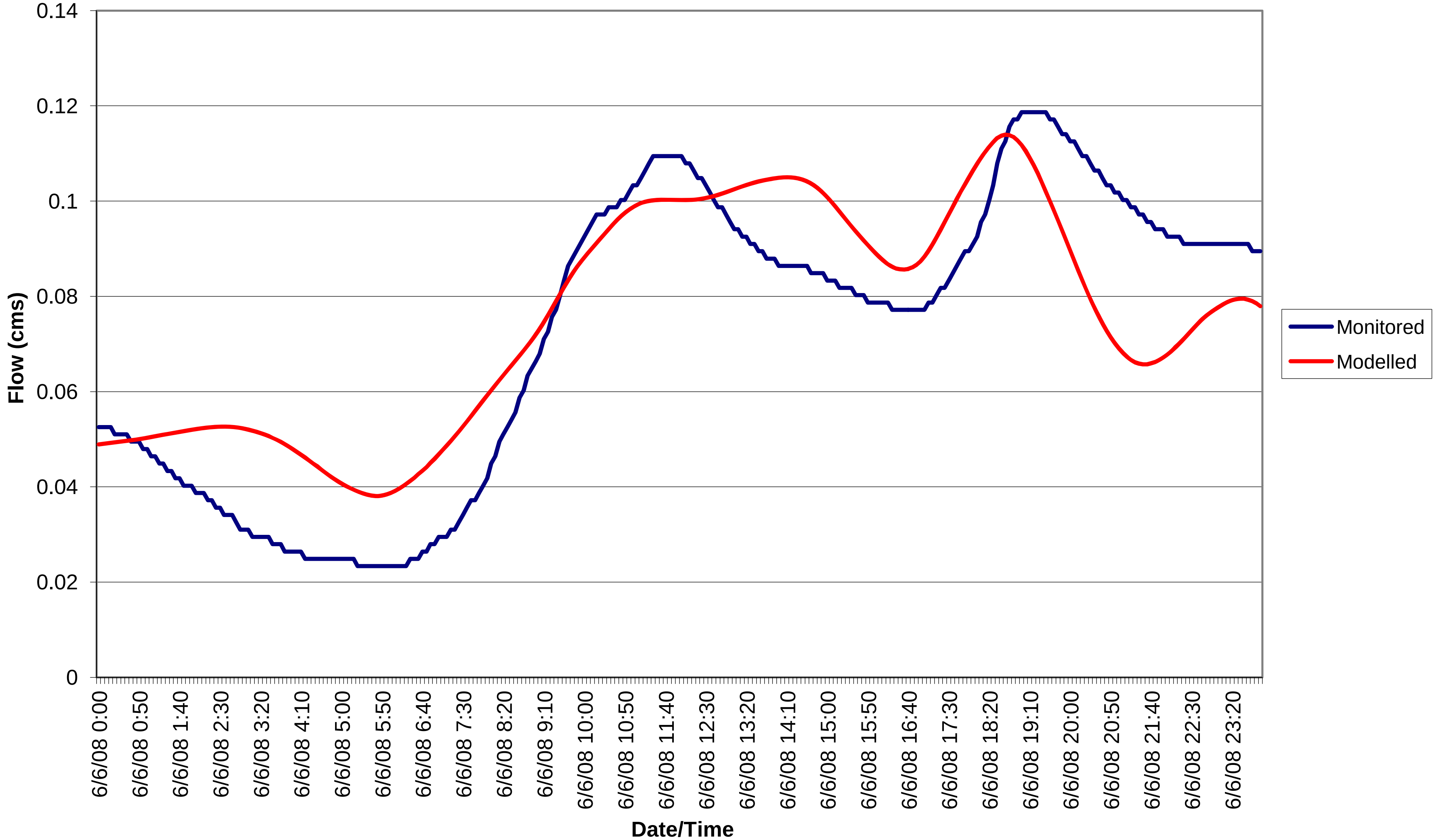


Figure F.2 - Wet Weather Flow Calibration
June 6, 2008



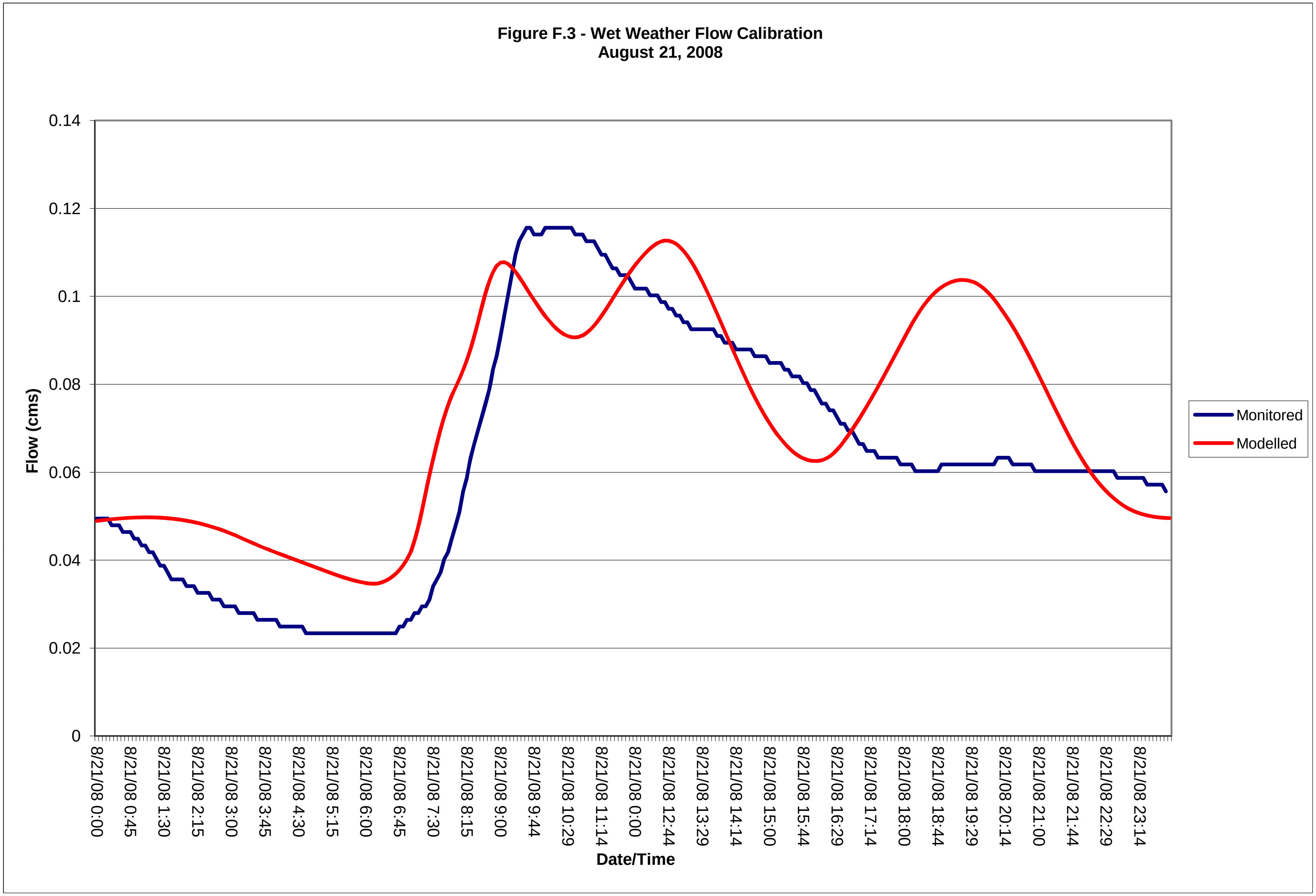


Figure F.3 - Wet Weather Flow Calibration
August 21, 2008
Flow (cms)
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
8/21/08 0:00
8/21/08 0:45
8/21/08 1:30
8/21/08 2:15
8/21/08 3:00
8/21/08 3:45
8/21/08 4:30
8/21/08 5:15
8/21/08 6:00
8/21/08 6:45
8/21/08 7:30
8/21/08 8:15
8/21/08 9:00
8/21/08 9:44
8/21/08 10:29
8/21/08 11:14
8/21/08 0:00
8/21/08 12:44
8/21/08 13:29
8/21/08 14:14
8/21/08 15:00
8/21/08 15:44
8/21/08 16:29
8/21/08 17:14
8/21/08 18:00
8/21/08 18:44
8/21/08 19:29
8/21/08 20:14
8/21/08 21:00
8/21/08 21:44
8/21/08 22:29
8/21/08 23:14
Date/Time
Monitored
Modelled

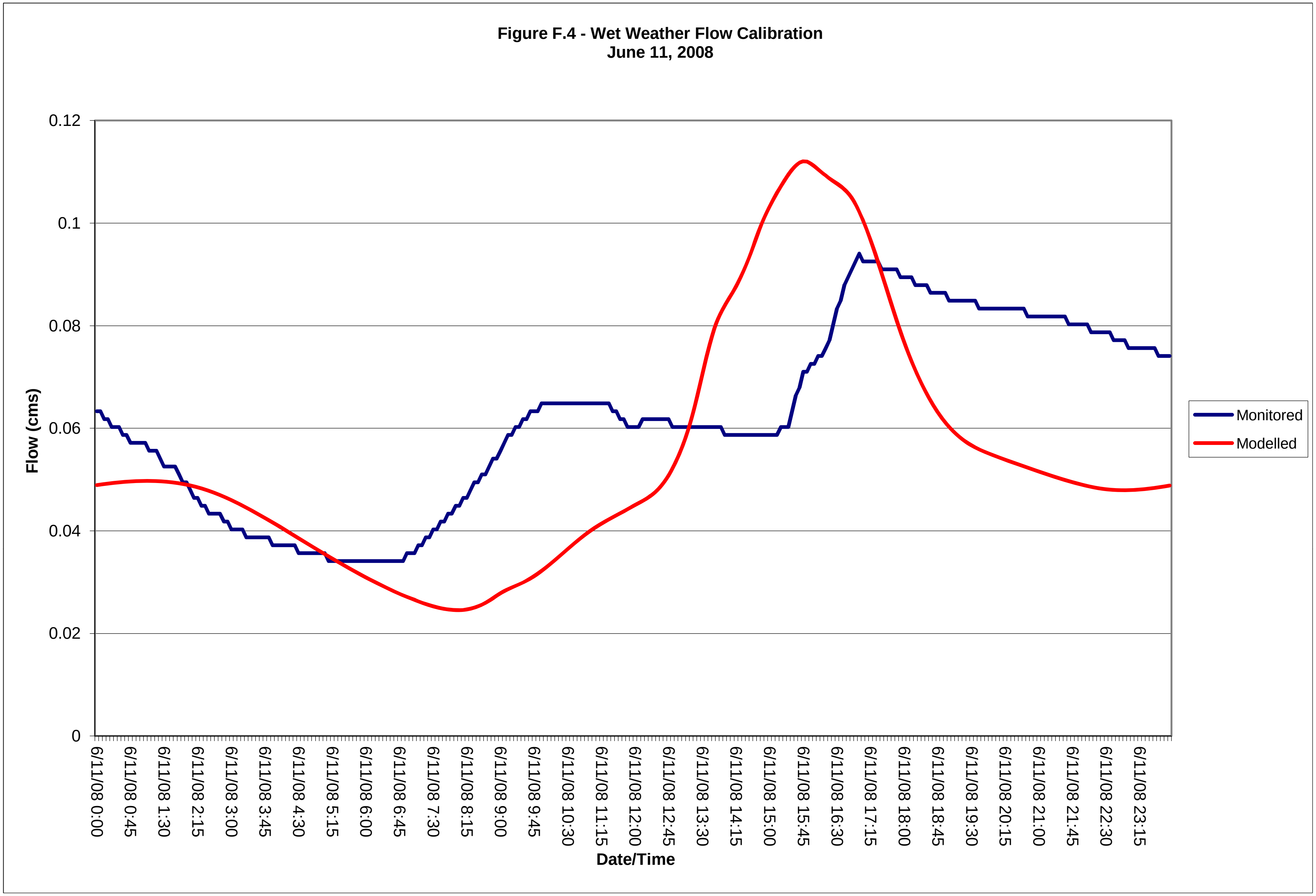


Figure F.4 - Wet Weather Flow Calibration
June 11, 2008
Flow (cms)
0.12
0.1
0.08
0.06
0.04
0.02
0
6/11/08 0:00
6/11/08 0:45
6/11/08 1:30
6/11/08 2:15
6/11/08 3:00
6/11/08 3:45
6/11/08 4:30
6/11/08 5:15
6/11/08 6:00
6/11/08 6:45
6/11/08 7:30
6/11/08 8:15
6/11/08 9:00
6/11/08 9:45
6/11/08 10:30
6/11/08 11:15
6/11/08 12:00
6/11/08 12:45
6/11/08 13:30
6/11/08 14:15
6/11/08 15:00
6/11/08 15:45
6/11/08 16:30
6/11/08 17:15
6/11/08 18:00
6/11/08 18:45
6/11/08 19:30
6/11/08 20:15
6/11/08 21:00
6/11/08 21:45
6/11/08 22:30
6/11/08 23:15
Date/Time
Monitored
Modelled

Appendix G

Cost Estimates – Sanitary Sewer System

Appendix G
Table G.1 - Edson Improvements - Phase 1
5 Year 4 Hour - HGL to 1.0 m below ground and below

Main trunk - 1st Avenue

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L348	N349	N348	903.22	902.944	172.5	0.2	0.45	908.85	908.5	6	900	$3,490	$602,095	900	$3,490	$602,095
L347	N348	N347	902.944	902.668	172.5	0.2	0.45	908.5	908.2	6	900	$3,490	$602,095	900	$3,490	$602,095
L346	N347	N346	902.668	902.392	172.5	0.2	0.45	908.2	908	6	900	$3,490	$602,095	900	$3,490	$602,095
L345	N346	N345	902.392	902.216	172.6	0.1	0.45	908	907.8	6	900	$3,490	$602,339	900	$3,490	$602,339
L344	N345	N344	902.216	901.94	172.5	0.2	0.45	907.8	906	5	900	$2,570	$443,376	900	$2,570	$443,376
L343	N344	N342	901.94	901.9	141.7	0.0	0.45	906	905	4	900	$2,150	$304,720	900	$2,150	$304,720
L340	N342	N341	901.9	901.8	67.9	0.1	0.45	905	905.27	3	900	$1,935	$131,464	900	$1,935	$131,464
					1072.3							Sub-Total	$3,288,183			$3,288,183
												Contingency (25%)	$822,046			$822,046
												Engineering and Administration (10%)	$328,818			$328,818
												CAPITAL COST	$4,440,000			$4,440,000

42nd Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L266	N267	N266	911	910.37	111.5	0.6	0.2	913.04	914.55	3	375	$710	$79,193	375	$710	$79,193
L295	N266	N297	910.37	908.61	111.6	1.6	0.2	914.55	912.27	4	375	$825	$92,037	375	$825	$92,037
L302	N297	N304	908.61	907.68	111.5	0.8	0.2	912.27	910.5	3	375	$710	$79,193	375	$710	$79,193
L311	N304	N313	907.68	904.93	111.6	2.5	0.2	910.5	907.542	3	375	$710	$79,250	375	$710	$79,250
L312	N313	N314	904.723	902.448	103.8	2.2	0.2	907.542	905.1	3	375	$710	$73,677	375	$710	$73,677
					550.0							Sub-Total	$403,351			$403,351
												Contingency (25%)	$100,838			$100,838
												Engineering and Administration (10%)	$40,335			$40,335
												CAPITAL COST	$545,000			$545,000

49th Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L373	N373	N372	915.975	914.78	97.1	1.2	0.2	919	917.13	3	375	$710	$68,974	375	$710	$68,974
L372	N372	N371	913.18	910.74	148.8	1.6	0.25	917.13	914.09	4	375	$825	$122,744	300	$575	$85,549
L371	N371	N370	910.74	909.68	111.9	0.9	0.25	914.09	911.96	3	375	$710	$79,472	300	$460	$51,489
L370	N370	N350	909.68	908.61	113.0	0.9	0.25	911.96	910.51	3	375	$710	$80,221	300	$460	$51,974
					470.8							Sub-Total	$351,410			$257,985
												Contingency (25%)	$87,853			$64,496
												Engineering and Administration (10%)	$35,141			$25,799
												CAPITAL COST	$475,000			$349,000

51st Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L250	N248	N251	921.87	921.41	112.2	0.4	0.25	924.72	924.58	3	450	$960	$107,707	375	$710	$79,658
L381	N251	N378	921.41	919.89	194.9	1.1	0.25	924.58	922.98	3	450	$960	$187,081	375	$710	$138,362
					307.1							Sub-Total	$294,788			$218,020
												Contingency (25%)	$73,697			$54,505
												Engineering and Administration (10%)	$29,479			$21,802
												CAPITAL COST	$398,000			$295,000

52 Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L101	N100	N102	917.48	917.04	111.6	0.4	0.2	920.13	919.6	3	300	$460	$51,318	250	$350	$39,046
L104	N102	N105	917	916.55	111.5	0.4	0.2	919.6	918.9	3	300	$460	$51,304	250	$350	$39,036
					223.1							Sub-Total	$102,621			$78,082
												Contingency (25%)	$25,655			$19,520
												Engineering and Administration (10%)	$10,262			$7,808
												CAPITAL COST	$139,000			$106,000

Appendix G
Table G.1 - Edson Improvements - Phase 1
5 Year 4 Hour - HGL to 1.0 m below ground and below

53rd Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L497	N495	N500	909.62	909.3	81.2	0.5	0.25	912.42	911.63	3	525	$1,000	$81,188	525	$1,000	$81,188
												Contingency (25%)	$20,297			$20,297
												Engineering and Administration (10%)	$8,119			$8,119
												CAPITAL COST	$110,000			$110,000

55/54 Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L509	N512	N511	907.71	907.35	111.5	0.3	0.375	910.3	909.96	3	525	$1,000	$111,540	375	$710	$79,193
L508	N511	N510	907.35	907.07	89.2	0.3	0.375	909.96	909.71	3	525	$1,000	$89,230	375	$710	$63,353
L767	N510	N775	907.04	906.84	55.8	0.4	0.375	909.71	909.7	3	525	$1,000	$55,780	375	$710	$39,604
L768	N775	N528	906.84	905.94	63.4	1.4	0.375	909.7	909.68	3	525	$1,000	$63,400	375	$710	$45,014
L531	N528	N530	905.94	905.57	85.3	0.4	0.375	909.68	911	5	525	$1,550	$132,277	375	$965	$82,353
L532	N530	N531	905.57	905.34	113.7	0.2	0.375	911	910.8	5	525	$1,550	$176,220	375	$965	$109,711
					519.0							Sub-Total	$628,447			$419,228
												Contingency (25%)	$157,112			$104,807
												Engineering and Administration (10%)	$62,845			$41,923
												CAPITAL COST	$849,000			$566,000

55 street flow equalization pipe

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L822	N207	N793	913.16	912.4	10.2	7.5	NA	918.79	918.75	6	300	$580	$5,916	250	$570	$5,814
												Contingency (25%)	$1,479			$1,454
												Engineering and Administration (10%)	$592			$581
												CAPITAL COST	$8,000			$8,000

70th Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L599	N600	N594	920.52	919.48	58.2	1.8	0.2	924.5	922.2	3	300	$460	$26,781	250	$350	$20,377
L823	N594	N807	919.48	917	154.0	1.6	NA	922.2	922	4	300	$575	$88,556	300	$575	$88,556
					212.2							Sub-Total	$115,337			$108,933
												Contingency (25%)	$28,834			$27,233
												Engineering and Administration (10%)	$11,534			$10,893
												CAPITAL COST	$156,000			$148,000

New pipe from West End Option (Alternative 2)

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L824	N807	N805	916	907.44	283.0	3.0	NA	922	913	6	375	$645	$182,561	375	$645	$182,561
L833	N805	N812	907.44	906.73	321.1	0.2	NA	913	912.5	6	375	$645	$207,122	375	$645	$207,122
L834	N812	N811	906.73	906.3	195.1	0.2	NA	912.5	915	7	375	$705	$137,517	375	$705	$137,517
L835	N811	N810	906.3	905.9	175.0	0.2	NA	915	910.5	7	375	$3,000	$525,000	375	$3,000	$525,000
L836	N810	N803	905.9	905.77	60.8	0.2	NA	910.5	911	5	375	$3,000	$182,250	375	$3,000	$182,250
L8282	N803	N425	905.77	902.51	1482.6	0.22	NA	911	907.59	5	375	$585	$867,309	375	$585	$867,309
					2517.6							Sub-Total	$2,101,760			$2,101,760
												Contingency (25%)	$525,440			$525,440
												Engineering and Administration (10%)	$210,176			$210,176
												CAPITAL COST	$2,838,000			$2,838,000

Appendix G
Table G.1 - Edson Improvements - Phase 1
5 Year 4 Hour - HGL to 1.0 m below ground and below

Upgrade pipe from west end (Alternative 1)

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L594	N594	N593	919.48	919.09	97.5	0.4	0.2	922.2	922	3	375	$710	$69,253	375	710	$69,253
L593	N593	N592	919.09	918.74	81.1	0.4	0.2	922	921.8	3	375	$710	$57,567	375	710	$57,567
L592	N592	N591	918.74	918.26	114.3	0.4	0.2	921.8	921.5	3	375	$710	$81,153	375	710	$81,153
L591	N591	N590	918.26	917.85	110.0	0.4	0.2	921.5	920.8	3	375	$710	$78,121	375	710	$78,121
L590	N590	N589	917.85	917.19	173.4	0.4	0.2	920.8	920.5	3	375	$710	$123,135	375	710	$123,135
L589	N589	N588	917.19	916.065	112.5	1.0	0.2	920.5	920	4	375	$825	$92,788	375	825	$92,788
L588	N588	N587	916.065	913.657	120.4	2.0	0.2	920	917	4	375	$825	$99,330	375	825	$99,330
L587	N587	N586	913.657	913.489	46.6	0.4	0.2	917	916.5	3	375	$710	$33,110	375	710	$33,110
L749	N586	N585	913.489	912.987	110.0	0.5	0.25	916.5	915.7	3	375	$710	$78,100	300	460	$50,600
L585	N585	N581	912.987	912.845	86.3	0.2	0.25	915.7	915.15	3	375	$710	$61,243	300	460	$39,679
L581	N581	N580	912.845	912.3	86.0	0.6	0.25	915.15	915.9	3	375	$710	$61,060	300	460	$39,560
L580	N580	N579	912.3	911.96	107.6	0.3	0.25	915.9	915	3	375	$710	$76,396	300	460	$49,496
L579	N579	N564	911.96	911.643	89.3	0.4	0.25	915	915.7	4	375	$825	$73,678	300	575	$51,352
L567	N564	N563	911.643	911.412	79.6	0.3	0.2	915.7	915.5	4	375	$825	$65,645	375	825	$65,645
					1414.6							Sub-Total	$1,050,580			$930,789
												Contingency (25%)	$262,645			$232,697
												Engineering and Administration (10%)	$105,058			$93,079
												CAPITAL COST	$1,419,000			$1,257,000

Phase 1 Summary	Length	Replacement Cost	Twinning Cost
Main trunk - 1st Avenue	1072	$3,288,183	$3,288,183
42nd Street	550	$403,351	$403,351
49th Street	471	$351,410	$257,985
51st Street	307	$294,788	$218,020
52 Street	223	$102,621	$78,082
53rd Street	81	$81,188	$81,188
55/54 Street	519	$628,447	$419,228
55 street flow equalization pipe	10	$5,916	$5,814
70th Street	212	$115,337	$108,933
Upgrade pipe from west end (Alternative 1)	1415	$1,050,580	$930,789
New pipe from West End (Alternative 2)	2518	$2,101,760	$2,101,760
Upsize new pipe for new development (Alternative 3)*	2518	$3,432,522	$3,432,522
Subtotal	5964	$8,703,763	$8,293,306
Engineering (10%) & Contingencies (25%)		$3,046,317	$2,902,657
Total		$11,750,080	$11,195,964

*detailed cost estimate shown in Table F.4

Appendix G
Table G.2 - Edson Improvements - Phase 2
24 Year 4 Hour - HGL to 1.0 m below ground and below

42nd Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L269	N270	N269	914.32	914.03	52.9	0.5	0.2	918.06	917.43	4	300	$575	$30,406	250	$550	$29,084
L268	N269	N268	914.03	912.63	59.3	2.4	0.2	917.43	916.35	4	300	$575	$34,103	250	$550	$32,621
L267	N268	N267	912.63	911	112.8	1.4	0.2	916.35	913.04	3	300	$460	$51,879	250	$350	$39,473
L483	N321	N487	901.265	901.121	177.4	0.1	0.45	903.602	903.9	3	675	$1,060	$188,012	525	$350	$62,080
					402.3							Sub-Total	$304,400			$163,257
												Contingency (25%)	$76,100			$40,814
												Engineering and Administration (10%)	$30,440			$16,326
												CAPITAL COST	$411,000			$221,000

2nd Avenue

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L354	N354	N355	904.117	903.872	98.2	0.2	0.375	906.3	905.8	3	525	$1,000	$98,150	375	$800	$78,520
L355	N355	N356	903.872	903.722	74.7	0.2	0.375	905.8	905.7	3	525	$1,000	$74,700	375	$800	$59,760
L356	N356	N357	903.722	903.245	59.7	0.8	0.375	905.7	905.62	3	525	$1,000	$59,700	375	$800	$47,760
L357	N357	N319	903.245	902.953	100.6	0.3	0.375	905.62	907.246	3	525	$1,000	$100,600	375	$710	$71,426
					333.2							Sub-Total	$333,150			$257,466
												Contingency (25%)	$83,288			$64,367
												Engineering and Administration (10%)	$33,315			$25,747
												CAPITAL COST	$450,000			$348,000

47th Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L88	N62	N91	922.36	921.44	114.6	0.8	0.2	925.02	924.2	3	300	$460	$52,716	250	$350	$40,110
L89	N91	N70	921.44	920.99	75.3	0.6	0.2	924.2	923.54	3	300	$460	$34,652	250	$350	$26,366
L70	N70	N71	920.99	920.69	64.1	0.5	0.2	923.54	923.33	3	300	$460	$29,477	250	$350	$22,428
L71	N71	N72	920.69	920.37	57.6	0.6	0.2	923.33	923.6	3	300	$460	$26,496	250	$350	$20,160
L73	N72	N74	920.31	920.036	113.4	0.2	0.2	923.6	922.8	3	300	$460	$52,146	250	$350	$39,676
L78	N74	N79	920.036	919.586	111.6	0.4	0.2	922.8	923.5	3	300	$460	$51,318	250	$350	$39,046
L80	N79	N81	919.586	918.366	56.2	2.2	0.2	923.5	921.72	4	300	$575	$32,298	250	$550	$30,894
L81	N81	N82	918.366	918.046	78.3	0.4	0.2	921.72	921.8	4	300	$575	$45,023	250	$550	$43,065
L781	N82	N782	918.046	917.336	83.5	0.9	0.2	921.8	920.5	3	300	$460	$38,419	250	$350	$29,232
L782	N782	N83	917.336	916.97	48.8	0.8	0.2	920.5	920	3	300	$460	$22,434	250	$350	$17,070
L374	N83	N373	916.97	915.975	51.8	1.9	0.2	920	919	3	300	$460	$23,837	250	$350	$18,137
					855.1							Sub-Total	$408,815			$326,183
												Contingency (25%)	$102,204			$81,546
												Engineering and Administration (10%)	$40,881			$32,618
												CAPITAL COST	$552,000			$441,000

48th Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L394	N389	N390	912.51	911.58	232.5	0.4	0.2	915.1	914.27	3	300	$460	$106,950	250	$350	$81,375
L396	N390	N371	911.58	910.89	172.5	0.4	0.2	914.27	911.96	2	300	$400	$69,000	250	$350	$60,375
L371	N371	N370	910.74	909.68	111.9	0.9	0.375	914.09	911.96	3	450	$960	$107,455	250	$350	$39,176
L370	N370	N350	909.68	908.61	113.0	0.9	0.375	911.96	910.51	2	450	$900	$101,689	250	$350	$39,546
L349	N350	N349	905.87	904.74	115.8	1.0	0.25	910.51	908.85	4	450	$1,075	$124,453	375	$825	$95,510
L410	N403	N349	903.52	903.31	172.4	0.1	0.45	909.22	908.85	6	600	$2,190	$377,644	450	$1,910	$329,360
					918.1							Sub-Total	$887,190			$645,343
												Contingency (25%)	$221,798			$161,336
												Engineering and Administration (10%)	$88,719			$64,534
												CAPITAL COST	$1,198,000			$872,000

Appendix G
Table G.2 - Edson Improvements - Phase 2
24 Year 4 Hour - HGL to 1.0 m below ground and below

50th Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L247	N247	N248	922.63	921.87	126.0	0.6	0.2	925.3	924.72	3	300	$460	$57,977	250	$350	$44,113
L246	N239	N247	923.93	922.63	111.1	1.2	0.2	924.72	926.55	2	300	$400	$44,444	250	$350	$38,889
					237.1							Sub-Total	$102,421			$83,001
												Contingency (25%)	$25,605			$20,750
												Engineering and Administration (10%)	$10,242			$8,300
												CAPITAL COST	$139,000			$113,000

1 Ave

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L414	N407	N405	904.49	904.2	172.0	0.2	0.375	908.49	908.55	4	525	$1,115	$191,747	375	$825	$141,875
												Contingency (25%)	$47,937			$35,469
												Engineering and Administration (10%)	$19,175			$14,188
												CAPITAL COST	$259,000			$192,000

52 Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L521	N500	N522	909.3	907.99	111.9	1.1	0.25	911.63	911.47	3	375	$710	$79,468	300	$460	$51,486
L492	N110	N495	910.4	909.62	111.6	0.7	0.25	913.006	912.42	3	375	$710	$79,208	300	$460	$51,318
					223.5							Sub-Total	$158,676			$102,804
												Contingency (25%)	$39,669			$25,701
												Engineering and Administration (10%)	$15,868			$10,280
												CAPITAL COST	$215,000			$139,000

next to 55th Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L810	N798	N796	909.48	908.99	86.5	0.6	0.375	914.2	913.05	4	525	$1,115	$96,448	375	$710	$61,415
L808	N796	N515	908.99	908.84	7.5	2.0	0.375	913.05	911.69	3	525	$1,000	$7,500	375	$710	$5,325
L512	N515	N514	908.84	908.35	71.6	0.7	0.375	911.69	910.6	3	525	$1,000	$71,630	375	$710	$50,857
L511	N514	N512	908.35	907.71	76.2	0.8	0.375	910.6	910.3	3	525	$1,000	$76,200	375	$710	$54,102
					241.8							Sub-Total	$251,778			$171,699
												Contingency (25%)	$62,944			$42,925
												Engineering and Administration (10%)	$25,178			$17,170
												CAPITAL COST	$340,000			$232,000

54th Street and 13th Avenue

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L759	N24	N190	923.84	923.5	94.2	0.4	0.2	927.22	926.09	3	375	$710	$66,868	375	$710	$66,868
L765	N190	N191	923.5	923.325	86.0	0.2	0.2	926.09	926.1	3	375	$710	$61,060	375	$710	$61,060
L766	N190	N115	923.5	923.39	114.3	0.1	0.2	926.1	927.66	3	375	$710	$81,153	375	$710	$81,153
					294.5							Sub-Total	$209,081			$209,081
												Contingency (25%)	$52,270			$52,270
												Engineering and Administration (10%)	$20,908			$20,908
												CAPITAL COST	$283,000			$283,000

Appendix G
Table G.2 - Edson Improvements - Phase 2
24 Year 4 Hour - HGL to 1.0 m below ground and below

10th Avenue

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L210	N212	N211	920	918.62	68.3	2.0	0.2	923.76	921.84	3	300	$460	$31,436	250	$350	$23,919
L209	N211	N208	918.62	915.06	69.8	5.1	0.2	921.84	920.59	4	300	$575	$40,135	250	$550	$38,390
L752	N208	N769	915.06	914.75	66.4	0.5	0.25	920.59	919.52	5	375	$965	$64,066	300	$740	$49,129
L753	N769	N207	914.75	913.14	66.9	2.4	0.25	919.52	918.79	5	375	$965	$64,530	300	$740	$49,484
					271.4							Sub-Total	$200,167			$160,921
												Contingency (25%)	$50,042			$40,230
												Engineering and Administration (10%)	$20,017			$16,092
												CAPITAL COST	$271,000			$218,000

10th Avenue

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L817	N800	N204	914.106	914.01	24.0	0.4	0.2	917.5	917.2	3	300	$460	$11,040	250	$350	$8,400
												Contingency (25%)	$2,760			$2,100
												Engineering and Administration (10%)	$1,104			$840
												CAPITAL COST	$15,000			$12,000

10th Avenue

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L801	N791	N792	913.44	912.8	100.0	0.6	0.2	916.8	917.85	4	300	$575	$57,500	250	$550	$55,000
L813	N792	N799	912.8	912.55	45.2	0.6	0.2	917.85	918.73	6	300	$740	$33,448	250	$600	$27,120
					145.2							Sub-Total	$90,948			$82,120
												Contingency (25%)	$22,737			$20,530
												Engineering and Administration (10%)	$9,095			$8,212
												CAPITAL COST	$123,000			$111,000

10th Avenue

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L198	N198	N200	914.72	914.4	96.3	0.3	0.2	917.83	917.54	3	300	$460	$44,298	250	$350	$33,705
L816	N200	N800	914.4	914.106	40.9	0.7	0.2	917.54	917.5	3	300	$460	$18,814	250	$350	$14,315
					137.2							Sub-Total	$63,112			$48,020
												Contingency (25%)	$15,778			$12,005
												Engineering and Administration (10%)	$6,311			$4,802
												CAPITAL COST	$86,000			$65,000

4th Avenue West

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L608	N615	N607	926.86	926.58	89.9	0.3	0.2	929.5	930	3	300	$460	$41,361	250	$350	$31,471
L603	N610	N607	927.26	926.58	85.0	0.8	0.2	931.5	930	4	300	$575	$48,898	250	$405	$34,441
L602	N607	N609	926.58	924	103.0	2.5	0.2	930	926.5	3	300	$460	$47,389	250	$350	$36,057
L601	N609	N604	924	921.52	88.4	2.8	0.2	926.5	925.5	3	300	$460	$40,659	250	$350	$30,937
L600	N604	N600	921.52	920.52	18.1	5.5	0.2	925.5	926	5	300	$700	$12,670	250	$460	$8,326
L622	N631	N630	922.474	922.03	111.0	0.4	0.2	926	925.5	3	300	$460	$51,037	250	$350	$38,833
L621	N630	N629	922.03	921.81	73.5	0.3	0.2	925.5	925.3	3	300	$460	$33,792	250	$350	$25,711
L620	N629	N628	921.81	921.52	121.0	0.2	0.2	925.3	924	3	300	$460	$55,660	250	$350	$42,350
L619	N628	N625	921.523	921.18	122.5	0.3	0.25	924	924	3	300	$460	$56,364	250	$350	$42,886
L618	N625	N600	921.181	920.52	188.9	0.4	0.25	924	924.5	3	300	$460	$86,880	300	$460	$86,880
					1001.3							Sub-Total	$474,711			$377,891
												Contingency (25%)	$118,678			$94,473
												Engineering and Administration (10%)	$47,471			$37,789
												CAPITAL COST	$641,000			$511,000

Appendix G
Table G.2 - Edson Improvements - Phase 2
24 Year 4 Hour - HGL to 1.0 m below ground and below

3rd Avenue

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L585	N585	N581	912.987	912.845	86.3	0.2	0.2	915.7	915.15	3	300	$460	$39,679	250	$350	$30,190
												Contingency (25%)	$9,920			$7,548
												Engineering and Administration (10%)	$3,968			$3,019
												CAPITAL COST	$54,000			$41,000

64th Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L639	N650	N649	923.257	922.885	103.3	0.4	0.2	925.695	925.3	3	300	$460	$47,527	250	$350	$36,162
												Contingency (25%)	$11,882			$9,041
												Engineering and Administration (10%)	$4,753			$3,616
												CAPITAL COST	$65,000			$49,000

4A Avenue

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L625	N634	N633	927.23	925.84	111.9	1.2	0.2	929.5	928.5	3	300	$460	$51,456	250	$350	$39,151
												Contingency (25%)	$12,864			$9,788
												Engineering and Administration (10%)	$5,146			$3,915
												CAPITAL COST	$70,000			$53,000

Phase 2 Summary	Length	Replacement Cost	
42nd Street	402	$304,400	
2nd Avenue	333	$333,150	
47th Street	855	$408,815	
48th Street +	918	$887,190	
50th Street	237	$102,421	
1 Ave	172	$191,747	
52 Street	223	$158,676	
next to 55th Street	242	$251,778	
54th Street and 13th Avenue	294	$209,081	
10th Avenue	271	$200,167	
10th Avenue	24	$11,040	
10th Avenue	145	$90,948	
10th Avenue	137	$63,112	
4th Avenue West	1001	$474,711	
3rd Avenue	86	$39,679	
64th Street	103	$47,527	
4A Avenue	112	$51,456	
Subtotal	5558	$3,825,896	$2,883,564.00
Engineering (10%) & Contingencies (25%)		$1,339,064	$1,009,247
Total		**$5,164,960**	**$3,892,811**

Appendix G
Table G.3 - Edson Improvements - Phase 3
5 Year 4 Hour - HGL to 2.5 m below ground and below

42nd Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L271	N272	N271	915.11	914.82	83.8	0.3	0.2	918.09	918.49	3	300	$460	$38,557	250	$350	$29,337
												Contingency (25%)	$9,639			$7,334
												Engineering and Administration (10%)	$3,856			$2,934
												CAPITAL COST	$53,000			$40,000

43rd Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L45	N45	N46	924.214	923.837	62.9	0.6	0.2	927	926.5	3	300	$460	$28,934	250	$350	$22,015
L56	N46	N58	923.837	923.28	111.3	0.5	0.2	926.5	925.9	3	300	$460	$51,198	250	$350	$38,955
					174.2							Sub-Total	$80,132			$60,970
												Contingency (25%)	$20,033			$15,243
												Engineering and Administration (10%)	$8,013			$6,097
												CAPITAL COST	$109,000			$83,000

47th Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L99	N56	N100	920.4	917.48	110.0	2.7		923.24	920.13	3	300	$460	$50,614	300	$460	$50,614
												Contingency (25%)	$12,653			$12,653
												Engineering and Administration (10%)	$5,061			$5,061
												CAPITAL COST	$69,000			$69,000

48th Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L108	N108	N109	910.875	910.71	27.6	0.6	0.25	913.5	913.3	3	375	$710	$19,561	300	$460	$12,673
L109	N109	N110	910.71	910.4	57.8	0.5	0.25	913.3	913.006	3	375	$710	$41,045	300	$460	$26,593
					85.4							Sub-Total	$60,606			$39,266
												Contingency (25%)	$15,151			$9,816
												Engineering and Administration (10%)	$6,061			$3,927
												CAPITAL COST	$82,000			$54,000

54th Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L399	N379	N394	916.99	914.09	111.5	2.6	0.2	919.83	916.75	3	300	$460	$51,307	250	$350	$39,038
L400	N394	N395	914.09	909.83	151.1	2.8	0.2	916.75	912.8	3	300	$460	$69,489	250	$350	$52,872
					262.6							Sub-Total	$120,797			$91,911
												Contingency (25%)	$30,199			$22,978
												Engineering and Administration (10%)	$12,080			$9,191
												CAPITAL COST	$164,000			$125,000

4th Ave (between 52nd and 53rd Street)

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L526	N526	N522	908.6	908	171.4	0.4	0.2	911.3	911.47	3	300	$460	$78,857	250	$350	$60,000
												Contingency (25%)	$19,714			$15,000
												Engineering and Administration (10%)	$7,886			$6,000
												CAPITAL COST	$107,000			$82,000

Appendix G
Table G.3 - Edson Improvements - Phase 3
5 Year 4 Hour - HGL to 2.5 m below ground and below

41st Street

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L323	N324	N323	902.57	902.31	48.5	0.5	0.2	905.4	905.08	3	300	$460	$22,292	250	$350	$16,961
												Contingency (25%)	$5,573			$4,240
												Engineering and Administration (10%)	$2,229			$1,696
												CAPITAL COST	$31,000			$23,000

Phase 3 Summary	Length	Replacement Cost
42nd Street	84	$38,557
43rd Street	174	$80,132
47th Street	110	$50,614
48th Street	85	$60,606
54th Street	263	$120,797
4th Ave (between 52nd and 53rd Street)	171	$78,857
41st Street	48	$22,292
Subtotal	936	$451,854
Engineering (10%) & Contingencies (25%)		$158,149
Total		$610,004

Appendix G
Table G.4
Edson Improvements - 2015 and 2025
5 Year 4 Hour - HGL to 2.5 m below ground and below

New pipe from West End - 750 mm (Alternative 3)

Name	Upstream Node Name	Downstream Node Name	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Length (m)	Slope (%)	Existing Diameter (m)	US Ground Elevation (m)	DS Ground Elevation (m)	Average Depth (m)	Replacement Diameter (mm)	Unit Cost	Replacement Cost	Twin Diameter (mm)	Unit Cost	Twinning Cost
L824	N807	N805	916	907.44	283.0	3.0	NA	922	913	6	750	1055	$298,607	750	1055	$298,607
L833	N805	N812	907.44	906.73	321.1	0.2	NA	913	912.5	6	750	1055	$338,782	750	1055	$338,782
L834	N812	N811	906.73	906.3	195.1	0.2	NA	912.5	915	7	750	1180	$230,171	750	1180	$230,171
L835	N811	N810	906.3	905.9	175.0	0.2	NA	915	910.5	7	750	5000	$875,000	750	5000	$875,000
L836	N810	N803	905.9	905.77	60.8	0.2	NA	910.5	911	5	750	5000	$303,750	750	5000	$303,750
L8282	N803	N425	905.77	902.51	1482.6	0.2	NA	911	907.59	5	750	935	$1,386,212	750	935	$1,386,212
					2517.6							Sub-Total	$3,432,522			$3,432,522
												Contingency (25%)	$858,130			$858,130
												Engineering and Administration (10%)	$343,252			$343,252
												CAPITAL COST	$4,634,000			$4,634,000

Appendix G
Table G.5
New Development - 2015 and 2025

New Development 2015/2025

Development Stage	Development Area	Pipe Type	Pipe Length (m)	Approximate Average Depth (m)	Diameter (mm)	Unit Cost	Installation Cost
2015	Area 1	Gravity	250	3	200	350	$87,500.00
2015	Area 2	Gravity	250	3	200	350	$87,500.00
2015	Area 4	Gravity	150	3	200	350	$52,500.00
2015	Area 5	Gravity	200	4	200	400	$80,000.00
2015	Area 8	Gravity	200	4	200	400	$80,000.00
2015	Area 9	Gravity	100	3	200	350	$35,000.00
2015	Area 14	Gravity	300	5	200	450	$135,000.00
2015	Area 15	Gravity	750	4	200	400	$300,000.00
2015	Area 15	Gravity	400	4	300	475	$190,000.00
2015	Area 17	Gravity	800	4	200	400	$320,000.00
2015	East - Area 15,17	Force Main	1700	3	150	300	$510,000.00
2015	East - Area 15,17	Lift Station 28 L/s					$300,000.00
2015	West Areas 11,12,13,18,23	Gravity	800	4	675	800	$640,000.00
2015	West Areas 11,12,13,18,24	Gravity	1600	4	600	700	$1,120,000.00
2015	West Areas 11,12,13,18,25	Gravity	3800	4	450	575	$2,185,000.00
2015	West Areas 11,12,13,18,26	Gravity (HWY)	200	4	450	4000	$800,000.00
2015	West Areas 11,12,13,18,26	Gravity	1200	4	375	525	$630,000.00
2015	West Area 13	Force Main	1300	3	200	350	$455,000.00
2015	West Area 13	Force Main (HWY)	200	3	200	3000	$600,000.00
2015	West Area 13	Lift Station 41 L/s					$500,000.00
2025	Area 3	Gravity	150	3	200	350	$52,500.00
2025	Area 6	Gravity	100	4	250	450	$45,000.00
2025	Area 7	Gravity	350	4	200	400	$140,000.00
2025	Area 10	Gravity	200	4	200	400	$80,000.00
2025	West Areas 19,20,21,22	Gravity	1600	4	375	525	$840,000.00
2025	West Areas 19,20,21,22	Gravity	1700	4	300	475	$807,500.00
2025	East - Area 16	Force Main	1700	3	200	350	$595,000.00
2025	East - Area 16	Lift Station 43 L/s					$500,000.00
2025	East - Area 16	Gravity	1000	4	300	475	$475,000.00

2015 Total Length	14200	2015 Cost	$9,107,500.00	
2025 Total Length	6800	10% Contingency	$910,750.00	
		25% Engineering	$2,276,875.00	
		Total 2015 Cost	$12,295,125.00	
				$4,424,875.00
		2025 Cost	$3,535,000.00	
		10% Contingency	$353,500.00	
		25% Engineering	$883,750.00	
		Total 2025 Cost	$4,772,250.00	
		2015 and 2025 Total	**$17,067,375.00**	

**Pipe invert, depth and slope conceptual only and must be determined upon design*

Appendix H

Hydraulic Analysis Results

Appendix H
Table H.1
Hydrologic Analysis Results

Basin Name	Development Year	Basin Area (ha)	100 Year 4 Hour Event			100 Year 24 Hour Event		
			Runoff Depth (mm)	Runoff/ Rainfall Ratio (%)	Runoff Volume (m3)	Runoff Depth (mm)	Runoff/ Rainfall Ratio (%)	Runoff Volume (m3)
BasinA	2015	11.4	28.6	54%	3,300	46.1	53%	5,300
BasinA	2025	60.1	46.6	87%	28,000	78.2	90%	47,000
BasinB	2025	51.3	46.7	88%	23,900	78.2	90%	40,100
BasinC	2025	63.7	46.6	87%	29,700	78.1	90%	49,800
BasinD	2025	58.6	46.6	88%	27,300	78.2	90%	45,800
BasinE	2025	57.3	46.6	88%	26,700	78.2	90%	44,800
BasinF	2025	75.4	46.7	88%	35,200	78.2	90%	59,000
BasinG	2025	53.1	46.7	88%	24,800	78.3	90%	41,600
BasinH	2025	74.1	46.6	87%	34,500	78.1	90%	57,900
BasinI	2025	60.8	46.6	87%	28,300	78.2	90%	47,500
BasinJ	2025	91.7	46.6	87%	42,700	78.2	90%	71,700
BasinK	2025	53.0	46.7	88%	24,800	78.3	90%	41,500
BasinL	2025	77.1	46.6	87%	35,900	78.1	90%	60,200
BasinM	2025	33.7	28.5	54%	9,600	46.0	53%	15,500
BasinN	2025	58.1	46.6	88%	27,100	78.2	90%	45,400
BasinO	2025	37.1	46.7	88%	17,300	78.3	90%	29,100
BasinP	2015	15.1	28.5	53%	4,300	45.9	53%	6,900
BasinP	2025	22.0	29.1	55%	6,400	46.4	54%	10,200
BasinR	2025	17.3	27.6	52%	4,800	45.1	52%	7,800
BasinS	2015	13.0	28.8	54%	3,700	46.2	53%	6,000
BasinS	2025	39.9	27.4	51%	10,900	45.0	52%	17,900
BasinT	2015	12.5	28.0	53%	3,500	45.5	53%	5,700
BasinT	2025	29.0	26.9	51%	7,800	44.5	51%	12,900
BasinU	2025	18.1	46.6	87%	8,400	78.1	90%	14,100
BasinV	2025	20.0	46.7	88%	9,300	78.3	90%	15,700
BasinW	2025	26.3	46.5	87%	12,200	77.9	90%	20,500
BasinX	2025	90.2	45.8	86%	41,300	76.6	88%	69,100
BasinY	2015	56.1	46.4	87%	26,000	77.9	90%	43,700
BasinY	2025	93.5	46.2	87%	43,200	77.5	89%	72,400

Appendix I

Hydraulic Analysis Results

Appendix I
Table I.1
Hydraulic Analysis Results

Basin Name	Year of Development	Basin Area (ha)	100 Year 4 Hour Event				100 Year 24 Hour Event			
			Runoff Depth (mm)	Runoff/ Rainfall Ratio (%)	Maximum Outflow (m3/s)	Required Storage (m3)	Runoff Depth (mm)	Runoff/Rainfall Ratio (%)	Maximum Outflow (m3/s)	Required Storage (m3)
BasinA	2015	11.4	28.6	54%	0.03	2,900	46.1	53%	0.03	3,400
BasinA	2025	60.1	46.6	87%	0.15	25,200	78.2	90%	0.17	34,300
BasinB	2025	51.3	46.7	88%	0.12	21,700	78.2	90%	0.14	29,500
BasinC	2025	63.7	46.6	87%	0.16	26,600	78.1	90%	0.18	36,000
BasinD	2025	58.6	46.6	88%	0.14	24,700	78.2	90%	0.16	33,900
BasinE	2025	57.3	46.6	88%	0.14	24,000	78.2	90%	0.16	32,500
BasinF	2025	75.4	46.7	88%	0.18	31,300	78.2	90%	0.21	42,100
BasinG	2025	53.1	46.7	88%	0.13	22,600	78.3	90%	0.15	30,400
BasinH	2025	74.1	46.6	87%	0.18	30,300	78.1	90%	0.21	41,000
BasinI	2025	60.8	46.6	87%	0.15	25,500	78.2	90%	0.17	34,700
BasinJ	2025	91.7	46.6	87%	0.22	38,300	78.2	90%	0.26	52,000
BasinK	2025	53.0	46.7	88%	0.13	22,500	78.3	90%	0.15	30,300
BasinL	2025	77.1	46.6	87%	0.19	31,500	78.1	90%	0.23	42,400
BasinM	2025	33.7	28.5	54%	0.09	8,500	46.0	53%	0.09	10,000
BasinN	2025	58.1	46.6	88%	0.14	24,400	78.2	90%	0.16	33,300
BasinO	2025	37.1	46.7	88%	0.09	15,800	78.3	90%	0.10	21,400
BasinP	2015	15.1	28.5	53%	0.04	3,800	45.9	53%	0.04	4,400
BasinP	2025	22.0	29.1	55%	0.06	5,500	46.4	54%	0.06	6,400
BasinR	2025	17.3	27.6	52%	0.16	3,000	45.1	52%	0.15	2,400
BasinS	2015	13.0	28.8	54%	0.12	2,500	46.2	53%	0.11	2,100
BasinS	2025	39.9	27.4	51%	0.35	6,900	45.0	52%	0.34	6,000
BasinT	2015	12.5	28.0	53%	0.11	2,300	45.5	53%	0.10	2,000
BasinT	2025	29.0	26.9	51%	0.26	4,800	44.5	51%	0.25	4,100
BasinU	2025	18.1	46.6	87%	0.16	6,200	78.1	90%	0.16	6,700
BasinV	2025	20.0	46.7	88%	0.06	8,300	78.3	90%	0.06	10,900
BasinW	2025	26.3	46.5	87%	0.06	10,800	77.9	90%	0.07	14,900
BasinX	2025	90.2	45.8	86%	0.21	33,400	76.6	88%	0.25	48,900
BasinY	2015	56.1	46.4	87%	0.14	22,900	77.9	90%	0.16	31,400
BasinY	2025	93.5	46.2	87%	0.22	36,900	77.5	89%	0.26	52,000

Appendix J

Cost Estimates – Stormwater Management Plan

Appendix J
Table J-1
Cost Estimates - Stormwater Management Plan

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
2-002	124.99	0.450	0.600	1.396	3.62	1080	75	$144,363
2-013	56.84	0.450	0.525	2.270	2.23	920	60	$55,703
2-014	55.27	0.450	0.525	2.086	2.13	920	60	$54,165
2-015	94.00	0.525	0.750	1.311	2.25	1100	70	$109,980
2-003	89.28	0.600	0.675	0.868	2.31	1000	60	$94,637
2-004	107.60	0.600	0.675	0.702	2.38	1000	60	$114,056
2-009	97.50	0.900	1.050	0.277	5.07	3400	115	$342,713
2-010	54.40	0.900	1.050	0.377	3.69	2535	100	$143,344
							Sub-Total	$1,059,000
							Contingency (25%)	$264,750
							Engineering and Administration (10%)	$105,900
							CAPITAL COST	$1,430,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
1-004	30.00	0.450	0.525	3.833	2.90	920	60	$29,400
1-005	145.92	0.450	0.600	4.146	2.80	940	60	$145,920
1-008	76.97	0.450	0.600	0.780	2.80	940	60	$76,970
1-014	87.20	0.450	0.525	0.500	2.16	920	60	$85,456
1-015	65.00	0.675	0.750	2.385	3.20	1100	70	$76,050
1-016	65.00	0.675	0.750	3.170	2.35	1100	70	$76,050
1-017	47.50	0.675	0.750	3.170	2.00	1100	70	$55,575
							Sub-Total	$546,000
							Contingency (25%)	$136,500
							Engineering and Administration (10%)	$54,600
							CAPITAL COST	$738,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
1-018	48.16	0.300	0.525	1.599	3.67	1040	75	$53,698
1-019	83.82	0.375	0.525	1.300	3.90	1040	75	$93,459
1-020	54.86	0.375	0.525	1.440	3.86	1040	75	$61,169
1-021	31.76	0.375	0.600	1.610	3.89	1080	75	$36,683
1-022	84.28	0.375	0.600	2.883	3.25	940	60	$84,280
1-023	85.34	0.300	0.450	1.594	3.39	900	60	$81,926
1-024	85.64	0.300	0.525	3.059	4.03	1040	75	$95,489
1-025	67.39	0.450	0.600	0.757	3.51	1080	75	$77,835
1-026	66.51	0.450	0.675	0.571	3.58	1200	75	$84,800
1-027	69.89	0.525	0.750	2.018	3.69	1475	85	$109,028
1-028	72.91	0.525	0.900	1.783	3.21	1860	75	$141,081
1-029	103.90	0.525	0.900	1.886	3.53	2060	90	$223,385
1-031	57.42	0.600	0.900	1.602	3.45	1860	75	$111,108
1-035	118.87	0.600	0.900	1.817	4.19	2060	90	$255,571
1-061	152.40	1.050	1.350	1.017	4.64	4160	130	$653,796
1-062	60.05	1.050	1.350	1.080	3.62	3470	115	$215,279
1-069	97.53	0.525	0.900	0.174	4.75	2460	110	$250,652
1-074	118.90	0.600	0.900	0.404	4.75	2460	110	$305,573
1-075	59.74	0.750	0.900	0.180	2.86	1860	75	$115,597
1-076	68.32	0.750	0.900	0.270	2.84	1860	75	$132,199
1-077	33.83	0.750	0.900	0.070	4.41	2060	90	$72,735
							Sub-Total	$3,256,000
							Contingency (25%)	$814,000
							Engineering and Administration (10%)	$325,600
							CAPITAL COST	$4,396,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
3-008	89.36	0.375	0.525	0.996	2.45	920	60	$87,573
3-009	114.60	0.375	0.750	1.001	2.79	1100	70	$134,082
3-010	88.39	0.525	0.750	0.900	2.58	1100	70	$103,416
3-011	81.00	0.525	0.750	0.900	3.38	1100	70	$94,770
3-012	95.46	0.375	0.525	0.995	2.91	920	60	$93,551
3-013	101.19	0.450	0.900	0.860	2.91	1860	75	$195,803
3-014	74.98	0.450	0.900	1.280	2.96	1860	75	$145,086
3-015	113.84	0.300	0.450	0.720	2.27	900	60	$109,286
3-016	62.48	0.450	0.900	1.793	2.41	1860	75	$120,899
							Sub-Total	$1,085,000
							Contingency (25%)	$271,250
							Engineering and Administration (10%)	$108,500
							CAPITAL COST	$1,465,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
3-017	86.26	0.450	0.525	1.906	3.28	920	60	$84,535
3-018	85.32	0.450	0.525	1.868	3.28	920	60	$83,614
3-019	86.87	0.450	0.600	1.530	2.30	940	60	$86,870
3-020	86.87	0.450	0.600	1.000	2.72	940	60	$86,870
3-021	99.97	0.450	0.675	1.070	2.79	1000	60	$105,968
3-022	23.99	0.450	0.675	1.000	2.33	1000	60	$25,429
							Sub-Total	$474,000
							Contingency (25%)	$118,500
							Engineering and Administration (10%)	$47,400
							CAPITAL COST	$640,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
4-001	76.00	0.375	0.450	0.712	2.36	900	60	$72,960
4-002	72.77	0.375	0.450	1.060	2.52	900	60	$69,859
4-003	62.84	0.375	0.525	1.500	2.65	920	60	$61,583
4-004	69.49	0.375	0.525	2.724	2.67	920	60	$68,100
4-005	49.99	0.375	0.525	3.231	2.67	920	60	$48,990
4-006	80.16	0.525	0.675	0.749	2.69	1000	60	$84,970
4-007	87.38	0.600	0.675	1.280	2.46	1000	60	$92,623
4-008	89.92	0.600	0.675	1.730	2.53	1000	60	$95,315
							Sub-Total	$595,000
							Contingency (25%)	$148,750
							Engineering and Administration (10%)	$59,500
							CAPITAL COST	$804,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
1-044	89.92	0.300	0.450	2.875	2.33	900	60	$86,323
1-045	95.27	0.300	0.450	2.470	2.33	900	60	$91,459
1-047	63.00	0.375	0.525	0.170	3.45	920	60	$61,740
1-046	47.67	0.450	0.525	0.830	2.66	920	60	$46,717
1-048	92.66	0.450	0.675	1.061	3.13	1000	60	$98,220
1-049	88.09	0.525	0.750	0.550	3.13	1100	70	$103,065
1-050	93.57	0.525	0.750	0.556	3.13	1100	70	$109,477
1-051	59.44	0.525	0.750	1.666	3.08	1100	70	$69,545
Link374	262.13	-	0.750	0.770	4.44	1475	85	$408,923
Link375	149.35	-	1.200	0.375	5.97	4820	145	$741,523
1-036	85.04	0.300	0.450	1.380	2.24	900	60	$81,638
1-037	69.80	0.300	0.525	1.400	2.23	920	60	$68,404
1-038	85.25	0.375	0.600	2.540	2.34	940	60	$85,250
1-039	97.54	0.450	0.600	1.190	2.41	940	60	$97,540
1-040	101.50	0.450	0.600	2.600	2.39	940	60	$101,500
1-042	87.17	0.300	0.450	2.120	2.19	900	60	$83,683
1-041	94.17	0.300	0.525	0.850	2.19	920	60	$92,287
1-078	154.73	0.300	0.450	0.500	2.82	900	60	$148,541
1-079	169.98	0.300	0.600	1.500	5.34	1240	90	$226,073
1-080	102.11	0.300	0.600	2.977	5.35	1240	90	$135,806
1-081	111.86	0.375	0.750	2.289	2.39	1100	70	$130,876
1-082	86.08	0.300	0.450	3.950	2.24	900	60	$82,633
1-083	87.78	0.300	0.525	2.700	2.19	920	60	$86,022
1-084	111.25	0.525	1.050	1.481	2.44	2100	80	$242,525
1-085	79.74	0.300	0.450	3.900	2.71	900	60	$76,554
1-086	90.00	0.300	0.450	3.400	2.30	900	60	$86,400
1-088	80.92	0.300	0.450	3.250	1.74	900	60	$77,686
1-089	89.05	0.300	0.450	4.256	2.70	900	60	$85,491
Link376	112.48	-	1.050	1.078	2.79	2100	80	$245,200
Link377	112.47	-	1.200	1.076	3.14	2420	90	$282,300
Link378	127.92	-	1.200	1.079	3.53	3020	110	$400,390
1-095	62.90	0.300	0.375	3.800	2.11	650	60	$44,655
1-096	113.47	0.300	0.375	5.059	3.49	650	60	$80,560
1-097	91.74	0.600	1.200	1.581	3.53	3020	110	$287,146
1-098	62.79	0.750	1.350	0.844	1.99	2780	95	$180,521
1-099	123.14	0.375	0.525	0.800	2.76	920	60	$120,677
1-100	88.39	0.450	0.750	0.300	2.75	1100	70	$103,416
1-101	111.56	0.450	0.900	0.296	2.71	1860	75	$215,869
Link379	95.71	-	1.350	1.640	2.15	2780	95	$275,166
Link380	78.94	-	1.350	1.153	2.15	2780	95	$226,953
Link381	124.97	-	1.350	1.152	2.26	2780	95	$359,289
							Sub-Total	$6,629,000
							Contingency (25%)	$1,657,250
							Engineering and Administration (10%)	$662,900
							CAPITAL COST	$8,950,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
1-108	82.30	0.300	0.525	0.377	1.75	920	60	$80,654
1-109	81.67	0.300	0.525	1.150	2.84	920	60	$80,037
1-110	86.26	0.375	0.525	0.410	2.84	920	60	$84,535
1-111	87.17	0.375	0.675	0.350	2.07	1000	60	$92,400
1-112	107.90	0.450	0.675	1.250	2.80	1000	60	$114,374
							Sub-Total	$452,000
							Contingency (25%)	$113,000
							Engineering and Administration (10%)	$45,200
							CAPITAL COST	$611,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
1-114	105.62	0.375	0.525	1.000	1.93	920	60	$103,508
1-115	113.88	0.375	0.750	0.600	1.93	1100	70	$133,240
1-116	124.35	0.600	0.900	0.520	1.91	1860	75	$240,617
							Sub-Total	$478,000
							Contingency (25%)	$119,500
							Engineering and Administration (10%)	$47,800
							CAPITAL COST	$646,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
4-011	47.83	0.300	0.450	0.320	2.43	900	60	$45,913
4-012	100.74	0.375	0.525	0.782	2.53	920	60	$98,725
4-013	106.38	0.375	0.525	1.239	3.08	920	60	$104,252
4-014	81.99	0.450	0.600	0.922	3.08	940	60	$81,990
							Sub-Total	$331,000
							Contingency (25%)	$82,750
							Engineering and Administration (10%)	$33,100
							CAPITAL COST	$447,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
1-118	92.96	0.300	0.525	1.689	2.14	920	60	$91,101
1-119	26.21	0.450	0.525	3.625	1.81	920	60	$25,686
1-120	103.94	0.375	0.600	3.300	1.81	940	60	$103,940
1-121	100.58	0.525	0.750	1.034	1.74	1100	70	$117,679
1-122	130.45	0.675	0.900	0.424	1.86	1860	75	$252,421
1-123	126.80	0.375	0.600	0.560	1.98	940	60	$126,800
1-124	178.92	0.525	0.750	0.326	2.12	1100	70	$209,336
1-125	173.00	0.900	1.200	0.200	2.22	2420	90	$434,230
							Sub-Total	$1,362,000
							Contingency (25%)	$340,500
							Engineering and Administration (10%)	$136,200
							CAPITAL COST	$1,839,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
4-016	107.90	0.300	0.450	1.168	2.18	900	60	$103,584
4-017	81.99	0.375	0.525	1.769	2.19	920	60	$80,350
4-018	75.90	0.375	0.600	1.858	2.02	940	60	$75,900
4-019	31.09	0.375	0.600	1.351	2.05	940	60	$31,090
							Sub-Total	$291,000
							Contingency (25%)	$72,750
							Engineering and Administration (10%)	$29,100
							CAPITAL COST	$393,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
4-020	138.68	0.300	0.450	1.642	2.20	900	60	$133,133
							Sub-Total	$134,000
							Contingency (25%)	$33,500
							Engineering and Administration (10%)	$13,400
							CAPITAL COST	$181,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
4-032	77.72	0.300	0.525	0.978	1.84	920	60	$76,166
4-033	79.25	0.300	0.450	4.479	1.86	900	60	$76,080
4-034	111.56	0.300	0.525	1.694	1.89	920	60	$109,329
4-035	38.10	0.300	0.450	9.869	2.45	900	60	$36,576
							Sub-Total	$299,000
							Contingency (25%)	$74,750
							Engineering and Administration (10%)	$29,900
							CAPITAL COST	$404,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
4-036	113.39	0.300	0.525	0.485	3.20	920	60	$111,122
4-037	90.22	0.300	0.600	1.053	3.20	940	60	$90,220
4-038	87.78	0.300	0.525	2.256	2.16	920	60	$86,024
4-039	15.24	0.300	0.525	8.455	1.66	920	60	$14,935
							Sub-Total	$303,000
							Contingency (25%)	$75,750
							Engineering and Administration (10%)	$30,300
							CAPITAL COST	$410,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
4-041	103.02	0.250	0.600	0.400	2.06	940	60	$103,020
4-042	50.08	0.450	0.525	3.000	2.02	920	60	$49,078
4-043	60.35	0.450	0.750	0.878	2.02	1100	70	$70,610
4-044	51.21	0.450	0.750	1.035	1.77	1100	70	$59,916
4-045	115.82	0.600	1.050	0.449	1.51	2100	80	$252,488
4-046	170.38	0.675	1.050	0.305	1.65	2100	80	$371,428
							Sub-Total	$907,000
							Contingency (25%)	$226,750
							Engineering and Administration (10%)	$90,700
							CAPITAL COST	$1,225,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
4-022	42.50	0.300	0.600	0.400	2.21	940	60	$42,500
4-024	53.34	0.300	0.600	0.500	2.54	940	60	$53,340
4-026	82.90	0.375	0.750	0.300	2.98	1100	70	$96,993
4-027	80.00	0.375	0.750	0.450	3.22	1100	70	$93,600
4-028	52.38	0.450	0.750	0.420	3.28	1100	70	$61,286
4-029	76.74	0.450	0.750	0.430	3.19	1100	70	$89,790
4-030	35.36	0.450	0.600	1.300	2.94	940	60	$35,360
							Sub-Total	$473,000
							Contingency (25%)	$118,250
							Engineering and Administration (10%)	$47,300
							CAPITAL COST	$639,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
4-048	95.45	0.300	0.450	3.100	2.16	900	60	$91,632
4-049	26.52	0.375	0.450	2.000	2.15	900	60	$25,459
4-050	13.44	0.450	0.525	2.976	2.15	920	60	$13,171
4-051	84.73	0.450	0.600	2.254	2.10	940	60	$84,730
4-052	95.60	0.450	0.675	2.500	2.60	1000	60	$101,340
4-053	124.36	0.750	0.900	1.240	2.60	1860	75	$240,637
							Sub-Total	$557,000
							Contingency (25%)	$139,250
							Engineering and Administration (10%)	$55,700
							CAPITAL COST	$752,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
4-054	114.40	0.300	0.450	2.500	1.94	900	60	$109,824
4-055	94.57	0.300	0.450	3.500	1.87	900	60	$90,788
							Sub-Total	$201,000
							Contingency (25%)	$50,250
							Engineering and Administration (10%)	$20,100
							CAPITAL COST	$272,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
5-001	47.55	0.300	0.450	0.379	1.10	900	60	$45,648
5-002	42.67	0.300	0.450	0.563	1.34	900	60	$40,963
5-003	76.04	0.375	0.525	1.170	1.76	920	60	$74,519
5-004	30.32	0.450	0.525	1.850	1.76	920	60	$29,714
							Sub-Total	$191,000
							Contingency (25%)	$47,750
							Engineering and Administration (10%)	$19,100
							CAPITAL COST	$258,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
4-061	98.45	0.300	0.525	2.300	2.18	920	60	$96,481
4-063a	127.65	0.600	0.675	1.214	3.70	1200	75	$162,754
4-064	131.06	0.600	0.900	0.900	2.00	1860	75	$253,601
							Sub-Total	$513,000
							Contingency (25%)	$128,250
							Engineering and Administration (10%)	$51,300
							CAPITAL COST	$693,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
5-005	38.71	0.300	0.375	0.400	2.41	650	60	$27,484
5-006	53.57	0.300	0.450	0.560	2.24	900	60	$51,428
5-007	117.65	0.300	0.525	0.700	3.22	920	60	$115,297
5-008	81.38	0.375	0.525	1.450	3.51	1040	75	$90,738
5-009	81.02	0.375	0.525	2.950	2.99	920	60	$79,397
5-010	29.96	0.375	0.450	2.270	3.36	900	60	$28,758
							Sub-Total	$394,000
							Contingency (25%)	$98,500
							Engineering and Administration (10%)	$39,400
							CAPITAL COST	$532,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
5-013	111.40	0.300	0.450	4.000	1.95	900	60	$106,944
5-014	101.11	0.300	0.600	1.800	1.95	940	60	$101,111
5-015	25.91	0.600	0.900	0.500	1.00	1860	75	$50,136
							Sub-Total	$259,000
							Contingency (25%)	$64,750
							Engineering and Administration (10%)	$25,900
							CAPITAL COST	$350,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
5-017	112.78	0.300	0.450	1.800	2.58	900	60	$108,267
5-018	110.00	0.300	0.525	2.100	2.64	920	60	$107,800
5-019	89.60	0.375	0.750	1.250	2.71	1100	70	$104,832
5-020	91.77	0.375	0.750	1.700	2.21	1100	70	$107,365
5-022	110.50	0.450	0.750	3.810	2.79	1100	70	$129,284
							Sub-Total	$558,000
							Contingency (25%)	$139,500
							Engineering and Administration (10%)	$55,800
							CAPITAL COST	$754,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
6-001	72.78	0.200	0.450	2.968	2.70	900	60	$69,869
6-002	115.53	0.200	0.525	1.420	2.75	920	60	$113,219
6-003	48.50	0.200	0.525	2.000	2.75	920	60	$47,530
6-004	37.00	0.525	0.600	0.649	2.82	940	60	$37,000
6-005	53.00	0.525	0.600	0.800	2.82	940	60	$53,000
6-006	53.00	0.525	0.675	1.377	2.50	1000	60	$56,180
6-010	85.45	0.300	0.450	0.351	2.50	900	60	$82,032
							Sub-Total	$459,000
							Contingency (25%)	$114,750
							Engineering and Administration (10%)	$45,900
							CAPITAL COST	$620,000

Pipe (Link Name)	Length (m)	Existing Diameter (m)	Proposed Diameter (m)	Pipe Slope (%)	Max Depth (m)	Pipe Installation Unit Cost ($/m)	Manhole Installation Unit Cost ($/m)	Total Cost ($)
6-021	64.78	0.300	0.450	2.300	2.16	900	60	$62,192
6-022	47.27	0.300	0.450	2.200	2.04	900	60	$45,382
6-023	53.44	0.300	0.525	3.200	2.34	920	60	$52,369
6-024	87.50	0.375	0.525	4.000	2.47	920	60	$85,750
							Sub-Total	$246,000
							Contingency (25%)	$61,500
							Engineering and Administration (10%)	$24,600
							CAPITAL COST	$333,000

$29,770,090.65

Appendix J
Table J-2
Cost Estimates for 2025 Ultimate Development - Future Stormwater Management Facilities

SWMF A

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 75,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.37	$ 14,224
3	Topsoil Stripping	m^3	$ 7.50	7,900	$ 59,250
4	Excavation and Disposal	m^3	$ 16.00	68,300	$ 1,092,800
5	Landscaping	ha	$ 144,000.00	1.06	$ 152,603
6	Outlet Pipe (525 mm diameter)	m	$ 625.00	100	$ 62,500
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,557,000**
25% Contingency	**$ 389,250**
10% Engineering and Overhead	**$ 155,700**
Total (2009 $)	**$ 2,102,000**

SWMF B

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 65,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.08	$ 12,489
3	Topsoil Stripping	m^3	$ 7.50	6,900	$ 51,750
4	Excavation and Disposal	m^3	$ 16.00	57,800	$ 924,800
5	Landscaping	ha	$ 144,000.00	0.98	$ 140,761
6	Outlet Pipe (525 mm diameter)	m	$ 625.00	100	$ 62,500
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,358,000**
25% Contingency	**$ 339,500**
10% Engineering and Overhead	**$ 135,800**
Total (2009 $)	**$ 1,834,000**

SWMF C

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 78,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.47	$ 14,833
3	Topsoil Stripping	m^3	$ 7.50	8,200	$ 61,500
4	Excavation and Disposal	m^3	$ 16.00	72,200	$ 1,155,200
5	Landscaping	ha	$ 144,000.00	1.09	$ 156,604
6	Outlet Pipe (525 mm diameter)	m	$ 625.00	100	$ 62,500
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,629,000**
25% Contingency	**$ 407,250**
10% Engineering and Overhead	**$ 162,900**
Total (2009 $)	**$ 2,200,000**

Appendix J
Table J-2
Cost Estimates for 2025 Ultimate Development - Future Stormwater Management Facilities

SWMF D

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 74,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.35	$ 14,080
3	Topsoil Stripping	m^3	$ 7.50	7,800	$ 58,500
4	Excavation and Disposal	m^3	$ 16.00	67,500	$ 1,080,000
5	Landscaping	ha	$ 144,000.00	1.05	$ 151,648
6	Outlet Pipe (525 mm diameter)	m	$ 625.00	100	$ 62,500
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,541,000**
25% Contingency	**$ 385,250**
10% Engineering and Overhead	**$ 154,100**
Total (2009 $)	**$ 2,081,000**

SWMF E

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 71,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.26	$ 13,576
3	Topsoil Stripping	m^3	$ 7.50	7,500	$ 56,250
4	Excavation and Disposal	m^3	$ 16.00	64,400	$ 1,030,400
5	Landscaping	ha	$ 144,000.00	1.03	$ 148,262
6	Outlet Pipe (525 mm diameter)	m	$ 625.00	100	$ 62,500
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,482,000**
25% Contingency	**$ 370,500**
10% Engineering and Overhead	**$ 148,200**
Total (2009 $)	**$ 2,001,000**

SWMF F

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 91,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.83	$ 17,003
3	Topsoil Stripping	m^3	$ 7.50	9,400	$ 70,500
4	Excavation and Disposal	m^3	$ 16.00	85,800	$ 1,372,800
5	Landscaping	ha	$ 144,000.00	1.18	$ 170,259
6	Outlet Pipe (600 mm diameter)	m	$ 700.00	100	$ 70,000
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,892,000**
25% Contingency	**$ 473,000**
10% Engineering and Overhead	**$ 189,200**
Total (2009 $)	**$ 2,555,000**

Appendix J
Table J-2
Cost Estimates for 2025 Ultimate Development - Future Stormwater Management Facilities

SWMF G

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 67,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.14	$ 12,816
3	Topsoil Stripping	m^3	$ 7.50	7,100	$ 53,250
4	Excavation and Disposal	m^3	$ 16.00	59,700	$ 955,200
5	Landscaping	ha	$ 144,000.00	0.99	$ 143,047
6	Outlet Pipe (525 mm diameter)	m	$ 625.00	100	$ 62,500
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,394,000**
25% Contingency	**$ 348,500**
10% Engineering and Overhead	**$ 139,400**
Total (2009 $)	**$ 1,882,000**

SWMF H

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 88,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.77	$ 16,613
3	Topsoil Stripping	m^3	$ 7.50	9,200	$ 69,000
4	Excavation and Disposal	m^3	$ 16.00	83,300	$ 1,332,800
5	Landscaping	ha	$ 144,000.00	1.17	$ 167,871
6	Outlet Pipe (600 mm diameter)	m	$ 700.00	100	$ 70,000
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,845,000**
25% Contingency	**$ 461,250**
10% Engineering and Overhead	**$ 184,500**
Total (2009 $)	**$ 2,491,000**

SWMF I

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 75,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.39	$ 14,367
3	Topsoil Stripping	m^3	$ 7.50	8,000	$ 60,000
4	Excavation and Disposal	m^3	$ 16.00	69,200	$ 1,107,200
5	Landscaping	ha	$ 144,000.00	1.07	$ 153,552
6	Outlet Pipe (525 mm diameter)	m	$ 625.00	100	$ 62,500
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,573,000**
25% Contingency	**$ 393,250**
10% Engineering and Overhead	**$ 157,300**
Total (2009 $)	**$ 2,124,000**

Appendix J
Table J-2
Cost Estimates for 2025 Ultimate Development - Future Stormwater Management Facilities

SWMF J

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 111,000
2	Clearing and Grubbing	ha	$ 6,000.00	3.42	$ 20,546
3	Topsoil Stripping	m^3	$ 7.50	11,400	$ 85,500
4	Excavation and Disposal	m^3	$ 16.00	108,700	$ 1,739,200
5	Landscaping	ha	$ 144,000.00	1.33	$ 190,909
6	Outlet Pipe (600 mm diameter)	m	$ 700.00	100	$ 70,000
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 2,318,000**
25% Contingency	**$ 579,500**
10% Engineering and Overhead	**$ 231,800**
Total (2009 $)	**$ 3,130,000**

SWMF K

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 67,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.13	$ 12,780
3	Topsoil Stripping	m^3	$ 7.50	7,100	$ 53,250
4	Excavation and Disposal	m^3	$ 16.00	59,500	$ 952,000
5	Landscaping	ha	$ 144,000.00	0.99	$ 142,795
6	Outlet Pipe (525 mm diameter)	m	$ 625.00	100	$ 62,500
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,391,000**
25% Contingency	**$ 347,750**
10% Engineering and Overhead	**$ 139,100**
Total (2009 $)	**$ 1,878,000**

SWMF L

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 91,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.85	$ 17,109
3	Topsoil Stripping	m^3	$ 7.50	9,500	$ 71,250
4	Excavation and Disposal	m^3	$ 16.00	86,500	$ 1,384,000
5	Landscaping	ha	$ 144,000.00	1.19	$ 170,905
6	Outlet Pipe (600 mm diameter)	m	$ 700.00	100	$ 70,000
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,905,000**
25% Contingency	**$ 476,250**
10% Engineering and Overhead	**$ 190,500**
Total (2009 $)	**$ 2,572,000**

Appendix J
Table J-2
Cost Estimates for 2025 Ultimate Development - Future Stormwater Management Facilities

SWMF M

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 27,000
2	Clearing and Grubbing	ha	$ 6,000.00	0.85	$ 5,091
3	Topsoil Stripping	m^3	$ 7.50	2,800	$ 21,000
4	Excavation and Disposal	m^3	$ 16.00	16,800	$ 268,800
5	Landscaping	ha	$ 144,000.00	0.55	$ 78,618
6	Outlet Pipe (450 mm diameter)	m	$ 575.00	100	$ 57,500
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 559,000**
25% Contingency	**$ 139,750**
10% Engineering and Overhead	**$ 55,900**
Total (2009 $)	**$ 755,000**

SWMF N

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 73,000
2	Clearing and Grubbing	ha	$ 6,000.00	2.31	$ 13,864
3	Topsoil Stripping	m^3	$ 7.50	7,700	$ 57,750
4	Excavation and Disposal	m^3	$ 16.00	66,100	$ 1,057,600
5	Landscaping	ha	$ 144,000.00	1.04	$ 150,205
6	Outlet Pipe (525 mm diameter)	m	$ 625.00	100	$ 62,500
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,515,000**
25% Contingency	**$ 378,750**
10% Engineering and Overhead	**$ 151,500**
Total (2009 $)	**$ 2,046,000**

SWMF O

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 49,000
2	Clearing and Grubbing	ha	$ 6,000.00	1.58	$ 9,504
3	Topsoil Stripping	m^3	$ 7.50	5,300	$ 39,750
4	Excavation and Disposal	m^3	$ 16.00	40,200	$ 643,200
5	Landscaping	ha	$ 144,000.00	0.82	$ 118,451
6	Outlet Pipe (450 mm diameter)	m	$ 575.00	100	$ 57,500
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 1,018,000**
25% Contingency	**$ 254,500**
10% Engineering and Overhead	**$ 101,800**
Total (2009 $)	**$ 1,375,000**

Appendix J
Table J-2
Cost Estimates for 2025 Ultimate Development - Future Stormwater Management Facilities

SWMF P

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 19,000
2	Clearing and Grubbing	ha	$ 6,000.00	0.60	$ 3,595
3	Topsoil Stripping	m^3	$ 7.50	2,000	$ 15,000
4	Excavation and Disposal	m^3	$ 16.00	10,000	$ 160,000
5	Landscaping	ha	$ 144,000.00	0.43	$ 61,972
6	Outlet Pipe (450 mm diameter)	m	$ 575.00	50	$ 28,750
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 389,000**
25% Contingency	**$ 97,250**
10% Engineering and Overhead	**$ 38,900**
Total (2009 $)	**$ 526,000**

SWMF R

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 15,000
2	Clearing and Grubbing	ha	$ 6,000.00	0.48	$ 2,867
3	Topsoil Stripping	m^3	$ 7.50	1,600	$ 12,000
4	Excavation and Disposal	m^3	$ 16.00	6,900	$ 110,400
5	Landscaping	ha	$ 144,000.00	0.26	$ 36,962
6	Outlet Pipe (525 mm diameter)	m	$ 625.00	50	$ 31,250
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 309,000**
25% Contingency	**$ 77,250**
10% Engineering and Overhead	**$ 30,900**
Total (2009 $)	**$ 418,000**

SWMF S

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 27,000
2	Clearing and Grubbing	ha	$ 6,000.00	0.96	$ 5,774
3	Topsoil Stripping	m^3	$ 7.50	3,200	$ 24,000
4	Excavation and Disposal	m^3	$ 16.00	18,800	$ 300,800
5	Landscaping	ha	$ 144,000.00	0.40	$ 57,505
6	Outlet Pipe (600 mm diameter)	m	$ 700.00	50	$ 35,000
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 551,000**
25% Contingency	**$ 137,750**
10% Engineering and Overhead	**$ 55,100**
Total (2009 $)	**$ 744,000**

Appendix J
Table J-2
Cost Estimates for 2025 Ultimate Development - Future Stormwater Management Facilities

SWMF T

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 20,000
2	Clearing and Grubbing	ha	$ 6,000.00	0.71	$ 4,235
3	Topsoil Stripping	m^3	$ 7.50	2,400	$ 18,000
4	Excavation and Disposal	m^3	$ 16.00	12,200	$ 195,200
5	Landscaping	ha	$ 144,000.00	0.33	$ 47,395
6	Outlet Pipe (600 mm diameter)	m	$ 700.00	50	$ 35,000
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 420,000**
25% Contingency	**$ 105,000**
10% Engineering and Overhead	**$ 42,000**
Total (2009 $)	**$ 567,000**

SWMF U

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 20,000
2	Clearing and Grubbing	ha	$ 6,000.00	0.62	$ 3,723
3	Topsoil Stripping	m^3	$ 7.50	2,100	$ 15,750
4	Excavation and Disposal	m^3	$ 16.00	10,500	$ 168,000
5	Landscaping	ha	$ 144,000.00	0.44	$ 63,489
6	Outlet Pipe (525 mm diameter)	m	$ 625.00	50	$ 31,250
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 403,000**
25% Contingency	**$ 100,750**
10% Engineering and Overhead	**$ 40,300**
Total (2009 $)	**$ 545,000**

SWMF V

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 27,000
2	Clearing and Grubbing	ha	$ 6,000.00	0.91	$ 5,454
3	Topsoil Stripping	m^3	$ 7.50	3,000	$ 22,500
4	Excavation and Disposal	m^3	$ 16.00	18,600	$ 297,600
5	Landscaping	ha	$ 144,000.00	0.57	$ 82,345
6	Outlet Pipe (450 mm diameter)	m	$ 575.00	50	$ 28,750
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 564,000**
25% Contingency	**$ 141,000**
10% Engineering and Overhead	**$ 56,400**
Total (2009 $)	**$ 762,000**

Appendix J
Table J-2
Cost Estimates for 2025 Ultimate Development - Future Stormwater Management Facilities

SWMF W

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 35,000
2	Clearing and Grubbing	ha	$ 6,000.00	1.17	$ 7,031
3	Topsoil Stripping	m^3	$ 7.50	3,900	$ 29,250
4	Excavation and Disposal	m^3	$ 16.00	26,600	$ 425,600
5	Landscaping	ha	$ 144,000.00	0.68	$ 97,450
6	Outlet Pipe (450 mm diameter)	m	$ 575.00	50	$ 28,750
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 724,000**
25% Contingency	**$ 181,000**
10% Engineering and Overhead	**$ 72,400**
Total (2009 $)	**$ 978,000**

SWMF X

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 104,000
2	Clearing and Grubbing	ha	$ 6,000.00	3.23	$ 19,394
3	Topsoil Stripping	m^3	$ 7.50	10,700	$ 80,250
4	Excavation and Disposal	m^3	$ 16.00	101,200	$ 1,619,200
5	Landscaping	ha	$ 144,000.00	1.28	$ 184,394
6	Outlet Pipe (600 mm diameter)	m	$ 700.00	100	$ 70,000
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 2,178,000**
25% Contingency	**$ 544,500**
10% Engineering and Overhead	**$ 217,800**
Total (2009 $)	**$ 2,941,000**

SWMF Y

Item	Description	Unit	Unit Price	Estimated Quantity	Amount
1	Mobilization & Demobilization (5%)				$ 110,000
2	Clearing and Grubbing	ha	$ 6,000.00	3.41	$ 20,476
3	Topsoil Stripping	m^3	$ 7.50	11,300	$ 84,750
4	Excavation and Disposal	m^3	$ 16.00	108,300	$ 1,732,800
5	Landscaping	ha	$ 144,000.00	1.32	$ 190,519
6	Outlet Pipe (600 mm diameter)	m	$ 700.00	100	$ 70,000
7	Control Structure	each	$ 100,000.00	1	$ 100,000

Sub Total	**$ 2,309,000**
25% Contingency	**$ 577,250**
10% Engineering and Overhead	**$ 230,900**
Total (2009 $)	**$ 3,118,000**