Water

Town of Edson

Municipal Servicing Plan Update

Prepared by:

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Project Number:

4193-033-00-4.6.1

Date:

December 2011

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December 21, 2011

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

Dear Mr. Solomon:

Project No: 4193-033-00-4.6.1

Regarding: Municipal Servicing Plan Update

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

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

Sincerely,
AECOM Canada Ltd.

Ahtesham Shirazi, M.Eng., P.Eng.
Project Manager
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AS:blb
Encl.

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Revision Log

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

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Project Manager

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AECOM CANADA LTD.

Signature ____________

Date ____________

PERMIT NUMBER: P10450
The Association of Professional Engineers, Geologists and Geophysicists of Alberta.

Report Reviewed By:

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

Executive Summary

Water Supply and Distribution System

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Wastewater Collection System

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

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

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

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

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

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

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

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

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

Table ES 2: Wastewater Collection System - Cost Estimate Summary

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

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

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

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

Stormwater Management System

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

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

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

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

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

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

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

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

Servicing of infill developments can be achieved through:

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

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

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1. Introduction

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

1.1 Background

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

1.2 Scope of Work

The scope of work includes the following:

<u>Water Supply and Distribution System</u>

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

<u>Wastewater Collection System</u>

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

Stormwater Management System

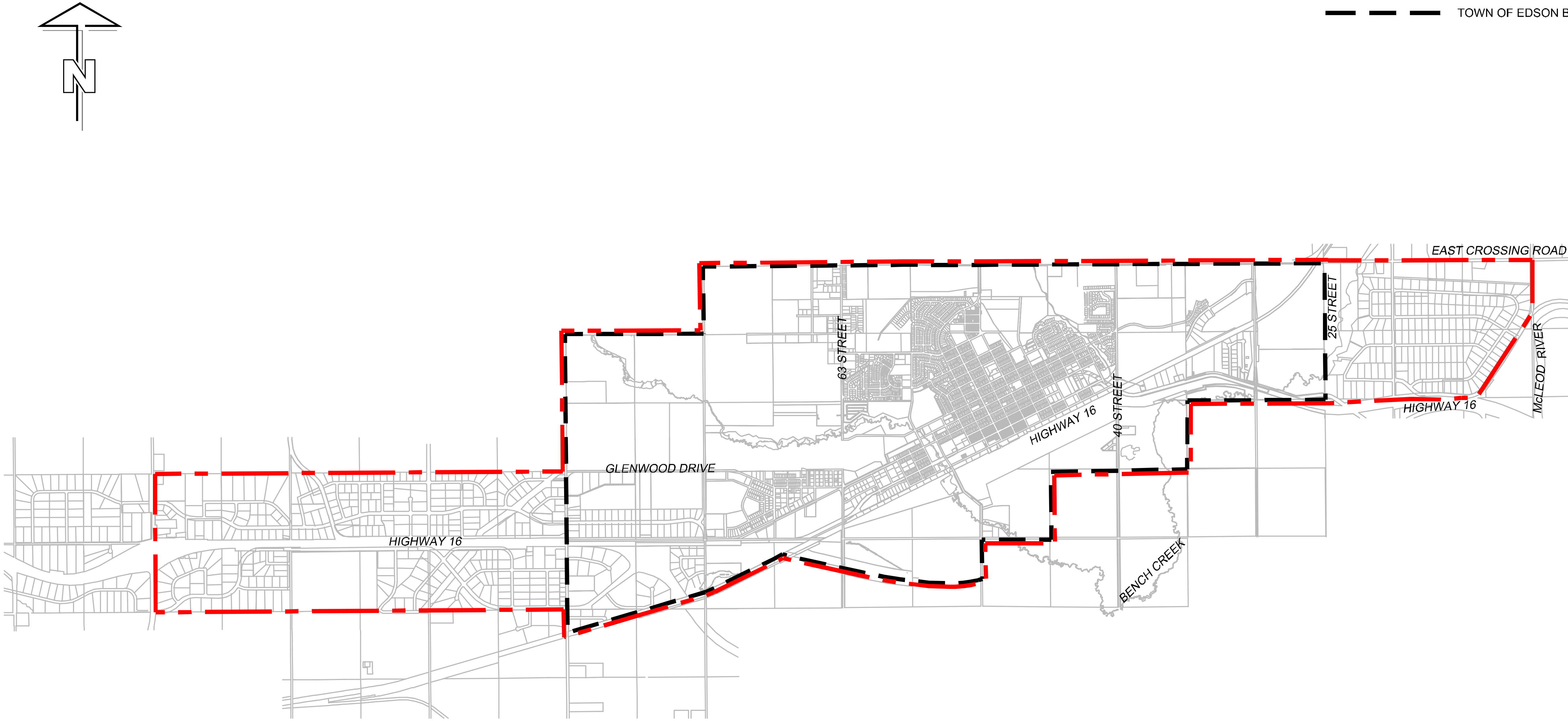
- Collection and review of all relevant data, including the July 2005 Stormwater Management Plan, existing as-builts and mapping.
- A field reconnaissance will be conducted, as well as any field survey that may be required.
- Develop growth scenarios for the ultimate development of the Town of Edson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

1.3 Study Area

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

LEGEND

- STUDY AREA BOUNDARY
- TOWN OF EDSON BOUNDARY



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Town of Edson
Municipal Servicing Plan Update

AECOM

Location Plan
Figure 1.1

2. Data Collection and Review

2.1 General

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

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

2.2 Relevant Reports

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

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

2.3 Population Projections

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

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

Table 2.1: Projected Growth for the Years 2015 and 2025

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

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

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

2.4 Land Use

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

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

3. Water Supply and Distribution System

3.1 General

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

3.2 Hydrant Testing

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

Table 3.1: Summary of Hydrant Flow Test Results

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

3.3 Design Criteria

Water Consumption Rates

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

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

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

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

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

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

Table 3.2: Water Consumption Rates

Demand	Town of Edson	Yellowhead County
Residential Demands		
Average Day Demand (L/capita/day)	330	375
Maximum Day Demand (L/capita/day)	660	750
Peak Hour Demand (L/capita/day)	990	1500
Non-Residential Demands (Industrial/Commercial)		
Average Day Demand (L/ha/day)	10,000	17,280
Maximum Day Peaking Factor	2.0	$10*Q^{-0.45}$
Peak Hour Peaking Factor	3.0	

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

Fire Flows

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

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

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

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

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

Pressure Requirements

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

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

3.4 Existing System Description

Supply System

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

Table 3.3: Groundwater Well Data

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

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

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

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

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

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

Storage Reservoirs

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

Table 3.4: Existing Reservoir Storage

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

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

Table 3.5: Storage Requirements – Existing Development Condition

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

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

Pumphouse Facilities

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

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

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

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

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

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

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

Table 3.6: Flow Requirements – Existing Development Condition

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

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

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

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

Water Distribution System

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

3.5 System Modelling

3.5.1 Existing Model Development

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

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

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

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

3.5.2 Model Calibration

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

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

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

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

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

Table 3.7: Model Calibration

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

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

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

3.6 System Evaluation under Existing Development Conditions

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

- Peak Hour Demand
- Maximum Day Demand Plus Fire Flow

3.6.1 Peak Hour Demand

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

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

3.6.2 Maximum Day Demand plus Fire Flows

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

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

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

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

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

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

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

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

3.7 System Deficiencies

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

3.8 System Improvements

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

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

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

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

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

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

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

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

3.9 Future Servicing

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

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

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

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

3.9.1 Alternative 1 – Design with Town of Edson Standards

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

3.9.1.1 Supply System

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

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

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

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

3.9.1.2 Storage Reservoirs

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

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

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

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

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

2015 Development Condition

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

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

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

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

2025 Development Condition

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

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

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

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

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

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

3.9.1.3 Pumphouse Facilities

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

Table 3.13: Future Pumping Requirements – Alternative 1

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

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

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

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

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

3.9.1.4 Water Distribution System

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

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

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

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

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

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

3.9.2.1 Supply System

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

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

3.9.2.2 Storage Reservoirs

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

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

2015 Development Condition

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

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

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

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

2025 Development Condition

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

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

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

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

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

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

Table 3.18: Water Storage Summary

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

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

3.9.2.3 Pumphouse Facilities

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

Table 3.19: Future Pumping Requirements – Alternative 3

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

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

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

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

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

Table 3.20: Pumping Requirements Summary

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

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

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

3.9.2.4 Water Distribution System

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

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

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

3.9.3 Pressure Zones

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

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

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

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

Table 3.21: PRV Settings - Existing With Improvements

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

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

Table 3.22: PRV Settings - Alternative 1 (2025)

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

Table 3.23: PRV Settings - Alternative 3 (2025)

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

3.10 Cost Estimates

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

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

Table 3.24: Cost Estimates – Groundwater Wells

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

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

Table 3.25: Cost Estimates – Reservoirs

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

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

Table 3.26: Cost Estimates – Pumping

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

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

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

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

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

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

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

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

Table 3.29: Cost Estimates – Pressure Reducing Valves

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

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

Table 3.30: Cost Estimate Summary

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

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

3.11 Implementation Plan

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

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

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

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

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

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

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

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

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

Figure 3.11: 2015 Water Distribution System Schematic Alternative 1 – Figure 1 of 3

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

Figure 3.13: 2015 Water Distribution System Schematic Alternative 1 – Figure 3 of 3

Figure 3.14: 2015 Water Distribution System Schematic Peak Hour Demand Alternative 1

Figure 3.15: 2015 Water Distribution System Schematic Maximum Day Demand and Fire Flow Alternative 1

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

Figure 3.17: 2015 Water Distribution System Schematic Alternative 3 – Figure 2 of 2

Figure 3.18: 2015 Water Distribution System Schematic Peak Hour Demand Alternative 3

Figure 3.19: 2015 Water Distribution System Schematic Maximum Day Demand and Fire Flow Alternative 3

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

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

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

Figure 3.23: 2025 Water Distribution System Schematic Peak Hour Demand Alternative 1

Figure 3.24: 2025 Water Distribution System Schematic Maximum Day Demand and Fire Flow Alternative 1

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

Figure 3.26: 2025 Water Distribution System Schematic Alternative 3 – Figure 2 of 2

Figure 3.27: 2025 Water Distribution System Schematic Peak Hour Demand Alternative 3

Figure 3.28: 2025 Water Distribution System Schematic Maximum Day Demand and Fire Flow Alternative 3

Figure 3.29. 2025 Pressure Zone Boundaries Alternative 1

Figure 3.30. 2025 Pressure Zone Boundaries Alternative 3

4. Wastewater Collection System

4.1 General

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

4.2 Study Data

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

4.3 Existing System Description

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

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

4.4 Model Development

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

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

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

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

4.4.1 Dry Weather Flow Model

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

4.4.2 Wet Weather Flow Model

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

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

Table 4.1: Infiltration Parameters

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

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

4.5 Model Calibration

This section outlines the calibration of the Town of Edson sanitary sewer model. The calibration consists of a two step process: identification of the dry weather flows and identification of the wet weather flows for the selected rainfall events.

4.5.1 Dry Weather Flow Calibration

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

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

Table 4.2: Dry Weather Flow Calibration Summary

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

4.5.2 Wet Weather Flow

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

Table 4.3: Summary of 2008 Rainfall Events

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

June 6, 2008 Event

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

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

Table 4.4: June 6, 2008 Event Calibration Summary

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

August 21, 2008

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

Table 4.5: August 21, 2008 Event Calibration Summary

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

June 11, 2008

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

Table 4.6: June 11, 2008 Event Calibration Summary

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

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

4.5.3 Summary of Model Calibration Criteria

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

Table 4.7: Summary of Calibration Criteria

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

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

4.6 Existing System Evaluation

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

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

Table 4.8: Design Rainfall Events

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

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

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

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

Table 4.9: Surcharging Levels

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

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

Table 4.10: Capacity Utilization Levels

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

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

4.6.1 Dry Weather Flow Results

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

4.6.2 5 Year Event Results

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

4.6.3 25 Year Event Results

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

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

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

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

4.7 Existing System Improvements

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

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

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

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

4.7.1 Existing System Improvements – Phase 1

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

Table 4.12: Summary of Phase 1 Existing System Upgrades

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

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

4.7.2 Existing System Improvements – Phase 2

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

Table 4.13: Summary of Phase 2 Existing System Upgrades

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

4.7.3 Existing System Improvements – Phase 3

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

Table 4.14: Summary of Phase 3 Existing System Upgrades

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

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

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

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

4.7.4 Existing System Improvements Cost Estimates

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

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

Table 4.15: Phase 1 Cost Summary

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

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

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

Table 4.16: Phase 2 Cost Summary

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

Table 4.17: Phase 3 Cost Summary

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

4.8 Future Sanitary Servicing Plan

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

Table 4.18: Sanitary Sewer Design Values

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

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

4.8.1 2015 and 2025 Development

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

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

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

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

Table 4.19: 2015 and 2025 Development Scenario Pipe Upgrades

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

4.8.2 2015 and 2025 Development Cost Estimates

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

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

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

Tunnelling was assumed under HWY 16

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

Table 4.21: 2015 and 2025 Development Scenario Cost Estimates

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

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

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

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

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

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

4.9 Cost Estimate Summary and Implementation Plan

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

Table 4.23: Cost Estimate Summary

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

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

Implementation Plan

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

Figure 4.1: Existing Sanitary System and Service Area

Figure 4.2: Existing Sanitary Catchment Areas

Figure 4.3: Dry Weather Flow Diurnal Flow Pattern

Figure 4.4: Existing System Dry Weather Flow Results

Figure 4.5: Existing System 5 Year 4 Hour Event Results

Figure 4.6: Existing System 5 Year 24 Hour Event Results

Figure 4.7: Existing System 25 Year 4 Hour Event Results

Figure 4.8: Existing System 25 Year 24 Hour Event Results

Figure 4.9: Phase 1 Improvements

Figure 4.10: Phase 2 Improvements

Figure 4.11: Phase 3 Improvements

Figure 4.12: 2015 and 2025 Sanitary Servicing

5. Stormwater Management System

5.1 General

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

5.2 Field Reconnaissance

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

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

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

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

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

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

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

5.3 Existing System Modelling and Evaluation

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

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

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

Table 5.1: Design Rainfall Events

Return Period	Duration (hours)	Total Rainfall (mm)	Peak Intensity (mm/hr)
5	4	34.0	91.4
	24	59.7	8.3
25	4	44.5	125.8
	24	75.5	10.5
100	4	53.3	154.6
	24	88.6	12.3

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

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

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

Table 5.2: Storm Sewer Capacity Utilization Levels

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

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

Table 5.3: Storm Sewer Levels of Surcharging

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

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

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

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

5.3.1 5 Year Rainfall Event Results

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

Figure 5.2 shows the results for the 5 year 24 hour rainfall event. The system performs significantly better during the 24 hour rainfall event. For the 24 hour event, many nodes are blue, however, these pipes are within 2.5 m of ground and the hydraulic grade line is still within the diameter of the pipe. A few select locations also indicate a HGL within 1.0 m below ground, as represented by a yellow node, however, these are shallow systems and the pipes have adequate capacity. Along 51st Street, there is one pipe surcharging above the pipe obvert, with the node indicating a HGL within 1.0 m below ground. In general, the system has adequate capacity to convey the 5 year 24 hour rainfall event.

5.3.2 25 Year Rainfall Event Results

Figure 5.3 shows that storm system performance during the 25 year 4 hour rainfall event is slightly worse than during the 5 year 4 hour rainfall. The system is under higher surcharge conditions and several additional areas experience surface flooding. However, minor systems are not designed for this large of an event.

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

5.3.3 100 Year Rainfall Event Results

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

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

5.4 Existing System Improvements

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

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

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

Table 5.4: Proposed Storm Sewer Improvements

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

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

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

5.5 Future Storm Servicing Plan

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

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

Table 5.5: Stormwater Drainage Basins

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

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

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

5.5.1 Allowable Discharge Rate

Allowable discharge rates for the Bench, Wase, and Poplar Creeks were established in the Town of Edson Stormwater Management Plan, completed by UMA Engineering in 2005. A regional analysis was performed, which established a flow versus drainage area relationship. This hydrologic relationship was used to determine the estimated 100 year peak flow for the creeks. Table 5.6 summarizes the allowable discharge rates for Bench, Wase and Poplar Creek.

Table 5.6: Receiving Watercourse Allowable Discharge Rates

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

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

Table 5.7: Stormwater Management Facilities Allowable Discharge Rates

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

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

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

The drainage system was assessed using XP-SWMM, an industry accepted stormwater management model, for the 100 year rainfall event with durations of 4 hours and 24 hours. As the SWMF's are proposed to be wet facilities, they were simulated as having 2.5 m of dead storage, 2.0 m of live storage, and 0.5 m of freeboard. The SWMFs were modelled as having a trapezoidal shape with side slopes of 7:1 (H:V) from the high water level to 1.0 m below the normal water level (NWL) and side slopes of 3:1 (H:V) from 1.0 m below the NWL to the bottom of the pond. The configuration of the SWMFs can be addressed in detail during the preliminary design phase.

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

Table 5.8: Storage Volumes

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

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

5.6 Cost Estimates

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

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

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

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

Table 5.9: Storm Sewer Improvements Cost Estimates

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

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

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

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

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

Table 5.10: Stormwater Management Facility Cost Estimates

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

5.7 Flood Mapping

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

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

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

5.8 Infill Development Guidelines

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Figure 5.1: Existing System 5 Year 4 Hour Rainfall Event Results

Figure 5.2: Existing System 5 Year 24 Hour Rainfall Event Results

Figure 5.3: Existing System 25 Year 4 Hour Rainfall Event Results

Figure 5.4: Existing System 25 Year 24 Hour Rainfall Event Results

Figure 5.5: Existing System 100 Year 4 Hour Rainfall Event Results

Figure 5.6: Existing System 100 Year 24 Hour Rainfall Event Results

Figure 5.7: Proposed Improvements

Figure 5.8: Improved System 5 Year 4 Hour Rainfall Event Results

Figure 5.9: Future Stormwater Drainage Basins

Figure 5.10: Future Storm Servicing Plan

6. Conclusions and Recommendations

6.1 Water Supply and Distribution System

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

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

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

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

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

6.2 Wastewater Collection System

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

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

6.3 Stormwater Management System

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

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