City of Leduc

Water Master Plan Update 2021

Final Report

Prepared by:

AECOM Canada Ltd.
101 – 18817 Stony Plain Road NW
Edmonton, AB T5S 0C2
Canada

T: 780 486 7000

www.aecom.com

Prepared for:

City of Leduc
#1 Alexandra Park
Leduc, AB T9E 4C4

Attention: Ryan Graham

Date: October 2021

Project #: 60609727

Distribution List

# Hard Copies	PDF Required	Association / Company Name
	✓	City of Leduc
	✓	AECOM Canada Ltd.

Revision History

Rev #	Date	Revised By:	Revision Description
0	May 4, 2021	SF	DRAFT 1 Report
1	June 2, 2021	SF	DRAFT 2 Report – for City Operations Review
2	July 26, 2021	SF	DRAFT 2 Report
3	Sept 30, 2021	SF	Final Draft Report
4	Oct 29, 2021	SF	Final Report

AECOM Imagine it. Delivered.

AECOM Canada Ltd.
101 – 18817 Stony Plain Road NW
Edmonton, AB T5S 0C2
Canada

T: 780 486 7000

www.aecom.com

October 29, 2021

Project #
60609727

Ryan Graham
Infrastructure Manager
City of Leduc
#1 Alexandra Park
Leduc, AB T9E 4C4

Attention: Ryan Graham

Dear Ryan:

Subject: Water Master Plan Update 2021
Final Report

AECOM is pleased to submit our final report for the City of Leduc Water Master Plan Update. If you have any comments or questions, please contact the undersigned.

Sincerely,
AECOM Canada Ltd.

Sean Frank

Sean Frank, P.Eng.
Water Resources Engineer
Sean.Frank@aecom.com

KStLouis

Kristin St. Louis, P.Eng.
Senior Water Resources Engineer
Kristin.Stlouis@aecom.com

SF:blb
Encl.

Statement of Qualifications and Limitations

The attached Report (the "Report") has been prepared by AECOM Canada Ltd. ("AECOM") for the benefit of the Client ("Client") in accordance with the agreement between AECOM and Client, including the scope of work detailed therein (the "Agreement").

The information, data, recommendations and conclusions contained in the Report (collectively, the "Information"):

- is subject to the scope, schedule, and other constraints and limitations in the Agreement and the qualifications contained in the Report (the "Limitations");
- represents AECOM's professional judgement in light of the Limitations and industry standards for the preparation of similar reports;
- may be based on information provided to AECOM which has not been independently verified;
- has not been updated since the date of issuance of the Report and its accuracy is limited to the time period and circumstances in which it was collected, processed, made or issued;
- must be read as a whole and sections thereof should not be read out of such context;
- was prepared for the specific purposes described in the Report and the Agreement; and
- in the case of subsurface, environmental or geotechnical conditions, may be based on limited testing and on the assumption that such conditions are uniform and not variable either geographically or over time.

AECOM shall be entitled to rely upon the accuracy and completeness of information that was provided to it and has no obligation to update such information. AECOM accepts no responsibility for any events or circumstances that may have occurred since the date on which the Report was prepared and, in the case of subsurface, environmental or geotechnical conditions, is not responsible for any variability in such conditions, geographically or over time.

AECOM agrees that the Report represents its professional judgement as described above and that the Information has been prepared for the specific purpose and use described in the Report and the Agreement, but AECOM makes no other representations, or any guarantees or warranties whatsoever, whether express or implied, with respect to the Report, the Information or any part thereof.

Without in any way limiting the generality of the foregoing, any estimates or opinions regarding probable construction costs or construction schedule provided by AECOM represent AECOM's professional judgement in light of its experience and the knowledge and information available to it at the time of preparation. Since AECOM has no control over market or economic conditions, prices for construction labour, equipment or materials or bidding procedures, AECOM, its directors, officers and employees are not able to, nor do they, make any representations, warranties or guarantees whatsoever, whether express or implied, with respect to such estimates or opinions, or their variance from actual construction costs or schedules, and accept no responsibility for any loss or damage arising therefrom or in any way related thereto. Persons relying on such estimates or opinions do so at their own risk.

Except (1) as agreed to in writing by AECOM and Client; (2) as required by-law; or (3) to the extent used by governmental reviewing agencies for the purpose of obtaining permits or approvals, the Report and the Information may be used and relied upon only by Client.

AECOM accepts no responsibility, and denies any liability whatsoever, to parties other than Client who may obtain access to the Report or the Information for any injury, loss or damage suffered by such parties arising from their use of, reliance upon, or decisions or actions based on the Report or any of the Information ("improper use of the Report"), except to the extent those parties have obtained the prior written consent of AECOM to use and rely upon the Report and the Information. Any injury, loss or damages arising from improper use of the Report shall be borne by the party making such use.

This Statement of Qualifications and Limitations is attached to and forms part of the Report and any use of the Report is subject to the terms hereof.

AECOM: 2015-04-13

Authors

PROFESSIONAL ENGINEER ALBERTA
SEAN T. FRANK
29 Oct. 2021 ID:118437

Report Prepared By:

Sean Frank, P.Eng.
Water Resources Engineer

PROFESSIONAL ENGINEER ALBERTA
KRISTIN J. ST. LOUIS
2021-10-29
ID 77973

Report Reviewed By:

Kristin St. Louis, P.Eng.
Senior Water Resources Engineer

PERMIT TO PRACTICE
AECOM CANADA LTD.
RM SIGNATURE: Chudarchuk
RM APEGA ID #: M77586
DATE: 2021-10-29
PERMIT NUMBER: P010450
The Association of Professional Engineers and Geoscientists of Alberta (APEGA)

Executive Summary

The City of Leduc (the City) retained AECOM Canada Ltd. to update the Water Master Plan. The City of Leduc receives potable water from EPCOR Water, supplied by the Capital Region Southwest Water Services Commission (CRSWSC). The CRSWSC is updating their water policy with an intent to ensure that adequate flow is supplied by the CRSWSC to existing connection locations whereas a municipality is required to fund pipeline extensions for new connections. The City's water distribution system includes three reservoirs and pumphouses. Two of the reservoirs are directly supplied by the CRSWSC and the third is filled off the distribution system. The City's distribution network is currently located within one pressure zone and consists of over 175 km of water mains.

The City of Leduc Water Master Plan was last updated in 2014. Since the completion of the 2014 Water Master Plan, the City has experienced significant development, as well as construction of a new reservoir and pumphouse.

This report provides an update to the Water Master Plan. The overall goal of the assessment is to update and assess the existing system with recently completed projects and provide a road map for future development.

Design Criteria

Existing land use and water consumption data was assessed to determine the water demand of the water distribution system, as well as peaking factors for maximum day demand and peak hour demand. Table ES.1 summarizes water consumption rates and peaking factors for the City based on the existing development condition.

Table ES.1. City of Leduc Average Day Demand and Peaking Factors

Average Day	Maximum Day		Peak Hour	
Demand (L/s)	Demand (L/s)	Peak Factor	Demand (L/s)	Peak Factor
86.6	141.0	1.63	222.1	2.56

Utilizing water consumption records, the historical demand was split between residential and non-residential land uses. In general, the existing water consumption rates were found to be lower than recommended for design standards. The historical consumption rates were used for the existing system, and development standards were utilized for future development area. Design standards for future development have been recommended and summarized in Table ES.2.

Table ES.2. Recommended Design Standards Summary

Criteria	Unit	Value	Referenced Standard
Minimum Peak Hour Pressure	kPa	280	EPCOR
Minimum Maximum Day + Fire Pressure	kPa	140	EPCOR
Minimum Maximum Day Pressure (Fire Sprinklers)	kPa	350	EPCOR
Maximum Pressure (Distribution System)	kPa	570	Leduc
Maximum Allowable Pressure (for Services)	kPa	570	Leduc
Average Day Demand (ADD) - Residential	L/c/d	250	Leduc
Average Day Demand (ADD) – Commercial	L/Ha/d	22,500	Leduc
Average Day Demand (ADD) – Industrial/Institutional	L/Ha/d	11,000	Leduc
Maximum Day Demand (MDD) Peaking Factor	-	1.8	Leduc
Peak Hour Demand (PHD) Peaking Factor	-	3.0	EPCOR
Maximum Hazen-William's Coefficient	-	120	EPCOR
Fire Flow - Single Family	L/s	115	Leduc
Fire Flow - Mid-Value Multi-Family	L/s	227	Leduc

Criteria	Unit	Value	Referenced Standard
Fire Flow - High Value Multi-Family	L/s	227	Leduc
Fire Flow - High Value Properties/Non-Res	L/s	227	Leduc

The above recommended design standards were reviewed with the City of Leduc prior to the publication of this Master Plan and were included in the City of Leduc Engineering Design Standards dated May 2021.

Existing System Description

The water distribution system is supplied by two connections to the CRSWSC water transmission system at the North Reservoir and the Robinson Reservoir. The Corinthia Reservoir fills from the distribution system.

The existing storage capacity is 24,900 m^3, which is sufficient for the existing storage requirement of 14,212 m^3. The existing pumping capacity is 1240.5 L/s. With the largest pump at the North Reservoir (227 L/s) out of service, the capacity is 1013.5 L/s. The existing pumping capacity is sufficient for the existing development condition, which is governed by the maximum day plus fire demand scenario at 363 L/s.

The North and Corinthia Reservoirs operate in tandem, maintaining a setpoint pressure at the Civic Centre of 570 kPa. As demand in the system increases and pressure within the system drops, additional pumps activate to supplement system pressure until the demand decreases. The Robinson reservoir operates to maintain a setpoint located at the Robinson Reservoir of approximately 370 kPa.

The existing water distribution system is comprised of mainly Polyvinyl Chloride (PVC), Asbestos Cement (AC), with some Steel and High Density Polyethylene (HDPE) pipe. The watermains range in size from 100 mm in diameter to 600 mm in diameter with installation dates as early as 1958.

Existing System Assessment

The existing water distribution system in general is sufficient to provide adequate fire flows to the system with the following exceptions:

- There are a number of dead ends within cul-de-sacs which can be opportunistically upgraded with neighborhood improvements. However, in general the adjacent watermains had sufficient fire flows and thus immediate improvements are not recommended.
- The Linsford Park school, the Civic Centre, and the East Elementary school. Pipe upgrades are proposed in these areas to increase the available fire flow.

The system pressure during average day and peak hour scenarios exceeds the maximum recommended pressure of 570 kPa. Pressures within the existing system are high in the northwest portion of the City, west of the QEII, as well as in the northeast portion of the northern industrial area. Pressures within these areas reach up to 675 kPa.

Additional scenarios were assessed to determine the existing systems capability to supply water during emergency scenarios. In general, the emergency assessment concluded that the existing system has sufficient redundancy to supply the water distribution system in each emergency scenario.

Future System Assessment

The future system assessment was developed to provide a roadmap for growth. The development plan is for sequential growth and can adjusted if growth progresses differently than is presented in this report. The future system was assessed in five development horizons as follows:

- Stage 1: 5-10 Year Development Horizon
- Stage 2: 10-20 Year Development Horizon
- Stage 3: 20-30 Year Development Horizon
- Stage 4: 30+ Years within the Current Municipal Boundary
- Stage 5: Potential Growth Areas Outside Current Municipal Boundary

Throughout the future development horizons, additional piping, PRVs, storage, and pumping is required as the demand of the system increases. The existing water distribution system pipes are sufficient to provide servicing for the future development areas without major upgrades.

It is recommended to create pressure zones to mitigate the high pressures observed at the edges of the existing development and as development extends further. The ground elevation generally decreases as development progresses north and west for the area west of the QEII. System pressure will increase proportionally as ground elevations increase.

The pressure zones are proposed to be implemented during the Stage 1 development horizon. The pressure zone boundary is proposed on the north side of Black Gold Drive, utilizing four new PRVs (PRVs A through D) for Stage 1 and the closure of two smaller diameter pipes. Once constructed, the pressure of the north pressure zone will be regulated by the North Pumphouse. The setpoint of the North Pumphouse is proposed to be lowered to a hydraulic grade line (HGL) of 770 m (lowered from 785 m). The pressure of the south zone will be regulated by the Robinson and Corinthia Reservoirs. The setpoint of the south zone is proposed to be increased to an HGL of 790 m (increased from 785 m).

The PRVs should be oriented such that the south zone is able to supplement flows to the north zone during periods of high demand. The pressure setpoint of the proposed PRVs has been selected slightly lower than the operating pressure of the North Reservoir so that during average day demand, the north reservoir supplies all water to the north zone. As demand increases dropping system pressure, the PRVs will open, supplementing flows to the north zone. As future development occurs, additional PRVs are required (PRV-F1, PRV-F2 and PRV-F3) to maintain the boundary of the pressure zones.

An implementation plan has been developed detailing the required water distribution infrastructure as development progresses. The implementation plan is summarized as follows:

- Water main improvements for the Civic Centre and the East Elementary School should be completed as soon as practical to improve local fire flows.
- Stage 1 Infrastructure Requirements:
  - Pressure zone implementation including the construction of PRVs A through D.
  - The Linsford Park School Upgrade.
  - The Willow Park upgrade.
  - Local water distribution pipes should be installed as required to provide servicing for the development as it progresses.
- Stage 2 Infrastructure Requirements:
  - An additional PRV (PRV-F1) on the east edge of Telford Lake.
  - Expansion of the Robinson and the North Reservoirs by 4,500 m^3 and 8,000 m^3, respectively. The timing and demand triggers for the reservoir expansion is summarized in Table ES.3.

  - An additional QEII crossing at 65 Avenue.
  - Local distribution pipes as required for development.
- Stage 3 Infrastructure Requirements:
  - An additional QEII crossing including PRV (PRV-F2) at 38 Avenue.
  - A Highway 2A crossing between Southfork and Tribute.
  - An additional expansion of the Robinson Reservoir of 14,000 m^3.
  - Upgrades of the existing distribution pumps at the Robinson Reservoir with associated pumphouse improvements required to accommodate the increased pumping.
  - Local distribution pipes as required for development.
- Stage 4 Infrastructure Requirements:
  - Local distribution pipes as required for development.
- Stage 5 Infrastructure Requirements:
  - An additional QEII crossing including PRV (PRV-F3).
  - A new reservoir with total capacity of 36,550 m^3.
  - A new pumphouse at the new reservoir with 500 HP pumping capacity.
  - Local distribution pipes as required for development.

Table ES.3 summarizes the demand triggers and expansion volumes for the proposed reservoir expansion.

Table ES.3. Reservoir and Pumphouse Upgrade Triggers

	Upgrade Size	ADD Trigger (L/s)	Development Stage
Robinson Reservoir Expansion 1	4500 m^3	144	Stage 2
North Reservoir Expansion	8000 m^3	173	Stage 2
Robinson Reservoir Expansion 2 and Pump Upgrades	14,000 m^3 200 HP	224	Stage 3
New Reservoir & Pumphouse	36,550 m^3 500 HP	315	Stage 5

Cost Estimates

Cost estimates have been developed for the proposed system improvements and expansions and are detailed in Section 6. For each of the proposed system improvements and expansions, the associated payment method has been indicated. It is assumed that funding will come from one of three sources:

- <u>City</u>: Improvements required to benefit the existing system to be paid from the City's capital budget.
- <u>Off-Site Levies</u>: Future system expansions, including reservoir upgrades, pump upgrades, pressure reducing valves, as well as highway crossing and associated connections back to the existing system will be paid through off-site levies.

A cost summary is provided in Table ES.4. Existing system improvement include 30% contingency, and all other costs include 40% contingency.

Table ES.4. Cost Summary

Upgrade	Estimated Cost (million $)						Payment Method
	Existing	Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	
Existing System Improvement	$1.10M	$0.89M	-	-	-	-	City
Pressure Reducing Valves	-	$2.36M	$0.5M	$0.5M	-	$0.5M	Off-Site Levies
Water Mains – Highway Crossings			$2.1M	$2.2M	-	$1.5M	Off-Site Levies
Reservoir/Pumphouse Expansion			$11.4M	$22.9M		$39.5M	Off-Site Levies

Table of Contents

List of Figures

List of Tables

Appendices

List of Acronyms

The following acronyms used in this report are defined as follows:

Abbreviation	Definition
%	Percent
AC	Asbestos Cement
ADD	Average Day Demand
ASP	Area Structure Plan
C Coefficient	Hazen Williams 'C' Coefficient
CP Rail	Canadian Pacific Railway Company
CRSWSC	Capital Region Southwest Water Services Commission
EIA	Edmonton International Airport
ft	Feet
GPM	(United States) Gallons Per Minute
GIS	Geographic Information System
h	hour
ha	Hectares
HDPE	High Density Polyethylene
HGL	Hydraulic Grade Line
HP	Horsepower
km	Kilometer
kPa	Kilopascal
L	Litre
m	Meters
m^3	Cubic Meters
MDD	Maximum Day Demand
mm	Millimeters
PHD	Peak Hour Demand
PRV	Pressure Reducing Valve
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
QEII	Queen Elizabeth II Highway
s	Second

1. Introduction

1.1 Background

The City of Leduc is located in the Edmonton Metropolitan Area and is one of the fastest growing communities in Alberta. The City population is over 33,000, and is located along major transportation corridors, including the adjacent Edmonton International Airport. The study area is shown on Figure 1.1.

The City of Leduc receives potable water from EPCOR Water, supplied by the Capital Region Southwest Water Services Commission (CRSWSC). The CRSWSC is updating their water policy with an intent to ensure that adequate flow is supplied by the CRSWSC to existing connection locations. Municipalities will be required to fund pipeline extensions for new connections. The City's water distribution system includes three reservoirs and pumphouses. Two of the reservoirs are directly supplied by the CRSWSC and the third is filled off the distribution system. The City's distribution network is currently located within one pressure zone and consists of over 175 km of water mains.

The City of Leduc Water Master Plan was last updated in 2014. Since the completion of the 2014 Water Master Plan, the City has experienced significant development, as well as construction of a new reservoir and pumphouse. In addition, many communities, including the City of Leduc, are seeing a decreasing trend in water use as water reduction measures are applied. An update to the Water Master Plan is required to recommend design criteria, validate system recommendations, and provide a road map for future development.

1.2 Scope of Work

The overall objective of the study is to update the water servicing concept for the City of Leduc and to review the existing system, propose improvements to address any existing system deficiencies as well as to support future development, and propose future expansion of the water distribution system. Growth scenarios for five growth scenarios were developed, and used to outline a framework for water servicing of future development.

The scope of work includes the following:

- Collect and review all data relevant to the project.
- Complete a water demand analysis to determine consumption rates and peaking factors.
- Conduct a field investigation of the pump stations.
- Assess City standards and recommend modifications as appropriate.
- Inventory the existing distribution system.
- Verify the existing water system model against hydrant flow data.
- Evaluate the existing water supply and distribution system and identify deficiencies.
- Develop a unidirectional flushing program.
- Assess and recommend system improvements.
- Recommend an implementation plan for proposed system improvements.
- Develop servicing concepts for future system expansion for the short, medium, and long term, and ultimate development scenarios.
- Review emergency scenarios, including closed valve connections to the Edmonton International Airport and Leduc County.
- Develop cost estimates for existing system upgrades, as well as future development scenarios.
- Prepare a report detailing the findings of the study and the proposed water servicing concepts.

1.3 Data Collection and Review

Relevant data was collected and reviewed, including existing reports, survey and topographic data, record information and as-built data, servicing standards, reservoir and pump data.

Relevant reports reviewed as part of the study include:

- City of Leduc PLC Control Narrative, North Reservoir Pumping Facility Upgrades 2019, Vector Group, December 2020.
- City of Leduc 2018 Sanitary Servicing Study, Cole Engineering Group, October 2019.
- City of Leduc Robinson Pump House and Reservoir Control Philosophy, Associated Engineering, April 2016.
- City of Leduc Benefit Analysis Technical Memorandum, Associated Engineering, March 2015.
- City of Leduc - Leduc Water Master Plan – Revised, Associated Engineering, December 2014.
- City of Leduc Facilities Assessment and Radio Path Study, Associated Engineering, June 2013.
- Capital Region Southwest Water Services Commission (CRSWSC) – Master Plan Update, January 2020.
- Capital Region Southwest Water Services Commission (CRSWSC) – Master Plan Update, November 2013.
- ASPs and Outline Plans.
- City of Leduc Engineering Design Standards.
- Other applicable legislation, regulations, and guidelines pertaining to water servicing within the City of Leduc.

The following drawings were reviewed for the three reservoir-pumphouses and existing water distribution system:

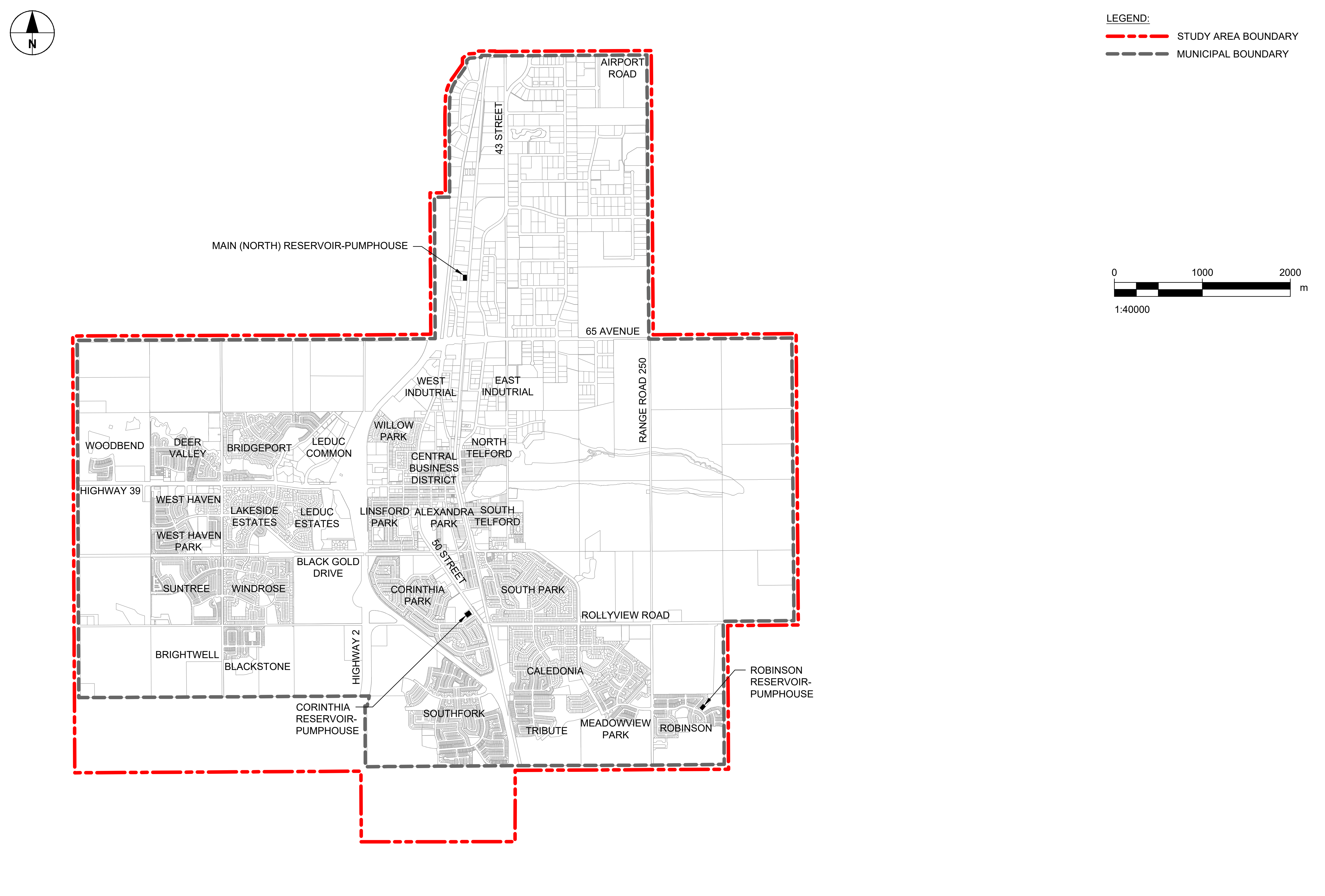
- City of Leduc Robinson Reservoir and Pump Station Issued for Tender and Construction Drawings, Associated Engineering, April 2016.
- Town of Leduc North Reservoir As-Built Drawings, Associated Engineering, July 1985.
- Town of Leduc Reservoir & Pumphouse As-Built Drawings, Associated Engineering, July 1975.
- City of Leduc Linsford Watermain Loop Issued for Construction Drawings, ISL Engineering, October 2018.
- Leduc 2017 Capital Utility Program Watermain Looping As-Built Drawings, WSP, April 2018.
- City of Leduc North Telford Neighbourhood Renewal Issued for Construction Drawings, Select Engineering, June 2018.
- Woodbend Stage 1 Issued for Approval Drawings, Select Engineering, August 2016.
- 2019 LiDAR data and contour maps.
- 2019 GIS base files and water distribution layers.
- Land Use Zoning as per Bylaw 809-2013.

The following information was reviewed, specific to the Edmonton International Airport:

- EIA Second Reservoir Proposed Control Philosophy, AECOM and Vector Electric, December 2014.
- Operation and Maintenance Data Manual Edmonton International Airport Central Utilities Plant – 2nd Reservoir, Stuart Olson Construction Ltd., January 2013.
- Edmonton Airports New Water Reservoir Expansion 2012 As-Built Drawings, AECOM, August 2013.
- 2021 CAD base files.

System data was reviewed and utilized for the existing system assessment, including:

- 2014 WaterCAD model
- 2016 – 2019 Reservoir Flow Data
- 2018 – 2019 Metered Billing Data



AECOM

Figure: 1.1

STUDY AREA

WATER MASTER PLAN UPDATE

CITY OF LEDUC

Project No.: 60609727

ANSI B 279.4mm x 431.8mm Approved: ____ Checked: ____ Designer: ____ Project Management Initials: ____

Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0101 - STUDY AREA.DWG

Last saved by: ANDREW.FLECK(2021-10-03) Last Plotted: 2021-10-03

2. Study Area

2.1 General

The study area includes the lands within the City of Leduc boundary, as well as potential future growth areas to the south of the current City limits. The City of Leduc is bordered by the Edmonton International Airport (EIA) to the northwest, and Leduc County on all other sides with Nisku to the north and agricultural areas to the east, south and west.

The Queen Elizabeth II (QEII) Highway runs north-south through the City, which also branches off to Highway 2A. Highway 39 runs east-west through the west half of the City, connecting to the QEII. The CP Rail also runs north-south through the City with a branch to the west. Telford Lake is located within the east half of the City, east of 46 Street.

2.2 Land Use

The west half of the City, including areas west of the QEII, is predominately single family residential. The north quarter sections, south of 65 Avenue, is primarily zoned for commercial development to provide a buffer between the City and the EIA. Future development to the west and south is expected to be single family residential.

The east half of the City has a blend of zoning. The downtown area along the 50 Street and 50 Avenue corridors contains commercial, institutional, and multi-family residential areas. North of 65 Avenue is all industrial development, and south of 50 Avenue and west of Black Gold Drive is primarily single family residential. Future areas around Telford Lake are zoned for non-residential development, with residential development anticipated to continue south.

The land use for the existing and future development conditions are shown in Figures 2.1 and 2.2, respectively. The area of land use was measured based on the provided land use map for 2019; the breakdown of existing land use by area is summarized in Table 2.1.

Table 2.1. Existing Land Use Summary

Land Use	Area (ha)	Percentage (%)
Single Family Residential	527	38
Multi-Residential	37	2.7
High-Density Residential	2.5	0.2
Commercial	131	10
Industrial	510	37
Institutional	165	12
TOTAL	1372.5	100

As shown in Table 2.1, approximately 40% of the existing development is residential and 60% is comprised of non-residential uses (commercial, industrial, and institutional).

The future land use was taken from the various ASPs and Outline Plans available at the time of this report. The future land use within the current municipal boundary is summarized in Table 2.2.

Table 2.2. Future Land Use Summary

Land Use	Area (ha)	Percentage (%)
Single Family Residential	484.2	36
Multi-Residential	22.5	1.7
High-Density Residential	2.2	0.2
Commercial	209.1	16
Industrial	545.6	41
Institutional	66.3	5
TOTAL	1,330	100

Both the existing and future land use areas have un-serviced areas such as parks, walkways, stormwater management facilities, etc. that have not been included in the above land use summaries. The tables are intended to be representative of the overall developed area serviced by the water distribution system.

2.3 Development Staging Scenarios

Five development staging scenarios were considered for this assessment:

- Stage 1 – 5-10 Year Development Horizon
- Stage 2 – 10-20 Year Development Horizon
- Stage 3 – 20-30 Year Development Horizon
- Stage 4 – 30+ Year Development Horizon within Municipal Boundary
- Stage 5 – Potential Growth Outside the Current Municipal Boundary

The development staging scenarios correlate to the staging considered in the 2018 Sanitary Sewer Master Plan and were developed based on input provided by the City of Leduc Planning and Economic Development, and Engineering departments. The development staging scenarios are illustrated on Figure 2.3.

The development staging scenarios help form the basis of the servicing framework and infrastructure staging.

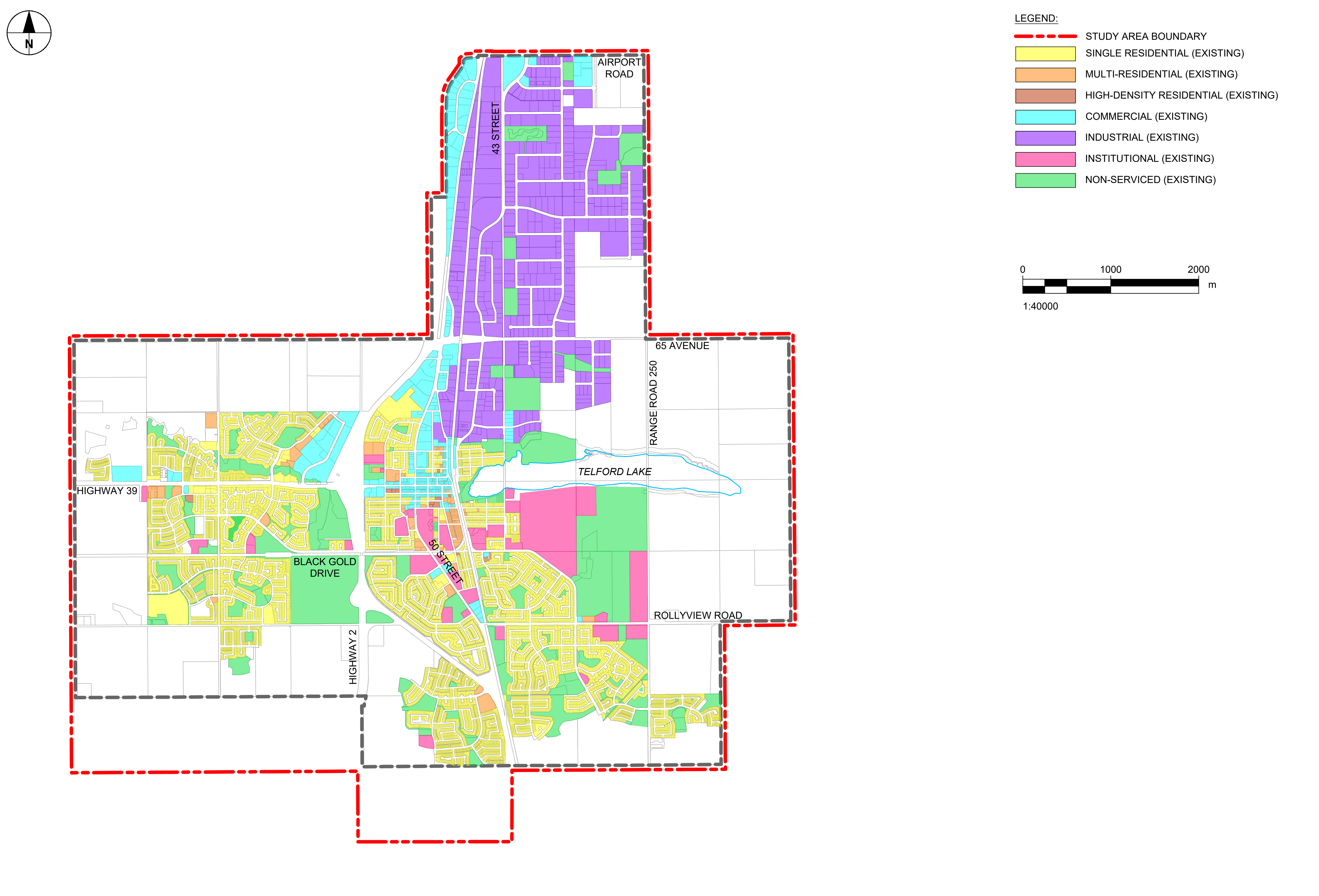


Figure: 2.1

LAND USE - EXISTING SYSTEM

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved:
Checked:
Designer:
Project Management Initials:
Last saved by: ANDREW.FLECK(2021-10-03) Last Plotted: 2021-10-03
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0201 - LAND USE - EXISTING SYSTEM.DWG

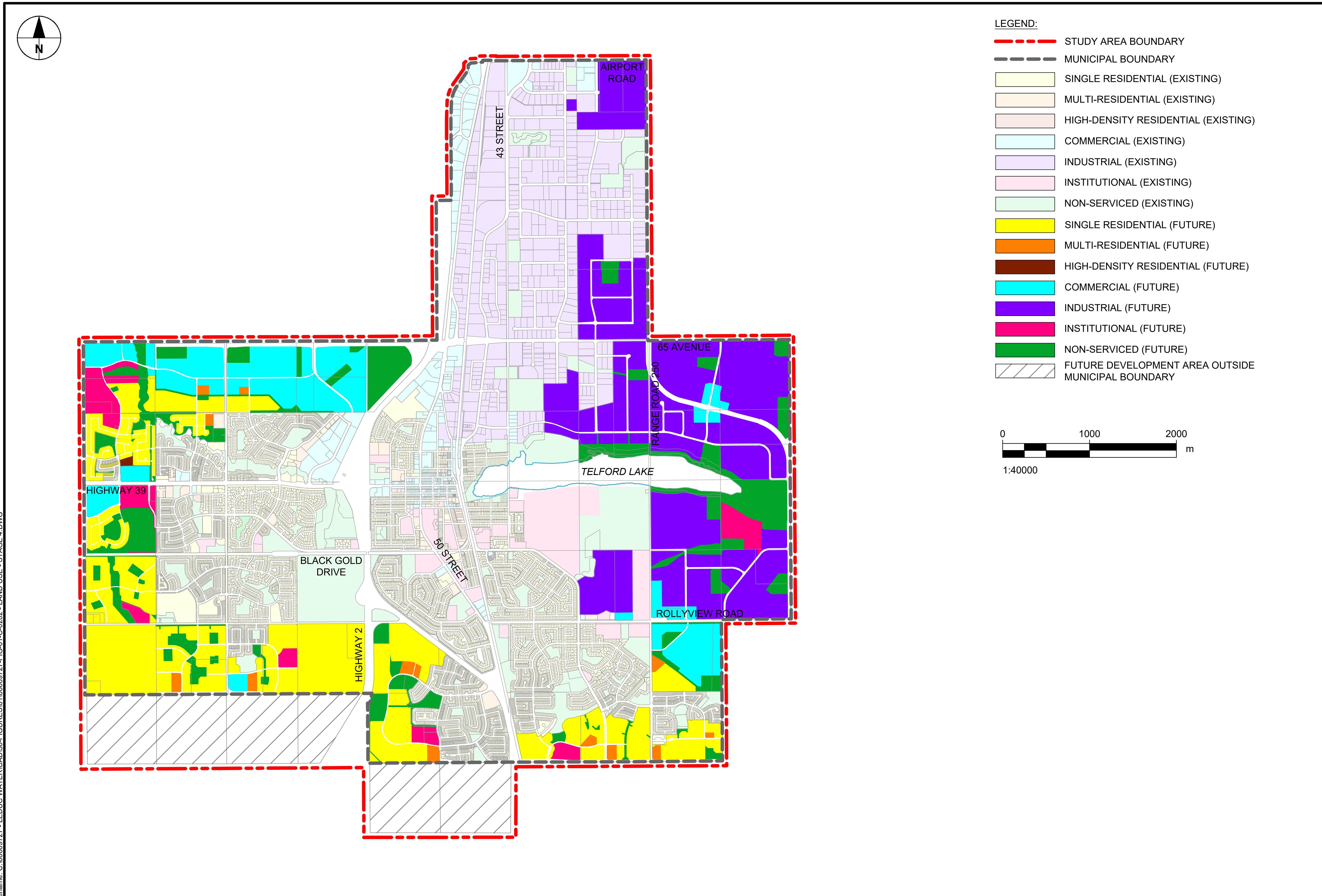


Figure: 2.2

AECOM

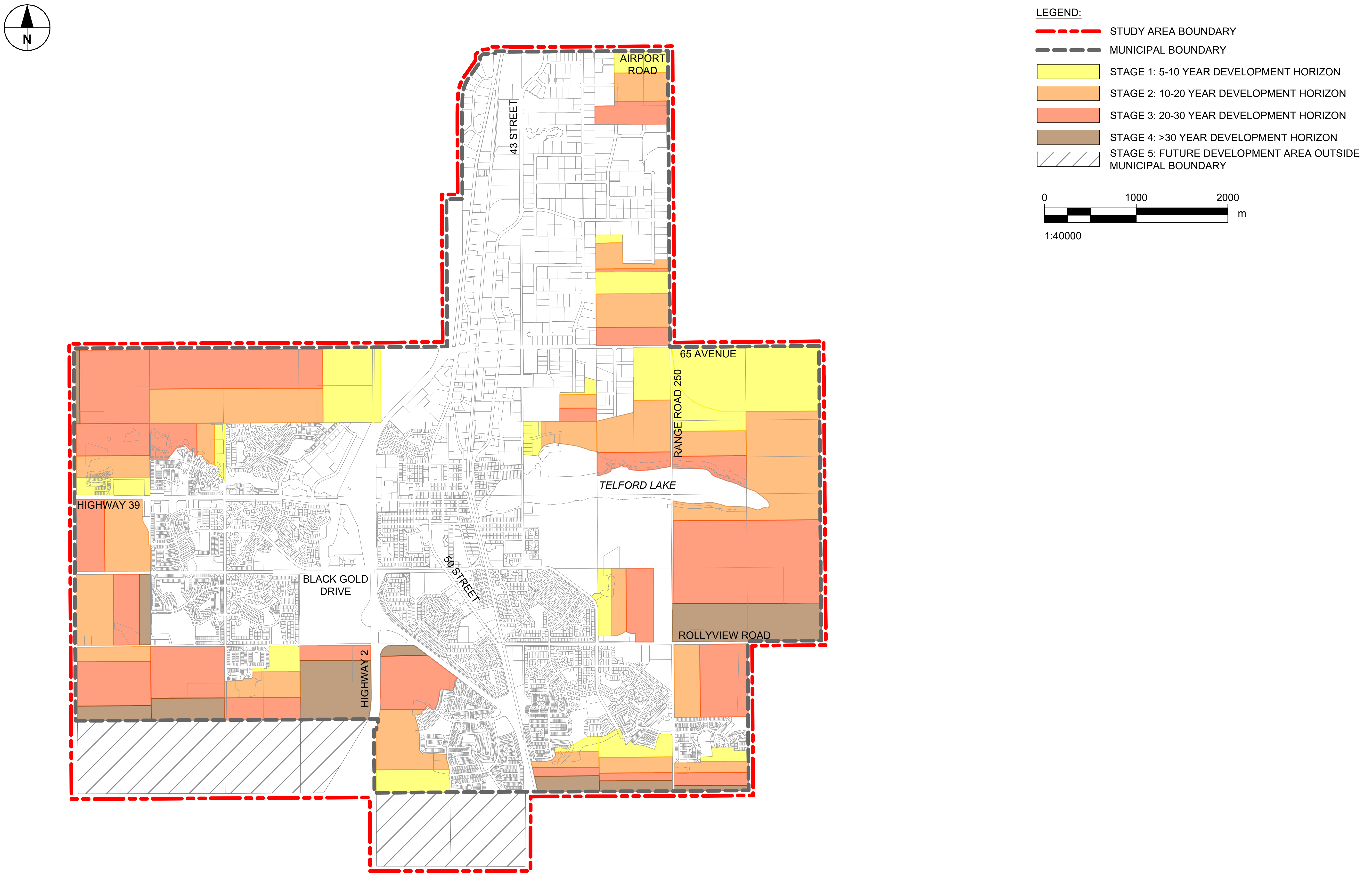
LAND USE - STAGE 4

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved: ____ Checked: ____ Designer: ____ Project Management Initials: ____

Last saved by: ANDREW.FLECK(2021-10-03) Last Plotted: 2021-10-03
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0202 - LAND USE - STAGE 4.DWG



AECOM

Figure: 2.3

DEVELOPMENT STAGING

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved: ____ Checked: ____ Designer: ____ Project Management Initials: ____

Last saved by: ANDREW.FLECK(2021-06-24) Last Plotted: 2021-06-24
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0203 - DEVELOPMENT STAGING.DWG

3. Design Criteria and Standards

This section provides the design criteria employed in the analysis of the water distribution system. Design criteria recommendations are provided for both the existing and future development scenarios. A review of the City of Leduc's existing Engineering Design Standards was conducted to assist in determining appropriate design criteria.

3.1 Population Projections and Development Staging

Based on the Leduc municipal census, the population in 2019 was 33,032 persons, and was considered to be the existing population.

The development anticipated to occur within each development stage was defined, and the corresponding population and annual growth rates were calculated. Table 3.1 summarizes the anticipated population to occur by the end of each development stage, and Table 3.2 summarized the anticipated development area to occur by the end of each development stage.

Table 3.1. Development Staging - Population Projections

Development Stage	Population*	Annual Growth Rate
Existing (2019)	33,032	-
Stage 1	38,550	1.60%
Stage 2	50,054	2.58%
Stage 3	65,201	2.62%
Stage 4	69,096	2.27%

**Populations shown are based on full build out of the development stage.*

Table 3.2. Development Staging - Development Projections

Development Stage	Developed Area*	Annual Growth Rate
Existing (2019)	1,373	-
Stage 1	1,619	1.67%
Stage 2	2,064	2.46%
Stage 3	2,635	2.47%
Stage 4	2,702	0.64%

**Populations shown are based on full build out of the development stage.*

3.2 Water Demand

AECOM assessed water consumption data from 2018 and 2019 to determine the existing water consumption rates for the City and form the basis of recommendation for future system design water consumption rates.

Water demands that were considered for this assessment are as follows:

Average Day Demand
The Average Day Demand (ADD) provides a baseline of the water consumption demands. It is calculated as the annual average water consumption rates. The ADD is used to analyze typical operating conditions and indicates trends in water consumption rates over time. ADD is also used in the sizing of storage reservoirs.

Maximum Day Demand
The Maximum Day Demand (MDD) is typically calculated as a 5-day rolling average. The water consumption over the maximum 5-day period is averaged and compared to the Average Day Demand to provide a peaking factor, usually 1.7 to 2 times the ADD. Use of a 5-day rolling average helps to account for atypically high peak days that may occur due to fire, water main breaks, etc. The MDD is used for the assessment of fire flow scenarios, such that the system can be designed to provide fire protection during high demand periods.

Peak Hour Demand
Peak Hour Demand (PHD) indicates the highest one-hour peak in daily water usage. The PHD is compared to the Average Day Demand to determine a peaking factor, usually 3 to 4 times the ADD. The PHD is used to assess the system during peak flow conditions, particularly the pumping systems.

3.2.1 Existing System Demand

The City provided water consumption data based on total outflow from the City's reservoir-pumphouses, as well as monthly billing records based on location and land use. The reservoir outflow data was utilized to determine the ADD, as well as peaking factors for MDD and PHD. The billing records were utilized to confirm the ADD as well as to determine the water consumption based on residential and various non-residential land uses. Reservoir outflow data was provided for 2019, and the billing records were provided for 2018 and 2019.

Table 3.3 summarizes the findings of the reservoir outflow analysis. The analysis assumed that since the City's South Reservoir operates by filling off of the Leduc water distribution system, flow in and out of the South Reservoir would be equivalent, and thus did not impact the overall consumption of the City. The North Reservoir and Robinson Reservoir fill off the Capital Region Southwest Water Services Commission (CRSWSC) water transmission main, and were used to determine the water consumption for Leduc. Additional details on existing system operation are provided in Section 4.

Table 3.3. City of Leduc Average Day Demand and Peaking Factors

Average Day			Maximum Day		Peak Hour	
Demand (m^3/year)	Demand (m^3/day)	Demand (L/s)	Demand (L/s)	Peak Factor	Demand (L/s)	Peak Factor
2,730,744	7,481	86.6	141.0	1.63	222.1	2.56

As seen in Table 3.3, the total average day demand for Leduc in 2019 was 86.6 L/s with peaking factors for maximum day demand and peak hour demand of 1.63 and 2.56, respectively.

As indicated previously, the 5-day rolling average is typically used to calculate the MDD peaking factor. The 5-day rolling average peaking factor was calculated to be 1.41, which is lower than a typical MDD peaking factor. Therefore, to be conservative, a 1 day rolling average was used to calculate the MDD peaking factor of 1.63.

Tables 3.4 and 3.5 summarize the findings of the billings analysis based on land use. Based on the Leduc municipal census, the population in 2018 and 2019 was 32,448 persons and 33,032 persons, respectively. Land use areas were measured based on the provided land use map for 2019, and used for both 2018 and 2019.

Table 3.4. Water Consumption Analysis Based on Land Use 2018

Criteria	Units	Single Family Residential	Low-Density Multi-Family	High-Density Multi-Family	Commercial	Industrial	Institutional	Total
Lots	(-)	10,112	-	-	-	-	-	-
Area	(ha)	527.5	36.5	2.5	131.5	210.0	164.7	1,372.7
Water Consumption	(m^3/year)	1,524,900	252,788	31,719	372,861	270,199	249,595	2,702,062
	(L/s)	48.4	8.0	1.0	11.8	8.6	7.9	85.7
	(L/ha/day)	7,920	18,995	34,767	7,767	1,452	4,149	5,393
	(L/person/day)	126	127*	139*	-	-	-	228

** Assumed density for medium and high-density residential areas was 150 persons/ha and 250 persons/ha respectively*

Table 3.5. Water Consumption Analysis Based on Land Use 2019

Criteria	Units	Single Family Residential	Low-Density Multi-Family	High-Density Multi-Family	Commercial	Industrial	Institutional	Total
Lots	(-)	10,112	-	-	-	-	-	-
Area	(ha)	527.5	36.5	2.5	131.5	210.0	164.7	1,372.7
Water Consumption	(m^3/year)	1,491,628	247,175	31,409	354,015	264,894	224,077	2,613,198
	(L/s)	47.3	7.8	1.0	11.2	8.4	7.1	82.9
	(L/ha/day)	7,748	18,568	34,422	7,380	1,423	3,729	5,215
	(L/person/day)	124	124*	138*	-	-	-	217

** Assumed density for medium and high-density residential areas was 150 persons/ha and 250 persons/ha respectively*

As shown in Tables 3.4 and 3.5, the total water demand dropped 2.8 L/s from 2018 to 2019. This is likely due to 2019 being a very rainy summer, resulting in lower water use from residents that would normally water their lawns, or from construction projects that would normally require water from trucks.

It can also be noted that the overall demand in 2019 observed from the reservoir outflow analysis (86.6 L/s) was higher than the billings analysis (82.9 L/s). This is likely due to losses in the water distribution system such as leaks in the water distribution system, unmetered water use from hydrants, as well as rounding at the individual billing meters. It is recommended to use 86.6 L/s for the overall demand of the City, and the billing data for the approximate split of demand by land use.

In future revisions of the Master Plan, climate change may become a factor in water distribution system planning and should be considered when planning for water use. Overall water use will likely be impacted by more extreme variations in temperatures which could result in floods and droughts. During drought conditions water restrictions may be required to manage regional water demands.

3.2.2 Historical Water Demand

When comparing the overall water consumption to the 2014 Water Master Plan, the City of Leduc has seen a drop in demand since 2014 by approximately 7.4 L/s from 94 L/s to 86.6 L/s. This is not uncommon and is seen in other similar municipalities, as the general trend of water use is decreasing due to better public practice and more efficient appliances such as washing machines and fixtures.

The drop in demand, together with an increase in population, results in a lower per capita consumption rate. The 2014 Water Master Plan reported decreasing water usage, from 330 L/person/day in 2009 to 290 L/person/day in 2013 as composite rates. As indicated in Table 3.4 and 3.5, the 2018 and 2019 rates are 228 L/person/day and 217 L/person/day respectively, and consider residential areas only.

3.2.3 Future System Demand

Table 3.6 summarizes the previously used design standards for water consumption used in the 2014 Water Master Plan and/or the City of Leduc Minimum Engineering Design Standards, the values determined for the existing system by this analysis, and the recommended values to be used for future system development. The blended water consumption rate was determined similar to the 2014 Water Master Plan, which included the total water consumption for all land use types divided by the population. Since water consumption for this study was available by land use type, it has been broken down for each land use type based on the measured area and the consumption for each land use.

Table 3.6. Water Consumption Standards Comparison

Standard	Previous Value	Existing System Value	Recommended Future Value
Total Blended Water Consumption	300 L/person/day	226 L/persons/day	-
Single Lot Residential Water Consumption	300 L/person/day	226 L/persons/day	250 L/person/day
Medium Density Residential Water Consumption	-	18,570* L/ha/day	250 L/person/day
High Density Residential Water Consumption	-	34,420* L/ha/day	250 L/person/day
Commercial Water Consumption	22,500 L/ha/day	7,380 L/ha/day	22,500 L/ha/day
Industrial Water Consumption	16,875 L/ha/day	1,420 L/ha/day	11,000 L/ha/day
Institutional Water Consumption	-	3,729 L/ha/day	11,000 L/ha/day
Maximum Day Demand Peaking Factor	1.8	1.6	1.8
Peak Hour Demand Peaking Factor	3.0	2.6	3.0

** Assumed density for medium and high-density residential areas was 150 persons/ha and 250 persons/ha respectively, at 124 L/person/day*

As shown in Table 3.6, the average water use determined based on land use type is considerably lower than the previous standards used in the 2014 Water Master Plan and as seen in the City of Leduc Minimum Engineering Standards. Based on this analysis, we have recommended to lower the residential consumption rate from 300 L/person/day to 250 L/person/day, and industrial consumption rate from 16,875 L/ha/day to 11,000 L/ha/day. It is recommended to keep the commercial development standard at 22,500 m^3/ha/day because the data provided indicated a wide range of consumption for commercial users with some users over 30,000 L/ha/day in a peak month. It is recommended to use the determined values for the existing system, and the higher recommended rates for the planning of future development to allow for some flexibility in design as well as safety factor.

The above recommended design standards were reviewed with the City of Leduc prior to the publication of this Master Plan and were included in the City of Leduc Engineering Design Standards dated May 2021.

3.3 Fire Flows

The fire flow requirements for the various land uses based on the City Standards are summarized below:

- Single Family Residential: 115 L/s
- Multi-Family Residential: 227 L/s
- Commercial: 227 L/s
- Industrial: 227 L/s
- Institutional: 227 L/s

Multi-family residential units include medium and high-density residential lots such as condominiums and apartments.

3.4 Pipe Requirements

The minimum required pipe diameter for distribution mains is 200 mm, with the exception of water mains on dead end lines, which are recommended to be reduced to 150 mm after the last hydrant.

Permitted pipe materials include thermoplastic pipes (PVC and HDPE) and steel. Ductile Iron (DI) pipe is not currently permitted to be used for new pipes within the City.

3.5 Pressure Requirements

The distribution system is recommended to be operated between 350 kPa (50 psi) and 570 kPa (82 psi) under normal operating conditions. The minimum recommended pressure of 350 kPa during Maximum Day Demand is recommended such that private sprinkler systems have adequate pressure to operate in an emergency scenario.

During Peak Hour Demand, the pressures are permitted to drop to 280 kPa (40 psi). During a Maximum Day Demand plus Fire Flow scenario, the pressures are permitted to drop to 140 kPa (20 psi).

3.6 Design Standards Summary

The design criteria used for this assessment are summarized in Table 3.7.

Table 3.7. Recommended Design Standards Summary

Criteria	Unit	Previous Value	Recommended Value	Referenced Standard
Minimum Peak Hour Pressure	kPa	280	280	EPCOR
Minimum Maximum Day + Fire Pressure	kPa	140/280*	140	EPCOR
Minimum Maximum Day Pressure (Fire Sprinklers)	kPa	-	350	EPCOR
Maximum Pressure (Distribution System)	kPa	550	570	Leduc
Maximum Allowable Pressure (for Services)	kPa	550	570	Leduc
Average Day Demand (ADD) - Residential	L/c/d	300	250	Leduc
Average Day Demand (ADD) – Commercial	L/Ha/d	-	22,500	Leduc
Average Day Demand (ADD) – Industrial/Institutional	L/Ha/d	-	11,000	Leduc
Maximum Day Demand (MDD) Peaking Factor	-	2	1.8	Leduc
Peak Hour Demand (PHD) Peaking Factor	-	2.5	3.0	EPCOR
Maximum Hazen-William's Coefficient	-	130	120	EPCOR
Fire Flow - Single Family	L/s	115	115	Leduc
Fire Flow - Mid-Value Multi-Family	L/s	227	227	Leduc
Fire Flow - High Value Multi-Family	L/s	227	227	Leduc
Fire Flow - High Value Properties/Non-Res	L/s	227	227	Leduc

**Maximum day + Fire Residual Pressure previously defined as 140 kPa at the flow hydrant and 280 kPa elsewhere in the system.*

4. Water Distribution System Assessment

4.1 Existing System Description

The City of Leduc operates a water distribution system providing water to the residents and businesses within the current City of Leduc Municipal Boundary. The water distribution system is supplied by the Capital Region Southwest Water Services Commission (CRSWSC) and operates utilizing three reservoir-pumphouses that distribute water throughout the City.

The following sections provide a description of the existing water supply, storage, and distribution system within the City of Leduc.

4.1.1 Water Supply System

The City of Leduc is supplied by the CRSWSC, which distributes water purchased from EPCOR Water services in the City of Edmonton. The Commission provides water to the following municipalities:

- Edmonton International Airport
- Nisku (Leduc County)
- City of Beaumont
- The Town of Calmar
- The Village of Hay Lakes
- Camrose County
- Town of Millet

The City of Leduc is located as the third most upstream connection location and is provided water by the Boundary Station Pumphouse located along QEII Hwy, approximately 1 km south of the City of Edmonton. The Boundary Station Pumphouse provides water to the North Reservoir through a 750 mm/600 mm diameter watermain that runs from the Boundary Station Pumphouse, along the west side of QEII Hwy, crossing at approximately 69 Avenue where it is tied into the North Reservoir.

The Robinson Reservoir is also supplied by the CRSWSC. The reservoir is supplied by a 450 mm diameter watermain that branches off the 550 mm diameter CRSWSC Millet Supply Line at approximately Rollyview Road on the west edge of NW ¼ 19-49-24-4.

The CRSWSC assesses the capacity of the fill lines and upgrades accordingly to supply their various members at existing connection locations. If a new connection is required, the member municipality is responsible for the construction of the new connection.

The Corinthia Reservoir is supplied through the distribution system through a common fill/distribution line from the reservoir that connects to the distribution system.

4.1.2 Reservoirs and Pumphouses

The City's water distribution system consists of a three reservoir-pumphouse system that provides water to the City. The reservoirs are located throughout the City as follows:

- North Reservoir, located at 6609 Sparrow Drive;
- Corinthia Reservoir, located at 179 Corinthia Drive; and
- Robinson Reservoir, located at 180A Robinson Drive.

The North Reservoir acts as the main reservoir for the City. The reservoir consists of a rectangular tank with a total capacity of 14,000 m^3. The pumps within the north reservoir were upgraded in late 2019. There are two electric driven variable speed distribution pumps, and one engine driven standby pump. Table 4.1 provides a summary of pump information for the pumps recently installed at the north reservoir.

Table 4.1. North Reservoir Pump Information

		Capacity		Hydraulic Head		Horsepower	Engine Speed
Pump Name	**Serial No.**	GPM	L/s	ft	m	HP	RPM
VSP-202	100212729	2393.4	151	190.3	58	150	1780
VSP-203	100212727	2393.4	151	190.3	58	150	1780
CSD-204	100212730	3598	227	190.3	58	250	1780

The Corinthia Reservoir acts as a secondary reservoir-pumphouse which supplements flow during high demand periods, as well as operates periodically to maintain water turnover. The reservoir consists of a rectangular tank with a total capacity of 6,400 m^3. The pumphouse is equipped with two constant speed distribution pumps and one engine driven standby pump. As the Corinthia Reservoir fills off the distribution system, it cannot fill and provide flows at the same time. In June 2013, Associated Engineering conducted a facilities assessment that included testing of the pumps at the Corinthia Reservoir. Table 4.2 provides a summary of the pump information provided the Corinthia Reservoir.

Table 4.2. Corinthia Reservoir Pump Information

	Capacity		Hydraulic Head		Engine Speed
Pump Name	GPM	L/s	ft	m	RPM
P-102	727	46	143	43.6	1770
P-103	772	48.7	145	44.2	1770
P-104 (Fire Pump)	3,598	227	144	43.9	-

The Robinson Reservoir was commissioned in June 2019. The reservoir consists of a reservoir with a total capacity of 4,500 m^3. There are 5 variable frequency drive pumps located at the Robinson Reservoir, three of which act as distribution pumps; the remaining two operate as standby pumps. Table 4.3 provides a summary of the pump information for the Robinson Reservoir.

Table 4.3. Robinson Reservoir Pump Information

	Capacity		Hydraulic Head		Engine Speed
Pump Name	GPM	L/s	ft	m	RPM
VSP-301	845	53.3	138	42.0	1770
VSP-302	845	53.3	138	42.0	1770
VSP-303	845	53.3	138	42.0	1770
HFP-304	1823	115	138	42.0	1770
HFP-304	1823	115	138	42.0	1770

The total existing storage capacity for the City of Leduc is 24,900 m^3, and the total pumping capacity is 1240.5 L/s (19,662 GPM) with all pumps in operation. Based on the Alberta Environment Guidelines for Municipal Waterworks, when assessing system pumping capacity in emergency scenarios, the largest pump should be considered out of service. Therefore, the existing total pumping capacity for the City of Leduc will be considered as 1013.5 L/s (16,064 GPM) with the Fire Pump in the North reservoir (CSD-204) out of service.

4.1.3 Water Distribution System

The existing water distribution system is comprised of mainly Polyvinyl Chloride (PVC), Asbestos Cement (AC), with some Steel and High Density Polyethylene (HDPE) pipe. The watermains range in size from 100 mm in diameter to 600 mm in diameter with installation dates as early as 1958. Figure 4.1 shows the existing water distribution system.

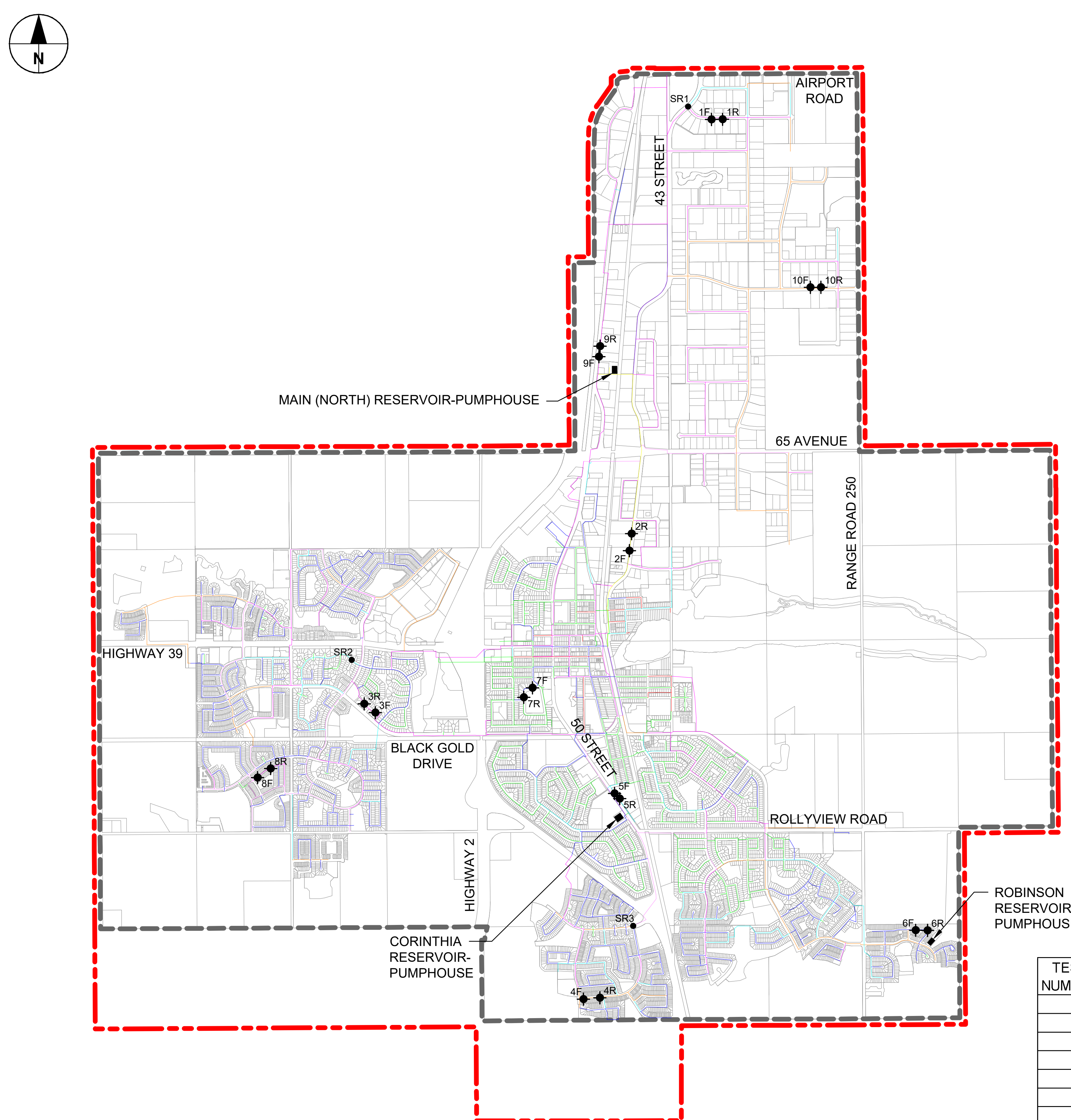
4.1.4 Operating Philosophy

The operating philosophy of the North Reservoir has been developed by The Vector Group in January 2020, and for the Robinson Reservoir by Associated Engineering in April 2016. A summary of the operating philosophy of the entire system is as follows:

- The North Reservoir operates as the main reservoir maintaining a system operating pressure of 570 kPa based on a pressure sensor located at the Civic Centre.
- At the North Reservoir, pumps start on low pressure when a VSP is running at maximum speed. A pump is stopped when a pump falls to minimum speed.
- The lead pump at the North Reservoir will always be on.
- As demand increases, dropping the pressure in the distribution system, additional pumps are turned on at the North and Corinthia Reservoirs once the active pumps reach 100% speed for a minimum of 60 seconds in the following order:
  - North Reservoir Lag Pump 1
  - Corinthia Reservoir Lead Pump
  - Corinthia Reservoir Lag Pump 1
  - North Reservoir Lag Pump 2
  - Corinthia Reservoir Fire Pump
- As additional pumps are activated, the previously active pumps are locked at 100% speed.
- As demand decreases, increasing the pressure in the system, pumps will turn off as they reach a minimum speed for three minutes in the order reverse to which were activated.
- The Robinson Reservoir operates separately from the North and Corinthia Reservoirs, maintaining a pressure setpoint as read by PIT-514 within the Robinson Pumphouse. The pressure setpoint at the Robinson Reservoir is approximately 370 kPa.

- The lead pump in operation at the Robinson Reservoir varies in speed to maintain pressure at PIT-514. If the lead pump operates at 100% speed for three minutes, the lead pump is locked, and additional lag pumps are activated.
- Once demand drops and pressure in the system exceeds the setpoint, lag pumps turn off in the reverse order in which they were active once running at the minimum speed for three minutes.

As noted in Section 4.1.2, the Corinthia Reservoir fills from the distribution system and cannot provide flows to the system when filling.



TEST NUMBER	FLOW HYDRANT	RESIDUAL HYDRANT	LOCATION	SIDE OF ROAD
1	LBP11	LBP10	82 AVE BETWEEN 42 ST & 39 ST	S
2	EI15	EI14	45 ST BETWEEN 61 AVE & 56 AVE	W
3	LE23	LE22	ALLEN PLACE	SW
4	SF91	SF92	SOUTHFORK DR BETWEEN SHEPPARD BLVD & STOUT LINK	S
5	AP04	AP03	50 ST SOUTH OF 42 AVE	NE
6	RB17	RB16	REED PLACE EAST OF ROTH STREETH	S
7	LP10	LP08	46 AVE AND 51A ST	W
8	ST15	ST21	SUNROSE LANE BETWEEN SCHUBERT ST & STRAWBERRY LANE	S
9	NI08	NI09	SPARROW DRIVE AT APPX 70 AVE	E
10	LBP155	LBP156	ALLARD AVE BETWEEN 36 ST & 33 ST	N

AECOM
Figure: 4.1

EXISTING WATER DISTRIBUTION SYSTEM

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved: ___
Checked: ___
Designer: ___
Project Management Initials: ___
Last saved by: ANDREW.FLECK(2021-06-24) Last Plotted: 2021-06-24
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0401 - EXISTING WATER DISTRIBUTION SYSTEM.DWG

4.2 Existing System Assessment

4.2.1 Existing Robinson Supply Capacity Assessment

As noted previously, the Robinson Reservoir is serviced by the CRSWSC. The 450 mm HDPE diameter supply line extends from the Robinson Reservoir and follows the east edge of ¼ Section NW 19-49-24 W4 to the north, tying into the 550 mm diameter CRSWSC Millet water main at Rollyview Road.

The supply line from the connection point to the Robinson Reservoir has the following physical information:

- A length of 1,200 m
- Elevation at the Millet water main tie in of approximately 736.25 m
- Elevation at the Robinson Reservoir supply point of approximately 744.75 m
- 450 mm diameter HDPE DR11 pipe (369 mm inside diameter)

As per the CRSWSC Master Plan Update (January 2020), a minimum pressure of 140 kPa will be maintained through the CRSWSC water distribution system, and it is assumed that the minimum pressure will be maintained at the Robinson Reservoir inlet.

The design standard for services from the CRSWSC indicates a maximum velocity of 1.5 m/s. As the overall demand of the City increases, the velocity of water within the service will increase. The velocity of water will reach 1.5 m/s when the fill rate reaches 160 L/s based on an inside diameter (ID) of the 450 mm diameter HDPE DR 11 service of 369 mm.

The existing average day demand from the Robinson Reservoir to the City's water distribution system in 2019 (during the months that it was in operation) was 19.5 L/s. It is estimated that the Robinson service will reach a maximum day demand fill rate of 160 L/s in 2045.

Bentley FlowMaster was utilized to estimate the required pressure at the Millet tie in location while maintaining 140 kPa at the Robinson Reservoir. At 1.5 m/s, assuming a Hazen Williams coefficient of 120, approximately 300 kPa should be maintained at the tie in location on the Millet water main. Note that this was a simplistic hydraulic analysis and additional losses will likely occur due to valves and bends along the service line. Based on the CRSWSC 2020 Master Plan, the pressure at the Robinson Reservoir tie in is approximately 640 kPa.

4.2.2 Existing Storage and Pumphouse Capacity Assessment

The North Reservoir, Corinthia Reservoir, and Robinson Reservoir have storage capacities of 14,000 m^3, 6,400 m^3, and 4,500 m^3, respectively. The total existing storage capacity within the City is 24,900 m^3.

The governing storage requirement within the City is based on providing storage for a maximum day demand plus fire storage. The required duration for fire storage is based on the Fire Underwriters Survey Water Supply for Public Fire Protection; a 3-hour storage duration is recommended for the fire flow requirement of 227 L/s. Table 4.4 summarizes the storage requirement for the existing development condition.

Table 4.4. Existing System Reservoir Storage Requirement

Description	Details/Description	Volume (m^3)
Fire Storage	227 L/s for 3 hours	2,452
Emergency Storage	Storage for Max Day (MDD=136 L/s)	11,761
Total Storage Requirement – Existing System	-	**14,212**
Storage Capacity less Storage Requirement	Surplus Existing Storage	10,668

Based on Table 4.4, the City has sufficient storage for the existing development condition.

Table 4.5 summarizes the pumping capacities at each reservoir.

Table 4.5. Existing System Pumping Capacity Summary

North Reservoir			Corinthia Reservoir			Robinson Reservoir		
Name	**Flow**	**Head**	**Name**	**Flow**	**Head**	**Name**	**Flow**	**Head**
VSP-202	151	58	P-102	48.7	44.2	VSP-301	53.3	42
VSP-203	151	58	P-103	45.9	43.6	VSP-302	53.3	42
CSD-204	227	58	P-104	227	43.89	VSP-303	53.3	42
						HFP-304	115	42
						HFP-305	115	42

Based on the information above the total pumping capacity is 1240.5 L/s (19,662 GPM) with all pumps in operation. Based on the Alberta Environment Guidelines for Municipal Waterworks, when assessing system pumping capacity in emergency scenarios, the largest pump should be considered out of service. Therefore, the existing total pumping capacity for the City of Leduc will be considered as 1013.5 L/s (16,064 GPM) with the Fire Pump in the North reservoir (CSD-204) out of service.

The governing demand scenario for the existing system is maximum day demand plus fire, requiring a pumping capacity of 363 L/s. The existing pump capacity is sufficient for the existing development.

4.2.3 Model Development

The water distribution system for the City of Leduc was modelled using WaterCAD Version 10 Connect Edition, developed by Bentley Systems Ltd. WaterCAD is an industry accepted hydraulic modelling software for water distribution systems using both steady state and extended period simulations.

The Leduc system was analyzed using steady state simulations. The City's water distribution system is represented utilizing a series of pipes and junction nodes that require various physical input properties including:

- Pipe diameter, length, material, and roughness coefficients
- Node elevation and spatial representation
- Reservoir elevation
- Pump elevation and capacity curves
- PRV set points

The model was originally developed by Associated Engineering for the 2014 City of Leduc Water Master Plan. At the time, data was not available to complete a complete model calibration and thus, as a part of this study, the model was updated and calibrated as discussed in Section 4.2.4 in order to validate the hydraulic modelling results.

WaterCAD functions by inputting the physical data of the system as well as water demand of the water distribution system, then calculates the resulting pressures, pipe flow and velocity, available fire flow at notes, and many other items that are used for hydraulic analysis. Therefore, based on the overall system demands described in Section 3.3, catchment areas were delineated, and the overall system demand was distributed throughout the water distribution system to spatially represent the demand for each neighborhood within the City.

4.2.4 Model Calibration

The WaterCAD model was calibrated using hydrant testing data. Hydrant testing was conducted on June 8, 2020 by SFE Global. Ten hydrants tests were conducted by opening each hydrant and measuring the residual pressure flowing from the hydrant. Based on the pressure measured at the hydrant, the flow rate was calculated, as well as the flow rate if the system residual pressure was drawn down to 140 kPa (20 psi). The static pressure was measured before the test, and the active pumps including the outgoing flow rate and pressure was recorded at the reservoir so that the field testing conditions could be replicated within the hydraulic model. In addition, three additional hydrants were equipped with pressure monitors to monitor the system pressure during the tests at various locations throughout the system. The testing locations and additional pressure monitor locations are shown on Figure 4.1. The test locations are summarized in Table 4.6.

Table 4.6. Hydrant Testing Locations

Test Number	Flow Hydrant	Residual Hydrant	Location	Side of Road
1	LBP11	LBP10	82 Avenue between 42 Street & 39 Street	S
2	EL15	EL14	45 Street between 61 Avenue & 56 Avenue	W
3	LE23	LE22	Allen Place	SW
4	SF91	SF92	Southfork Drive Between Sheppard Blvd & Stout Link	S
5	AP04	AP03	50 Street South of 42 Avenue	NE
6	RB17	RB16	Reed Place East of Roth Street	S
7	LP10	LP08	46 Avenue and 51a Street	W
8	ST15	ST21	Sunrose Lane between Schubert Street & Strawberry Lane	S
9	NI08	NI09	Sparrow Drive at approximately 70 Avenue	E
10	LBP155	LBP156	Allard Avenue between 36 Street & 33 Street	N

During testing, the Corinthia Reservoir was not in operation. One pump at the North Reservoir and one pump at the Robinson Reservoir were in operation prior to the testing and one additional pump at the Robinson Reservoir was activated during the test. A summary of the hydrant testing results is provided in Tables 4.7 to 4.9 detailing the results at the test location, system hydrants, and the reservoirs, respectively. The hydrant testing summary report developed by SFE Global is provided in Appendix A. Note that System Hydrant 2 lost connection during Tests 3 through 7 and was not able to provide data during those tests. The average flow was monitored the day of the testing to determine the average system demand. The average flow monitored the day of the testing was 107 L/s.

Table 4.7. Hydrant Testing Results – Test Locations

Test Number	Test Time	Hydrant Flow Rate	Hydrant Pressure	Residual Pressure	Flow at 140 kPa
(-)	(-)	(L/s)	(kPa)	(kPa)	(L/s)
1	11:02	117	103	586	242
2	11:44	131	131	496	277
3	13:49	130	128	469	198

Test Number	Test Time	Hydrant Flow Rate	Hydrant Pressure	Residual Pressure	Flow at 140 kPa
4	15:09	119	107	427	198
5	14:36	117	103	462	210
6	15:36	95	69	317	129
7	13:28	135	138	469	224
8	14:08	138	145	490	198
9	16:06	141	152	496	222
10	11:22	131	131	538	250

Table 4.8. Hydrant Testing Results – System Hydrants

Test Number	SR #1 - Pressure	SR #2 - Pressure	SR #3 - Pressure
1	674	632	523
2	681	614	509
3	671	-	525
4	632	-	500
5	653	-	541
6	628	-	487
7	658	-	507
8	683	-	526
9	680	650	521
10	692	649	537

Table 4.9. Hydrant Testing Results – Reservoir Data

Test Number	North Reservoir		Robinson Reservoir		Total Outgoing Flow (L/s)
	Outgoing Flow (L/s)	Outgoing Pressure (kPa)	Outgoing Flow (L/s)	Outgoing Pressure (kPa)	
1	147	578	102	367	249
2	141	590	104	373	245
3	138	590	102	369	240
4	131	589	98	369	229
5	131	588	96	369	227
6	79	587	114	370	193
7	128	575	82	363	210
8	141	588	107	369	248
9	143	584	106	369	249
10	141	585	103	369	245

Based on the hydrant testing results, the model was evaluated based on various pipe roughness coefficients (Hazen Williams ‘C’ Coefficients’) to determine the appropriate roughness coefficient to represent the system. The calibration was conducted by replicating the test conditions within the model, utilizing the average demand the day of the testing (107 L/s) and placing an additional demand at the flow hydrant. The modelled pressure was compared at the flow, residual, and additional system hydrants to the monitored data. Tables 4.10 and 4.11 show the model calibration results based on roughness coefficients of 120, and 130, respectively.

Table 4.10. Model Calibration Results – C = 120

Test No	Residual Hydrant		System Hydrant 1		System Hydrant 2		System Hydrant 3	
	Modelled Pressure	Percent Difference	Modelled Pressure	Percent Difference	Modelled Pressure	Percent Difference	Modelled Pressure	Percent Difference
(-)	(kPa)	(%)	(kPa)	(%)	(kPa)	(%)	(kPa)	(%)
1	565	-3.6	564	-16.32	550	-12.97	446	-14.72
2	489	-1.5	582	-14.54	542	-11.73	439	-13.75
3	460	-1.89	586	-12.67	491	-	438	-16.57
4	393	-8.07	594	-6.01	550	-	426	-14.8
5	454	-1.72	595	-8.88	550	-	443	-18.11
6	355	11.93	619	-1.43	576	-	469	-3.7
7	439	-6.37	583	-11.4	532	-	436	-14
8	497	1.52	581	-14.93	478	-	434	-17.49
9	525	5.75	572	-15.88	538	-17.23	434	-16.7
10	499	-7.22	573	-17.2	543	-16.33	439	-18.25
Average Difference	-1.1		-11.9		-14.6		-14.8	

Table 4.11. Model Calibration Results – C = 130

Test No	Residual Hydrant		System Hydrant 1		System Hydrant 2		System Hydrant 3	
	Modelled Pressure	Percent Difference	Modelled Pressure	Percent Difference	Modelled Pressure	Percent Difference	Modelled Pressure	Percent Difference
(-)	(kPa)	(%)	(kPa)	(%)	(kPa)	(%)	(kPa)	(%)
1	572	-2.4	570	-15.4	553	-12.5	448	-14.3
2	493	-0.7	586	-14.0	546	-11.1	442	-13.2
3	472	0.7	588	-12.4	502	-	442	-15.8
4	401	-6.2	596	-5.7	553	-	431	-13.8
5	459	-0.6	596	-8.7	554	-	446	-17.6
6	359	13.2	618	-1.6	576	-	469	-3.7
7	453	-3.4	586	-10.9	538	-	440	-13.2
8	512	4.6	584	-14.5	491	-	438	-16.7
9	532	7.2	576	-15.3	542	-16.6	438	-15.9
10	505	-6.1	577	-16.6	547	-15.7	443	-17.5
Average Difference	-0.6		-11.5		-14.0		-14.2	

The model calibration was also evaluated utilizing the calculated flow during the hydrant test if the system was drawn down to 140 kPa (20 psi) at the flow hydrant. Table 4.12 summarizes the comparison of the modelled flow vs the calculated flow from the hydrant testing data, for both C values.

Table 4.12. Model Calibration Results – Available Flow at 140 kPa

Test No.	Calculated Flow at 140 kPa	C = 120		C = 130	
		Simulated Flow at 140 kPa	Percent Difference	Simulated Flow at 140 kPa	Percent Difference
(-)	(L/s)	(L/s)	(%)	(L/s)	(%)
1	307	277	-9.8	284	-7.4
2	352	293	-16.8	296	-15.9
3	251	243	-3.2	251	-0.1
4	251	225	-10.4	231	-7.9
5	266	257	-3.4	263	-1.3
6	160	207	29.4	212	32.6
7	285	230	-19.3	238	-16.4
8	257	250	-2.7	259	0.9
9	282	286	1.4	292	3.5
10	318	275	-13.5	281	-11.7
Average Difference	-	-4.8		-2.4	

Based on the above information, the model is effectively representing the drawdown of the system at the flow hydrant; however, the pressures measured at the system hydrants in general are lower than what is modelled. This is likely caused by the pressure measured at the exact time of the test. It is possible that the system takes some additional time to respond to the test. It was also found that pressure at the system hydrants ranged significantly during the test and the pressure would drop from the static pressure when the hydrant began flowing, and then would slowly increase as the test occurred once the additional pump was turned on, and then would spike in pressure when the hydrant was turned off until the additional pump turned off.

Based on the above calibration, a C value of 130 for existing system pipes will be used for the remainder of the analysis. Future system pipes including proposed upgraded pipes within the existing system will be modelled using a C value of 120 based on design standards.

4.2.5 System Evaluation for Existing Development Conditions

A hydraulic analysis was conducted for the City of Leduc water distribution system for the following demand conditions:

- Average Day Demand (ADD)
- Maximum Day Demand Plus Fire Flow (MDD+Fire)
- Peak Hour Demand (PHD)

The existing system was assessed with a pressure setpoint at the reservoirs at a hydraulic grade line elevation of 785 m. As demand increases with each demand scenario, additional pumps were turned on to represent the system response. The active pumps for each scenario were as follows:

- ADD: One distribution pump at the north reservoir (VSP-202)
- MDD+Fire: One fire pump at both the North and Robinson Reservoirs (CSD-204 and HFP-304)
- PHD: One distribution pump at the North reservoir and two distribution pumps at the Robinson Reservoir (VSP-202, VSP-301 and VSP-303).

As discussed in Section 4.2.2, the existing pumping capacity greatly exceeds the pumping requirement. However, one of the largest pumps should remain inactive during the analysis to simulate an emergency scenario. During the simulations, the fire pump at the Corinthia Reservoir (among additional pumps) remained inactive.

Tables 4.13 and 4.14 summarize the existing system hydraulic assessment for each demand scenario, and the results are shown schematically on Figures 4.2 through 4.4. The ADD and PHD scenarios were assessed to determine the pressure and water velocity throughout the system. The MDD+Fire scenario was assessed to determine the available fire flow throughout the system. Note that, as per the design standards in Section 3.7, the minimum pressure during peak hour demand is 280 kPa, and a minimum pressure for average day demand is not specified. For the purposes of this assessment, the minimum allowable PHD pressure was used for the ADD assessment.

Table 4.13. Existing System Evaluation – ADD and PHD

Scenario	Total Number of Nodes	Minimum Pressure	Maximum Pressure	Nodes with High Pressure		Nodes with Low Pressure	
(-)	(No.)	(kPa)	(kPa)	(No.)	(%)	(No.)	(%)
ADD	1149	340	674	353	30.7	0	0
PHD		343	649	241	21.0	0	0

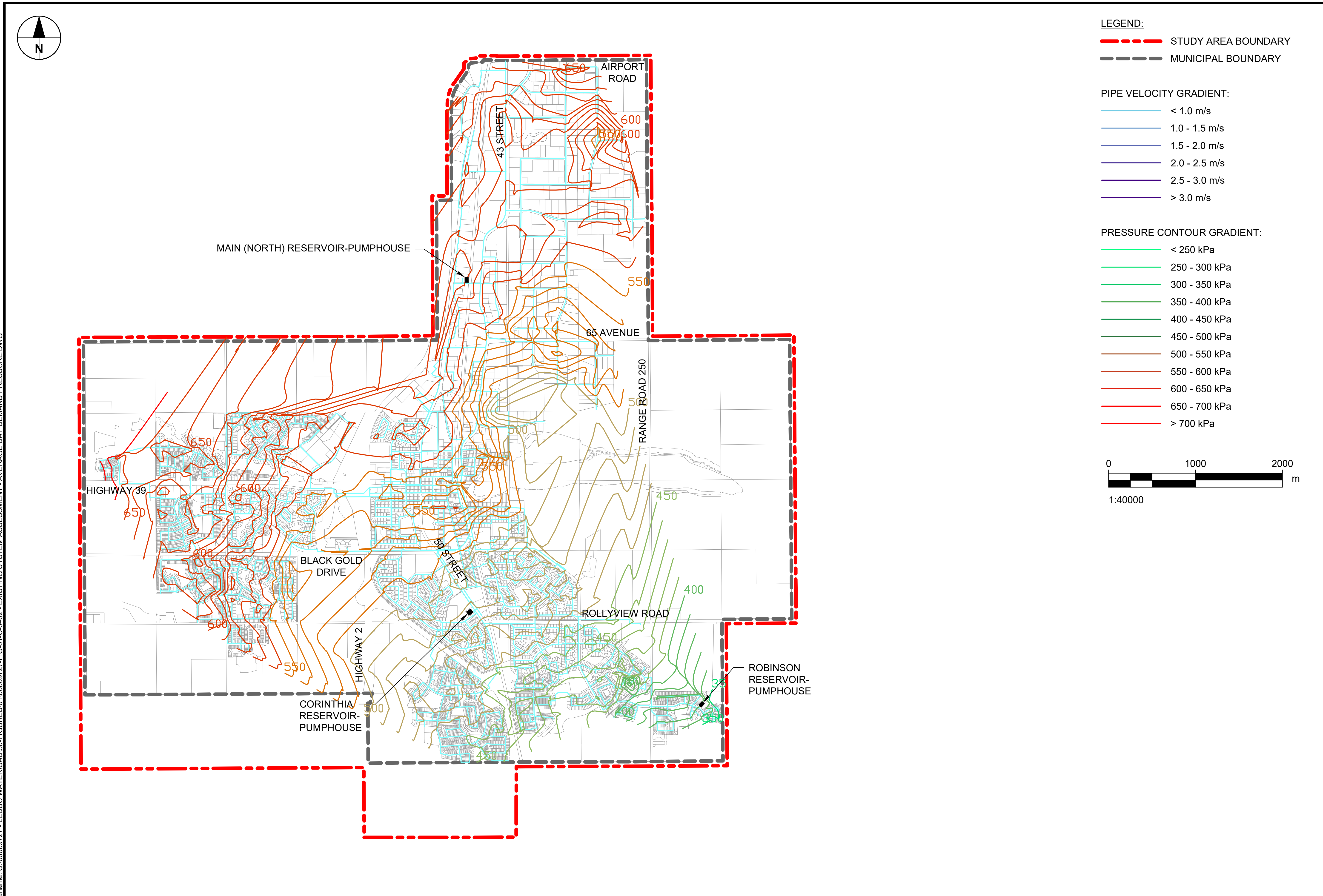
Table 4.14. Existing System Evaluation – MDD+Fire

Total Number of Nodes	Residential Nodes	Non-Residential Nodes	Residential Nodes Failing Fire Flow Requirements		Non-Residential Nodes Failing Fire Flow Requirements		Total Nodes Failing Fire Flow Requirements	
(No.)	(No.)	(kPa)	(No.)	(%)	(No.)	(%)	(No.)	(%)
1149	805	344	21	2.6	19	5.5	40	3.5

As seen in Table 4.13, based on the modelling results, low pressure is not a concern for the existing water distribution system; however, up to 30% of the system exceeds the recommended maximum pressure. In general, the areas that have high pressure are located in the NW corner of the neighborhoods west of QEII Hwy, and areas north of the North Reservoir in the industrial area.

During both the ADD and PHD scenarios, water velocity remains below 1.0 m/s in all pipes, and thus water velocity is not a concern for the existing system. The maximum velocity within the system during the ADD scenario is the north QEII crossing, with a velocity of 0.21 m/s. During the PHD scenario, the maximum velocity is the segment of 350 mm diameter pipe from the Robinson Reservoir until Gaetz Road. The velocity for this segment of pipe is approximately 0.71 m/s.

During the MDD+Fire scenario, a total of 40 nodes fail the fire flow requirement. As seen on Figure 4.3, the majority of the failing nodes are located within cul-de-sacs at locations with long dead ends. It is understood the Fire Department generally utilizes hydrants on main streets rather than cul-de-sacs. It is recommended that if there are other works occurring in the area, the benefit of up-sizing the pipes should be assessed on a case-by-case basis. An area of concern with respect to fire flow is the South Telford residential area/East Elementary School, as well as the non-residential areas just north of Black Gold Drive. System deficiencies and the proposed improvements are further discussed in Section 4.3.1 and Section 4.3.2, respectively.



AECOM

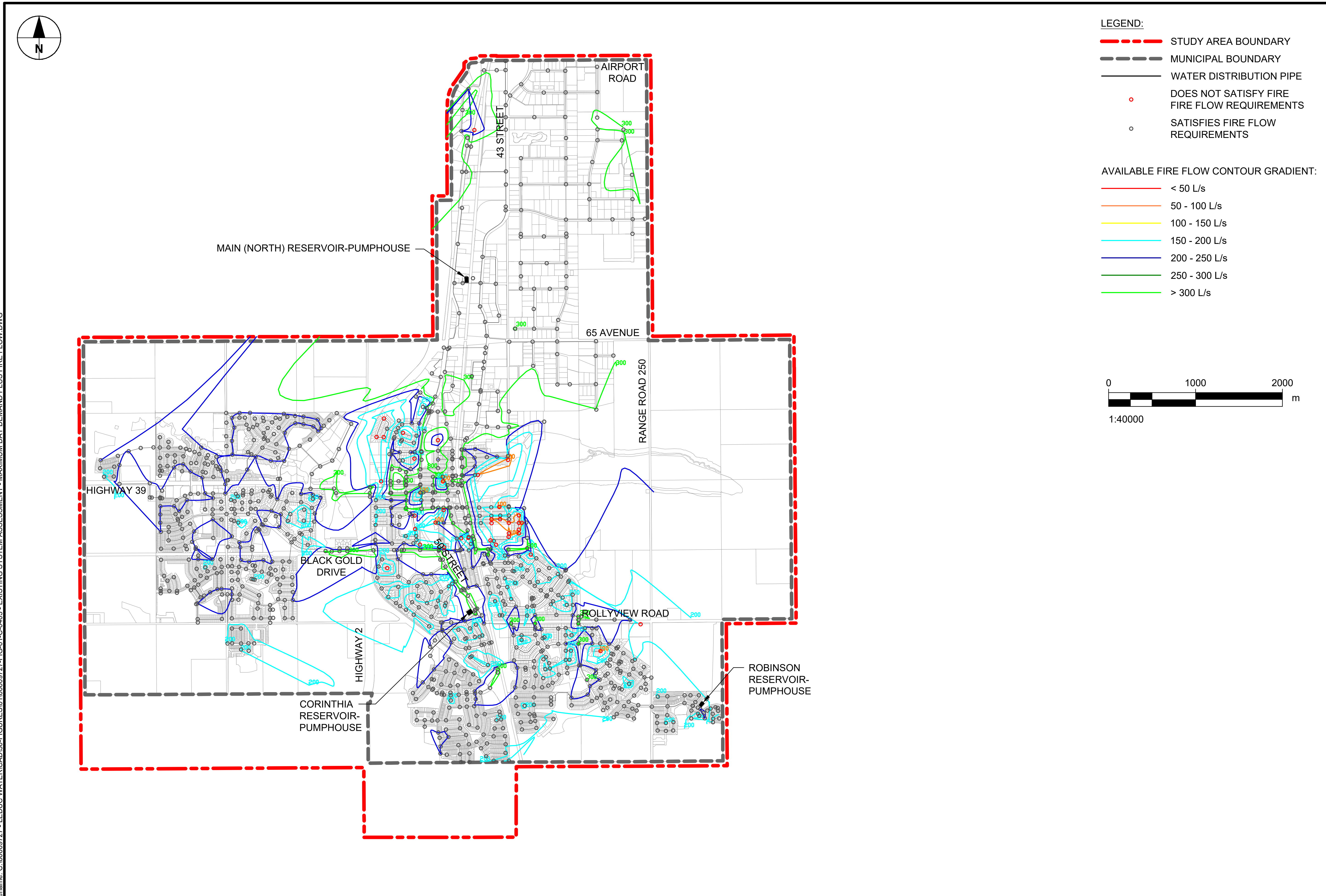
Figure: 4.2

EXISTING SYSTEM ASSESSMENT
AVERAGE DAY DEMAND

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved:
Checked:
Designer:
Project Management Initials:
Last saved by: ANDREW.FLECK(2021-06-24) Last Plotted: 2021-06-24
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0402 - EXISTING SYSTEM ASSESSMENT - AVERAGE DAY DEMAND PRESSURE.DWG



AECOM

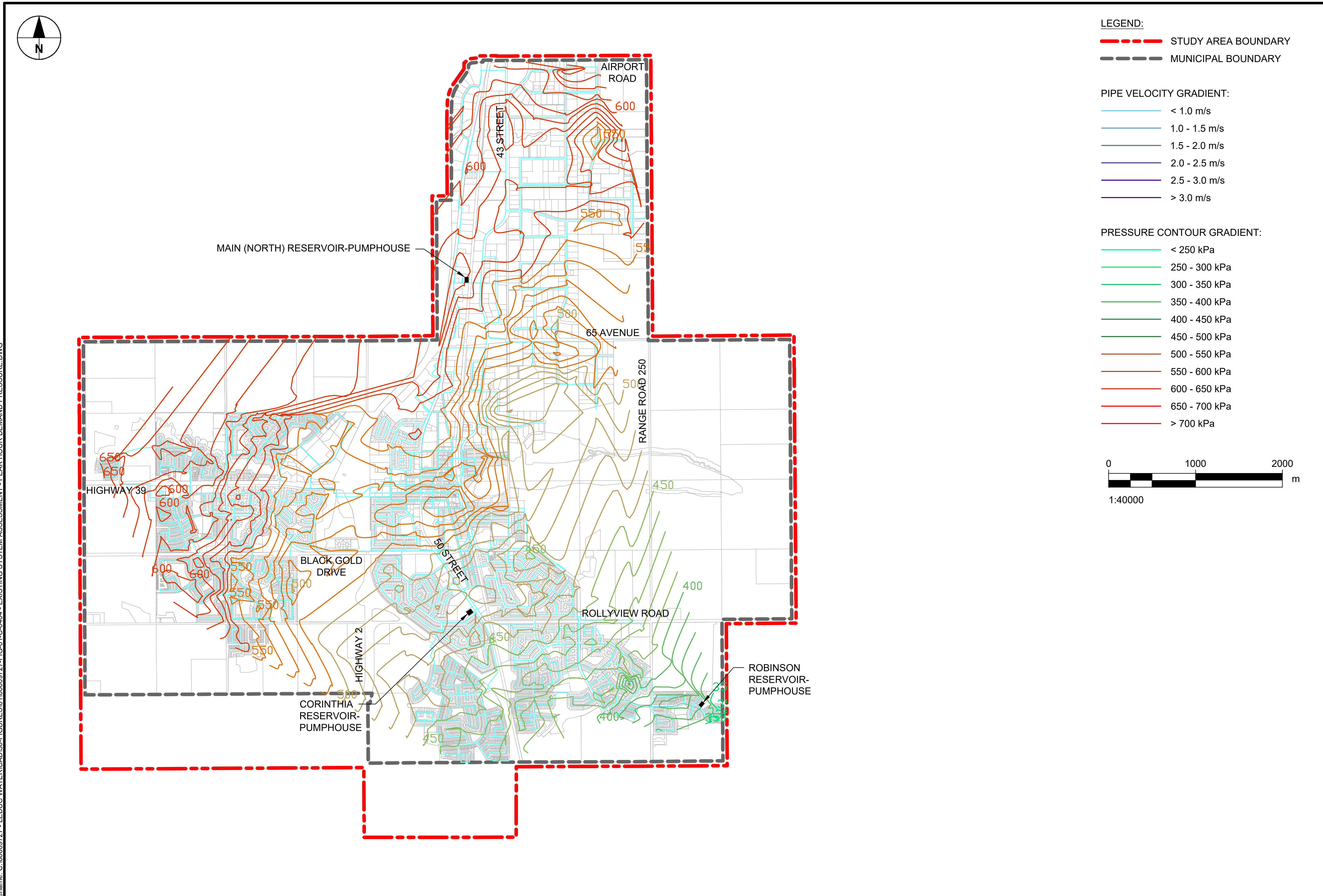
Figure: 4.3

EXISTING SYSTEM ASSESSMENT
MAXIMUM DAY DEMAND PLUS FIRE FLOW

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved: ____ Checked: ____ Designer: ____ Project Management Initials: ____
Last saved by: ANDREW.FLECK(2021-07-09) Last Plotted: 2021-07-09
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0403 - EXISTING SYSTEM ASSESSMENT - MAXIMUM DAY DEMAND PLUS FIRE FLOW.DWG



AECOM

Figure: 4.4

EXISTING SYSTEM ASSESSMENT
PEAK HOUR DEMAND

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved:
Checked:
Designer:
Project Management Initials:
Last saved by: ANDREW.FLECK(2021-06-24) Last Plotted: 2021-06-24
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0404 - EXISTING SYSTEM ASSESSMENT - PEAK HOUR DEMAND PRESSURE.DWG

4.3 Existing System – Emergency Preparedness

Disaster planning is a key consideration to determine how the system would respond if an individual high value asset was temporarily lost. The main areas of susceptibility to wide scale impact in an emergency would be the loss of a reservoir, highway crossing, or water supply from the CRSWSC.

Therefore, the following sections provide an assessment of the performance of the City's water distribution system for the following potential emergency scenarios:

- Single Reservoir Operation Assessment; determining system pressures and the time to empty during a time in which the City would need to operate a single reservoir.
- Single QEII Highway Crossing Assessment; determining system pressures in the neighborhoods located west of the QEII should a crossing be out of service.
- Supply to and from the EIA or Leduc County should either system be required to supply the other.

The City has taken significant steps as a municipality to increase the redundancy of their system. Historically, the City had a single connection from the CRSWSC. If it were to go down, the City would only have a number of hours to operate before storage would run out for both domestic use and fire protection. This was improved by the construction of the Robinson Reservoir where there is now a second feed to the City, and as a result if one connection was lost to one of the two primary reservoirs there is a backup.

Currently, the City has two QEII crossings that provide all the water to the residents located west of the QEII. If one of the crossings were to go down, the City would be relying on a single crossing. Two additional QEII crossings are planned in Stage 2 and Stage 3.

The following sections summarize the system performance during emergency scenarios as well as the potential connections and servicing options with the EIA and Leduc County. The intent is to understand the benefit of the steps that the City has taken to handle emergency scenarios and the feasibility of additional steps that could be taken.

4.3.1 Scenario 1 – Single Reservoir Operation Assessment

The existing system was assessed to determine the performance of the system if only a single reservoir was in operation. The system was assessed during the average day and peak hour demand scenario to determine the pressure within the system.

In general, each reservoir has sufficient pumping capacity to provide flows to the entire water distribution system.

When the North Reservoir is operating alone, it is able to service the City during PHD with only the two distribution pumps in operation and the fire pump on standby. The pressure in the system during this emergency scenario is very similar to the existing system evaluation, where pressures remain high west of QEII, as well as the industrial development area. During ADD, one distribution pump is sufficient to supply the City.

When the Corinthia Reservoir operates alone, it requires all three pumps including the fire pump to operate to provide sufficient pressures during PHD; however, is sufficient to service Leduc during all demand scenarios. The two distribution pumps operating without the fire pump are insufficient to provide sufficient flow to the existing system during PHD, however, are sufficient to supply the City during ADD.

When the Robinson Reservoir operates alone, during PHD the pressure in the southeast corner of Leduc drops to approximately 300-310 kPa compared to normal operation at 350 kPa. The pressure drop is due to increased velocities in the 350 mm diameter watermain from the reservoir to Gaetz Drive that has to supply water to the entire City, causing a drop in hydraulic grade line. The velocity in this watermain reaches 2.2 m/s during PHD and thus meets current requirements. However, due to the high elevation of the Robinson Reservoir, the more northern portions of the City remain at appropriate pressures (<570 kPa) due to the drop in elevation in the north end of Leduc.

The Corinthia Reservoir is not a viable reservoir to service the entire City despite the adequate pumping capacity. This is due to the limited storage capacity and the dependence of other reservoirs for water supply.

Table 4.15 shows the amount of time in which each reservoir will be able to provide the City with water while operating alone assuming that they cannot fill. Note that the North and Robinson Reservoir can fill at the maximum day demand rate from the CRSWSC and thus can technically run indefinitely without running out of storage. The time was calculated based on the maximum day demand rate (136 L/s) compared to the storage capacity of each reservoir less fire storage.

Table 4.15. Single Reservoir Operation Storage Capacity Assessment

Reservoir	Storage Capacity	Storage Capacity Less Fire Storage	Time to Empty (Without Fill)	Time to Empty (While Filling)
	(m^3)	(m^3)	(Hours)	(Hours)
North	14,000	11,548	23.6	Unlimited
Corinthia	6,400	3,948	8.1	n/a*
Robinson	4,500	2,048	4.2	Unlimited

*Corinthia Reservoir cannot supply while filling

Based on the above information, in the existing development condition both the North and Robinson have sufficient pumping capacity to supply water to the entire system as well as can operate indefinitely while filling at the maximum day demand rate. The Corinthia Reservoir cannot supply the entire system despite having adequate pumping capacity.

4.3.2 Scenario 2 – QEII Crossing Closure Assessment

The existing system was assessed to determine the impact to the system west of QEII Hwy if one of the highway crossings is out of operation for maintenance or during an emergency scenario. This condition was assessed during the average day and peak hour demand scenario to determine the impact to the system pressures west of QEII.

Table 4.16 summarizes the system performance comparing the existing system with both crossings in operation, versus if either crossing is closed. The maximum pressure was recorded within the Woodbend neighbourhood, as this location is at the lowest elevation and generally experiences the highest pressure, and the minimum pressure was recorded on the east edge of the Windrose neighbourhood as this area has the highest elevation and experiences the lowest pressures in the area.

Table 4.16. Single QEII Crossing Assessment

Scenario	Demand	Maximum Pressure (kPa)	Minimum Pressure (kPa)	North Pipe Velocity (m/s)	North Pipe Headloss (m/km)	South Pipe Velocity (m/s)	South Pipe Headloss (m/km)
Existing	ADD	674	519	0.21	0.21	0.14	0.10
	PHD	649	495	0.51	1.09	0.38	0.65
North Closed	ADD	667	513	-	-	0.35	0.54
	PHD	613	460	-	-	0.89	3.08
South Closed	ADD	670	516	0.35	0.54	-	-
	PHD	629	475	0.89	3.08	-	-

As shown in Table 4.16, during the average day scenario, the overall pressure within the system west of QEII is negligibly impacted with less than a 5 kPa drop in pressure throughout the system. During peak hour demand, the headloss across a single highway crossing increases to 3 m/km. This results in a pressure drop in the system of 30-40 kPa during peak hour demand; however, the minimum pressure within the system remains above the minimum pressure requirement of 280 kPa.

This analysis indicates that if one crossing is out of operation, a single crossing is sufficient to provide water to the areas west of QEII in the existing development condition. As demand increases through further development of the City, west of QEII, this will correspondingly increase the headloss that will occur in a single crossing. For this purpose, additional connections are proposed in future development conditions, the first of which is proposed in the 20-30 year development condition. The additional QEII crossings will provide redundancy and a factor of safety should one crossing not be in operation.

4.3.3 Scenario 3 – EIA Servicing and Supply Viability Assessment

There is the opportunity to provide a connection between the City of Leduc and the Edmonton International Airport (EIA) water distribution systems. This connection would be utilized to supplement water to either the City or to the EIA should an emergency situation occur causing one party to not be able to supply water to its own network from any of its own reservoirs.

The existing City of Leduc and EIA water distribution systems are shown on Figure 4.5. The following includes a summary of the EIAs existing water distribution system information collected to determine the feasibility of a potential connection between systems:

- The EIA operates 2 reservoir pumphouses, both have a fill connection to the CRSWSC.
- EIA Reservoir 1 (EIAr1) and EIA Reservoir 2 (EIAr2) have an accessible storage capacity of 3,600 m^3, and 4,200 m^3, respectively, for a total storage capacity of 7,800 m^3.
- The average day demand rate for the EIA from 2018-2020 were 12 L/s, 17 L/s, and 11.5 L/s, respectively. For the purposes of this assessment, the 2019 rate was used because the 2020 rate was lower due to the COVID-19 Pandemic.
- The EIAr2 is equipped with three distribution pumps that operate at 42 L/s, and two high capacity pumps that operate at 168 L/s.
- The elevation of the pumps at EIAr2 is approximately 721.25 m.
- EIAr2 operates at two operating pressures. Normally, it operates at 50 m of discharge head (490 kPa, 71 psi). When required, the pumps can operate at 70 m of discharge head (690 kPa, 100 psi)
- Therefore, the set point hydraulic grade line of the EIAr2 at the low and high operating points are 772 m and 792 m, respectively. The hydraulic grade line setpoint for the City is 785 m.

There are two connection alternatives proposed between the City and the EIA, which are shown on Figure 4.5:

- Connection alternative 1 includes a connection to the existing 400 mm diameter water main just south of EIAr2. The proposed connection will follow existing waterlines east and will cross Highway 2 connecting to the City's water distribution system at Sparrow Drive at approximately 75 Avenue. The alignment follows along the south edge of the EIA Outlet Mall Option Lands to avoid a future conflict. Alternative 1 has an estimated cost of $2.3 million.
- Connection alternative 2 includes a connection to the EIAs water distribution system at the existing 400 mm diameter water main adjacent to the STARS hangar. The proposed alignment will continue southeast then will cross QEII Hwy connecting to the City's system at Sparrow Drive and approximately 65 Avenue. Alternative 2 has an estimated cost of $2.5 million.

The assessment was conducted for the existing development condition during the average day demand scenario to determine if, with a single connection, both systems could supply flows to each other. It is preliminarily recommended to install a 450 mm diameter watermain connecting the systems to minimize headloss when the EIA is supplying the City. However, if a connection is to be implemented, future development and potential for additional connections should be assessed to determine a final pipe sizing.

When at the lower operating pressure (772 m), with the three distribution pumps and one high capacity pump at the reservoir active, the EIA is sufficient to provide flows to the entirety of the City during the existing average day demand scenario. However, the pressure in the southeast portion of the City (Robinson neighborhood) drops to approximately 200 kPa. When operating at the higher operating pressure (792 m) EIAr2 is capable of supplying the entire City without pressures dropping below the minimum recommended value. The minimum pressure in the City's system remains at approximately 400 kPa. The higher operating pressure results in areas of the City in the northern industrial area and west of QEII exceeding the maximum pressures. The pressure in these areas will reach up to approximately 725 kPa (105 psi).

When the City is supplying flows to the EIA, the hydraulic grade line at the connection points during ADD is approximately 785 m. This would result in the pressure at EIAr2 to be approximately 640 kPa (92.5 psi). It is understood that the ground elevation of the EIA is relatively constant at approximately 720 m. The City has sufficient pumping capacity to supply the EIA in all demand scenarios.

The storage capacity of the systems was assessed to determine the amount of time each system could supply the connected systems. For this assessment, it was assumed that the fire storage would be maintained. The Alberta Environment Guidelines for Municipal Waterworks Systems indicate that the level of fire protection is the responsibility of the municipality. The Fire Underwriters Water Supply for Public Fire Protection 1999 report emphasizes the importance of an adequate and reliable water system, with adequacy defined at the ability to deliver the necessary fire flow through the distribution system, and reliability defined as the ability of the system to provide necessary fire flow during certain emergency or unusual conditions. Complete loss of potable water within a jurisdiction is an outstanding emergency scenario, and should the supply not be able to be reinstated within the timeframes specified in Table 4.17, it is recommended that the use of fire storage be reviewed and considered. As the fire storage makes up approximately 10% of the total available storage, the drawdown times without fill could be extended by approximately 10%.

Table 4.17 summarizes the storage and times at which each system can supply at average day demand with and without filling. The fill rate is assumed to be 1.8 times the average day demand of the system supplying. The total existing ADD for the system would be 100.5 L/s.

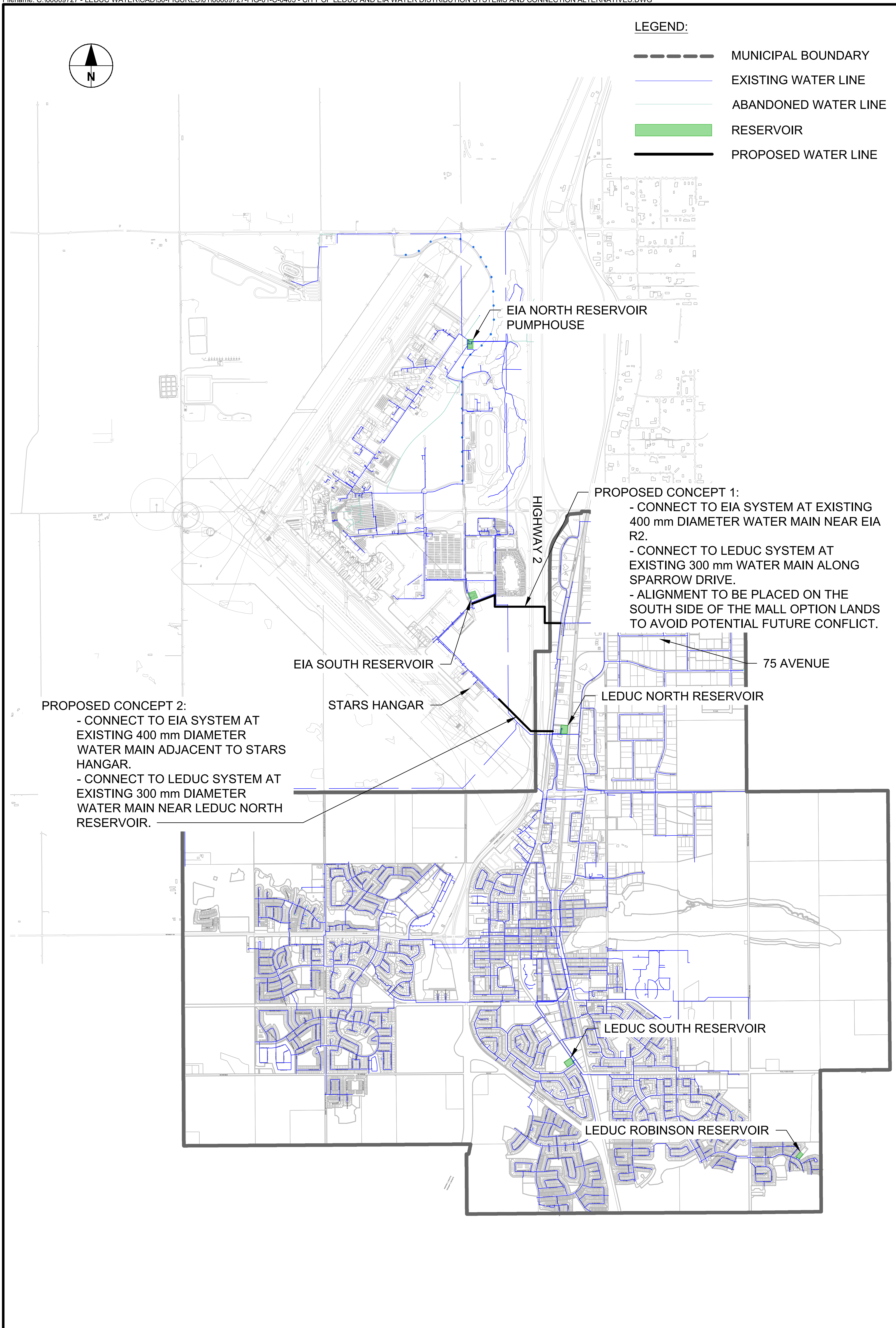
Table 4.17. EIA Servicing Assessment

	Leduc Supply	EIA Supply
Total Storage	24,900 m^3	7,800 m^3
Fire Storage	2,452 m^3	504 m^3
Total Storage less Fire	22,448 m^3	7,296 m^3
Drawdown Time (without fill)	62.1 hours	20.2 hours
Fill Rate	150 L/s	34 L/s
Drawdown Time (with fill)	n/a	30.5 hours

Based on Table 4.17, when the City is supplying the EIA without filling, the City can supply the connected system for approximately 2.6 days. When considering that the City's reservoirs can fill at the MDD fill rate, the City will not run out of storage in the existing development condition. When the EIA is supplying the City, the EIA will run out of storage in approximately 20 hours without filling, which can be extended to 30 hours when filling.

For the existing system, a connection to the EIA distribution system is not recommended to be pursued further due to the cost of the connection ($2.3-$2.5 million). As assessed in emergency scenario 1, the City distribution system can operate with only one reservoir-pumphouse in operation. The likelihood of having both the North Reservoir and Robinson Reservoir out of service at the same time is very low, and would likely be due to a catastrophic event (extreme weather, terrorism, etc.) that may cause the EIA reservoirs to be out of service as well due to the close proximity. Although a connection between the EIA and the City can provide emergency water supply between jurisdictions during emergency scenarios, the City currently has a connection with Leduc County that could be utilized if necessary (emergency scenario 4).

In the future, when development within both the City and EIA has occurred along 65 Avenue, consideration of a normally closed connection would become more cost-effective. Depending on the proximity of the distribution piping, the cost of the connection could potentially drop to approximately $200,000. However, a main benefit to the City of an existing system connection is the current surplus storage capacity at the EIA. Due to the undefined timing and type of development at the EIA, there may not be surplus storage available. Without surplus storage, the duration the EIA could supply the City would be significantly reduced. It is recommended that the benefit of a connection be re-evaluated once both systems have developed to 65 Avenue.



WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

CITY OF LEDUC AND EIA
WATER DISTRIBUTION SYSTEMS WITH PROPOSED CONNECTION ALTERNATIVES

AECOM

Figure: 4.5

4.3.4 Scenario 4 – Leduc County Servicing and Viability Assessment

There are existing connections between the City's and Leduc County's (the County) water distribution systems along Airport Road. The County's water distribution system begins immediately north of the City's system at Airport Road and the systems are currently connected with closed valves at two locations, at 42 Street and 36 Street. Similar to the EIA assessment, the viability of servicing each system from the other, should one systems reservoir-pumphouses be out of service was assessed.

The following information was collected as part of the County connection assessment:

- The total 2019 ADD for the County was 22.3 L/s. The combined ADD for the City and the County is approximately 105 L/s.
- The County operates two reservoir pumphouses, the west and east reservoirs. The east reservoir generally operates as the lead.
- The ADD and PHD boundary conditions were provided along 11 Avenue within Nisku at hydraulic grade line elevations of approximately 769 m and 761 m, respectively.
- The available fire flow at the boundary was provided at approximately 175 L/s.

The system was assessed during ADD and PHD scenarios utilizing boundary conditions to determine if both systems can provide service to each other.

When the County is providing water to the City, the existing system is sufficient to provide average day demand while maintaining approximately 200 kPa in the southeast (highest elevation) areas of Leduc. During ADD, the pressure within the City system remains above 280 kPa north and west of the intersection of Rollyview Road and 50 Street. The County is capable of providing average day demand to the City.

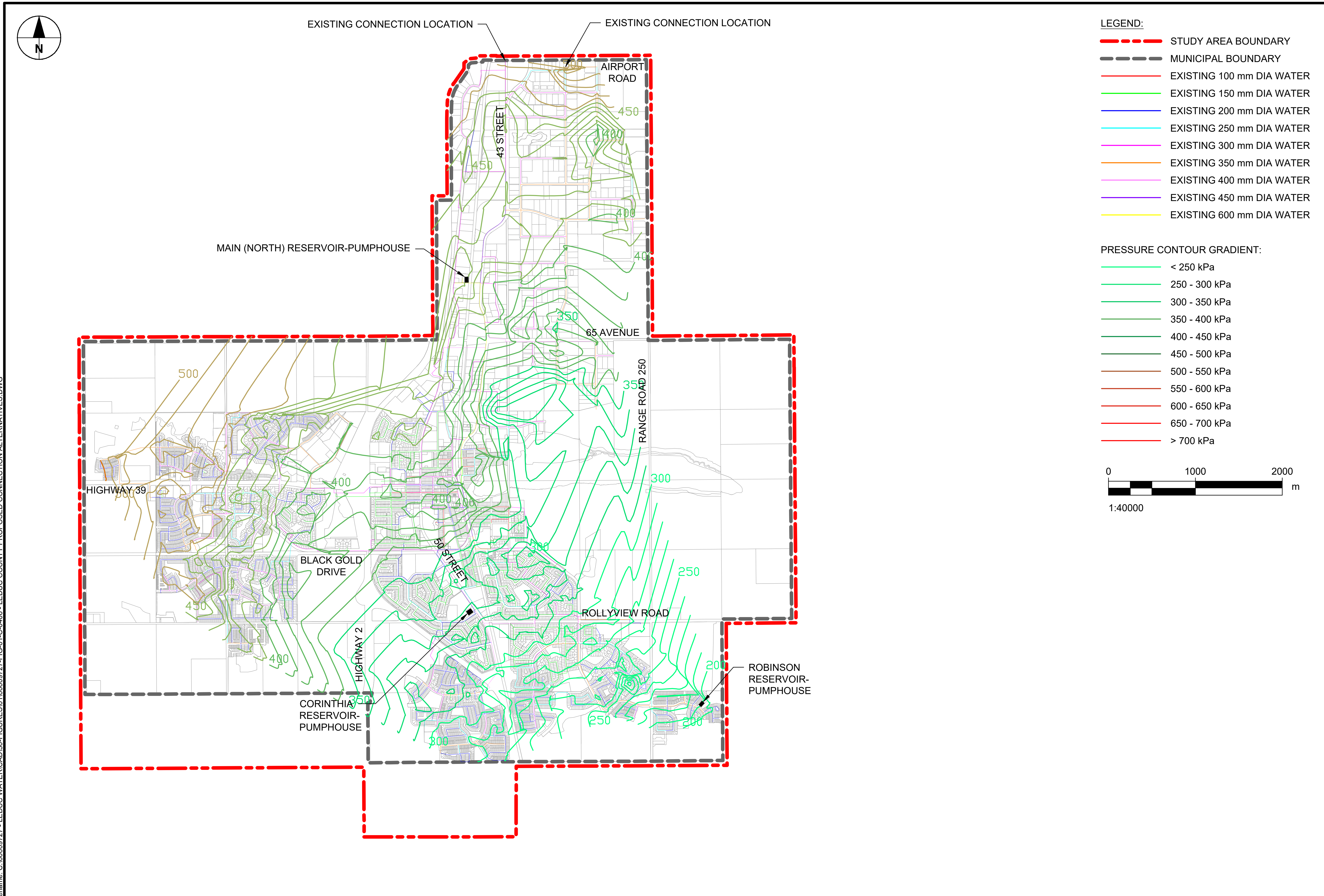
During peak hour demand, the pressure in southeast Leduc drops to approximately 50 kPa assuming that the County has sufficient pumping capacity to provide 200 L/s at the boundary; however, the available fire flow provided was 175 L/s, and thus is not considered feasible. The County is not able to provide peak hour pressures to the City.

The City has sufficient pumping capacity to provide flows to the County in all existing demand scenarios.

Figure 4.6 shows the connection locations to the County as well as the average day demand pressures within the City that would be provided if the County was to provide water to the City.

When the City is supplying flows to the County, during ADD the hydraulic grade line at the boundary is 785 m. Due to the elevation drop from the boundary to the elevation of the east reservoir (23 m), the pressure within the water distribution system at the north boundary of the County's system during ADD would reach up to 860 kPa (125 psi). These high pressures are not sustainable in the distribution network, and there is a significant risk of bursting services and hot water tanks. Pressures could be even higher during low flow situations, such as during the night. Therefore, caution should be used when supplying the County from the City of Leduc without additional infrastructure (PRVs) to lower the hydraulic grade line for flows provided to the County. It should be noted that the boundary valves have been safely opened in the past.

Further analysis between the two systems is not recommended due to the pressure differences between the County and City systems.



AECOM

Figure: 4.6

LEDUC COUNTY CONNECTION VIABILITY ASSESSMENT
AVERAGE DAY DEMAND ANALYSIS

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

4.4 Flushing Program

A unidirectional flushing program was developed for the City of Leduc using Bentley WaterCAD. Hydrants and hydrant laterals were imported from the City's GIS database and existing modelled pipes were split at the connection locations to accommodate them. Valves were imported from the Leduc Water Master Plan CAD. GIS ID's were imported for the existing modelled pipes based on proximity. Some short downtown pipe sections and some pipe alignments were not close enough to be captured by this process, however all hydrants and valves and the majority of pipes are associated with a GIS ID. The addition of future PRVs can be incorporated into the flushing program and would have the potential to impact event limits and timing. Due to the length of the flushing program, it has been provided as a separate document.

4.4.1 Development

Flushing pipe runs were generated by isolating a length of pipe with the objective of flushing approximately 200 m of pipe during each event, although this was restricted by hydrant and valve locations in some scenarios. See Section 4.4.3 for more discussion. The program was divided into two different scenarios – the "Mains," for flushing activities using the 4.5" steamer and the "Local," for flushing activities using the 2.5" nozzle. Water mains with a diameter of less than 300 mm had a target flush velocity of 1.5 m/s while water mains with a diameter of greater than 300 mm had a target flush velocity of 0.9 m/s. Generally, the "Mains" set of events involves flushing the larger diameter pipes and should be the first set of events carried out in each neighbourhood, as the program has been generated to flush from larger pipes to smaller pipes wherever possible.

4.4.2 Procedure

For each event the flushing field report book will indicate the flushing hydrant, which valves to open or close (assuming that the events are being completed sequentially), the recommended flushing time, and the predicted pressure and flowrate. Pressure and flowrate are determined by the model based on a single active pump, and pressures may in fact be higher if additional pumps are active. Each event report accounts for the state of the valves following the previous event, however at the end of each day it is important to reopen any closed valves. Flushing event information has also been added to the hydrant GIS shapefile to develop a figure that can be used as an aid for determining event location and sequencing.

4.4.3 Area Isolation

Difficult to isolate areas are generally identified by high required flushing times. These areas may be limited by the quantity of available valves along their length or at a bounding intersection. Three notable areas were identified during the development of the flushing model:

42 Street & Allard Avenue – The valve at the intersection (WAV1667) is classified as missing, so to isolate this section of Allard Avenue multiple valves at 42 Street and 75 Avenue will need to be closed, resulting in a relatively large area isolated area.

44 Street N of 66 Avenue – 44 Street appears to only have one valve along its length between 66 Avenue and 70 Avenue. The western approaches from the at 45 Street also cannot be isolated along the line and will require flow to be shut down along 45 Street in order to properly isolate this area.

46 Street N of Black Gold Drive – The model has a limited amount of hydrants in the area between Black Gold Drive and 49 Avenue. Nearby hydrants are located along 150 mm lines, which result in an extended flushing time for the 450 mm water main running along the street. This area is not difficult to isolate in the event of repairs or a burst pipe, only in terms of flushing program development.

4.5 Existing System Improvements

4.5.1 Existing System Deficiencies

The following items have been identified as deficiencies of the existing system.

- There are various dead ends at cul-de-sac's that do not meet the fire flow requirement. This is common for water distribution systems where looping is not provided within residential neighborhoods.
- The East Elementary school and residential areas within Telford Lake South do not meet the fire flow requirement. This area was developed with a water system primarily consisting of 100 mm diameter AC pipes that do not have the capacity to provide fire flows. In addition, hydrant spacing was assessed and a hydrant is required on the west edge of the school on 45 Street south of 46 Avenue to provide full coverage for the school.
- The watermain along Rollyview Road to Daystar Church on the east edge of the City does not meet fire flow requirements. In future development scenarios, looping is added to neighborhoods developed to the east and the available fire flow is subsequently increased in this area.
- The existing 150 mm diameter pipe at the Linsford Park School along 51 Street and 41 Avenue. This area does not have sufficient fire flow capacity for the school and other adjacent non-residential areas.
- The existing 100 mm diameter pipes at the Civic Centre along 48a Street between Black Gold Drive and 47 Avenue. This area does not have sufficient capacity for the school and other adjacent non-residential areas.

The existing system has pressures that exceed the maximum recommended value in the northwest corner of the neighborhoods located west of the QEII as well as the industrial area located north of the North Reservoir. These areas are located at the lowest elevations within the system. As seen on Figure 4.2, the pressures on the west edge of the City and in the industrial area at the north edge of the City reach approximately 650 kPa. Through discussions with the City, it is understood that there have not been complaints from residents or businesses due to issues related to high pressure, such as pipe bursting or complaints from residents of high pressure. However, as development west of the QEII continues to the north and west, as well as to the east of the north industrial area, the ground elevation continues to drop and pressures will increase.

4.5.2 Infill/Redevelopment of Existing Development Areas

To support infill development or redevelopment within the City's downtown and mature neighourhoods, upgrades are often required to the existing infrastructure. For water servicing, the required water flow rates for fire protection are much larger than the flow rates required for customer use and govern minimum service levels. The level of fire protection provided within each neighbourhood is dictated by the land use zoning and the standards in place at the time of the development. If rezoning is required for the infill development (for example from single family residential to multi-family residential), then upgrades to the distribution network will likely be required to increase the available fire flows. For infill or redevelopment that is proposed without a zoning change, or between zoning with the same fire flow standards, sufficient fire flows are likely already available. Figure 4.7 highlights the areas in the core areas of the City that currently meet the fire flow standard of 227 L/s for multi-family residential, commercial, industrial and institutional areas. Infill development or redevelopment in these areas would be more cost effective in terms of water servicing.

For infill or redevelopment that triggers rezoning within existing areas that cannot currently provide 227 L/s, upgrades to the distribution network would be required to increase the available fire flows to meet standards, and are historically completed at the developer's cost. Alternatively, individual buildings can install sprinkler systems and non-combustible building materials to lower the required fire protection.

For reference, the City of Edmonton and EPCOR Water Services Inc. have recently developed an Infill Fire Protection Program and Infill Fire Protection Assessment. The proposed Infill Fire Protection Program provides a methodology to fairly share the costs of upgrading fire protection infrastructure in older neighbourhoods to current standard amongst infill developers, water ratepayers and the City's Fire Rescue Services department. The proposed cost share approach recognizes that some fire protection upgrades to the water system that improve fire protection in established areas benefit the entire neighbourhood. The cost sharing approach will allow some infill projects to proceed that otherwise may have been cost-prohibitive to the infill developer.

The Infill Fire Protection Assessment is a new review process to determine whether water infrastructure for on-street fire protection is needed. For developments that require water infrastructure upgrades to meet City Standards, Fire Rescue Services can complete a site-specific review to assess existing hydrant spacing and fire flows, using the methodology outlined in the Fire Underwriters Survey. This assessment process provides a technical basis to relax the upgrade requirements, should the existing fire flows and hydrant spacing be found to be sufficient as a result of the site-specific assessment for the subject site, and can potentially eliminate or reduce the large financial barriers for projects posed by those upgrades.

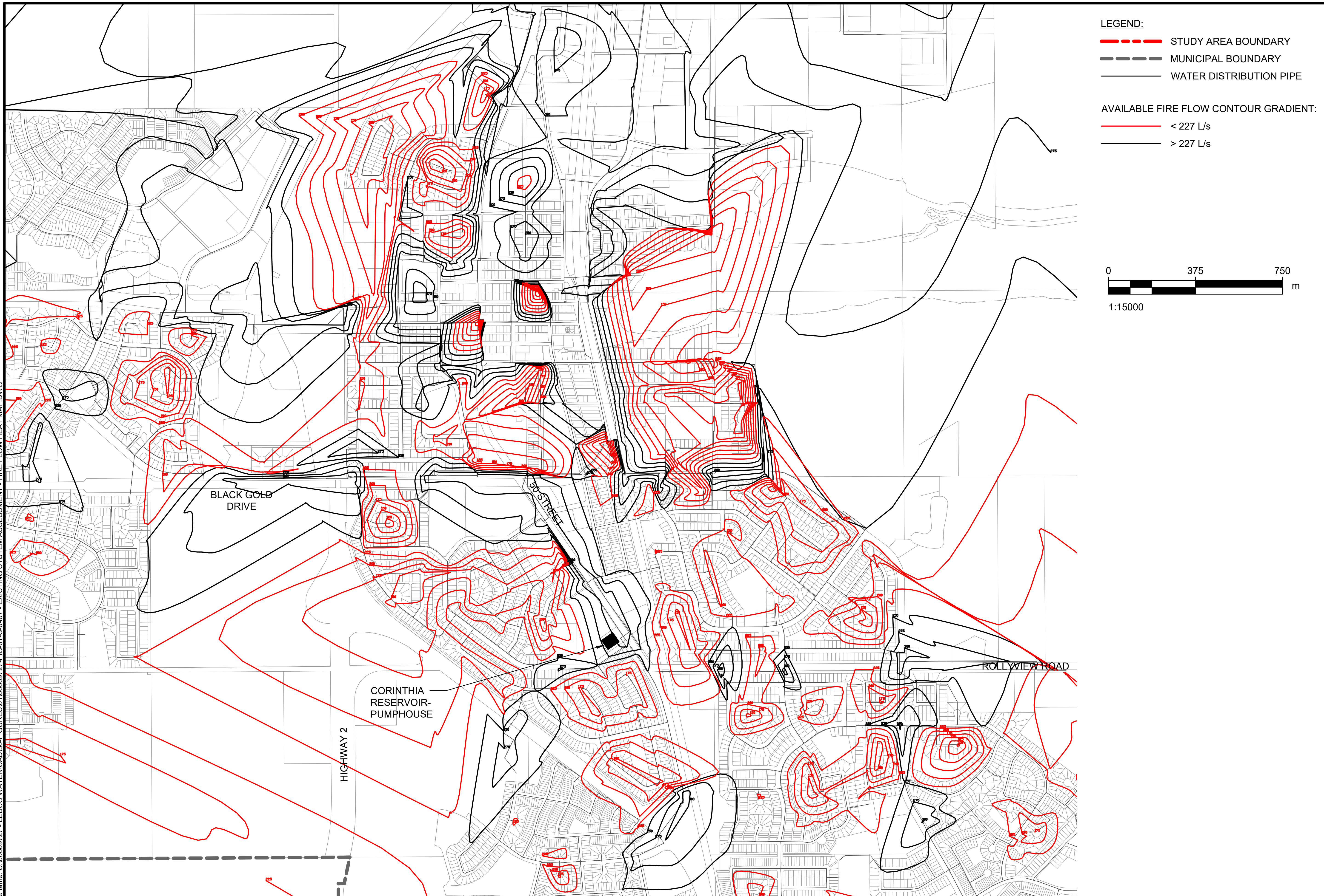


Figure: 4.7

EXISTING WATER DISTRIBUTION SYSTEM
FIRE FLOW MAP

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

4.5.3 Proposed Existing System Improvements

The following pipe improvements area proposed for the existing condition for the purposes of available fire flow. The proposed improvements are shown on Figure 4.8.

- Improvement 1 (Project IMP-1): Upsizing of the existing 100 mm diameter to 200 mm diameter watermain at the Civic Centre, along 48a Street between 47 Avenue and Black Gold Drive.
- Improvement 2 (Project IMP-2): A new 250 mm diameter pipe and upsizing of the existing 100 mm diameter watermain to 250 mm diameter along 46 Avenue between Maisonette Village and 45 Street within the South Telford residential area. Upsize the existing 100 mm diameter watermain to 250 mm along 45 Street from the alley north of 46 Avenue to the School including a fire hydrant on the south end of the improvement. The purpose of the upgrade is to provide fire flows to the East Elementary School.

To reduce high pressures within the water distribution system as discussed in Section 4.3.1, PRVs will be required that will create pressure zones for the north and south areas of the City. However, through discussions with the City, currently there are no reported issues related to high water pressure. Therefore, it is proposed to defer improvements related to splitting the water distribution system into two pressure zones until the Stage 1 development horizon. The proposed PRVs and pressure zones are further discussed in Section 4.6.4.1.

As discussed in Section 4.5.1, upsizing the watermain to the Linsford Park school is required for fire flow. This upgrade includes upgrading the existing 150 mm diameter watermain to 200 mm diameter along 51 Street and 43 Avenue, south of 46 Avenue. The portion of the Linsford Upgrade (Project ST1-1, shown on Figure 4.9) along Black Gold Drive, that involves tying in the watermain downstream of the PRV which keeps area north of Black Gold Drive in the north pressure zone must be completed in conjunction with the PRV installation. Based on discussions with the City, to minimize disruption to the area it is recommended to complete this upgrade as one project, and thus the Linsford upgrade has been shifted to the Stage 1 development horizon.

Similarly, the portion of the Civic Centre upgrade (Project IMP-1) along Black Gold Drive is required in conjunction with the PRV installation. However, there is currently an ongoing project along 48a Street near the Civic Centre and thus the portion of the upgrade along 48a Street is recommended to be implemented in the existing condition as part of that project.

With the implementation of the PRVs, the fire flow within Willow Park is reduced such that the fire flow requirements are not met. This residential area is serviced by a single 150 mm diameter pipe which is recommended to be upsized to 200 mm diameter. This upgrade is recommended during the Stage 1 development once the PRV’s have been implemented (Project ST1-3, shown on Figure 4.9).

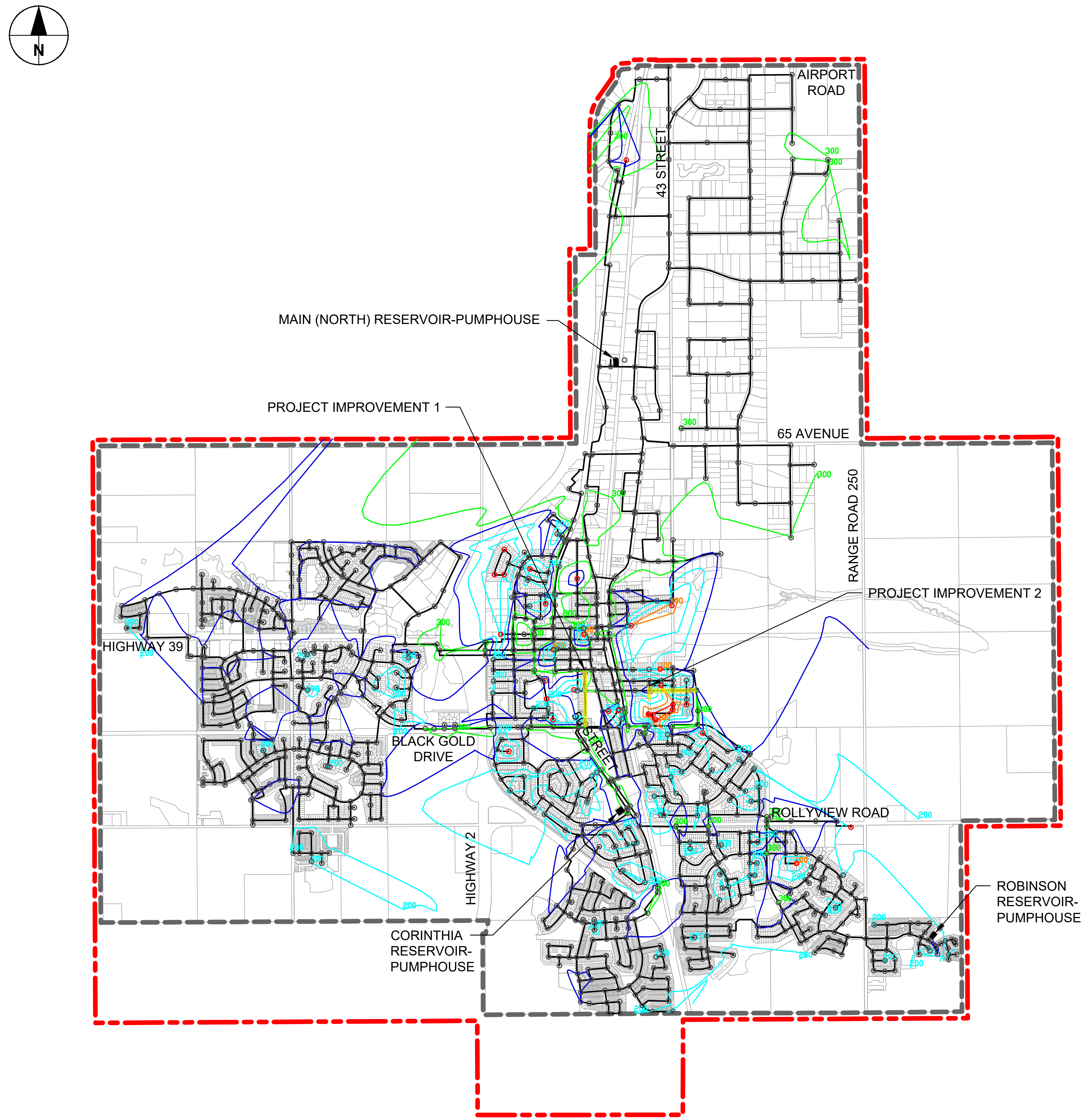
4.5.4 Existing System with Improvements – Hydraulic Assessment

The proposed system improvements are primarily directed towards improving the available fire flow at specific locations within the existing system, generally within the downtown area and South Telford residential area. Therefore, the hydraulic assessment for the Existing System with Improvements consisted of an evaluation of the maximum day demand plus fire flow scenario. Table 4.18 summarizes the hydraulic analysis results for the Existing System with Improvements.

Table 4.18. Existing System with Improvements Assessment – MDD+Fire

Total Number of Nodes	Residential Nodes	Non-Residential Nodes	Residential Nodes Failing Fire Flow Requirements		Non-Residential Nodes Failing Fire Flow Requirements		Total Nodes Failing Fire Flow Requirements	
(No.)	(No.)	(kPa)	(No.)	(%)	(No.)	(%)	(No.)	(%)
1149	805	344	21	2.6	19	5.5	40	3.5

Based on Table 4.18 when compared to the existing system, 10 junctions have improved fire flow and now meet the fire flow requirement, 7 of which are residential and 3 are non-residential. The available fire flow is shown schematically on Figure 4.8.



LEGEND:

- STUDY AREA BOUNDARY
- MUNICIPAL BOUNDARY
- WATER DISTRIBUTION PIPE
- DOES NOT SATISFY FIRE FIRE FLOW REQUIREMENTS
- SATISFIES FIRE FLOW REQUIREMENTS
- IMPROVEMENT PROJECTS

AVAILABLE FIRE FLOW CONTOUR GRADIENT:

- < 50 L/s
- 50 - 100 L/s
- 100 - 150 L/s
- 150 - 200 L/s
- 200 - 250 L/s
- 250 - 300 L/s
- > 300 L/s

0 1000 2000 m

1:40000

EXISTING SYSTEM IMPROVEMENT PROJECTS:

1. UPSIZE EXISTING 100 mm DIAMETER TO 200 mm DIAMETER WATERMAIN FOR FIRE FLOW AT CIVIC CENTRE.
2. NEW 250 mm WATERMAIN AND UPSIZE EXISTING 100 mm DIAMETER TO 250 mm DIAMETER FOR FIRE FLOW TO EAST ELEMENTARY SCHOOL

AECOM

Figure: 4.8

EXISTING SYSTEM WITH IMPROVEMENTS ASSESSMENT MAXIMUM DAY DEMAND PLUS FIRE FLOW

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm

Approved: Checked: Designer: Project Management Initials:

Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0408 - EXISTING SYSTEM WITH IMPROVEMENTS ASSESSMENT - MAXIMUM DAY DEMAND PLUS FIRE.DWG

4.6 Future Water Servicing Description

Future development in the City was assessed based on the following five development scenarios:

- Stage 1: 5-10 Year Development Horizon
- Stage 2: 10-20 Year Development Horizon
- Stage 3: 20-30 Year Development Horizon
- Stage 4: 30+ Year Development Horizon within the Current Municipal Boundary
- Stage 5: Potential Growth Areas Outside Current Municipal Boundary

The development staging of the various neighborhoods was been provided by the City. The approximate area of development for each stage as well as the general direction of development is shown on Figure 2.1.

Due to the relative uncertainty and timeline for the development of the potential growth areas outside the current Municipal Boundary, the demand, storage calculations, and hydraulic assessment for this area has been separated and provided in Section 4.5.

The following sections detail the future water system servicing and assessment.

4.6.1 Future System Demand

The future system demand was determined for each future development scenario based on the land use and development staging as described in Section 2.0. Table 4.19 shows the demand added for each development scenario in addition to the existing system demand.

Table 4.19. Future Additional System Demand

Development Stage	Area	ADD	MDD	PHD
-	**(ha)**	**(L/s)**	**(L/s)**	**(L/s)**
Stage 1	246	40.0	72.0	120.0
Stage 3	445	77.4	139.3	232.1
Stage 3	571	100.6	181.0	301.7
Stage 4	68	18.7	33.6	56.1

Based on Table 4.19, the total cumulative demand for the City during each development stage was determined and is presented in Table 4.20.

Table 4.20. Future Cumulative System Demand

Development Stage	Area	ADD	MDD	PHD
-	**(ha)**	**(L/s)**	**(L/s)**	**(L/s)**
Existing System	1,373	83.4	136.1	214.3
Stage 1	1,619	123.4	208.1	334.2
Stage 2	2,064	200.8	347.4	566.3
Stage 3	2,635	301.4	528.4	868.0
Stage 4	2,703	320.1	562.0	924.1

4.6.2 Future Storage Requirement

The storage requirement for each development stage was determined based on providing fire storage plus emergency storage (one maximum day). Table 4.21 provides the storage requirement for each development stage. The required storage was compared to the existing storage capacity (24,900 m^3) to determine the surplus or additional storage required as applicable.

Table 4.21. Future Reservoir Storage Requirement

Development Stage	Fire Storage	Emergency Storage	Volume Required	Surplus/Deficient Volume
-	(m^3)	(m^3)	(m^3)	(m^3)
Stage 1	2,450	17,980	20,430	Surplus 4,470
Stage 2	2,450	30,010	32,460	Deficient 7,560
Stage 3	2,450	45,650	48,100	Deficient 23,200
Stage 4	2,450	48,560	51,000	Deficient 26,100

As shown in Table 4.21, the existing storage capacity is sufficient for the Stage 1 development horizon; however, additional storage is required for the Stage 2 and further development horizons. Within the existing municipal boundary, there is the opportunity to expand the existing reservoirs. Storage recommendations by stage are discussed in detail in the following sections.

The implementation plan is further discussed in Section 5; however, to maintain water turnover within the reservoirs, storage expansion should be completed in stages based on the progression of development and should be re-assessed during further Master Plan Updates.

4.6.3 Future Pumping Requirement

The pumping requirement for the future development scenarios has been provided and is shown in Table 4.22, based on the largest existing pump out of service. The total existing pumping capacity within the City is 1,240.5 L/s with all pumps in operation. With the largest pump out of service, the total pumping capacity is 1,013.5 L/s.

The governing demand scenario was determined based on the largest calculated demand, which is either the MDD plus Fire or PHD scenario. During earlier development scenarios, the required pumping is governed by MDD plus Fire, and switches to PHD as the demand in the City increases.

Table 4.22. Future Pumping Requirement

Development Scenario	Governing Demand Scenario	Required Pumping Capacity	Surplus Pumping Capacity
		(L/s)	(L/s)
Stage 1	MDD + Fire	435	578
Stage 2	MDD + Fire	574	439
Stage 3	PHD	868	146
Stage 4	PHD	924	89

As shown in Table 4.22, the existing pumping capacity within the City is sufficient for the full buildout within the current municipal boundary. However, as discussed in Section 4.6.6.2, it is proposed to expand the Robinson Reservoir by up to 24,000 m^3 by Stage 3. Once the Robinson Reservoir is fully constructed and development proceeds, the Robinson Reservoir will act in tandem with the North Reservoir as the main reservoir for the City. As demand increases, the utilization of the Robinson Reservoir should be assessed to determine if additional pumps or pump upgrades should be implemented at the Robinson Reservoir.

4.6.4 Stage 1 – 5-10 Year Development Horizon

The proposed water distribution system for the Stage 1 development horizon is shown on Figure 4.9. The network has been laid out to show water distribution mains required to supply water to future areas; however, local distribution pipes have not been included in the assessment. Additional details on Project ST1-1 and ST1-3 are provided in Section 4.5.3.

4.6.4.1 Stage 1 Pressure Zone Implementation

As discussed in Section 4.5.3, it is recommended to implement pressure zones during the Stage 1 development. It is proposed to create two pressure zones, that divide the system into north and south pressure zones by the construction of PRVs as required along Black Gold Drive, as well as the closure of two existing pipes.

The proposed PRVs (Project ST1-2) and pipe closures (Project ST1-4) are shown on Figure 4.9, and include the following:

- PRV-A: Located at Black Gold Drive and the northwest corner of Kinsmen Park on the existing 300 mm diameter watermain along Black Gold Drive. This PRV requires the existing 200 mm diameter pipe in the alley to the north of the PRV to be reconnected on the downstream (west) side of the PRV. The proposed PRV should be installed west of the existing 200 mm diameter pipe to the south that connects Black Gold Drive to Corinthia Park.
- PRV-B: Located along the existing 400 mm diameter watermain at Black Gold Drive on the west side of 48 Street. To implement this PRV, the existing 200 mm diameter pipe along 48a Street north of Black Gold Drive must be disconnected at Black Gold drive and connected to the east at 48a Street on the downstream (east) side of the PRV at 48 Street. In addition, the existing connection of the pipe along 48 street should be closed at Black Gold Drive, with an additional connection made to the 400 mm diameter pipe on the downstream (east) side of the PRV. Both of these reconnections are required to maintain the pressure zone boundary without the implementation of additional PRVs.
- PRV-C: Located at Black Gold Drive on the east side of 46 Street along the existing 300 mm diameter watermain.
- PRV-D: Located on the north side of Black Gold Drive along the existing 300 mm diameter HDPE watermain between Maisonnette Village and Lede Park.
- Pipe Closure 1: The existing 150 mm diameter pipe along 47 Street to the north of Black Gold Drive should be closed. However, there are 4 lots located along this pipe segment that will continue to need service. At the time of this study it was unclear whether this segment of pipe is connected to the system to the north. Therefore, further investigation should be conducted to ensure that service is provided to these lots, while implementing the pressure zones.
- Pipe Closure 2: The existing 150 mm diameter pipe located on the north side of Black Gold Drive to the west of 45 Street should be closed.

With the implementation of the PRVs, the hydraulic grade line elevation within the system can be optimized to both reduce the pressure west of the QEII and in the north industrial area. It is critical to manage the pressure within the system to not overburden the Robinson Reservoir, because the PRVs in general will allow flow from the south pressure zone to the north pressure zone. i.e., the Robinson and Corinthia Reservoir will be able to provide water to the entire system, and the North reservoir, which currently has most of the City's water storage, will not normally be able to provide water to the south pressure zone without opening the valves to allow backflow.

Therefore, the hydraulic grade line settings for the PRVs within the system are proposed as follows:

- The North Reservoir HGL is proposed to be lowered to an elevation of 770 m.
- The HGL along the Black Gold Drive PRVs will be set slightly lower than the PRV at the North Reservoir so that they are normally closed, such that the north reservoir normally provides all water for the north pressure zone. During periods of high demand, as pressure in the north zone drops, the valves will allow flow from the south zone to the north to supplement flows.
- The Robinson and Corinthia Reservoir HGLs are proposed to be increased to an elevation of 790 m.

Table 4.23 summarizes the proposed pressure setting at each reservoir-pumphouse and PRV.

Table 4.23. Future Stage 1 System PRV Settings

PRV	HGL Elevation (m)	Ground Elevation (m)	Pressure Setting (kPa)	Pressure Setting (kPa)
North Reservoir	770	726	432	62.7
Corinthia Reservoir	790	735	540	78.3
Robinson Reservoir	790	747	422	61.2
PRV-A	766	732	334	48.4
PRV-B	768	732.5	348	50.5
PRV-C	768	734.25	331	48.0
PRV-D	768	736.5	309	44.8

The pressure setting at each PRV is critical to the function of the pressure zones. If the pressure settings at a PRV is off of the proposed HGL elevation, this could prevent some PRVs from opening properly limiting flow to some areas in the system. The pressure settings in the above table have been provided based on ground elevation for reference. However, during design and construction, the pressure setting should be set based on the desired hydraulic grade line, and the depth below ground of the PRV should be considered and compensated for when selecting the final PRV settings.

4.6.4.2 Stage 1 Hydraulic Assessment

Similar to the existing development condition, the Stage 1 development horizon was assessed for the ADD, MDD+Fire, and PHD demand scenarios. The Stage 1 assessment was conducted assuming that all improvements as recommended in Section 4.3 have been completed and the pressure zones have been implemented at the hydraulic grade lines as presented in Table 4.23. Additional watermains and associated water demands were added to the model to represent the anticipated development for the Stage 1 development horizon.

To provide flows, the following pumps were activated for each demand scenario as follows:

- ADD: One distribution pump at both the North and Robinson Reservoirs (VSP-202 and VSP-301)
- MDD+Fire: Two distribution pumps at the North Reservoir (VSP-202 and VSP-203), the Fire Pump at the Corinthia Reservoir (FP-104), and three distribution pumps at the Robinson Reservoir (VSP-301, VSP-302, VSP-303).
- PHD: Two distribution pumps at the North Reservoir (VSP-202 and VSP-203), two Distribution Pumps at the Corinthia Reservoir (P-102 and P-103), and three distribution pumps at the Robinson Reservoir (VSP-301, VSP-302, VSP-303).

Tables 4.24 and 4.25 summarize the existing system hydraulic assessment for each demand scenario, and the results are shown schematically on Figures 4.10 through 4.12.

Table 4.24. Stage 1 System Evaluation – ADD and PHD

Scenario	Total Number of Nodes	Minimum Pressure	Maximum Pressure	Nodes with High Pressure		Nodes with Low Pressure	
(-)	(No.)	(kPa)	(kPa)	(No.)	(%)	(No.)	(%)
ADD	1197	322	569	0	0	0	0
PHD		317	569	0	0	0	0

Table 4.25. Stage 1 System Evaluation – MDD+Fire

Total Number of Nodes	Residential Nodes	Non-Residential Nodes	Residential Nodes Failing Fire Flow Requirements		Non-Residential Nodes Failing Fire Flow Requirements		Total Nodes Failing Fire Flow Requirements	
(No.)	(No.)	(kPa)	(No.)	(%)	(No.)	(%)	(No.)	(%)
1197	382	815	20	2.5	18	4.7	38	3.2

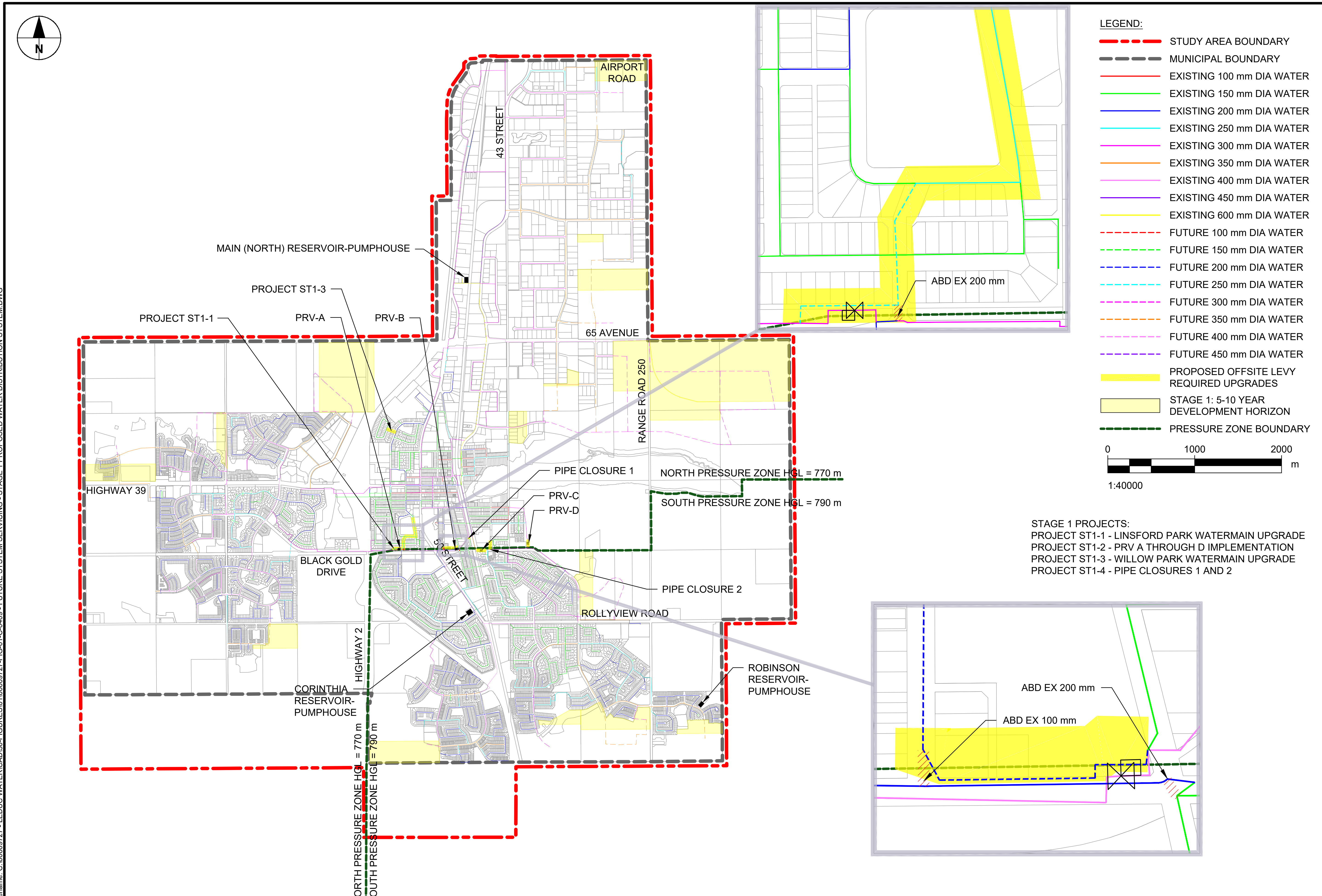
As shown in Table 4.24, based on the modelling results, with the implemented pressure zones, the pressure within the water distribution stays within the acceptable range of 317 kPa to 569 kPa. Figures 4.10 and 4.12 show the pressure contours for the water distribution system for ADD and PHD, respectively. In general, the highest pressures are in the northwest region of the development west of the QEII, the north portion of the industrial area, as well as on the upstream side of the proposed PRVs. The lowest pressures are located on the downstream end of the PRVs.

Pipe velocity is shown on Figures 4.10 and 4.12 for ADD and PHD, respectively. The maximum velocity during ADD is approximately 0.4 m/s; during PHD the highest velocity is approximately 1.0 m/s. Similar to the existing condition, the highest velocities are observed in the QEII crossings, however, some of the 150 mm diameter and smaller pipes within the downtown neighborhood are also reaching a velocity of approximately 0.8-1.0 m/s. In general, the maximum acceptable velocity is 3.0 m/s and thus the distribution system meets the design criteria.

During the MDD+Fire scenario, a total of 38 nodes fail the fire flow requirement. As seen on Figure 4.11, similarly to the existing development condition the majority of the failing nodes are located within cul-de-sacs at locations with longer dead ends. In general, these areas can be opportunistically upgraded to improve the fire flow to these areas, but immediate upgrades are not recommended.

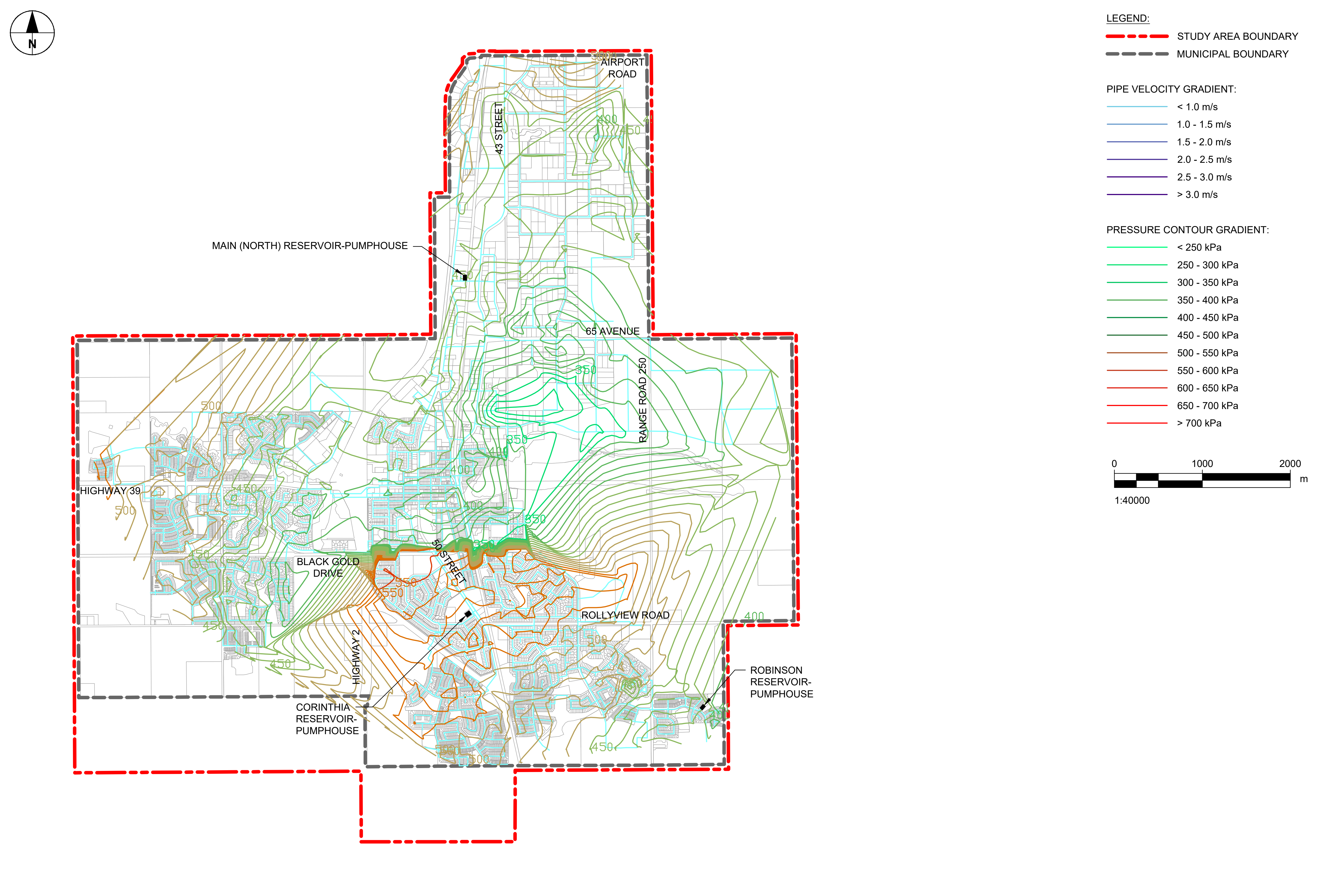
Figure: 4.9

FUTURE DISTRIBUTION SERVICING
STAGE 1 PROPOSED WATER DISTRIBUTION SYSTEM



WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727



AECOM

Figure: 4.10

FUTURE SYSTEM SERVICING
STAGE 1 ASSESSMENT - AVERAGE DAY DEMAND

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved: ____ Checked: ____ Designer: ____ Project Management Initials:
Last saved by: ANDREW.FLECK(2021-10-03) Last Plotted: 2021-10-03
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0410 - FUTURE SYSTEM SERVICING - STAGE 1 ASSESSMENT - AVERAGE DAY DEMAND PRESSURE.DWG

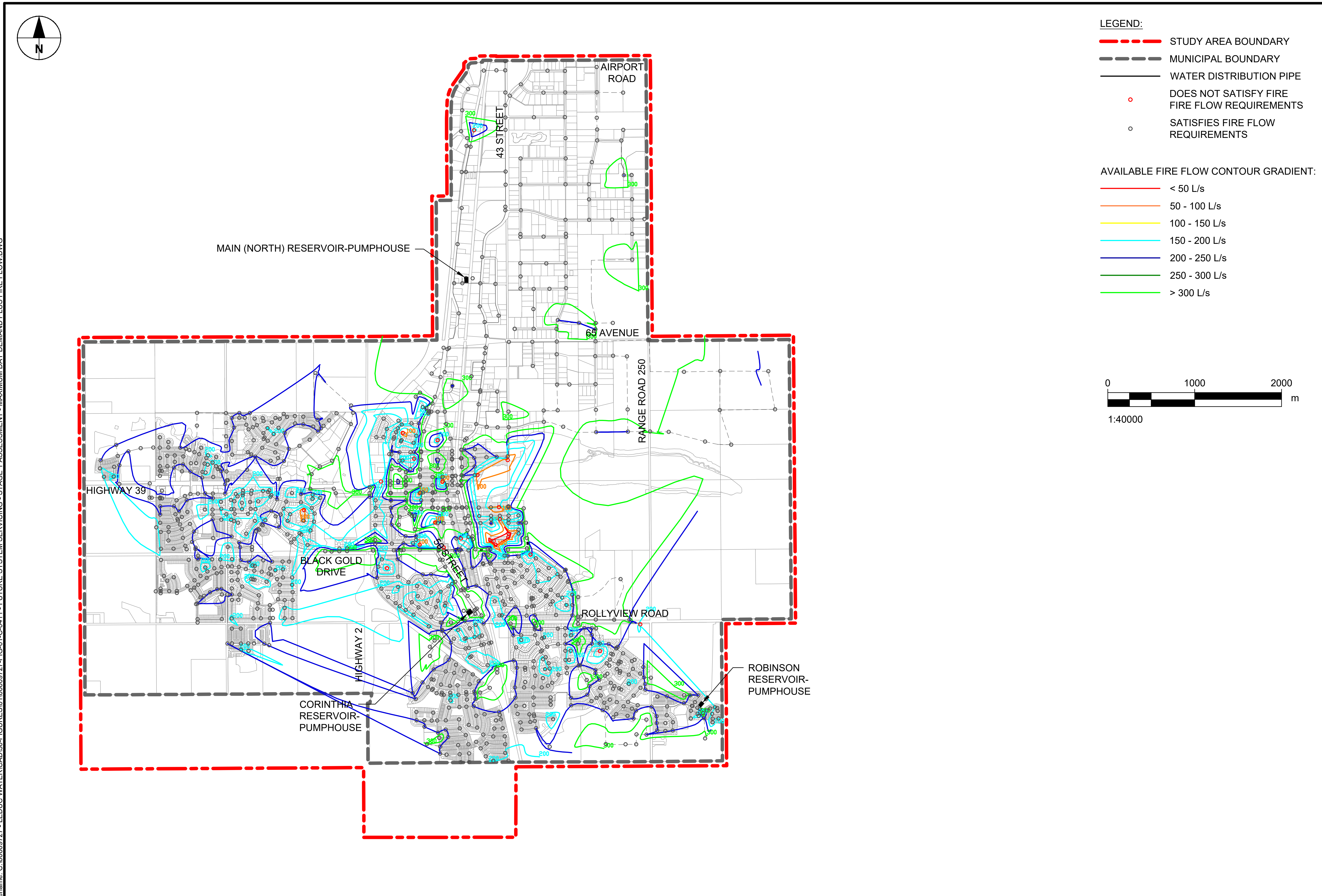


Figure: 4.11

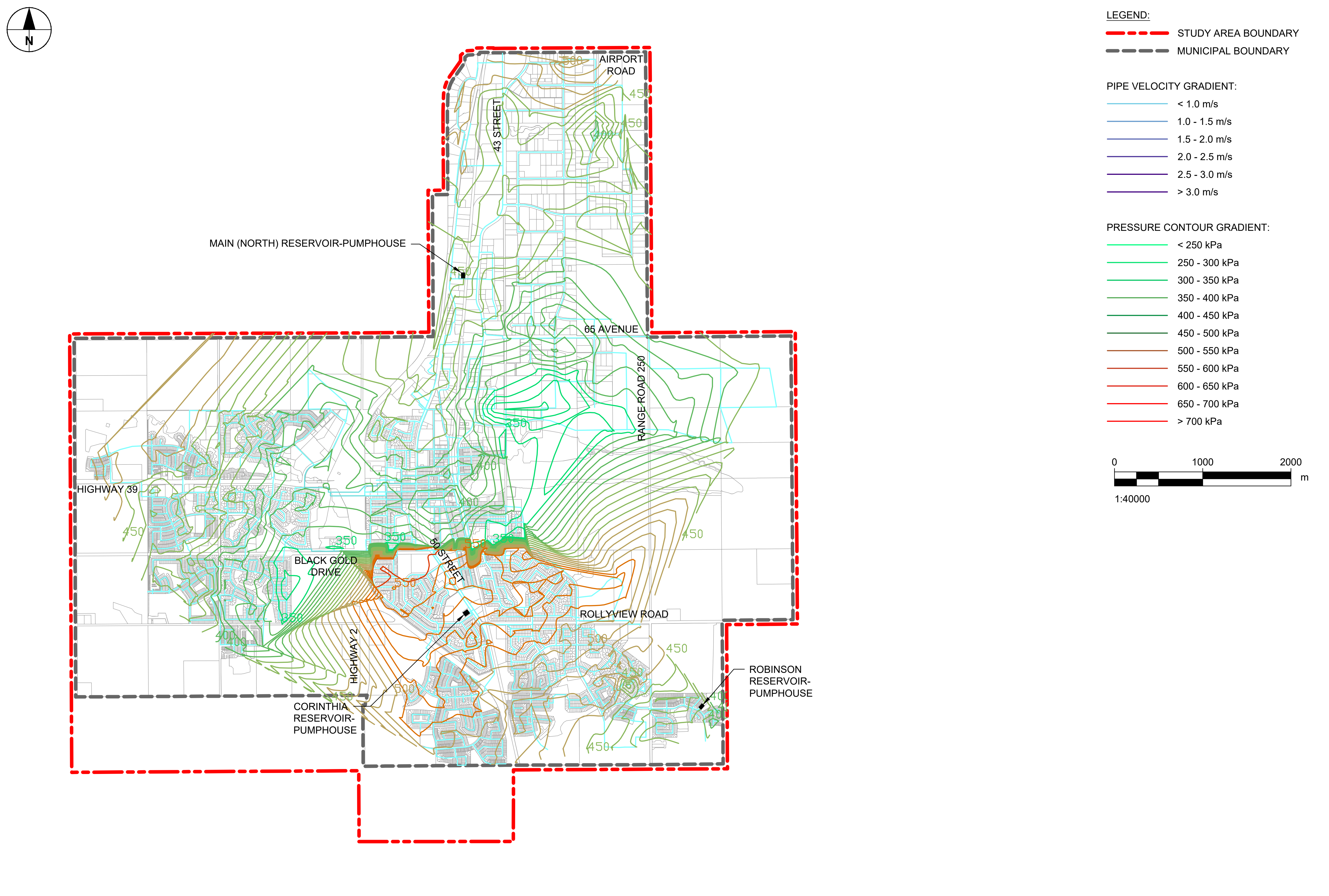
AECOM

FUTURE SYSTEM SERVICING
STAGE 1 ASSESSMENT - MAXIMUM DAY DEMAND PLUS FIRE FLOW

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved: Checked: Designer: Project Management Initials:
Last saved by: ANDREW.FLECK(2021-10-03) Last Plotted: 2021-10-03
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0411 - FUTURE SYSTEM SERVICING - STAGE 1 ASSESSMENT - MAXIMUM DAY DEMAND PLUS FIRE FLOW.DWG



AECOM

Figure: 4.12

FUTURE SYSTEM SERVICING
STAGE 1 ASSESSMENT - PEAK HOUR DEMAND PRESSURE

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved: ____ Checked: ____ Designer: ____ Project Management Initials: ____

Last saved by: ANDREW.FLECK(2021-10-03) Last Plotted: 2021-10-03
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0412 - FUTURE SYSTEM SERVICING - STAGE 1 ASSESSMENT - PEAK HOUR DEMAND PRESSURE.DWG

4.6.5 Stage 2 – 10-20 Year Development Horizon

The proposed water distribution system for the Stage 2 development horizon is shown on Figure 4.13 including the additional pipes required for the Stage 2 development. Similar to the Stage 1 development, the network has been laid out to show water distribution mains required to supply water to future areas; however, local distribution pipes have not been included in the assessment.

4.6.5.1 Stage 2 PRVs and Distribution System

To service future areas west of the QEII, it is recommended that additional highway crossings be added as development progresses. A 300 mm diameter water main crossing to service the 65 Avenue development areas is proposed in Stage 2 development (Project ST2-4).

The Stage 2 development includes providing water servicing to the planned industrial area on the southeast edge of Telford Lake (refer to Figure 2.3 for land use map). To service this area without a long dead end, a connection must be made between the north and south pressure zones requiring an additional PRV (Project ST2-3). The HGL setting at this PRV has been proposed at an elevation slightly lower than the normal pressure of the north zone so that the North Reservoir normally supplies the north zone and if the pressure drops the south zone can supplement flows.

Table 4.26 provides the proposed pressure setting for the Stage 2 PRV.

Table 4.26. Future Stage 2 Development - Additional PRV Settings

PRV	HGL Elevation (m)	Ground Elevation (m)	Pressure Setting (kPa)	Pressure Setting (kPa)
PRV-F1	768	730.5	368	53.4

4.6.5.2 Telford Lake Looping

As seen on Figures 4.13 through 4.16, looping is shown on the east side of Telford Lake, including the implementation of PRV-F1 (Project ST2-3). This looping and connection to Rollyview Road is required once development occurs south of Telford Lake. Based on the development area projections, a portion of the quarter sections on the south side will begin construction during Stage 2, however, may not occur until Stage 3. If adequate looping is provided for fire flows, as development proceeds south adequate ADD pressures can be maintained until development reaches a ground elevation of approximately 732 m, at which point the loop would be required.

4.6.5.3 Stage 2 Storage and Pumping

As shown in Table 4.21 in Section 4.6.2, the existing storage capacity (24,900 m^3) requires approximately 7,500 m^3 of additional storage to meet the storage requirement for Stage 2 (32,460 m^3). There is the opportunity to expand either the North Reservoir or the Robinson Reservoir to meet this storage requirement. Based on a preliminary assessment, the available space for an expansion at the North reservoir is 8,000 m^3, and an additional 18,500 m^3 (which would be constructed in stages) at the Robinson Reservoir. For the purposes of this assessment, it was assumed that during Stage 2 the North Reservoir would still be functioning as the primary reservoir for the City. Thus, for Stage 2 an expansion of the Robinson Reservoir of 4,500 m^3 (Project ST2-1), and then an expansion of the North Reservoir of 8,000 m^3 (Project ST2-2) is recommended for a total system storage capacity of 37,400 m^3, meeting the storage requirement for Stage 2.

During Stage 2, due to the implementation of the PRVs, the Corinthia Reservoir will need to be filled from the Robinson Reservoir. The existing capacity of the Corinthia Reservoir is 6,400 m^3, and thus to turn the water within the reservoir over within one week, it must both supply and be supplied at an average rate of 10.6 L/s. As development proceeds the location of additional storage should be reassessed.

As discussed in Section 4.6.3, the existing pumping capacity is sufficient for the Stage 2 development horizon.

4.6.5.4 Stage 2 Hydraulic Assessment

The Stage 2 water distribution system was assessed for the ADD, MDD+Fire, and PHD demand scenarios assuming that all additional watermains required to provide servicing for Stage 1 have been completed. Additional watermains and associated water demands were added to the model to represent the anticipated development for the Stage 2 development horizon.

To provide flows, the following pumps were activated for each demand scenario as follows:

- <u>ADD</u>: One distribution pump the North Reservoir (VSP-202) and two pumps at the Robinson Reservoir (VSP-301 and VSP-302)
- <u>MDD+Fire</u>: Two distribution pumps at the North Reservoir (VSP-202 and VSP-203), the Fire Pump at the Corinthia Reservoir (FP-104), and three distribution pumps at the Robinson Reservoir (VSP-301, VSP-302, VSP-303).
- <u>PHD</u>: All pumps at the North Reservoir (VSP-202, VSP-203, and CSD-204), two distribution pumps at the Corinthia Reservoir (P-102 and P-103), and three distribution pumps at the Robinson Reservoir (VSP-301, VSP-302, VSP-303).

Tables 4.27 and 4.28 summarize the existing system hydraulic assessment for each demand scenario, and the results are shown schematically on Figures 4.14 through 4.16.

Table 4.27. Stage 2 System Evaluation – ADD and PHD

Scenario	Total Number of Nodes	Minimum Pressure	Maximum Pressure	Nodes with High Pressure		Nodes with Low Pressure	
(-)	(No.)	(kPa)	(kPa)	(No.)	(%)	(No.)	(%)
ADD	1244	321	585	2	0.2	0	0
PHD		302	571	1	0.1	0	0

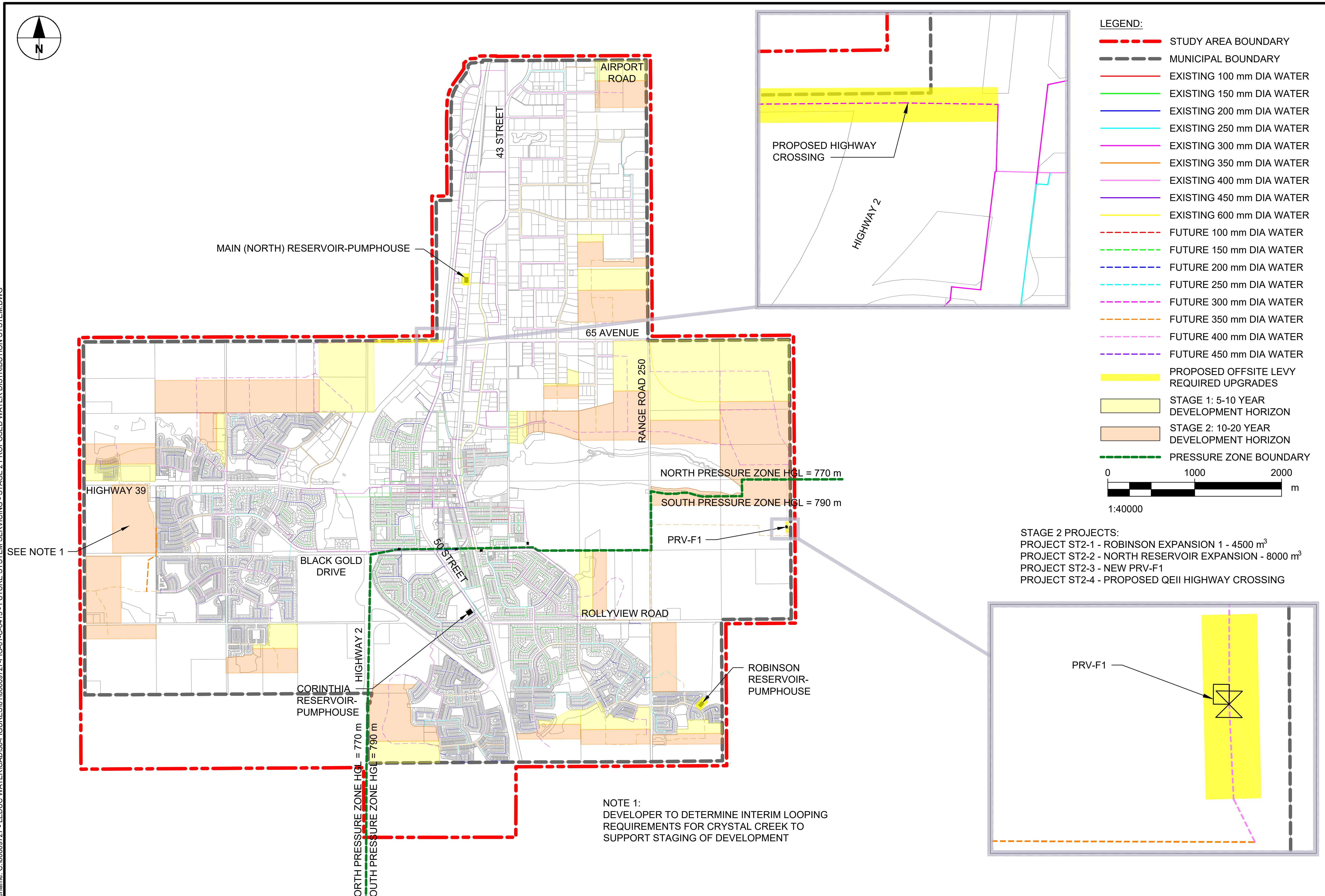
Table 4.28. Stage 2 System Evaluation – MDD+Fire

Total Number of Nodes	Residential Nodes	Non-Residential Nodes	Residential Nodes Failing Fire Flow Requirements		Non-Residential Nodes Failing Fire Flow Requirements		Total Nodes Failing Fire Flow Requirements	
(No.)	(No.)	(kPa)	(No.)	(%)	(No.)	(%)	(No.)	(%)
1244	826	420	20	2.4	22	5.2	42	3.4

As shown in Table 4.27, based on the modelling results, with the implemented pressure zones the maximum pressure within the water distribution is 585 kPa during ADD, with minimum pressure at 321 kPa. Figures 4.14 and 4.16 show the pressure contours for the water distribution system for ADD and PHD, respectively. Similar to previous development conditions, the highest pressures are in the northwest region of the development west of the QEII, the north portion of the industrial area, as well as on the upstream side of the proposed PRVs. The lowest pressures are located on the downstream end of the PRVs.

Pipe velocity is shown on Figure 4.14 and 4.16 during ADD the PHD, respectively. The maximum velocity during ADD is approximately 0.5 m/s; during PHD the highest velocity is approximately 1.3 m/s. During PHD, the highest velocities are in the 600 mm diameter outlet pipe from the North Reservoir, prior to splitting at the distribution system. Similar to the existing condition, velocities up to approximately 1.1 m/s are observed in the QEII crossings and some of the smaller diameter pipes downtown. In general, the maximum acceptable velocity is 3.0 m/s and thus the distribution system meets the design criteria.

During the MDD+Fire scenario, a total of 42 nodes fail the fire flow requirement. As seen on Figure 4.15, similarly to the existing development condition the majority of the failing nodes are located within cul-de-sacs at locations with longer dead ends. If other upgrades within the neighborhood are being implemented, it is recommended to assess the benefit of water upgrades in these areas on a case-by-case basis.



AECOM

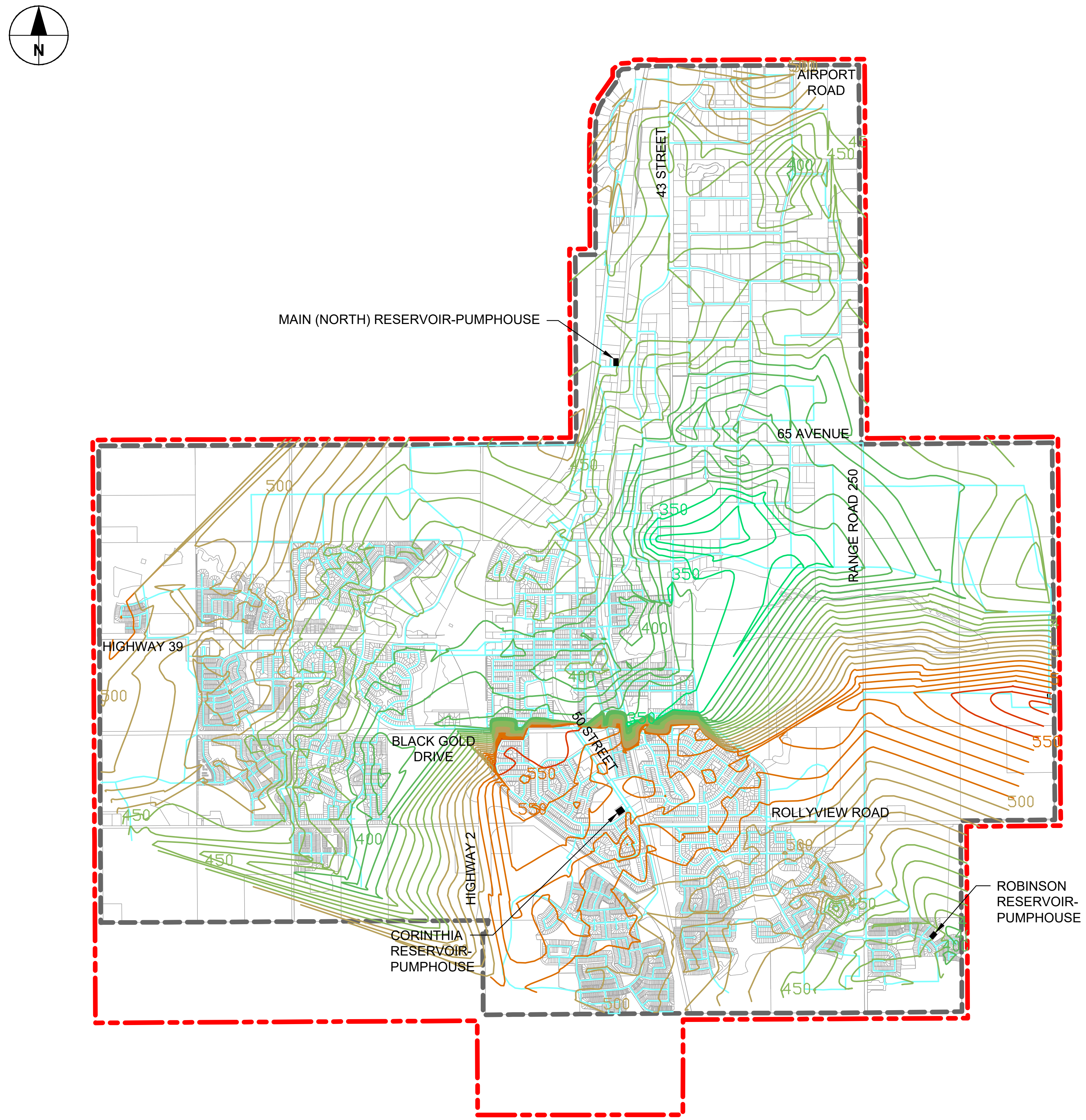
Figure: 4.13

FUTURE DISTRIBUTION SERVICING
STAGE 2 PROPOSED WATER DISTRIBUTION SYSTEM

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved:
Checked:
Designer:
Project Management Initials:
Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0413 - FUTURE SYSTEM SERVICING - STAGE 2 PROPOSED WATER DISTRIBUTION SYSTEM.DWG



LEGEND:

STUDY AREA BOUNDARY

MUNICIPAL BOUNDARY

PIPE VELOCITY GRADIENT:

- < 1.0 m/s
- 1.0 - 1.5 m/s
- 1.5 - 2.0 m/s
- 2.0 - 2.5 m/s
- 2.5 - 3.0 m/s
- > 3.0 m/s

PRESSURE CONTOUR GRADIENT:

- < 250 kPa
- 250 - 300 kPa
- 300 - 350 kPa
- 350 - 400 kPa
- 400 - 450 kPa
- 450 - 500 kPa
- 500 - 550 kPa
- 550 - 600 kPa
- 600 - 650 kPa
- 650 - 700 kPa
- > 700 kPa

0 1000 2000 m

1:40000

AECOM

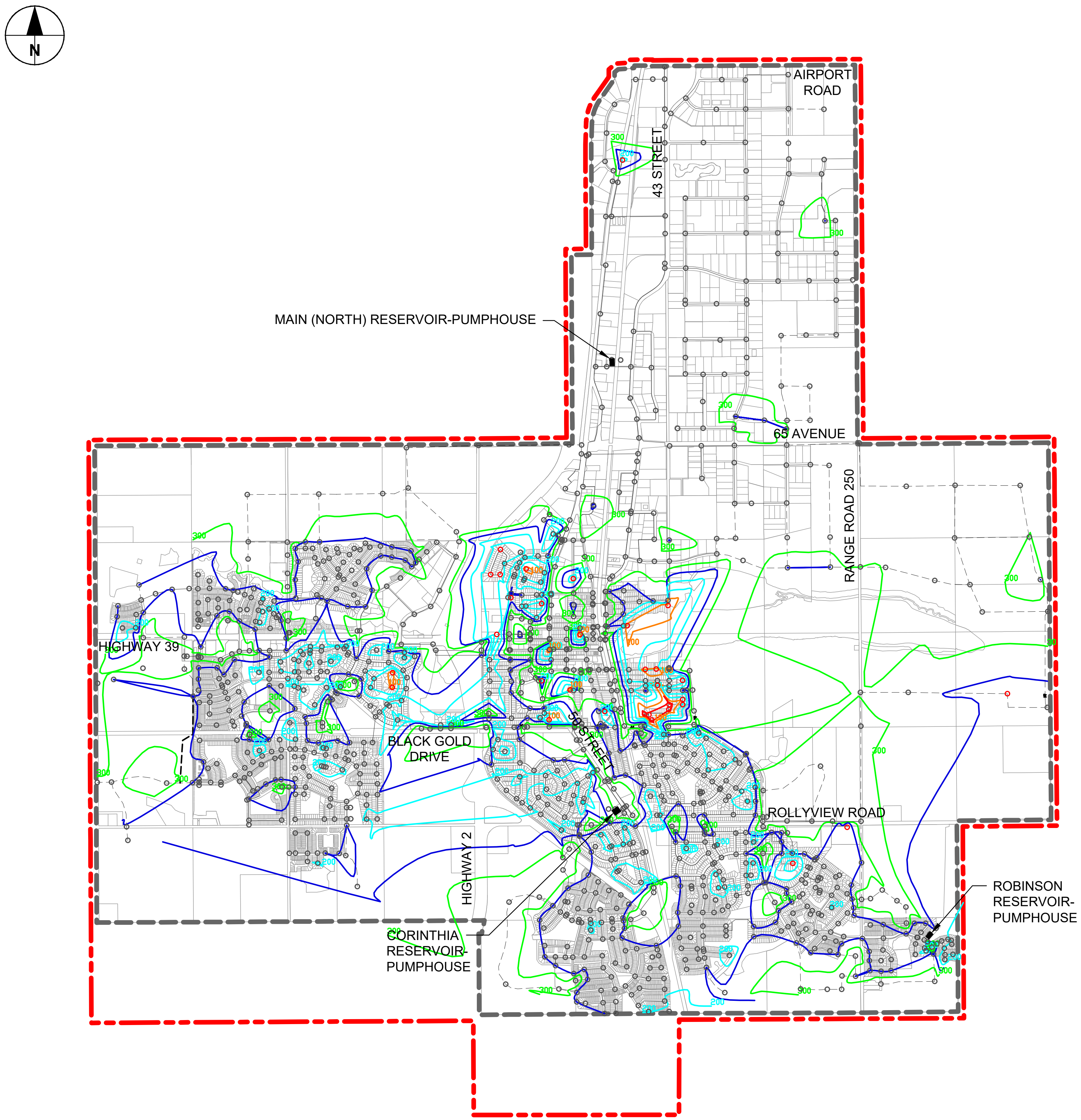
Figure: 4.14

FUTURE SYSTEM SERVICING
STAGE 2 ASSESSMENT - AVERAGE DAY DEMAND

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0414 - FUTURE SYSTEM SERVICING - STAGE 2 ASSESSMENT - AVERAGE DAY DEMAND PRESSURE.DWG
Project Management Initials: Designer: Checked: Approved: ANSI B 279.4mm x 431.8mm



LEGEND:

- STUDY AREA BOUNDARY
- MUNICIPAL BOUNDARY
- WATER DISTRIBUTION PIPE
- DOES NOT SATISFY FIRE FIRE FLOW REQUIREMENTS
- SATISFIES FIRE FLOW REQUIREMENTS

AVAILABLE FIRE FLOW CONTOUR GRADIENT:

- < 50 L/s
- 50 - 100 L/s
- 100 - 150 L/s
- 150 - 200 L/s
- 200 - 250 L/s
- 250 - 300 L/s
- > 300 L/s

0 1000 2000 m

1:40000

AECOM

Figure: 4.15

FUTURE SYSTEM SERVICING

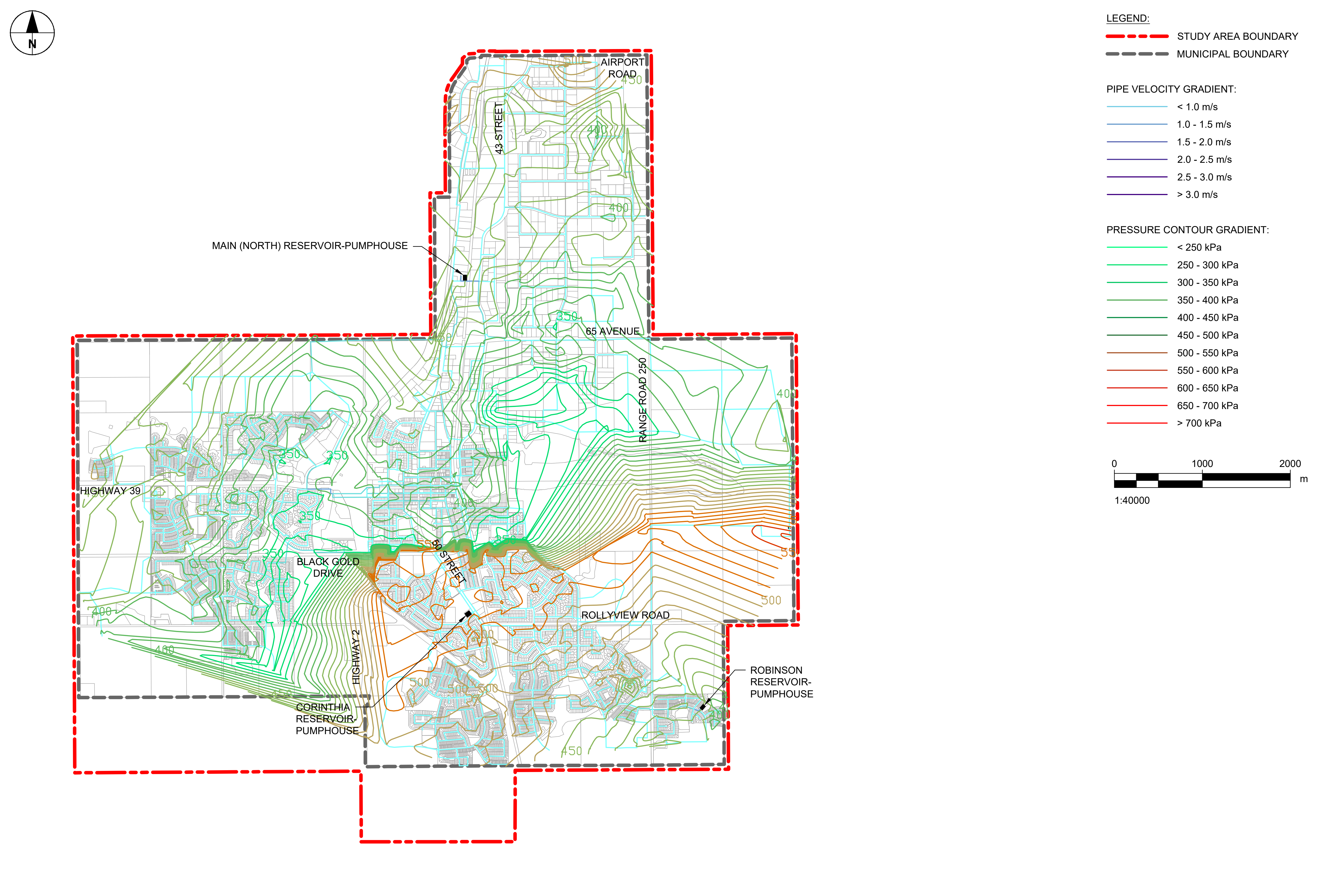
STAGE 2 ASSESSMENT - MAXIMUM DAY DEMAND PLUS FIRE FLOW

WATER MASTER PLAN UPDATE

CITY OF LEDUC

Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved:
Checked:
Designer:
Project Management Initials:
Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0415 - FUTURE SYSTEM SERVICING - STAGE 2 ASSESSMENT - MAXIMUM DAY DEMAND PLUS FIRE FLOW.DWG



AECOM

Figure: 4.16

FUTURE SYSTEM SERVICING
STAGE 2 ASSESSMENT - PEAK HOUR DEMAND

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved: ____ Checked: ____ Designer: ____ Project Management Initials: ____
Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0416 - FUTURE SYSTEM SERVICING - STAGE 2 ASSESSMENT - PEAK HOUR DEMAND PRESSURE.DWG

4.6.6 Stage 3 – 20-30 Year Development Horizon

The proposed water distribution system for the Stage 3 development horizon is shown on Figure 4.17 including the additional pipes required for the Stage 3 development. Similar to previous development stages, the network has been laid out to show water distribution mains required to supply water to future areas; however, local distribution pipes have not been included in the assessment.

4.6.6.1 Stage 3 PRVs and Distribution System

The Stage 3 development includes providing water servicing to the residential area west of the QEII to the south of the existing development (in the future Blackstone, Brightwell, Backs of Crystal Creek neighborhoods). To provide fire flows within these neighborhoods as well as some redundancy should a crossing require closure, an additional QEII crossing is proposed which will require a PRV to maintain the proposed pressure zones (Project ST3-1). The HGL setting at this PRV is shown in Table 4.29, and has been proposed similarly to the Stage 1 PRVs at an elevation slightly lower than the normal pressure of the north zone, so that the North Reservoir normally supplies the north zone and if the pressure drops the south zone can supplement flows.

Table 4.29. Future Stage 3 Development - Additional PRV Settings

PRV	HGL Elevation (m)	Ground Elevation (m)	Pressure Setting (kPa)	Pressure Setting (kPa)
PRV-F2	766	732	334	48.4

A 350 mm crossing of Hwy 2A at Stutz Link (Project ST3-2) is proposed for Stage 3 development as well, to support development in the Southfork ASP area.

4.6.6.2 Stage 3 Storage and Pumping

As discussed in Section 4.6.2, the existing storage capacity (24,900 m^3) requires approximately 23,000 m^3 of additional storage to meet the storage requirement for Stage 3 (48,100 m^3). Assuming the Stage 2 storage upgrades of 4,500 m^3 and 8,000 m^3 have been implemented at the Robinson and North Reservoirs respectively, an additional 10,700 m^3 of storage is required for Stage 3.

A second expansion of the Robinson Reservoir is proposed at 14,000 m^3 (Project ST3-3) bringing the total storage at the Robinson Reservoir to 23,000 m^3 and the total storage capacity for the City to 51,400 m^3. This storage expansion is recommended to be implemented near the end of Stage 3. The expansion would be sufficient for both the Stage 3 and Stage 4 development and is proposed to reduce storage expansion projects until the end of development within the current municipal boundary (end of Stage 4).

As discussed in Section 4.6.3, the total existing pumping capacity exceeds the pumping requirement for the Stage 3 development horizon. However, during peak hour demand, the pressure within the north zone is dropping such that the Robinson and South Reservoirs are supplementing flows to the north pressure zone, which results in all existing pumps at the Robinson Reservoir required to be in operation. To meet Alberta Environment requirements, with the largest pump out of service, the system should be sufficient to provide flows to the system. Therefore, pump upgrades are recommended at the Robinson Reservoir (Project ST3-4). Within the existing pumphouse, the connection of the three distribution pumps to the main header has been sized to allow for upgrades of the distribution pumps.

Design items that would need to be reviewed/addressed for the Robinson Reservoir Expansion include the following:

1. Fill line would need to be upgraded. Currently the fill line is 450 mm diameter to the building then drops to 250 mm inside the building.
2. Pumps would need to be upgraded. An upgrade of 200 HP has been assumed for cost purposes; however, this should be reviewed in detail during the next design phases.
3. The electrical service and generator sizing would need to be checked; they do not appear to be designed for increased load and the electrical room is tight. Based on the single line diagram for the Robinson Reservoir, the existing generator can run two small pumps and one large pump, or both of the large pumps. A generator upgrade would likely be required.
4. Confirm the capacity of the main header (currently 400 mm).
5. The overflow arrangement would need to be reviewed and replaced if the fill capacity is increased.
6. Confirm the existing structure can accommodate the cutting of large openings in the existing walls.
7. Remove and replace outdoor rink.

Existing constraints at the Robinson Reservoir site include the AltaGas facility to the northeast and the existing outdoor rink to the southeast. A preliminary site review indicates that the proposed reservoir expansions totalling 18,500 m^3 could be accommodated with relocation of one of these facilities; however, this would need to be confirmed during later design stages. The Robinson Reservoir could potentially be expanded by up to 24,000 m^3 if both facilities are relocated.

4.6.6.3 Stage 3 Hydraulic Assessment

The Stage 3 water distribution system was assessed for the ADD, MDD+Fire, and PHD demand scenarios assuming all additional watermains required to provide servicing for Stage 1 and 2 have been completed. Additional watermains and associated water demands were added to the model to represent the anticipated development for the Stage 3 development horizon.

To provide flows for each demand scenario, it was assumed that pumps at the Robinson Reservoir were upgraded to provide sufficient flows to the system with both distribution pumps in operation at the North Reservoir. During pressure scenarios (ADD and PHD), at the Corinthia Reservoir, the two distribution pumps were assumed to be in operation and during MDD plus Fire Flow the fire pump was assumed to be in operation.

Tables 4.30 and 4.31 summarize the Stage 3 system hydraulic assessment for each demand scenario, and the results are shown schematically on Figures 4.18 through 4.20.

Table 4.30. Stage 3 System Evaluation – ADD and PHD

Scenario	Total Number of Nodes	Minimum Pressure	Maximum Pressure	Nodes with High Pressure		Nodes with Low Pressure	
(-)	(No.)	(kPa)	(kPa)	(No.)	(%)	(No.)	(%)
ADD	1285	318	585	3	0.2	0	0
PHD		304	505	0	0	0	0

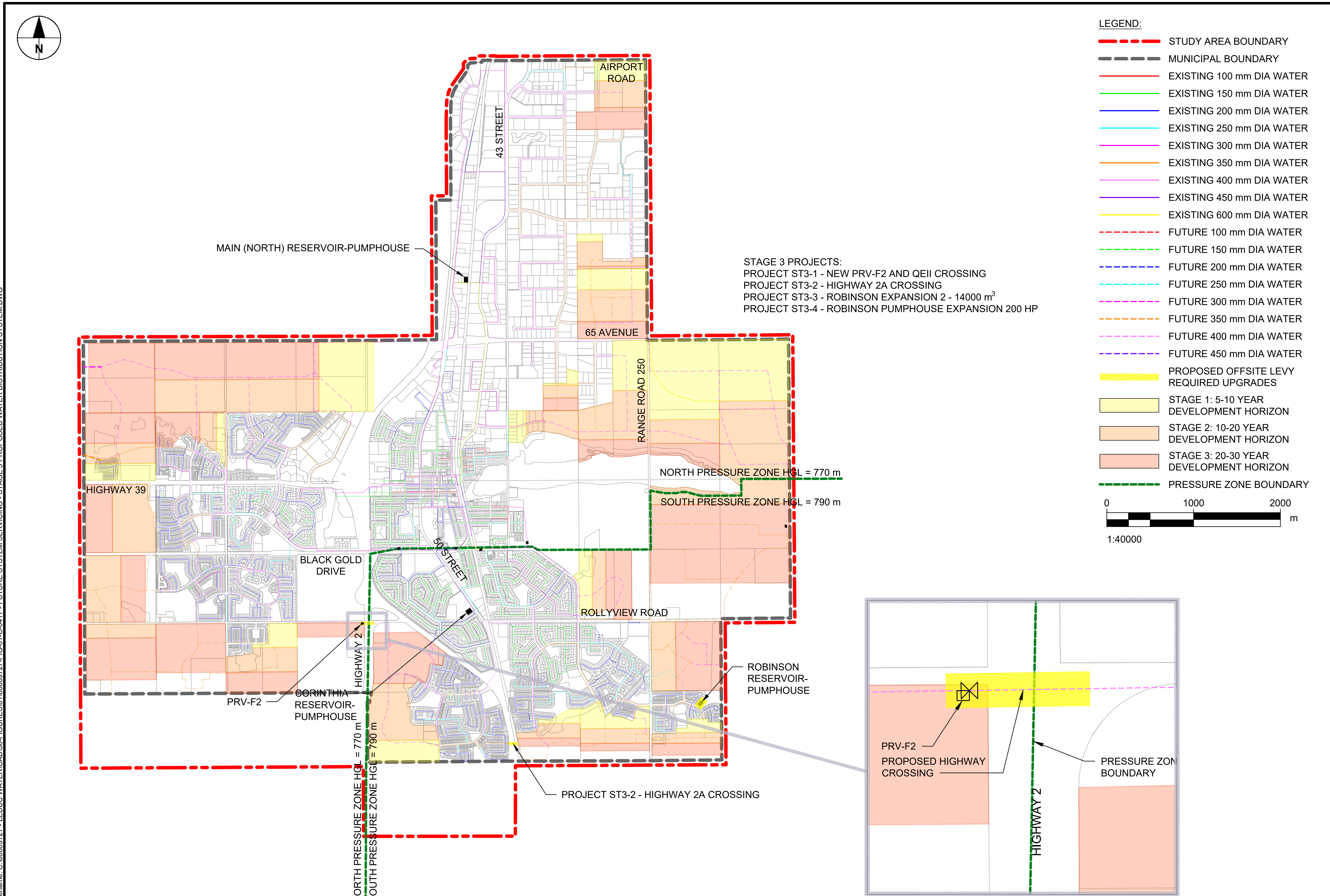
Table 4.31. Stage 3 System Evaluation – MDD+Fire

Total Number of Nodes	Residential Nodes	Non-Residential Nodes	Residential Nodes Failing Fire Flow Requirements		Non-Residential Nodes Failing Fire Flow Requirements		Total Nodes Failing Fire Flow Requirements	
(No.)	(No.)	(kPa)	(No.)	(%)	(No.)	(%)	(No.)	(%)
1285	827	458	22	2.7	24	5.2	46	3.6

As shown in Table 4.30, based on the modelling results, with the implemented pressure zones the maximum pressure within the water distribution is 585 kPa during ADD, with minimum pressure at 304 kPa during PHD. Figures 4.18 and 4.20 show the pressure contours for the water distribution system for ADD and PHD, respectively. Similar to previous development conditions, the highest pressures are in the northwest region of the development west of the QEII, the north portion of the industrial area, as well as on the upstream side of the proposed PRVs. The lowest pressures are located on the downstream end of the PRVs.

Pipe velocity is shown on Figures 4.18 and 4.20 during ADD the PHD, respectively. The maximum velocity during ADD is approximately 0.75 m/s; during PHD the highest velocity is approximately 3.3 m/s. During PHD, the system is exceeding the maximum recommended velocity in the Robinson Reservoir 400 mm diameter header. In addition, velocities in the existing 350 mm diameter pipe from the Robinson Reservoir along Robinson Drive are reaching approximately 2.8 m/s. It is recommended that the flow rates from the Robinson Reservoir be monitored, and if the peak hour flow rates approach 350 L/s out of the Robinson Reservoir the capacity of these pipes should be considered for upgrade or twinning. Due to the implementation of the additional QEII crossing, the velocities within the crossings remain within the acceptable range at approximately 1.5 m/s.

During the MDD+Fire scenario, a total of 46 nodes fail the fire flow requirement. As seen on Figure 4.19, similarly to the existing development condition the majority of the failing nodes are located within cul-de-sacs at locations with longer dead ends. If other upgrades within the neighborhood are being implemented, it is recommended to assess the benefit of water upgrades in these areas on a case-by-case basis.



AECOM

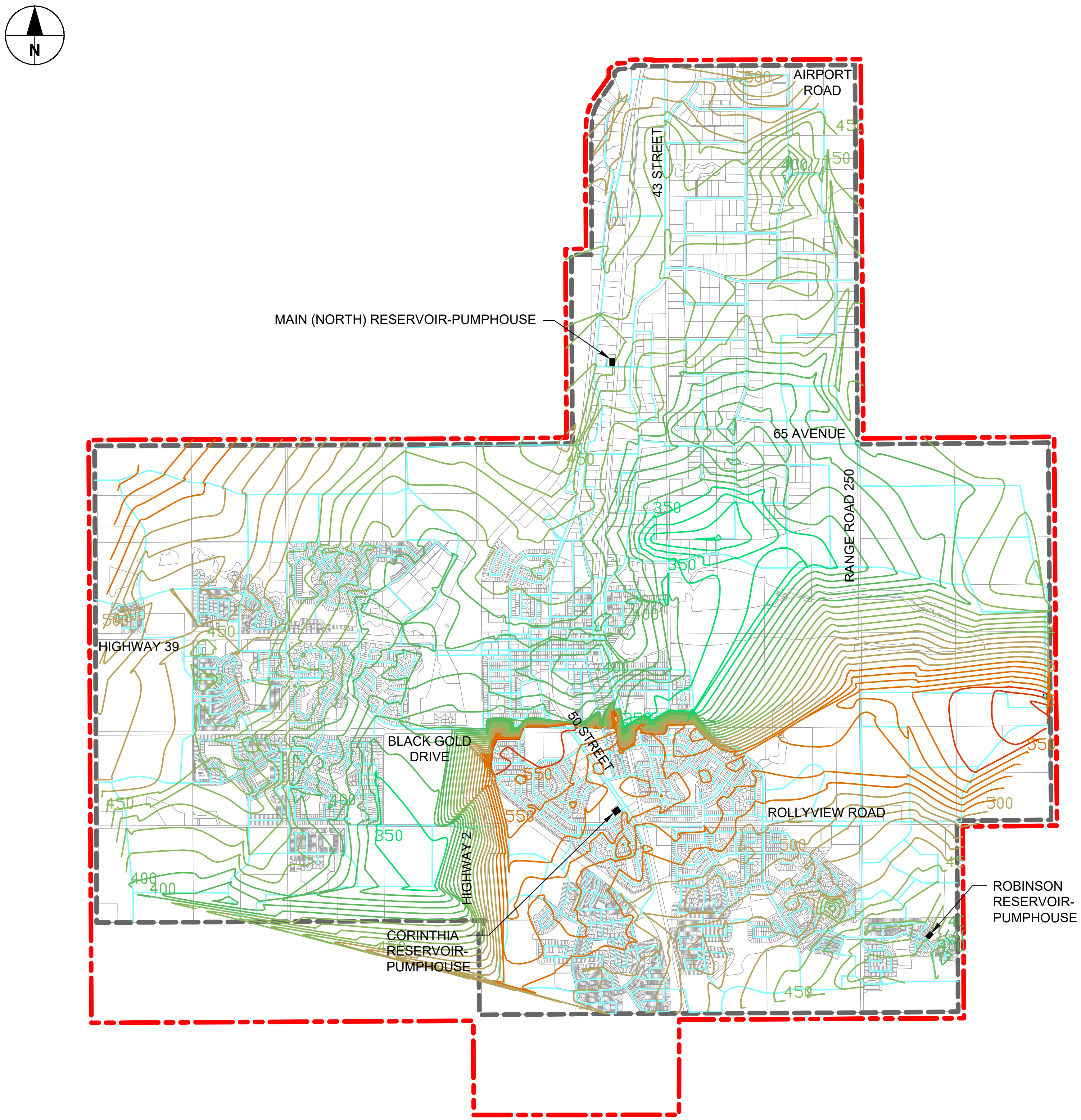
Figure: 4.17

FUTURE DISTRIBUTION SERVICING
STAGE 3 PROPOSED WATER DISTRIBUTION SYSTEM

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Approved:
Checked:
Designer:
Project Management Initials:
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0417 - FUTURE SYSTEM SERVICING - STAGE 3 PROPOSED WATER DISTRIBUTION SYSTEM.DWG
Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28



LEGEND:

STUDY AREA BOUNDARY

MUNICIPAL BOUNDARY

PIPE VELOCITY GRADIENT:

- < 1.0 m/s
- 1.0 - 1.5 m/s
- 1.5 - 2.0 m/s
- 2.0 - 2.5 m/s
- 2.5 - 3.0 m/s
- > 3.0 m/s

PRESSURE CONTOUR GRADIENT:

- < 250 kPa
- 250 - 300 kPa
- 300 - 350 kPa
- 350 - 400 kPa
- 400 - 450 kPa
- 450 - 500 kPa
- 500 - 550 kPa
- 550 - 600 kPa
- 600 - 650 kPa
- 650 - 700 kPa
- > 700 kPa

0 1000 2000 m

1:40000

AECOM

Figure: 4.18

FUTURE SYSTEM SERVICING
STAGE 3 ASSESSMENT - AVERAGE DAY DEMAND

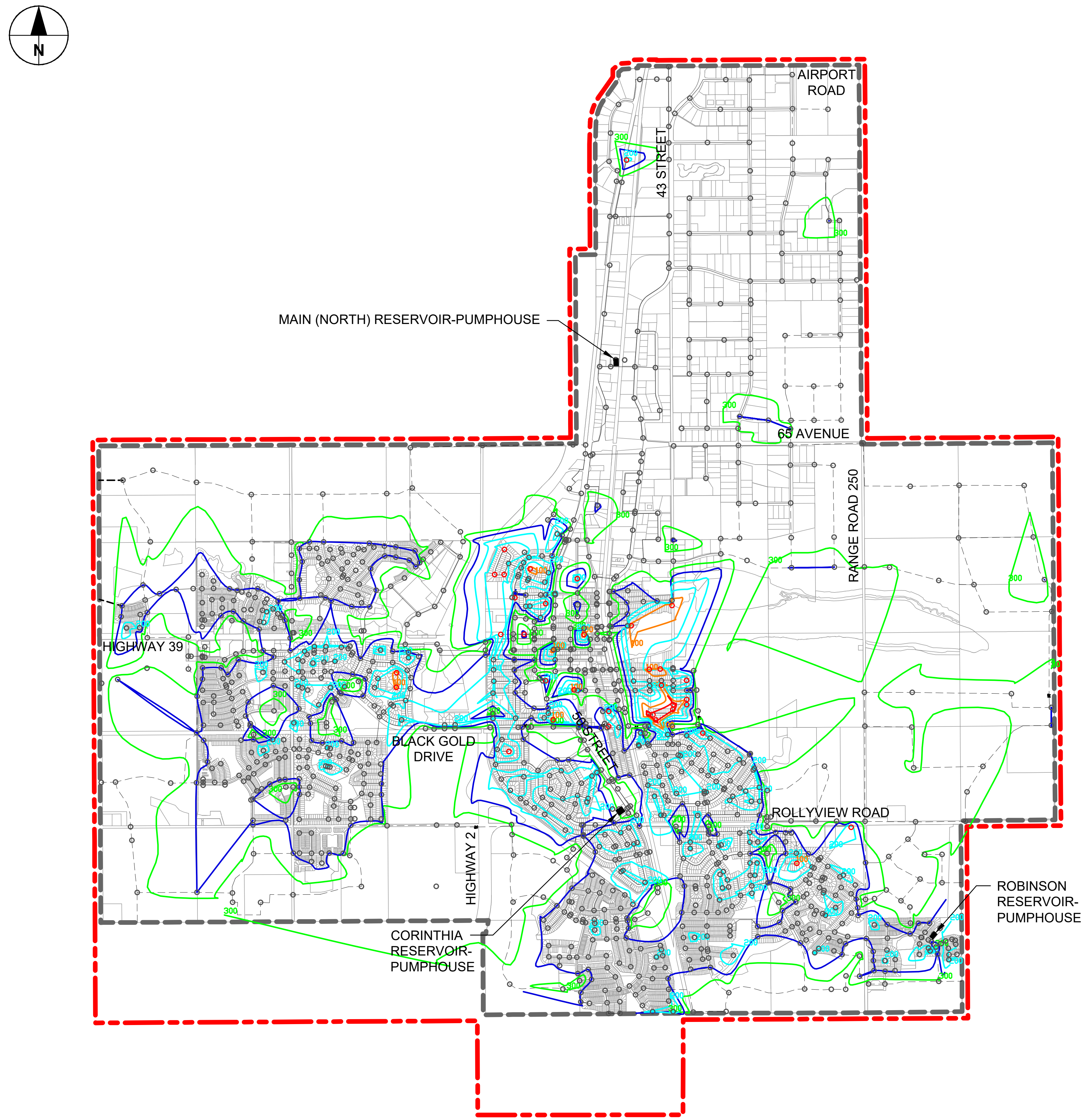
WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0418 - FUTURE SYSTEM SERVICING - STAGE 3 ASSESSMENT - AVERAGE DAY DEMAND PRESSURE.DWG

Project Management Initials: Designer: Checked: Approved:

ANSI B 279.4mm x 431.8mm



LEGEND:

- STUDY AREA BOUNDARY
- MUNICIPAL BOUNDARY
- WATER DISTRIBUTION PIPE
- DOES NOT SATISFY FIRE FIRE FLOW REQUIREMENTS
- SATISFIES FIRE FLOW REQUIREMENTS

AVAILABLE FIRE FLOW CONTOUR GRADIENT:

- < 50 L/s
- 50 - 100 L/s
- 100 - 150 L/s
- 150 - 200 L/s
- 200 - 250 L/s
- 250 - 300 L/s
- > 300 L/s

0 1000 2000 m
1:40000

AECOM

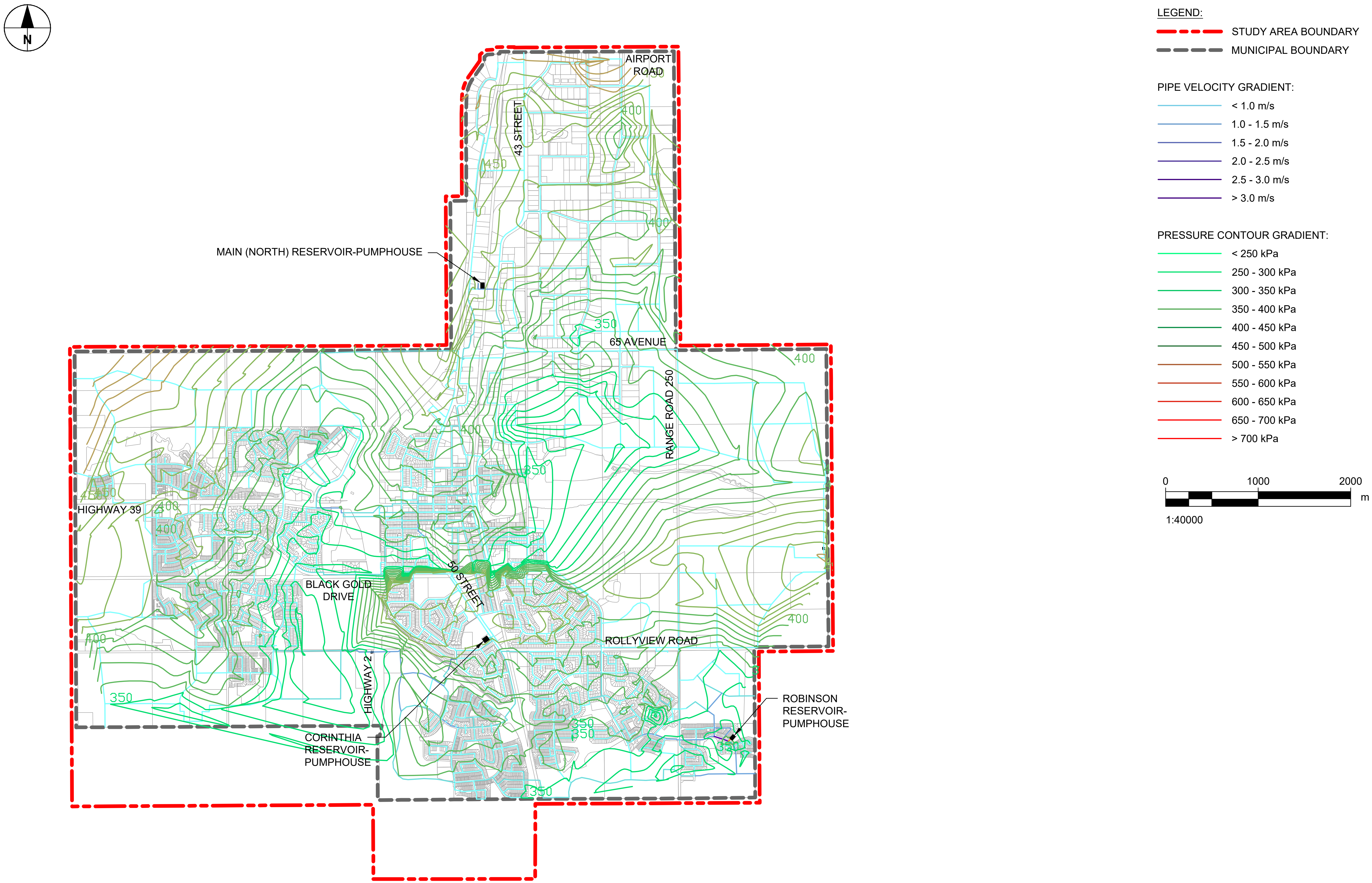
Figure: 4.19

FUTURE SYSTEM SERVICING
STAGE 3 ASSESSMENT - MAXIMUM DAY DEMAND PLUS FIRE FLOW

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0419 - FUTURE SYSTEM SERVICING - STAGE 3 ASSESSMENT - MAXIMUM DAY DEMAND PLUS FIRE FLOW.DWG
Project Management Initials: Designer: Checked: Approved:
ANSI B 279.4mm x 431.8mm



AECOM

Figure: 4.20

FUTURE SYSTEM SERVICING
STAGE 3 ASSESSMENT - PEAK HOUR DEMAND

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm

Project Management Initials: Designer: Checked: Approved:

Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0420 - FUTURE SYSTEM SERVICING - STAGE 3 ASSESSMENT - PEAK HOUR DEMAND PRESSURE.DWG

4.6.7 Stage 4 – 30+ Year Development Horizon Within Municipal Boundary

The proposed water distribution system for the Stage 4 is shown on Figure 4.21 including the additional pipes required for Stage 4 Similar to previous development stages, the network has been laid out to show water distribution mains required to supply water to future areas; however, local distribution pipes have not been included in the assessment.

4.6.7.1 Stage 4 Distribution System

Stage 4 primarily consists of a small increase in population and non-residential areas that finish the development within the municipal boundary and is only a marginal increase when compared to Stage 3. Therefore, it was assumed that all the necessary watermains, PRVs, and pump upgrades were required and constructed during Stage 3. Therefore, no additional infrastructure is proposed for the Ultimate development with the exception of additional storage required to meet the Ultimate Development.

The second expansion of the Robinson Reservoir proposed during Stage 3 is sufficient for the Stage 4 development condition. However, as noted in Section 4.6.6.2, there is additional space for an additional expansion of the Robinson reservoir of 5,500 m^3 if both the AltaGas facility and the outdoor rink are relocated, which would bring the total storage at the Robinson Reservoir to 28,500 m^3. At this time, this final expansion is not recommended; however, during subsequent Master Plan updates, depending on the rate and location of development the location of storage should be further assessed to optimize the system.

4.6.7.2 Stage 4 Hydraulic Assessment

The Stage 4 water distribution system was assessed for the ADD, MDD+Fire, and PHD demand scenarios assuming that all additional watermains required to provide servicing for the previous development stages have been completed.

To provide flows for each demand scenario, it was assumed that Robinson Pumphouse was upgraded to provide sufficient flows to the system with both distribution pumps in operation at the North Pumphouse. During pressure scenarios (ADD and PHD), at the Corinthia Reservoir, the two distribution pumps were assumed to be in operation and during MDD plus fire the fire pump was assumed to be in operation.

Tables 4.32 and 4.33 summarize the Stage 4 system hydraulic assessment for each demand scenario, and the results are shown schematically on Figures 4.22 through 4.24.

Table 4.32. Stage 4 System Evaluation – ADD and PHD

Scenario	Total Number of Nodes	Minimum Pressure	Maximum Pressure	Nodes with High Pressure		Nodes with Low Pressure	
(-)	(No.)	(kPa)	(kPa)	(No.)	(%)	(No.)	(%)
ADD	1285	318	584	3	0.2	0	0
PHD		297	497	0	0	0	0

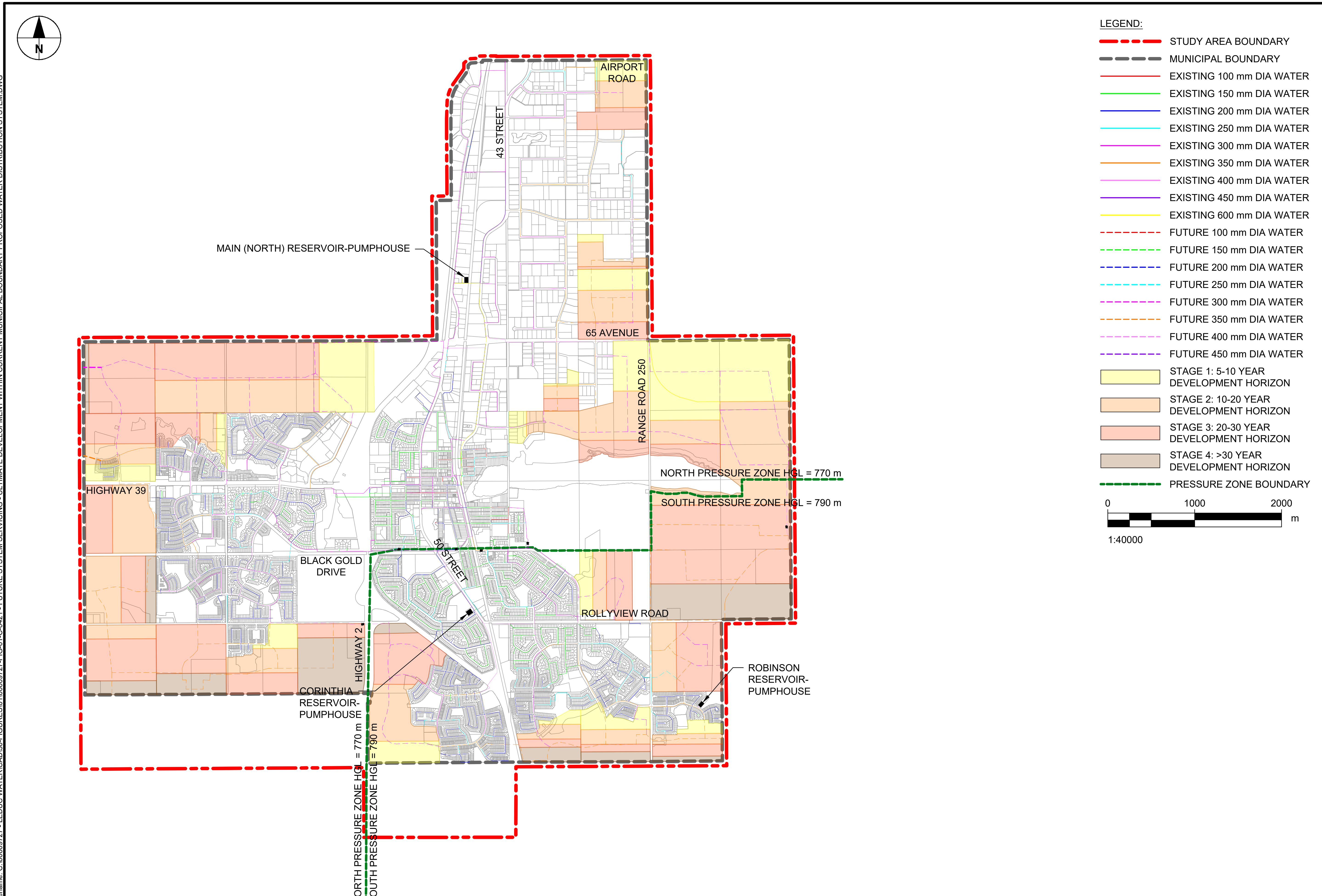
Table 4.33. Stage 4 System Evaluation – MDD+Fire

Total Number of Nodes	Residential Nodes	Non-Residential Nodes	Residential Nodes Failing Fire Flow Requirements		Non-Residential Nodes Failing Fire Flow Requirements		Total Nodes Failing Fire Flow Requirements	
(No.)	(No.)	(kPa)	(No.)	(%)	(No.)	(%)	(No.)	(%)
1285	827	458	22	2.7	24	5.2	46	3.6

As shown in Table 4.32, based on the modelling results, with the implemented pressure zones the maximum pressure within the water distribution system is 584 kPa during ADD, with minimum pressure at 297 kPa during PHD. Figures 4.22 and 4.24 show the pressure contours for the water distribution system for ADD and PHD, respectively. Similar to previous development conditions, the highest pressures are in the northwest region of the development west of the QEII, the north portion of the industrial area, as well as on the upstream side of the proposed PRVs. The lowest pressures are located on the downstream end of the PRVs.

Pipe velocity is shown on Figures 4.22 and 4.24 during ADD and PHD, respectively. The maximum velocity during ADD is approximately 0.82 m/s; during PHD the highest velocity is approximately 3.3 m/s. During PHD, the system is exceeding the maximum recommended velocity in the Robinson Reservoir 400 mm diameter header. In addition, high velocities in the existing 350 mm diameter pipe from the Robinson Reservoir along Robinson Drive is reaching approximately 2.9 m/s.

During the MDD+Fire scenario, a total of 46 nodes fail the fire flow requirement. As seen on Figure 4.23, similarly to the existing development condition the majority of the failing nodes are located within cul-de-sacs at locations with longer dead ends.



AECOM

Figure: 4.21

FUTURE DISTRIBUTION SERVICING
STAGE 4 PROPOSED WATER DISTRIBUTION SYSTEM

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0421 - FUTURE SYSTEM SERVICING - ULTIMATE DEVELOPMENT WITHIN CURRENT MUNICIPAL BOUNDARY PROPOSED WATER DISTRIBUTION SYSTEM.DWG
Project Management Initials: Designer: Checked: Approved:
ANSI B 279.4mm x 431.8mm

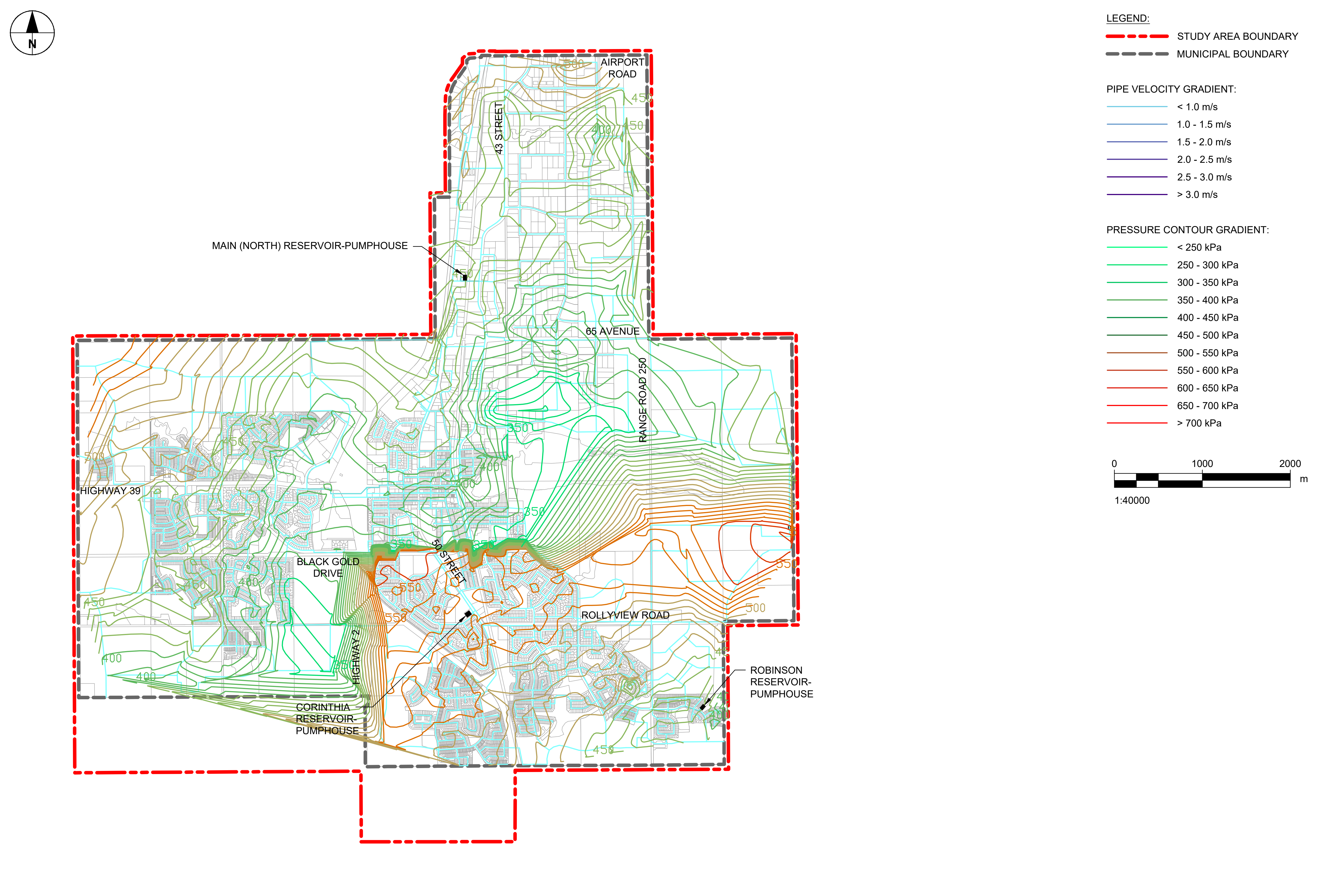


Figure: 4.22

FUTURE SYSTEM SERVICING
STAGE 4 ASSESSMENT - AVERAGE DAY DEMAND

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0422 - FUTURE SYSTEM SERVICING - ULTIMATE DEVELOPMENT WITHIN CURRENT MUNICIPAL BOUNDARY ASSESSMENT - AVERAGE DAY DEMAND PRESSURE.DWG
Project Management Initials: Designer: Checked: Approved:
ANSI B 279.4mm x 431.8mm

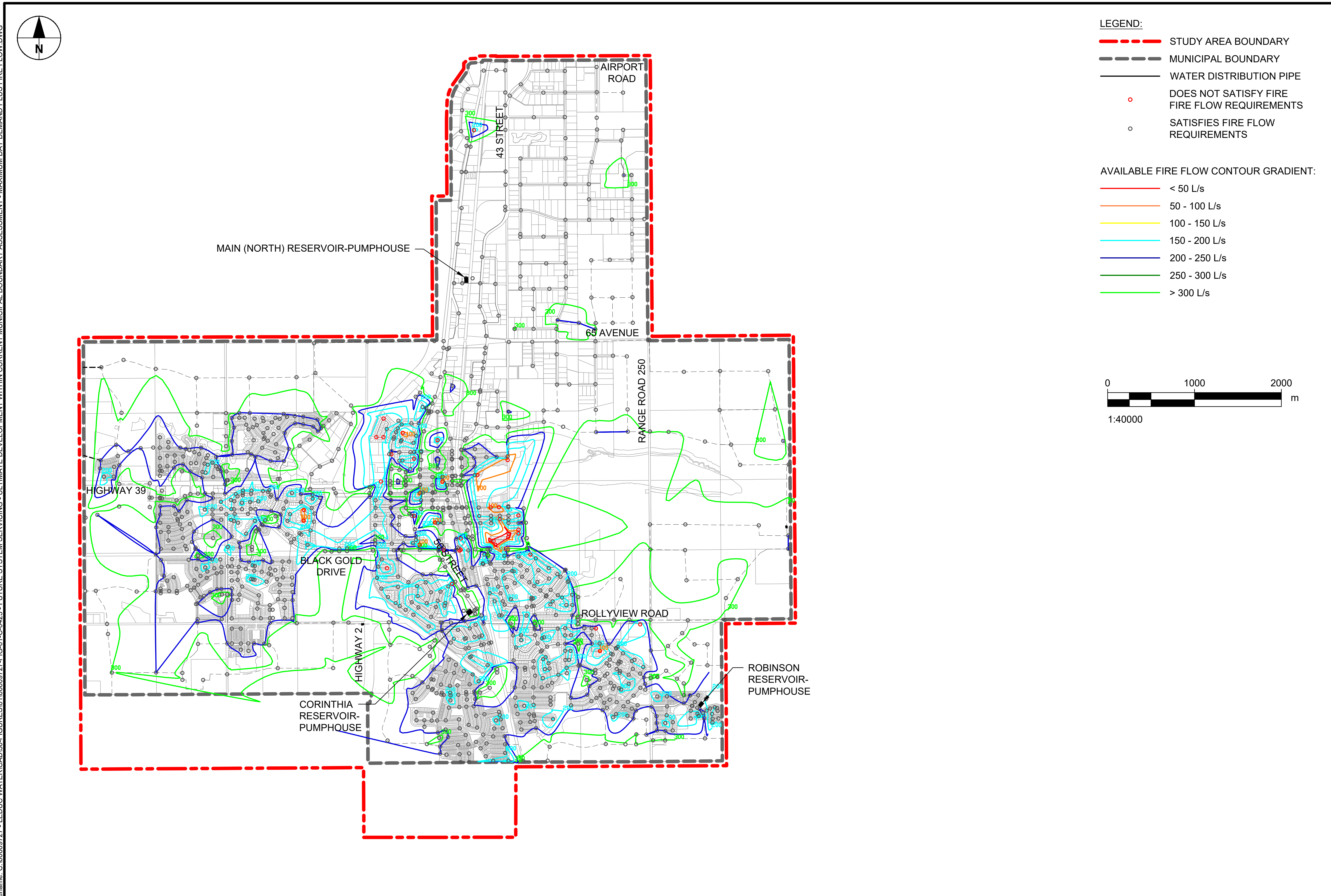


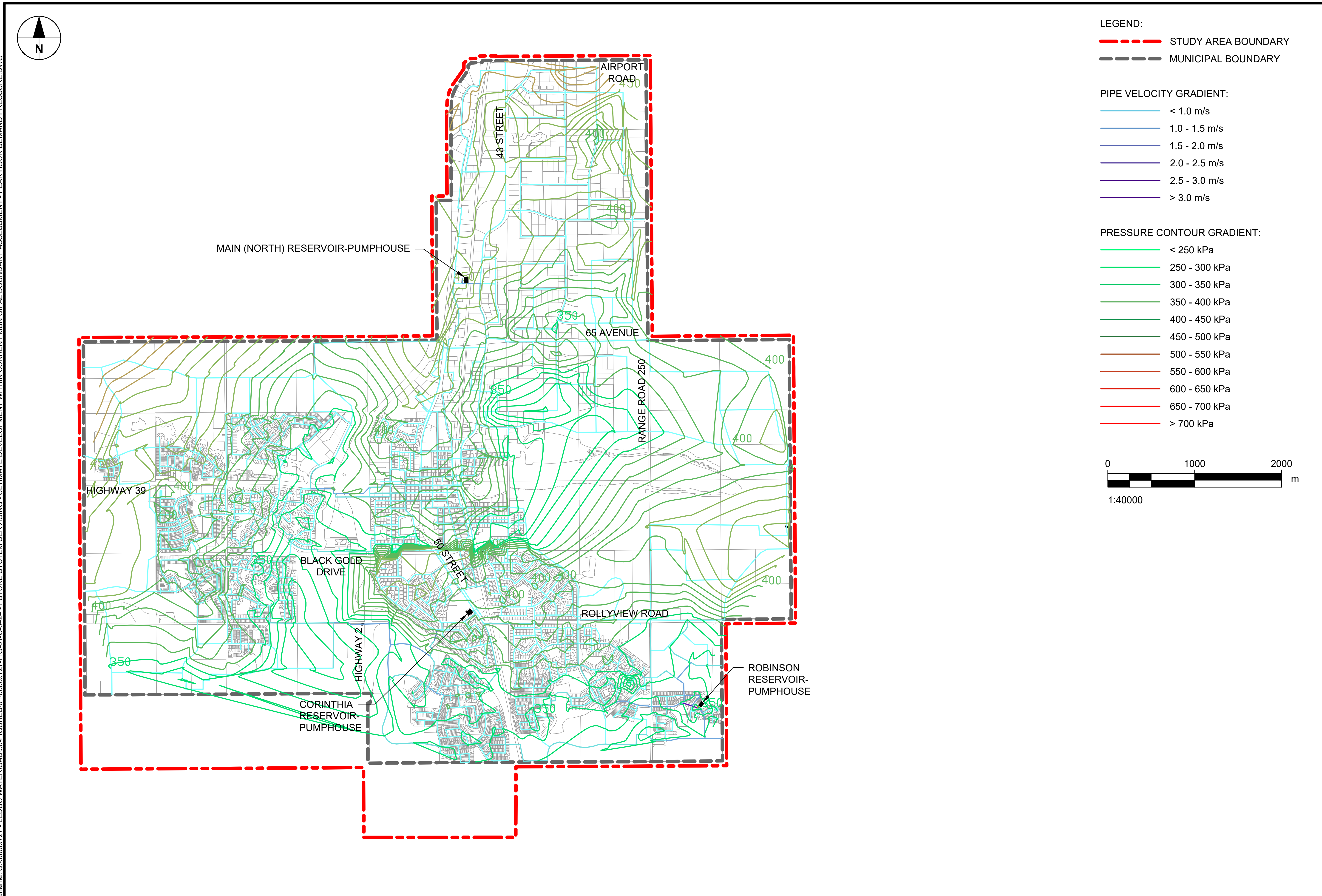
Figure: 4.23

AECOM

FUTURE SYSTEM SERVICING
STAGE 4 ASSESSMENT - MAXIMUM DAY DEMAND PLUS FIRE FLOW

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727



AECOM

Figure: 4.24

FUTURE SYSTEM SERVICING
STAGE 4 ASSESSMENT - PEAK HOUR DEMAND

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm
Project Management Initials: Designer: Checked: Approved:
Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28
Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\0160609727-FIG-01-C-0424 - FUTURE SYSTEM SERVICING - ULTIMATE DEVELOPMENT WITHIN CURRENT MUNICIPAL BOUNDARY ASSESSMENT - PEAK HOUR DEMAND PRESSURE.DWG

4.7 Stage 5 – Potential Growth Outside Current Municipal Boundary

Future growth assumptions are that growth will meet or exceed the regional growth plan requirements. There is a potential for future expansion of the Municipal Boundary for the City of Leduc in the southwest and southeast, described as the West and East Growth areas, respectively. The West Growth Area consists of approximately six quarter sections and is shown on Figure 2.2, and consists entirely of residential area. The East Growth Area is more uncertain and thus is not shown. The area assessed in the Sanitary Master Plan was considered, and was assumed to consist of approximately 66% residential and 33% industrial development.

The additional Stage 5 demand added to the system was calculated based on the design standards as discussed in Section 3, and is shown in Table 4.34, along with the total system demand for Stage 5.

Table 4.34. Stage 5 System Demand

Development Stage	ADD	MDD	PHD
-	(L/s)	(L/s)	(L/s)
Total (Stages 1-4)	320.1	562.0	924.1
Stage 5 Additional	237.4	427.3	712.2
Total (Stages 1-5)	557.5	989.3	1636.3

Based on the total system demand, the storage requirement for the Stage 5 areas was calculated and shown in Table 4.35.

Table 4.35. Stage 5 Reservoir Storage Requirement

Description	Details	Volume (m^3)
Fire Storage	227 L/s for 3 hours	2,450
Emergency Storage	Max Day Demand: 989 L/s	85,480
Total Storage Requirement – Stage 5	-	**87,930**

Assuming that the Stage 2 and 3 storage upgrades to the North and Robinson Reservoirs have been completed, the storage capacity within the current municipal boundary (Stage 4) would be 51,400 m^3. Therefore, a new reservoir would be required with a total storage capacity of 36,530 m^3 (Project ST5-2)

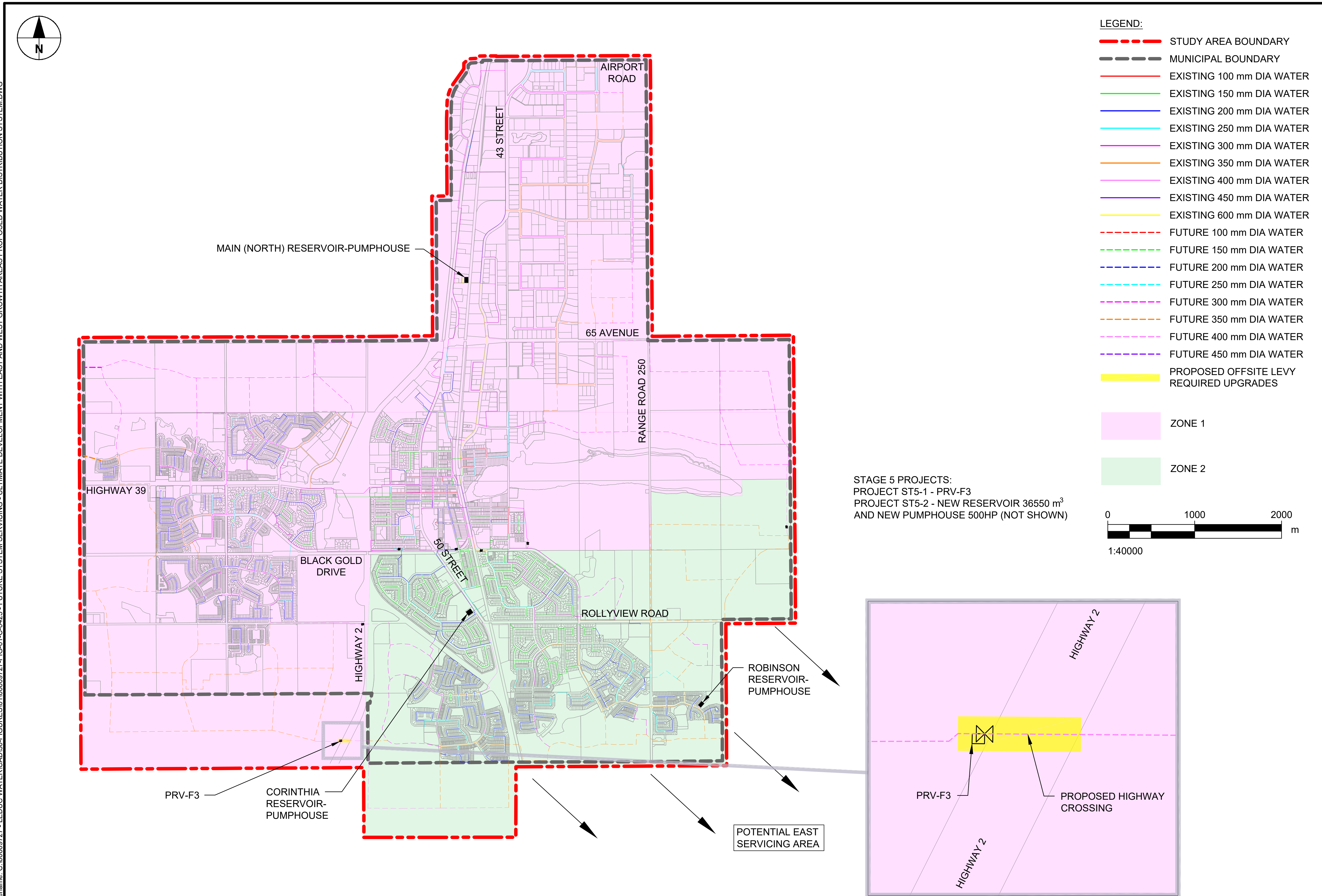
At the new reservoir, a pumphouse would be required. Assuming the pump upgrades at the Robinson Reservoir, the pumping capacity within the current municipal boundary would be a total of 1,355 L/s. Therefore, to provide sufficient pumping capacity for the Ultimate Development with Potential Growth Areas under peak hour demand, an additional 280 L/s would be required at the new pumphouse. Due to the size of the reservoir, and the requirement for redundant pumping, it is recommended to provide 400 L/s at the new pumphouse.

Figure 4.25 shows the water distribution system including the West Growth Area. Due to uncertainties and location of development of the East Growth Area, the pipe network is not shown, however, potential tie in locations for the distribution system are identified at multiple locations where potential development is anticipated. It is recommended to provide multiple tie-in points to the existing system to provide proper looping and system redundancy for emergency scenario.

The new reservoir is assumed to be located within the East Growth Area due to the ground elevation within that area compared to the current municipal boundary. In general, the ground elevation increases as development proceeds to the southeast, and the construction of a new reservoir at higher elevations reduces energy requirements when compared to pumping from low elevations to high.

The following additional infrastructure requirements would be required as development proceeds through the potential growth areas:

- One additional QEII crossing (Project ST5-1) will be required connecting the proposed network for the West Growth Area.
- One additional PRV (Project ST5-1) will be required for the new QEII crossing to maintain pressure zone separation. The HGL of the proposed PRV-F3 is anticipated be 766 m to be consistent with the pressure zone boundary at the time.
- The new reservoir will require an additional service connection to the CRSWSC transmission system. If constructed in the East Growth Area, there is the opportunity to connect to the Millet supply line. It is recommended to connect to the existing connection location near the Robinson Reservoir and upgrading/upsizing as required for the existing fill line be completed.



AECOM

Figure: 4.25

FUTURE DISTRIBUTION SERVICING
STAGE 5 PROPOSED WATER DISTRIBUTION SYSTEM

WATER MASTER PLAN UPDATE

CITY OF LEDUC
Project No.: 60609727

ANSI B 279.4mm x 431.8mm

Project Management Initials: Designer: Checked: Approved:

Filename: C:\60609727 - LEDUC WATER\CAD\30-FIGURES\01\60609727-FIG-01-C-0425 - FUTURE SYSTEM SERVICING - ULTIMATE DEVELOPMENT WITH EAST AND WEST GROWTH AREAS PROPOSED WATER DISTRIBUTION SYSTEM.DWG

Last saved by: ANDREW.FLECK(2021-10-28) Last Plotted: 2021-10-28

5. Implementation Plan

Recommendations for implementation of improvements is based on the benefit the improvement provides to the system, either by increasing fire flow or maintaining available system pressures. However, consideration should be given to other factors, such as stakeholder acceptance, including public consultation and traffic disruptions, as well as opportunities to coordinate the improvements with other capital projects.

5.1 System Improvements – Distribution System

For the existing system improvements, it is recommended that the water main upgrades be implemented in the following order:

- Project IMP-2: East Elementary School
- Project IMP-1: Civic Centre Upgrade
- Project ST1-1: Linsford Park School
- Projects ST1-2 and ST1-4: PRVs A, B, C and D and pipe closures
- Project ST1-3: Willow Park Upgrade

The first two water main upgrades (IMP-1 and IMP-2) are recommended to be implemented as soon as practical to improve available fire flows to the East Elementary School and the Civic Centre. The Civic Centre water main upgrade is recommended to be completed as part of the other planned upgrades along 48a Street.

The implementation of the pressure zone boundary is recommended for the Stage 1 (5-10 year) development. When new developments occur at a ground elevation of 713.5 m or lower north of 65 Avenue on the west half of the City, the distribution pressures will start exceeding 700 kPa. It is recommended that PRVs A, B, C, and D be implemented at this time.

The Linsford Park School (ST1-1) and Willow Park water main upgrades (ST1-3) are recommended to be implemented in the Stage 1 development scenario. Project ST1-1 is recommended to be completed with PRV-A, as implementation of the PRV will reduce available fire flows to the school. When the pressure zone boundary is implemented, the available fire flows in Willow Park are reduced. This upgrade can be completed in conjunction with other renewal projects in the area.

5.2 System Expansions – Distribution System

As development progresses, new water mains will be implemented to service the new development areas, ranging in diameter from 200 mm to 400 mm.

To service future areas west of the QEII, it is recommended that additional highway crossings be added as development progresses. A 300 mm diameter water main crossing to service the 65 Avenue development areas is proposed in Stage 2 development (Project ST2-4).

A 400 mm diameter PRV is also required in Stage 2 development when the future water main loop around the east half of Telford Lake is implemented (Project ST2-3). The water main loop around the east half of Telford Lake is recommended to provide fire flows to the development areas south of Telford Lake within the Telford Lake ASP area.

A 400 mm QEII crossing at 38 Avenue complete with a 400 mm PRV is proposed in Stage 3 development to support development in the Blackstone and Brightwell ASP areas. The PRV is required to maintain the pressure zone boundary (Project ST3-1).

A 350 mm crossing of Hwy 2A at Stutz Link is proposed for Stage 3 development as well, to support development in the Southfork ASP area (Project ST3-2). However, depending on the sequencing of development, this crossing may be required sooner if looping is not provided east through the Robinson neighbourhood.

5.3 System Expansions – Reservoirs/Pumphouse

Based on the existing pumps and reservoirs, the limiting factor is reservoir capacity, not pumping capacity. As described in Section 4.6, reservoir and pumphouse upgrades will be required to service Stage 2 and 3 development, and with additional reservoir expansions required to service the Ultimate development areas within the municipal boundary. Potential growth areas outside the municipal boundary will require an additional reservoir-pumphouse.

A summary of the recommended storage and pumping expansions are provided in Table 5.1.

Table 5.1. Reservoir and Pumphouse Upgrade Triggers

	Upgrade Size	ADD Trigger (L/s)	Development Stage
Robinson Reservoir Expansion 1 (ST2-1)	4500 m^3	144	Stage 2
North Reservoir Expansion (ST2-2)	8000 m^3	173	Stage 2
Robinson Reservoir Expansion 2 (ST3-3) and Pump Upgrades (ST3-4)	14,000 m^3 200 HP	224	Stage 3
New Reservoir & Pumphouse (ST5-2)	36,530 m^3 500 HP	350	Stage 5

To provide the required storage for Stage 3, a second expansion to the Robinson Reservoir has been recommended. However, the location of storage for Stage 3 should be re-evaluated as development progresses to determine the most appropriate location, as there is also room for expansion at the Corinthia Reservoir.

The new reservoir and pumphouse required for Stage 5 will most likely be built in stages; however, due to the uncertainty in the location and timing of development and long growth horizon (over 25 years), the total volume is shown.

6. Cost Estimates

The following sections detail the costs for improvements for the Stage 1, Stage 2, Stage 3 and Ultimate development stages. Costs were developed in 2021 construction dollars; a detailed breakdown is provided in Appendix B.

6.1 Funding Mechanisms

For each of the proposed system improvements and expansions, the associated payment method has been indicated. It is assumed that funding will come from one of three sources:

- City: Improvements required to benefit the existing system to be paid from the City's capital budget.
- Off-Site Levies: Future system expansions, including reservoir upgrades, pump upgrades, pressure reducing valves, as well as highway crossings and associated connections back to the existing system will be paid through off-site levies.
- Developer: Watermain expansions required to service future development will be paid by developers.

6.2 Cost Estimates - Distribution System

The total costs for the proposed water main improvements are summarized in Table 6.1. Costs for existing system improvements include pipe supply, installation, and road restoration costs, and include a 30% contingency.

Table 6.1. Cost Estimate Summary: Distribution System Improvements

Upgrade	Location	Description	Cost ($)	Payment Method
Existing System Improvements				
East Elementary School (IMP-2)	46 Avenue, 45 Street to E of 43 Street	250 mm Watermain Upgrade	$786,000.00	City
Civic Centre Upgrade (IMP-1)	48a Street, Black Gold Drive to 47 Avenue	200 mm Watermain Upgrade	$313,000.00	City
Stage 1 Improvement				
Linsford Park School (ST1-1)	Black Gold Drive to 41 Avenue & 51 Street	250 mm Watermain Upgrade	$744,000.00	City
Willow Park Upgrade (ST1-3)	56 Avenue, 52 Street to 53 Street	200 mm Watermain Upgrade	$145,000.00	City
PRV A (ST1-2)	Black Gold Drive W of 51 Street	300 mm PRV and 250 mm Watermain	$577,600.00	Off-Site Levies
PRV B (ST1-2)	Black Gold Drive at 48 Street	400 mm PRV and 200 mm Watermain	$869,500.00	Off-Site Levies
PRV C (ST1-2)	Black Gold Drive E of 46 Street	350 mm PRV	$477,300.00	Off-Site Levies
PRV D (ST1-2)	Black Gold Drive at 43 Street	300 mm PRV	$438,500.00	Off-Site Levies

Costs for system expansion, summarized in Table 6.2, include pipe supply and installation and include a slightly higher, 40% contingency. Costs for highway crossings include casing pipes and considered trenchless installation methods.

Table 6.2. Cost Estimate Summary: Distribution System Expansion

Upgrade (Project #)	Description	Cost ($)	Payment Method
Stage 2			
300 mm Watermain (ST2-4)	Highway Crossing – QEII at 65 Avenue	$2,126,000.00	Off-Site Levies
PRV - F1 (ST2-3)	400 mm PRV, East of Telford Lake	$490,000.00	Off-Site Levies
Stage 3			
350 mm Watermain (ST3-2)	Highway Crossing – Hwy 2A at Sturtz Link	$686,000.00	Off-Site Levies
400 mm Watermain (ST3-1)	Highway Crossing - QEII at 38 Avenue	$1,557,000.00	Off-Site Levies
PRV - F2 (ST3-1)	400 mm PRV, West of QEII at 38 Avenue	$490,000.00	Off-Site Levies
Stage 5			
400 mm Watermain (ST5-1)	Highway Crossing – QEII at South Municipal Boundary	$1,462,000.00	Off-Site Levies
PRV – F3 (ST5-1)	400 mm PRV – West of QEII at South Municipal Boundary	$490,000.00	Off-Site Levies

6.3 Cost Estimates - Reservoir – Pumphouses

The total costs for the proposed Reservoir and Pumphouse expansions are summarized in Table 6.3. Costs for the expansions have been developed to reflect expansion versus new reservoir costs, and include a 40% contingency.

Table 6.3. Cost Estimate Summary: Reservoir – Pumphouse Expansions

Upgrade (Project #)	Description	Cost ($)	Payment Method
Stage 2			
Robinson Reservoir Expansion 1 (ST2-1)	4500 m^3 Storage Expansion	$4,095,000.00	Off-Site Levies
North Reservoir Expansion (ST2-2)	8000 m^3 Storage Expansion	$7,280,000.00	Off-Site Levies
Stage 3			
Robinson Reservoir & Pumphouse Expansion 2 (ST3-3, ST3-4)	14,000 m^3 Storage Expansion 200 HP	$19,110,000.00 $3,780,000.00	Off-Site Levies
Stage 5			
New Reservoir & Pumphouse (ST5-2)	36,550 m^3 500 HP	$33,220,000.00 $6,300,000.00	Off-Site Levies

Robinson Reservoir Expansion 2 is projected to be required when the average day demand in the City approaches 224 L/s. As long term development rates are somewhat aggressive, and consumption rates are decreasing over time, it is likely that Expansion 2 will not be required until the end of Stage 3, which is past the 25-year horizon considered for off-site levies. It is recommended that the timing for the expansion be reviewed in the next Water Master Plan and included in the levies at that time if appropriate.

7. Conclusions and Recommendations

The City of Leduc has retained AECOM Canada Ltd. to complete an update to the Water Master Plan. The City of Leduc receives potable water from EPCOR Water, supplied by the Capital Region Southwest Water Services Commission (CRSWSC). The City's water distribution system includes three reservoirs and pumphouses. Two of the reservoirs are directly supplied by the CRSWSC and the third is filled off the distribution system. The City's distribution network is currently located within one pressure zone and consists of over 175 km of water mains.

The City of Leduc Water Master Plan was last updated in 2014. Since the completion of the 2014 Water Master Plan, the City has experienced significant development, as well as construction of a new reservoir and pumphouse.

This report provides an update to the Water Master Plan. The overall goal of the assessment is to update and assess the existing system with recently completed projects and provide a road map for future development.

The work completed as part of this study includes the following:

- A water demand analysis to determine consumption rates and peaking factors.
- Update and verify the previously developed water distribution system model.
- Develop a uni-directional flushing program.
- Evaluate the existing water supply and distribution system and identify deficiencies.
- Review of emergency scenarios such as single reservoir operation, QEII crossing closures, and EIA and Leduc County connection viability.
- Develop and evaluate servicing concepts for system expansion.
- Provide an implementation plan for proposed system improvements.
- Develop cost estimates for system improvements and future infrastructure requirements.

7.1 Design Criteria

The existing land use and water consumption was assessed to determine the water demand on the water distribution system and peaking factors for maximum day demand and peak hour demand. Table 7.1 summarizes water consumption rates and peaking factors for the City based for the existing development condition.

Table 7.1. City of Leduc Average Day Demand and Peaking Factors

Average Day			Maximum Day		Peak Hour	
Demand (m^3/year)	Demand (m^3/day)	Demand (L/s)	Demand (L/s)	Peak Factor	Demand (L/s)	Peak Factor
2,730,744	7,481	86.6	141.0	1.63	222.1	2.56

Utilizing water consumption records, the historical demand was split between residential and non-residential land uses. In general, the existing water consumption rates were found to be lower than recommended for design standards, and the historical consumption rates were used for the existing system assessment. Design standards for future development have been recommended and summarized in Table 7.2.

Table 7.2. Recommended Design Standards Summary

Criteria	Unit	Value	Referenced Standard
Minimum Peak Hour Pressure	kPa	280	EPCOR
Minimum Maximum Day + Fire Pressure	kPa	140	EPCOR
Minimum Maximum Day Pressure (Fire Sprinklers)	kPa	350	EPCOR
Maximum Pressure (Distribution System)	kPa	570	Leduc
Maximum Allowable Pressure (for Services)	kPa	570	Leduc
Average Day Demand (ADD) - Residential	L/c/d	250	Leduc
Average Day Demand (ADD) – Commercial	L/Ha/d	22,500	Leduc
Average Day Demand (ADD) – Industrial/Institutional	L/Ha/d	11,000	Leduc
Maximum Day Demand (MDD) Peaking Factor	-	1.8	Leduc
Peak Hour Demand (PHD) Peaking Factor	-	3.0	EPCOR
Maximum Hazen-William's Coefficient	-	120	EPCOR
Fire Flow - Single Family	L/s	115	Leduc
Fire Flow - Mid-Value Multi-Family	L/s	227	Leduc
Fire Flow - High Value Multi-Family	L/s	227	Leduc
Fire Flow - High Value Properties/Non-Res	L/s	227	Leduc

7.2 Existing System

The following is a summary of the conclusions of the existing water distribution system:

- The water supply system has sufficient capacity for the existing development condition. Currently, the system has two supply lines from the CRSWSC. A 600 mm diameter connection to the North Reservoir and a 450 mm diameter connection to the Robinson Reservoir. The Corinthia Reservoir fills off of the distribution system.
- The existing storage capacity is 24,900 m^3, which is sufficient for the existing development storage requirement of 14,212 m^3.
- The existing pumping capacity is 1,240.5 L/s. With the largest pump at the north reservoir (227 L/s) out of service, the capacity is 1,013.5 L/s. The existing pumping capacity is sufficient for the existing development condition, which is governed by the maximum day plus fire demand scenario at 363 L/s.
- The existing water distribution system in general is sufficient to provide adequate fire flows to the system with the following exceptions:
  - There are a number of dead ends within cul-de-sacs which can be opportunistically upgraded with neighborhood improvements. However, in general the adjacent watermains have sufficient fire flows and thus immediate improvements are not recommended.
  - The Linsford Park school, the Civic Centre, and the South Telford residential area. Pipe upgrades are proposed in these areas to increase the available fire flow.
- The system pressure during ADD and PHD scenarios exceeds the maximum allowable pressure of 570 kPa. Pressures within the existing system are high in the northwest portion of the development west of the QEII, as well as in the northeast portion of the northern industrial area. Pressures within these areas reach up to 674 kPa.
- Pressure zones are proposed utilizing PRVs to create a boundary at Black Gold Drive to reduce the pressures within the system. However, the implementation of the pressure zone boundary has been deferred to the Stage 1 development horizon.
- Each reservoir has the pumping capacity to supply the entire system when operating alone. The North and Robinson Reservoirs can fill from the CRSWSC while operating and thus storage capacity is not an issue. The Corinthia Reservoir cannot supply and fill at the same time, and will run out of storage when operating at ADD in approximately 8 hours.

- If one of the existing QEII crossings is out of service, a single operational crossing is sufficient to provide flows to the area west of the QEII without a significant drop in pressure. As demand increases, the velocity in a single operational crossing will increase. Therefore, additional QEII crossings are proposed in future development scenarios to provide redundancy as well as additional available fire flow.
- A potential connection to the EIA water distribution system is feasible and could benefit both parties in an emergency scenario requiring one system to supply the other.
- The existing connections to the Leduc County water distribution system is feasible to provide servicing during emergency scenarios. However, due to the lower elevation of Leduc County, the system pressure in the south areas of the City would drop to 200 kPa during ADD and 50 kPa during PHD.
- Caution should be used when supplying the County from the City of Leduc without additional infrastructure (PRVs) to lower the hydraulic grade line for flows provided to the County.

7.3 Future System

The following is a summary of the conclusions of the Future System and assessment:

- In general, the existing water distribution system pipes are sufficient to provide servicing for expansion without any major upgrades within the existing development to support future development.
- Any system improvements required within the existing development area would be implemented opportunistically to improve local fire flows.
- Proposed watermains for future development are proposed to provide servicing to future development areas. For the purposes of this study, watermains have been sized and assessed, but local distribution pipes have not been assessed for future development areas.
- The Future System was assessed in five development horizons as follows:
  - Stage 1: 5-10 Year Development Horizon
  - Stage 2: 10-20 Year Development Horizon
  - Stage 3: 20-30 Year Development Horizon
  - Stage 4 – 30+ Year Development Horizon within Municipal Boundary
  - Stage 5 – Potential Growth Outside the Current Municipal Boundary
- Throughout the future development horizons, additional, piping, PRV, storage, and pumping requirements were determined and an implementation plan for future water distribution infrastructure was developed. The implementation plan is summarized as follows:
  - System improvements for the Civic Centre, and the East Elementary School should be completed as soon as practical to improve local fire flows.
  - Stage 1 Infrastructure Requirements:
    - Linsford Park Upgrade to be completed during Stage 1 in conjunction with PRV A installation.
    - Pressure Zone Implementation including the construction of PRVs A through D.
    - The Willow Park upgrade.
    - Local water distribution pipes should be installed as required to provide servicing for the development as it progresses.
  - Stage 2 Infrastructure Requirements:
    - An additional PRV (PRV-F1) on the east edge of Telford Lake.
    - Expansion of the Robinson and the North Reservoirs by 4,500 m^3 and 8,000 m^3, respectively. The timing and demand triggers for the reservoir expansion is summarized in Table 7.3.
    - An additional QEII crossing at 65 Avenue.
    - Local distribution pipes as required for development.
  - Stage 3 Infrastructure Requirements:
    - An additional PRV (PRV-F2) and QEII crossing at 38 Ave.
    - A Hwy 2A crossing at between Southfork and Tribute.
    - An additional expansion of the Robinson Reservoir of 14,000 m^3.

  - Upgrades of the existing distribution pumps at the Robinson Reservoir with associated pumphouse improvements required to accommodate the increased pumping.
  - Local distribution pipes as required for development
- Stage 4 Infrastructure Requirements
  - Local distribution pipes as required for development.
- Stage 5 Infrastructure Requirements
  - An additional PRV (PRV-F3) and QEII crossing south of current municipal boundary.
  - A new reservoir with total capacity of 36,550 m^3.
  - A new pumphouse at the new reservoir with 500 HP pumping capacity.
  - A new supply line from the new reservoir to an existing tie-in location
  - Local distribution pipes as required for development.

- Table 7.3 summarizes the demand triggers and expansion volumes for the proposed reservoir expansion.
- Table 7.4 summarizes the proposed PRV settings for the proposed pressure zones.

Table 7.3. Reservoir and Pumphouse Upgrade Triggers

Upgrade (Project #)	Upgrade Size	ADD Trigger (L/s)	Development Stage
Robinson Reservoir Expansion 1 (ST2-1)	4500 m^3	144	Stage 2
North Reservoir Expansion (ST2-2)	8000 m^3	173	Stage 2
Robinson Reservoir Expansion 2 and Pump Upgrades (ST3-3, ST3-4)	14,000 m^3 200 HP	224	Stage 3
New Reservoir & Pumphouse (ST5-2)	36,550 m^3 500 HP	350	Stage 5

Table 7.4. Reservoir and PRV Pressure Settings Summary

PRV	Development Stage	HGL Elevation (m)	Ground Elevation (m)	Pressure Setting (kPa)	Pressure Setting (psi)
North Reservoir	Existing	770	726	432	62.7
Corinthia Reservoir	Existing	790	735	540	78.3
Robinson Reservoir	Existing	790	747	422	61.2
PRV-A	Stage 1	766	732	334	48.4
PRV-B	Stage 1	768	732.5	348	50.5
PRV-C	Stage 1	768	734.25	331	48.0
PRV-D	Stage 1	768	736.5	309	44.8
PRV-F1	Stage 2	768	730.5	368	53.4
PRV-F2	Stage 3	766	732	334	48.4
PRV-F3	Stage 5	766	TBD	TBD	TBD

7.4 Cost Estimates

Cost estimates have been developed for the proposed system improvements and expansions and are detailed in Section 6. For each of the proposed system improvements and expansions, the associated payment method has been indicated. It is assumed that funding will come from one of three sources:

- City: Improvements required to benefit the existing system to be paid from the City's capital budget.
- Off-Site Levies: Future system expansions, including reservoir upgrades, pump upgrades, pressure reducing valves, as well as highway crossing and associated connections back to the existing system will be paid through off-site levies.

A cost summary is provided in Table 7.5. Existing system improvement include 30% contingency, and all other costs include 40% contingency.

Table 7.5. Cost Summary

Upgrade	Estimated Cost (million $)						Payment Method
	Existing	Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	
Existing System Improvement	$1.10M	$0.89M	-	-	-	-	City
Pressure Reducing Valves	-	$2.36M	$0.5M	$0.5M	-	$0.5M	Off-Site Levies
Water Mains – Highway Crossings			$2.1M	$2.2M	-	$1.5M	Off-Site Levies
Reservoir/Pumphouse Expansion			$11.4M	$22.9M	-	$39.5M	Off-Site Levies

AECOM Imagine it. Delivered.

Appendix A

Hydrant Testing Report, SFE Global June 2020

Final Report for
AECOM

Attn: **Sean Frank, P.Eng.**

Water Resources Engineer, Water

Leduc, Alberta

Fire Hydrant Flow Testing

June 2020

Prepared and submitted by:

SFE Global

10707 - 181th Street

Edmonton, Alberta T5S 1N3

Phone (780) 461-0171 Fax (780) 443-4613

Toll Free: 1-877-293-0173

Alberta Head Office
10707-181 Street
Edmonton, Alberta T5S 1N3
Ph (780) 461-0171 *Fx* (780) 443-4613

British Columbia Head Office
#201 – 26641 Fraser Hwy
Aldergrove, British Columbia V4W 3L1
Ph (604) 856-2220 *Fx* (604) 856-3003

June 17, 2020

Sean Frank, P.Eng.
Water Resources Engineer, Water

AECOM
101, 18817 Stony Plain Road NW
Edmonton, Alberta
T5S 0C2

FINAL REPORT: 2020 Fire Hydrant Flow Testing, Leduc, Alberta

Dear Mr. Frank;

Please find enclosed SFE's Final Report for the above mentioned project. If you have any questions, comments or concerns, please do not hesitate to contact us at your earliest convenience.

Thank you for having SFE conduct this work on your behalf. We are appreciative of the opportunity to work with you and your team on this project. We look forward to working together again in the near future.

Sincerely,
SFE Global

Kevin McMillan
Vice President
(780) 461-0171
Kevin.McMillan@sfeglobal.com
www.sfeglobal.com

1. Executive Summary

This report provides details of the hydrant fire flow testing conducted in Leduc, Alberta. SFE Global was retained by AECOM under the direction of Mr. Sean Frank, P.Eng.. Kevin McMillan represented SFE Global as Project Manager during this project.

As requested, SFE conducted ten fire hydrant fire flow tests on June 8th, 2020. The flow hydrants and test hydrants were indicated to SFE by maps supplied by the client. The fire flow tests were conducted according to National Fire Protection Association (NFPA) 291 standards.

2. Summary of Testing

SFE Technicians met representatives of AECOM and the City of Leduc on-site to perform testing. The testing plan was discussed and location maps reviewed by all participants.

Testing Equipment

Testing was performed on flow hydrants utilizing a Hydro Flow Products 4-inch Hose Monster system with integral de-chlorinator. These are fixed pitot devices to measure flow, de-chlorinate and diffuse in one process. The benefit of this system is the ability to provide repeatable results and manage discharge water conditions.

The configuration for the Hose Monster System consisted of one four-inch hose monster on the Flow hydrant pumper port. To digitally log pressure on the residual hydrant SFE Technicians installed one (1) Telog HPR hydrant pressure logger. This device was set to ten second logging intervals and one second sampling intervals. Each interval logs the minimum, maximum and average pressure for that time stamp.

Testing Procedure

The client selected all flow and residual hydrants for each test. SFE Technicians installed flow testing equipment on each flow hydrant and residual pressure measurement equipment on the residual hydrant.

The tests were performed by recording system static pressure then flowing the four-inch port on the flow hydrant until fire pumps activated and flow and pressure stabilized. Residual and pitot(flow) pressures were then obtained. Upon closure of the flow hydrant, static pressure was obtained to determine actual fire pump static pressure. Total flow and extrapolated flow to 20 psi residual pressure are calculated with all pumps running and using fire pump static pressure.

Flow testing summary sheets are included in Appendix I.

3. Data

The testing reports included in Appendix I contain all test results and photos. All pressure readings are in psi and all flow values are reported in IGPM. All hydrants were returned to as found condition upon completion of testing.

4. Safety

A pre-job safety inspection and meeting was conducted by SFE personnel, and the following potential hazards were identified:

- Need for Personal Protective Equipment
- Working with water under pressure
- Pedestrian and vehicular traffic conditions
- Safe operation and shut down of fire hydrants

This project was conducted in accordance with the WCB and OSHA safety standards as documented in SFE's Safety Procedures Manual. The SFE crew reviewed the work to be completed and safety requirements at a tail-gate safety meeting held prior to commencing work.

Report End
June 2020

SFE Global
Project A19-029

Appendix I

Test Results

Final Report

Fire Flow Test Report

Client Name:	AECOM	Hyd 1 - #/Port Size	4 inch	SR#1 Hyd Addr.	82nd Ave - Vallard Const.
Project Location:	Leduc, Alberta	Hyd 2 - #/Port Size		SR#2 Hyd Addr.	13 Bamber Pl.
SFE Project #:	A19-029	Hyd 1 - Pito Types	4 inch HM	SR#3 Hyd Addr.	131 Southfork Dr.
SFE Technicians:	KM / AS	Hyd 2 - Pito Types		Fire Pump Status	Auto
		Test Procedure	NFPA 291	(circle one)	Force On

Test ID: System Residual Test : of Date: 8-Jun-20

		Flow Hyd 1		Flow Hyd 2		Residual Hydrant		
Start Time	End Time	Port 1-1 psi	Port 1-2 psi	Port 2-1 psi	Port 2-2 psi	Static psi	Residual psi	Static psi

Flow Summary (igpm)	
Flow 1-1	
Flow 1-2	
Flow 2-1	
Flow 2-2	
Total Flow	
Flow @ 20 psi	

Notes: Note: Some data loss at SR#2 between 12:00 to 15:00 hrs.

SR#1

GPS 53.3056 -113.5342

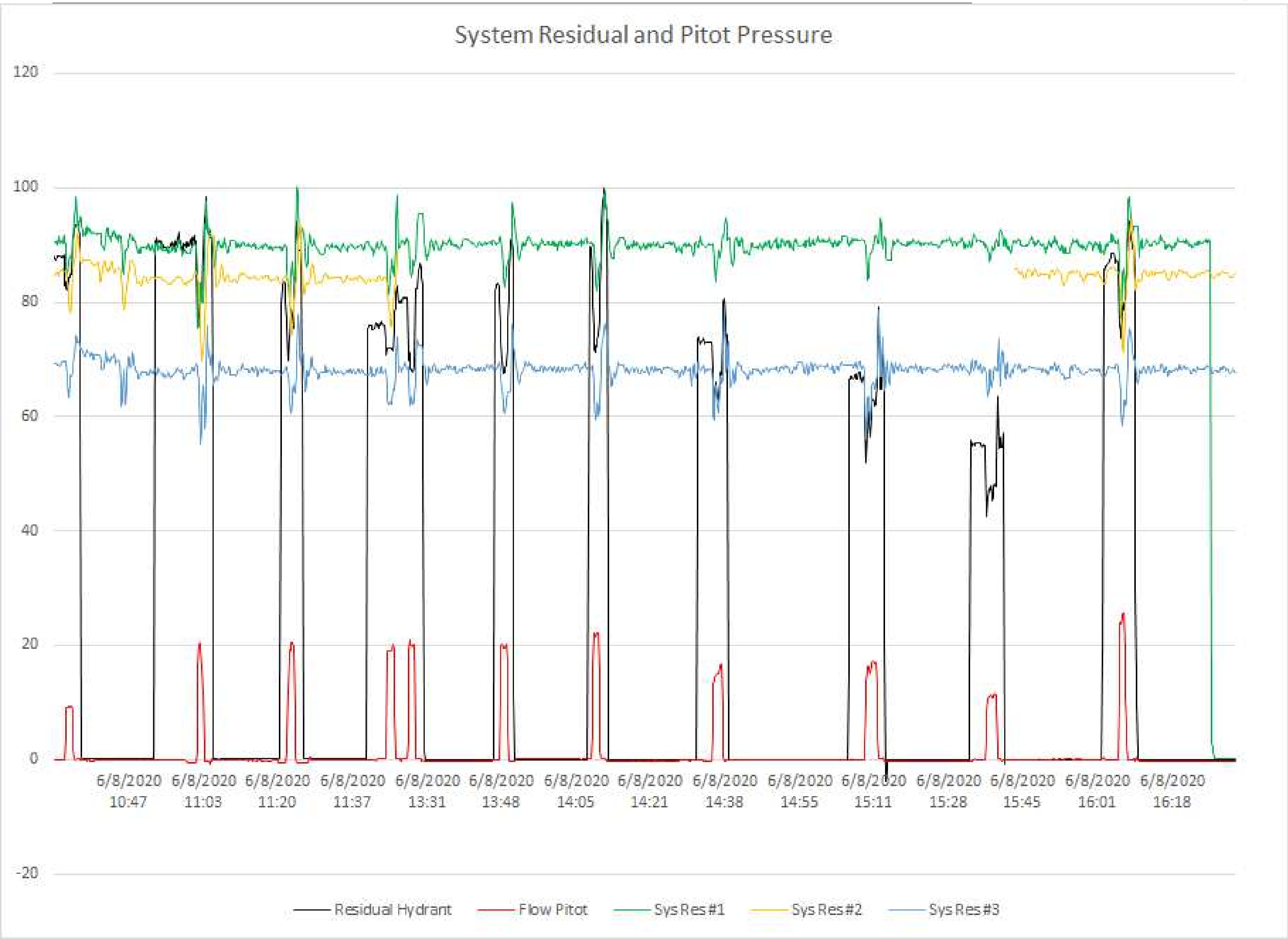
SR#2

GPS 53.2638 -113.5783

SR#3

GPS 53.2425 -113.5421

Final Report



Final Report

Fire Flow Test Report

Client Name:	AECOM	Hyd 1 - #/Port Size	4 inch	Flow Hyd 1 Addr	LBP11 - 3904 82nd Ave
Project Location:	Leduc, Alberta	Hyd 2 - #/Port Size		Flow Hyd 2 Addr	
SFE Project #:	A19-029	Hyd 1 - Pito Types	4 inch HM	Resid Hyd Addr.	LBP10 - 3908 82nd Ave
SFE Technicians:	KM / AS	Hyd 2 - Pito Types		Fire Pump Status	Auto
		Test Procedure	NFPA 291	(circle one)	Force On

Test ID: T-1 - H#LBP11 Test : 2 of 10 Date: 8-Jun-20

		Flow Hyd 1		Flow Hyd 2		Residual Hydrant		
Start Time	End Time	Port 1-1 psi	Port 1-2 psi	Port 2-1 psi	Port 2-2 psi	Static psi	Residual psi	Static psi
11:02	11:05	15				98	85	98

Flow Summary (igpm)	
Flow 1-1	1213
Flow 1-2	
Flow 2-1	
Flow 2-2	
Total Flow	1213
Flow @ 20 psi	3192

Notes: Pre-firepump static pressure: 92

Flow Hydrant 1

GPS 53.3051 -113.529

Flow Hydrant 2

GPS

Residual Hydrant

GPS 53.3052 -113.5305

Final Report

Fire Flow Test Report

Client Name:	AECOM	Hyd 1 - #/Port Size	4 inch	Flow Hyd 1 Addr	El-15 - 5905 45th St.
Project Location:	Leduc, Alberta	Hyd 2 - #/Port Size		Flow Hyd 2 Addr	
SFE Project #:	A19-029	Hyd 1 - Pito Types	4 inch HM	Resid Hyd Addr.	El14 - 5611 45th St.
SFE Technicians:	KM / AS	Hyd 2 - Pito Types		Fire Pump Status	Auto
		Test Procedure	NFPA 291	(circle one)	Force On

Test ID: T-2 - H#El15 Test : 4 of 10 Date: 8-Jun-20

		Flow Hyd 1		Flow Hyd 2		Residual Hydrant		
Start Time	End Time	Port 1-1 psi	Port 1-2 psi	Port 2-1 psi	Port 2-2 psi	Static psi	Residual psi	Static psi
11:44	11:46	19				82	72	82

Flow Summary (igpm)	
Flow 1-1	1365
Flow 1-2	
Flow 2-1	
Flow 2-2	
Total Flow	1365
Flow @ 20 psi	3656

Notes: Pre-firepump static pressure: 76

Flow Hydrant 1

GPS 53.2721 -113.5425

Flow Hydrant 2

GPS

Residual Hydrant

GPS 53.2737 -113.5417

Final Report

Fire Flow Test Report

Client Name:	AECOM	Hyd 1 - #/Port Size	4 inch	Flow Hyd 1 Addr	LE-23 - #6 Allen Pl.
Project Location:	Leduc, Alberta	Hyd 2 - #/Port Size		Flow Hyd 2 Addr	
SFE Project #:	A19-029	Hyd 1 - Pito Types	4 inch HM	Resid Hyd Addr.	LE-22 - #22 Allen Pl.
SFE Technicians:	KM / AS	Hyd 2 - Pito Types		Fire Pump Status	Auto
		Test Procedure	NFPA 291	(circle one)	Force On

Test ID: T-3 - H#LE23 Test : 6 of 10 Date: 8-Jun-20

		Flow Hyd 1		Flow Hyd 2		Residual Hydrant		
Start Time	End Time	Port 1-1 psi	Port 1-2 psi	Port 2-1 psi	Port 2-2 psi	Static psi	Residual psi	Static psi
13:49	13:51	18.5				88	68	88

Flow Summary (igpm)	
Flow 1-1	1347
Flow 1-2	
Flow 2-1	
Flow 2-2	
Total Flow	1347
Flow @ 20 psi	2608

Notes: Pre-firepump static pressure: 82

Flow Hydrant 1

GPS 53.2598 -113.5753

Flow Hydrant 2

GPS

Residual Hydrant

GPS 53.2604 -113.5767

Final Report

Fire Flow Test Report

Client Name:	AECOM	Hyd 1 - #/Port Size	4 inch	Flow Hyd 1 Addr	SF-91 - #184 Southfork
Project Location:	Leduc, Alberta	Hyd 2 - #/Port Size		Flow Hyd 2 Addr	
SFE Project #:	A19-029	Hyd 1 - Pito Types	4 inch HM	Resid Hyd Addr.	SF-92 - #227 Southfork
SFE Technicians:	KM / AS	Hyd 2 - Pito Types		Fire Pump Status	Auto
		Test Procedure	NFPA 291	(circle one)	Force On

Test ID: T-4 - H#SF91 | Test : 9 of 10 | Date: 8-Jun-20

		Flow Hyd 1		Flow Hyd 2		Residual Hydrant		
Start Time	End Time	Port 1-1 psi	Port 1-2 psi	Port 2-1 psi	Port 2-2 psi	Static psi	Residual psi	Static psi
15:09	15:11	15.5				76	62	76

Flow Summary (igpm)	
Flow 1-1	1233
Flow 1-2	
Flow 2-1	
Flow 2-2	
Total Flow	1233
Flow @ 20 psi	2607

Notes: Pre-firepump static pressure: 65

Flow Hydrant 1

GPS 53.2377 -113.5486

Flow Hydrant 2

GPS

Residual Hydrant

GPS 53.2378 -113.5466

Fire Flow Test Report

Client Name:	AECOM	Hyd 1 - #/Port Size	4 inch	Flow Hyd 1 Addr	AP-04 - 50th Street
Project Location:	Leduc, Alberta	Hyd 2 - #/Port Size		Flow Hyd 2 Addr	
SFE Project #:	A19-029	Hyd 1 - Pito Types	4 inch HM	Resid Hyd Addr.	AP-03 - Firehall
SFE Technicians:	KM / AS	Hyd 2 - Pito Types		Fire Pump Status	Auto
		Test Procedure	NFPA 291	(circle one)	Force On

Test ID: T-5 - H#AP04 Test : 8 of 10 Date: 8-Jun-20

		Flow Hyd 1		Flow Hyd 2		Residual Hydrant		
Start Time	End Time	Port 1-1 psi	Port 1-2 psi	Port 2-1 psi	Port 2-2 psi	Static psi	Residual psi	Static psi
14:36	14:38	15				80	67	80

Flow Summary (igpm)	
Flow 1-1	1213
Flow 1-2	
Flow 2-1	
Flow 2-2	
Total Flow	1213
Flow @ 20 psi	2770

Notes: Pre-firepump static pressure: 70

Flow Hydrant 1

GPS 53.2534 -113.5446

Flow Hydrant 2

GPS

Residual Hydrant

GPS 53.2530 -113.5440

Final Report

Fire Flow Test Report

Client Name:	AECOM	Hyd 1 - #/Port Size	4 inch	Flow Hyd 1 Addr	RB-17 - #189 Reed Pl.
Project Location:	Leduc, Alberta	Hyd 2 - #/Port Size		Flow Hyd 2 Addr	
SFE Project #:	A19-029	Hyd 1 - Pito Types	4 inch HM	Resid Hyd Addr.	RB-16 - #142 Reed Pl.
SFE Technicians:	KM / AS	Hyd 2 - Pito Types		Fire Pump Status	Auto
		Test Procedure	NFPA 291	(circle one)	Force On

Test ID: T-6 - H#RB17 Test : 10 of 10 Date: 8-Jun-20

		Flow Hyd 1		Flow Hyd 2		Residual Hydrant		
Start Time	End Time	Port 1-1 psi	Port 1-2 psi	Port 2-1 psi	Port 2-2 psi	Static psi	Residual psi	Static psi
15:36	15:38	10				62	46	62

Notes: Pre-firepump static pressure: 54

Flow Summary (igpm)	
Flow 1-1	1009
Flow 1-2	
Flow 2-1	
Flow 2-2	
Total Flow	1009
Flow @ 20 psi	1699

Flow Hydrant 1

GPS 53.2377 -113.5486

Flow Hydrant 2

GPS

Residual Hydrant

GPS 53.2428 -113.5050

Final Report

Fire Flow Test Report

Client Name:	AECOM	Hyd 1 - #/Port Size	4 inch	Flow Hyd 1 Addr	LP10 - 5104 46th Ave
Project Location:	Leduc, Alberta	Hyd 2 - #/Port Size		Flow Hyd 2 Addr	
SFE Project #:	A19-029	Hyd 1 - Pito Types	4 inch HM	Resid Hyd Addr.	LP-08 - 4504 51A St.
SFE Technicians:	KM / AS	Hyd 2 - Pito Types		Fire Pump Status	Auto
		Test Procedure	NFPA 291	(circle one)	Force On

Test ID: T-7 - H#LP10 Test : 5 of 10 Date: 8-Jun-20

		Flow Hyd 1		Flow Hyd 2		Residual Hydrant		
Start Time	End Time	Port 1-1 psi	Port 1-2 psi	Port 2-1 psi	Port 2-2 psi	Static psi	Residual psi	Static psi
13:28	13:30	20				84	68	83

Flow Summary (igpm)	
Flow 1-1	1401
Flow 1-2	
Flow 2-1	
Flow 2-2	
Total Flow	1401
Flow @ 20 psi	2962

Notes: Pre-firepump static pressure: 80

Flow Hydrant 1

GPS 53.2616 -113.5551

Flow Hydrant 2

GPS

Residual Hydrant

GPS 53.2613 -113.5563

Final Report

Fire Flow Test Report

Client Name:	AECOM	Hyd 1 - #/Port Size	4 inch	Flow Hyd 1 Addr	ST-15 - #105 Sunrose Ln.
Project Location:	Leduc, Alberta	Hyd 2 - #/Port Size		Flow Hyd 2 Addr	
SFE Project #:	A19-029	Hyd 1 - Pito Types	4 inch HM	Resid Hyd Addr.	ST-21 - #23 Sunrose Ln.
SFE Technicians:	KM / AS	Hyd 2 - Pito Types		Fire Pump Status	Auto
		Test Procedure	NFPA 291	(circle one)	Force On

Test ID: T-8 - H#ST15 Test : 7 of 10 Date: 8-Jun-20

		Flow Hyd 1		Flow Hyd 2		Residual Hydrant		
Start Time	End Time	Port 1-1 psi	Port 1-2 psi	Port 2-1 psi	Port 2-2 psi	Static psi	Residual psi	Static psi
14:08	14:10	21				96	71	96

Flow Summary (igpm)	
Flow 1-1	1435
Flow 1-2	
Flow 2-1	
Flow 2-2	
Total Flow	1435
Flow @ 20 psi	2616

Notes: Pre-firepump static pressure: 88

Flow Hydrant 1

GPS 53.2548 -113.5903

Flow Hydrant 2

GPS

Residual Hydrant

GPS 53.2556 -113.5885

Final Report

Fire Flow Test Report

Client Name:	AECOM	Hyd 1 - #/Port Size	4 inch	Flow Hyd 1 Addr	7120 Sparrow Dr.
Project Location:	Leduc, Alberta	Hyd 2 - #/Port Size		Flow Hyd 2 Addr	
SFE Project #:	A19-029	Hyd 1 - Pito Types	4 inch HM	Resid Hyd Addr.	NW Crane
SFE Technicians:	KM / AS	Hyd 2 - Pito Types		Fire Pump Status	Auto
		Test Procedure	NFPA 291	(circle one)	Force On

Test ID: T-9 - H#Paradise Inn   Test : 1 of 10   Date: 8-Jun-20

		Flow Hyd 1		Flow Hyd 2		Residual Hydrant		
Start Time	End Time	Port 1-1 psi	Port 1-2 psi	Port 2-1 psi	Port 2-2 psi	Static psi	Residual psi	Static psi
16:06	16:08	22				92	72	92

Flow Summary (igpm)	
Flow 1-1	1469
Flow 1-2	
Flow 2-1	
Flow 2-2	
Total Flow	1469
Flow @ 20 psi	2934

Notes: Pre-firepump static pressure: 87

Flow Hydrant 1

GPS 53.2901 -113.5457

Flow Hydrant 2

GPS

Residual Hydrant

GPS 53.2912 -113.5463

Final Report

Fire Flow Test Report

Client Name:	AECOM	Hyd 1 - #/Port Size	4 inch	Flow Hyd 1 Addr	LBP155 - Allard Ave.
Project Location:	Leduc, Alberta	Hyd 2 - #/Port Size		Flow Hyd 2 Addr	
SFE Project #:	A19-029	Hyd 1 - Pito Types	4 inch HM	Resid Hyd Addr.	LBP156 - Allarad Ave.
SFE Technicians:	KM / AS	Hyd 2 - Pito Types		Fire Pump Status	Auto
		Test Procedure	NFPA 291	(circle one)	Force On

Test ID: T-10 - H#LBP155 Test : 3 of 10 Date: 8-Jun-20

		Flow Hyd 1		Flow Hyd 2		Residual Hydrant		
Start Time	End Time	Port 1-1 psi	Port 1-2 psi	Port 2-1 psi	Port 2-2 psi	Static psi	Residual psi	Static psi
11:22	11:24	19				92	78	94

Flow Summary (igpm)	
Flow 1-1	1365
Flow 1-2	
Flow 2-1	
Flow 2-2	
Total Flow	1365
Flow @ 20 psi	3305

Notes: Pre-firepump static pressure: 82

Flow Hydrant 1

GPS 53.2926 -113.5193

Flow Hydrant 2

GPS

Residual Hydrant

GPS 53.2914 -113.5201

AECOM Imagine it. Delivered.

Appendix B

Cost Estimates

Table B.1 - Cost Estimate Summary

Upgrade	Project	Location	Description	Trigger / Upgrade Note	Cost ($)	Stage	Payment Method	Recommended Cost Split Developer	Recommended Cost Split City
East Elementary School	IMP-2	46 Ave, 45 St to E of 43 St	250 mm Watermain Upgrade	Existing FF to School ~100 L/s deficient.	$786,000.00	Existing	City		100%
Civic Centre Upgrade	IMP-1	48a St, Black Gold Dr to 47 Ave	200 mm Watermain Upgrade	Existing FF to Civic Centre ~80 L/s deficient. To be coordinated with local renewal project, road removal and restoration costs are not included.	$313,000.00	Existing	City		100%
Linsford Park School	ST1-1	Black Gold Dr to 41 Ave & 51 St	250 mm Watermain Upgrade	Existing FF to School ~40 L/s deficient. FFs drop with implementation of pressure zone.	$744,000.00	Stage 1	City		100%
Willow Park - Local Fire Flows	ST1-3	56 Ave, 52 St to 53 St	200 mm Watermain Upgrade	Local FFs drop with implementation of pressure zone. To be coordinated with local renewal project.	$145,000.00	Stage 1	City		100%
PRV A	ST1-2	Black Gold Dr W of 51 St	300 mm PRV and 250 mm Watermain	PRVs to be implemented together to create pressure zone boundary. Required when new developments are at an El. of 714 m and lower.	$577,600.00	Stage 1	Off-Site Levies	100%	
PRV B	ST1-2	Black Gold Dr at 48 St	400 mm PRV and 200 mm Watermain		$869,500.00	Stage 1	Off-Site Levies	100%	
PRV C	ST1-2	Black Gold Dr E of 46 St	350 mm PRV		$477,300.00	Stage 1	Off-Site Levies	100%	
PRV D	ST1-2	Black Gold Dr at 43 St	300 mm PRV		$438,500.00	Stage 1	Off-Site Levies	100%	
300 mm Watermain Extension - Hwy Crossing	ST2-4	QEII Crossing at 65 Avenue	Watermain Extension for Future Development	Cost inclusive of casing under QEII and 1700 m for connection to existing system	$2,126,000.00	Stage 2	Off-Site Levies	100%	
PRV - F1	ST2-3	East of Telford Lake	400 mm PRV		$490,000.00	Stage 2	Off-Site Levies	100%	
Robinson Reservoir Expansion 1	ST2-1		4500 m^3 Storage Expansion	ADD trigger 144 L/s	$4,095,000.00	Stage 2	Off-Site Levies	100%	
North Reservoir Expansion	ST2-2		8000 m^3 Storage Expansion	ADD trigger 173 L/s	$7,280,000.00	Stage 2	Off-Site Levies	100%	
350 mm Watermain Extension - Hwy Crossing	ST3-2	Hwy 2A Crossing at Sturtz Link	Watermain Extension for Future Development	Cost inclusive of casing under Hwy 2A and 200 m for connection to existing system.	$686,000.00	Stage 3	Off-Site Levies	100%	
400 mm Watermain Extension - Hwy Crossing	ST3-1	QEII Crossing at 38 Avenue	Watermain Extension for Future Development	Cost inclusive of casing under QEII and 720 m for connection to existing system	$1,557,000.00	Stage 3	Off-Site Levies	100%	
PRV - F2	ST3-1	QEII Crossing at 38 Avenue	400 mm PRV		$490,000.00	Stage 3	Off-Site Levies	100%	
Robinson Reservoir Expansion 2	ST3-3		14,0000 m^3 Storage Expansion	ADD trigger 224 L/s	$19,110,000.00	Stage 3	Off-Site Levies	100%	
Robinson Pump Upgrades	ST3-4		Pump Upgrades	Recommended with Expansion 2	$3,780,000.00	Stage 3	Off-Site Levies	100%	
400 mm Watermain Extension - Hwy Crossing	ST5-1	QEII Crossing at South Municipal Boundary	Watermain Extension for Future Development	Cost inclusive of casing under QEII and 640 m for connection to existing system	$1,462,000.00	Stage 5	Off-Site Levies	100%	
PRV - F3	ST5-1	QEII Crossing at South Municipal Boundary	400 mm PRV		$490,000.00	Stage 5	Off-Site Levies	100%	
New Reservoir and Pumphouse	ST5-2		36500 m^3 Storage and 500 HP	ADD trigger 350 L/s	$39,520,000.00	Stage 5	Off-Site Levies	100%	

-Existing system upgrades are listed in order of recommended priority

-Existing improvements include 30% contingency, Stage 1-5 include 40% contingency

Table B.2 - System Improvement - Cost Breakdown - Water Mains

		East Elementary School		Linsford Park School		Civic Centre Upgrade		Willow Park Upgrade	
Description	**Unit Cost**	**Qty**	**Total**	**Qty**	**Total**	**Qty**	**Total**	**Qty**	**Total**
Pavement Removals (m^2/lm)	$111	521	$ 57,831	420	$ 46,620	0	$ -	115	$ 12,765
Landscape Removals (m^2)	$20	0	$ -	90	$ 1,800	485	$ 9,700	0	$ -
Concrete Removals (l.m.)	$70	443	$ 31,010	400	$ 28,000	0	$ -	115	$ 8,050
200 mm Water Main Supply & Install (l.m.)	$325	43	$ 13,975	0	$ -	485	$ 157,625	115	$ 37,375
250 mm Water Main Supply & Install (l.m.)	$372	478	$ 177,816	510	$ 189,720	0	$ -	0	$ -
Abandon Sewer (l.m.)	$200	0	$ -	20	$ 4,000	0	$ -	0	$ -
Curb and Gutter Repairs (l.m.)	$100	285	$ 28,500	20	$ 2,000	0	$ -	115	$ 11,500
Mono Walk, Curb and Gutter (l.m.)	$350	163	$ 57,050	350	$ 122,500	0	$ -	0	$ -
Pavement Repairs - 200 mm Pipe (m2/lm)	$270	43	$ 11,610	0	$ -	0	$ -	115	$ 31,050
Pavement Repairs - 250 mm Pipe (m2/lm)	$275	478	$ 131,450	420	$ 115,500	0	$ -	0	$ -
Hydrant (1 every 150 m 250 mm down)	$70	2	$ 140	0	$ -	0	$ -	0	$ -
Landscape Repairs (m^2)	$100	0	$ -	90	$ 9,000	485	$ 48,500	0	$ -
Connections (ea)	$20,000	2	$ 40,000	0	$ -	0	$ -	0	$ -
Reconnect Services (ea)	$100	0	$ -	10	$ 1,000	25	$ 2,500	0	$ -
Miscellaneous	10%		$ 54,900		$ 52,000		$ 21,800		$ 10,100
SubTotal			$ 604,282		$ 572,140		$ 240,125		$ 110,840
Contingency	30%		$ 181,285		$ 171,642		$ 72,038		$ 33,252
Total			**$ 786,000**		**$ 744,000**		**$ 313,000**		**$ 145,000**
Composite Unit Price per lineal metre ($/l.m.) including Contingency			$ 1,509		$ 1,459		$ 645		$ 1,261

Table B.3 - System Improvement - Cost Breakdown - PRVs and Associated Connections

		PRV-A		PRV-B		PRV-C		PRV-D	
Description	**Unit Cost**	**Qty**	**Total**	**Qty**	**Total**	**Qty**	**Total**	**Qty**	**Total**
Pavement Removals (m^2)	$111	135	$ 14,985	195	$ 21,645	20	$ 2,220	0	$ -
Landscape Removals (m^2)	$20	0	$ -	0	$ -	0	$ -	100	$ 2,000
Concrete Removals (l.m.)	$70	0	$ -	195	$ 13,650	10	$ 700	0	$ -
200 mm Water Main Supply & Install (l.m.)	$325	0	$ -	195	$ 63,375	0	$ -	0	$ -
250 mm Water Main Supply & Install (l.m.)	$372	135	$ 50,220	0	$ -	0	$ -	0	$ -
Abandon Sewer (l.m.)	$200	0	$ -	25	$ 5,000	0	$ -	0	$ -
Curb and Gutter Repairs (l.m.)	$100	0	$ -	0	$ -	0	$ -	0	$ -
Mono Walk, Curb and Gutter (l.m.)	$350	0	$ -	195	$ 68,250	20	$ 7,000	0	$ -
Pavement Repairs - 200 mm Pipe (l.m.)	$270	0	$ -	195	$ 52,650	0	$ -	0	$ -
Pavement Repairs - 250 mm Pipe (l.m.)	$275	135	$ 37,125	0	$ -	0	$ -	0	$ -
Landscape Repairs (m^2)	$100	0	$ -	0	$ -	0	$ -	100	$ 10,000
Connections (ea)	$20,000	0	$ -	2	$ 40,000	0	$ -	0	$ -
300 mm PRV (ea)	$300,000	1	$ 300,000	0	$ -	0	$ -	1	$ 300,000
350 mm PRV (ea)	$330,000	0	$ -	1	$ 330,000	1	$ 330,000		$ -
Miscellaneous	10%		$ 10,200		$ 26,500		$ 1,000		$ 1,200
SubTotal			$ 412,530		$ 621,070		$ 340,920		$ 313,200
Contingency	40%		$ 165,012		$ 248,428		$ 136,368		$ 125,280
Total			**$ 577,600**		**$ 869,500**		**$ 477,300**		**$ 438,500**

Table B.4 - System Improvement - Cost Breakdown - Highway Crossings

Description	Unit Rate ($/m)	Length (m)	Sub-Total	Contingency (40%)	Total
Stage 2					
300 mm Watermain Extension - QEII Crossing Trenchless	$2,000	200	$400,000	$160,000	$560,000
300 mm Watermain Extension - QEII Crossing Connections	$650	1,720	$1,118,000	$447,200	$1,566,000
Stage 3					
350 mm Watermain Extension - Hwy 2A Crossing Trenchless	$2,000	175	$350,000	$140,000	$490,000
350 mm Watermain Extension - Hwy 2A Crossing Connections	$700	200	$140,000	$56,000	$196,000
400 mm Watermain Extension - QEII Crossing Trenchless	$2,500	200	$500,000	$200,000	$700,000
400 mm Watermain Extension - QEII Crossing Connections	$850	720	$612,000	$244,800	$857,000
Stage 5					
400 mm Watermain Extension - QEII Crossing Trenchless	$2,500	200	$500,000	$200,000	$700,000
400 mm Watermain Extension - QEII Crossing Connections	$850	640	$544,000	$217,600	$762,000

Table B.5 - System Improvement - Cost Breakdown - PRVs & Reservoir-Pumphouse Expansions

Description	Unit Rate	Qty	Sub-Total	Contingency (40%)	Total
Stage 2					
Robinson Reservoir Expansion 1 (m^3)	$650	4,500	$2,925,000	$1,170,000	$4,095,000
North Reservoir Expansion (m^3)	$650	8,000	$5,200,000	$2,080,000	$7,280,000
400 mm PRV (PRV-F1)	$350,000	1	$350,000	$140,000	$490,000
Stage 3					
Robinson Reservoir Expansion 2 (m^3)	$975	14,000	$13,650,000	$5,460,000	$19,110,000
Robinson Pump Upgrades (HP)	$13,500	200	$2,700,000	$1,080,000	$3,780,000
400 mm PRV (PRV-F2)	$350,000	1	$350,000	$140,000	$490,000
Stage 5					
New Reservoir (m^3)	$650	36,500	$23,725,000	$9,490,000	$33,220,000
New Pumphouse (HP)	$9,000	500	$4,500,000	$1,800,000	$6,300,000
400 mm PRV (PRV-F3)	$350,000	400	$350,000	$140,000	$490,000

*Unit rates for the Robinson Expansion 2 is higher than Expansion 1 as modifications will have to be made to allow for additional expansions

AECOM Imagine it. Delivered.

Appendix C

Model Results

aecom.com