City of Lacombe 2013 Water Model Update

Draft Report

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Engineering Services

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Draft

Sign-off Sheet

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1.0 Introduction

1.1 SYSTEM OVERVIEW

The City of Lacombe water distribution system provides potable water and fire protection services to a population of approximately 12,000 people. The City of Lacombe potable water source is supplied entirely from the North Red Deer Regional Water System Commission (NRDRWSC).

The components of the water distribution system include two pressure zones, three reservoir and pumping facilities, underground piping, and appurtenances such as valves and hydrants.

1.2 STUDY OBJECTIVES

The primary objectives of the City of Lacombe 2013 Water Model Update include:

- Update the existing City of Lacombe Water Model based on as-built drawings and current water demands;
- Complete a modeling analysis using the updated water model representing existing infrastructure for average day demand, maximum day demand, peak hour demand, and maximum day demand with fire flows;
- Provide recommendations regarding the operation of the existing Water Distribution System based on modeling results;
- Using future land use plans and the City of Lacombe MDP, complete a modeling analysis based on future demand projections to the year 2038 (25 years) for average day demand, maximum day demand, peak hour demand, and maximum day demand with fire flows;
- Provide recommendations regarding the future operation of the Water Distribution System based on modeling results;
- Deliver a final report outlining the complete findings from the study including an operational component that will describe how the system could be operated for optimize the performance of the water distribution system; and

Submit a working Water CAD V.8i Model to the City of Lacombe including all scenarios that were developed and analyzed for the Study.

2.0 Model Update

2.1 DISTRIBUTION SYSTEM UPDATE

The City of Lacombe provided a WaterCAD model that was last updated by the City in 2006, based on the original model developed by Stantec in 2003. The physical model components such as pipes, junctions, pumps and associated pump curves, PRVs, etc. were used as the basis for the 2013 model update.

The first step in the update was the conversion of the original model's coordinate system to be consistent with the City's current coordinate projection (NAD 83 UTM Zone 12N). This was required so that the various background layers (CAD and GIS) can be used directly within the WaterCAD software.

Once the model was re-projected into the NAD 83 UTM Zone 12 coordinate system, the piping and junction network was compared against the City's current CAD drawings to ensure the model accurately represents the current state of infrastructure. Several locations required reconfiguration of the network connectivity and updating actual pipe sizes based on the CAD drawing and as confirmed by the City operations department. In several cases, the diameter of existing pipes was unclear from both CAD drawings and operator knowledge of the system. As such, the unknown pipe diameters were assumed based on sizing of adjacent pipes.

The 2013 model update comprises approximately 82 km of water distribution mains ranging in size from 50mm to 300mm. Drawing 2.1 provides a breakdown of main diameters by length, and is summarized in Table 2-1.

Table 2-1 Distribution System Breakdown

Diameter (mm)	Total Length (m)
50	546
100	2,011
150	39,178
200	17,094
250	11,816
300	10,989
Grand Total	**81,871**

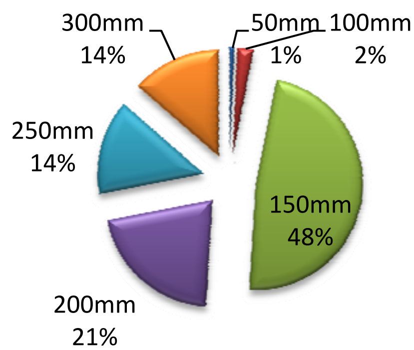


Table 2-2 provides a summary of the model pipe material and corresponding Hazen-Williams roughness value, as calibrated in the 2003 model update.

Table 2-2 Pipe Material Breakdown

Material Type	Hazen - Williams C	Total Length (m)
Asbestos Cement	135	9,542
Cast iron	95	1,237
PVC	140	71,146

The model junction elevations were updated in the WaterCAD model based on surface elevations and not actual pipe invert elevations. The elevations for each junction were spatially referenced from the nearest contour extracted from the Town's digital contour map. The purpose for using surface elevations is that the minimum pressure requirements, especially fire flows, can only be verified at a fire hydrant and are slightly conservative.

2.2 WATER DEMAND UPDATE

In order to update the water demands in the model, a water use analysis was completed for the City of Lacombe using reservoir meter records, water meter billing readings and bulk water sales records provided by the City of Lacombe from the NRDRWSC for a three year period (2010 – 2012).

The original 2003 model water demands were developed and based on an analysis of average water usage within distinct land uses (Residential, Commercial, and Industrial). The original model demands were applied to model junctions based on the average water meter demands within each land use and applied consistently over each area.

The intent of assessing the individual water meter billings as part of this update was to enable a more accurate allocation of water demands from individual meters. The City's water meter billing data was provided in Excel format from 2010 through April 2013. For each water meter, water consumption is measured on an approximate bi-monthly basis and references the Billing Account Number and Tax Folio Number. The data for each meter was summarized on an annual basis for 2010, 2011, and 2012 to develop the average annual demand at each meter. In some cases, the consumption totals between years varied significantly (i.e. 2010 total consumption was 10x the consumption of 2011 or 2012, etc). In cases where annual consumption of one of the years was inconsistent with the other two years, it was ignored as part of the average consumption for that meter. The summation of the annual water meter consumption was compared and noted to be 7 – 11% higher than to the total Reservoir Inflows and Outflows, as compared in Table 2-3. This discrepancy was unexpected, as typically the total consumption from water meter records is generally less than the total flow supplied from reservoirs, as it does not account for water losses (unmetered connections, pipe leaks, watermain breaks, etc).

Each meter Account Number and annual average consumption was referenced to its legal parcel in GIS. This resulted in the City's GIS parcel layer shapefile including the annual average metered demand for 2010, 2011, and 2012. The individual parcel demands for 2010, 2011, and 2012 are shown in **Drawings 2.2, 2.3, and 2.4** and compared in terms of standard deviation analysis.

Table 2-3 Annual Water Meter Consumption vs. Reservoir Flows

Year	Combined City Reservoir Inflow [Million m^3/year]	Combined City Reservoir Outflow [Million m^3/year]	Water Billing from Parcel Water Meters [Million m^3/year]	Variance from Meters to Reservoir Flows
2010	1.14	1.14	1.27	11%
2011	1.19	1.16	1.30	9%
2012	1.17	1.19	1.25	7%

Since the water meter consumption totals exceeded the total reservoir flows pumped to the system, the reservoir outflows were used as the correct total annual average flows.

Table 2-4 provides a summary of the annual average flows and gross per capita water consumption for the City of Lacombe. The average flow for 2010 through 2012 increased very proportionately with the increasing population, resulting in a gross per capita demand of 272 Lpcd.

Table 2-4 Annual Average Consumption and Per Capita Flows

Year	Population	Combined City Reservoir Outflow [Million m^3/year]	Average Flow [L/s]	Average Per Capita Demand [Lpcd]
2010	11,491	1.14	36.1	272
2011	11,707	1.16	36.8	271
2012	12,000	1.19	37.7	272

Given the discrepancy between meter consumption and total consumption, the individual parcel meter consumption was normalized as a percentage of the total Combined Reservoir Outflow such that the sum of the normalized water meter demands results in the total Combined Reservoir Outflow Flow.

The normalized average annual water meter consumption for 2012 from each water meter was then allocated to the nearest model junction and imported into the WaterCAD model as the basis for the Average Daily Demand (ADD) scenario. The total modeled **ADD is 36.7 L/s.**

For the purposes of evaluating the performance of the water distribution system, the allocated demands for the ADD scenario was also used as the basis for Maximum Daily Demand (MDD) and Peak Hour Demand (PHD) scenarios as follows:

- **MDD = ADD x 2 = 73.3 L/s**
- **PHD = ADD x 4 = 146.7 L/s**

2.3 EXISTING SYSTEM MODEL SCENARIOS

The previous WaterCAD model included numerous scenarios and alternatives which were completely removed and restructured as follows for the existing system (2013) model:

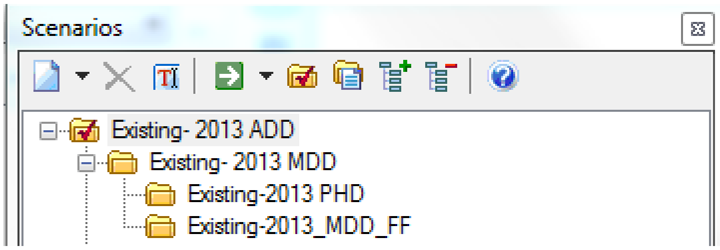


Table 2-5 Existing System Model Scenarios

		Model Alternative			
Scenario	**Active Topology**	**Physical**	**Demand**	**Initial Settings**	**Fire Flow**
ADD	2013	2013	2013ADD	2013ADD	N/A
MDD	2013	2013	2013MDD	2013MDD	N/A
MDD + FIREFLOW	2013	2013	2013ADD	2013MDD_Fire	MDD_FF
PHD	2013	2013	2013PHD	2013PHD	N/A

2.4 RESERVOIR AND PUMPING FACILITIES

The City of Lacombe water distribution system is separated into high and a low pressure zones. Pump Station C maintains pressure in the high pressure zone with an HGL of 910m while Pump Station A and B operate the lower pressure zone at an HGL of 903m. Pump Station C also provides supply to the low pressure zone through a PRV in the pumping station at an HGL of 903m.

2.4.1 Pump Station A

Pump station A is located at C&E Trail and includes one VFD pump and four constant speed vertical turbine pumps with a firm capacity (largest pumping unit out of service) of 140 L/s, and a total capacity of 203 L/s. The reservoir capacity of this facility is 4,545 m^3.

Table 2-6 provides a summary of the Pump Station A operational data.

Table 2-6 Reservoir Pump Station A Operational Data

Pump	VFD?	Reservoir HGL [m]	Pump Elevation [m]	Discharge HGL [m]	Pump Head [m]	Design Flow [L/s]
PWP-101	No	851.61	852.6	903	51.39	26
PWP-102	Yes	851.61	852.6	903	51.39	63
PWP-103	No	851.61	852.6	903	51.39	38
PWP-104	No	851.61	852.6	903	51.39	38
PWP-105	No	851.61	852.6	903	51.39	38

The sequencing of Pump Station A pumps are as follows:

1. PWP-102 (Variable Frequency Drive)
2. PWP-105
3. PWP-101
4. PWP-103
5. PWP-104

2.4.2 Pump Station B

Pump station B is located near the Hospital and includes five constant speed pumps that are designed to start automatically when pressure drops below 65 psi. The firm capacity of Pump Station B is 70 L/s with a total capacity of 90 L/s. The reservoir capacity of this facility is 2,275 m^3.

Table 2-7 provides a summary of the Pump Station B operational data.

Table 2-7 Reservoir Pump Station B Operational Data

Pump	VFD?	Reservoir HGL [m]	Pump Elevation [m]	Discharge HGL [m]	Pump Head [m]	Design Flow [L/s]
PWP-201	No	853.6	855	903	49.4	10
PWP-202	No	853.6	855	903	49.4	20
PWP-203	No	853.6	855	903	49.4	20
PWP-204	No	853.6	855	903	49.4	20
PWP-205	No	853.6	855	903	49.4	20

The sequencing of Pump Station B pumps are as follows:

1. PWP-201
2. PWP-204
3. PWP-202
4. PWP-203
5. PWP-205

2.4.3 Pump Station C

Pump Station C is located at College Heights and was constructed in 2002 to service the Upper Pressure Zone. The Upper Pressure Zone has an operating HGL of 910 m. Pump Station C was designed to operate as a dual facility. This facility has the capability to pump water from its reservoir and/or boost the pressure from the lower zone. The initial HGL of the PRV is set at 903 m, the same as operating HGL in Lower Pressure Zone.

The firm capacity of Pump Station C is 92 L/s with a total capacity of 187 L/s. The reservoir capacity of this facility is 7,120 m^3.

Table 2-8 provides a summary of the Pump Station C operational data.

Table 2-8 Reservoir Pump Station C Operational Data

Pump	VFD?	Reservoir HGL [m]	Pump Elevation [m]	Discharge HGL [m]	Pump Head [m]	Design Flow [L/s]
PWP-301	Yes	856	858	910	54	30
PWP-302	Yes	856	858	910	54	62
PWP-303	Yes	856	858	910	54	95

The sequencing of Pump Station C pumps are as follows:

1. PWP-301 (Variable Frequency Drive)
2. PWP-302 (Variable Frequency Drive)
3. PWP-303 (Variable Frequency Drive)

2.4.4 Model Verification

The model was verified for general accuracy by comparing model results to previous pressure reports provided by the City in the "2006 Water Model Summary" document, as shown in Table 2-9 on the following page.

Model Verification Conclusions and Recommendations:

- Under average day demand conditions, the model is predicting pressures within a reasonable level of accuracy. Additional data (hydrant pressure and flow data) would be required to calibrate the model and obtain more accurate results.
- The verification demonstrates that the model is generally sufficient for planning purposes, i.e. concept development of servicing options and order of magnitude costs.
- Additional hydrant testing during high demand periods should be completed to increase the level of calibration and accuracy of the model.
- It is recommended that the model be calibrated and confirmed prior to the design of improvements as recommended in this study.

Table 2-9 Model Verification Summary

Street Address	Node Name	Junction Elevation	Field Value HGL [m]	Field Value Pressure [kPa]	Model Value HGL [m]	Model Value Pressure [kPa]	Difference [kPa]	Difference [%]
2169 45th St	Monitor #1	845.5	902.3	557	903	562.7	5.7	1.02%
5934 45th Ave	Monitor #2	859.5	902.1	418	903	425.7	7.7	1.84%
5718 50th Ave	Monitor #3	857.5	901.7	434	903	445.3	11.3	2.60%
On water main North of 56th Ave, South of Cranna Lake	Monitor #4	860	903	422	903.01	421	-1	-0.24%
30 Westview Dr.	**Monitor #5****	**871**	**902.1**	**305**	**909.67**	**378.5**	**73.5**	24.10%
51 Woodland Dr.	Monitor #6	855.5	902.2	458	903.01	465	7	1.53%
17 Beardsley Ave.	Monitor #7	865.5	910.1	438	909.99	435.4	-2.6	-0.59%
5226 52nd Ave	Monitor #8	853	903	490	903	489.3	-0.7	-0.14%

** *Monitor #5 is shown to be in the high pressure zone. The Field Value HGL is likely incorrect.*

3.0 Analysis of Existing Water Distribution System

3.1 ANALYSIS METHODOLOGY

The WaterCAD model was used to examine the performance of the system with respect to appropriate levels of service and common water distribution system metrics.

The existing water distribution system was examined during Average Day Demand (ADD), Maximum Day Demand (MDD), Maximum Day Demand plus fire flows, and Peak Hour Demand (PHD) flow conditions.

The level of service criteria used in both the existing system and future system analyses was developed in accordance with industry standards and with discussion with City staff. The adopted level of service criteria is shown in Table 3-1.

Table 3-1 Level of Service Criteria

Level of Service Criteria	Level of Service
Minimum Allowable Pressure in Distribution System	276 kPa (40 psi)
Maximum Allowable Pressure in Distribution System	620 kPa (90 psi)
Maximum Allowable Hydraulic gradient (headloss)	0.5% (5m/km of headloss)
Maximum Allowable Velocity at PHD	1.5 m/s
Minimum Allowable Fire Flow (Residential)	75 L/s
**Minimum Allowable Fire Flow (MultiFamily)	135 L/s
**Minimum Allowable Fire Flow (Commercial / Industrial)	233 L/s
**Maximum Allowable Velocity in Distribution System During Fire Flow	2.5 m/s (3.0 m/s)

** For discussion with City of Lacombe

3.2 PUMPING STATION OPERATIONAL STATUS

Table 3-3 through Table 3-4 show the status of each pumping facility for ADD, MDD, and PHD conditions used in the existing system analysis.

Table 3-2 Pumping Station Operation for ADD

Pumping Station	Pump	VFD?	Design Flow [L/s]	Modeled Flow [L/s]	Status (Initial)
Reservoir A	PWP-101	No	26		Off
	PWP-102	**Yes**	**63**	**9.8**	**ON**
	PWP-103	No	38		Off
	PWP-104	No	38		Off
	PWP-105	No	38		Off
Reservoir B	**PWP-201**	**No**	**10**	**9.5**	**ON**
	PWP-202	No	20		Off
	PWP-203	No	20		Off
	PWP-204	No	20		Off
	PWP-205	No	20		Off
Reservoir C	**PWP-301**	**Yes**	**30**	**17.4**	**ON**
	PWP-302	Yes	62		Off
	PWP-303	Yes	95		Off
TOTAL FLOW				**36.7**	

Table 3-3 Pumping Station Operation for MDD

Pumping Station	Pump	VFD?	Design Flow [L/s]	Modeled Flow [L/s]	Status (Initial)
Reservoir A	**PWP-101**	**No**	**26**	**26.4**	**ON**
	PWP-102	Yes	63		Off
	PWP-103	No	38		Off
	PWP-104	No	38		Off
	PWP-105	No	38		Off
Reservoir B	PWP-201	No	10		Off
	PWP-202	**No**	**20**	**18.4**	**ON**
	PWP-203	No	20		Off
	PWP-204	No	20		Off
	PWP-205	No	20		Off
Reservoir C	**PWP-301**	**Yes**	**30**	**28.5**	**ON**
	PWP-302	Yes	62		Off
	PWP-303	Yes	95		Off
TOTAL FLOW				**73.3**	

Table 3-4 Pumping Station Operation for PHD

Pumping Station	Pump	VFD?	Design Flow [L/s]	Modeled Flow [L/s]	Status (Initial)
Reservoir A	PWP-101	No	26		Off
	PWP-102	Yes	63		Off
	PWP-103	**No**	**38**	**36.2**	**ON**
	PWP-104	No	38		Off
	PWP-105	No	38		Off
Reservoir B	**PWP-201**	**No**	**10**	**9.3**	**ON**
	PWP-202	**No**	**20**	**17.4**	**ON**
	PWP-203	**No**	**20**	**17.4**	**ON**
	PWP-204	No	20		Off
	PWP-205	No	20		Off
Reservoir C	PWP-301	Yes	30		Off
	PWP-302	**Yes**	**62**	**66.4**	**ON**
	PWP-303	Yes	95		Off
TOTAL FLOW				**146.7**	

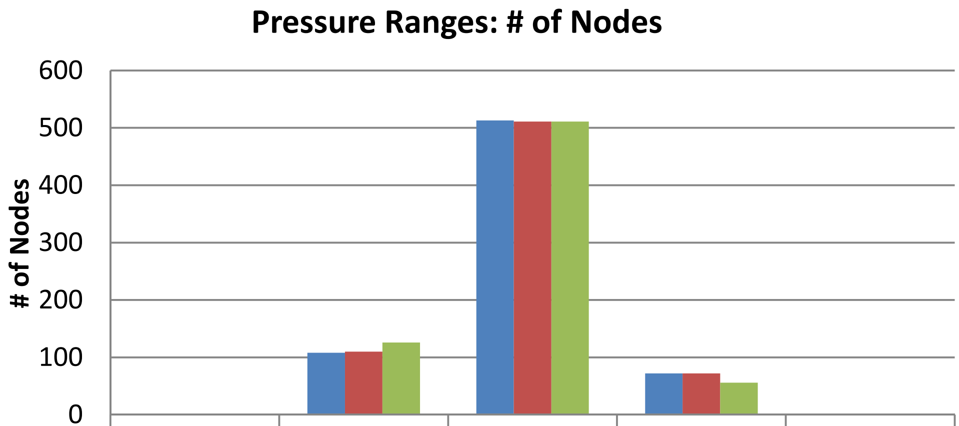
3.3 EXISTING DISTRIBUTION SYSTEM ANALYSIS

3.3.1 System Pressures

System pressures were evaluated in the model to determine the level of service provided throughout the City. A minimum of 276 kPa (40 psi) is required for most normal residential water use. Pressures above 620 kPa (90 psi) require pressure reducing valves to protect plumbing systems.

The following graph summarizes the results of the existing system pressure analysis under ADD, MDD, and PHD scenarios. These results are shown in **Drawings 3.1, 3.2 and 3.3** and summarized in Figure 3-1 below. In general, the existing distribution system provides adequate level of service to the majority of the City.

Figure 3-1 Pressure Range for Each Scenario



	< 40 psi	40 - 60 psi	60 - 80 psi	80 - 90 psi	> 90 psi
ADD	0	108	513	72	0
MDD	0	110	511	72	0
PHD	0	126	511	56	0

3.3.2 Watermain Headloss

Watermain headloss is used to indicate localized areas with relatively high headloss per km of pipe. This performance measure is used to identify undersized mains or capacity bottlenecks in the water distribution system.

Drawing 3.4 shows watermain headloss during ADD conditions. Pipe headlosses are all less than 1.7 m/km.

Drawing 3.5 shows watermain headloss during MDD conditions. There are several pipes with headloss exceeding 2.0 m/km, but these are limited to pipes within pumping stations.

Drawing 3.6 shows watermain headloss during PHD conditions. There are several pipes with headloss exceeding 2.0 m/km, but these are limited to pipes within pumping stations.

Although these results alone would not provide a rationale for improvements to the existing water system, they provide valuable insight into which watermains should be targeted when developing improvements for other levels of service deficiencies for system pressures or fire flow availability.

3.3.3 Watermain Velocity

Drawing 3.7 shows velocities in the existing water distribution system under PHD conditions. All distribution pipes for PHD are within 1.0 m/s.

3.3.4 Fire Flow Availability

The recommended fire flow servicing criteria established in the Fire Underwriters' Survey (FUS) considers varying land uses. Below are the recommended RFF for residential and commercial / industrial land uses:

- Single family residential area hydrants shall be required to produce a minimum required fire flow (RFF) of 75 L/s.
- Multifamily residential area hydrants shall be required to produce a minimum required fire flow (RFF) of 135 L/s.
- Commercial / industrial / College area hydrants shall be required to produce a minimum RFF of 233 L/s.

The City of Lacombe Engineering Design Standards states the maximum allowable velocity of 2.5 m/s and a residual pressure of 150 kPa.

Drawing 3.8 shows the fire flow availability under MDD plus fire flow demand conditions using the maximum velocity constraint of 2.5 m/s.

As **Drawing 3.8** shows, there are approximately 159 junctions with available fire flows that do not meet the required rate. Table 3-5 shows the number of nodes that failed the fire flow requirements in each zone and land use using the 2.5 m/s velocity constraint.

Table 3-5 # of Failed Fire Flow Nodes in Each Pressure Zone (Drawing 3.8)

Required Fire Flow	High Pressure Zone	Low Pressure Zone	Grand Total
Residential (75 L/s)	16	59	75
MultiFamily (135 L/s)	4	8	12
Commercial / Industrial (233 L/s)	8	64	72
Grand Total	**28**	**131**	**159**

For comparison purposes, Table 3-6 shows the maximum fireflow that can be attained from a single pipe using the velocity constraint of 2.5 m/s and 3.0 m/s. As this table shows, the maximum fire flow that can be available on a 150mm dead-end main is only 44 L/s using the 2.5 m/s constraint, hence why numerous locations are failing the minimum fire flow requirement.

Table 3-6 Maximum Fire Flow Attainable using Maximum Velocity Constraint

Diameter	Max FireFlow with V = 2.5 m/s on single or dead-end main [L/s]	Max FireFlow with V = 2.5 m/s on looped main [L/s]	Max FireFlow with V = 3 m/s on single or dead-end main [L/s]	Max FireFlow with V = 3 m/s on looped main [L/s]
150	44.2	88.4	53.0	106.0
200	78.5	157.1	94.2	188.5
250	122.7	245.4	147.3	294.5
300	176.7	353.4	212.1	424.1

If the maximum allowable velocity of at MDD + fire flow is increased to 3.0 m/s, the number of nodes that fail to meet the fire flow requirements is reduced to 130 nodes. The maximum velocity of 3.0 m/s is a common metric used in other smaller municipalities and could be considered by the City of Lacombe.

Drawing 3.9 shows the fire flow availability under MDD plus fire flow demand conditions using the maximum velocity constraint of 3.0 m/s.

As **Drawing 3.9** shows, there are approximately 130 junctions with available fire flows that do not meet the required rate. Table 3-7 shows the number of nodes that failed the fire flow requirements in each zone and land use using the 3.0 m/s velocity constraint.

Table 3-7 # of Failed Fire Flow Nodes in Each Pressure Zone (Drawing 3.9)

Required Fire Flow	High Pressure Zone	Low Pressure Zone	Grand Total
Residential (75 L/s)	11	40	51
MultiFamily (135 L/s)	4	4	8
Commercial / Industrial (233 L/s)	8	63	71
Grand Total	**23**	**107**	**130**

These locations are generally deficient due to the maximum velocity constraint, with pipe diameters of 200mm and smaller, incomplete looping (i.e. dead-ends or closed zone valves), or hydrants located at the higher elevations of each zone. Only four locations are deficient as a result of insufficient residual pressure.

3.3.5 Storage Capacity Analysis

Fire storage requirements have been calculated and reserved for each of the facilities as shown in Table 3-8.

Table 3-8 Fire Storage Requirements

Reservoir	Storage Criteria	Storage Volume Required (m^3)
Reservoir A/B	233 L/s for 3.0 hours	2,500
Reservoir C	233 L/s for 3.0 hours	2,500
Total		**5,000**

The City of Lacombe currently has an estimated water storage capacity of 13,940 m^3:

- Reservoir A: 4,545 m^3
- Reservoir B: 2,275 m^3
- Reservoir C: 7,120 m^3

The required water storage as provided by Alberta Environment Sustainable Resources Department (AESRD) is as follows:

S = A + B + C where:

- **A = Fire Storage**: water should be held in reserve for critical fire demands as they occur from time to time. This fire storage water is not available for consumptive use and should always be present in the system. As fire storage water is permanently required, it effectively reduces the usable reservoir volume. The required Fire Storage Capacity in each zone should equal 233 L/s for 3 hours, as per the Fire Underwriters Survey, equivalent to 2,500 m^3 that must be stored in each Pressure Zone for a total of 5,000 m^3 for the City

- **B = Equalization Storage**: when demand fluctuates above the supply of water being input to the system, equalization or peaking storage is used to meet the temporary shortfall. The typical volume of water required for peaking or equalization storage is to be 25% of the MDD condition. With MDD of 73.3 L/s, the required equalization storage is 1,583 m^3.

- **C = Emergency Storage**: Emergency storage is usually needed to provide potable water to the system in case of a supply line failure or maintenance shut down. An emergency storage of 15% of the ADD is required. With an ADD of 36.7 L/s, the required equalization storage is approximately 476 m^3.

- **S = Total Storage Required = 5,000 m^3 + 1,583 m^3 + 476 m^3 = 7,059 m^3**

The total water storage requirement for the City of Lacombe by combining the storage requirements as noted above is 7,059 m^3.

As the NRDRWSC is a long regional water supply line that currently extends from the City of Red Deer to the Town of Ponoka, a contingency for water supply, additional storage defined as (D) for the City, should also be allowed for to account for unlikely supply interruptions. A common criterion for this additional required storage is 1.25 ADD. Given this criterion, the total storage calculation is as follows:

$$S = A + B + C + D$$

Based on current ADD of 3,171 m^3/day, D would equal 3,964 m^3.

Therefore S = Total Storage Required including = 7,059 m^3 + 3,964 m^3 = 11,023 m^3

The current storage available within the City of Lacombe currently meets the requirements for fire flow, emergency and equalization storage as required by AESRD.

3.4 EXISTING SYSTEM IMPROVEMENTS

According to the analysis of the existing system, the level of service criteria can be satisfied except for fire flow requirements.

Drawing 3.10 shows the fire flow availability under MDD plus fire flow conditions without using velocity constraints.

As **Drawing 3.10** shows, there are approximately 50 junctions with available fire flow that do not meet the required rate. Thus, the following pipe network improvements are recommended relating to the insufficient fire flow junctions.

Several pipe network improvements have been completed to provide an adequate fire flow rate for the existing system under no velocity constraint. Each improvement provides additional capacity for fire protection. Table 3-9 summarizes the proposed pipe network improvements. **Drawing 3.11** shows the location and proposed diameter for each upgraded pipe.

There are still areas that do not meet fire flow requirements. One of the main areas is located in the College Heights Area (North of the City), as a result of the fire flow demand of 233 L/s for Canadian University College. Apart from this area, another eight junctions have insufficient fire flow; this is generally due to incomplete looping (i.e. dead-ends or closed zone valves) or insufficient residual pressure.

Table 3-9 Existing Pipe Network Improvements

No.	Improvement Description	Pipe Length (m)	Improved Diameter (mm)
1	**Willow Ridge (Sunset Way)**	167	200
	This includes upgrading 167 m of existing 150 mm dead-end mains to 200 mm to increase the available fire flow to the properties in the area.		
2	**Willow Ridge (Beside Garden Road)**	170	200
	This includes upgrading 170 m of existing 150 mm dead-end mains to 200 mm to increase the available fire flow to the properties in the area.		
3	**Willow Ridge (Beside Sunset Way)**	153	200
	This includes upgrading 153 m of existing 150 mm dead-end mains to 200 mm to increase the available fire flow to the properties in the area.		
4	**Fairway Heights (Garden Road)**	275	200
	This includes upgrading 275 m of existing 150 mm mains to 200 mm to increase the available fire flow to the properties in the area.		
5	**58 Street & 54 Ave**	187	150
	This includes upgrading 187 m of existing 100 mm mains to 150 mm to increase the available fire flow to the properties in the area.		
6	**Lacombe Jr. High School**	49	250
	This includes upgrading 49 m of existing 150 mm dead-end mains to 250 mm to increase the available fire flow to the properties in the area.		
7	**52 Street & 54 Ave**	118	250
	This includes upgrading 118 m of existing 100 mm mains to 250 mm to increase the available fire flow to the properties in the area.		
8	**Len Thompson Dr & Wolf Creek Dr**	96	250
	This includes upgrading 96 m of existing 200 mm mains to 250 mm to increase the available fire flow to the properties in the area.		
9	**51 Street & 54 Ave**	233	250
	This includes upgrading 233 m of existing 100 mm mains to 250 mm to increase the available fire flow to the properties in the area.		
10	**Lacombe Composite High School**	272	250
	This includes upgrading 272 m of existing 150 mm mains to 250 mm to increase the available fire flow to the properties in the area.		
11	**58 Street & 53 Ave**	193	250
	This includes upgrading 193 m of existing 150 mm mains to 250 mm to increase the available fire flow to the properties in the area.		
12	**46 Ave & HWY 2A**	67	250

	This includes upgrading 67 m of existing 150 mm to 250 mm to increase the available fire flow to the properties in the area.		
13	**Wolf Creek Dr & 53 Ave**	211	250
	This includes upgrading 211 m of existing 200 mm mains to 250 mm to increase the available fire flow to the properties in the area.		
14	**Len Thompson Dr**	233	250
	This includes upgrading 233 m of existing 150 mm mains to 250 mm to increase the available fire flow to the properties in the area.		
15	**Len Thompson Dr**	400	200
	This includes upgrading 400 m of existing 150 mm mains to 200 mm to increase the available fire flow to the properties in the area.		
16	**48 Ave**	136	200
	This includes upgrading 136 m of existing 100 mm mains to 200 mm to increase the available fire flow to the properties in the area.		
17	**Beside 47 Street & 52 Ave**	276	200
	This includes upgrading 276 m of existing 150 mm mains to 200 mm to increase the available fire flow to the properties in the area.		
18	**Woodland Drive & Beside HWY 2A**	417	200
	This includes upgrading 417 m of existing 150 mm mains to 200 mm to increase the available fire flow to the properties in the area.		
19	**Woodland Drive & HWY 2A**	197	200
	This includes building a new 200 m main connecting the existing 150 mm mains with existing 250 mm main to increase the available fire flow to the properties in the area.		
20	**51 Street & 55 Ave**	48	200
	This includes upgrading 48 m of existing 100 mm mains to 200 mm to increase the available fire flow to the properties in the area.		
21	**Beside HWY 2A & 49b Ave**	45	200
	This includes upgrading 45 m of existing 150 mm mains to 200 mm to increase the available fire flow to the properties in the area.		
22	**46 St & 48 Ave**	1027	200
	This includes upgrading 1027 m of existing 150 mm mains to 200 mm to increase the available fire flow to the properties in the area.		
23	**46 Ave & 45 Street**	319	250
	This includes upgrading 319 m of existing 200 mm mains to 250 mm to increase the available fire flow to the properties in the area.		

4.0 Future Distribution System Development

4.1 FUTURE GROWTH

The future growth analysis for this study is consistent with the population projections developed for the "City of Lacombe - 2013 Transportation Master Plan" (Stantec, 2013). The Transportation Master Plan (TMP), developed three population horizons: 15,000 Persons (Year 2018), 17,500 Persons (Year 2023) and 22,500 Persons (Year 2033). Growth areas and proposed population developed in this analysis is mainly based on future growth planning information available. In order to project the water demands in the future system, a future analysis for growth areas was completed based on the Design Guidelines from City of Lacombe. The following **Table 4-1** summarizes consumption rate applied in the model under ADD, MDD, and PHD scenarios.

Table 4-1 Future Growth Areas Consumption Rate

	ADD (LPCD)	MDD (LPCD)	PHD (LPCD)	Industrial/Commercial/Institutional (L/S/Ha)
Consumption Rate	375	750	1500	0.2

Drawing 4.1 displays a color coding map of growth areas and **Table 4-2** shows the proposed zoning, area and population information, as well as projected ADD water demands for future growth areas.

Table 4-2 Future Growth Area Breakdown

Name	Area (ha)	Population	Zoning	ADD (L/s)
1	6.2	154	Residential	0.5
2	1.9	48	Residential	0.2
3	21.3	506	Residential	1.6
4	2.4	0	Industrial/Commercial	0.5
5	7.3	143	Residential	0.3
6	3.9	97	Residential	3.9
7	55.6	1228	Residential	8.1
8 (I/C)	50.1	0	Industrial/Commercial	6.3
8 (R)	134.8	2548	Residential	0.4
9A	31.0	771	Outside MDB Boundary - Assumed Residential	2.5
9B(I/C)	7.5	0	Industrial/Commercial	3.6
9B(R)	24.4	794	Outside MDB Boundary - Assumed Residential	2.5
10(I/C)	7.9	0	Industrial/Commercial	2.5

Name	Area (ha)	Population	Zoning	ADD (L/s)
10(R)	26.6	860	Outside MDB Boundary - Assumed Residential	2.7
11A	64.7	0	Industrial/Commercial	8.0
11B	65.4	0	Industrial/Commercial	0.2
12	85.6	1973	Residential	5.0
13	7.4	112	Residential	0.8
14A	38.4	787	Residential	0.8
14B	22.9	399	College	6.5
15	140.1	2506	Outside MDB Boundary - Assumed Residential	6.5
16 (I/C)	58.7	0	Industrial/Commercial	5.9
16 (R)	135.6	1116	Residential	1.4
17	14.0	0	Industrial/Commercial	6.1
18	60.8	0	Industrial/Commercial	1.3
14B			College	0.5
Total of Growth Areas	**1074.5**	**14042**		77
Total of Existing System		**12000**		37
Total of Ultimate		**26,042**		**114**

4.2 FUTURE WATER DEMAND

The projected water demands for ADD condition are equally allocated to the model junctions within each growth area. The total future modeled **ADD is 131 L/s**. The water demand for the ADD scenario was also used as the basis for MDD and PHD conditions as follows:

- **MDD = ADD x 2 = 262 L/s**
- **PHD = ADD x 4 = 524 L/s**

4.3 FUTURE STORAGE REQUIREMENTS

The existing distribution system in the City of Lacombe has an estimated water storage capacity of 13,940 m^3:

- Reservoir A: 4,545 m^3
- Reservoir B: 2,275 m^3
- Reservoir C: 7,120 m^3

Check the cross reference as numbers changed

The required water storage, as discussed in Section 3.3.5, consists of Fire Storage (A), Equalization Storage (B), Emergency Storage (C) and Additional Storage (D) for the regional water supply line of the NRDRWSC. As a result, total storage required is the sum of A, B, C and D.

Table 4-3 summarizes the storage requirement for the ultimate water system. In terms of the future system, given the deficiencies in storage capacity for both pressure zones, it is necessary to install two reservoirs and pump facilities in each pressure zone to provide sufficient storage and pumping capacity to the future system.

As Pump Station B was approaching the end of its design life, as stated in the "2003 Water Distribution Study", it is preferable to decommission Pump Station B and incorporate the necessary storage volumes and pumping capacities in the new facilities from a long-term perspective.

The proposed reservoirs and pumping facilities' details are discussed in the following section.

Table 4-3 Future System Storage Analysis

Storage Type	Method	Storage Volume (m3)
A = Fire Storage	Fire Storage (233 L/s for 3 hours) 2,516 m3 in each Zone	5,033
B = Equalization Storage	0.25MDD	4,928
C = Emergency Storage	0.15ADD	1,478
D = Additional Storage	1.23ADD	12,319
TOTAL STORAGE REQUIRED	S=A+B+C+D	23,758
Existing Reservoir Capacity		11,665
Surplus (Deficit)		12,319

4.4 PROPOSED RESERVOIRS AND PUMPING FACILITIES

4.4.1 Future Pressure Zones

As discussed in Section 2.4, the future water distribution system in the City of Lacombe is still split into two pressure zones: a high and a low pressure zone. Pump Station C primarily maintains pressure in the high pressure zone at an HGL of 910 m and also provides supply to the low pressure zone through a PRV with an HGL of 903 m; while Pump Station A operates the low pressure zone at an HGL of 903 m.

With decommissioned Pump Station B, two reservoirs and pump stations F1 and F2 are introduced and developed in the future model to better service future distribution systems in the City of Lacombe. Pump Station F1 is proposed to maintain pressure in the low pressure zone with an HGL of 903 m; while Pump Station F2 is planned to operate the high pressure zone at an HGL of 910 m.

Given the topography and feasibility of the future system, a small part of Lacombe Golf Course is proposed to be in the High Pressure Zone; while the rest of the growth areas are included in the Low Pressure Zone. The High and Low Pressure Zones in the existing system remain the same.

Drawing 4.1 shows the proposed pressure zone boundary in the future system.

4.4.2 Pump Station F1

Pump Station F1 is proposed to locate at the southeast corner of the City in Area 8 (residential zone) to service the Low Pressure zone with an operating HGL of 903 m. The firm capacity of Pump Station F1 is 150 L/s with a total capacity of 200 L/s.

Table 4-4 provides a summary of the Pump Station F1 operational data.

Table 4-4 Reservoir Pump Station F1 Operational Data

Pump	VFD?	Reservoir HGL [m]	Pump Elevation [m]	Discharge HGL [m]	Pump Head [m]	Design Flow [L/s]
PWP-303	Yes	847	846.5	903	56	50
PWP-303	Yes	847	846.5	903	56	50
PWP-303	Yes	847	846.5	903	56	50
PWP-303	Yes	847	846.5	903	56	50

4.4.3 Pump Station F2

Pump Station F2 is proposed to locate at the southwest corner of the City beside the Lacombe Golf and Country Club to service the High Pressure zone. The operating HGL of PRV is set at 910 m, the same as the operating HGL in the High Pressure Zone.

The firm capacity of Pump Station F2 is 130 L/s with a total capacity of 195 L/s. Table 4-5 provides a summary of the Pump Station F2 operational data.

Table 4-5 Reservoir Pump Station F2 Operational Data

Pump	VFD?	Reservoir HGL [m]	Pump Elevation [m]	Discharge HGL [m]	Pump Head [m]	Design Flow [L/s]
PWP-303	Yes	871.49	872	910	38.51	65
PWP-303	Yes	871.49	872	910	38.51	65
PWP-303	Yes	871.49	872	910	38.51	65

4.5 FUTURE SERVICING

The future distribution system is constructed based on an upgraded existing system discussed in Section 3.4 with pipe network improvements. The projected water demands under ADD, MDD and PHD conditions for growth areas were added to the junctions equally in each growth area. The long-term plan of decommissioning Pump Station B and proposed two reservoirs and pumping facilities in the southeast corner and southwest corner of the City was applied to the future system analysis in the water model.

Two main future developments are planned to occur; the first along Highway 2 west of the City all the way to the C&E Trail north of the City; the second is proposed to be in the southeast corner of the City along Highway 2A. There are only a few small areas that are included in future growth scenarios within the boundary of the existing system.

The layout of the water mains in the future system is based on the Future Watermains Planning Drawing. The proposed diameters are according to the future land use plan and design guidelines from the City.

4.5.1 West Area

Pump Station F2 is proposed to construct in the southwest corner of the City, beside the Lacombe Golf Club, to provide capacity for the proposed High Pressure Zone. In order to provide additional capacity for the existing system, three watermains are proposed to extend from the existing system to connect with the future system in the west area of the City, at Woodland Drive, 56 Ave and 50 Ave.

Specifically, a 300 mm pipe is recommended to extend from the pump station northward to Woodland Drive, connected with another new 250 mm pipe around Fairway Heights, to serve the new High Pressure zone (#1a and #1b, Drawing 4.2). Two 250 mm diameter pipes are recommended to extend from Woodland Drive and 56 Ave and to connect with above pipes from Pump Station F2 (#2 and #3, Drawing 4.2).

The third extended pipe from 50 Ave is proposed to be 300 mm diameter (#4a, Drawing 4.2). In addition, a 200 mm diameter pipe would be constructed on 45 Ave and loop back to 50 Ave to increase the flow along Highway 12 (#4b, Drawing 4.2). The #4a pipe is proposed to connect with another new 300 mm pipe westward from Pump Station F2 (#5, Drawing 4.2) and loop all the way to the North of the

University, adjacent to Highway 2, with several 300mm pipes (#6, Drawing 4.2). These new pipes would be proposed to connect to the existing 300 mm pipe on Mobel Drive. Two loops can be accomplished by connecting the extended pipes on Woodland Drive and 56 Ave to the new 300mm pipes with two 250 mm pipes (#7a and #7b, Drawing 4.2).

Three PRVs of HGL at 903 m are proposed to be constructed to separate the High and Low pressure zones in the West Area (#8, #9, #10, Drawing 4.2). The fourth proposed PRV is to be installed on Mobel Drive, also with a HGL of 903 m (#11, Drawing 4.2). In Area 5, the fifth proposed PRV with HGL of 903 m is proposed to construct to separate the two pressure zones (#12, Drawing 4.2).

4.5.2 North Area (College Heights and Terrace Heights)

Due to the configuration of the existing water distribution network, the existing 300 mm main on C& E Trail and College Ave is needed to extend to Area 12 with 250 mm pipes (#13, Drawing 4.2), connecting to new 300 mm pipes. Thus, Pump Station C could provide additional capacity to the new low pressure growth areas alongside Highway 2.

4.5.3 South East Area

Reservoir and Pump Station F1 are proposed for installation in the southeast corner of the City (Area 8 R), to provide sufficient capacity for growth areas in the southeast corner. A 400 mm main is proposed to extend from the Pump Station F2 northward and connect with the existing system on 50 Ave (Highway 12) through 300 mm mains (#14 and # 15, Drawing 4.2). The 300 mm main is required to extend southward from Pump Station F1 and loop back to the new 400 mm main discussed above and also extend to connect Area 11A, Area 11B and 18. Finally, it would be connected with the existing pipe on Dickens Ln and Petitcoat Ln (#16, Drawing 4.2). Four loops located in Area 7, Area 9, Area10 and Area 11, are proposed to be constructed with 250 mm pipes and connect to 300 mm new mains (#17, Drawing 4.2).

4.5.4 Small Future Developments

There are three small developments (Area 5, 6 and 13) around Fairway Heights. Given the land use is residential, 200mm diameter pipes are proposed to connect the growth areas to the existing systems (#18, #19, #20, Drawing 4.2). The other four developments are proposed to be in the southeast of the City alongside Highway 2A. Due to different land use plans in these areas, pipes with different diameters are proposed. The 200 mm pipes are recommended in Area 1 and 2 (#21 and #22, Drawing 4.2); while the Area 4 and 17 would have 250 mm diameter pipes (#23 and #24, Drawing 4.2).

4.6 PIPE NETWORK IMPROVEMENTS REQUIRED TO SUPPORT FUTURE SYSTEM

In order to facilitate future development, several pipe network improvements are required. These requirements are in addition to the improvements proposed for the existing distribution system and are required as development meets appropriate level of service criteria.

Table 4-6 summarizes the proposed pipe network improvements. **Drawing 4.3** shows the location and proposed diameter for each upgraded pipe.

As additional capacity is provided by the two proposed reservoir and pumping facilities, the inadequate fire flow issues occurring in the upgraded existing system, noted in Drawing 3.11, could be improved by the following first four items (A-D). In addition, the loop from 46 St & 50 Ave to 48 Ave & 45 St, a part of Item E (Table 5-5), has also been proposed to upgrade from 150 mm to 200 mm in diameter for existing system improvement (Section 3.4, Item 22). Therefore, it is preferable for the City to upgrade this part of pipes right t0 250 mm, if the existing system improvements are only to be considered.

Table 4-6 Pipe Network Improvements for Future System

No.	Improvement	Pipe Length (m)	Pipe Diameter (mm)
A	**College Heights**	1133	300
	This includes upgrading 1131 m of existing 150 mm and 250 mm mains to 300 mm to increase the available fire flow to the properties in the area.		
B	**College Heights**	1081	250
	This includes upgrading 1351 m of existing 150 mm mains to 200 mm to increase the available fire flow to the properties in the area.		
C	**Fairway Heights (Eagle Rd & Northstar Dr)**	145	150
	This includes upgrading 145 m of existing 150 mm dead-end mains to 200 mm to increase the available fire flow to the properties in the area.		
D	**46 St & 48 Ave & 45 St**	1585	250
	This includes upgrading 286 m of upgraded 200 mm mains to 250 mm to increase the available fire flow to the properties in the area.		
E	**Outlet of Pumpstation C**	41	400
	This includes upgrading 41 m of existing 300 mm and 400 mm mains to 300 mm to satisfy velocity constraint under PHD condition.		
F	**61 Ave & Petitcoat Ln**	106	250
	This includes upgrading 106 m of existing 200 mm mains to 250 mm to satisfy headloss constraint.		
G	**Fairway Dr. & 58 St**	44	200
	This includes upgrading 44 m of existing 150 mm dead-end mains to 200 mm to satisfy headloss constraint.		

5.0 Future System Evaluation

The Water CAD model was used to examine the performance of the future system with respect to appropriate levels of service and common water distribution system metrics, as discussed in Section 3.1. The future water distribution system was also tested during ADD, MDD, MDD plus fire flow, and PHD flow conditions.

5.1 FUTURE SYSTEM MODEL SCENARIOS

The following screen-shot shows the structure of the future model scenarios in the model and **Table 5-1** illustrates the alternatives applied to the future scenarios.

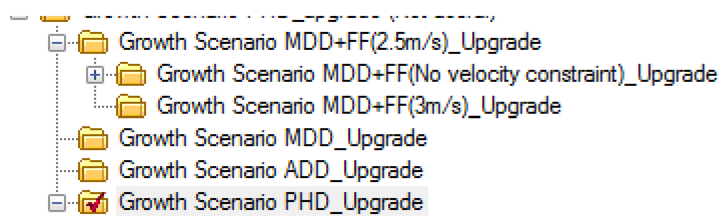


Table 5-1 Future System Scenarios

		Model Alternative			
Scenario	**Active Topology**	**Physical**	**Demand**	**Initial Settings**	**Fire Flow**
ADD	Growth Scenario Upgrade_ADD	Growth Scenario Upgrade	Growth Scenario ADD	Growth Scenario Upgrade	N/A
MDD	Growth Scenario Upgrade_MDD	Growth Scenario Upgrade	Growth Scenario MDD	Growth Scenario Upgrade	N/A
MDD + FIREFLOW	Growth Scenario Upgrade_MDD+FF	Growth Scenario Upgrade	Growth Scenario MDD	Growth Scenario MDD+FF	Growth MDD+FF
PHD	Growth Scenario Upgrade_PHD	Growth Scenario Upgrade	Growth Scenario PHD	Growth Scenario Upgrade	N/A

5.2 PUMPING STATION OPERATIONAL STATUS

Table 5-2, Table 5-3 and Table 5-4 show the status of each pumping facility for ADD, MDD and PHD conditions used in the future system analysis.

Table 5-2 Pumping Station Operation for ADD

Pump Station	Pump	VFD?	Design Flow [L/s]	Modeled Flow [L/s]	Status (Initial)
Reservoir A	PWP-101	No	26		Off
	PWP-102	Yes	63		Off
	PWP-103	**No**	**38**	**36.04**	**ON**
	PWP-104	No	38		Off
	PWP-105	No	38		Off
Reservoir C	PWP-301	Yes	30		ON
	PWP-302	**Yes**	**62**	**40.88**	**OFF**
	PWP-303	Yes	95		Off
Reservoir F1	**PWP-303**	**Yes**	**95**	**59.95**	**ON**
	PWP-303	Yes	95		Off
	PWP-303	Yes	95		Off
	PWP-303	Yes	95		Off
Reservoir F2	**PWP-303**	**Yes**	**95**	**32.21**	**ON**
	PWP-303	Yes	95		Off
	PWP-303	Yes	95		Off
TOTAL FLOW				**169.08**	

Table 5-3 Pumping Station Operation for MDD

Pump Station	Pump	VFD?	Design Flow [L/s]	Modeled Flow [L/s]	Status (Initial)
Reservoir A	**PWP-101**	**No**	**26**	**25.73**	**ON**
	PWP-102	Yes	63		Off
	PWP-103	**No**	**38**	**33.56**	**ON**
	PWP-104	**No**	**38**	**33.56**	**ON**
	PWP-105	No	38		Off
Reservoir C	**PWP-301**	**Yes**	**30**	**24.78**	**ON**
	PWP-302	**Yes**	**62**	**49.52**	**ON**
	PWP-303	Yes	95		Off
Reservoir F1	**PWP-303**	**Yes**	**95**	**56.78**	**ON**
	PWP-303	**Yes**	**95**	**56.78**	**OFF**
	PWP-303	Yes	95		Off
	PWP-303	Yes	95		Off
Reservoir F2	**PWP-303**	**NO**	**95**	**57.46**	**ON**
	PWP-303	NO	95		Off

	PWP-303	NO	95		Off
TOTAL FLOW				**338.17**	

Table 5-4 Pumping Station Operation for PHD

Pump Station	Pump	VFD?	Design Flow [L/s]	Modeled Flow [L/s]	Status (Initial)
Reservoir A	**PWP-101**	**No**	**26**	**25.69**	**ON**
	PWP-102	Yes	63		Off
	PWP-103	**No**	**38**	**33.4**	**ON**
	PWP-104	**No**	**38**	**33.4**	**ON**
	PWP-105	**No**	**38**	**33.4**	**ON**
Reservoir C	**PWP-301**	**Yes**	**30**	**27.55**	**ON**
	PWP-302	**Yes**	**62**	**55.66**	**ON**
	PWP-303	**Yes**	**95**	**87.4**	**ON**
Reservoir F1	**PWP-303**	**Yes**	**95**	**79.65**	**ON**
	PWP-303	**Yes**	**95**	**79.65**	**ON**
	PWP-303	**Yes**	**95**	**79.65**	**ON**
	PWP-303	Yes	95		Off
Reservoir F2	**PWP-303**	**Yes**	**95**	**70.44**	**ON**
	PWP-303	**Yes**	**95**	**70.44**	**ON**
	PWP-303	Yes	95		Off
TOTAL FLOW				**676.33**	

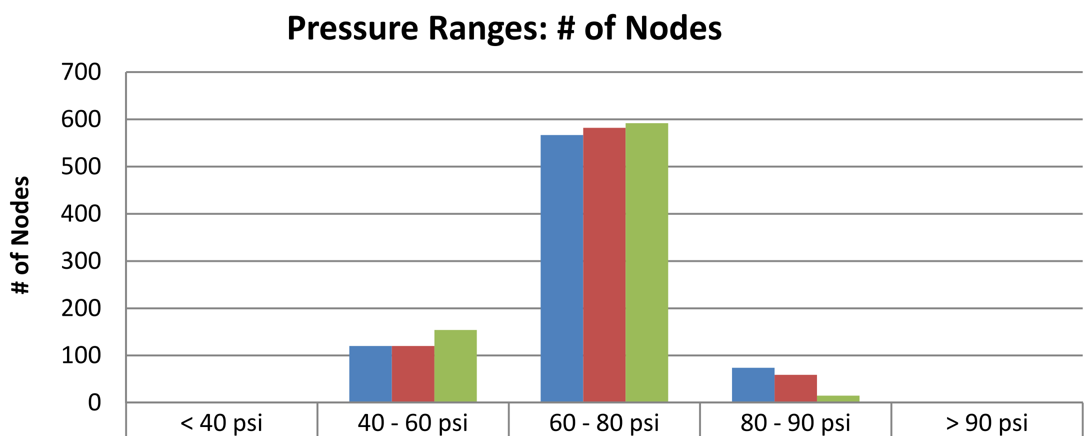
It should be noted that under MDD plus fire flow conditions, all the pumps in Pump Station C should be On to provide sufficient fire flow for the High Pressure Zone in the area of College Heights. As for PHD conditions, it is necessary to open all the pumps as well in Pump Station C to provide adequate pressure for High Pressure Zone in the future system.

5.3 FUTURE DISTRIBUTION SYSTEM ANALYSIS

5.3.1 System Pressures

The following graph summarizes the results of the future system pressure analysis under ADD, MDD, and PHD scenarios. These results are shown in **Drawings 5.1, 5.2 and 5.3** and are summarized in Figure 5-1 below. The future distribution system generally provides an adequate level of service to the majority of the City.

Figure 5-1Pressure Range for Each Scenario



	< 40 psi	40 - 60 psi	60 - 80 psi	80 - 90 psi	> 90 psi
ADD	0	120	567	74	0
MDD	0	120	582	59	0
PHD	0	154	592	15	0

5.3.2 Watermain Headloss

Do you want to duplicate 3.3.2?

Drawing 5.4 shows watermain headloss during ADD conditions. There are several pipes with headloss exceeding 2 m/km, but are limited to pipes within pumping stations.

Drawing 5.5 shows watermain headloss during MDD conditions. There are several pipes with headloss exceeding 2 m/km, but are limited to pipes within pumping stations.

Drawing 5.6 shows watermain headloss during PHD conditions. There are several pipes with headloss exceeding 2.0 m/km, but are limited to pipes within pumping stations.

Although these results alone would not provide a rationale for improvements to the existing water system, they provide valuable insight into which watermains should be targeted when developing improvements for other level of service deficiencies for system pressures or fire flow availability.

5.3.3 Watermain Velocity

Drawing 5.7 shows velocities in the future water distribution system under PHD conditions. All distribution pipes for PHD are within 1.2 m/s.

5.3.4 Fire Flow Availability

Drawing 5.8 shows the fire flow availability under MDD plus fire flow demand conditions using the maximum velocity constraint of 2.5 m/s.

As **Drawing 5.8** shows, there are approximately 154 junctions with available fire flow that do not meet the required rate. Table 5-5 shows the number of nodes that failed the fire flow requirements in each zone and land use using the 2.5 m/s velocity constraint.

Table 5-5 # of Failed Fire Flow Nodes in Each Pressure Zone (Drawing 5.8)

Required Fire Flow	High Pressure Zone	Low Pressure Zone	Grand Total
Residential (75 L/s)	10	55	66
MultiFamily (135 L/s)	7	12	19
Commercial / Industrial (233 L/s)	6	64	70
Grand Total	**23**	**131**	**154**

Drawing 5.9 shows the fire flow availability under MDD plus fire flow demand condition using the maximum velocity constraint of 3.0 m/s.

As Drawing 5.9 shows, there are approximately 109 junctions with available fire flow that do not meet the required rate. Table 5-6 shows the number of nodes that failed the fire flow requirements in each zone and land use using the 3.0 m/s velocity constraint

Table 5-6 # of Failed Fire Flow Nodes in Each Pressure Zone (Drawing 5.9)

Required Fire Flow	High Pressure Zone	Low Pressure Zone	Grand Total
Residential (75 L/s)	6	37	43
MultiFamily (135 L/s)	4	11	15
Commercial / Industrial (233 L/s)	3	48	51
Grand Total	**13**	**96**	**109**

Drawing 5.10 shows the fire flow availability under MDD plus fire flow demand conditions without velocity constraint. It is noted that all the fire flow junctions meet the requirement under this condition. As a result, the insufficient fire flow availability in first two conditions are mainly due to velocity constraint.

6.0 Conclusions and Recommendations

6.1 CONCLUSIONS

In this study, the existing water distribution system model was reconstructed based on as-built drawings and actual water demands. A modeling analysis using the updated water model was completed to evaluate the capacity of the existing infrastructure for ADD, MDD, MDD plus fire flow and PHD conditions. A series of pipe network improvements have been bought forward due to deficiencies of fire flow availability under MDD conditions. The area of the Canadian University College is considered as a problematic location for fire flow availability in the existing system.

A conceptual plan for future growth scenarios has been developed based on the updated existing water model. The pressure zone boundary was expanded based on the existing zone boundary alongside future developments to better service each growth area. Two reservoirs and pumping facilities are proposed at the southeast and southwest corner of the City, in order to provide sufficient capacity for growth areas, as well as adequate fire flow for the existing system. In addition, several pipe network improvements are also required for growth scenarios. The performance of the future system has been evaluated under ADD, MDD, MDD plus fire flow and PHD conditions. The fire flow demand can be satisfied for future growth scenarios without velocity constraints. As future development grows, the recommendations for both systems should be considered by the City.

6.2 RECOMMENDATIONS

There are a number of improvements illustrated in this report. In addition, it is also recommended that the City replace all existing 50 mm and 100 mm diameter pipes with minimum 150 mm diameter PVC watermain as a water main replacement strategy for future development purposes.

Table 6-1 summaries the proposed improvements with an associated Opinion of Probable Cost in this study, as illustrated in Table 3-9 and Table 4-6.

Table 6-1 Summary of Improvements

No.	Pipe Length (m)	Improved Diameter (mm)	Opinion of Probable Cost
1	167	200	
2	170	200	
3	153	200	
4	275	200	
5	187	150	
6	49	250	
7	118	250	
8	96	250	

No.	Pipe Length (m)	Improved Diameter (mm)	Opinion of Probable Cost
9	233	250	
10	272	250	
11	193	250	
12	67	250	
13	211	250	
14	233	250	
15	400	200	
16	136	200	
17	276	200	
18	417	200	
19	197	200	
20	48	200	
21	45	200	
22	1027	200	
23	319	250	
A	1133	300	
B	1081	250	
C	145	150	
D	1585	250	
E	41	400	
F	106	250	
G	44	200	

Table 6-2 summaries the proposed future pipes in growth areas with corresponding Opinion of Probable Cost.

Table 6-2 Future Proposed Pipes Breakdown

Future Pipe Diameter (mm)	Length (m)	Opinion of Probable Cost
200	3499	
250	13900	
300	14244	
400	278	
Grand Total	31921	

Appendix A

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A.1 HEADING 8

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A.1.1 Heading 9

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