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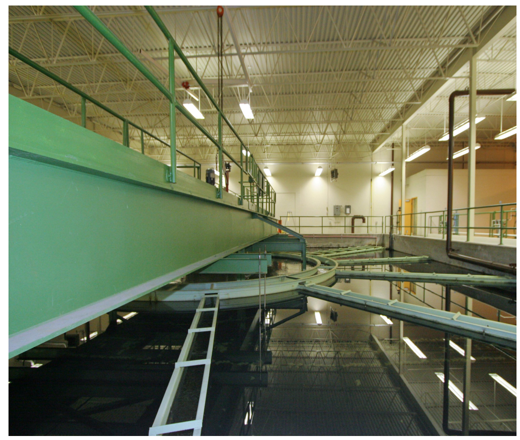

City of Lloydminster

Final Report

Waterworks Master Plan and System Assessment 2016

October 2016

ISL Engineering and Land Services

ISL Engineering and Land Services Ltd. is an award-winning full-service consulting firm dedicated to working with all levels of government and the private sector to deliver planning and design solutions for transportation, water, and land projects.

Corporate Authorization

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PROFESSIONAL ENGINEER
R.S. TOMBS
MEMBER 30802
16 10 19
YR. MN. DAY
SASKATCHEWAN

Richard Tombs, P.Eng., C.Eng., MIChemE
Senior Project Manager

Association of Professional Engineers & Geoscientists of Saskatchewan
CERTIFICATE OF AUTHORIZATION
ISL Engineering and Land Services Ltd.
Number C1096
Permission to Consult held by:

Discipline	Sk. Reg. No.	Signature
CIVIL	30802	

ISL Engineering and Land Services

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Suite 100, 7909 – 51 Avenue Edmonton, AB T6E 5L9 T: 780.438.9000 F: 780.438.3700

October 19, 2016

Our Reference: 14228

City of Lloydminster
6623 – 52 Street
Lloydminster, Alberta
T0B 0L0

Attention: Abdelqader Abdelqader, M.Sc., P.Eng
Branch Manager, Utilities.

Dear Sir:

Reference: City of Lloydminster Waterworks Master Plan and System Assessment Final Report Submission

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ISL Engineering and Land Services Ltd. (ISL) is pleased to submit the final Lloydminster Waterworks Master Plan and System Assessment for your records. In addition to meeting the requirements of the Saskatchewan Water Security Agency, this report provides the information and background behind the basis and projections used to assess the historical performance and future capacity of the waterworks system. Sections 7 and 8 of this report develop a capital plan and a series of recommendations based upon the requirements of the waterworks system.

ISL would like to extend their thanks to the City of Lloydminster for providing ISL with the opportunity to be involved in this assessment for the City's Waterworks System. ISL hopes to continue working with the City on other projects in the future and looks forward to building on and strengthening their relationship with the City.

Should you have any questions or queries with regards to this report or the project as a whole please do not hesitate to call me at 780 438 9000.

Sincerely,

Richard Tombs, P.Eng., C.Eng., MIChemE
Senior Project Manager

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1.0 Introduction

1.1 Objective

As indicated in Section 35 (1) of The Water Regulations (2002), the permittee of a waterworks system supplying water for human consumption is required to submit an independent engineering assessment of the system to the Saskatchewan Water Security Agency (SWSA) at least once every five years.

Round 1 of the Waterworks System Assessment (WSA) for the City of Lloydminster was conducted by Associated Engineering in 2005 (City of Lloydminster Waterworks System Assessment, 2006). This was followed by a Round 2 WSA, which was completed by Worley Parsons in 2010 (City of Lloydminster Waterworks System Assessment and Capital Plan, 2010). ISL Engineering and Land Services Ltd. (ISL) was retained by the City of Lloydminster to re-evaluate the condition, efficiency, and capacity of the different components of the City's waterworks system, which are listed below.

- Raw Water Supply System
- Water Treatment System including Disinfection
- Treated Water System including Storage and Distribution
- Waterworks System Infrastructure

This assessment process will allow the City to anticipate any upcoming upgrades and maintenance requirements in order to provide raw and high quality treated water to consumers, which complies with the Saskatchewan standards and regulations. Planning for future upgrades will allow the City to formulate a financial plan and re-assess their water rates in advance of commencing any of the upgrades or changes identified within this report.

1.2 References

- A Guide to Waterworks Design EPB 201, Saskatchewan Ministry of Environment, 2012.
- City of Lloydminster Waterworks System Assessment, Associated Engineering, 2006.
- City of Lloydminster Waterworks System Assessment and Capital Plan, Worley Parsons, 2010.
- City of Lloydminster Waterworks Emergency Response Plan, 2014.
- City of Lloydminster Growth Study, ISL Engineering and Land Services, 2013.
- Disinfection Profiling and Benchmarking Guidance Manual, Environmental Protection Agency, 1999.
- Guidelines for Chlorine Gas Use in Water and Wastewater Treatment EPB 265, Government of Saskatchewan, 2004 Revision.
- New West Reservoir Preliminary Design Report, ISL Engineering and Land Services, 2004.
- North Saskatchewan River Watershed Source Water Protection Plan, Saskatchewan Watershed Authority, 2008.
- Permit to Operate a Waterworks, Water Security Agency, 2011.
- The Occupational Health and Safety Regulations, Government of Saskatchewan, 1996.
- The Water Regulations, Saskatchewan Ministry of Environment, 2002.

2.0 Waterworks System Overview

2.1 System Description

The City's existing Waterworks Systems consists of the following components:

- Raw Water Supply System
- Water Treatment and Disinfection System
- Treated Water Storage and Distribution System

The following subsections will provide, in detail, a description of each of the sub-components within the three systems listed above. A Process Flow Diagram (PFD) entailing all the components described in this section is provided below (Figure 1.1).

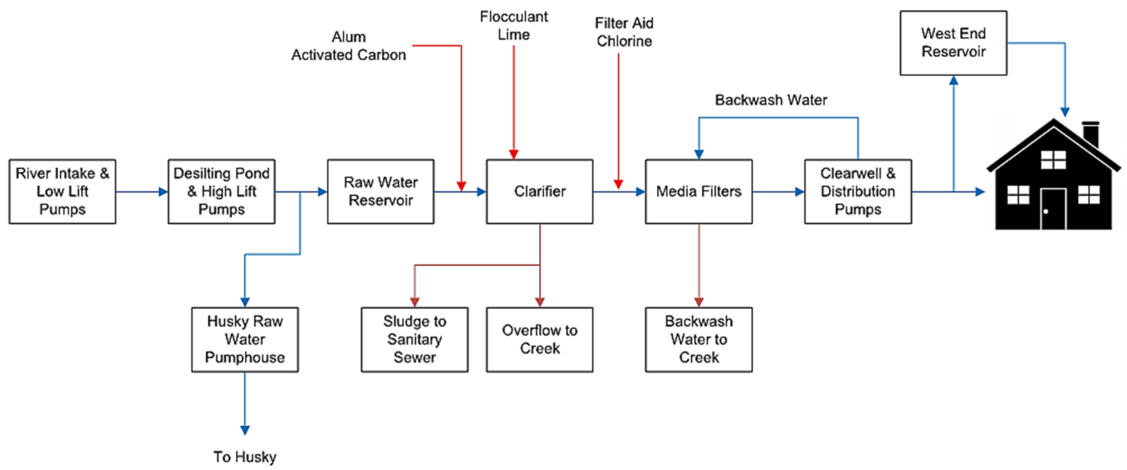


Figure 1.1: Process Flow Diagram

2.1.1 Raw Water Supply System

The Raw Water Supply System, which provides raw water to both the City of Lloydminster (City) and Husky Energy Upgrader (Husky), was constructed in the summer of 1983 and placed into service during the spring of 1984. Following commissioning, the system was upgraded first in 1991 and finally in 1999 to address the increasing Husky raw water demand. The major infrastructure and elements of the Raw Water Supply System are listed below:

- River Intake Structure
- River Intake Pump Station
- Desilting Pond
- Raw Water Supply Pipeline
- Husky Raw Water Pumphouse
- Other Raw Water Allocation
- Raw Water Reservoir
- Contingency and Source Water Protection Plans

River Intake Structure

The River Intake Structure is the first stage of the City's raw water supply system. Positioned in the channel of the North Saskatchewan River, this concrete rectangular structure is responsible for drawing and directing surface water from the river into the river intake pump station. The intake structure draws in raw water from the river using four (4) intake ports, two (2) on each side of the intake structure. Following entry into the intake structure, raw water is directed to the river intake pump station via a 1,050 mm diameter high-density Polyethylene (HDPE) pipe. The intake structure as well as the transfer pipe have a design capacity of 1,042 L/sec (90,000 m^3/day).

River Intake Pump Station

The River Intake Pump Station comprises of the 3 components.

- A 10 mm mesh size travelling screen with a rated capacity of 1,040 L/sec (89,856 m^3/day), which screens out and prevents larger debris from entering the rest of the downstream raw water system.
- Two VFD-driven low-lift pumps, each with a rated capacity of 347 L/sec (30,000 m^3/day) at a discharge pressure of 9 m which pump the raw water influent to the desilting pond. Provision to direct raw water from the low lift pump wet well straight to the high lift pumps wet well is provided in the event the desilting pond is undergoing maintenance.
- Three fixed speed high lift pumps, two pumps each with a capacity of 232 L/sec (20,000 m^3/day) at a discharge pressure of 542.5 m, and one with a capacity of 116 L/sec (10,000 m^3/day) at a discharge pressure of 542.5 m. These units pump the screened and de-silted river water to the raw water reservoir adjacent to the Water Treatment Plant (WTP) via a 750 mm raw water supply pipeline.

Desilting Pond

The single U-shaped desilting pond is designed to remove particulate matter such as sediment and silt from the raw water, prior to being pumped to the raw water reservoir by the high lift pumps. Following desilting, the outlet from the pond flows by gravity into the high lift pump wet well. The volumetric capacity of the desilting pond is 122,000 m^3. As per the 2010 WSA, the desilting pond was last dredged in 2007.

Raw Water Supply Pipeline

The high lift pumps have the capability to move both de-silted and screened river water from the high lift pump wet well to the raw water reservoir via a 36 Km long 750 mm diameter yellow jacketed, epoxy lined steel pipe with a design capacity of 694 L/sec (60,000 m^3/day). At this capacity, the velocity of the flow within the pipe would be approximately 1.6 m/s. A provision is provided to bypass the raw water reservoir and directly feed the WTP, in the event the raw water reservoir is undergoing maintenance.

Raw Water Reservoir

Constructed and commissioned in 1974, the raw water reservoir is located directly west of the City's Water Treatment Plant (WTP). The volumetric capacity of the raw water reservoir is approximately 204,500 m^3, of which the lower 2 m cannot be used due to allowances for solids accumulation and position of the outlet pipework. In accounting for 1 m of ice cover this results in a working raw water storage capacity of 155,000 m^3 in winter, and 188,000 m^3 in summer

Copper Sulfate is added periodically to the raw water reservoir to control algal growth within the raw water, typically during the spring and summer seasons. The City currently adds one 25 kg bag of Copper Sulfate to the raw water reservoir during each application, which translates to a dose of 2.2 mg/L. Over the past year, Copper Sulfate was added to the raw water reservoir twice in 2014 (May 28 and July 3) and thrice in 2015 (May 13, July 8, and September 3). A maximum of 12 bags, each containing 25 kg of Copper Sulfate is typically stored on site.

Husky Raw Water Pump House

The Husky Raw Water Pump House is located adjacent to both the WTP and the raw water reservoir. This pump house is used to supply raw water to the Husky Lloydminster Upgrader (HLU) and is comprised of two horizontal split case pumps, each with a capacity of 91 L/sec (7,872 m^3/d) at a discharge pressure of 57 m. Prior to the 1999 upgrade, raw water supply for HLU was taken from the raw water reservoir. However, following the upgrade, the HLU was provided with only the option of taking raw water directly off the raw water supply pipeline. The split case pumps direct raw water from the raw water supply pipeline to the HLU via a 9 km long 350 mm diameter PVC pipeline. The HLU supply pipeline is rated for a maximum flow rate of 15,720 m^3 of raw water per day. At this flow rate, the velocity of the flow within the pipe will be approximately 1.9 m/s.

Other Raw Water Allocation

In addition to the HLU, raw water is drawn directly off the raw water pipeline and supplied for agricultural operations to the following establishments.

- Don Whiting Farm
- Jack Whiting Farm
- Manley Farms Ltd.
- L&A Farms
- Five L Farms Ltd.

Quantock Cattle Co. is an additional agricultural establishment that holds an account to draw raw water directly off the raw water pipeline; however historical data supplied by the City suggests that Quantock Cattle Co. has not drawn any raw water from the Pipeline since 2011.

Two additional businesses, Legion Ball Park and the City's golf course also draw raw water off the Husky Raw Water Pipeline.

Contingency and Source Water Protection Plans

The City of Lloydminster has developed a Waterworks Emergency Response Plan which ensures water quality, safety and adequate supply for consumers who use water from the drinking water system. The Emergency Plan is reviewed and updated on an annual basis and provides the City with an action plan to address emergencies or equipment failures that would affect source water, supply, distribution or the quality of drinking water for the City.

In addition to the plan above, the North Saskatchewan River Watershed Source Water Protection Plan has been developed by the Watershed Advisory Committees for the North Saskatchewan River watershed. The Committee's objective is to recognize and analyze any threats/issues to the North Saskatchewan River and develop recommendations and actions to address these threats.

As per ISL's correspondence with the City, no major issues with the quality of the source water have been observed or reported to date.

2.1.2 Water Treatment and Disinfection System

The water treatment and disinfection system (i.e. the Water Treatment Plant) is located east of 50th Avenue / Highway 17, on 67th Street in the Province of Saskatchewan. Construction of the treatment facility began in late 1981 and the process was commissioned in March of 1984. The WTP was designed to treat a flow of 30,000 m^3/d, when it was commissioned. As per the Water Treatment Plant Assessment completed in July 2003 by Associated Engineering, the rated net production capacity of the water treatment plant was established at 21,800 m^3/d. The net production capacity was reduced further to 20,125 m^3/d as a result of the Waterworks System Assessment performed by Worley Parsons in 2010. Based upon the available data from 2009 and 2014, the highest treated water flow produced by the WTP was 18,176 m^3/day in 2014. The WTP is typically operated for about 16 hours a day at an established flowrate, and the number of hours of operation per day are adjusted to account for variations in daily demands.

Originally the raw water from the raw water reservoir was pumped into the WTP using combinations of three vertical inline centrifugal pumps, each with the capacity to provide 173.5 L/sec of flow (14,990 m^3/day). However, due to the presence of significant head (elevation difference) between the raw water reservoir and the WTP, the City Operations' staff replaced one of the raw water pumps with a section of pipework. During normal operation, raw water now flows by gravity from the raw water reservoir into the WTP. Should the City experience high demands, the remaining two raw water pumps can be used to provide additional flow.

The WTP accomplishes treatment of raw river water by subjecting it to the following treatment stages:

- Coagulation
- Flocculation and clarification
- Media filtration
- Disinfection

Raw Water Gravity Feed

Figure 1.2 below provides a simplified schematic of the water treatment process employed at the City of Lloydminster.

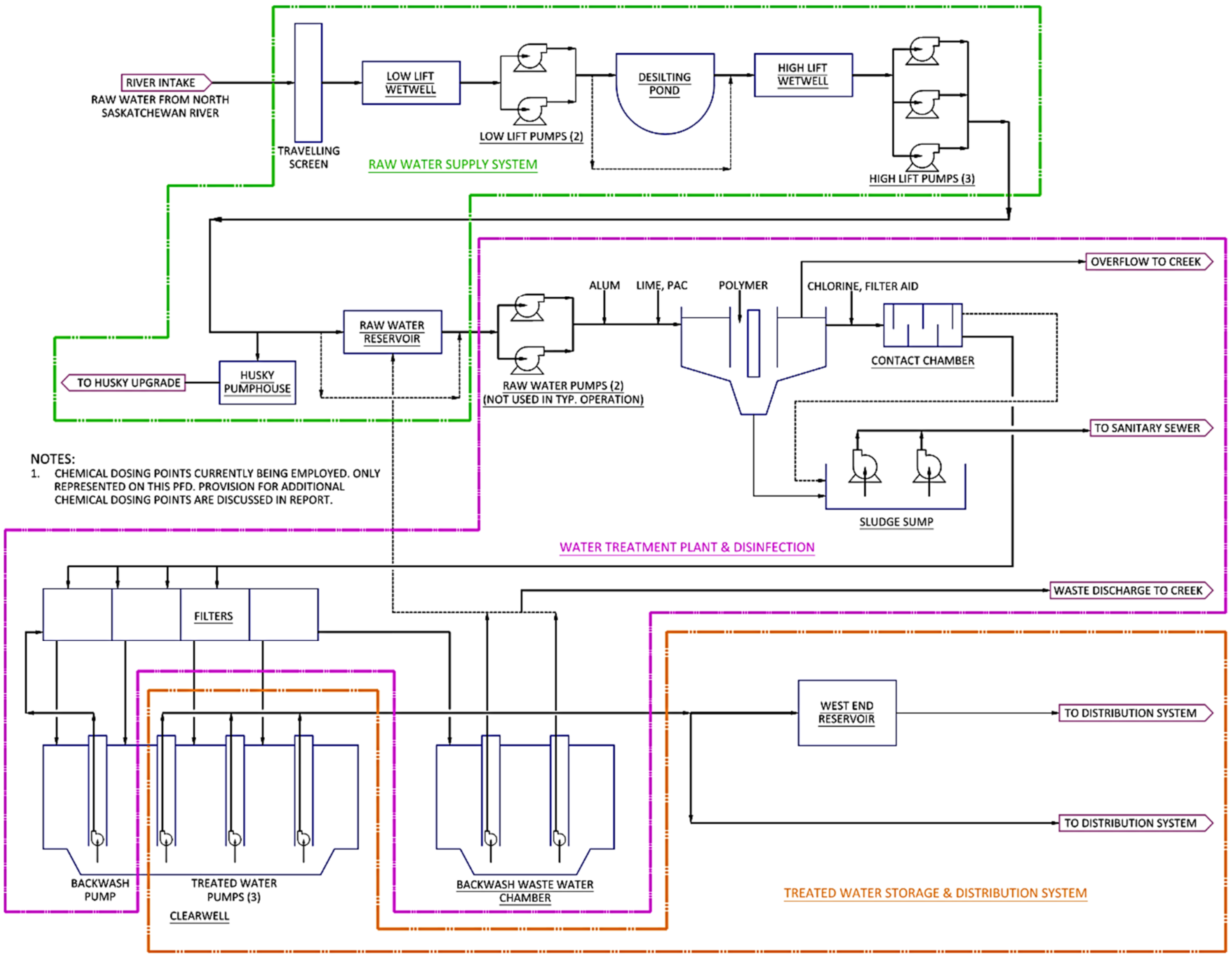


Figure 1.2: Water Treatment Schematic

Coagulation

Within this process stage, the coagulant Aluminum Sulphate (Alum) is first added to the water to neutralize the charge of particles and compounds within the water, which results in the formation of pin-floc. This is followed by the addition of Hydrated Lime which assists with the downstream flocculation process and adjusted the raw water's pH. With the capability of adding a maximum of 13 L/min of Alum and 2,000 kg/day of Lime, Powdered Activated Carbon can also be added up to a maximum rate of 2,000 kg/day to assist in the removal of organics and taste and odor compounds.

The WTP is equipped with a maximum Alum storage capacity of 65,000 kg. Currently, the facility typically stores about 40,000 kg of Alum on site. The average dose of Alum from January 1 2014 until December 21 2105 was 34.69 mg/L, which corresponds to an average raw water flowrate of 11,208 m^3/day for the same period. Based upon this flow and dosage, the WTP is equipped with an Alum storage capacity of approximately 167 days. As per EPB 201, every WTP is to ensure that a minimum of 30 days of inventory is available in the facility for any chemicals that are to be used in the treatment process. The Alum storage capacity exceeds the minimum 30-day chemical inventory as stated in the EPB 201. In addition, as per the NSF/ANSI Standard 60 the Maximum Use Level (MUL) of liquid Alum is 330 mg/L. The maximum dose of Alum applied between January 1 2014 and December 31 2015 was 88 mg/L, which is well below the assigned MUL. In reviewing this data with the City, it was confirmed that this high dosage did not actually occur, but was caused by a large Alum delivery that resulted in a false value.

With regards to Lime, the WTP is capable of storing a maximum of 27,750 kg, with approximately 20,000 kg of on site at any particular time. The average dose of Lime from January 1 2014 until December 31 2015 was 7.8 mg/L at an average raw water flowrate of 11,208 m^3/day. Based on this flow and dose, the WTP is equipped with a Lime storage capacity of approximately 317 days, which exceeds the minimum 30-day chemical inventory in a WTP as stated in the EPB 201. The Lime supplied to the facility is certified as a direct food substance by the FDA and no MUL has been identified

Alum, Carbon and Hydrated Lime Dosing Location

The City's WTP has the capability to store up to 3,000 kg of Powder Activated Carbon (PAC) on site. The average dose of PAC applied between January 1 2014 and December 31 2015 was 1.88 mg/L at the average raw water flow conditions. On this basis the WTP can store approximately 142 days of storage at this usage. This exceeds the minimum 30-day chemical inventory in a WTP as stated in the EPB 201. With regards to the MUL for the PAC, no maximum concentration is noted within the MSDS.

Flocculation and Clarification

Following coagulation, the next process stage in the City's WTP consists of a single solids contact clarifier where the coagulated water undergoes flocculation and settlement. An anionic polymer is added to the water within the mixing zone of the clarifier, which encourages the micro floc to grow into macro-floc and settle using the recirculation of solids that are already within the clarifier. At the manufacturer's recommended maximum loading rate of 4.2 m/h and with an effective surface area of 210 m^2, the clarifier is currently rated to treat up to 21,168 m^3/day, based upon a 24 hours of continuous operation.

Solids Contact Clarifier

In the event that the single clarifier has to be taken off line for maintenance, the clarifier can be bypassed and direct filtration used to treat the water. Once isolated, the two sludge pumps that are used to waste excess sludge from the clarifier to the sanitary sewer are used to completely drain the clarifier.

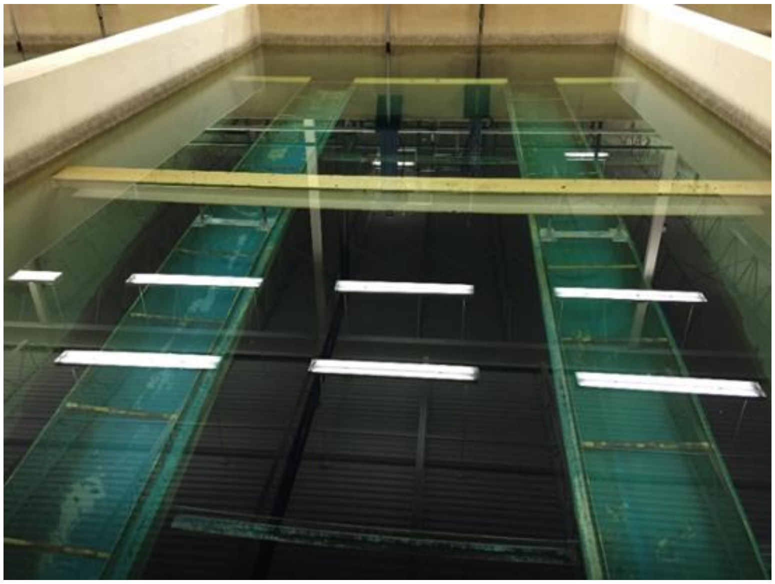
A total of 408 kg of polymer is stored on site. The average dose of polymer between January 1 2014 and December 31, 2015 was 0.68 mg/L at average flow conditions. On this basis the WTP stores approximately 53 days of polymer on site, which exceeds the minimum 30-day chemical inventory required within EPB 201. The Maximum Use Level of the Polymer employed at the WTP (Clearfloc AE3055) is 1.0 mg/L, which in reviewing the daily values appears to be exceeded a number of times during the provided data set. Upon reviewing with the City it has been clarified that these "exceedances" are a result of the batch counter which is sensitive and registers the make-up of a batch when a power interruption occurs, or when maintenance is performed on the system. As such the identified exceedance of the MUL did not occur.

Filtration

During normal operation, the water leaving the clarifier is directed to a set of four dual media filters that operate in parallel. Any solids that are not removed via the conventional solids contact clarifier are removed during this stage. At the manufacturer's recommended maximum loading rate of 11.2 m/hr and with an effective filtration area of 27.0 m^2, all four filters have a combined capacity of 28,995 m^3/day based upon 24 hours of operation. With only three filters online this reduces to 21,747 m^3/day.

The four media filters are backwashed using a separate air and water backwash cycle. Once isolated a single air blower is used to agitate the media and solids. Once agitated the blower is switched off a single backwash pump (rated at 1,070 m^3/hr) is used to flush the solids out of the filter before the filter is filter to waste and returned to service. All backwashes are currently performed manually by WTP Operations Staff based on the head loss across the filter, or the length of time the filter has been in service. Typically, any filter that has been in service for more than 8 days is backwashed.

As the water leaves the solids contact clarifier filter aid is added at a dose of 0.05 mg/L to help promote the filtration process. Currently, the filter aid is withdrawn directly from a 55 Gallon drum which contains 204 kg of the chemical. Typically one drum is in use, with a second full drum immediately available for use. Based on the average daily raw water flowrate and the applied dose above, approximately 357 days of filter aid is provided by a single drum. This exceeds the minimum 30-day chemical inventory in a WTP as stated in the EPB 201. As per the NSF/ANSI Standard 60, the Maximum Use Level of the Filter Aid employed at the WTP (CTI CL2410) is 50 mg/L. The maximum dose of this filter aid that the City historically applied was 0.1 mg/L, which is well below the assigned MUL.



Dual Media Filter

Disinfection

Gaseous Chlorine is used at the City's WTP to achieve disinfection of the water prior to entering the distribution system. The operations team has the option to inject Chlorine either at the outlet of the solids contact clarifier, or at the outlet of the clearwell. Each dosing point has a dedicated chlorinator, each with a capacity of 90 kg/day which are interconnected to provide a stand-by capacity if required.

The WTP has the capacity to store up to 12 Chlorine Drums (Tonners) on site, each holding 907 kg of liquefied Chlorine. Only three Tonners are currently stored at the facility, with two drums always online. Both online drums are located on weigh scales to allow monitoring of the amount of chlorine used, and are both connected to vacuum regulators in a duty / stand-by configuration.

Under normal operating conditions, Chlorine is added to the water as it leaves the clarifier, in proportion to the total filter outlet flowrate (i.e. flow paced). The addition of Chlorine is set such that the water passing through the media filters and the downstream clearwell achieve a specific free chlorine residual. Should the free chlorine residual drop through the filters and clearwell, additional chlorine can be added at the exit of the clearwell.

As the filter outlet flowrates are not recorded, the raw water flowrates into the WTP will be used to assess the addition of chlorine in this Section, as at the point of chlorine addition, the large withdraw of partially treated water due to filter washing has not occurred. As noted previously the average daily raw water flowrate from January 1 2014 until December 31 2015 was 11,208 m^3/day. Based upon the recorded usage the average chlorine dose of Chlorine applied over that time period was 2.15 mg/L. Based upon the above information and 12 drums onsite, the WTP is equipped with a chlorine storage capacity of approximately 452 days. With either three or two tonners on site this storage capacity reduces to 113 and 75 days respectively. Under all these scenarios this exceeds the minimum 30-day chemical inventory in a WTP as stated in the EPB 201.

Within NSF/ANSI Standard 60 the Maximum Use Level applied for Chlorine Gas is 30 mg/L. The maximum dose of Chlorine applied between January 1 2014 and December 31 2015 was approximately 5.6 mg/L, which was a single day event and well below the assigned MUL.

Disposal of Waterworks Generated Wastewater

Excess/Waste sludge from the clarifier is disposed to the sanitary sewer, which is discharged into the inlet of the City's Wastewater Treatment Facility.

Wastewater generated from backwashing the filters is pumped to a natural stream adjacent to the WTP. As per the recommendation by the 2005 WSA, the City undertook a downstream impact assessment study, which consisted of measuring the Free Chlorine Residual within the natural stream downstream of the discharge of backwash water. No chlorine residual was detected at this location. Therefore, it was concluded that disposal of the backwash generated wastewater into this stream did not result in any environmental concerns and the City can continue disposal of this wastewater though this route.

2.1.3 Treated Water Storage and Distribution System

The following sections will provide an outline of the Treated Water Storage and Distribution System for the City of Lloydminster.

Water Treatment Distribution Pumps

Three (3) vertical turbine pumps, provided in a two duty / one standby configuration, pump the treated water from the clearwell and into the distribution system. Two of the three pumps are operated using constant speed drives and are rated to operate at a duty flow of 16,532 m^3/d at 61 m. The third pump is operated using a Variable Frequency Drive (VFD) and is rated as operating at a duty flow of 13,488 m^3/d at 59 m.

The clearwell from which the Water Treatment Distribution Pumps draw has a capacity of 1,090 m^3. This clearwell holds treated water, however the contents of the clearwell are primarily used as a source of filter backwash water. Due to the small volumetric capacity of the clearwell, it does not contribute towards the treated water storage capacity of the distribution system.

West End Reservoir and Distribution Pumps

Treated Water from the WTP is pumped into the distribution system to meet the treated water demands of the City's residents and commercial users, and to refill the West End Reservoir which stores treated water to support high system and fire flow demands.

The West End Reservoir has a total storage capacity of 24,746 m^3. An above ground concrete structure constructed in 1974 provides 4,545 m3 of storage, with an additional 20,201 m^3 of storage provided within two below ground reservoirs which were installed in 2006. Treated water from the West End Reservoir can be supplied to distribution system using combinations of the four pumps installed adjacent to the reservoir. Each pump has a capacity of 103 L/sec (371 m^3/h) at 43.2 m. Two of the pumps are operated using variable frequency drives and two by constant speed drives.

During periods of high demand (i.e. during the day), treated water is primarily supplied into the distribution system from the WTP, with the distribution pumps at the West End Reservoir making up any shortfall. As the demands declined towards the end of the day, the excess treated water from the WTP is used to replace the treated water that has been provided by the West End Reservoir during the day. Typically, the WTP shuts down at approximately 11:00 pm each night, with the overnight demands of the City met by the West End Reservoir only, until 6:00 am the following morning when the WTP restarts. If necessary, it is possible to easily run the WTP for longer hours to meet criteria of higher than typical demands.

To clarify, there is no control link or control communications between the West End Reservoir and the WTP, such that the West End Reservoir can cause the WTP to automatically shut down. In addition, while the West End Reservoir is being filled, it is not possible to supply the distribution system with water from the West End Reservoir.

2.1.4 Operations and Maintenance

The City maintains up to date records of all maintenance procedures electronically, which includes a description of the maintenance work being performed, the materials and time spent on performing maintenance. Standard Operating Procedures (SOPs) for operation and maintenance are accessible and up to date.

3.0 Disinfection Assessment

As per Saskatchewan Water Security Agency document EPB 201, water treatment facilities employing coagulation, flocculation, sedimentation, filtration, and disinfection stages to treat a surface water sources must ensure that a 3-log reduction of *Giardia Lamblia* cysts and *Cryptosporidium parvum* oocycts, and a 4-log reduction of viruses is achieved by the whole treatment process.

The guiding principle in that log reductions are to be achieved through the application of multiple treatment process stages to provide multiple barriers. Within the system operated by the City of Lloydminster, these required log reductions are achieved through the application of 2 stages:

- Coagulation, flocculation clarification and media filtration, and
- Chlorine and contact time.

Using a combination of clarification and media filtration, the turbidity of the water leaving each media filter maintained below 0.3 NTU as prescribed within the City's Permit to Operate a Waterworks. The clarification and media filtration steps provide the majority of the log reduction for Cryptosporidium and Giardia, leaving the disinfection stage, comprised of chlorine and contact, to provide required 2-log reduction in viruses and a 0.5-log reduction in Giardia as stated within SWSA document EPB 501.

Within the City of Lloydminster's treatment process, chlorine is added at the inlet channel of the media filters. The water subsequently passes through the filters and into the clearwell, before being pumped into the distribution system. Once discharged from the distribution pumps, the first customer is 1,213 m from the WTP and as such the City uses the intermediate pipeline length as part of its disinfection process. The free chlorine residual within this step is measured and monitored as the treated water is discharged from the distribution pumps.

The log reduction achieved by a treatment process using a chlorine-based disinfection system is a function of a several factors including the disinfectant applied, the baffling of the tank / pipe, temperature, pH, free chlorine residual, contact volume and the outlet flowrate. The City of Lloydminster records the peak hourly flowrates from the WTP which have been recorded as a maximum of 954 m^3/hr (265 L/sec) in July 2011. As this report looks at both the historical performance and future possibilities, the rated capacity of the existing backwash and distribution pumps exceeds this historical peak hourly flow and will therefore be used for this assessment. In addition, using the context of which pumps are running will provide clarity, as the objective is to also highlight potential scenarios where one of the variables stated above needs to be monitored and adjusted to account for a limitation of another (i.e. at high flows and elevated chlorine residual is required).

Table 3.1: Rated Capacity of Distribution and Backwash Pumps

	Rated Capacity (L/sec)	Rated Capacity	Number Installed
Small Distribution Pump	156	13,488 m3/day	1
Large Distribution Pump	191	16,532 m3/day	2
Backwash Pump	297	1,070 m3/hr	1

The backwash pump that is used to wash the media filters, withdraws water from the clearwell at a rate of 1,070 m^3/hr for the first 6 to 10 minutes of the upwash phased, before reducing to 300 m^3/hr for a further 10 minutes to regrade the media. The backwash supply pump takes water from the clearwell partway through the disinfection step, just before it enters the pipeline. The City has confirmed that when the backwash supply pump is running, the larger distribution pump is switched off such that only the smaller distribution pump supplies water to the distribution system.

Therefore, under these conditions the maximum flow of water leaving the clearwell only could be 1,562 m^3/hr. As this will exceed the 600 to 800 m^3/hr entering the clearwell, its water level will drop during the first 10 minutes of the up-wash phased of the filter backwash cycle. Based upon a clearwell foot print of 352 m^2, a clearwell inlet flow of 600 m^3/hr and a backwash pump run time of 10 minutes, the clearwell level will drop approximately 0.488 m during the first 10 minutes of the upwash phase before recovering.

3.1 Virus Log Reduction

The SWSA requirements state that in that in order to provide the required log reduction for viruses, a specific CT value must be achieved. The required CT values vary upon the process conditions applied and the contaminant being addressed. For example, to achieve a 3-log reduction in viruses using a free chlorine residual at a water temperature of 0.5 degrees Celsius and a pH between 6.0 to 9.0, a CT value of 9 mg-min/L must be achieved.

Comparing this to the situation where a 3-log reduction in Giardia is required using free chlorine residual of 1.0 mg/L at a water temperature of 0.5 degrees Celsius and a pH 6.0, a CT value of 148 mg-min/L must be achieved. By just change in the pH from 6.0 to 9.0 the CT value for the same contaminant and the same remaining process conditions increases to 437 mg-min/L. As can be observed the CT requirements for Giardia are higher than those for viruses and vary considerably more depending upon the actual process conditions applied.

In considering the virus log reduction from January 2009 to December 2015, the minimum temperature of the water was recorded as 0.5 degrees Celsius and the pH remained between 6.0 and 9.0. SWSA documents EPB 201 and EPB 501 state that a 4.0-log reduction in viruses must be provided by all treatment processes that are supplied by a surface raw water. The same documents state that a 2.0-log reduction in viruses can be recognized for a conventional sedimentation / filtration (i.e. Lloydminster's first process stage) and a further 2.0-log reduction must be provided across the disinfection stage.

To provide a more stringent analysis of the system (both historical and future) ISL will be assessing on the basis that only a 4-log reduction across the disinfection stage is required, which according EPB 201 and EPB 501 requires a CT of 12 mg-min/L for a water temperature of 0.5 degrees Celsius and a pH between 6.0 and 9.0.

The remaining components of the CT calculation include peak hourly flowrates, chlorine residual and baffling factors. As noted above, the capacity of the pumping equipment will be used for this analysis instead of historical peak hourly flows. With regards to the condition where the filter backwash supply pump is running it is important to note that the high flowrate will only be applied to the clearwell, as the water for filter washing will be removed at the end of the clearwell. As such the flow from one small distribution pump will be applied to the pipeline section under this scenario.

Therefore, for the purpose of this evaluation four flow conditions will be evaluated as per Table 3.2 below.

Table 3.2: Disinfection Flow Scenarios

	Water Flowrate (L/sec)	Water Flowrate (L/min)
One Small Distribution Pump	156	9,367
One Small and One Large Distribution Pump	347	20,847
Two Large Distribution Pumps 1	383	22,691
One Small Distribution Pump and One Backwash Pump2	453	27,200

1. This is a theoretical scenario as at this time, it is not possible to run two large pumps due to a restriction within the distribution system.
2. Stated flowrate to be applied to the clearwell only, as backwash water is removed from the clearwell. Under these circumstances the flow from one small distribution pumps will be applied to the pipeline.

For chlorine residual, the 2009 to 2015 data was reviewed and the maximum, minimum and average values were identified as shown in Table 3.3 below. These values will each be applied to each of the flow scenarios above as part of the evaluation.

Table 3.3: Free Chlorine Residuals, 2009 to 2015

	Free Chlorine Residual (mg/L)
Minimum	0.64
Average	1.12
Maximum	1.92

To establish the contact volume of the disinfection stage, the 1,213 m section of pipeline and the clearwell will be used within the assessment, with the upstream filter volume providing an undefined safety buffer. The pipeline component of the contact volume is comprised of a section of pipe 30 inches in diameter and 1,050 m long, and a second section 300 mm dimeter and 163 m long. This provides a total volume of 1,962 m^3 and due to the high width to length ratio encountered within the pipeline, a baffling factor of 1 can be assigned.

The clearwell located after the filters serves as both a contact tank and pump wet well. The operating objective is to maintain the clearwell level at 2.50 m. There is a low level alarm at 1.50 m and a high level alarm at 2.95 m. The WTP will automatically such down when the clearwell level reaches 3.0 m. Using the record drawings for the WTP, the clearwell has a footprint of 352 m^2 (17.45 m by 20.15m). Using the information above, Table 3.4 calculates the volumes at the different water levels that can applied.

Table 3.4: Clearwell Volumes

Clearwell Water Level (m)	Water Volume (m3)	Water Volume (L)
1.5	527	527,426
2.0	703	703,235
2.5	879	879,044
2.95	1,037	1,037,272
3	1,055	1,054,853

For the purpose of this evaluation the basis has been applied that the clearwell level is maintained at a depth of 2.50 m for scenarios where only the distribution pumps are running. For the scenario where the backwash pump is running, the reduction in clearwell level due to the filter washing will be accounted for by using a clearwell level of 2.0 m to calculate the contact volume. The clearwell itself does have a number of compartments, however no specific baffling or isolation points exist. Therefore, a baffling factor of 0.3 (poor) will be applied.

Tables 3.5 to 3.7 provide the calculation and CT ratios with regards to a 4-log reduction in viruses for the clearwell, pipeline and the overall process respectively. The calculation performed within these table is based upon

$$CT\ Value = \frac{Volume}{Flowrate} \times Baffling\ Factor\ \times Free\ Chlorine\ Residual$$

Table 3.5: Clearwell Virus CT Calculation (4-log Reduction, 0.5 ⁰ C, pH of 6.0 to 9.0, Baffling Factor of 0.3)

Flow Condition	Flow rate (L/sec)	Flow rate (L/min)	Contact Volume (L)	Time (T10) (Minutes)1	Free Chlorine Residual (mg/L)		CT (mg-min/L)	CT Ratio
One Small Pump	156	9,367	879,044	28.2	Min	0.64	18.0	1.5
					Ave	1.12	31.6	2.6
					Max	1.92	54.1	4.5
One Small and One Large Pump	347	20,847	879,044	12.6	Min	0.64	8.1	0.7
					Ave	1.12	14.2	1.2
					Max	1.92	24.3	2.0
Two Large Pumps2	383	22,961	879,044	11.5	Min	0.64	7.4	0.6
					Ave	1.12	12.9	1.1
					Max	1.92	22.1	1.8
One Small Pump and One Backwash Pump3	453	27,200	703,235	7.8	Min	0.64	5.0	0.4
					Ave	1.12	8.7	0.7
					Max	1.92	14.9	1.2

1. T10 = Contact Time × Baffling Factor
2. This is a theoretical scenario this time as it is not possible to run two large pumps due to a restriction within the distribution system.
3. Clearwell level at 2.0m instead of 2.5m

Table 3.6: Pipeline Virus CT Calculation (4-log Reduction, 0.5 ⁰ C, pH of 6.0 to 9.0, Baffling Factor of 1.0)

Flow Condition	Flow rate (L/sec)	Flow rate (L/min)	Contact Volume (L)	Time (T 10) (Minutes)1	Free Chlorine Residual (mg/L)		CT (mg-min/L)	CT Ratio
One Small Pump	156	9,367	1,962,231	209.5	Min	0.64	134.1	11.2
					Ave	1.12	235.1	19.6
					Max	1.92	402.2	33.5
One Small and One Large Pump	347	20,847	1,962,231	94.1	Min	0.64	60.2	5.0
					Ave	1.12	105.6	8.8
					Max	1.92	180.7	15.1
Two Large Pumps2	383	22,961	1,962,231	85.5	Min	0.64	54.7	4.6
					Ave	1.12	95.9	8.0
					Max	1.92	164.1	13.7
One Small Pump and One Backwash Pump3	156	9,367	1,962,231	209.5	Min	0.64	134.1	11.2
					Ave	1.12	235.1	19.6
					Max	1.92	402.2	33.5

1. T10 = Contact Time × Baffling Factor
2. This is a theoretical scenario at this time as it is not possible to run two large pumps due to a restriction within the distribution system
3. Backwash water removed prior to entering pipeline. Only small distribution pump flow passes through pipeline. Clearwell level at 2.0m instead of 2.5m

Table 3.7: Combined Virus CT Calculation (4-log Reduction, 0.5 ^{0} C, pH of 6.0 to 9.0, Baffling Factor of 0.3 for Clearwell & 1.0 for Pipeline)

Flow Condition	Flow rate (L/sec)	Flow rate (L/min)	Contact Volume (L)	Time (T 10) (Minutes)1	Free Chlorine Residual (mg/L)		CT (mg-min/L)	CT Ratio
One Small Pump	156	9,367	2,841,275	237.6	Min	0.64	152.1	12.7
					Avg	1.12	266.6	22.2
					Max	1.92	456.3	38.0
One Small and One Large Pump	347	20,847	2,841,275	106.8	Min	0.64	68.3	5.7
					Avg	1.12	119.8	10.0
					Max	1.92	205.0	17.1
Two Large Pumps2	383	22,961	2,841,275	96.9	Min	0.64	62.0	5.2
					Avg	1.12	108.8	9.1
					Max	1.92	186.1	15.5
One Small Pump and One Backwash Pump3	453	27,200	2,665,466	217.2	Min	0.64	139.0	11.6
					Avg	1.12	243.8	20.3
					Max	1.92	417.1	34.8

1. T10 = Contact Time × Baffling Factor
2. This is a theoretical scenario at this time, as it is not possible to run two large pumps due to a restriction within the distribution system
3. Backwash water removed prior to entering pipeline. Only small distribution pump flow passes through pipeline. Clearwell level at 2.0m instead of 2.5m

The above tables demonstrate that the combined use of both the clearwell and the pipeline provided the required CT for a 4-log virus reduction. In the event that only the clearwell is used for disinfection then a free chlorine residual greater than 1.7 mg/L must be provided to achieve a CT of 12 mg-min/L under the maximum flow conditions defined above (i.e. with the backwash pump running).

3.2 Giardia Log Reduction

The application of a CT calculation for Giardia is more complex as the required CT values are highly dependent on the pH and temperature of the water, and its free chlorine residual. In order assess the worst case scenario, CT calculations were performed using the maximum recorded pH of 8.22 and the minimum water temperature of 0.5ºC. However, it is important to stress that under normal circumstances when the raw water cycles through its annual patterns, the low water temperatures that occur in winter do not coincide with the with the high pH values of summer. This can be observed within Figure 4.6.

In a similar approach to virus reduction, EPB 201 and EPB 501 state that water treatment processes that are supplied with raw water from a surface water must achieve a minimum 3-log reduction for Giardia, of which 0.5-log reduction must be achieve across the disinfection stage. Tables 3.8 to 3.10 below illustrate the variables used, the required CT, and the achieved CT ratios with regards to achieving a 0.5-log reduction for Giardia for the clearwell, pipeline and the overall process respectively. The 0.5-log reduction values were interpolated from the CT Tables provided within the Alberta Environment Standards and Guidelines 2012.

Table 3.8: Clearwell Giardia CT Calculation (0.5-log reduction, 0.5 ^{0} C, pH of 8.22, Baffling Factor of 0.3)

Flow Condition	Flow rate (L/sec)	Flow rate (L/min)	Contact Volume (L)	Time (T 10) (Minutes)1	Free Chlorine Residual (mg/L)		CT (mg-min/L)	CT Required (mg-min/L)	CT Ratio
One Small Pump	156	9,367	879,044	28.2	Min	0.64	18.0	52.2	0.3
					Avg	1.12	31.6	56.3	0.6
					Max	1.92	54.1	62.5	0.9
One Small and One Large Pump	347	20,847	879,044	12.6	Min	0.64	8.1	52.2	0.2
					Avg	1.12	14.2	56.3	0.3
					Max	1.92	24.3	62.5	0.4
Two Large Pumps2	383	22,961	879,044	11.5	Min	0.64	7.4	52.2	0.1
					Avg	1.12	12.9	56.3	0.2
					Max	1.92	22.1	62.5	0.4
One Small Pump and One Backwash Pump3	453	27,200	703,235	7.8	Min	0.64	5.0	52.2	0.1
					Avg	1.12	8.7	56.3	0.2
					Max	1.92	14.9	62.5	0.2

1. T10 = Contact Time × Baffling Factor
2. This is a theoretical scenario at this time as it is not possible to run two large pumps due to a restriction within the distribution system
3. Clearwell level at 2.0m instead of 2.5m

Table 3.9: Pipeline Giardia CT Calculation (0.5-log reduction, 0.5 ^{0} C, pH of 8.22, Baffling Factor of 1.0)

Flow Condition	Flow rate (L/sec)	Flow rate (L/min)	Contact Volume (L)	Time (T 10) (Minutes)1	Free Chlorine Residual (mg/L)		CT (mg-min/L)	CT Required (mg-min/L)	CT Ratio
One Small Pump	156	9,367	1,962,231	209.5	Min	0.64	134.1	52.2	2.6
					Avg	1.12	235.1	56.3	4.2
					Max	1.92	402.2	62.5	6.4
One Small and One Large Pump	347	20,847	1,962,231	94.1	Min	0.64	60.2	52.2	1.2
					Avg	1.12	105.6	56.3	1.9
					Max	1.92	180.7	62.5	2.9
Two Large Pumps2	383	22,961	1,962,231	85.5	Min	0.64	54.7	52.2	1.0
					Avg	1.12	95.9	56.3	1.7
					Max	1.92	164.1	62.5	2.6
One Small Pump and One Backwash Pump3	156	9,367	1,962,231	209.5	Min	0.64	134.1	52.2	2.6
					Avg	1.12	235.1	56.3	4.2
					Max	1.92	402.2	62.5	6.4

1. T10 = Contact Time × Baffling Factor
2. This is a theoretical scenario at this time as it is not possible to run two large pumps due to a restriction within the distribution system
3. Backwash water removed prior to entering pipeline. Only small distribution pump flow passes through pipeline.

Table 3.10: Combined Giardia CT Calculation (0.5-log Reduction, 0.5 ^{0} C, pH of 8.22, Baffling Factor of 0.3 for Clearwell & 1.0 for Pipeline)

Flow Condition	Flow rate (L/sec)	Flow rate (L/min)	Contact Volume (L)	Time (T 10) (Minutes)1	Free Chlorine Residual (mg/L)		CT (mg-min/L)	CT Required (mg-min/L)	CT Ratio
One Small Pump	156	9,367	2,841,275	237.6	Min	0.64	152.1	52.2	2.9
					Avg	1.12	266.6	56.3	4.7
					Max	1.92	456.3	62.5	7.3
One Small and One Large Pump	347	20,847	2,841,275	106.8	Min	0.64	68.3	52.2	1.3
					Avg	1.12	119.8	56.3	2.1
					Max	1.92	205.0	62.5	3.3
Two Large Pumps2	383	22,961	2,841,275	96.9	Min	0.64	62.0	52.2	1.2
					Avg	1.12	108.8	56.3	1.9
					Max	1.92	186.1	62.5	3.0
One Small Pump and One Backwash Pump3	453	27,200	2,665,466	217.2	Min	0.64	139.0	52.2	2.7
					Avg	1.12	243.8	56.3	4.3
					Max	1.92	417.1	62.5	6.7

1. T10 = Contact Time × Baffling Factor
2. This is a theoretical scenario at this time as it is not possible to run two large pumps due to a restriction within the distribution system
3. Backwash water removed prior to entering pipeline. Only small distribution pump flow passes through pipeline. Clearwell level at 2.0m instead of 2.5m

Table 3.8 to 3.10 demonstrate that a 0.5-log reduction in Giardia can be achieved using chlorine and contact at the Lloydminster WTP, however this does require both the clearwell and pipeline to be online. In reviewing this information it should be noted that the majority of the disinfection for Giardia is provided by the pipeline section, which compensates for the real world variation in the clearwell level. During situations where any two distribution pumps are running, a free chlorine residual of 0.64 mg/L provides only a 20 to 30% safety factor when an elevated pH occurs at the minimum temperature of 0.5^{0}C. However it must be noted as stated above that the occurrence of a high pH and a low temperature is very unlikely.

3.3 Removal of Clarifier from Service

Situations have previously occurred and may occur in the future where it is necessary for the City to remove the existing clarifier from service (i.e. unexpected repairs or critical maintenance). When the clarifier is by-passed and the filters are operating as a direct filtration process, coagulant is added to the filter inlet channel. SWSA Document EPB 501, Table 3.2 states that a correctly operating and performing direct filtration process can receive a 2.5-log reduction credit for Giardia and Cryptosporidium, and a 1.0-log reduction credit for viruses. When the clarifier is in service a 3.0-log reduction for Giardia and Cryptosporidium, and a 2.0-log reduction credit for viruses is recognized.

Therefore, when the clarifier is out of service and media filters are being performing as a direct filtration process, the disinfection stage must provide a 0.5-log reduction for Giardia and a 3.0-log reduction in viruses. As demonstrated within Section 3.1 and 3.2 above, both of these disinfection requirements for when the clarifier is removed from service can be achieved using the existing process. Should a further safety factor be necessary, the flow through the WTP can be reduced and the operation of the facility changed to 24 hours a day to increase the achieved CT values.

It is important to note that chlorine is not effective at providing a log-reduction for Cryptosporidium, and as such the log reduction for this parameter can be achieved through direct filtration only. Within the City's Permit to Operate, the log reduction requirements are not specified. Instead a surrogate requirement is placed on each filter's outlet turbidity, such that it is monitored continuously and is required to be less than 0.3 NTU for 95% of the time. The City has informed ISL that no contraventions of this limit occurred within the reporting period.

3.4 Bacteriological Sampling

In accordance with its Permit to Operate, the WTP is also required to provide a treated water that contains of 0 Total Coliforms per 100 mL and of 0 E. coli per 100 mL. As per the requirements of the Permit, the City conducts sampling and monitoring for bacteriological content in the treated water once a week. During the 2009 to 2015 period all samples of the treated water were reported that the presence of both total coliforms and E.coli were non-detectible.

4.0 Waterworks System Water Quality

As part of the Round 3 WSA, ISL assessed the January 2009 to December 2015 water quality data supplied by the City of Lloydminster for the following areas:

- Raw Water (river, desilting pond and raw water reservoir)
- Water Treatment Plant (post clarification, post filter and post disinfection)
- Treated Water within the distribution system

Within this section, a summary of water quality data for each of the areas listed above will be provided and reviewed.

4.1 Raw Water Quality

The City of Lloydminster conducts routine analysis and monitoring of its raw water quality, which is either directed to the City's WTP for treatment or to other consumers. The City monitors raw water quality at three locations:

- The North Saskatchewan River as the water enters the intake structure,
- The outlet of the desilting pond, and
- The outlet of the raw water reservoir.

At each location the following parameters are monitored:

- North Saskatchewan River
  - Turbidity, Color, Conductivity, pH, Total Coliform, and Escherichia coli (E. coli)
- Desilting Pond Outlet
  - Turbidity and Color
- Raw Water Reservoir Outlet
  - Turbidity, Color, Temperature, pH, UV Transmissivity, Total Coliform, E. coli, *Cryptosporidium* and *Giardia*

Table 4.1 below summarizes the 2009 to 2015 minimum, average, and maximum values for each of the raw water parameters listed above.

Table 4.1: Historical Raw Water Quality, 2009 to 2015

	River			Desilting Pond			Raw Water Reservoir		
	Min.	**Avg.**	**Max.**	**Min.**	**Avg.**	**Max.**	**Min.**	**Avg.**	**Max.**
Color (ACU)	4.50	351	18,200	5.50	197	2,763	3	66.7	1,790
Turbidity (NTU)	1.25	45	1,054	1.21	20	310	1.11	5.8	126
UV Transmittance (%)	-	-	-	-	-	-	8.6	79.0	97.5
pH	7.7	8.2	8.6	-	-	-	6.69	8.2	9.0
Temperature (°C)				-	-	-	0	8.3	22.3
Total Coliform (MPN/100 ml)	47	765	3,400	-	-	-	1	1,129	44,000
E. Coli (MPN/100 ml)	1	21	200	-	-	-	0	15	82
Cryptosporidium (oocysts/100 L)	-	-	-	-	-	-	0.9	4.1	28
Giardia (oocysts/100 L)	-	-	-	-	-	-	0.9	15	69
Conductivity (µs/cm)	312	379	493	-	-	-	-	-	-

This information has also been characterized graphically in the following pages to clearly illustrate the seasonal trends, to correlate any differences and identify any abnormalities.

Figure 4.1 illustrates the information for raw water turbidity from 2009 through to 2015 at all three locations. As can be seen the expected spike in river turbidity occurs at the start of each spring and continues to be observed until mid-summer. These peaks continue through the raw water system and can be observed at a lower level as the water passes through the desilting pond and the raw water reservoir. As such it can be concluded that the desilting pond is effectively removing a substantial portion of material that contributes to turbidity within the river water.

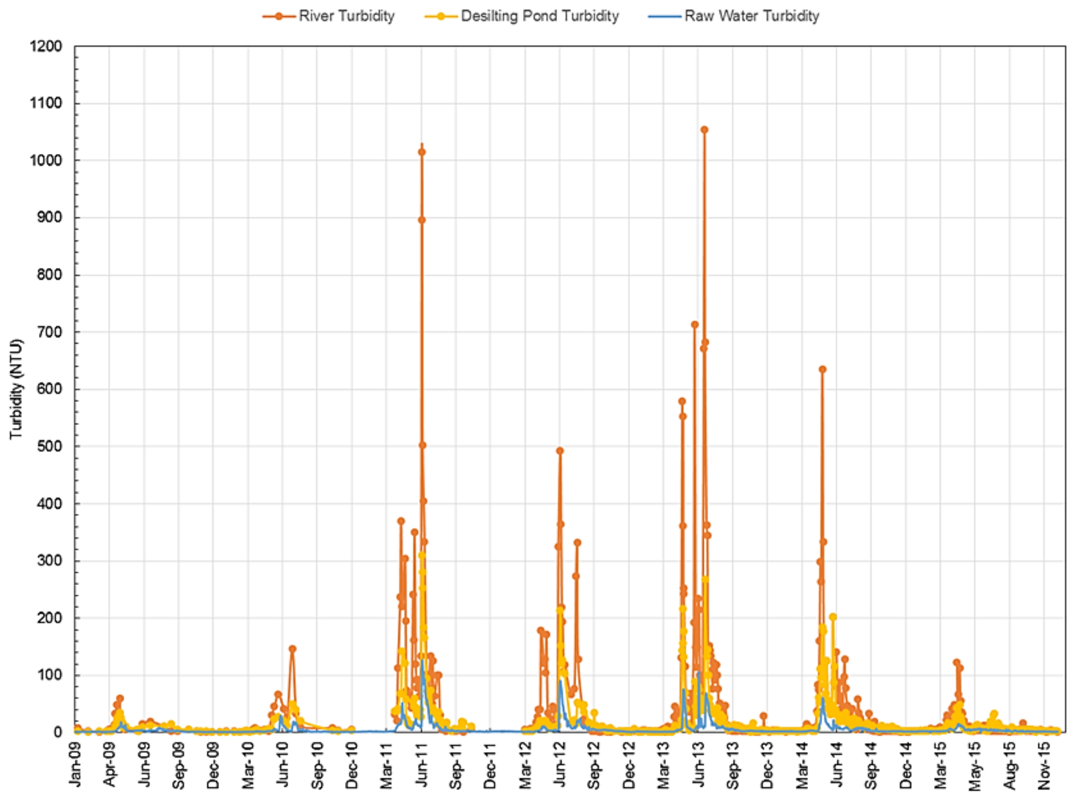


Figure 4.1: Historical River, Desilting and Raw Water Turbidities, 2009 to 2015

Figure 4.2 represents the total coliform and E. coli data for the same period for both the river water and the outlet of the raw water reservoir.

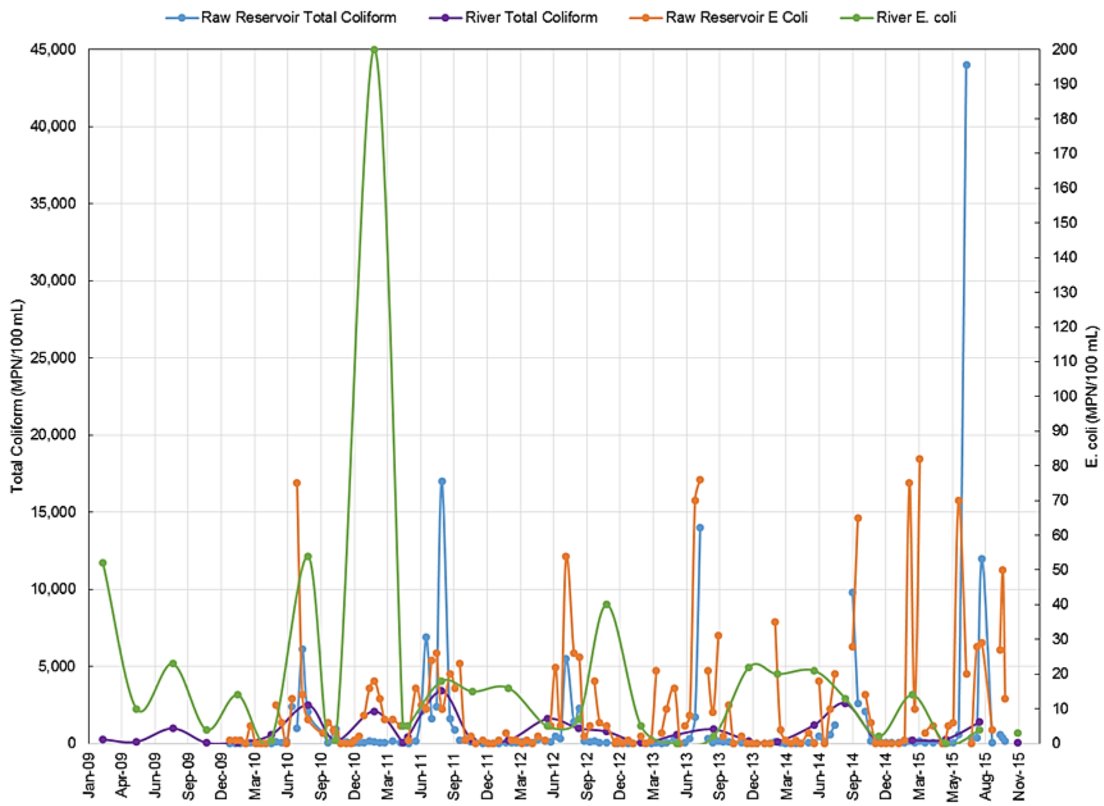


Figure 4.2: Historical River and Raw Water Bacteriological Data, 2009 to 2015

As is evident from the above figure, peaks in total coliform and E. coli are observed in both the river and the reservoir in the summer months. The figure above also shows that in summer, the numbers of total coliform and E. coli in the reservoir are greater than those observed in the river for a large number of instances. This increase is likely due to the presence of birds around the raw water reservoir during spring and summer. The City has installed a bird scaring device, in addition to working with the local conservation office to address the birds gathering around the raw water reservoir

Rodent activity around the raw water reservoir had been identified as an issue in the 2005 and 2010 WSAs. To clarify, this issue was with regards specifically to presence of muskrats within the raw water reservoir which has been addressed and is no longer an issue

Figure 4.3 below shows the results of the Cryptosporidium and Giardia samples from the raw water reservoir outlet from 2009 through to 2015. Whilst the higher concentrations align with the spikes in turbidity, the values of these concentrations provides no further relevance to the required log reduction values for these parameters, as the Saskatchewan Water Security Agency (SWSA) requires a standard 3-log reduction for facilities that use surface waters or ground waters under the direct influence of a surface water. This is interpreted by the application of specific turbidity requirements at each filter outlet as established by the SWSA.

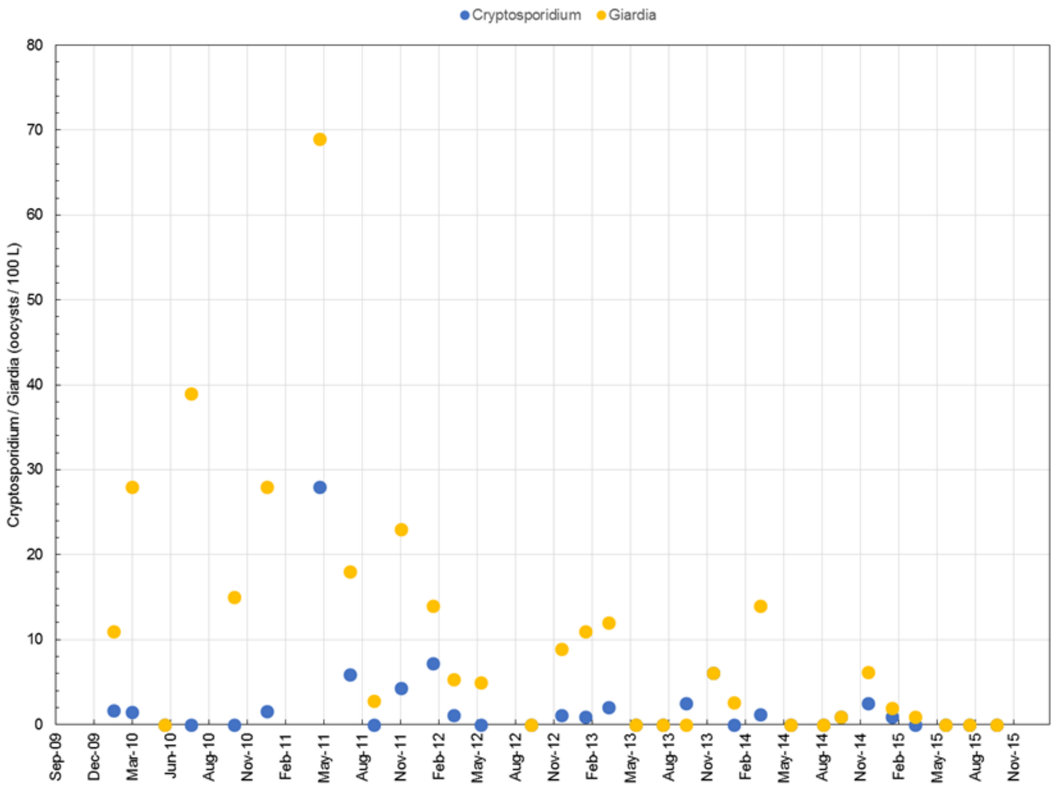


Figure 4.3: Raw Water Cryptosporidium and Giardia Concentrations, 2009 to 2015

In focusing on the raw water which enters the WTP, Figure 4.5 below illustrates the reservoir outlet results for UV transmittance, turbidity and colour. As expected, the drop in transmittance corresponds with peaks in colour and turbidity. This indicates that actual colour rather than true colour is being measured and that changes in UV transmittance is associated with the normal natural cycle.

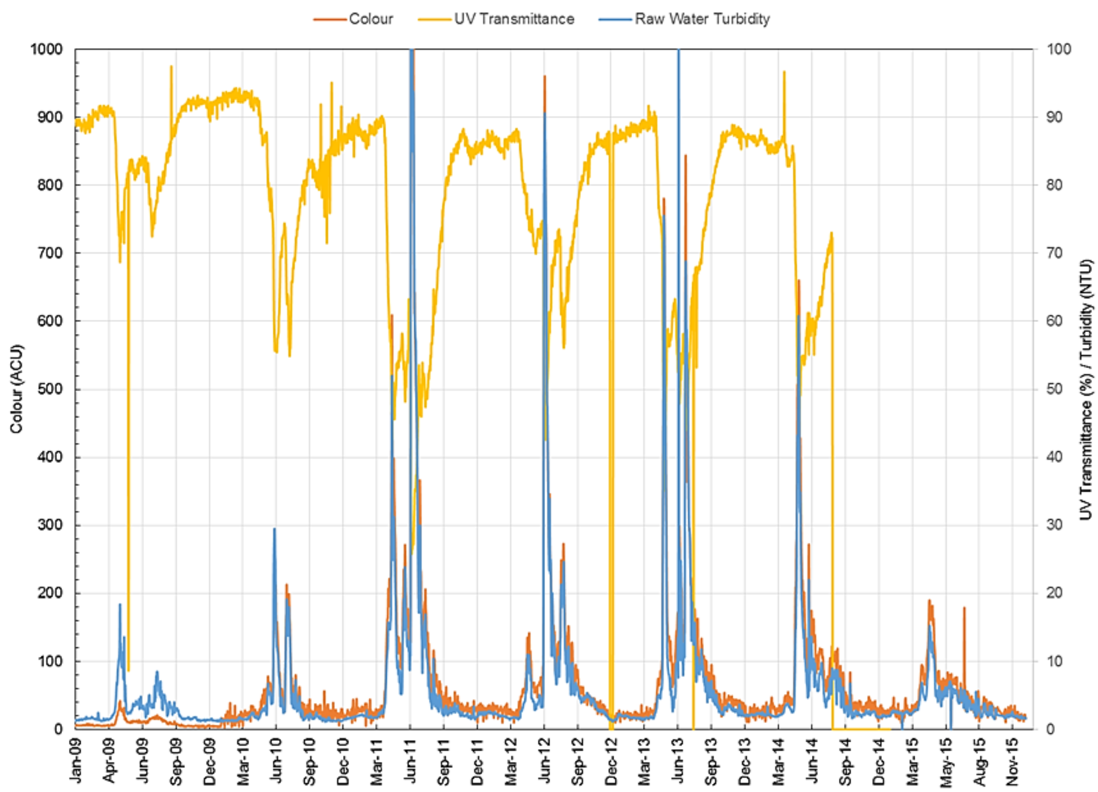
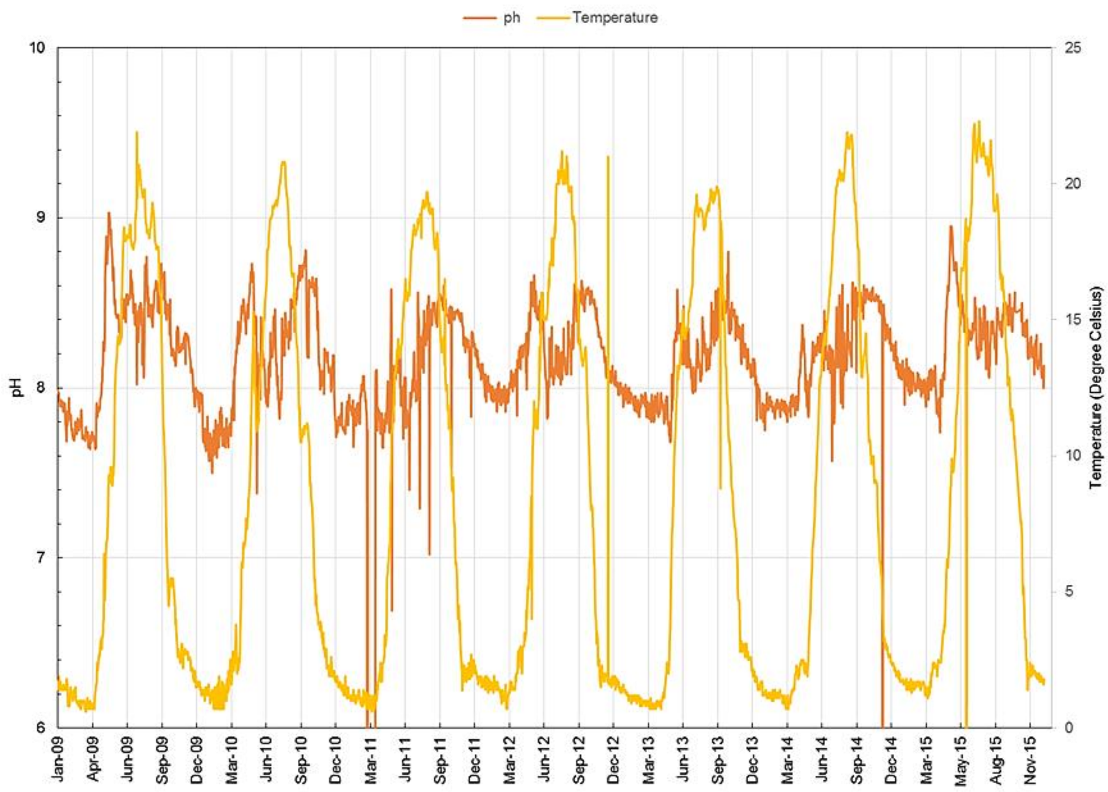


Figure 4.4: Raw Water UV Transmittance, Turbidity and Colour, 2009 to 2015

With regards to the pH and temperature of the raw water, no abnormalities have been noted within Figure 4.6 below. There was a significant spike in pH in 2009 and 2015, which exceeded a pH of 9 in 2009. Whilst this could have been an anomaly, any reoccurrence to this high value would warrant further investigation as to the source and assessment would be required to determine the effect a continued elevated pH would have on the treatment process.



Figures 4.5: Raw Water pH and Temperature, 2009 to 2015

In summary and based upon the information presented above, the raw water withdrawn from the North Saskatchewan River cycles annually in a fashion that is typical for a large, glacier fed Western Canada River. There is no indication from the analysis undertaken that indicates that there should be any concerns with regards to its treatability and the production of potable water for the City.

4.2 Water Treatment Plant Quality

As discussed in Section 2 of this document, the City's existing WTP provides treatment by subjecting the water from the North Saskatchewan River to coagulation, flocculation, clarification, filtration, and disinfection. ISL has reviewed and summarized the water quality data supplied by the City from 2009 through to 2015. Within the WTP, the following parameters are analyzed at the following locations:

- Clarifier Effluent
  - Turbidity, Colour, pH, UV Transmissivity, Total Coliform, E. coli and Aluminum
- Filter Outlet
  - Continuous turbidity monitoring on each filter outlet
- Treated Water

- Turbidity (grab and continuous), Colour, Temperature, pH, Alkalinity, Hardness, UV Transmissivity, Free Chlorine (Cl_2) (grab and continuous), Total Chlorine (Cl_2), Nitrogen, Bicarbonate, Sulfate, Sodium, Calcium, Chloride, Fluoride, Total Dissolved Solids (TDS), Manganese, Aluminum, Arsenic, Barium, Boron, Cadmium, Chromium, Copper, Lead, Selenium, Uranium, Zinc, Cyanide, Mercury, Total Halo Acetic Acids, Total Organic Carbon (TOC), Dissolved Organic Carbon (DOC), Synthetic Organic Chemicals, and Pesticides

Table 4.2 below summarizes the 2009 to 2015 minimum, average, and maximum values for each of these parameters, which are obtained from grab samples. The raw water values previously noted have also been added to this table to provide context.

Table 4.2: Historical WTP Water Quality, 2009 to 2015

	Raw Water			Clarifier Effluent			Treated Water		
	Min.	Avg.	Max.	Min.	Avg.	Max.	Min.	Avg.	Max.
Color (ACU)	3	66.7	1,790	< 1	5.43	107.6	< 1	1.47	16
Turbidity (NTU)	1.11	5.8	126	0.05	0.31	2.69	0.02	0.04	0.26
UV Transmittance (%)	8.6	79.0	97.5	2.4	90.18	97.4	79.6	92.70	99.9
pH (unitless)	6.69	8.2	9.0	6.51	7.64	13.8	6.6	7.54	8.22
Temperature (°C)	0	8.3	22.3	-	-	-	0.5	8.65	22.2
Total Coliform (MPN/100 ml)	1	1,137	44,000	0	154.78	6400	-	-	-
E. Coli (MPN/100 ml)	0	15	82	0	4.02	23	-	-	-
Free Chlorine (mg/L)	-	-	-	-	-	-	0.64	1.12	1.92
Total Chlorine (mg/L)	-	-	-	-	-	-	0.82	1.31	2.16
Alkalinity (mg/L of CaCO3)	-	-	-	-	-	-	18	118.00	152
Hardness (mg/L of CaCO3)	-	-	-	-	-	-	19	182.67	236
Manganese (mg/L)	-	-	-	-	-	-	0.0005	0.02	0.067
Aluminum (mg/L)	-	-	-	0.016	0.15	0.453	0.001	0.05	0.342
Total Halo Acetic Acids (µg/L)	-	-	-	-	-	-	6.7	22.52	61
Total Organic Carbon (mg/L)	-	-	-	-	-	-	1	2.18	4.6
Dissolved Organic Carbon (mg/L)	-	-	-	-	-	-	1	2.18	4.7

Figure 4.7 illustrates the turbidity of water within the clarifier outlet and treated water, from the daily grab samples that were taken at the WTP from 2009 through to 2015.

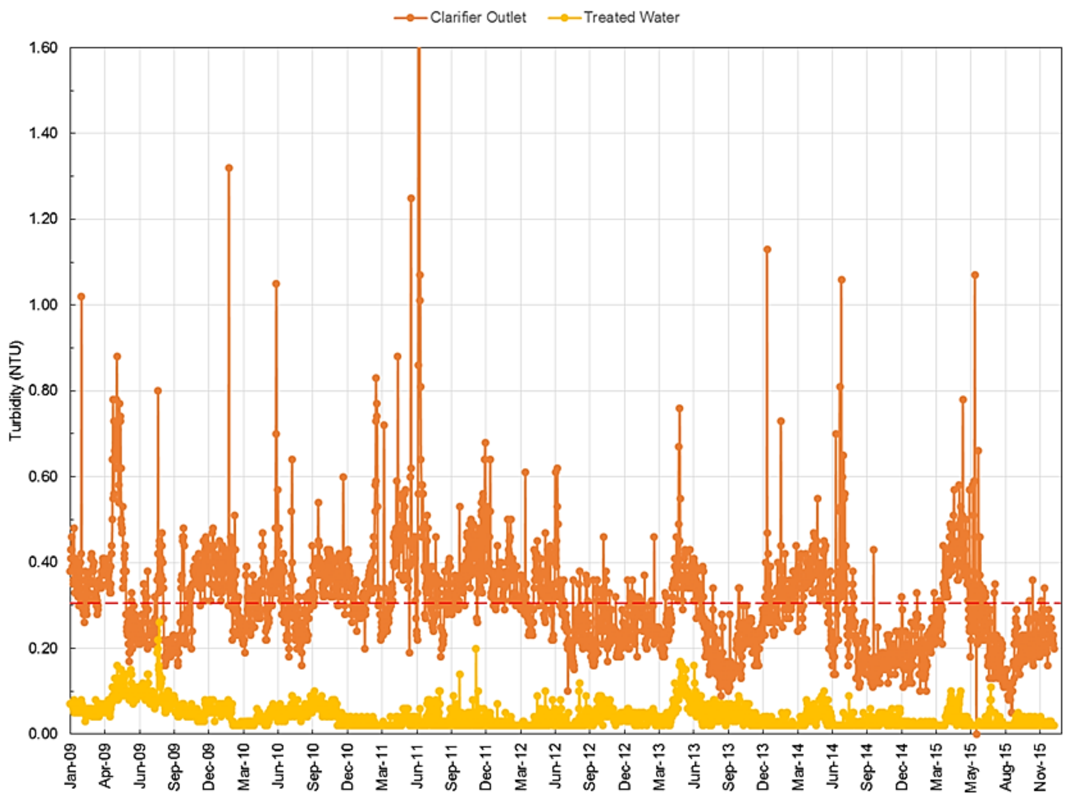


Figure 4.6: Historical WTP Turbidity, 2009 to 2015

The Water Regulations (2002) and the City's Permit to Operate Waterworks (2011) for a surface water source with a monthly average turbidity of 1.5 NTU or greater using chemically assisted filtration, state that the filter outlet turbidity of each individual filter must be less than 0.3 NTU. As the City continuously monitors each filter's outlet turbidity, the standard of less than 0.3 NTU must be met 95% of the time.

As can be seen from the figure above, the effluent turbidity standard was consistently met. The highest effluent turbidity of 0.26 NTU was recorded in August 2009.

Equally important parameters within the water treatment process include the bacteriological content of the treated water and its free chlorine residual. No non-compliances were reported with regards to the presence of total coliforms and E. coli within the treated water during the reporting period. Figure 4.7 below shows the free chlorine residual within the treated water from 2009 through to 2015. The Water Regulations (2002) and the City's Permit to Operate Waterworks (2011) stated that a minimum free chlorine residual of 0.1 mg/L must be maintained for all water entering a distribution system. The figure below is based upon the results of the daily grab samples. Between January 2009 and December 2015 no issues have been reported with regards to failure to maintain the free chlorine residual within the treated water and no bacteriological failures occurred with regards to the water leaving the WTP.

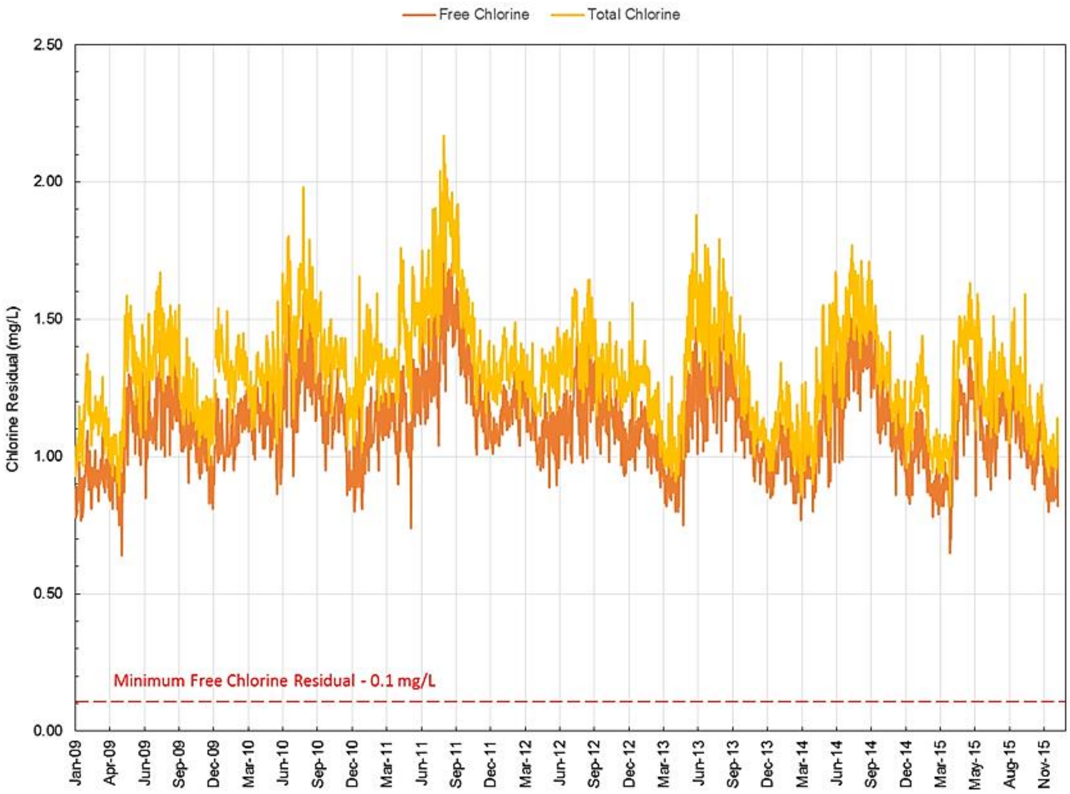


Figure 4.7: Historical Treated Effluent Free Chlorine Residual, 2009 to 2015

Within the City's Permit to Operate a Waterworks (2011) there are additional requirements to monitor and report organics, ions, metals and pesticides within the treated water, as per the sampling schedule defined within the Permit. Table 4.3 below summarizes the minimum, average and maximum concentration of the ions and metals specified and compares them with their respective Maximum Acceptable Concentration (MAC) or Interim Maximum Acceptable Concentration (IMAC). As can be noted within the table, no contraventions were noted.

Table 4.3: Historical Treated Water Metal and Ion Concentrations, 2009 to 2015

	Minimum	Average	Maximum	MAC	IMAC
Arsenic (µg/L)	0	0.188	0.300	10	
Barium (mg/L)	0.052	0.107	0.520	1	
Boron (mg/L)	0.010	0.019	0.030		5
Cadmium (mg/L)	0	<0.001	< 0.001	0.005	
Chromium (mg/L)	0.0005	0.0	0.0	0.05	
Cyanide (µg/L)	< 1	< 1	< 1	200	
Fluoride (mg/L)	0.010	0.094	0.160	1.5	
Lead (mg/L)	< 0.001	< 0.001	< 0.002	0.01	
Mercury (µg/L)	<0.01	< 0.01	< 0.05	1	

	Minimum	Average	Maximum	MAC	IMAC
Nitrate (mg N/L)				10	
Selenium (mg/L)	0.0001	0.0	0.002	0.01	
Uranium (µg/L)	0.0002	0.2	0.50	20	

The minimum, average and maximum concentrations for the pesticides specified within the Permit and their comparison with their respective Maximum Acceptable Concentration (MAC) or Interim Maximum Acceptable Concentration (IMAC) are provide below in Table 4.4. Again as can be seen no contraventions have been observed in this group.

Table 4.4: Historical Treated Water Pesticides Concentrations, 2009 to 2015

	Minimum	Average	Maximum	MAC	IMAC
Atrazine (µg/L)	< 1	< 1	< 2		5
Bromoxynil (µg/L)	< 0.5	< 0.5	< 0.5		5
Carbofuran (µg/L)	< 0.02	< 2	< 2	90	
Chlorpyrifos (µg/L)	< 2	< 2	< 2	90	
Dicamba (µg/L)	< 0.5	< 0.5	< 0.5	120	
2,4-D* (µg/L)	< 0.5	< 0.5	< 0.5		100
Diclofop-methyl (µg/L)	< 1	< 3	< 3	9	
Dimethoate (µg/L)	< 1	< 3	< 5		20
Malathion (µg/L)	< 2	< 2	< 2	190	
Pentachlorophenol (µg/L)	< 2	< 2	< 2	60	
Picloram (µg/L)	< 1	< 1	< 1		190
Trifluralin (µg/L)	< 1	< 1	< 1		45

Table 4.5 summarizes the minimum, average and maximum concentrations for the synthetic organics specified in the Permit and again compares them with their respective Maximum Acceptable Concentration (MAC) or Interim Maximum Acceptable Concentration (IMAC).

Table 4.5: Historical Treated Water Synthetic Organic Concentrations, 2009 to 2015

	Minimum	Average	Maximum	MAC	IMAC
Benzene (µg/L)	< 0.2	< 0.2	< 0.2	5	
Benzo(a)pyrene (µg/L)	< 0.01	< 0.01	< 0.01	0.01	
Carbon Tetrachloride (µg/L)	< 2	< 2	< 2	5	
Dichlorobenzene, 1, 2 (µg/L)	< 0.5	< 0.5	< 0.5	200	
Dichlorobenzene, 1, 4 (µg/L)	< 0.5	< 0.5	< 0.5	5	
Dichloroethane, 1, 2 (µg/L)	< 0.5	< 0.5	< 0.5		5.00
Dichloroethylene, 1,1 (µg/L)	< 0.5	< 0.5	< 0.5	14	

	Minimum	Average	Maximum	MAC	IMAC
Dichloromethane (µg/L)	< 0.5	< 0.5	< 0.5	50	
Dichlorophenol, 2, 4 (µg/L)	< 0.2	< 1	< 1	900	
Monochlorobenzene(µg/L)	< 0.5	< 0.5	< 0.5	80	
Tetrachlorophenol, 2,3,4,6 (µg/L)	< 0.5	< 0.75	< 1	100	
Trichloroethylene (µg/L)	< 0.5	< 0.5	< 0.5	50	
Trichlorophenol, 2,4,6 (µg/L)	< 1	< 1	< 1	5	
Vinyl Chloride (µg/L)	< 0.5	< 0.5	< 0.5	2	

As can be seen from Tables 4.3, 4.4, and 4.5, the treated water produced by the City's WTP from January 2009 through to December 2015 did not exceed the MAC or IMAC for each of the species identified.

In addition to the standards outlined above, the Guidelines for Canadian Drinking Water Quality (GCDWQ) has objectives/guidelines for certain parameters within drinking water. Within the Saskatchewan Water Security Agency requirements, compliance with objectives/guidelines is also not mandatory. However, the exceedance of these species beyond their respective objectives/guidelines may affect the acceptance of water by consumers.

Table 4.6 summarizes the minimum, average and maximum concentrations for the objectives monitored by the City from January 2009 through until December 2015. The values are compared with their respective objectives as specified in the GCDWQ.

Table 4.6: Comparison of Species with GCDWQ Objectives and Guidelines, 2009 to 2015

	Minimum	Average	Maximum	Objectives/Guidelines
Aluminum (mg/L)	0.001	0.054	0.342	<0.1
Chloride (mg/L)	5	7.6	14.0	< or equal to 250
Color (ACU)	0	1.5	16.0	< or equal to 15 True Colour Units
Copper (mg/L)	< 0.0005	< 0.0005	< 0.001	< 1.0
Manganese (mg/L)	0.001	0.019	0.067	< or equal to 0.05
pH (unitless)	6.6	7.5	8.2	6.5-8.5
Sodium (mg/L)	7	11.5	62.0	< or equal to 200
Sulfate (mg/L)	1.1	67.2	86.0	< or equal to 500
TDS (mg/L)	206	259.1	338.0	< or equal to 500

As can be seen in Table 4.6, the objectives for Aluminum and Manganese were slightly exceeded in this period. To clarify the colour parameter, colour has been measured in apparent colour units (ACU) whereas the objective is measures in true colour units (TCU). Colour measured in water that contains suspended matter is defined as "apparent colour". "True colour" is measured in water samples from which particulate matter has been removed by centrifuges. In general, the true colour of a given water sample is substantially less than its apparent colour.

Figures 4.9 below illustrates the concentrations of Aluminum and Manganese within the treated water for the reporting period. It is important to note that while these concentrations exceed the objectives, these are not enforceable standards and that the 2009 to 2015 average values for each of these parameters is significantly below their respective objectives/guidelines. Therefore, it is anticipated that these parameters will not pose a threat to the quality of drinking water supplied to the consumers.

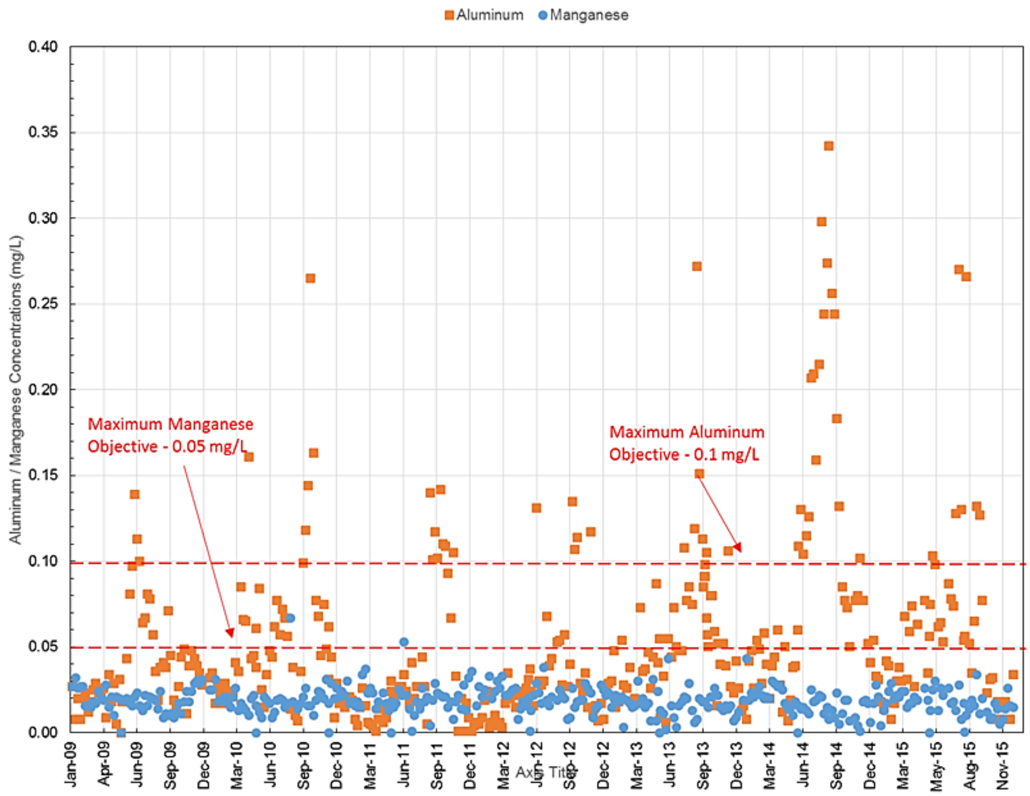


Figure 4.8: Treated Water Aluminum and Manganese Concentrations

If the concentrations in Figure 4.9 are plotted against the pH of the treated water (see Figure 4.10), it is possible to observe that the elevated aluminum concentrations occur either in fall or summer, and during periods when the pH of the treated water is also high. As the solubility of aluminum is directly related to the pH of the water, it is likely that soluble (non-particulate) aluminum is passing through the filters and being measure in the treated water. Should the City wish to address this elevation in aluminum, a small degree of pH control to reduce and stabilize the pH of the water prior to clarification should reduce the aluminum concentration in the treated water.

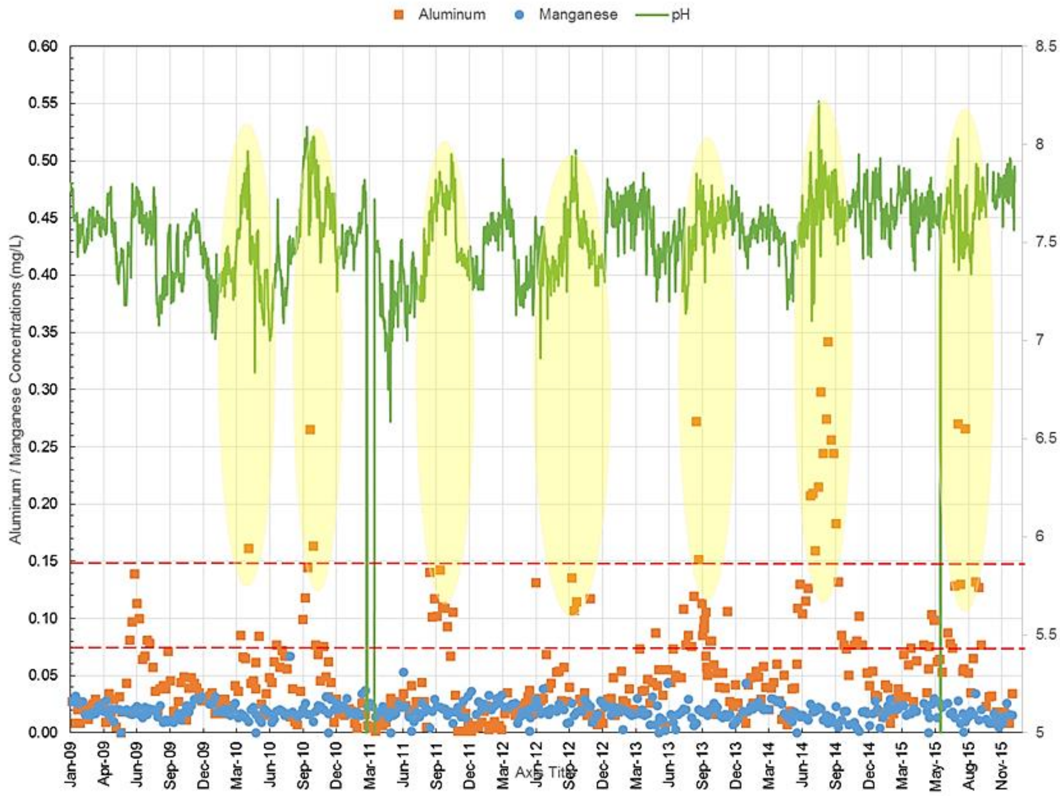


Figure 4.9: Treated Water Aluminum and Manganese Concentrations, with pH.

Based upon the information presented with regards to the quality of water within the treatment process, no conceptual recommendations for improvement are made at this time. In order to align with the GCDWQ it is suggested that the colour analysis move to true colour units rather than apparent colour units.

4.3 Treated Water Distribution Quality

Treated water is taken from the clearwell at the end of the City's WTP is directed into the distribution system using a combination of three distribution pumps. Once within the distribution system the City conducts routine analysis for total coliforms, E coli, free chlorine, total chlorine, turbidity and Trihalomethanes at a total of five representative locations within the system. These are:

- Servus Center
- West End Reservoir
- Redhead Equipment
- Home Depot
- Leisure Center

In addition to the locations listed above, the City performs sampling and testing for free and total chlorine at other locations. Table 4.7 below provides the free and total chlorine residuals measures at all locations within the distribution system from January 2009 through to December 2015. For locations where testing has been performed only once between 2009 and 2015, only the average concentration has been provided within the table below.

Table 4.7: Historical Treated Water Quality, 2009 to 2015

	Free Chlorine (mg/L)			Total Chlorine (mg/L)		
	Min.	Avg.	Max.	Min.	Avg.	Max.
Servus Center	0.28	0.66	1.09	0.42	0.79	1.29
West End Reservoir	0.50	0.89	1.31	0.65	1.03	1.52
Redhead Equipment	0.12	0.71	1.05	0.22	0.83	1.16
Home Depot	0.25	0.59	0.98	0.43	0.69	1.11
Leisure Center	0.21	0.66	1.04	0.33	0.79	1.21
Lakeland College	0.27	0.58	0.97	0.48	0.72	1.08
Sobeys	0.44	0.77	1.12	0.64	0.93	1.28
Husky	0.52	0.89	1.25	0.7	1.05	1.42
WTP	0.81	1.22	2.05	0.6	1.38	2.2
5515 49th Street	0.69	0.87	1.01	0.81	1.05	1.26
5619 46th Street	0.60	0.82	0.96	0.68	0.93	1.02
4708 13th Street	0.26	0.51	0.75	0.46	0.64	0.86
3007 57A Avenue (Single Sample)	-	0.46	-	-	0.61	-
Bottle Depot (Single Sample)	-	0.29	-	-	0.48	-
City Hall	0.62	0.78	0.93	0.84	0.97	1.13
City Shop (Single Sample)	-	0.86	-	-	1.12	-
Holy Rosary (Single Sample)	-	0.67	-	-	0.88	-
City Hospital (Single Sample)	-	0.53	-	-	0.72	-
City Mall (Single Sample)	-	0.85	-	-	1.02	-
United Rentals (Single Sample)	-	0.89	-	-	1.00	-
West Harvest (Single Sample)	-	0.45	-	-	0.57	-
RSC (Single Sample)	-	0.80	-	-	1.05	-
Other	0.03	0.49	1.04	0.21	0.64	1.11
Hydrant 648 (Single Sample)	-	0.39	-	-	0.63	-
Hydrant 753 (Single Sample)	-	0.51	-	-	0.83	-
Hydrant 736 (Single Sample)	-	0.36	-	-	0.65	-
Hydrant 705 (Single Sample)	-	0.34	-	-	0.6	-

Figures 4.11 and 4.12 provide the free and total chlorine residuals at the routine locations in a graphical format.

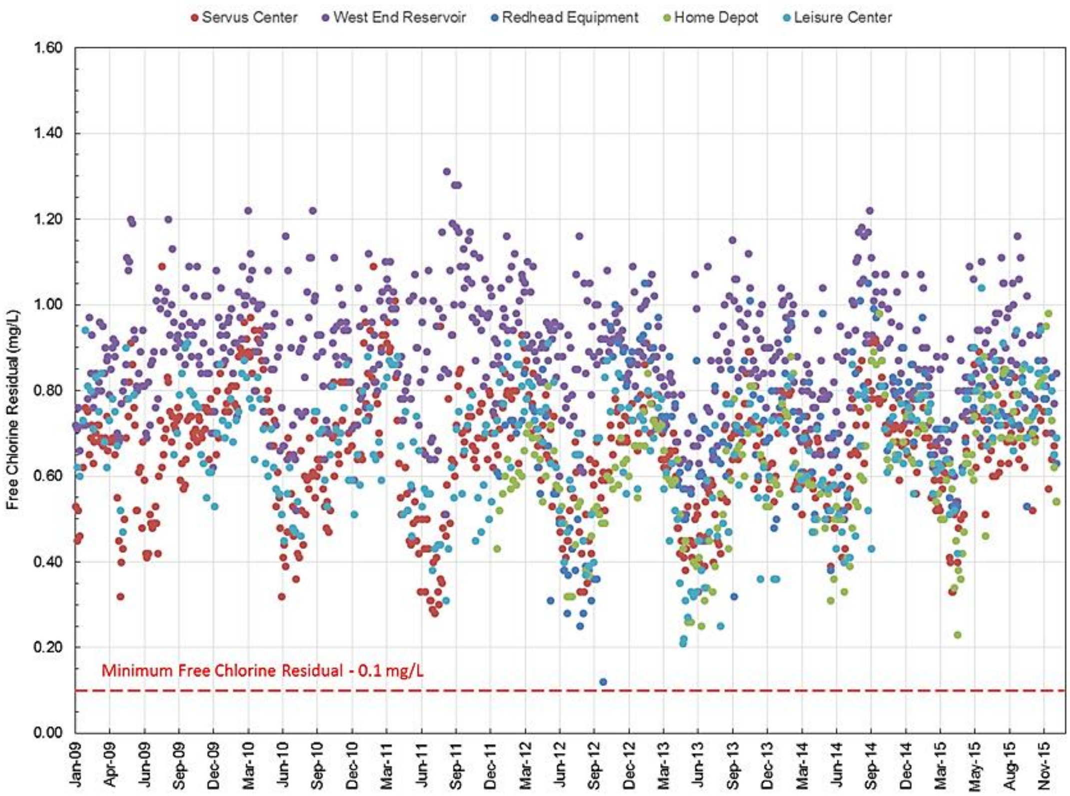


Figure 4.10: Free Chlorine Residual at Routine Locations within the Distribution System, 2009 to 2015

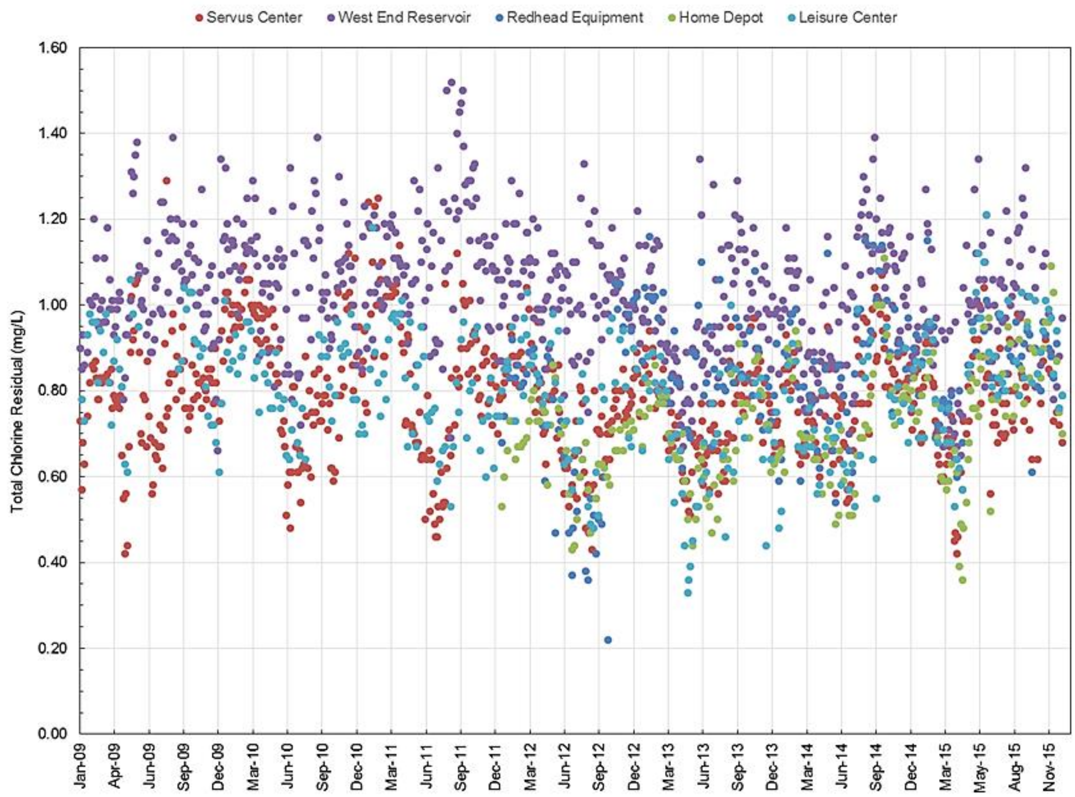


Figure 4.11: Total Chlorine Residual at Routine Locations within the Distribution System, 2009 to 2015

The Saskatchewan Drinking Water Quality Standards and Objectives and the City's Permit to Operate a Waterworks state that a minimum of 0.1 mg/L of free chlorine residual or 0.5 mg/L of total chlorine residual must be maintained at all times within the distribution system. As can be seen from the figures above, the City is thorough in ensuring that either the minimum free chlorine residual is provided throughout their distribution system.

When samplings for free and total chlorine, the City is also required to sample and analyze for turbidity. Table 4.8 and Figure 4.13 below, illustrate the results for turbidity at the six representative locations listed above.

Table 4.8: Turbidity within the Distribution System, 2009 to 2015

	Turbidity (NTU)		
	Min.	Avg.	Max.
Servus Center	0.06	0.12	0.80
West End Reservoir	0.05	0.11	0.90
Redhead Equipment	0.07	0.13	1.09
Home Depot	0.09	0.14	1.31
Leisure Center	0.06	0.12	0.36
Lakeland College	0.11	0.18	0.60

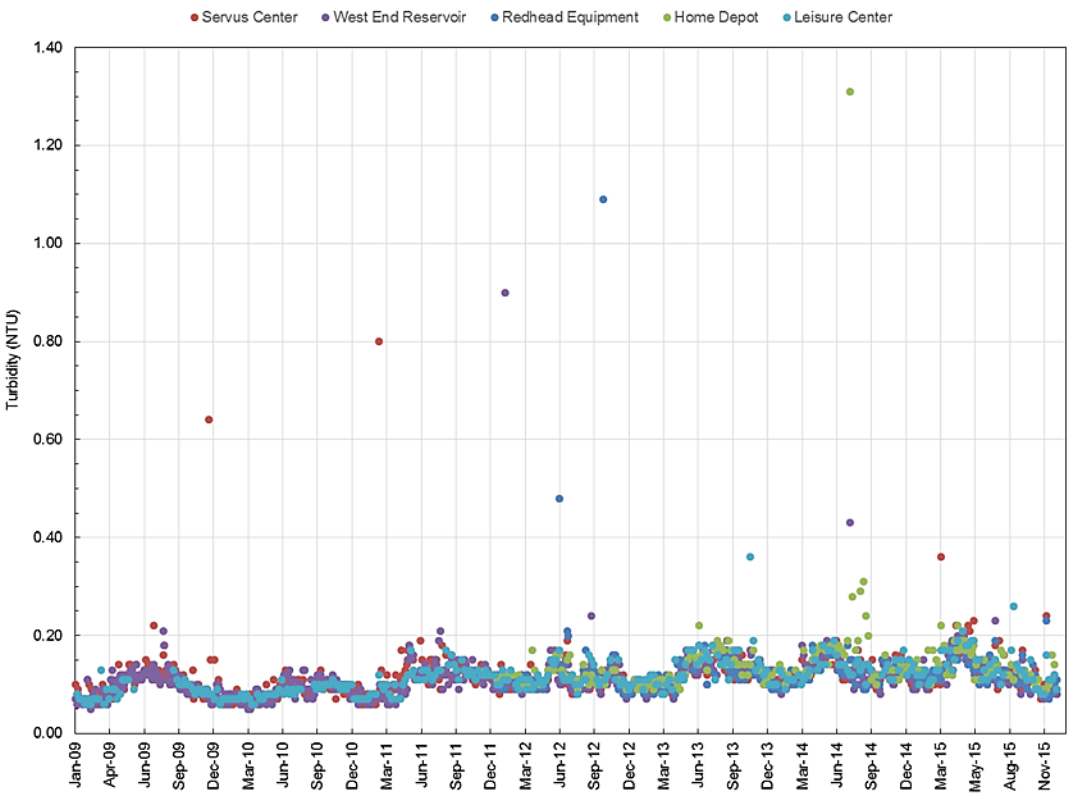


Figure 4.12: Turbidity within the Distribution System, 2009 to 2015

For the majority of the reported period the turbidity in the distribution system low, with an occasional elevate value. In August 2014 there is some elevated values at the Home Depot sample location, however the results drop back to normal in the fall of 2014.

Trihalomethanes (THMs) within the distribution system are monitored by the City at the Servus Center and the West End Reservoir. Table 4.9 below provides the individual and annual average concentrations for THMs at these locations from 2009 through to 2015. As per the SWSA standards, the annual average THM concentration in the treated water measured over all 4 seasons must fall below the Interim Maximum Acceptable Concentration (IMAC) of 100 µg/L.

Table 4.9: Trihalomethanes Concentrations with the Distribution System, 2009 to 2014

	Servus Center		West End Reservoir	
	Concentration (µg/L)	Annual Average (µg/L)	Concentration (µg/L)	Annual Average (µg/L)
10-Feb-09	30.7	49.2	17.1	34.15
12-May-09	32.4		36.9	
18-Aug-09	110		63.5	
17-Nov-09	23.7		19.1	
08-Feb-10	22.2	51.3	19.5	49.78
18-May-10	26.8		26.8	
17-Aug-10	98.1		66.3	
02-Nov-10	58.1		86.5	
15-Feb-11	41.8	41.8	26.1	58.15
31-May-11	-		80	
16-Aug-11	-		98	
08-Nov-11	-		28.5	
14-Feb-12	29.8	51.13	24.7	40.25
29-May-12	44.2		43.3	
28-Aug-12	98.7		67.9	
04-Dec-12	31.8		25.1	
05-Feb-13	32.6	62.65	28.7	52.35
21-May-13	91		82.5	
27-Aug-13	93.4		73.2	
26-Nov-13	33.6		25	
11-Feb-14	39	40.98	32.3	35.43
20-May-14	68.4		68.1	
16-Sep-14	31.2		26.5	
12-Nov-14	25.3		14.8	
10-Feb-15	27.7	38.57	19.2	31.9
12-May-15	43.5		51.7	
11-Aug-15	44.5		24.8	

Whilst the annual average value for the concentration of THMs is below the IMAC, the concentration of individual samples does exceed or get close to exceeding the IMAC. Figures 4.14 below illustrated these high values and the variations between the seasons, as well as highlighting how the location of the sample can vary the result on the same day. The concentration of THMs within the distribution system is directly affected by the age of the water within the system, the temperature of the water and the free chlorine residual that is present.

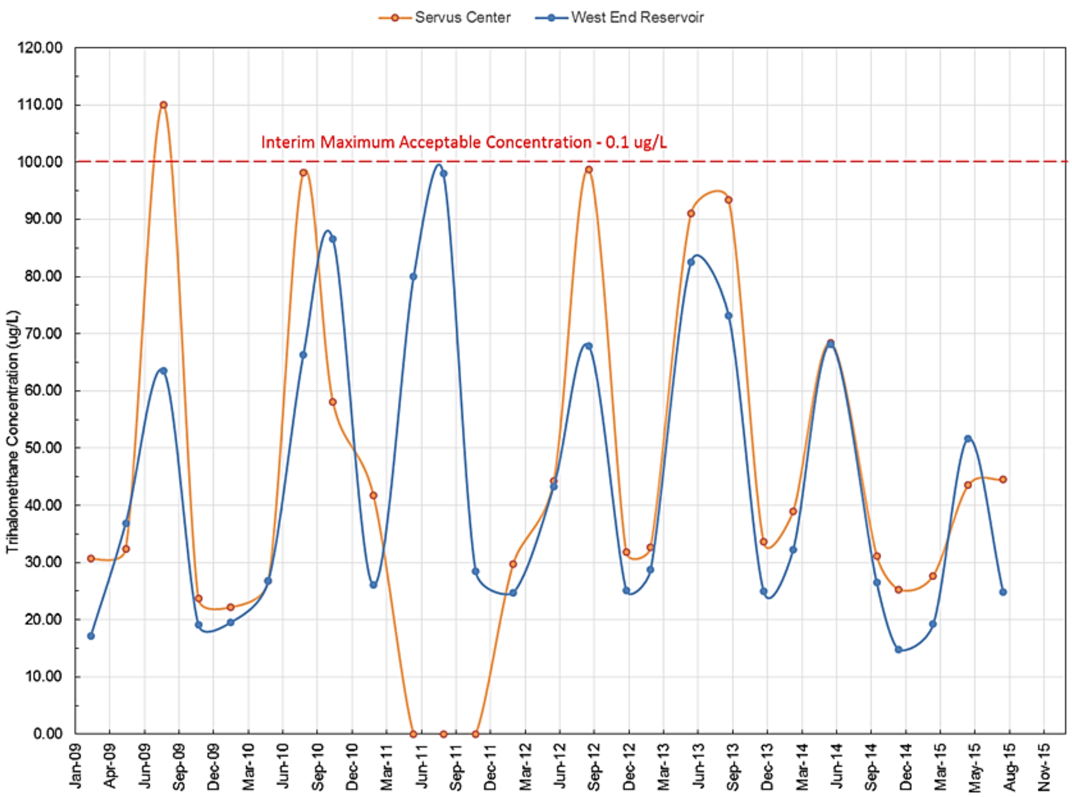


Figure 4.14: Trihalomethanes Concentrations within the Distribution System, 2009 to 2015

Based upon the information presented with regards to the quality of water within the distribution system, there is no basis to make any conceptual recommendations at this time. ISL does note however the elevated THM concentrations within the summer months and suggests that the City vary the location of the sample to take into account the water with the longest age within the distribution system.

4.4 Non-Compliant Issues / Incidences

As part of the Round 3 WSA, ISL is required to report and examine any non-compliant issues or incidences that have occurred in the City's waterworks system within the review period. Upon reviewing the available information from 2009 through to 2015, positive results for Total Coliforms were recorded on

- June 9th, 2013 at the Home Depot location
- December 29th, 2014 at the Leisure Center, and
- July 14th, 2015 at the Leisure Center

Repeat samples taken as a result of all three positive results did not detect the presence of coliforms in the water. As review of the information shows no issues with both turbidity and free chlorine concentrations, and in all three cases either stand-in samplers or an alternate sample locations were used. In conclusion the positive results were likely to be the result of external contamination.

5.0
Waterworks Demand Projections

One of the objectives of the Round 3 WSA is to estimate the remaining service life of all the equipment and components within the City's waterworks system. This requires an assessment of each component of the waterworks system not only in terms of its remaining operating life, but also its capacity with regards to the projected growth of the City. This section will summarize the expected future demands to be placed upon both the raw water supply system and the Water Treatment Plant (WTP). As noted previously the raw water system provides river water to a number of parties, which include the City, Husky Energy and a number of smaller users.

As established within Section 2, a number of agricultural establishments draw raw water from the pipeline between the intake and the raw water reservoir. In reviewing the 2009 to 2015 historical data, the amount of raw water used by the smaller users is less than 1%. The Legion Ball Park and the City's Golf Course draw raw water from the pipeline to Husky Energy after the WTP takeoff, which amounts to about 1% of the annual raw water volume. Therefore, based upon the small amounts by which these other users impact the raw water volumes, they will not be included further within the future raw water projections and as such only the City and the Husky Energy will be considered for the remainder of this Section.

5.1 Husky Energy Upgrader Water Demands

As discussed in Section 2.0, the Husky Energy Upgrader (Husky) is supplied with river water directly from the raw water supply pipeline for industrial use. The historical raw water consumption data for Husky from 2009 to 2015 was supplied to ISL by the City, and has been represented in Figure 5.1 below.

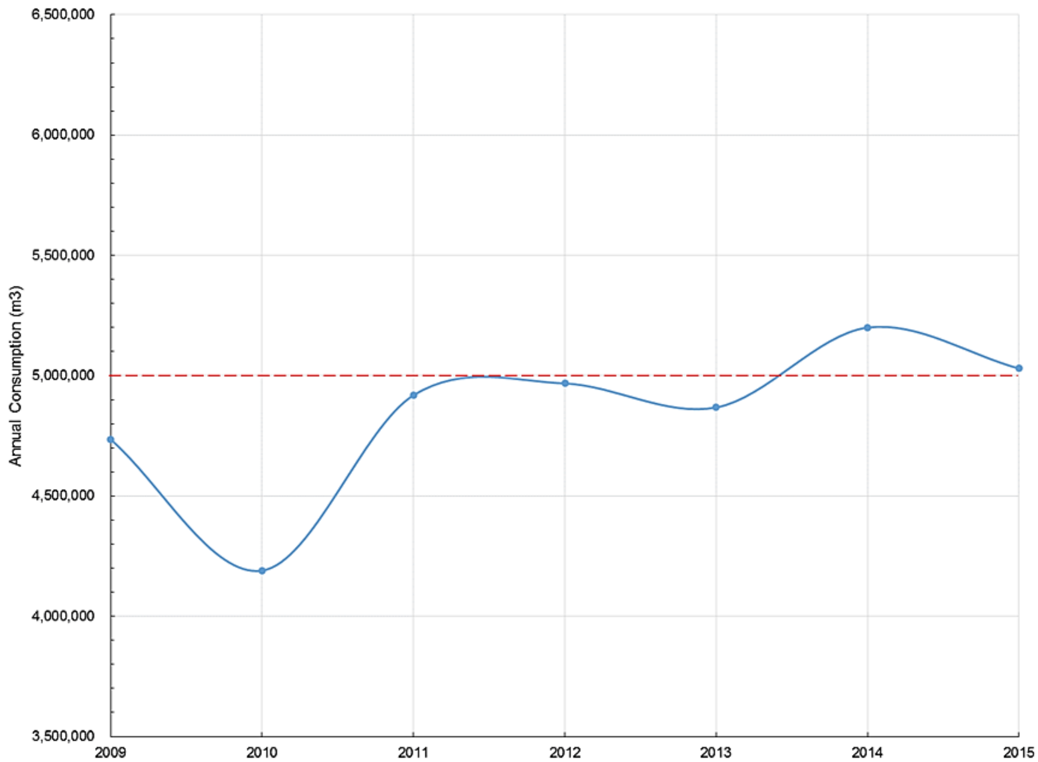


Figure 5.1: Historical Annual Raw Water Consumption by Husky Upgrader, 2009 to 2015

Inspiring sustainable thinking

As can be seen from the Figure 5.1, the average consumption by Husky since 2011 has consistently been around 5,000,000 cubic meters per year, however in reality Husky typically wishes to divert as much was as possible for their operations. For the purposes of this assessment and using the recent historical information as a basis, the future annual consumption of raw water by Husky will be maintained at 5,000,000 m^3 as per year, as there are no known expansions planned at the upgrader. This will provide an average daily volume to Husky of 13,699 m^3/d.

With the average daily volume established, the next step is to determine the maximum daily volume that will be supplied to Husky in the future. Table 5.2 provides a further breakdown of average daily and maximum daily volumes of raw water supplied to Husky between 2009 and 2015. Using both the 100th and 99th percentiles to eliminate any possibilities of outliers, the historical peaking factors for maximum daily volume to Husky were established as shown. The variance between the maximum daily values noted below and the stated rating of the Husky pumps can be attributed to pump moving along its pump curve, as the suction pressure vary and discharge friction losses change (pressure control valves are installed downstream). This can result in a lower pressure difference (boost) across the pumps allowing a higher flowrate to be achieved.

Table 5.1: Husky Upgrader Historical Daily Volumes & Peaking Factor

	100th Percentile			99th Percentile		
Year	**Annual Average Volume (m^3/d)**	**Maximum Daily Volume (m^3/d)**	**Peaking Factor**	**Annual Average Volume (m^3/d)**	**Maximum Daily Volume (m^3/d)**	**Peaking Factor**
2009	13,042	23,428	1.8	13,042	18,062	1.4
2010	11,479	27,109	2.4	11,479	17,871	1.6
2011	13,550	17,944	1.3	13,550	17,584	1.3
2012	13,573	20,304	1.5	13,573	19,035	1.4
2013	13,337	19,457	1.5	13,337	18,636	1.4
2014	14,246	20,303	1.4	14,246	18,611	1.3
2015	13,780	19,603	1.4	13,780	18,887	1.4
Avg. Values	13,204	21,424	1.6	13,204	18,300	1.4

Based upon the above information, a maximum daily volume peaking factor of 1.4 is a practical value based upon the historical data. On this basis, the assessment will therefore be based upon future raw water flows to Husky of 13,966 m^3/d for the average daily volume, and 19,179 m^3/d for the maximum daily volume.

5.2 City of Lloydminster Water Demands

Projections completed for a Water Master Plan with regards to pipe sizes, pressure zones and reservoirs are typically based upon the end use and developed using future land use at specific years or design horizons. In this planning exercise the average daily volumes for future land uses are assigned based upon industry standards and comparisons with similar sized communities. This results in an average day demand (ADD) for the distribution system to which peaking factors are applied to establish both the maximum day demand (MDD) and the peak hour demand (PHD).

Within the City's Water Distribution Master Plan 2016, an agreed MDD peaking factor of 2 times the ADD and a PHD peaking factor of 3 times the ADD was applied to establish flowrates for evaluating components of the distribution system. In applying this basis to the existing system, the following flowrates for 2015 were established for the supply of water to the distribution system,

- Average Day Demand of 127 L/sec (10,973 m^3/d),
- Maximum Day Demand of 253 L/sec (21,859 m^3/d), and
- Peak Hour Demand of 380 L/sec (32,832 m^3/d).

Through discussions with the City's operational team there has been a historical disconnect between what is projected as part of assessments and studies, and what happens in reality. This has resulted in water treatment capacity upgrades being recommended and pursued far earlier than is actually necessary. For example, within the City's 2010 WSA report, a consumption rate of 421 L/capita/day was applied along with a MDD peaking factor of 1.8 and a population growth of 3%. For the year of 2015 this resulted in a projected ADD of 13,323 m^3/d and a MDD of 23,981 m^3/d.

In 2015 the recorded ADD based upon flow instrumentation at the WTP was 10,862 m^3/d and the MDD was 18,241 m^3/d. Based upon the population of 31,377, the consumption rate was 346 L/capita/day. With regards to MDD peaking factors observed in 2015, the MDD peaking factor based upon a single day was 1.68 times the ADD, whereas the MDD peaking factor over a 5-day average was 1.50 times the ADD. In our discussions with the City, the operations staff have been clear that the single day based maximum daily demand is not a true representation of demand, as a high single day maximum daily volume is usually the result of refilling the West End Reservoir after the water level has been intentionally allowed to drop.

In comparing the projected flows and volumes from 2010 with the actual recorded flows and volumes, a significant difference can be observed as shown in Table 5.2 below. At the time the 2010 WSA was completed the available information was utilized, however input from the operations staff whose actions directly affect the recorded information might not have been given the recognition needed. As such this resulted in the recommendation within the 2010 WSA Report that the WTP required a capacity upgrade by 2012, whereas today in 2016 the plant continues to operate on an average of 16 hours per day to meet the treated water demands of the City.

Table 5.2: Comparison of 2015 Projected and Actual Water Supply Values

	2015 Projected Values from 2010	2015 Projections from 2016 Master Plan	Actual 2015 Values
Population	31,646	-	31,377
Consumption (L/capita/d)	421	-	346
Average Daily Demand (m^3/d)	13,323	10,942	10,862
Maximum Daily Demand (m^3/d)	23,981	21,886	18,241
Single Day Peaking Factor	1.8	2	1.68
Five Day Average Peaking Factor	-	-	1.50

In determining the design flow for a WTP and when a capacity upgrade should occur, it is the maximum daily demand values that is used as the design value. As can be noted within the above table a significant discrepancy occurs between high level projections and actual data, thus providing a skewed view of the timing for future capacity upgrades.

In order to provide a realistic projection with regards to the future demands and performance of the existing WTP that serves the City of Lloydminster, the projected distribution demand forecast used within the 2016 Distribution Master Plan will not be applied to this water treatment facility assessment. The application of the distribution system projections would in this case provide a misguided and premature assessment of the future upgrade of the City's WTP. Instead the historical information gathered and reviewed in conjunction with the City's operational staff will be used for the projection as summarized below. This data was gathered following an exercise in 2007 where the City calibrated and validated their key water flowmeters thus providing them with confidence in their readings.

In determining the water consumption rate (L/capita/d) and a suitable MDD peaking factor, the historical data was considered. Table 5.3 and Figure 5.2 below provides a summary of the historical daily volume of treated water distributed from the WTP between 2008 and 2015. As can be observed within the data below, the average and maximum daily volumes have not varied considerably over the last eight years.

Table 5.3: Historical Average and Maximum Daily Volumes from WTP

	2008	2009	2010	2011	2012	2013	2014	2015	Avg. Values
Average Daily Volume (m^3)	11,021	10,840	10,418	10,889	10,754	10,760	10,942	10,862	10,811
Max Daily Volume (m^3)	16,554	17,887	15,438	16,804	15,000	16,507	18,176	18,241	16,826

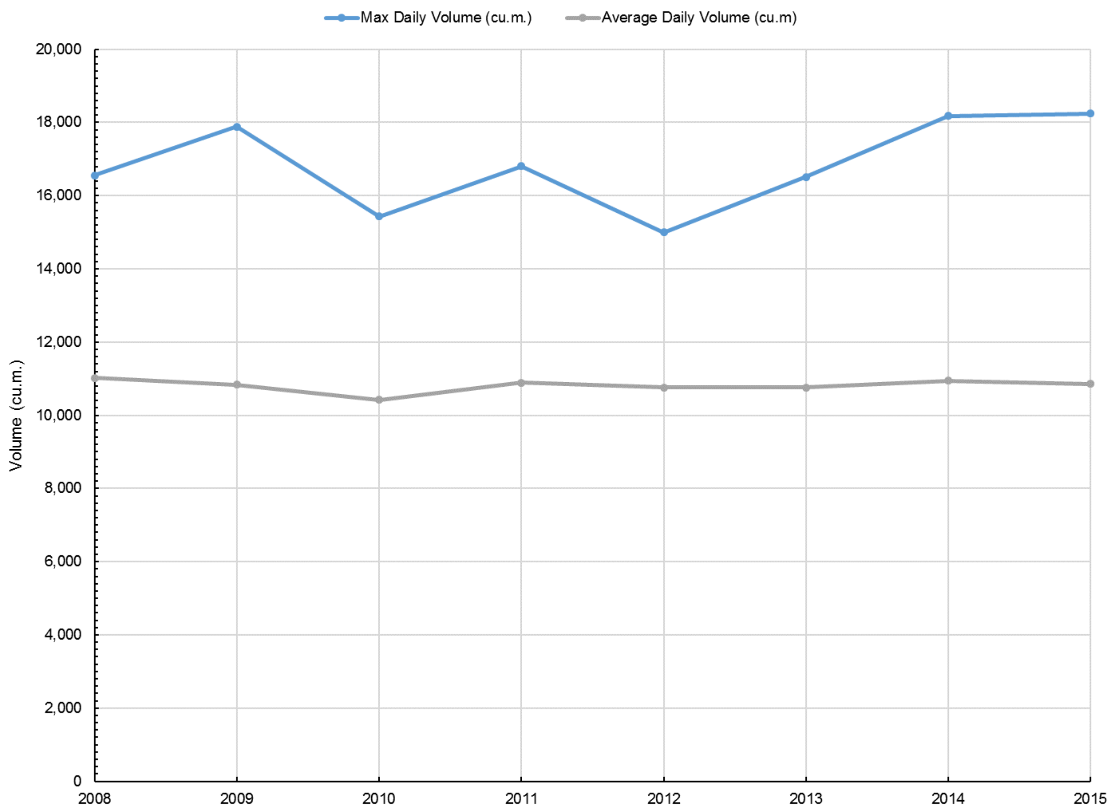


Figure 5.2: Historical Average and Maximum Average Daily Volumes from WTP

In utilizing the City census information for 2007, 2009, 2011 2013 and 2015 and linearly interpolating the population for the missing years the historical water consumption rate for each year has been calculated within Table 5.4 below.

Table 5.4: Historical Water Consumption Rates

	2008	2009	2010	2011	2012	2013	2014	2015
Average Daily Volume (m^3)	11,021	10,840	10,418	10,889	10,754	10,760	10,942	10,862
Census and Interpolated Population	26,013	26,502	27,153	27,804	29,644	31,483	31,390	31,377
Water Consumption (L/capita/d)	424	409	384	392	363	342	349	346

Since 2007 the City has been monitoring and reviewing data on water usage within the distribution system, and working with consumers on addressing high consumption. As can be observed within Table 5.4 above, the consumption rate over the past 3 years has stabilized around 340 to 350 Liters per capita per day (L/capita/d). Based upon a partial set of data for 2016 the average consumption up to the end of July 2016 is 336 L/capita/d. On this basis the City believes that the treated water consumption rate within the City has stabilized and that a consumption rate of 340 L/capita/d shall be applied for the City's future potable water projections.

In most situations the calculation of peaking factors for the maximum daily demand is based upon a single day only. As noted previously, caution must be used when using this approach as a single high daily volume may be the result of an operational event such a reservoir cleaning or a recovery in water storage following an intentional drop in reservoir levels. In addition, a single day value does not take fully into account the available potable water storage within the treated water reservoirs.

Following the design requirements set out by Regulatory bodies, a significant amount of the water held in reservoirs is for the purpose of fire protection. However, in reality a reservoir may not be operated in this way and in the case of the City of Lloydminster, the maximum daily volume averaged over five days is monitored and the system is operated in a way that maximizes the available water storage and reduces water age. Using the historical information from 2008 onwards, Table 5.5 below illustrates the peaking factors based upon both a single day and a five-day average maximum value.

Table 5.5: Historical Maximum Daily Volumes with Single and 5 Day Average Peaking Factors

	2008	2009	2010	2011	2012	2013	2014	2015	Avg. Values
Average Daily Volume (m^3)	11,021	10,840	10,418	10,889	10,754	10,760	10,942	10,862	10,811
Maximum Daily Volume - Single Day (m^3)	16,554	17,887	15,438	16,804	15,000	16,507	18,176	18,241	16,826
Maximum Daily Volume – 5 Day Rolling Average (m^3)	14,680	15,252	13,577	15,066	13,441	13,789	15,012	16,326	14,643
Peaking Factor (Single Day)	1.50	1.65	1.48	1.54	1.39	1.53	1.66	1.68	1.56
Peaking Factor (Five Day Rolling Average)	1.33	1.41	1.30	1.38	1.25	1.28	1.37	1.50	1.35

Following the most common approach to determining the peaking factor (single day), the City of Lloydminster experiences a peaking factor between 1.4 and 1.7, whereas when considered over 5-days the peaking factor is lower. Figure 5.3 below is a graphical representation of the daily volumes for each day from January 1, 2008 to December 31, 2015. This Figure shows that when a high daily volume occurs it is typically preceded or followed by daily volumes which are significantly less. This indicates that a high daily volume is off-set by the daily volume preceding it or following it.

The red line within the Figure 5.3 below illustrates a peaking factor of 1.5 applied to the average of the average daily volume from 2008 to 2015 (i.e. 10,811 m^3 multiplied by 1.5). As can be seen, on days where the daily volume historically exceeds the red line, the daily volume before or after the peak is a lot lower, supporting the conclusion that over 2 to 3 days the peaking factor does not exceed 1.5.

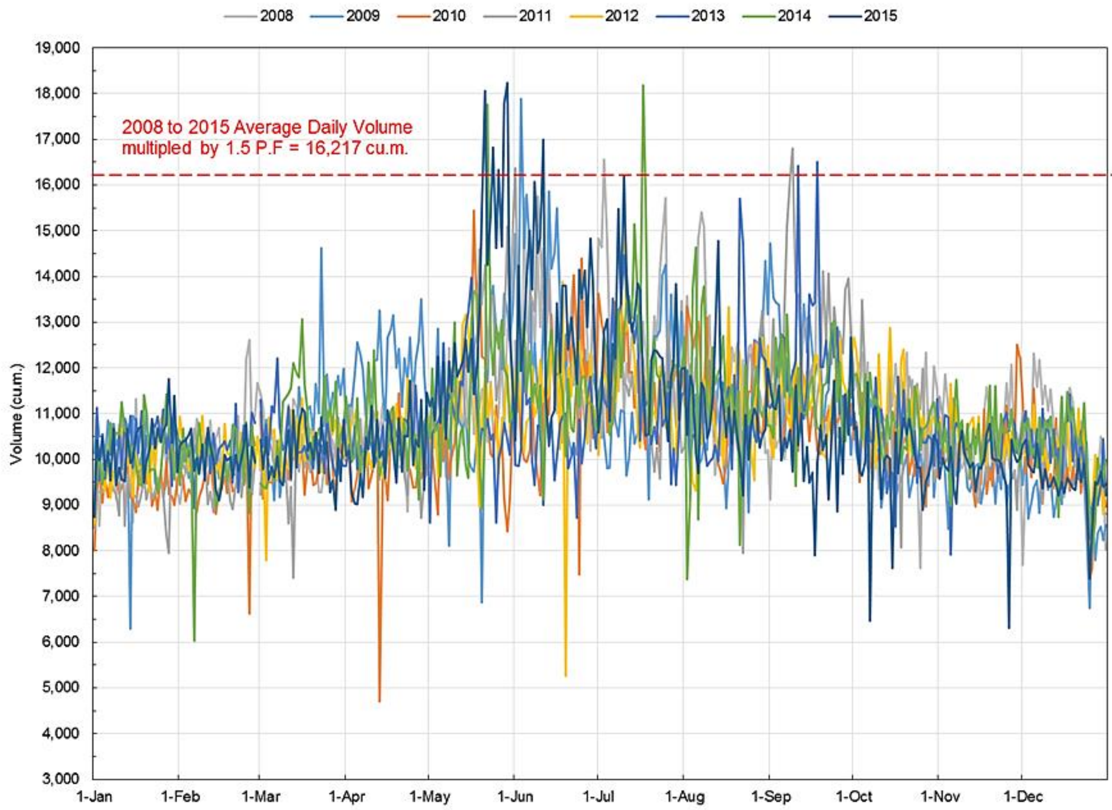


Figure 5.3: Historical Daily Volume with 1.5 Peaking Factor

Based upon this information and the data presented above, the City and ISL agreed to apply a maximum day factor of 1.5 times the ADD, based upon a 5-day rolling average, to the future average day projections with regards to the future potable water requirements for the City of Lloydminster.

The final part required for this projection is the future population numbers. The typical approach applied for WTP projections is to select a constant population growth rate and apply it over a number of years. For the draft version of this report, varying population projections were established based upon future land use by the City's planning department in conjunction with ISL. These growth rate were 3.3% for 2015 and 2016, 3.0% from 2017 until 2021, 2.6% from 2022 until 2026, 2.30% from 2027 until 2031, 2.10% from 2032 until 2036, 1.90% from 2037 until 2041, and 1.70% until 2042 until 2046.

Since establishing the above growth basis, the provincial economy has experienced a downturn and the population within the City has reduced slightly. As such the City has determined that the above growth rates projections are too aggressive and shall be revised to a medium single rate of 2.1% for the purposes of this report. Furthermore, the City confirmed that the base population for 2015 of 32,515 which was derived from the 2013 census population and the average growth rate since 1950 until 2014 of 3.2%, will be replaced with the actual census population for 2015 of 31,377.

In summary the future daily volumes projections for the City of Lloydminster's WTP will therefore be established using:

- A base population of 31,377 in 2015 (i.e. municipal census figure for 2015)
- A constant medium annual growth rate of 2.1%
- A water consumption rate of 340 Liters per capita per day (L/capita/d)
- A maximum day peaking factor of 1.5 (5-day rolling average)

In applying this basis the projected average and maximum daily volumes project at 0, 3, 5, 10, 20 and 30 years are provided below in Table 5.6 and Figure 5.4, which will be used within this Waterworks System Assessment. ISL has noted that the projected daily volumes for 2015 are less than what was recorded, however it is important to refer to Figure 5.2 and note how the daily volumes have remained stable over the past eight years

Table 5.6: Projected Future Average and Maximum Daily Treated Water Volumes for WTP

	2015	2018	2020	2025	2035	2045
Year	0	3	5	10	20	30
Projected Population	31,377	33,396	34,813	38,625	47,547	58,531
Average Daily Treated Water Volume (m3)	10,668	11,354	11,836	13,133	16,166	19,900
Maximum Daily Treated Water Volume (m3) (5 Day Rolling Average)	16,002	17,032	17,755	19,699	24,249	29,851

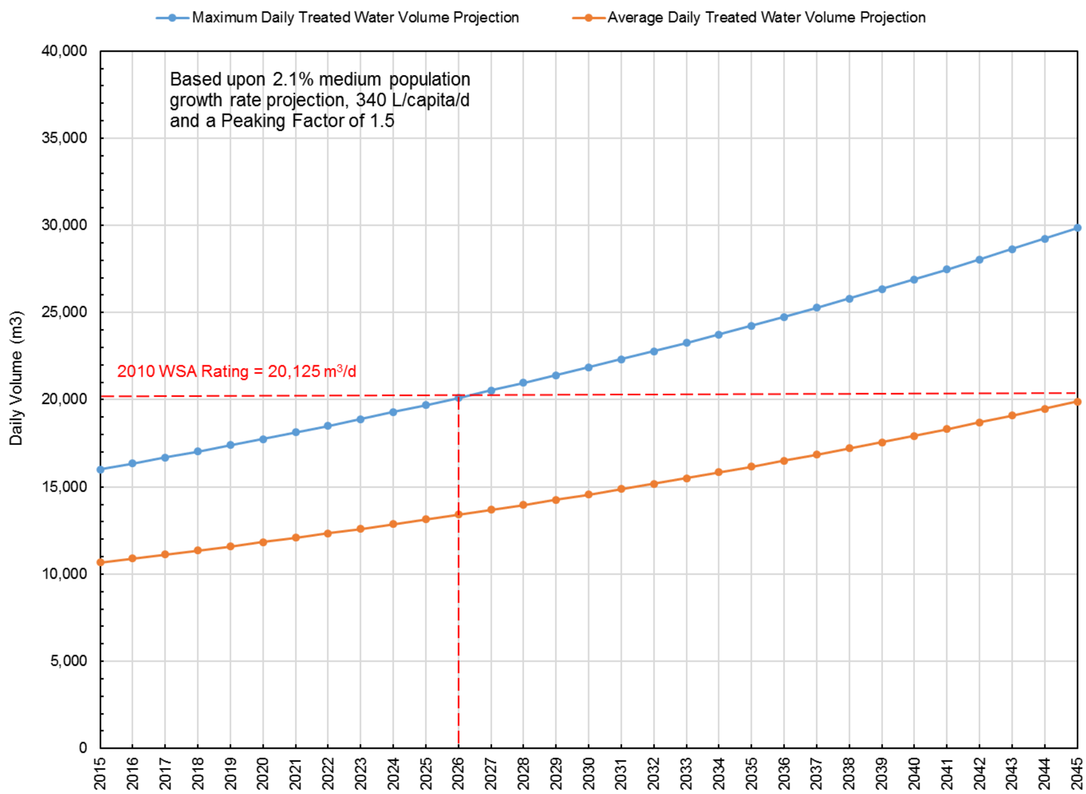


Figure 5.4: Projected Future Maximum and Average Daily Treated Water Volumes for WTP

In order to ascertain the impact of the City's potable water demand on the raw water system, the losses across the WTP must be reviewed and an appropriate amount applied to the future projections. Table 5.7 and Figure 5.5 below illustrates the daily percentage losses that have occurred across the water treatment plant from 2009 through to 2015.

Table 5.7: Daily Losses across Water Treatment Plant, 2009 to 2015

	Volume (m^3/d)	Percentage Losses
Minimum	-738	-8
Average	339	3
Maximum	2,459	16
99 Percentile	1,341	11
95 Percentile	993	9

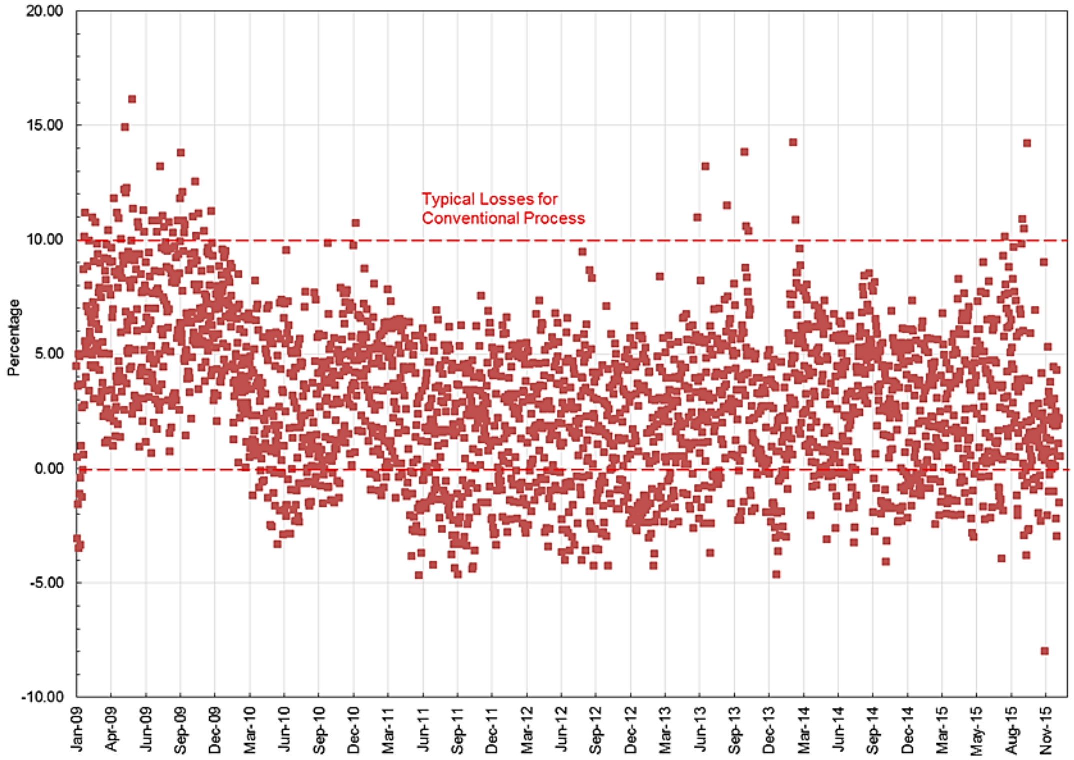


Figure 5.5: Daily Losses across Water Treatment Plant, 2009 to 2015

When evaluating losses across a conventional water treatment process, similar to that employed by the City, a raw water loss of 10% is typically applied to account for desludging, filter washing and plant service water. Based upon Figure 5.5 above, a values of 10% for the losses across the WTP would be very conservative in this situation.

Analysis of the numbers within Table 5.7 provides no firm indication as to what percentage to apply to the future projections. In considering the average value for the losses, the distribution shown within Figure 5.5 and the typical value applied, a slightly above average value of 5% is deemed appropriate for this process and situation.

In applying this loss to the future daily treated water volumes projected above, Table 5.8 and Figure 5.6 illustrate the raw water flows to the water treatment plant in years 0, 5, 10, 20 and 30.

Table 5.8: Projected Future Average and Maximum Daily Raw Water Volumes for WTP

	2015	2018	2020	2025	2035	2045
Year	0	3	5	10	20	30
Average Daily Raw Water Volume (m^3)	11,202	11,922	12,428	13,789	16,974	20,895
Maximum Daily Raw Water Volume (m^3)	16,802	17,883	18,642	20,684	25,462	31,343

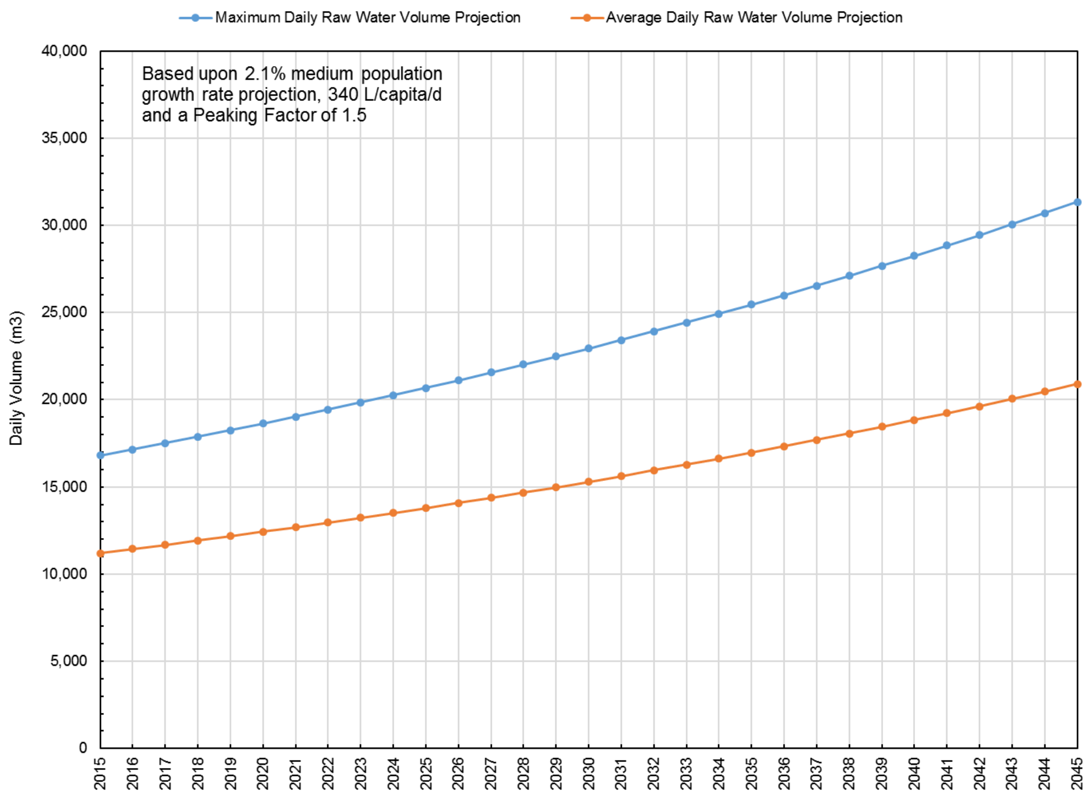


Figure 5.6: Projected Future Maximum and Average Daily Raw Water Volumes for WTP

5.3 Assessment Basis

Using upon the information provided and develop within this Section, the volumes and flows that will be applied to the evaluation of the raw water system and the water treatment plant are summarized below in Table 5.9 and 5.10. As shown in the table below the raw water system volumes are calculated from the summation of the projections for the Husky and the City, whereas the treated water projections just apply to the City of Lloydminster.

Table 5.9: Future Average and Maximum Daily Raw Water Volumes for WTP Assessment

	2015	2018	2020	2025	2035	2045
Year	0	3	5	10	20	30
Average Daily Raw Water Volume to Husky (m3)	13,966	13,966	13,966	13,966	13,966	13,966
Average Daily Raw Water Volume to City (m3)	11,202	11,922	12,428	13,789	16,974	20,895
Total Average Daily Raw Water Volume (m3)	**25,168**	**25,888**	**26,394**	**27,755**	**30,940**	**34,861**
Maximum Daily Raw Water Volume to Husky (m3)	19,179	19,179	19,179	19,179	19,179	19,179
Maximum Daily Raw Water Volume to City (m3)	16,802	17,883	18,642	20,684	25,462	31,343
Total Maximum Daily Raw Water Volume (m3)	**35,981**	**37,062**	**37,821**	**39,863**	**44,641**	**50,522**

Table 5.10: Future Average and Maximum Treated Water Volumes for WTP Assessment

	2015	2018	2020	2025	2035	2045
Year	0	3	5	10	20	30
Projected Population	31,377	33,396	34,813	38,625	47,547	58,531
Average Daily Treated Water Volume to City (m3)	10,668	11,354	11,836	13,133	16,166	19,900
Maximum Daily Treated Water Volume to City (m3) (5 Day Rolling Average)	16,002	17,032	17,755	19,699	24,249	29,851

6.0
Waterworks System Capacity and Assessment

One of the objectives Waterworks System Assessment is to assess the performance of each major component and determine its capabilities and condition with regards to meeting the future demands for the supply of potable water. For each stage within the City of Lloydminster's system, the following aspects will be established and reviewed within this Section:

1. Design basis or rated capacity
2. Performance issues or non-compliance issues
3. Condition
4. Future performance
5. Remaining service life
6. Upgrade requirements

As required, future performance capabilities will be based upon both flow and populations using the projections established within Section 5.0 of this report.

In completing this assessment, it is important to note that the remaining service life stated within this section does not mean a complete replacement of the item or component. The City's operational staff will explore and undertake options to overhaul and rebuild equipment before resorting to replacing it with completely new items. This specific approach is very relevant when considering and reviewing screens, pumps and other mechanical equipment.

6.1 Raw Water Supply System

The major infrastructure and elements of the Raw Water Supply System which will be assessed are the:

- River intake
- River intake screen
- Low lift pumps and desilting pond
- High lift pumps and raw water supply pipeline
- River intake building
- Husky raw water pump station
- Raw water reservoir

6.1.1 River Intake Structure

The River intake structure comprises of a concrete rectangular structure located within the North Saskatchewan River, which draws river water into the intake structure. The intake ports, a 1,050 mm diameter HDPE pipeline and the structure have a reported design capacity of 1,042 L/sec (90,000 m^3/day).

As noted in the previous WSA, the buildup of sand and sediment in and around the intake structure is an ongoing issue. Discussions with the operational staff as part of this WSA have highlighted that sand is drawn into the intake structure when more than one low lift pumps is running. Due to this consequence the operational staff strive to avoid running more than one low lift pump, as they are concerned about excessive wear occurring with regards to the installed mechanical equipment, and sand building up within the structure that could lead to a restriction or prevention of river water flowing through the intake structure.

A previous inspection, review and assessment of this issue recommended the installation of a "wing dam" to increase the velocity of the water passing across the face if the intake, scouring the sand and sediment downstream such that it does not enter the intake. At this time, the design of the wing dam is completed, although the work has not been executed. The City's operational staff continue to address the sediment issue through monitoring, frequent flushing of the sediment back into the river and the removal of sediment from the wet wells using a submersible pump. It is this practice of backflushing the intake, started in 2004, that has minimized wear and prolonged the life of the mechanical equipment within the raw water pumphouse.

ISL were not able to conduct a physical inspection of the intake structure during this review, however City staff reported no ongoing issues, other than sediment accumulation with regards to the intake structure. In 2004 the City videoed the condition of the intake structure, a review of which show the structure to be in good condition with no damage, with no other issues raised.

Table 6.1 below provides a comparison of the projected future raw water flowrates (as established in Section 5.0) for comparison with the rated design capacity of the intake structure. Within the projected 30-year time frame there is no hydraulic limitation anticipated, other than that related to sedimentation build up.

Table 6.1: Future Performance of Intake Structure

Years	Design Capacity (m^3/day)	2018	2020	2025	2035	2045
Horizon		**3**	**5**	**10**	**20**	**30**
Projected Maximum Day Raw Water Flowrate (m^3/d)	90,000	37,062	37,821	39,863	44,641	50,522
Projected Population	-	33,396	34,813	38,625	47,547	58,531

Currently there are two Licenses under the Water Act, which are related to this intake structure. One issued to the City, the other to Husky. Whilst annual average volumes stated within these documents raises no concerns, the maximum rate of diversion on both Licenses is 60,000 m^3/day. Therefore, should both parties wish to withdraw at the maximum rate, the rated design capacity of the intake structure would be exceeded. ISL do not see this as an issues with regards to the future operation of the raw water system, however it is something that the City should be aware of.

The intake structure was brought into service in 1984 and based upon a 50 year typical operating life for a concrete structure, has approximately 9 years of service life remaining. However, this is a conservative estimate and the City staff have provided no indication that the condition of the structure is a concern to them.

With regards to the future improvements, ISL recommends that the City move forward with a review, tender and subsequent installation of the wing dam. The presence sand and sediment around the intake structure is limiting the operation of the raw water system when a flow through the low lift pumps greater than 30,000 m^3/day is required. This operational restriction placed on the incoming flow does not impact the volume of water withdrawn today, however it will start to restrict future supply of sufficient water in the immediate future. Until the movement of sand and sediment into the intake is addressed, there is no benefit to increasing the capacity of the equipment within the raw water pumphouse, as the resulting wear from the sand and sediment will drastically reduce the life of the installed equipment and possible restrict the flow of incoming water.

6.1.2 River Intake Screen

The Rexnord traveling screen immediately downstream of the intake structure, has an effective mesh size of 10 mm and a hydraulic design capacity of 89,856 m^3/day, just below the design capacity of the intake structure. Whilst the screen was installed in 1984 and is now 11 years beyond the typical 20-year design life applied to mechanical equipment, the City staff maintain the screen such that it continues to operate and remove large items that enter the intake with the river water.

In terms of future hydraulic capacity, the values presented in Table 6.1 also apply to the screen, which is capable of meeting the project demands beyond the next 30 years. Based upon the observations made by ISL, the screen is not in any serious state of disrepair, apart from spillage resulting from the oil bath, which is being contained by adsorption pads. In 2013 the City's staff undertook an inspection of the screen below the main floor, from which no serious concerns or issues were raised.

On the basis that the screen is maintained and parts /components are fabricated and replaced when issues are identified, the raw water intake screen should continue to operate beyond next 10 years.

6.1.3 Low Lift Pumps and Desilting Pond

Once through the intake screen, the river water is lifted, using two low lift pumps, into a U-shaped desilting pond where sediment and other material is settled out of the water. Both low lift pumps are installed with variable frequency drives and have a pumping capacity of 347 L/sec (30,000 m^3/day) at a discharge pressure of 9 m.

The desilting pond has a volumetric capacity of 120,000 m^3 and was last dredged in 2007. During this exercise approximately 90% of the settled material (sand) was removed within the first 25% of the pond length. Based upon the actual location of the deposited material, the whole pond was not dredged as minimal amounts were found towards the end of the pond. Based upon changes in operating practices to prevent material from being pulled into the system, future dredging of the desilting pond is not foreseen by the operations team. This is supported by frequency of low lift pump overhauls due to wear, which since the implementation of these practices have dropped from once every three years to no longer required. Isolation of the desilting pond if required, can be achieved through a bypass which directs river water to the high lift pump wet well.

The low lift pumps have historically met the demands of the system and have the potential capacity to withdraw water at the maximum licensed removal rate when both are running. On an ongoing basis the hydraulic performance of the low lift pumps is monitored by comparing them with high lift pumps flowrates and the level in the desilting pond. This allows any performance issues to be identified and addressed early. However, as noted within Section 6.1.1, the operational staff are very reluctant to run two low lift pumps together, as under these conditions sand and sediment is pulled into the intake structure, increasing equipment wear and potentially restricting the flow of water.

Historical turbidity results, shown in Table 4.1, show that the desilting pond is efficient in removing suspended solids from the river water, thus reducing the solids loading on the water treatment plant (WTP). In reviewing this data, the variations in the pond outlet turbidity correlated with the variations in the inlet turbidity rather than the flowrate through the pond.

Table 6.2 shows the projected average and maximum raw water flowrates that will pass through the system in the next 30 years. As a single pump provides a flowrate of 30,000 m^3/day, it will be necessary to run the second low lift pump on a more regular basis as raw water demand increases (i.e. within the next 5 to 10 years). As the operation of two low lift pumps is a condition that the operational staff wish to avoid at this time, addressing the sand and sediment infiltration issue becomes a higher priority for both operational and maintenance reasons.

Table 6.2: Projected Average and Maximum Raw Water Flowrates, with Population

Years	2018	2020	2025	2035	2045
Horizon	**3**	**5**	**10**	**20**	**30**
Projected Average Day Raw Water Flowrate (m^3/d)	25,888	26,394	27,755	30,940	34,861
Projected Maximum Day Raw Water Flowrate (m^3/d)	37,062	37,821	39,863	44,641	50,522
Projected Population	33,396	34,813	38,625	47,547	58,531

On the basis that the wing dam is installed and the sand / sediment infiltration is addressed, the situation will be eventually reach that due to the ongoing operation of two low lift pumps, the City will have lost its stand-by low lift pump capability.

Using their understanding and experience from moving water between the ponds / reservoirs and through the WTP, ISL recommends that the City develop, test and document a series of steps and procedures to avoid the use of the second low lift pump. Subsequently a construction schedule for the wing dam should be developed for planning purposes. By undertaking this task, the City will be able to clearly communicate the criteria and timing for when the use of a second low lift pump will actually be required. The primary objective however is to proceed with the installation of the wing dam before the operation of two low lift pumps is required, such that sand and sediment is not pulled into the intake structure causing more problems and issues.

Once the sand and sediment issue is address, and the point has been reached where the operation of the second low lift pump is consistently required, the City will need to address the lost stand-by low lift pump capability. The City currently has a spare impeller for the low lift pumps which could be replaced within one to two weeks, should an issue with an existing pump's impeller occur. Other parts of the pumps could be obtained or manufactured with relative ease. However, for the future conditions when two low lift pumps are required, the City could either upgrade both pumps to achieve a duty / stand-by configuration again, or procure a complete "boxed spare" pump that can be stored by the City and installed at short notice on a low lift pump failure.

Table 6.3 below provides a summary of the review undertaken by ISL as part of this assessment and the subsequent discussions with the operations team. With regards to the low lift pumps, the remaining service life is also identified within this table along with any actions that are required.

Table 6.3: Low Lift Pumps and Desilting Review, Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Low Lift Pump LLP-101	Re-built recently. Pump was running during visit. No excessive noises and no sign of leakage or scale on pump	Pump overhauled in June 2010. New 150 Hp VFD motors installed in February 2009. One spare impeller on site for both low lift pumps.	10	Plan for pump and motor overhaul in 10 years.
Low Lift Pump LLP-102	Evidence of pump has been maintained / worked upon. No leakage or scale on pump. Not running during inspection.	New motors 150 Hp VFD motor installed in January 2009. Motor and pump overhauled in May 2002. One spare impeller on site for both low lift pumps	< 5	Plan for pump overhaul in less than 5 years.

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Low Lift Pumps Pipework & Valves	Pipework in good condition, few areas require touch up of epoxy coating. Discharge air valves have surface corrosion at flanges joints. Original isolation and check valves in use		10+	Valves, instrumentation and pipework will need replacement as issues are identified.
De-silting Pond	No signs of issues, banks are all in place, and dredged in 2007	A lot of maintenance is performed on the banks, to prevent erosion from water movement. Constantly maintained	25+	Monitor sediment and remove when required. Address any future issues with bank stability and wildlife as identified

6.1.4 High Lift Pumps and Raw Water Supply Pipeline

Water from the desilting pond flows by gravity into the high lift pump well from where three high lift pumps are used to pump the water 36 km to the raw water reservoir. Two pumps each have the capacity to deliver 232 L/sec (20,000 m^3/day) at a 542.5 m, and one has a capacity of 116 L/sec (10,000 m^3/day) at 542.5 m. During our discussions with the operations team it was noted that the motors on the larger pump are running close to their capacity (i.e. 690 Hp on a 700 Hp motor).

The raw water pipeline is a 750 mm diameter yellow jacketed epoxy lined steel pipe. The design capacity of the pipeline is reported to be of 694 L/sec (60,000 m^3/day), which results in a typical water velocity through the pipe of 1.6 m/sec. Air release stations are located along the length of the pipeline and the pipeline effectively operates on the principle that the water is pumped to the top of the hill and then flows by gravity to the raw water reservoir. The cathodic protection system for the raw water pipeline is inspected annually and any issues addressed.

No day to day issues have been report by the operational staff with regards to the raw water pipeline, however it is recognized as a critical piece of infrastructure. A previous inspection and assessment of the raw water pipeline identified a section where the pipeline was "buckled". A further analysis of the pipewall thickness at the "buckle" identified no evidence of any detrimental issues. Replacement of this section has not been pursued due to the criticality of the pipeline and the fact that there have been no issues reported with regards maintaining raw water flow and pressure. The materials to repair the pipeline should a failure occur have been procured and are stored by the City. The City also has the skills and capabilities to undertake a repair at short notice.

In 2010 a corrosion assessment of the pipeline was completed using a current device. Several "hot spots" were identified, which based upon the assessment were exposed and inspected. It was determined that in these locations the yellow jacket had been breach and the corrosion was only at the surface of the pipeline. These hot spots were addressed when the pipeline was exposed and inspected. A subsequent internal inspection of the pipeline using the installed vaults at the high point, confirmed the condition of the pipeline to be "like new" in the inspected locations and no further issues were identified.

In reviewing the capacity of high lift pumps, a similar situation to that identified with the low lift pumps occurs. Overall the average day raw water flowrate can be achieved using two of the three high lift pumps, however after 2025 (10 years) both of the large pumps could be required to meet average flow, as using the smaller pump with a larger pump will only provide a total flowrate of 30,000 m^3/day. Currently the City runs the two larger high lift pumps once every 3 to 5 years for a short period (i.e. a few days), and uses the buffering capacity of the desilting pond and the raw water reservoir to normally address the few days of the year when maximum day demands currently occur. Running two large high lift pumps eventually requires the operation of two low lift pumps, which results in sand entering the system. A situation that the operations team wishes to avoid.

In considering the projected values as stated, an increase in the application of the available two large high lift pumps could occur within the next 5-10 years such that maximum day demand flowrates can be met. The raw water max day demands are based upon a future 5-day rolling average of treated water flows plus 5%, and it anticipated that the buffering capacity within the desilting pond and raw water reservoir can be used to reduce the maximum day raw water flow further. The requirement to use all three high lift pumps to meet the maximum day raw water flowrates is likely to occur substantially further into the future, and should this need occur the intent of the City would be to replace the smaller high lift pump with a larger unit, and continue to increase pumping capacity as required to maintain the availability of a stand-by pump.

Table 6.4: Average and Maximum Day Raw Water Flowrates

Years	2018	2020	2025	2035	2045
Horizon	**3**	**5**	**10**	**20**	**30**
Projected Average Day Raw Water Flowrate (m^3/d)	25,888	26,394	27,755	30,940	34,861
Projected Maximum Day Raw Water Flowrate (m^3/d)	37,062	37,821	39,863	44,641	50,522
Projected Population	33,396	34,813	38,625	47,547	58,531

The projected average and maximum day flowrates in 2045 within the table above, are below the design capacity of the raw water pipeline resulting in velocities of 0.91 and 1.32 m/sec for the average and maximum daily flowrates in 2045. With the application of fixed speed high lift pumps, the operation and integrity of the raw water pipeline is dependent upon the successful performance of the pump start-up and pressure relief valves. Whilst these valves are serviced and inspected on an annual basis their importance within the system cannot be underestimated.

Table 6.5 below provides comments from both ISL and the operation's team review of the high lift pumps and raw water pipeline, as well the remaining service life and actions that are required.

Table 6.5: High Lift Pump and Raw Water Pipeline Comments, Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
High Lift Pump HLP-101	Scale build up on pump head and base plate from seal. Corrosion developing on pump discharge head. Corrosion on grounding cable connection	Pump and motor overhauled in June 2009	< 10	Plan for overhaul of pump and motor in less than 10 years. Address corrosion issues
High Lift Pump HLP-102	Corrosion and scale on pump not as bad as High Lift Pump P-101.	Pump and motor overhauled in June 2011	10+	Replace smaller pump with larger unit such that all three pumps match, as future demands will result in loss of standby capability
High Lift Pump HLP-103	Recently refurbished. Seal water is handled differently. No leakage on the pump. Pump was running smoothly during visit	Pump and motor overhauled in June 2015. Epoxy coated internal and external column surfaces	15+	Perform maintenance and address issues when they arise
Pump Start-up and Pressure Relief Valves	Staining and surface corrosion observed from leaking fittings	Valves are inspected and tested on a regular basis. Repairs are made when identified. New valve was installed in one location a few years ago.	10+	Replace parts and completed repairs as needed
High lift Pump Pipework & Valves	Pipework looks ok, few areas require touch up of epoxy coating. Staining of pipework from water leaking from valves and fittings. Original isolation and check valves in use		10+	Valves, instrumentation and pipework will need replacement as issues are identified.

6.1.5 River Intake Building

The river intake building that houses all of the above equipment, as well as the electrical and HVAC system was built in 1984. The building was constructed from masonry block with sections of metal cladding, and a sloped membrane roof. The above grounds structure remains in very good condition and there was no evidence of a leakage or damage to the building itself. The below ground structure was not observed by ISL.

The operational staff reported no issues with the HVAC system, and there is currently a project underway to replace the electrical switchgear as the sourcing of spare parts has become an issue. Although typically a building is expected to have a service life of 30 years, this building should therefore be theoretically replaced soon. However, there is no obvious reason why the building and structure should not be serviceable, if maintained for the next 20 years.

6.1.6 Husky Raw Water Pumphouse and Pipeline

Prior to entering the raw water reservoir, a portion of the raw water flow is diverted to the Husky Raw Water Pumphouse. Constructed in 1999, to replace the original pumphouse, the station is equipment with two split-case horizontal pumps each with a capacity of 91 L/sec (7,840 m^3/d) at a discharge pressure of 57 m (559 kPa), with space for a further two pumps in the future. The associated raw water supply pipeline to the Husky Facility is constructed for 350 mm dimeter PVC and is approximately 9 km long.

In considering the future average and maximum day flowrates to Husky (13,966 m^3/d and 19,179 m^3/d respectively, see Section 5), it is noted that maximum day projection exceeds the reported duty of two pumps running together (15,680 m^3/d), which is possible to achieve the with existing pumps providing the pressure increase across the pump is reduced to approximately 545 kPa. At these future flowrates the respective water velocities are approximately 1.68 and 2.31 m/sec. Whilst it is recognized that there is some variation between theoretical and real world values, and that there is usually some flexibility with pump curves, these figures illustrate that the Husky raw water system is operating at or close to its design capacity.

As Husky is responsible for the upkeep and maintenance of their pumphouse and pipeline, its condition and the future costs to maintain it do not affect directly affect the City of Lloydminster. Should Husky wish to increase the flow of water to it facility, this would require further discussion with the City of Lloydminster and the impact to the raw water system reassessed, specifically with regards to the components upstream.

6.1.7 Raw Water Reservoir

With a design volumetric capacity of 204,000 m3, the raw water reservoir has been assigned a working storage capacity of 188,000 m^3 in summer and 155,000 m^3 in winter, which accounts for sediment and ice cover as described in Section 2.0. The raw water reservoir has not been drained and inspected since 1983, however in 2006 divers did enter the reservoir and determined that there were no issues or concerns with the condition of the pipework and other structures within it.

Under normal operation the reservoir is kept full and all raw water passes through the reservoir before entering the WTP. However, in an emergency the raw water reservoir can be bypassed, such that raw water is supplied directly to the WTP. Whilst there are no specific compliance requirements for the reservoir, the City staff have to add copper sulphate to the reservoir to address algae and weed growth. In reviewing the water quality data (Section 4.0), the reduction in raw water turbidity across the reservoir was noted, however elevated E. coli counts have also been observed in the reservoir outlet. Whilst the presence of E. coli is addressed by the treatment process and confirmed by sampling, the increase is unusual and is likely related to the bird activity within and around the reservoir.

Table 6.6 below illustrates the number of days storage within the raw water reservoir based upon the future average and maximum day raw water flowrates to the WTP. The table is based upon the raw water reservoir volume for summer, however this value is an untested theoretical volume. It is recognized that a maximum day flowrate is unlikely to occur in winter, and thus the number of days storage within the raw water reservoir in winter will be more, unless there is a serious break in the distribution system.

Table 6.6: Future Days of Storage in Raw Water Reservoir

Year	2018	2020	2025	2035	2045
Horizon	3	5	10	20	30
Average Day Raw Water Flowrate (m3/d)	11,922	12,428	13,789	16,974	20,895
Maximum Day Raw Water Flowrate (m3/d)	17,883	18,642	20,684	25,462	31,343
Projected Population	33,396	34,813	38,625	47,547	58,531
Number of Days Storage (Average Day Summer)	15.8	15.1	13.6	11.1	9.0
Number of Days Storage (Maximum Day Summer)	10.5	10.1	9.1	7.4	6.0

The water supply requirements for Saskatchewan make no specific mention the number of days of off-stream storage that is required, other than to state that off-stream storage should be based on the hydrological and quality characteristics of the primary supply. Whilst the North Saskatchewan River is a large river with sufficient volume at this time, this table illustrates the number of days the City has to address any issues or problems that would prevent raw water from being moved from the river to the reservoir.

Based upon this summary, ISL recommends that the City establish the purpose of the raw water reservoir (short term storage, long term storage, water quality improvement etc.), how many days of storage are required and what would trigger an increase in the storage volume.

Table 6.7 below provides comments from both ISL and the City's staff review of the raw water reservoir, as well the remaining service life and actions that are required.

Table 6.7: Raw Water Reservoir Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Raw Water Reservoir	Not visually observed. Elevated E.coli counts	A significant amount of work has been completed to repair the erosion damage to the reservoir banks. No deterioration has been observed since this work was completed over 10 years ago.	N/A – Complete replacement not envisaged. Ongoing inspections and repairs are expected for the foreseeable future	Continue to work with Conservation Office on bird control methods.

6.2 Water Treatment Plant

The water treatment plant located 67th Street, to the east of 50th Avenue, was commissioned in 1984 with a design capacity of 30,000 m^3/d, and replaced the WTP located on 49th Avenue. Comprised of coagulation, flocculation, clarification, media filtration and disinfection, the City's WTP operates for part of each day (about 16 hours) at a pre-established flowrate (600 to 800 m^3/hr) that allows the process to perform effectively. The hours run per day are shortened or extend on a daily basis to account for daily fluctuations in demand.

Using the same design basis as the raw water supply section, each major component will be revised and assessed to determine its capabilities and condition with regards to meeting the future demands for the supply of potable water.

6.2.1 Raw Water Pumping

Raw water can be transferred from the raw water reservoir to the WTP by either gravity or by pumping. To facilitate the pumping of raw water, two vertical inline pumps (RWP-101 and RWP-103) each with a capacity of 173.5 L/sec (14,990 m^3/d) at a discharge pressure of 9 m are available. For a flowrate of less than 1,000 m^3/hr (24,000 m^3/day), the gravity pipework is used until the water level in the raw water reservoir drops too low to push the water through to the clarifier. In 2015 a unique set of circumstances occurred for the first time in 30 years, which resulted in the level within the raw water reservoir dropping too low for the gravity connection to be effective. For a period of one week in 2015, both RWP-101 and RWP-103 were run with no issues, to supply water to the WTP when the raw water reservoir was low.

For those consumers on the raw water pipeline, a "jockey pump" (JSP-101) is also included within the raw water pumping set up, which is used to maintain a raw water supply when the river intake high lift pumps are not running. The turbidity of the raw water entering the WTP is monitored on the gravity connection only, which is also the location where Alum is normally added to the process. There is also the option to add powdered activated carbon at this point, if required.

Table 6.8 illustrates the projected average and maximum day raw water flowrates until 2045. Based upon these figures and the previously established production capacity, the current gravity system can meet the demand provided that the raw water reservoir level is kept high. As previous shown, should the reservoir level drop, the raw water pumps can be used to meet the required raw water demand up to and beyond the previously established WTP production capacity of 20,125 m^3/d. With two raw water pumps running, it is possible to meet the projected average and maximum raw water flow demands beyond the next 20 years, with the average daily demand met for the next 30 years.

Table 6.8: Average and Maximum Day Raw Water Flowrates into WTP

Years	2018	2020	2025	2035	2045
Horizon	**3**	**5**	**10**	**20**	**30**
Average Day Raw Water Flowrate (m3/d)	11,922	12,428	13,789	16,974	20,895
Maximum Day Raw Water Flowrate (m3/d)	17,883	18,642	20,684	25,462	31,343
Projected Population	33,396	34,813	38,625	47,547	58,531

Table 6.9 below provides the results of both ISL's and the City's review of the WTP raw water pumps, as well the remaining service life and actions that are required.

Table 6.9: WTP Raw Pumps Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Jockey Pump JSP – 101	No indication of any current water leakage from the pump, but there is some evidence of prior water leaks. Coating has been chipped off in some locations	No issues operationally.	10+	
WTP Raw Water Pump RWP- 101	No indication of current water leakage from the pump. Pump has been worked upon based on condition of bolts and fixings. Some surface rust on pump case/ body where coating has been chipped away. Discharge fitting is corroded.	All original pumps, mechanical seals are replaced as needed. Pump ran for 1 week with no issues in 2015	10+	Review pump condition when pumps is used for a prolonged period.
WTP Raw Water Gravity Pipework (Formally RWP -102)	Replaced with pipework to allow gravity flow through the process. Raw water turbidity sampled on this pipe leg only. Alum and optional carbon dosing point also located on this pipe section.	Aware of limitation of dosing location and raw water turbidity monitoring.	N/A	No actions, alternative dosing points for alum have been identified and are available.
WTP Raw Water Pump. RWP-103	Evidence of seal leaking and scale build up over pump. Pump has been worked upon based on condition of bolts and fixings. Some surface rust on pump case/ body where coating has been chipped away and on the discharge fitting	No operational issues. City staff is aware of seal leaking. Pump ran for 1 week with no issues in 2015	10+	City's operations team to monitor seal leakage and address when serious.

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Associated Pipework and Valves	Original discharge valves in use on RWP-101 and RWP-103, whereas other valves have been replaced. Oil residue on the gearbox of these valve, but not on the floor.	City's operations team is aware of the oil leak. The valves were not change as the flanges are 300 lb Spec and there is not enough room for conversation piece	10+	Identify replacements for 300 lb valves, and their lead time.
Associated Building / Structure	Building is in very good condition and is well maintained. Coated floor has peeled way in some locations which normal wear for 30 years of use.		30+	
Associated Electrical and Controls	No issues noted	Annual inspection by 3rd party	10+	
Associated Instrumentation	Raw water turbidity meter on gravity pipework only.	City operations team is aware of this.	10+	Turbidity instrument will be moved / re-plumbed if raw water pumps are used.

Based upon the information provide above with respect to the raw water pumps, no upgrade requirements have been identified.

6.2.2 Pre Clarification Flowmeter and Chemical Dosing Point

In following the treatment process, the raw water flowrate is measured and chemicals are added to the water. This section will comment on the condition of the equipment in this area of the WTP, with the capacity and performance of the chemical systems addressed in a later section. Table 6.10 below comments on the condition of the raw water flowmeter and chemical dosing point, as well the remaining service and actions that are required.

Table 6.10: Pre Clarification Flowmeter and Chemical Dosing Point Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Raw Water Flowmeter	Old magnetic flowmeter replaced with new clamp on ultrasonic flowmeter	No issues with the operation of this unit. Meter is calibrated as per AWWA water audit procedures and flow tested on an annual basis.	15+	

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
pH Meter	Hach instrument unit installed (new).	No issues with the operation of this instrument. Check on a daily basis	10+	
Flash Mixer FM-101	No longer used and removed	Flash mixer was removed in mid-1980's. Removal had no detrimental effect on the performance of the treatment process	N/A	
Chemical Dosing Point	Some chemical leakage on fittings with evidence of crystallization and corrosion of fittings. Only Activated Carbon is typically added here on an as required basis. There are options available to add all chemicals at this point if needed.	No issues raised.	10+	
Associated Pipework and Valves	No sign of corrosion on steel pipework or valves. Some oil sweating on valves with gear boxes prior to clarifier.	No issues raised	15+	
Associated Building / Structure	Coated floor has peeled away in some locations which is normal for 30 years of wear. Water leakage / seepage from clarifier has discoloured floor (no standing water observed). This leakage has always been present since construction and has been confirmed as not an issue by a specialist. Some cracks and staining on walls and ceiling (normal for 30 years of use)	Leakage from clarifier has slowed down over the years and self-sealed the leaks. When assessed, the downtime of clarifier was too long for sealing the cracks to be performed from inside the clarifier. Some work has been completed sealing the cracks in the clarifier from outside.	20+	Leakage from clarifier to be monitored
Associated Electrical and Controls	No issues identified	Annual inspection by 3rd party	10+	
Associated Instrumentation	Not applicable	No issues raised.		

Based upon the information provide above with respect to the Pre Clarification Flowmeter and Chemical Dosing Point, no upgrade requirements have been identified.

6.2.3 Solids Contact Clarifier

When originally installed in 1984, the rated capacity of the clarifier and tube settlers was 30,000 m^3/day, with an effective surface area of 210 m^2 (clarifier area minus mixing zone area), which results in surface loading rate of 5.95 m/hr. Since installation and commissioning, this surface loading rate has been revised by the supplier to 4.2 m/hr.

The solids contact clarifier has two purposes:

1. To remove as much of the solid and organic material from the raw water, such that the loading on the downstream media filtration stage is reduced, providing the filters with more of an opportunity to meet the required turbidity targets, and
2. To prepare and condition any material that does carryover from the clarifier, such that it can be captured by the downstream media filters.

By working and experimenting with the operation of the solids contact clarifier, the City's operations staff have determined that maintaining the integrity of the sludge blanket is the key parameter by which they can ensure the performance of the solids contact clarifier, rather than focusing on the turbidity of the water leaving. By varying the chemical dose rates, mixing speed, desludging frequency etc. in response to changes in flowrates and raw water quality, efforts are focused on the sludge blanket, with the knowledge from experience that with this approach the material carried over will be captured on the media filters.

Within the water treatment industry, it is the turbidity of the water leaving the clarifier by which performance is measured. However, while an operator can tell the difference between turbidity that is caused by poor process performance and sludge carry-over, an instrument cannot.

Using the data collected between 2009 and 2015, instantaneous flowrates based on a 16 hour and 22 hour daily run times were calculated and plotted against the clarifier outlet and treated water turbidity. As shown in Figure 6.1, the clarifier outlet turbidity varies throughout the years, and consistently stays below 1.0 NTU within only a few exceedances above this self-imposed operational guideline.

As noted above, Turbidity is not really true representation of what is occurring within clarifier and how it impacts the process downstream. It is sludge carry over and the sludge blanket integrity that are the important controlling factors with regards to this type of clarification technique. At certain times of the year when conditions are right the clarifier can treat more water than it is rated for (red line within Figure 6.1), however a change in a single water quality parameter (i.e. higher organics) can unbalance the alum dose and the now incorrect mixing speed or a deep sludge blanket for this change in raw water quality can cause the sludge blanket to break up and carry over onto the filters, blinding them. This is further illustrated within Figures 6.2 to 6.4, which show variations in measured parameters across the clarifier and how in some cases they affect one another, or not.

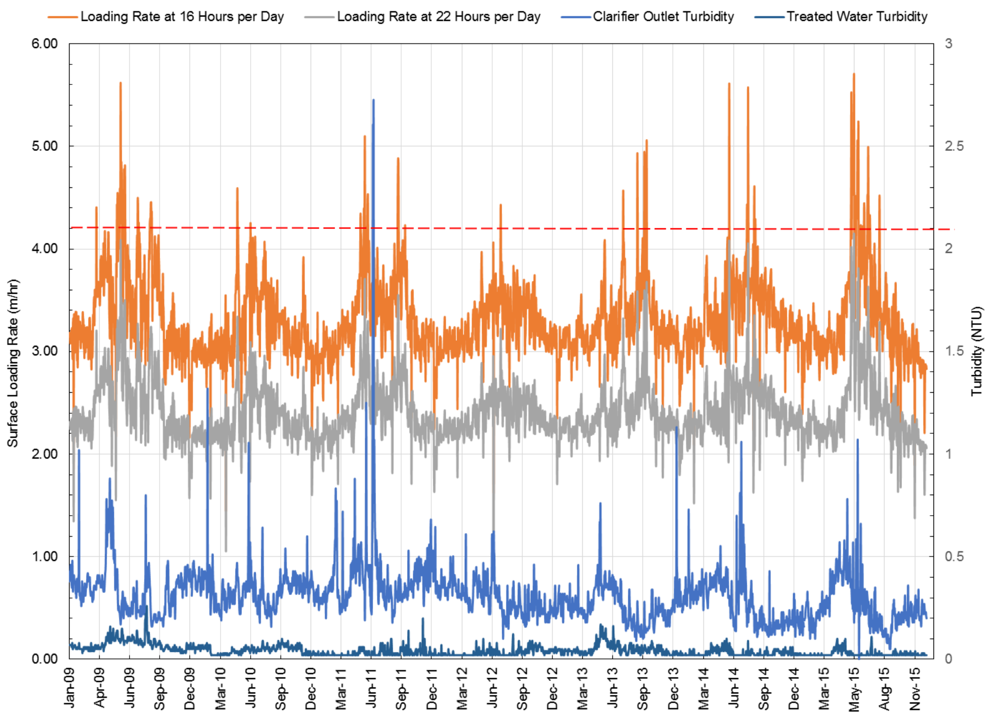


Figure 6.1: Solids Contact Clarifier Surface Loading Rate and Outlet Turbidity

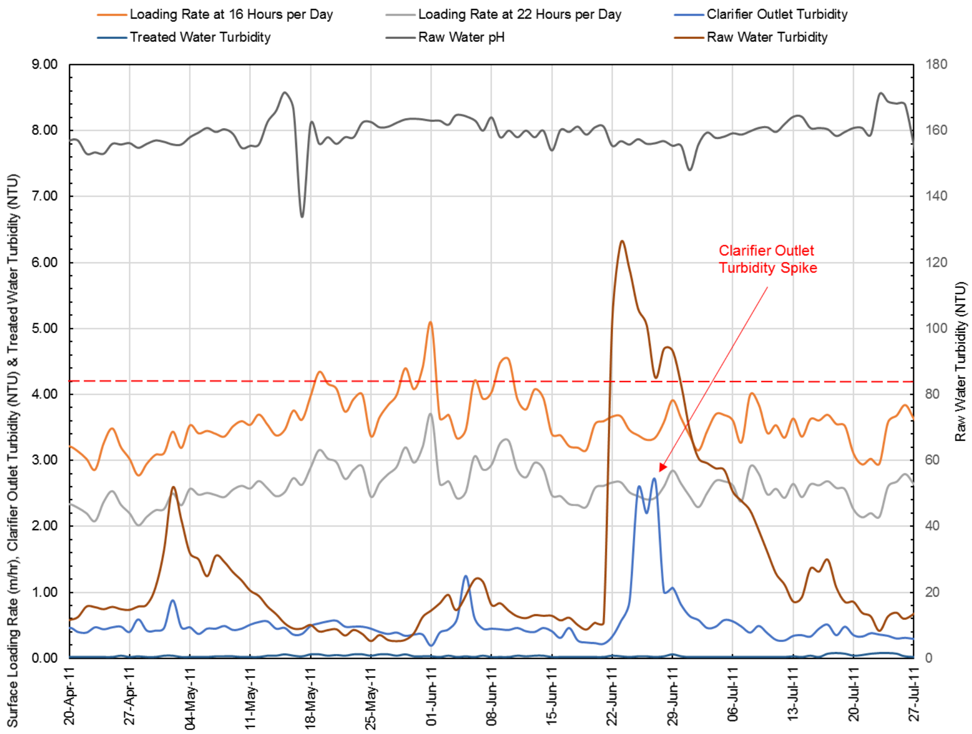


Figure 6.2: April to July 2011 Clarifier Measured Parameters

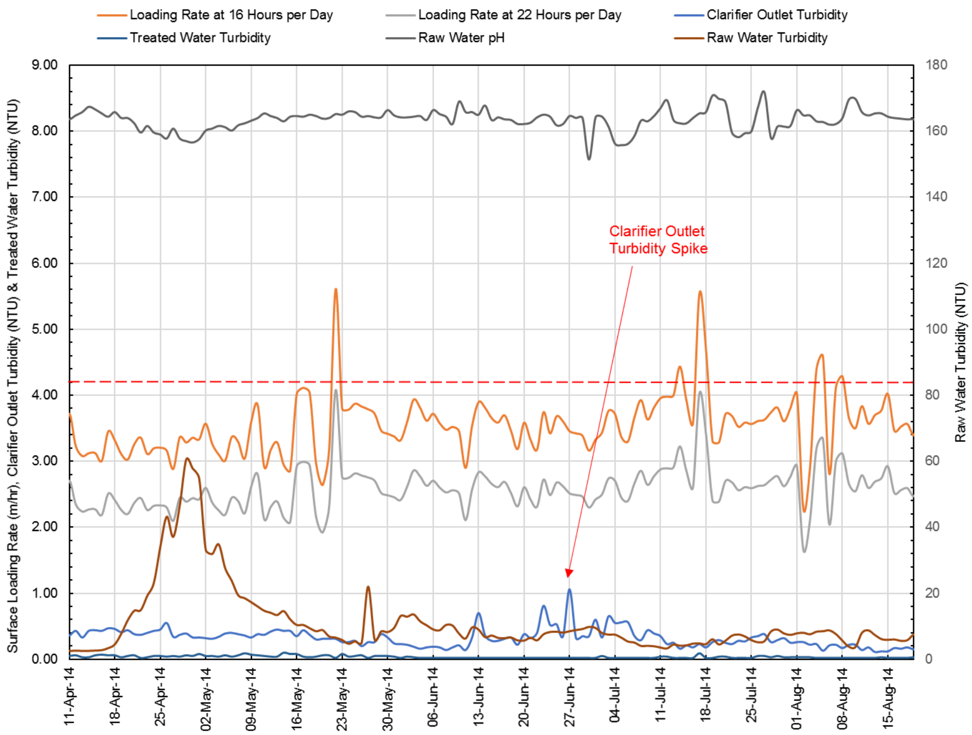


Figure 6.3: April to August 2014 Clarifier Measured Parameters

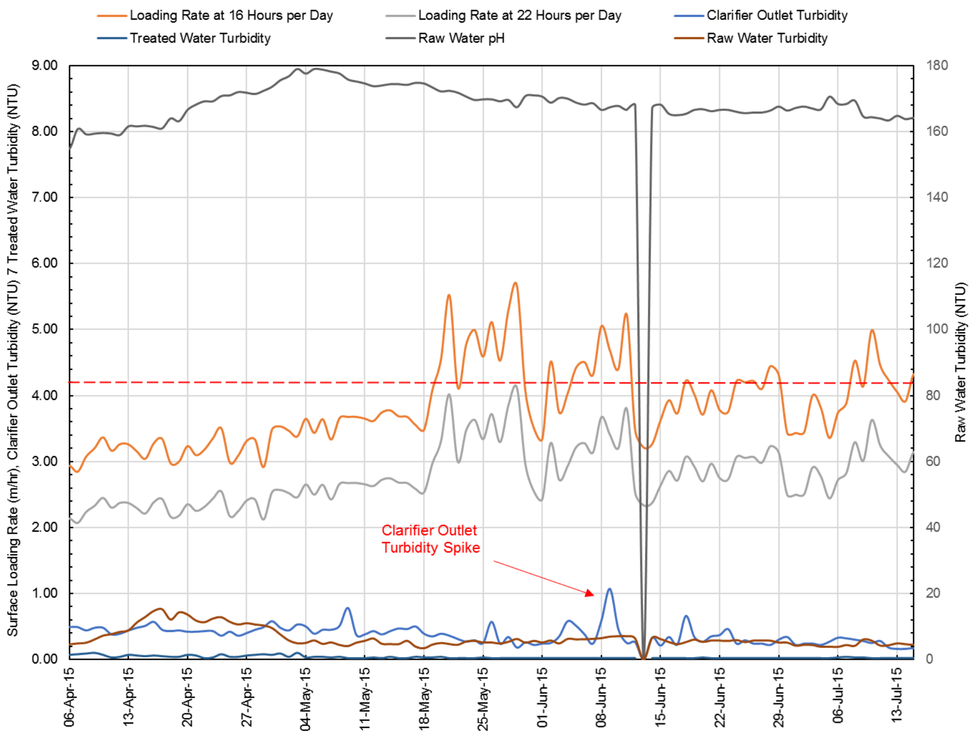


Figure 6.4: April to July 2015 Clarifier Measured Parameters

In considering the downstream impact from variations in the clarifier's performance, Table 6.11 shows that the regulatory requirements for the filter outlet turbidity were met with during the two "high clarifier outlet turbidity" events in July 2014 and June 2015, which are shown in Figures 6.5 and 6.6.

It is very important to note that the outlet turbidity value from each filter is measured 24/7. As such, elevated turbidity values are recorded during filter washing, and are likely due to movement of air and washwater through the filter. As no water enters the clearwell during filter washing and filters are first ripened by filtering to waste before the water is allowed to enter the clearwell, these occasional spikes should not be a considered further. The City is currently working towards only recording filter turbidity when the filter outlet valve is "not closed", and as shown in Table 6.11 below, the 95th percentile outlet turbidity requirements of less than 0.3 NTU were met during both of these events based upon the 24/7 data.

Table 6.11: Individual Filter Outlet Turbidity during High Clarifier Outlet Turbidity Events

	Filter Outlet Turbidity (NTU)		
	Maximum	95th Percentile	Average
June 26th to 29th 2014			
Filter 1	1.01	0.08	0.06
Filter 2	0.13	0.04	0.03
Filter 2	1.01	0.04	0.02
Filter 4	1.00	0.06	0.03
June 8th to 10th 2015			
Filter 1	0.28	0.04	0.03
Filter 2	1.01	0.04	0.02
Filter 2	0.06	0.03	0.01
Filter 4	0.26	0.03	0.02

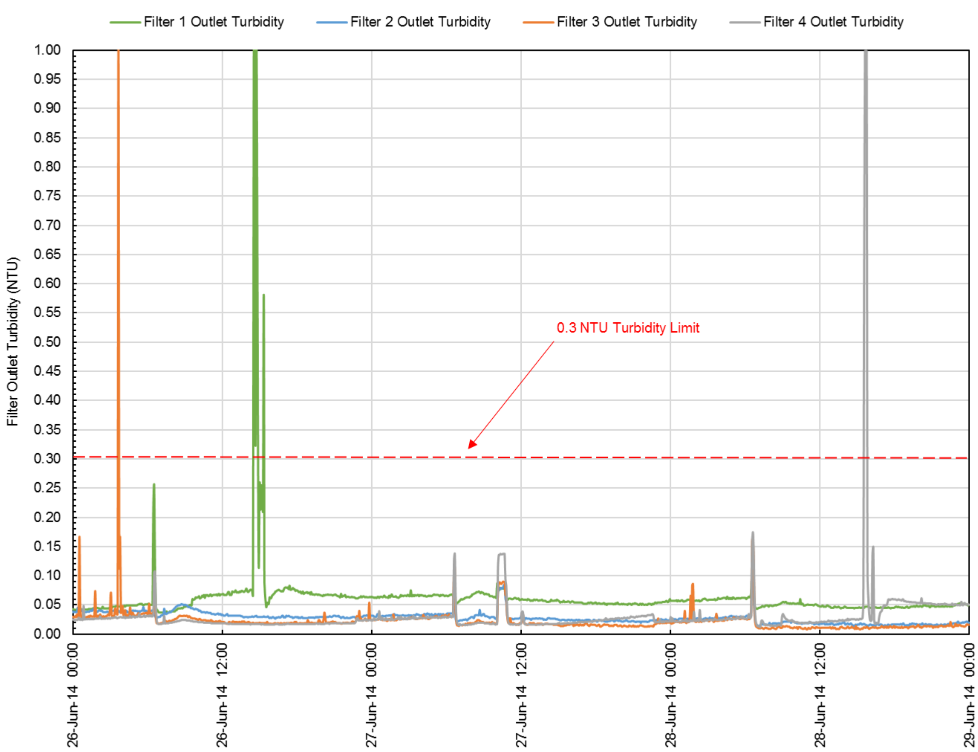


Figure 6.5: Filter Outlet Turbidity from June 26th to June 29th 2014

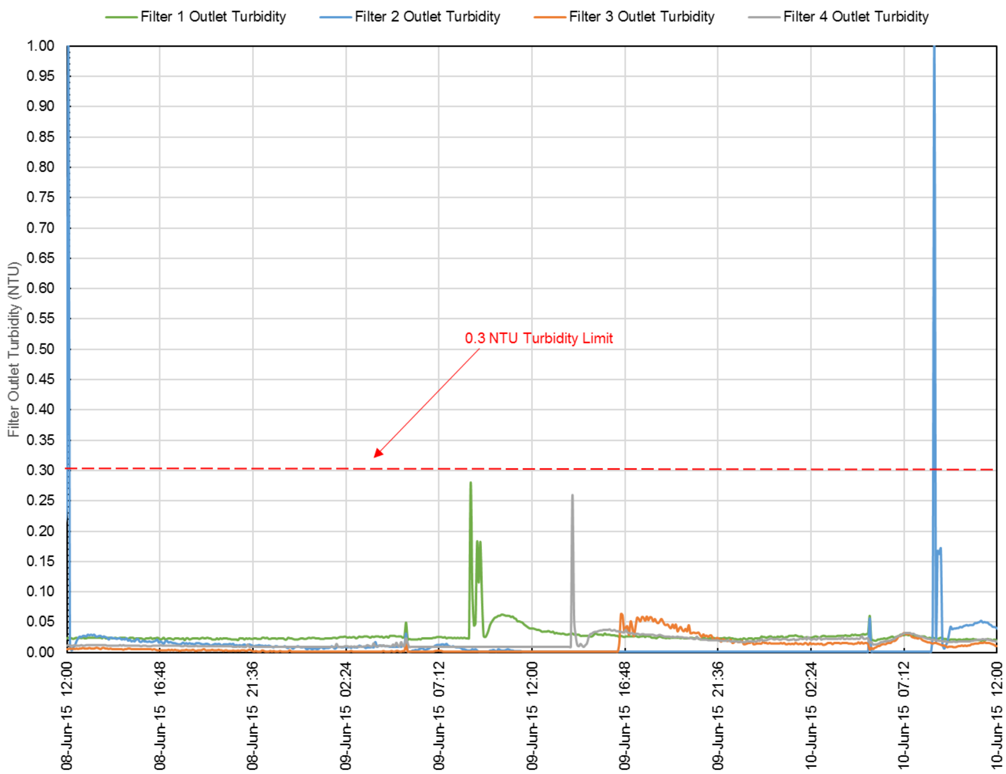


Figure 6.6: Filter Outlet Turbidity from June 8th to June 10th 2015

The presented information highlights that applying a specific surface loading rate to the clarifier does not fully account for the variables that have to be balanced when operating it. In addition, both the impact of the filtration stage on water quality and the requirements of compliance point at the outlet of the media filters is not accounted for by applying only a surface loading rate.

Therefore, on this basis ISL recommends that further work is undertaken to test and validate the joint performance of the solids contact clarifier and the media filters, under different raw water conditions such that the City can be better informed on determining the combined rating of these process stage.

For the purpose of this WSA, the previously established surface loading rates of 4.2 m/hr will be applied to provide a full and complete assessment. Table 6.12 below shows the daily raw water and clarifier production volumes based upon variations in the hours run time and a 2% loss across the clarifier only, due to desludging. Within Section 5.0, a 5% loss was applied to accounted for losses across the whole WTP which includes losses for both clarifier desludging and filter washing.

Table 6.12: Clarifier Daily Volumes based upon a 4.2 m/hr Surface Loading Rate

Run time (hrs)	Raw Water Daily Volume (m^3)	Clarifier Daily Production Volume (m^3)	Equivalent Population
16	14,112	13,835	27,128
18	15,876	15,565	30,519
20	17,640	17,294	33,910
22	19,404	19,024	37,301
24	21,168	20,753	40,692

The values presented within the above table align with the conclusions established in the previous WSAs, which identify the solids contact clarifier as the limiting treatment factor within the City's water treatment process. Based upon these figures and the projections provided in Section 5.0, the treatment capacity of the solids contact clarifier will be exceeded due to maximum day demands in 2026, based upon 24-hour operation of the WTP. As 2026 approaches the required flowrate to meet demands will increase the challenges to the operations staff to maintain the performance of the solids contact clarifier.

Table 6.13 below comments on the condition of the solids contact clarifier, as well the remaining service life and actions that are required. Whilst the table below focuses on the installed equipment, it is very important to note that there is only one solids contact clarifier employed within the WTP. Failure of this unit process requiring its removal from service will significantly impair the performance of the water treatment process, possibly resulting in a reduced treated water quality or production volume.

Table 6.13: Solids Contact Clarifier Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Polymer Dosing Point	PVC pipework places polymer in feed well of Clarifier. Drop tube in place to introduce the polymer into the mixing zone	No issues raised	10+	
Lime / Carbon Dosing Point	Addition points for lime and carbon which are used seasonally to add chemicals to the feed well. Lime is normally added at this location.	No issues raised	10+	
Settling Tubes	There are patches of sludge on the surface of the tubes. No holes or missing spaces observed	Presence of sludge on surface of tubes noted and monitored.	10+	

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Drive / Rake Mechanism	Running smoothly, looks well maintained.	Gear box oil changed twice a year, and analysis completed regularly to check bearing conditions	15+	
Sludge Pump P-123	No leaks on pump, new motor has been installed.	No issues raised	10+	
Sludge Pump P-124	No leaks on pump, some surface corrosion on the pumps observed.	No issues raised	10+	
Associated Pipework and Valves	No issues observed	No issues raised	15+	
Associated Building / Structure	Evidence of seepage from clarifier on exterior surface of basin Calcification and paint peeling on outside walls.	Clarifier has been reviewed by specialist and further investigations will be required in the future. Coatings were recommended inside the clarifier, however cure time was 7 days and it is not possible to accommodate this requirements at this time.	Building - 30+ Clarifier Basin – 10+	City's Operations Team to monitor the condition of Clarifier basin
Electrical and Controls	No issues observed	No issues raised	10+	

In assessing the performance of the solids contact clarifier with the future projected flows, it is clear that the clarifier is a key restriction that will prevent the water treatment plant from meeting the future demands after 2026. Based upon the operation and historical data, the current solids contact clarifier is operating in the upper range of its treatment capacity and no further reasonable modifications could be made to increase its treatment capacity. Therefore, ISL recommends that the City consider and develop a concept design to increase the treatment capacity of the clarification and filtration stages. This may include:

- The design and construction of a second solids contact clarifier to provide the addition treatment capacity. Additional filtration and disinfection capacity may be required (see subsequent sections) to provide sufficient treatment, which might be possible to implement in a phased approach.
- Focusing on the optimization of the media filtration to increase its solids removal. Through pilot studies investigate and demonstrate the implementation of changes to the media configuration such that higher solids loading can be accommodated, thus allowing water with a higher solids content to leave the clarifier.
- Retrofitting the existing media filters with membrane technology thus allowing the solids contact clarifier to operate at a higher surface loading rate. The additional solids loading on the membrane filtration stage would be prevented from passing into the disinfection stage by the semi-permeable membrane barrier, which will also provide a minimum of 3-log reduction for both Cryptosporidium and Giardia.

6.2.4 Media Filtration

Upon leaving the solids contact clarifier both a filter aid and chlorine is added to the water ahead of the media filtration stage to improve filter performance and disinfect the water respectively. The filtration stage at the City's WTP is comprise of four rapid gravity dual media filters, which were refurbished in 2004. As part of the refurbishment, a new underdrain system was installed, the media was replaced and the redundant surface water wash was replaced with an air scour system. Each filter has an effective filtration area of 27 m^2, based upon the media length of 6.7 m and a media width of 4.025 m which were obtained from the supplier's drawings. The installed sand depth upon completion of the refurbishment in 2004 was 18" (457 mm), which was covered with 24" (610 mm) of anthracite. This exceeds the minimum requirements specified by Saskatchewan Water Security Agency (SWSA).

The current operation of the process allows washing of the media filters to be manually initiated upon a high pressure drop across the media, or if the filter is in service for more than 8 days. The whole sequence from taking a filter off line and returning it to service (after filter to waste) takes approximately an hour and ten minutes to complete. During this time the flow of water to the distribution system is reduced by only running the smaller variable speed distribution pumps and the pumps at the West End Reservoir are started to meet the remaining distribution system demand. In summer all four filters are required to be in service, however during the winter months only three filters are online, which are rotated in and out of service as filter washing is required. Since their refurbishment the filters have been monitored, maintained with any excess material removed from the filters. As such they are in excellent condition.

The turbidity of the water leaving each filter is continually monitored using online instruments, which records the turbidity value of the water within the outlet pipework whether the filtered water is directed to the downstream clearwell or not. Upon completion of a filter backwash, the filter is entered into a "rewash mode" where the filter is allowed to ripen while filtering to waste. Tables 6.14 to 6.17 below summarized each filter's outlet turbidity from 2012 to 2015. Based upon the provided one, four or five-minute data collection intervals, the data below shows that for each year the performance of the filter stage meets the regulatory requirements of meeting less than 0.3 NTU for 95% of the time (i.e. the 95th percentile).

With regards to the maximum values shown below, it should be noted again that the outlet turbidity from each filter is monitored 24/7. As such, elevated turbidity values are recorded during an instrument's maintenance / cleaning and during a filter backwash. During periods of filter washing no water enters the clearwell, and each filter is first ripened by filtering to waste before the water is allowed to enter the clearwell. As each high value reads 1.01 NTU it is very likely that the instrument is over range when this value was recorded, and a true turbidity reading is not being provided.

Table 6.14: Summary of 2012 Filter Outlet Turbidity Data (One Minute Intervals)

	Filter 1 Outlet Turbidity (NTU)	Filter 2 Outlet Turbidity (NTU)	Filter 3 Outlet Turbidity (NTU)	Filter 4 Outlet Turbidity (NTU)
Maximum	1.01	1.01	1.01	1.01
Minimum	0.00	0.00	0.00	0.01
Average	0.06	0.05	0.06	0.06
99th Percentile	0.18	0.15	0.16	0.15
95th Percentile	0.12	0.09	0.10	0.11

Table 6.15: Summary of 2013 Filter Outlet Turbidity Data (Four Minute Intervals)

	Filter 1 Outlet Turbidity (NTU)	Filter 2 Outlet Turbidity (NTU)	Filter 3 Outlet Turbidity (NTU)	Filter 4 Outlet Turbidity (NTU)
Maximum	1.01	1.01	1.01	1.01
Minimum	0.02	0.01	0.00	0.00
Average	0.09	0.07	0.07	0.07
99th Percentile	0.34	0.20	0.27	0.23
95th Percentile	0.16	0.13	0.14	0.14

Table 6.16: Summary of 2014 Filter Outlet Turbidity Data (Four Minute Intervals)

	Filter 1 Outlet Turbidity (NTU)	Filter 2 Outlet Turbidity (NTU)	Filter 3 Outlet Turbidity (NTU)	Filter 4 Outlet Turbidity (NTU)
Maximum	1.01	1.01	1.01	1.00
Minimum	0.01	0.00	0.00	0.01
Average	0.07	0.05	0.05	0.05
99th Percentile	0.30	0.20	0.26	0.23
95th Percentile	0.14	0.10	0.13	0.11

Table 6.17: Summary of 2015 Filter Outlet Turbidity Data (Five Minute Intervals)

	Filter 1 Outlet Turbidity (NTU)	Filter 2 Outlet Turbidity (NTU)	Filter 3 Outlet Turbidity (NTU)	Filter 4 Outlet Turbidity (NTU)
Maximum	1.01	1.01	1.01	1.00
Minimum	0.00	0.00	0.00	0.01
Average	0.05	0.05	0.03	0.04
99th Percentile	0.21	0.22	0.15	0.15
95th Percentile	0.11	0.12	0.09	0.09

The water treatment plant has been historically operated over the past 5 years at a raw water flowrate between 600 and 800 m^3/hr, with the number of hours it runs varied each day to meet the daily demands. Based upon these flowrates this results in a filtration rate of between:

- 5.56 and 7.42 m/hr when all four filters are in service, and
- 7.42 and 9.89 m/hr when three filters are in service.

Whilst these historical values are less than the supplier's recommended filtration rate (11.2 m/hr), the Saskatchewan Water Security Agency state within EPB-501 that "production capacity shall be equal to or greater than the maximum plant capacity with the largest filter removed from service". In applying the SWSA basis (three filters in service) and the supplier's recommended filtration rates, the rated hourly production capacity of the filter stage is 906 m^3/hr, which is equivalent to 21,746 m^3/day (i.e. is just above the rated maximum flowrate through the solids contact clarifier).

There are a number of ways that the design and operational basis of the media filter stage can be considered when looking at the future requirements. Within Section 6.2.3, it was established that the maximum instantaneous outlet flowrate from the solids contact clarifier was 21,168 m^3/d, or 882 m^3/hr, when no desludging is occurring. This results in a filtration rate of:

- 8.18 m/hr when all four filter are in service, and
- 10.9 m/hr when three filters are in service.

These filtration rates are below the supplier's recommended filtration rate of 11.2 m/hr. In allowing for 1 filter to be washed per day (i.e. no water entering into the distribution system for 1 hour) and 24-hour operation of the WTP, the daily production volume from the filters can be estimated as:

- 28,693 m^3 with all four filters online, and
- 21,455 m^3 with three filters online

As noted above, the filtration rates historically occurring within the WTP are below the supplier's recommendation. The Saskatchewan Water Security Agency do permit filtration rates between 5 to 12.5 m/hr as noted within EPB-501. In applying the upper limit of this SWSA permitted range with only one filter washing per day, the hourly treatment capacity of the filters can be considered as,

- 1,334 m^3/hr (32,024 m^3/d) with all four filters online, and
- 997 m^3/hr (23,934 m^3/d) with three filters online

The above information regarding filtration rates and flowrates shows that in order to match the flowrates of water leaving the solids contact clarifier at its rate capacity, the media filters operated just below the supplier's recommendation when strictly complying with the requirements of SWSA (i.e. one filter offline). However, it is important to note that in reality when all filters are available significantly more water, or water with an improved water quality, can be produced.

The loading and filtration rates applied to both the solids contact clarifier and media filters only partially take into account the site specific conditions and water quality. Therefore, to aligning with the solids contact clarifier section, ISL recommends as noted previously, that further work is undertaken by the City to test and validate the joint performance of the solids contact clarifier and the media filters, under different raw water conditions and flowrates such that the City can be better informed on determining the rating of these process stage. As part of this assessment the filtration system supplier should be consulted and asked to reassess their recommended filtration rates for the system, based upon past performance.

Table 6.18 below summarizes the condition of the media filtration stage (inc. pumps and blower), as well the remaining service life and actions that are required, based upon the information collected by ISL with input from the City's staff.

Table 6.18: Media Filters Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Filter 101 Internals	Surface of media is clean, no deposits. No media in backwash troughs. Some pitting in the corner of the concrete walls and some patches of coatings are pealing. Underdrain system not observed.	All washes are observed and walls are washed down during a backwash. Media levels are checked on an annual basis. Mild acid washes are untaken on an as needed basis to remove buildup.	15+	

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Filter 101 Pipework & Valves	No sign of corrosion / dis-colorization on pipework. Some oil leakage has dripped onto pipework from air actuators. Original valves in use. A lot of the coating is coming off the floor around this area.	No issues raised.	10+	Operations Team to continue to monitor leaks on actuators. Address as required
Filter 102 Internals	Surface of media is clean, no deposits. No media in backwash troughs. Some patches of coatings are pealing away from the wall. Underdrain system not observed.	All washes are observed and walls are washed down during a backwash. Media levels are checked on an annual basis. Mild acid washes are untaken on an as needed basis to remove buildup.	15+	
Filter 102 Pipework & Valves	No sign of corrosion / dis-colorization on pipework. Original valves in use. A lot of the coating is coming off the floor around this area	No issues raised.	10+	Operations Team to continue to monitor leaks on actuators. Address as required
Filter 103 Internals	Surface of media is clean, no deposits. No media in backwash troughs. Some patches of coatings are coming away from the wall. Underdrain system not observed.	All washes are observed and walls are washed down during a backwash. Media levels are checked on an annual basis. Mild acid washes are untaken on an as needed basis to remove buildup.	15+	
Filter 103 Pipework & Valves	No sign of corrosion / dis-colorization on pipework. Some oil leakage has dripped onto pipework from air actuators. Original valves in use. A lot of the coating is coming off the floor. Coating come away from pipework in one location and bare metal exposed	No issues raised.	10+	Operations Team to continue to monitor leaks on actuators and corrosion on pipework. Address as required
Filter 104 Internals	Surface of media is clean in all filters, no deposits. Some material remains in the backwash trough. Some patches of coatings are coming away from the wall.	All washes are observed and walls are washed down during a backwash. Media levels are checked on an annual basis. Mild acid washes are untaken on	15+	

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
	Underdrain system not observed.	an as needed basis to remove buildup.		
Filter 104 Pipework & Valves	No sign of corrosion / dis-colorization on pipework. Some oil leakage has dripped onto pipework from air actuators. Original valves in use. A lot of the coating is coming off the floor.	No issues raised.	10+	Operations Team to continue to monitor leaks on actuators. Address as required
Filter Media	Filter media is clean and shows no evidence of sludge build up.	No issues raised	5+	Filter media should be replace very 5 to 10 years as its looses is effectiveness.
Filter Air Blowers	One unit, installed in 2004. Looks like new, pipework into filters also in place and in use.	No operational issues	15+	
Backwash Supply Pump BSP-101	Pump is not run very intermittently. No indication of water leakage from the pump. Pump has never been removed based upon condition of bolts and fixings. Some surface rust on pump case/ body where coating has been chipped away and on the discharge fittings	Inspection of clearwell noted that the column for this pump had significant scale and rust build up in the space above the water line. There is no backup pump. Water is drawn from distribution system through pressure reducing valve should this pump fail.	15+	Re-inspect pump when the clearwell is next drained and inspected. Include the provision of a stand-by backwash pump in any future expansions.
Backwash Waste Pump BWP-101	No water leakage noted on the pump itself. More corrosion noted than seen on other pumps around the base plate. Some corrosion also on the discharge head. Corrosion has spread to electrical conduit. Pump has never been taken apart / out	No operational issues. Smaller pump that is easily replaced and there is a backup in place	5+	Continue to monitor performance and repair / replace as necessary
Backwash Waste Pump BWP-102	No water leakage noted on the pump itself. Corrosion noted around the base plate. Some corrosion also inside on the discharge head.	No operational issues Smaller pump that is easily replaced and there is a full stand-by unit is in place	5+	Continue to monitor performance and repair / replace as necessary

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
	Corrosion has spread to electrical conduit.			
Associated Pipework and Valves	No issues notes, other than comments noted above	No issues raised.	10+	
Associated Building / Structure	Some spalling is evident on the exterior walls of the filters at the lower level. Paint is peeling considerably where the discharge pipes pass through the filer wall. Coated floor has peeled away in some locations (normal wear for 30 years).	No issues raised.	30+	
Electrical and Controls	No issues noted, apart from corrosion on electrical conduit.	No issues raised.	10+	

The historical performance of the media filters shows that the combination of the solids contact clarifier and the media filtration stages is producing water in compliance with the assigned Permit to Operate. In strictly assessing the current treatment capacity of the media filtration stage with the SWSA requirements and the manufacturer's recommendations, the media filters appear to have the same treatment capacity that the solids contact clarifier. However as previously noted, the true treatment capability of the media filter is unknown based upon the site specific conditions.

It is clear from the information presented within this report that the treatment capacity of both the solid contact clarifier and the media filters are similar and strongly linked, and that further work on-site is required to ascertain what their true treatment capacity is. A capacity upgrade to the clarification stage, will also require a similar capacity upgrades to the filtration stage. In the event of such an upgrade the City could consider:

- The expansion of the current media filtration stage, using the same design basis to meet the SWSA requirements, with the addition of a second backwash pump and improvements to the supply of water for backwashing.
- Retrofitting the existing media filters with membrane technology thus allowing the solids contact clarifier to operate at a higher surface loading rate. The additional solids loading on the filtration stage would be prevented from passing into the disinfection stage by the semi-permeable barrier and membranes will also provide a minimum of 3-log reduction for both Cryptosporidium and Giardia, independently of the solid contact clarifier's availability.

6.2.5 Disinfection

Within the City's WTP, disinfection of Giardia and viruses is provided by the application of chlorine gas and contact time. As demonstrated within Section 3.0 of this report, the application of the clearwell in conjunction with the pipeline volume to the first customer has historically provided a 4-log reduction in viruses and a 0.5-log reduction in Giardia. The information provided within previous sections illustrated that in order to provide the required CT for the installed pumping equipment under the regimes they operate, and using the historical free chlorine residuals, both the clearwell and pipeline are required for all conditions when assessing the Giardia log reduction, and in most of the conditions when assessing the virus log reduction.

In situations where the clarifier was or will be off-line, the performance requirements for the disinfection stage are adjusted as listed below, and as stated within EPB-501, to account for the application of direct filtration. As stated above the existing disinfection configuration has the ability to meet these revised requirements without any special modification to the disinfection system.

- Log reduction for viruses reduced from 2-log to 1-log, requiring 3-log virus reduction to be provided across disinfection.
- Log reduction for Giardia reduced from 3-log to 2.5-log, requiring 0.5-log Giardia reduction to be provided across disinfection.

In reviewing the requirements for log reductions in Cryptosporidium, ISL noted that the SWSA requirements call for 3.0-log reduction in Cryptosporidium for all surface waters. Within EPB-201 (October 2012), Table 3.2 states a 2.0-log reduction credit can be recognized for conventional sedimentation / filtration, and a 1.0-log reduction is required across disinfection. However, within EPB-501 (November 2015), Table 3.2 states that a 3.0-log reduction credit can be recognized for conventional sedimentation / filtration, which aligns with the standards in place within other provinces within Canada.

On the basis that the City's Permit to Operate only refers to a turbidity requirement for each filter outlet, which is a requirement of the above Cryptosporidium log reduction credits, ISL is working on the basis that the conventional sedimentation / filtration applied at the City's WTP provides the required 3-log reduction in Cryptosporidium. However, it is noted that in the situation where the clarifier is remove from service, the direction filtration employed by the City is only recognized as providing a 2.5-log reduction for Cryptosporidium. As chlorine and contact has no impact on Cryptosporidium, other measures such as increased monitoring and timing of work must be used to mitigate the risks to water quality in this situation.

With regards to the current processes capabilities to achieve the required level of disinfection in the future, Table 3.5 to 3.10 within Section 3.0 show that both the clearwell and the existing pipeline volume to the first customer must be used. With the 0.5-log reduction for Giardia placing the highest criteria on the disinfection stage, Table 6.19 below shows the calculated future CT using combinations of the existing installed pumping equipment and the historical free chlorine residuals. To put these flowrates in perspective, two large pumps running provides a flow of 383 L/sec has the capacity to meet the projected maximum day demand for a population of 64,831, which is forecast to occur in 2050.

Table 6.19: Combined Giardia CT Calculation (0.5-log Reduction, 0.5 ^{0} C, pH of 8.22, Baffling Factor of 0.3 for Clearwell & 1.0 for Pipeline).

Flow Condition	Flow rate (L/sec)	Flow rate (L/min)	Contact Volume (L)	Time (T10) (Minutes)	Free Chlorine Residual (mg/L)		CT (mg-min/L)	CT Required (mg-min/L)	CT Ratio
One Small Distribution Pump	156	9,367 (Max Day Pop 26,477)	2,841,275	237.6	Min	0.64	152.1	52.2	2.9
					Ave	1.12	266.6	56.3	4.7
					Max	1.92	456.3	62.5	7.3
One Small and One Large Distribution Pump	347	20,847 (Max Day Pop 58,863)	2,841,275	106.8	Min	0.64	68.3	52.2	1.3
					Ave	1.12	119.8	56.3	2.1
					Max	1.92	205.0	62.5	3.3
Two Large Distribution Pumps	383	22,961 (Max Day Pop 64,831)	2,841,275	96.9	Min	0.64	62.0	52.2	1.2
					Ave	1.12	108.8	56.3	1.9
					Max	1.92	186.1	62.5	3.0
All Distribution Pumps Running (3 in total)	539	32,328 (Max Day Pop 91,278	2,841,275	68.9	Min	0.64	44.1	52.2	0.8
					Ave	1.12	77.3	56.3	1.4
					Max	1.92	132.2	62.5	2.1

The calculation for CT is influence by two key variables, the free chlorine residual and the flowrate of water through the contact volume. As can be observed in the above table, when all three pumps are running (which is a theoretical scenario), the chlorine residual must be kept elevated above 0.8 mg/l to ensure that the required 0.5-log reduction in Giardia is achieved.

These calculations show that both the clearwell and pipeline must be maintained to ensure adequate disinfection occurs with regards to Giardia. This has particular importance, as a future modification of the pipeline to the first customer must be such that the provision of CT for Giardia is not adversely affected and aligns with the future operation of the system. Therefore, ISL recommends that no connections are made to the distribution system prior to the existing first customer, unless the required contact volume and conditions required to achieved the required CT are maintained.

It is the intent of the City to modify the supply of water to the distribution system, by installing a dedicated fill line from the WTP to the West End Reservoir, such that the whole distribution system would be fed under normal operation with potable water from the reservoir. Within the Master Plan that focuses on the distribution system provided by ISL in May 2016, the dedicated fill line would likely connect to existing system at 49th Avenue and 62nd Street.

The initial advantage of the dedicated fill line would be the additional 6.31 km of 750 mm diameter pipework, plus the volume within the West End Reservoir that would be provided as additional contact volume. However, the assessment of future distribution system within the Distribution Master Plan was undertaken on the basis that under certain circumstances, fire flow would be provided by from both the WTP and the West End Reservoir (i.e. two water supplies into the distribution system). As there is possibility that water entering the distribution system under fire flow conditions would bypass the additional 6.31 km of pipeline and the West End Reservoir, this additional contact volume cannot be accounted for within any future CT calculations.

During the ISL's review of the water treatment plant it was not possible to inspect the clearwell or the contact pipework. Through our discussions with the operations staff, comments were gathered and have been summarized along with the estimated remaining service life and actions within Table 6.20 below.

Table 6.20: Disinfection Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Primary Chlorine Dosing Point	Chlorine is added to the outlet of the clarifier ahead of the filters. There is no backup dosing line / pipework to this dosing point.	There is a secondary dosing point for chlorine and there is sufficient material on site run temporary lines in an emergency.	10	Operations staff to monitor and replace when required
Secondary Chlorine Dosing Point	Chlorine can be added partway through the clearwell. There is no backup dosing line / pipework to this dosing point.	This is a backup dosing point chlorine additional and there is sufficient material on site run temporary lines in an emergency.	10	Operations staff to monitor and replace when required
Clearwell	Not observed.	Cleaned out in 2015 as it was believed to be causing issues with Algae / Taste and Odour. Material was removed. Baffle channels between clarifier and filters are difficult to access and clean.	20+	
Contact Pipeline	Not observed. Installed in 1984, 32 years old.	No issues raised	20+	
Associated Pipework and Valves	Not applicable			
Associated Building / Structure	Not observed			
Electrical and Controls	Not applicable			

In conclusion the configuration of the existing disinfection stage has the capability to achieve the required log-reduction for Giardia and viruses, and provide coliform free water as demonstrated within the previous sections. The continued use of both the clearwell and the pipeline will be required for the foreseeable future.

The comments above make reference to the treatment requirements for Cryptosporidium, and highlights the vulnerability of the process relying upon on single contact clarifier. In the event that the clarifier is removed from service, the remaining treatment process will fall short of the requirements by 0.5-log reduction in Cryptosporidium. In this event, the City would continue to work with the Saskatchewan Water Security Agency to meet their requirements.

Recommendations for improvements of the baffling within the clearwell could be made by ISL, however the improvement in performance does not appear to be a regulatory requirement at this time or be cost effective. Therefore, ISL recommends that:

- No new direct connections are made to the WTP upstream of the current first customer, unless additional contact volume or safeguards to ensure the required CT are included within the design
- The City determine the course of action to provide adequate treatment should the clarifier be removed from service

In addition, the free chlorine residual used for the CT calculation is measured and monitored on the outlet of the distribution pumps, partway through the disinfection stage. The ideal location to measure this variable when assessing the applied CT is at the end of the contact volume, such that adequate disinfection is ensured. Whilst this is not practical in this case, ISL recommends that on a regular basis the City staff measure the free chlorine residual at the first customer and compare the reading to the recorded values from the online instrument.

6.2.6 Chemical Dosing Systems

The water treatment process employed by the City of Lloydminster includes the storage and addition of several chemicals including alum, lime, powered activated carbon, polymer, filter aid and chlorine gas. Within this section of the report the complete chemical dosing system will be reviewed and assessed from storage system through to the dosing equipment and dosing points.

Alum Dosing

Alum is delivered to the WTP in articulated tankers and stored within a single 65,000 kg tank, which is maintained at around 40,000 kg of alum. The alum is added to the raw water using one of two Wallace and Tiernan dosing pumps, which were part of the original installation in 1984. Each dosing pump has the capacity to add 13 L/min of alum to the water. Table 6.21 below summarizes both ISL and the City's comments on the alum dosing system, as well as estimating the remaining service life.

Table 6.21: Alum Storage and Dosing System Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Alum Unloading Point	Unlocked connection point, no containment beneath unloading point. Indication has been provided to show when tank is full. No safety shower in proximity to unloading point	Loading points are labelled and are different sizes to avoid mixing. Loading point has been reviewed by supplier on more than one occasion and SOP is in place to address safety concerns.	20+	
Alum Storage Tank	Overflow goes to sanitary system. No leaks or damage noted. Liner recently replaced.	There is a secondary smaller tank available for short term storage in the event that the primary tank fails.	20+	Replace liner as per suppliers recommendation

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Alum Dosing Pump P-109	Original pumps, some of the PVC pipework has been replaced. Pumps are operated using VFD.	Pumps are maintained and spares can be obtained. No issues reported	5+	Pumps can be rebuilt provided spares continue to be available
Alum Dosing Pump P-110	Original pumps, some of the PVC pipework has been replaced. Pumps are operated using VFD.	Pumps are maintained and spares can be obtained. No issues reported	5+	Pumps can be rebuilt provided spares continue to be available
Associated Pipework and Valves	No leakage or other issues observed. Sections replaced as required.	No issues reported	10+	Replace pipework sections and fittings as required
Alum Dosing Point	Flexible hose and PVC fitting used. Located on raw water gravity line. Will need to be moved in raw water pumps are used.	No further issues reported	5	Replace hose and fittings as required
Associated Building / Structure	No chemical containment with regards to the dosing. Any leakage goes to sanitary drains. Eye wash bottles and hoses are present.	No issues reported	30+	
Associated Electrical and Controls	No issues observed	No issues reported	10+	

Using the historical chemical consumption information and the projected maximum day flowrates included within this report, the existing Alum pump capacity of 13 L/min is sufficient to meet the projected maximum alum dose of 88 mg/L in 2045, which is equivalent to a population of 58,531.

Lime

Hydrated Lime is delivered to site and unloaded into a single 27,750 kg lime silo, which is maintained at around 20,000 kg. Incorporating both a feeder/ slurry system beneath the lime silo (rated at 2,000 kg/d) and a single transfer pump, lime is added to the mixing zone within the solids contact clarifier. The City staff ensure the operation of this system by descaling on a regular basis using hydrochloric acid and replacing components when required. For example, the centrifugal end suction lime transfer pump has been replaced a number of times and is set up in such a way as to ensure a quick change over using the boxed spare that is onsite. Table 6.22 summarizes both ISL and the City's comments on the lime dosing system, as well as estimating the remaining service life.

Table 6.22: Lime Storage and Dosing System Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Lime Unloading Point	Unlocked connection point, no containment beneath unloading point. Indication has been provided to show when tank is full. No safety shower in proximity to unloading point	Loading points are labelled and are different sizes to avoid mixing. SOP is in place to address safety concerns.	20+	
Lime Hopper	Some surface corrosion noted. No leaks observed.	Internal inspection complete previously and no corrosion has reported.	20+	
Lime Feeder / Slurry System	System works. Slurry concentration is varied for control.	Replacement parts for the clutch are no longer available.	< 5	New feeder / slurry system is required as existing cannot be maintained
Lime Dosing Pump P-112	New centrifugal pump, constant speed installed. Boxed spare on site ready to go.	Spare pump available on site.	< 5	High wear item. Replace with boxed spare when required.
Associated Pipework and Valves	Large dimeter tubing is in use with large radius bends to minimized buildup and blockages. No issues noted	Operational staff experienced with blockages. Spare tubing on site	< 5	Prone to blockages, replacement on site.
Lime Dosing Point	Combination of hoses and PVC pipework used.	No issues raised	10+	
Associated Building / Structure	Acid has eaten floor beneath Hydrochloric Acid Container. Some corrosion and discoloration due to lime, especially around the drain covers	No issues noted	30+	
Electrical and Controls	No issues noted	No issues noted	10+	

Using the historical chemical consumption information and the projected maximum day flowrates included within this report, the existing capacity of the lime feeder (2,000 kg/d) is sufficient to meet the projected maximum day dose rate of 19 mg/L in 2045, population of 58,531.

Powered Activated Carbon

Powdered Activated Carbon is used within the water treatment process on a seasonal basis. Delivered to site in 25 kg bags, approximately 3,000 kg is stored on site for use when required. With a feeder capacity of 2,000 kg, the carbon is wetted and added to the process using a progress cavity pump. The normal dosing point is located ahead of the clarifier where the flash mixer (FM-101) used to be installed. The areas where the activated carbon is loaded and wetted are coated with a fine layer of black dust. Table 6.23 summarizes both ISL and the City's comments on the powdered activated carbon dosing system, as well as estimating the remaining service life.

Table 6.23: Powdered Activated Carbon Storage and Dosing System Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Carbon Make-up Tank ACDT-101 & MX-104	System is running, carbon is covering a lot of equipment. Duct tape has been used to seal the system and stem the build-up of carbon in the room.	Replacement parts for the clutch are no longer available.	< 5	New makeup system required as existing system cannot be maintained
Carbon Dosing Pump P-117	Pumps are running, some sections of PVC pipework have been replaced	Pumps are not original pumps, replaced when needed	< 10	
Carbon Dosing Pump P-118	Pumps are running, some sections of PVC pipework have been replaced	Pumps are not original pumps, replaced when needed	< 10	
Associated Pipework and Valves	No issues noted, flushing system in place	No issued noted	10	
Carbon Dosing Point	Combination of hose and PVC fitting used.	No issues noted	5	Replace hose and fittings when required
Associated Building / Structure	No issue noted.	No issues noted	30+	
Electrical and Controls	No issue noted.	No issues noted	10+	

Using the historical chemical consumption information and the projected maximum day flowrates included within this report, the existing capacity of the powdered activated carbon (2,000 kg/d) is sufficient to meet the projected maximum day dose rate of 34.7 mg/L in 2045 with a population of 58,531

Polymer (Flocculation)

The polymer system employed at the water treatment plant utilizes a concentrated polymer emulsion which is diluted and applied to the process within the mixing zone of the solids contact clarifier. Delivered to the WTP in a 55 gallon drums containing 204 kg of polymer, the emulsion is transferred into the mixing unit and allowed to mature. The polymer is applied to the process using one of two Wallace and Tiernan pumps each rated for 4.4 L/min. Table 6.24 summarizes both ISL and the City's comments on the polymer dosing system, as well as estimating the remaining service life.

Table 6.24: Polymer Makeup and Dosing System Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Make-up Unit	Original equipment still in used and is well maintained	No issues raised	10+	
Polyelectrolyte Dosing Pump P-120	Original Pumps, still working using VFDs	Pumps are maintained and spares can be obtained. No issues reported	5+	Pumps can be rebuilt provided spares continue to be available
Polyelectrolyte Dosing Pump P-121	Original Pumps, still working using VFDs	Pumps are maintained and spares can be obtained. No issues reported	5+	Pumps can be rebuilt provided spares continue to be available
Associated Pipework and Valves	Some of the original pipework has been replaced. No other issues noted	No issues raised	10+	Replace pipework sections and fittings as required
Polymer Dosing Point	PVC pipework that delivers polymer directly in to the mixing zone	No issues raised	10	Replace pipework sections and fittings as required
Associated Building / Structure	No chemical containment in place. Safety shower in chlorine area across the hall. Hose and eye wash station in room. Coated floor has peeled way in some locations (normal wear for 30 years).	No issues raised	30+	
Electrical and Controls	No issues noted	No issues noted	10+	

Using the historical dosing information and the projected maximum day flowrates included within this report, the existing capacity of the polymer make up and dosing system is sufficient to meet the maximum treatment capacity of the clarifier (21,168 m^3/d, population of 41,505). Should a second clarifier be added to the process, the existing make-up unit can be utilized, however separate dosing pumps should be sized and added to serve the new clarifier.

Filter Aid

On leaving the solids contact clarifier a filter aid is added to the water at a fixed dose rate of 0.05 mg/L to enhance the performance of the downstream media filtration stage. Consisting of a 55 gallon drum and simple dosing pump arrangement, the filter aid (CTI CL2410) is added to the water in proportion to the flow of water leaving the clarifier. Table 6.25 summarizes both ISL and the City's comments on the filter dosing system, as well as estimating the remaining service life.

Table 6.25: Filter Aid Dosing System Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Filter Aid Storage	Filter is supplied directly from the drum, no containment in place. Any spills will go to the sanitary system.	No issues raised	N/A	Consider portable containment system for drum.
Filter Aid Dosing Pump P-109	Small Grundfos DDI Pump, feeds chemical based upon the filter flow	No issues raised	10+	Replace when required
Associated Pipework and Valves	No issues notes, single length of tube running to dosing point.	Spare tubing on site.	10	Replace when required
Filter Air Dosing Point	Tubing drip feeding chemical into clarifier outlet. Mixing provide by downstream channel ahead of the media filters	No issues raised	5	Replace when required
Associated Building / Structure	No comment	No issues raised	30+	
Electrical and Controls	Single receptacle in use with control wiring.	No issues raised	30+	

The dosing pump currently used to add the filter aid is a Grundfos DDI-209 with a capacity of 0.11 Gal/hr. Applying the fixed dose of 0.05 mg/L and the projected maximum day flowrates, the existing dosing pump has the capacity to meet the maximum day demands beyond 2045 (population 58,831).

Chlorine

Supplied under pressure as liquefied chlorine gas in 1 ton drums (tonners), the chlorine gas is drawn from the drum using vacuum based chlorinators and eductors to create a chlorinated solution which is then directed to one of two dosing points. With space for 12 drums, only three drums are ever on site at any time. Two are online in a duty / stand-by configuration, with the third ready to replace the duty drum when it is emptied. The chlorine dosing equipment is configured as two parallel dosing systems, that are dedicated to the addition of chlorine at either outlet of the clarifier, or partway through the clearwell. The systems are cross-connected and normally isolated from one another, such that the two systems back each other up.

The previous WSA confirmed that the exhaust system for the chlorine storage room has been modified to a manual initiation on detection of a gas leak, however the interior access remains along with no observation window within the exterior door. The Guidelines for Chlorine Gas Use in Water and Wastewater Treatment (issued by the Government of Saskatchewan) is clear under clause 1.3.11 that access to the chlorine storage room shall only be provided from directly outside. The City has confirmed that this issue will be addressed when the facility is expanded in the future.

Table 6.26 below summarizes the comments collected with regards to the chlorine dosing system, as well as estimating the remaining service life.

Table 6.26: Chorine Dosing System Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Gas Tonner Storage	Self-contained room with both interior and exterior access (interior access through vestibule). Space is available for 12 drums. Chlorine detection system is online and connected to alarm system.	No issues raised	N/A	
Chlorinator CFD-101	V10k unit which runs on vacuum. Normal operation is to add chlorine to post clarification only.	Serviced annually and a complete spare unit is on site for repairs as necessary	15+	
Chlorinator CFD-102	V10k unit which runs on vacuum. Set up for post chlorination, but can be switched to post-clarification if needed.	Serviced annually and a complete spare unit is on site for repairs as necessary	15+	
Chlorine Dosing Points	Pipework is in good condition. There is only a single pipe to the dosing point.	Repairs / replacements are made as condition deteriorates. Materials are on site for a quick installation	10+	
Associated Pipework and Valves	Pipework in good condition. Eductors are serviced on a regular basis	No issues raised	10+	
Associated Building / Structure	Safety shower, emergency kit and SCBA available. No other issues noted	No issues raised	30+	
Electrical and Controls	No issues noted. No high electrical loads in use	No issues raised	15+	

The existing chlorinator are configured to supply a maximum chlorine flow of 90 Kg/d each. In using the 99th percentile value of the chlorine historical chemical consumption to remove any outliers (i.e. a dose of 3.87 mg/L) and the projected maximum day flowrates included within this report, a single chlorination will reach its capacity around 2034 which is equivalent to a population of 46,569.

6.2.7 Water Treatment Plant Building & Control System

The Water Treatment Plant Building houses all of the treatment process units described previously as well as a laboratory, office space, workshops and control room. The building is constructed from masonry block with sections of metal cladding, and a flat membrane roof. The above ground structure remains in very good condition and there was no evidence of a leakage or damage to the building itself. In reviewing the below ground structure with the City Staff no significant issues were observed as noted previously.

Heating is provided to the WTP by boilers which are installed on the ground floor. The heating system is service twice a year by a third party, with operational staff completing daily checks. Once a month safety checks are completed and the two boilers (duty / stand-by) are rotated. The original boilers were replaced in 2013 as they were at the end of their design life. There are number of makeup air and ventilation that serve sections of the building, with the powdered activated carbon areas served with its own unit.

In addition to the main power incomer, a single diesel generator was installed in 2010 which has the capability of running the entire process using an automatic transfer switch. The original smaller stand-by generator was disconnected in 2014. The new generator has been placed on the preventative maintenance schedule and is inspected / started by the operations staff every month. Every third month the WTP is operated by the staff using the standby generator. On an annual basis the generator is serviced and run in conjunction with a load bank by a third party.

The upgrade of the control system and SCADA for the water supply system is nearing the completion. The upgrade has been completed by the City staff and a local third party who will continue to support the system once completed.

The City's staff have reported no issues with the building and the associated systems. Typically, a building of this type is expected to have a service life of 30 years, but there is no obvious reason why the building and structure should not continue to be serviceable, if maintained, for the next 30 years.

6.2.8 Treatment Capacity

One of the key objectives of the WSA is to determine the treatment capacity of the WTP under assessment, and identify when an upgrade of the facility is required based upon past performance and the rated design capacities. As demonstrated, the WTP operated by the City of Lloydminster has met the historical water demands and performed to a standard that either meets or exceeds the conditions stated within their Permit to Operate, while only operating for part of a normal day.

The combination of the solids contact clarifier and the media filtration stages has resulted in a filtered water quality that is significant less than the required individual filter outlet turbidity of 0.3 NTU, and in most cases is as low as 0.1 NTU. Previous WSAs have identified the solids contact clarifier as the limiting factor with regards to treatment capacity, which is support by this WSA. However, it does appear that based upon information provided recently by the supplier, the refurbishment of the media filters in 2004 was based on the requirement that the filters matched the treatment capacity of solids contact clarifier, which defined the filter media configuration.

The 2010 WSA established that the water treatment plant has the capacity to produce 20,125 m^3 of treated water per day, based upon continuous operation. The evaluation completed within this report by ISL has identified no basis upon which to change this value. Table 6.27 below summarizes the hourly and daily volumes that can be treated by the stage within the treatment plant as it is configured and operated today. As can be observed it is the capacity of the solids contact clarifier and media filtration stages that limit the treatment capacity, and as such the 2026 expansion date predicted within Section 5 of this report stands.

Table 6.27: Summary of Treatment Capacity of WTP Stages

	Hourly Treatment Capacity (m^3)	Stage Losses	Daily Production Volume (m^3)
Raw Water Pumps	1,245	0%	29,895
Solids Contact Clarifier	882	2%	20,753
Media Filtration[1]	906	1 Filter Washing per Day	21,445
Disinfection[2]	1,360	0%	32,659

Notes

1. Based upon three out of four filters online
2. Based upon chlorinator modifications and increase in free chlorine residual.

6.3 Distribution System

The major items of infrastructure that will be assessed within this Section with regards to the distribution system include the:

- WTP distribution pumps
- West End Reservoir
- West End Reservoir pumphouse
- Distribution system

6.3.1 WTP Distribution Pumps

Water from within the clearwell is pumped into the distribution system using a combination of the three pumps originally installed in 1984 and overhaul in 2003. The installation is comprised of two fixed speed pumps (PWP-101 and PWP-102) each with a capacity of 16,532 m^3/d at a discharge pressure of 61 m (598 kPa) and one variable speed pump (PWP-103) which has a capacity of 13,488 m^3/d at a discharge pressure of 59 m (578 kPa).

The water from the WTP is currently discharged directly into the distribution system, with any unused demand entering the West End Reservoir (Reservoir). Under a "normal day" the water demands of the system are currently met by water supplied from both the WTP and the West End Reservoir, as there is another set of pumps at the Reservoir that also feed the distribution system. As demand drops away in the evening, the surplus water from the WTP is used to refill the Reservoir. Once full, the water from the WTP continues to meet the distribution system demands until it is shut down for the evening. Overnight demands are then solely met by the West End Reservoir until the WTP restarts the following morning. Due to the current pipework configuration, the West End Reservoir cannot be filled and used to maintain pressure in the distribution system at the same time.

As noted within the previous WSAs, the supply of water to the distribution system from the WTP, which also feeds the West End Reservoir, is restricted in some way. Work has continued since the last WSA to identify the restriction within the distribution system, however conflicting results and the inability to repeat previous test results have prevented the City from solving this issue. With regards to the rest of the operation of the WTP distribution pumps no other issues were raised by the City's staff.

In moving forward, it is the City's intent to change the basis by which water is supplied from the WTP. As discussed within Section 6.2.5, a dedicated fill line will be installed through which water will be fed directly to and through the West End Reservoir, before entering the distribution system. Section 5.2 of this report established the projected demands on the WTP and highlights the historical disconnect between previously projected flowrates and the flowrates which have actually occurred. On this basis the projections established for the WTP assessment moved away from those used for the distribution system within the Distribution Master Plan. In developing a concept for the dedicated reservoir fill line within the Distribution Master Plans, a similar approach was taken to that applied for the WTP assessment, in that the peaking factor between the Maximum Day Demand and Average Day Demand reduced from 2.0 to 1.5.

Table 6.28: WSA and Master Plans Maximum Day Demand Projections

Year	WSA WTP Projected Maximum Day Demand (m^3/d)	Distribution Water Master Plan Projected Maximum Day Demand (m^3/d)
2018	17,032	20,811
2020	17,755	24,046
2025	19,699	29,754
2035	24,249	40,130
2045	29,851	-

As can be observed in above in Table 6.28 a significant discrepancy still remains between the two projections, which is a result of approaching the projections from two difference perspectives (i.e. historical data versus standard rates). Within the Distribution Master Plan, the capabilities of the pumping equipment at the WTP were assessed with regards to meeting the higher projected maximum day demands in Table 6.28 above. Table 6.10 within the Distribution Master Plan shows that the project demand up to 2025 can be met using two of the currently installed WTP distribution pumps (1 small and 1 large), soon after which all three pumps are required to meet the projected demand, eliminating the stand-by or backup pump capability in maximum day conditions.

Alternatively, in comparing the WSA projections with the Table 6.10 in the Distribution Master Plan, the 2045 maximum day demand of 29,754 m^3/d (1,240 m^3/hr), can be achieved using two of the existing distribution pumps (1 small and 1 large), thus maintaining the availability of a stand-by or backup pump for a significantly period.

Based upon the two sets of projected flowrates presented above and that the dedicated fill line is installed, the existing WTP pumping equipment is capable of lifting water to the West End Reservoir until 2025 in both predictions, after which they deviate. If the WSA projections are used the existing pumping equipment can meet the maximum daily demand until 2045. If the distribution Master Plan is used then additional stand-by pumping capabilities are needed after 2025. To clarify, this analysis focuses on the movement of the maximum required volume from the WTP in any one day (i.e. the maximum day which is based on a 5-day rolling average). In completing this work, the City noted that they wish to understand the implication of not proceeding with the dedicated fill line to the West End Reservoir. As this analysis would now move to looking at the peak hourly flows, this analysis has been completed later in the report under Section 6.3.4

It is important to highlight that the Distribution Master Plan and this assessment was based upon the dedicated fill line being in place, and currently no schedule has been established for the design and installation of this fill line. Therefore, ISL recommends that an analysis should be performed which determines when the dedicated reservoir fill line must be installed, based upon the existing system's response to increasing demands, and both the WTP and reservoir pumping equipment's ability to meet those demands. One established a decision to proceed or not should be made and a schedule established to ensure implementation of the dedicated fill line.

In reviewing the condition of the existing WTP distribution pumps, Table 6.29 below summarizes the comments of both ISL and the City, in addition to estimating their remaining service life.

Table 6.29: WTP Distribution Pump Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
PWP-101	Pump was running evenly with no excessive noise during inspection. Evidence of corrosion starting on base plate.	Pump overhaul in 2003 and lager impellers installed	20	Monitor condition and performance
PWP-102	Corrosion on base plate was observed.	Pump overhaul in 2003 and lager impellers installed	20	Monitor condition and performance
PWP-103	Corrosion on base plate was noted.	Pump overhauled in 2003	20	Monitor condition and performance
Associated Pipework and Valves	No major issues. Some of the epoxy coating is coming away to leave exposed metal. No major pitting noted	No issues noted	10+	
Associated Building / Structure	Coated floor has peeled way in some locations (normal wear for 30 years).	No issues noted	30+	
Electrical and Controls	Conduit on some of the pumps / solenoid wiring are corroding.	Electrical equipment inspected on an annual basis	10+	
Associated Instrumentation	No issues noted	No issues noted	10+	

On the basis that the dedicated fill line to the West End Reservoir is installed, the existing WTP distribution pumps have the pumping capacity to meet the demands placed upon them using the WSA historical basis. Therefore, based upon the information presented above ISL recommends that the City:

- Undertakes an assessment to confirm when the dedicated reservoir fill line has to be in place, based upon both the distribution's system and pumping equipment responses to future demands
- Subsequently make a decision to proceed with the dedicated fill line or not, and if they are to proceed then establish a schedule for implementation of the dedicated fill line

6.3.2 West End Reservoir

The West End Reservoir is comprised of two structures, an above ground circular concrete structure with a capacity of 4,545 m^3 which was constructed in 1971, and below ground compartmented concrete structure built in 2006 with a capacity of 20,201m^3. The reservoirs are configured such that water from the distribution system is directed to the above ground reservoir, from which it flows into the below ground structure before being pumped into the distribution system.

Within the Distribution Master Plan (ISL, 2016) the capacity of the current reservoir storage was assessed against the Saskatchewan requirements using the demands established from the projected land use. The assessment, shown below in Table 6.30, indicated that within 3 years the storage capacity within the distribution system (i.e. at the West End Reservoir) will not meet the regulatory requirements of twice the average daily demand.

Table 6.30: Reservoir Capacity Assessment using Distribution Master Plan Basis

Year	Minimum Storage Requirement (m^3)	Storage Surplus / Deficit (+/-)	% Increase in Storage Needed
2018	27,748	-3,002	12%
2020	32,062	-7,316	30%
2025	39,672	-14,926	60%
2035	53,507	-28,761	116%

As noted previously, the Distribution Master Plan is based upon future average and maximum daily demands developed using standard consumption rates and standard peaking factors for the designated land use, which can lead to conservative projections. Table 6.31 below summarizes the same reservoir sizing, but using the projected ADD applied to the WSA based upon historical data. The information within this table places the need for additional reservoir storage around 2022, according to the Saskatchewan Water Security Agency requirements.

Table 6.31: Reservoir Capacity Assessment Using WSA Basis

Year	Minimum Storage Requirement (m3)	Storage Surplus / Deficit (+/-)	% Increase in Storage Needed
2018	22,708	2,038	0%
2020	23,672	1,074	0%
2025	26,266	-1,520	6%
2035	32,332	-7,586	31%

In 2010, a structural assessment of the above ground reservoir was undertaken to assess the adequacy of the structure and identify any significant problems. The conclusion of the assessment was that the reservoir was in a poor to fair condition with evidence of micro-cracks and exposed rebar. The remaining service life of the reservoir was estimated to be order to 10 years in its assessed condition, and requirements for further analysis and assessment were noted prior to providing recommendations for remedial work to prolong the life of the reservoir. The 2006 below ground reservoir can still be classified as a recent construction and as such has an expected service life in excess of 30 years.

As part of previous work by ISL, the subsequent development of the West End Reservoir was planned in two further stages:

- Stage 2 – The addition of 9,850 m^3 to the existing underground reservoir providing a total storage capacity of 34,596 m^3, and
- Stage 3 – Demolition of the above ground reservoir and construction of a further 11,000 m^3 of storage, providing a total capacity of 41,051 m^3.

Using the WSA projected demands for future requirements, there is no immediate need to increase the storage capacity at the West End Reservoir. However, the condition of the original above ground reservoir is a concern and as such ISL recommend that within the next five years the City add a further 9,850 m^3 to the existing below ground structure and then subsequently demolish the above ground reservoir. Once complete this would provide the City with 30,051 m^3 of storage at the West End Reservoir, which based upon the projections used within this WSA would be sufficient until 2031, which is equivalent to a population of 43,755.

6.3.3 West End Reservoir Pumphouse

The infrastructure installed at the West End Reservoir in 2006, also included a pumphouse that is used to supply water to the distribution system either overnight when the WTP is switched off, or when pressure in the distribution system is low and requires boosting. The pumphouse at the reservoir is comprised of two fixed speed pumps, two variable speed pumps, a re-chlorination system and a diesel stand-by power generator. All four of the installed pumps each have the capability of delivering 371 m3/hr (8,923 m3/d) at a discharge pressure of 42.5 m (416 kPa). The analysis completed as part of the Distribution Master Plan determined that to meet the system demands in the future, the West Reservoir discharge pressure would have to be approximately 38.3m of water (375 kPa). Using the installed equipment's pump curves, the discharge flow from each pump at this pressure will adjust to 416 m3/hr, which will be used for this analysis.

Since installation in 2006 Distribution Pump #1 has required further attention to address bearing and vibration issues. No issues were reported with regards to the remaining three pumps. Based upon the hours run meters on the pump, the two VFD operated pumps, numbers #1 and #2 run the most often with 13,122 and 18,614 hours respectively, whereas the fixed speed pumps run a lot less, numbers #3 and #4 with 118 and 124 hours. On the basis of 10 years of regular daily operation and that only one pump runs at any point in time, these pumps run for between 8 to 9 hours a day.

With regards to the future operations at the West End Reservoir, the City's intent is for the whole distribution system to be fed from the West End Reservoir, with the exception of a significant fire event. When a fire occurs that requires significant water (i.e. maximum fire flow = 225 L/sec), the system would be adjusted such that water would be fed into the distribution system from both the WTP and the West End Reservoir at the same time. This is the basis upon which the Distribution Master Plan was completed.

The consequence of this approach is that all the water required to meet the forecasted hourly demands will have to be supplied on a normal day through the West End Reservoir pumphouse only. To establish the peaking factor for hourly demands in the future, the highest ten hourly volumes from 2013 to 2016 were compiled and reviewed. Tables 6.2 to 6.35 show the highest ten hourly volumes from both the water treatment plant and the West End Reservoir for each year from 2013 to 2016. These values were then compare and divided by the Average Day Demand for their year to establish a historical peaking factor for peak hour demands.

Table 6.32: 2013 Peak Hourly Volume and Historical Peaking Factor

Hourly Volume (m^3)			Peaking Factor (ADD)
WTP	West End Reservoir	Total	ADD = 448.3 m^3/hr
492	463	955	2.13
949	0	949	2.12
946	0	946	2.11
934	0	934	2.08
931	0	931	2.08
929	0	929	2.07
929	0	929	2.07
927	0	927	2.07
910	0	910	2.03
908	0	908	2.03

Table 6.33: 2014 Peak Hourly Volume and Historical Peaking Factor

Hourly Volume (m^3)			Peaking Factor (ADD)
WTP	West End Reservoir	Total	ADD = 455.9 m^3/hr
657	896	1553	3.41
823	314	1137	2.49
954	182	1136	2.49
818	223	1041	2.28
817	212	1029	2.26
787	203	990	2.17
927	0	927	2.03
923	0	923	2.02
743	177	920	2.02
381	473	854	1.87

Table 6.34: 2015 Peak Hourly Volume and Historical Peaking Factor

Hourly Volume (m^3)			Peaking Factor (ADD)
WTP	West End Reservoir	Total	ADD = 452.6 m^3/hr
860	326	1186	2.62
811	375	1186	2.62
847	338	1185	2.62
852	290	1142	2.52
357	751	1108	2.45
808	278	1086	2.40
857	223	1080	2.39
821	255	1076	2.38
795	255	1050	2.32
456	590	1046	2.31

Table 6.35: 2016 Peak Hourly Volume and Historical Peaking Factor

Hourly Volume (m^3)			Peaking Factor (ADD)
WTP	**West End Reservoir**	**Total**	**ADD = 440.9 m^3/hr**
261	910	1171	2.66
235	895	1130	2.56
790	211	1001	2.27
501	482	983	2.23
953	0	953	2.16
952	0	952	2.16
509	405	914	2.07
893	0	893	2.03
852	0	852	1.93
264	528	792	1.80

Using the above information, each year from 2013 to 2015 has been displayed alongside one another in Figure 6.7 below. In reviewing this data with the City, it has been confirmed that the peaking factor of 3.41 that occurred in 2014 was the result of the reservoir fill valve sticking open which resulted in water recirculating back into the Reservoir. As such this high value will be discounted from the analysis. In considering the remaining information, the highest hourly peaking factor for each year occurs in the 2.2 to 2.67 range, which depending on the year is either repeated or the succeeding highest values slowly reduces. This indicates that the peak hourly flows experienced within the system are consistent and thus can be used for establishing an hourly peaking factor. On the basis of this information and to provide suitable safety factor, the City agreed to ISL recommendation of applying a peaking factor of 3.0 times the ADD to the future predictions for hourly demands. This aligns with the peaking factor for hourly demands applied to the Distribution Master Plan, and the suggestion of 2 to 5 times the ADD provided by the Alberta Environment Guidelines (There is no indication in the Saskatchewan Guidelines).

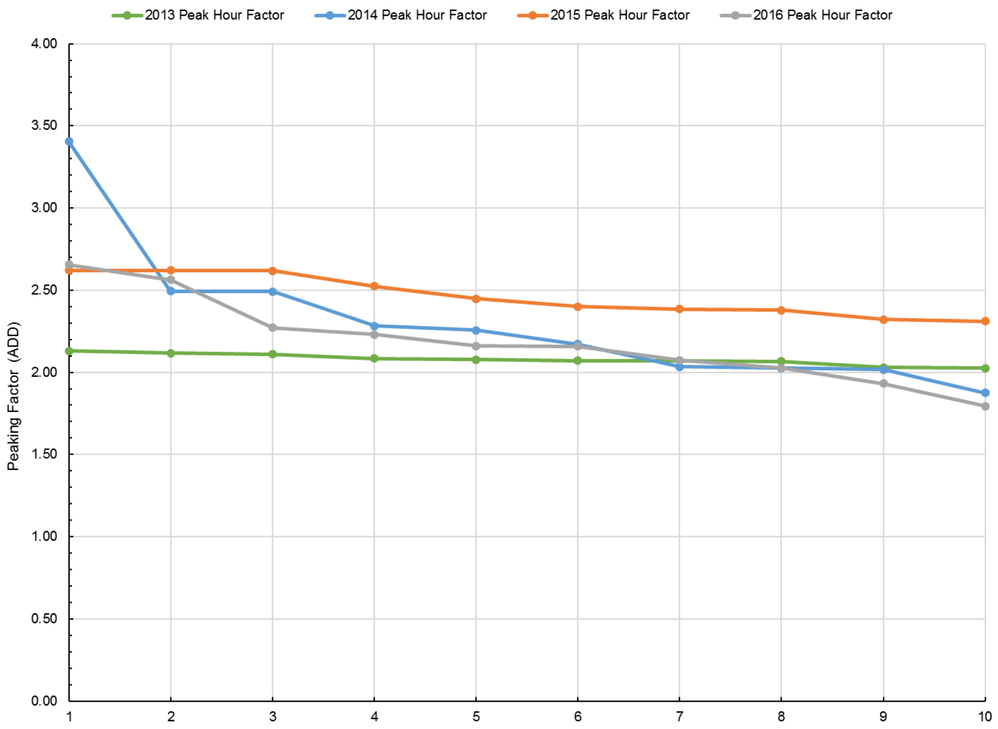


Figure 6.7: Historical Peaking Factor for Hourly Demands

Table 6.36 below shows the projected hourly daily demands derived from the WSA and those projected from the Distribution Master Plan. The comparison shows that the projections using the WSA basis are substantially lower than those developed for the Distribution Master Plan, due to the basis upon which the Average Day Demands are established (i.e. historical data vs. standard rates). To be consistent with the previous sections of the report the assessment of the West End Reservoir will therefore be based upon the projections derived from the WSA basis.

Table 6.36: Projected Peak Hour and Maximum Daily Demands

Year	WSA Projected Average Daily Demand (m^3/d)	WSA Projected Hourly Demands (m^3/hr) (P.F. = 3.0 ADD)	Dist. Master Plan Projected Peak Hour Demands (m^3/hr)
2018	11,354	1,419	1,734
2020	11,836	1,480	2,004
2025	13,133	1,642	2,479
2035	16,166	2,021	3,344

As noted previously the established discharge pressure from the West End Reservoir Pumphouse will be (according to the Distribution Water Master Plan) 38.3m of water (375 kPa), which based upon the provided pump curves will result in a flowrate of 416 m^3/hr. Table 6.37 below shows the number of current installed pumps required to meet the peak hour demands, if water is solely supplied from the West End Reservoir. The corresponding velocity within the 500 mm dimeter outlet pipework is also shown.

Table 6.37: Peak Hour - Number of Pumps and Outlet Pipework Velocity

Year	WSA Projected Peak Hour Demands (m^3/hr)	Number of Existing Pumps Required (416 m^3/hr per pump at 38.3 m (375 kPa))	Outlet Pipework (m/sec)
2018	1,419	4	2.01
2020	1,480	4	2.09
2025	1,642	4	2.32
2035	2,021	5	2.86

In conclusion this information shows that should a dedicated fill line be installed, the existing pumping equipment within the West End Reservoir has sufficient capacity to meet the peak hour demand. However, when a peak hour flows are required, at a peaking factor of 3.0 times the ADD, then all of the existing pumps would need to run. As such this would require the installation of the 5th pump as a standby pump, space for which was include within the original design. With regards to the pipeline velocities, the 500 mm diameter connection is suitable for those short periods when peak hourly flowrates are required. However, after 2025, when the velocity within the pipework reaches 2.5 m/sec, the pipework should be upsized or twinned.

As the above assessment shows that the hourly demands for the City can be addressed using most of the infrastructure currently installed within the West End Reservoir, ISL recommends that the full implications of the implementing the dedicated fill line (i.e. power, spacing, stand-by power) are evaluated and a concept design completed, such that a schedule for design and installation of the dedicated fill line can be developed.

Should the operation of the West End Reservoir continue in line with the current configuration, the installed pumping equipment is capable, in ISL opinion, of meeting the demands of the system for the next 20 years (i.e. an upgrade will be needed when the 5th pump needs to run to meet the peak hour demand). Table 6.38 below summarizes the comments of both ISL and the City, in addition to estimating the remaining service life of the equipment installed within the West End Reservoir pumphouse.

Table 6.38: West End Reservoir Pumphouse Comments, Remaining Service Life and Actions

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Distribution Pump 1	Variable speed pump, which has run for 13,122 hours since installation. Issues with bearing overheating due to arcing resulted in a new specialist bearing being installed. New pump based also installed as the pump was visibly shaking. There is still some vibration that still remains.	No further issues noted	10	City to continue monitoring of pump.
Distribution Pump 2	Variable speed pump, which has run for 18,614 hours since installation. Seal water scaling on pump, sole plate is rusting / corroding	No issues noted	15+ Based upon low hours run.	

Equipment Description	Visual Inspection Comments	Operation's Team Comments	Remaining Service Life (Years)	Actions
Distribution Pump 3	Fixed speed pump. 118 hours of operations since installation. Deposition of rust on pump start up valve	No issues noted	15+ Based upon low hours run.	
Distribution Pump 4	Fixed speed pump. 124 hours of operation since installation. Deposition of rust on pump star up valve	No issues noted	15+ Based upon low hours run.	
Chlorine Dosing System	Install too high up the wall. Cannot access and maintain. Some joints are leaking chemical and crystalizing. No containment for chemical drum.	No further issues noted	< 5	Provide drum containment and plan for replacement.
Pipework and Valves	Most of the welded joints on the pipework are rusting / corroding.	No further issues noted	10+	City to monitor corrosion.
Associated Building /Structure	No issues noted with structure. Eye station and wash down hose available. All hatches to reservoir raised or behind curbs	No issues noted	30+	
Associated Electrical & Controls	No issues noted with controls. Actuators operate with no issues, but several cards have failed and had to be replaced	No further issues noted	15+	
Associated Instrumentation	No issues with chlorine instruments (ATI)	No issues noted	15+	
Stand-by Power	Diesel powered, with separate tank in containment. ATS available. No issues observed	No issues noted	15+	

The current system installed within the pumphouse has the capability to meet the City's demand in its current configuration for the next 15 years and quite possibly longer. The introduction of a dedicated fill line from the WTP will require come change and modifications to be made at the West End Reservoir. Therefore, in alignment with the previously made recommendations, the City should undertake an assessment to determine when the dedicated fill line will need to be installed by, based upon the projected demands within the system. In addition, the concept for modifying the West End Reservoir and pumphouse should also be developed, such that the complete scope of the project can be established and budgeted for.

6.3.4 Non-pursuit of Dedicated Fill Line

On the basis that the dedication fill line to the West End Reservoir is not pursued, then the existing approach to supply water from both the WTP and the West End Reservoir would continue. On this basis (i.e. with no dedicated fill line) the projected hourly demands shown in Table 6.37 would still apply. These demands would have to be meet by a combined operation of the pumping equipment at both the WTP and the West End Reservoir. The availed pumping equipment at both location has been summarized below in Table 6.39 below.

Table 6.39: WTP and West End Reservoir Distribution Pumping Equipment

Location	Pump Tag	Drive	Pump Capacity	Facility Capacity (m^3/hr)
WTP	PWP-101	Fixed Speed	688 m3/hr @ 61m (598 kPa)	1,938
WTP	PWP-102	Fixed Speed	688 m3/hr @ 61m (598 kPa)	
WTP	PWP-103	Variable Speed	562 m3/hr @ 59m (578 kPa)	
West End	DP-1	Variable Speed	416 m^3/hr @ 38m (375 kPa)	1,664
West End	DP-2	Variable Speed	416 m^3/hr @ 38m (375 kPa)	
West End	DP-3	Fixed Speed	416 m^3/hr @ 38m (375 kPa)	
West End	DP-4	Fixed Speed	416 m^3/hr @ 38m (375 kPa)	
Total				**3,602**

Based upon the values present within Table 6.37 and 6.38, the pumping equipment installed in both the WTP and the West End Reservoir have the ability to work in conjunction to exceed the predicted hourly demand for 2035 (i.e. 2,021 m^3/hr). However as noted previously there is a restriction within the distribution system that restricts the flow of water from the WTP into the distribution system that must be resolved such that the carrying capacity of the distribution system can be fully used.

Looking at the historical peak hourly volumes shown in Table 6.32 to 6.35, the maximum flow leaving the WTP is approximately 954 m^3/hr. On the basis that this is the maximum flow of water that can be discharged from the WTP at this time due to the restriction in the distribution system, the remaining 1,067 m^3/hr would have to be address through the West End Reservoir. At a discharge pressure of 375 kPa, three of the four currently installed pumps at the West End Reservoir would need to run to meet this demand.

6.3.5 Distribution System

Within the Distribution Water Master Plan completed by ISL in 2016, the details and future requirements of the distribution system to meet future demands were capture and reported upon. The basis for the Distribution Master Plan was established from the City's GIS database of pipe data which was provided as "shapefiles". Figures 6.7 to 6.9 summarize the key water pipe data including year of construction (age), pipe material and pipe diameter. From this information, it can be seen that much of the distribution system was constructed in the mid to late 1970's and the 2000's, which were periods of rapid development for the City. As a consequence, much of the system consists of asbestos cement and PVC pipes. Less than 7% of the system has cast iron pipes, however these are clustered in the older downtown zone.

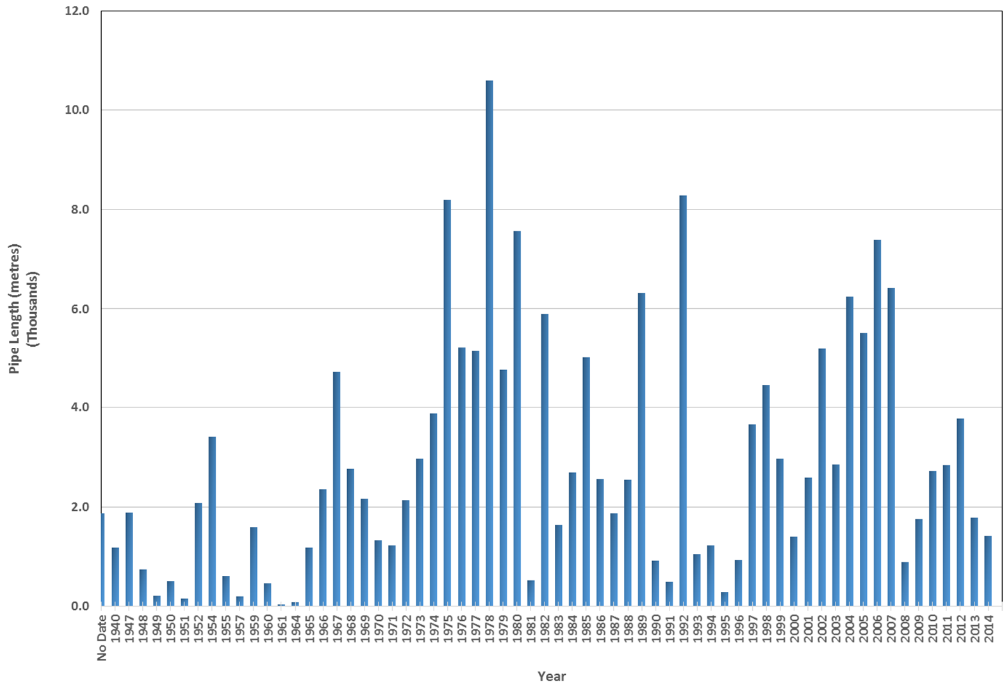


Figure 6.8: Length of Distribution System Pipe Construction by Year from City's GIS (Dated Feb.13, 2015)

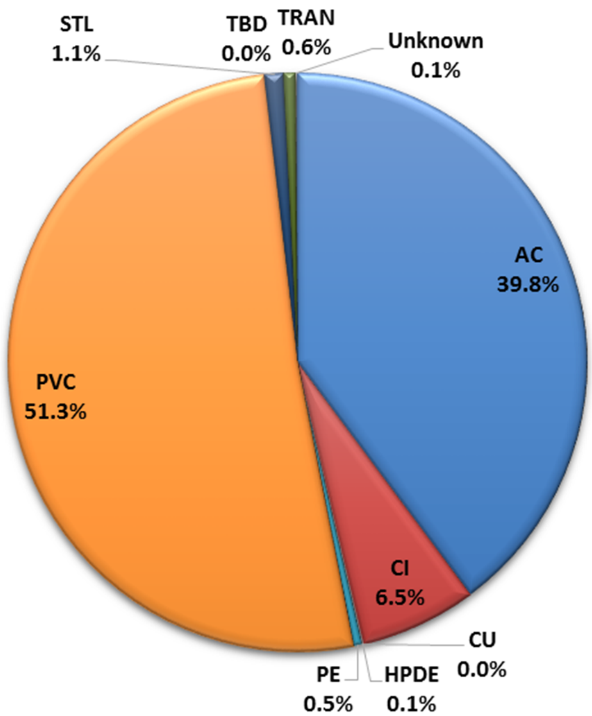


Figure 6.9: Breakdown of Distribution System Pipe Materials from City's GIS (Dated Feb. 13, 2015)

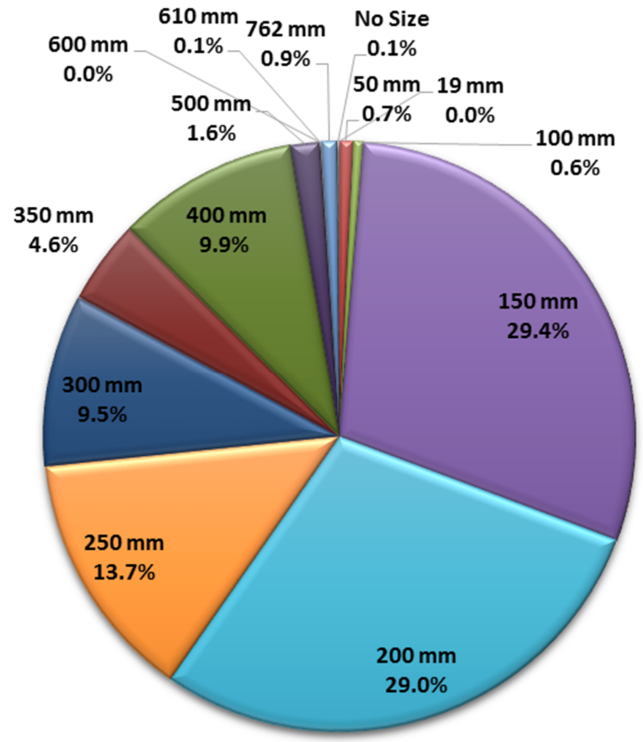


Figure 6.10: Breakdown of Distribution System Pipe Sizes from City's GIS (Dated Feb. 13, 2015)

Modeling of the existing and future distribution system using WaterCAD identified that the distribution system was capable of meeting the residential and commercial demands from 2018 through to 2035. However, with regards to meeting the agreed fire flow requirements in this period, some further upgrades were required. These upgrades have been grouped based upon location, prioritized and included within Appendix A and Appendix B of this report.

7.0 Financial Analysis

This section of the WSA will focus on the annual operating and maintenance costs of the City's Waterworks system, as well as developing a capital plan for the major waterworks components that are expected to be upgraded within the next 20 years (2035). The included costs are based upon historical costs provided by the City or from similar projects completed by ISL. All the cost estimates included within this section are in 2015 dollars and are conceptual, unless otherwise stated.

7.1 Annual Operation and Maintenance Costs

Upon requesting information from the City with regards to their costs for operating and maintaining the waterworks system in previous years, ISL was provided with the budgets that were established for 2014, 2015 and 2016, which were broken down into River, Husky, Water Treatment Plant, West End Reservoir and Distribution System. In reviewing this information with the City, it was confirmed that the actual total costs for 2014 and 2015 were very close to the assigned budget, something that has been repeated regularly, providing no major unforeseen events occur.

Tables 7.1 to 7.3 below provides a breakdown of the assigned budgets for 2014, 2015 and 2016. As can be observed within these tables, the salary component of the budget for each year accounts for a significant portion of the budget. This is followed by the utilities portion and then maintenance & inspections.

Table 7.1: Operation and Maintenance Budget for 2014

	River	Husky	WTP	West End	Distribution	Line Totals
Site Maintenance	$7,500	$530	$3,950	$1,200	$0	$13,180
Utilities	$781,100	$157,600	$317,800	$64,050	$11,000	$1,331,550
Monitoring	$0	$0	$0	$0	$0	$0
Laboratory & Sampling	$0	$0	$14,850	$10,250	$0	$25,100
Chemicals	$0	$0	$165,000	$5,550	$0	$170,550
Research and Office Work	$0	$0	$5,500	$0	$0	$5,500
Maintenance & Inspections	$52,400	$20,800	$37,200	$25,700	$1,079,850	$1,215,950
Building Upkeep & Janitorial	$37,000	$0	$65,000	$2,450	$0	$104,450
Training	$0	$0	$10,250	$0	$8,000	$18,250
Insurance & Claims	$8,400	$1,500	$16,550	$14,200	$20,500	$61,150
Salaries	$39,955	$83,166	$336,524	$85,314	$557,600	$1,102,560
Staff Equipment (Vehicles, Tools, Safety etc.)	$8,059	$5,173	$13,649	$5,173	$136,600	$168,654
Total Budget (Husky and City)	**$934,415**	**$268,769**	**$986,272**	**$213,887**	**$1,813,550**	**$4,216,894**
Potable Water Supply Budget (City Only)	**$498,387**	**$0**	**$986,272**	**$213,887**	**$1,813,550**	**$3,512,097**

Table 7.2: Operation and Maintenance Budget for 2015

	River	Husky	WTP	West End	Distribution	Line Totals
Site Maintenance	$7,500	$800	$6,450	$2,500	$0	$17,250
Utilities	$727,500	$167,000	$289,500	$48,950	$10,700	$1,243,650
Monitoring	$0	$0	$0	$0	$0	$0
Laboratory & Sampling	$0	$0	$15,300	$10,000	$0	$25,300
Chemicals	$0	$0	$179,215	$7,100	$0	$186,315
Research and Office Work	$0	$0	$7,250	$0	$2,500	$9,750
Maintenance & Inspections	$72,150	$20,050	$45,550	$29,465	$665,650	$832,865
Building Upkeep & Janitorial	$12,250	$0	$46,000	$2,700	$0	$60,950
Training	$0	$0	$11,000	$0	$7,000	$18,000
Insurance & Claims	$11,475	$1,925	$22,657	$19,000	$5,500	$60,557
Salaries	$38,931	$80,060	$540,237	$88,670	$749,700	$1,497,599
Staff Equipment (Vehicles, Tools, Safety etc.)	$8,550	$5,700	$31,001	$5,173	$139,701	$190,125
Total Budget (Husky and City)	**$878,357**	**$275,535**	**$1,194,160**	**$213,558**	**$1,580,751**	**$4,142,361**
Potable Water Supply Budget (City Only)	**$476,127**	**$0**	**$1,194,160**	**$213,558**	**$1,580,751**	**$3,464,596**

Table 7.3: Operation and Maintenance Budget for 2016

	River	Husky	WTP	West End	Distribution	Line Totals
Site Maintenance	$8,250	$800	$6,950	$2,000	$0	$18,000
Utilities	$692,250	$173,000	$295,000	$54,300	$11,000	$1,225,550
Monitoring	$200	$0	$0	$0	$0	$200
Laboratory & Sampling	$0	$0	$18,500	$10,000	$0	$28,500
Chemicals	$0	$0	$186,500	$6,500	$0	$193,000
Research and Office Work	$0	$0	$7,250	$0	$5,500	$12,750
Maintenance & Inspections	$82,900	$19,450	$46,000	$23,400	$814,500	$986,250
Building Upkeep & Janitorial	$12,250	$0	$37,000	$2,700	$0	$51,950
Training	$0	$0	$13,000	$0	$7,000	$20,000
Insurance & Claims	$11,467	$1,925	$22,657	$19,000	$12,134	$67,183
Salaries	$44,298	$96,317	$586,371	$91,761	$715,018	$1,533,765
Staff Equipment (Vehicles, Tools, Safety etc.)	$7,961	$5,307	$40,601	$7,650	$144,209	$205,728
Total Budget (Husky and City)	**$859,576**	**$296,799**	**$1,259,829**	**$217,312**	**$1,709,361**	**$4,342,877**

The agreement between the City and Husky is such that Husky covers the costs paid by the City to supply them with raw water. Where these costs are shared, i.e. the River Intake, Husky pays for the percentage of the costs based upon the percentage of raw water supplied to them. Using the data provided, ISL has determined that in 2014 and 2015 respectively, 54% and 55% of the raw water withdrawn from the river was supplied to Husky.

The impact of this shared costs has been illustrated within Tables 7.1 and 7.2 by providing totals for the whole water supply budget, i.e. Husky and the City, and the budget for just supplying potable water to the City of Lloydminster. By using the potable water supply budget, Table 7.4 shows the production cost of water based upon the recorded volume of treated water supplied to the distribution system and the allocated budgets in 2014 and 2015.

Table 7.4: 2014 and 2015 Water Production Rate

Year	Annual Budget	Annual Treated Water Volume (m^3)	Cost of Water ($ per m^3)
2014	$3,603,052	3,993,982	$0.88
2015	$3,541,556	3,976,611	$0.87

7.2 Non-Routine Equipment Overhaul and Replacement Plan

Within Section 6, each major component of the waterworks system was assessed and assigned a service life to identify the point in time at which it needs to be overhauled or replaced. Using this basis, Tables 7.3 to

7.5 below illustrate the estimated overhaul or replacement costs for non-routine maintenance components based upon five year increments up to 2030. The values below are provided on the basis that the installation work will be completed by the City Staff unless otherwise stated. In addition, this plan does not include costs for replacement of valves, instrument, dosing points and sections of pipework that require replacement on an ad-hoc basis over time.

By splitting each group into five years blocks some flexibility is provided to the replacement plan with regards to budgeting when each task is to be completed. Once the City decides when each component will be overhauled or replaced within the five-year cycle, the appropriate value of the work will have to be added to the individual annual budgets.

Table 7.5: Less Than 5 Year (2016 to 2020) Non-Routine Equipment Overhaul / Replacement Plan

Equipment Description	Basis for Inclusion	Actions	Estimated Cost
River Intake Low Lift Pumps & Desilting Pond			
Low Lift Pump LLP-102	VFD motor installed in January 2009. Pump overhauled in May 2002. LLP-101 overhauled in 2010Overhaul needed to maintain reliability.	Plan for pump overhaul in less than 5 years.	$25,000
Lime Storage and Dosing			
Lime Feeder / Slurry System	Replacement parts for the clutch are no longer available.	New feeder / slurry system is required as existing system cannot be maintained. Installation by Contractor.	$150,000
Lime Dosing Pump P-112	High wear item.	Boxed spare required for replacement when required).	$5,000
Carbon Storage and Dosing			
Carbon Make-up Tank, ACDT-101 & MX-104	Replacement parts for the clutch are no longer available.	New makeup system required as existing system cannot be maintained Installation by Contractor.	$100,000
West End Reservoir Pumphouse			
Chlorine Dosing System	Installed too high up the wall. Cannot access and maintain. Some joints are leaking chemical and crystalizing. No containment for drum.	Provide drum containment. Replace / modify dosing skid.	$30,000
Total			**$310,000**

Table 7.6: 6 to 10 Year (2021 to 2025) Non-Routine Equipment Overhaul / Replacement Plan

Equipment Description	Basis for Inclusion	Actions	Estimated Cost
River Intake Low Lift Pumps & Desilting Pond			
Low Lift Pump LLP-101	Pump overhauled in June 2010. New 150 Hp VFD motors install in 2009. Overhaul needed to maintain reliability.	Plan for pump and motor overhaul in 10 years.	$25,000
River Intake High Lift Pumps			
High Lift Pump HLP-101	Pump and motor overhauled in June 2009. Raw Water demands will result in loss of standby capability	Replace smaller pump with larger unit such that all three pumps match	$350,000
Media Filtration			
Filter Media	Filter media effectiveness is lost over time and should be replaced to maintain performance.	Plan to replace all media in 5 to 10 years. Installation by supplier.	$150,000
Backwash Waste Pump BWP-101	Conveyed Material results in high wear. Pumps were part of the original installation in 1984	Continue to monitor performance and repair / replace as necessary	$30,000
Backwash Waste Pump BWP-102	Conveyed Material results in high wear. Pumps were part of the original installation in 1984	Continue to monitor performance and repair / replace as necessary	$30,000
Alum Dosing			
Alum Dosing Pump P-109	Pumps installed as part of original construction in 1984. Pumps can be rebuilt provided spares continue to be available.	Allow for new pumps in less than 10 years.	$9,000
Alum Dosing Pump P-110	Pumps installed as part of original construction in 1984. Pumps can be rebuilt provided spares continue to be available.	Allow for new pumps in less than 10 years.	$9,000
Powered Activated Carbon Dosing			
Carbon Dosing Pump P-117	Conveyed material causes high wear. Pump located in high dust content conditions	Continue to monitor performance and repair / replace as necessary	$5,000
Carbon Dosing Pump P-118	Conveyed material causes high wear. Pump located in high dust content conditions	Continue to monitor performance and repair / replace as necessary	$5,000
Polymer Dosing (Flocculation)			
Polyelectrolyte Dosing Pump, P-120	Pumps installed as part of original construction in 1984. Pumps can be rebuilt provided spares continue to be available.	Allow for new pumps in less than 10 years.	$9,000

Equipment Description	Basis for Inclusion	Actions	Estimated Cost
Polyelectrolyte Dosing Pump, P-121	Pumps installed as part of original construction in 1984. Pumps can be rebuilt provided spares continue to be available.	Allow for new pumps in less than 10 years.	$9,000
West End Reservoir Pumphouse			
Distribution Pump 1	Pump installed in 2006. Hours run is low. Overhaul needed to maintain reliability	If dedicated fill line not in place, plan for pump and motor overhaul in 10 years.	No allowance made as dedicated fill line should be in place.
Total			$631,000

Table 7.7: 11 to 15 (2026 to 2030) Year Non-Routine Equipment Overhaul / Replacement Plan

Equipment Description	Basis for Inclusion	Actions	Estimated Cost
River Intake High Lift Pumps			
High Lift Pump HLP-102	Pump and motor overhauled in 2011. Overhaul required to maintain reliability	Plan for pump and motor overhaul in 15 years.	$100,000
River Intake Low Lift Pumps and Desilting Pond			
Desilting Pond	Previous effort was required to maintain / upkeep the integrity of the desilting pond banks.	Continue to monitor and plan to undertake significant bank maintenance in 10 to 15 years	$30,000
WTP Raw Water Pumps			
Jockey Pump, JSP – 101	Pumps installed as part of original construction in 1981. Overhaul required to maintain reliability	Plan for pump and motor overhaul in 10 to 15 years, as this is a low use pump	$10,000
WTP Raw Water Pump, RWP- 101	Pumps installed as part of original construction in 1984. Overhaul required to maintain reliability.	Plan for pump and motor overhaul in 10 to 15 years, as this is a low use pump	$30,000
WTP Raw Water Pump. RWP-103	Pumps installed as part of original construction in 1984. Overhaul required to maintain reliability.	Plan for pump and motor overhaul in 10 to 15 years, as this is a low use pump.	$30,000
Solids Contact Clarifier			
Settling Tubes	Tubes and Clarifier Internal part of original construction in 1984.	Plan for internals replacement in 10 to 15 years. Installation by Contractor.	$150,000
Sludge Pump, P-123	Overhaul required to maintain reliability.	Plan for pump and motor overhaul in 10 to 15 years.	$10,000

Equipment Description	Basis for Inclusion	Actions	Estimated Cost
Sludge Pump, P-124	Overhaul required to maintain reliability.	Plan for pump and motor overhaul in 10 to 15 years.	$10,000
Clarifier Basin	Evidence of seepage from clarifier on exterior surface of basin Calcification and paint peeling on outside walls.	Plan for existing clarifier refurbishment as part of WTP upgrade in 2026	$0
Polymer Dosing (Flocculant)			
Make-up Unit	Installed as part of the original installation in 1984. Parts may no longer be available.	Plans for replacement unit.	$30,000
Filter Aid Dosing			
Filter Aid Dosing Pump P-109	Replacement required to maintain reliability.	Monitor and plan to replace in 10 to 15 years.	$3,000
West End Reservoir Pumphouse			
Pipework and Valves	Corrosion on pipework welds is very visible and requires further monitoring	If dedicated fill line not in place, plan for pipework replacement in 10 to 15 years. Installation by Contractor	No allowance made as dedicated fill line should be in place, which will result in pumphouse upgrade
Total			**$403,000**

7.3 Capital Cost Estimates and Plan

In addition to the budgets that are required for normal operation / maintenance, and replacement of key components, the implication and costs for future expansions and engineering work must also be accounted for. Within the previous sections requirements for capital projects and engineering work were identified. Tables 7.8 to 7.11 below summarize these items and identified an associate cost and timing for them.

Within the Distribution Master Plan, a series of upgrades and improvements to the existing distribution system were prioritized and recommended for implementation. These have been included within Appendix A of this report for reference. The upgrades and improvements to the existing distribution system (totaling $35,310,000) have been split equally across the first two five-year capital planning cycles and added to the capital projects budget below.

Table 7.8: Less Than 5 Years (2020) Years Capital Project Cost Estimated and Plan

Project Title	Project Basis	Actions	Estimated Cost
River Intake Wing Dam	Both River Intake Low Lift Pumps are not operated together as their combined flow results in sand / sediment being pulled into the intake, increasing equipment wear and the risk of blockages / plugging.	Design of a Wing Dam has been completed. Update design, tender and complete construction	$2,000,000
Low Lift Pump Capacity	Off-set application of second low lift pumps which pulls sand and sediment into the River Intake.	City to use its understanding and experience to maximising retention and buffering capacity of desilting pond and raw water reservoir to shave off peaks in raw water demand.	$0 - Internal Exercise
Raw Water Reservoir	The purpose of the raw water reservoir and the trigger for its expansion is undefined. As such the raw water reservoir is not included in any future planning exercises.	City to establish the function of the raw water reservoir and its design basis (i.e. how many days of storage) to allow inclusion in planning model	$0 - Internal Exercise
Clarifier and Filter Treatment Capacity	Future expansion timing has been established on the basis of the supplier providing rates and loadings for two separate process stages. The rates and loadings for the two process stages working together needs to be established, to understand the real timing of the upgrade and the risks involved.	City and Suppliers to undertake testing of the solids contact clarifier and media filtration stage together to determined site specific loading rates under different conditions, whilst meeting the requirements of the Permit to Operate	$50,000
WTP Expansion Concept Design	This WSA has established an expansion is required by 2025 based upon treatment capacity and treated water demands. This will required an expansion of a combination of the clarification, filtration and disinfection stages	Engage consultant to establish design basis and establish comprehensive concept design to expand the City's WTP	$300,000
West End Reservoir Expansion (Stage 2)	To allow the demolition of the 1974 reservoir, additional storage capacity is required.	Engage Consultant to supply engineering services, then tender and construct	$6,309,000
Demolition of 1974 West End Reservoir	Assessment completed in 2010, assessed the life of the 1974 reservoir as 10 years. Remedial work can be completed however, this reservoir will need to be removed for future expansion, once Stage 2 of the expansion is completed	Engage Consultant to supply engineering services, then tender and construct.	$600,000

Project Title	Project Basis	Actions	Estimated Cost
West End Reservoir Fill Line Implementation	The implementation of the dedicated fill line to the West End Reservoir has been included in planning documents since 2006. The current Master Plan was based upon the application of the fill line. The date by which this must be installed in unknown.	Engage Consultant to identify the year by which the dedicated fill line to the West End Reservoir must be implemented	$50,000
West End Reservoir Fill Line and Pumphouse Upgrade	Design, Tender and Construction of West End Reservoir Fill Line and Pumphouse upgrade, such that WTP supplies the West End Reservoir and water is the distributed from the West End Reservoir only	Engage Consultant to supply engineering services, then tender and construct	$17,140,000
Existing Distribution System Upgrades (Part 1)	Improvements and upgrades to the Water Distribution System to improve fire flow within the network. Refer to Appendix A for details	Engage Consultant to supply engineering services, then tender and construct	$17,655,000
Future Distribution System (Network) Upgrade (Part 1)	Additional Capacity required to ensure system pressures at peak hour in northwest indusial area, and fire flows requirements. Refer to Appendix B for details	Engage Consultant to supply engineering services, then tender and construct	$2,020,000
Less Than 5 Years (2015 to 2020) Total			**$46,124,000**

Table 7.9: 5 to 10 Years (2020 to 2025) Capital Project Cost Estimated and Plan

Project Title	Project Basis	Actions	Estimated Cost
WTP Expansion Preliminary and Detailed Design	Building upon the concept design, develop the preliminary and detailed design for the WTP expansion.	Engage consultant to complete design of WTP expansion	$1,300,000
WTP Expansion Tender and Construction	Construction of WTP Expansion	Tender and construction of WTP expansion (Value based upon clarifier and media filtration)	$19,000,000
Existing Distribution System Upgrades (Part 2)	Improvements and upgrades to the Water Distribution System to improve fire flow within the network. Refer to Appendix A for details.	Engage Consultant to supply engineering services, then tender and construct	$17,655,000
Future Distribution System (Network) Upgrade (Part 2)	Additional Capacity required to minimize head loss and ensure system pressures are achieved, plus twinning of West End Reservoir Outlet. Refer to Appendix B for details	Engage Consultant to supply engineering services, then tender and construct	$1,610,000
5 to 10 Years (2020 to 2025) Total			**$39,565,000**

Table 7.10: 10 to 15 Years (2025 to 2030) Capital Project Cost Estimated and Plan

Project Title	Project Basis	Actions	Estimated Cost
WTP Electrical Upgrade	Electrical Equipment installed as part of original construction. Condition and availability of spares might become and issue	Allow for electrical upgrade within WTP	$1,000,000
West End Reservoir Expansion (Stage 3)	Additional storage and pumping capacity required to meet peak hour demands	Engage Consultant to supply engineering services, then tender and construct	$8,750,000
10 to 15 Year (2025 to 2030) Total			**$9,750,000**

Table 7.11: 15 to 20 Years (2030 to 2035) Capital Project Cost Estimated and Plan

Project Title	Project Basis	Actions	Estimated Cost
West End Reservoir Expansion (New Location) & Pump House Upgrades. From Distribution Water Master Plan	Additional storage and pumping capacity required to meet peak hour demands	Engage Consultant to supply engineering services, then tender and construct	$13,500,000
Future Distribution System (Network) Upgrades (Part 3)	Additional Capacity required to minimize head loss and ensure system pressures are achieved. Refer to Appendix B for details	Engage Consultant to supply engineering services, then tender and construct	$2,730,000
15 to 20 Years (2030 to 2035) Total			**$16,230,000**

7.4 Recommended Water Rates

A requirement of the WSA is to estimate the water rates necessary to operate and maintain the water supply system. To achieve this objective, the operational and capital expenditures were grouped together in five year blocks and a 2% annual inflation rate was applied to the estimates above (which are in 2015 dollars) to establish both the future capital budgets. The costs for producing water in each year was established by using the future treated water projections applied to the WSA and the 2015 historical cost of $0.89 per m^3 from Table 7.4, a future cost per cubic meter was established. The combination of these two aspects was subsequently used to provide an estimate of the average water rates that would be required for that five-year cycle to operate and maintain the water supply system, and complete the identified capital projects.

Tables 7.12 to 7.15 below provides a summary of each five-year cycle up to 2035. In addition to each table, the required average "water rates" for the specific five-year cycle has been provided using the following basis.

- 10% of the treated water produced by the WTP is non-revenue water (i.e. water upon which a revenue cannot be claim due to operational activities, flushing etc.)
- Based upon historical data from 2014 and 2015, the operation and maintenance of the wastewater system is an additional 52.5% of the costs for the operation and maintenance of the water supply system. These costs for the wastewater system are recovered by the City through the water rates.

Table 7.12: Estimated Water Rates required for 2016 to 2020

Year	Annual Volume (m3)	Water Production Cost ($ per m3)	Non-routine Operational & Maintenance Costs	Equipment Replacement Budget	Capital Projects Budget
Value in 2015 Dollars				**$310,000**	**$46,124,000**
2016	3,975,657	0.91	$3,527,998	$62,000	$9,224,800
2017	4,059,146	0.93	$3,674,128	$63,240	$9,409,296
2018	4,144,388	0.94	$3,826,310	$64,505	$9,597,482
2019	4,231,420	0.96	$3,984,796	$65,795	$9,789,432
2020	4,320,280	0.98	$4,149,846	$67,111	$9,985,220
Total	**20,730,892**		**$19,163,079**	**$322,650**	**$48,006,230**

For the period 2016 to 2020:

- The total estimated expenditure for this five-year cycle is $67,491,959 (i.e. $13,498,392 per year)
- Based upon the projected increase in water demands for this five-year cycle, the water supply portion of the assigned water rate is required to be a minimum of $3.62 per m^3 to maintain and operate the water supply system only and implement identified water supply capital program,
- Based upon the operation and maintenance of the wastewater system costing a further 52.5% of the water supply operating and maintenance budget, the average water rate for the five-year cycle required to operate and maintain the water and wastewater systems, and undertake the identified capital program for the water supply system would need to be $4.16 per m^3

Table 7.13: Estimated Water Rates required for 2021 to 2025

Year	Annual Volume (m3)	Water Production Cost ($ per m3)	Non-routine Operational & Maintenance Costs	Equipment Replacement Budget	Capital Projects Budget
Value in 2015 Dollars				**$631,000**	**$39,565,000**
2021	4,411,006	1.00	$4,321,733	$139,335	$8,736,591
2022	4,503,637	1.02	$4,500,739	$142,122	$8,911,323
2023	4,598,214	1.04	$4,687,160	$144,964	$9,089,550
2024	4,694,776	1.06	$4,881,302	$147,863	$9,271,341
2025	4,793,366	1.08	$5,083,486	$150,821	$9,456,767
Total	**23,000,999**		**$23,474,420**	**$725,105**	**$45,465,572**

For the period 2021 to 2025:

- The total estimated expenditure for this five-year cycle is $69,665,097 (i.e. $13,933,019 per year)
- Based upon the projected increase in water demands for this five-year cycle, the water supply portion of the assigned water rate is required to be a minimum of $3.37 per m^3 to maintain and operate the water supply system only and implement identified water supply capital program,
- Based upon the operation and maintenance of the wastewater system costing a further 52.5% of the water supply operating and maintenance budget, the average water rate for the five-year cycle required to operate and maintain the water and wastewater systems, and undertake the identified capital program for the water supply system would need to be $3.96 per m^3

Table 7.14: Estimated Water Rates required for 2026 to 2030

Year	Annual Volume (m3)	Water Production Cost ($ per m3)	Non-routine Operational & Maintenance Costs	Equipment Replacement Budget	Capital Projects Budget
Value in 2015 Dollars				**$403,000**	**$9,750,000**
2026	4,894,027	1.11	$5,294,043	$98,251	$2,377,039
2027	4,996,802	1.13	$5,513,323	$100,216	$2,424,580
2028	5,101,734	1.15	$5,741,685	$102,220	$2,473,071
2029	5,208,871	1.17	$5,979,505	$104,265	$2,522,533
2030	5,318,257	1.20	$6,227,176	$106,350	$2,572,984
Total	**25,519,691**		**$28,755,732**	**$511,302**	**$12,370,207**

For the period 2026 to 2030:

- The total estimated expenditure for this five-year cycle is $41,637,241 (i.e. $8,327,448 per year)
- Based upon the projected increase in water demands for this five-year cycle, the water supply portion of the assigned water rate is required to be a minimum of $1.81 per m^3 to maintain and operate the water supply system only and implement identified water supply capital program,
- Based upon the operation and maintenance of the wastewater system costing a further 52.5% of the water supply operating and maintenance budget, the average water rate for the five-year cycle required to operate and maintain the water and wastewater systems, and undertake the identified capital program for the water supply system would need to be $2.47 per m^3

For the 2031 to 2035 period it was not possible to estimate the budget to overhaul / replace non-routine equipment. After reviewing the list of items requiring overhaul or replacement within the next five years, it was determined that these items would be due to further overhaul or replacement again (as they are high wear items). As such the overhaul / replacement budget for non-routine equipment for the next five years was inflated and applied to 2031 to 2035.

Table 7.15: Estimated Water Rates required for 2031 to 2035

Year	Annual Volume (m3)	Water Production Cost ($ per m3)	Non-routine Operational & Maintenance Costs	Equipment Replacement Budget	Capital Projects Budget
Value in 2015 Dollars				**$310,000**	**$16,230,000**
2031	5,429,941	1.22	$6,485,106	$81,808	$4,283,028
2032	5,543,969	1.25	$6,753,719	$83,444	$4,368,689
2033	5,660,393	1.27	$7,033,458	$85,113	$4,456,062
2034	5,779,261	1.30	$7,324,784	$86,815	$4,545,184
2035	5,900,625	1.32	$7,628,176	$88,551	$4,636,087
Total	**28,314,189**		**$35,225,243**	**$425,730**	**$22,289,050**

For the period 2026 to 2030:

- The total estimated expenditure for this five-year cycle is $57,940,024 (i.e. $11,588,005 per year)

- Based upon the projected increase in water demands for this five-year cycle, the water supply portion of the assigned water rate is required to be a minimum of $2.27 per m^3 to maintain and operate the water supply system only and implement identified water supply capital program,
- Based upon the operation and maintenance of the wastewater system costing a further 52.5% of the water supply operating and maintenance budget, the average water rate for the five-year cycle required to operate and maintain the water and wastewater systems, and undertake the identified capital program for the water supply system would need to be $3.00 per m^3

In 2016 the water rates for the City of Lloydminster were established as a two tier system. Based upon a two monthly billing period

- Those properties that use less than $60m^3$ in the two-month period are charge $3.40 per m^3, and
- Those properties that use more than $60m^3$ in the two-month period are charge $3.55 per m^3

Based upon historical data developed by the City, 36% of the revenue water generated from the water rate each year is charged at the lower rate. In using this information, a combined or blended water rate of $3.50 per m^3 can be established for this assessment. Comparing the combined water rate of $3.50 per m^3 with the requirements identified above the following observations can be made.

- For the 2016 to 2020 period, the current blended water rate is insufficient to meet the projected expenditure for the identified water and wastewater budgets, which requires a water rate of $4.16 per m^3
- For the 2021 to the 2025 period, the current blended water rate provides sufficient revenue to meet the projected expenditure of the identified water supply budgets only, however it Is insufficient to meet the identified water and wastewater budgets, which requires a water rate of $3.96 per m^3
- For the 2026 to 2035 period the current blended water rate of $3.50 per m^3 is capable to providing sufficient revenue to meet the identified budget requirements for both the water supply system and the operation of the wastewater system.

8.0 Conclusions and Recommendations

8.1 Conclusions

The conclusion of this review and assessment is that the waterworks system operated by City of Lloydminster has historically provide treated water that meets the treated water demands of its residents and businesses, at a quality that exceeds the conditions stated within their Permit to Operate. Initially built in 1984, the equipment and buildings included within the waterworks system are maintained to a high standard, and as such there are no items that required urgent attention or replacement.

With regards to the upgrade or expansion of the water treatment plant, the established treatment capacity remains unchanged from the 2010 WSA at 20,125 m^3/d, due to the limitation of the solids contact clarifier and media filters. Based upon the future projections developed within Section 5.0 of this report, the upgrade is not required until 2026.

With regards to system capacity, the mitigation of sand and sediment build up within the river intake is of immediate concern, as the operations staff avoid running two low lift pumps at the River Intake. The operation of two low lift pumps together is known to increase the amount of material that is drawn into the intake, which in turn increase the wear on the installed equipment and the risk of fouling or plugging the intake structure.

8.2 Impact of Husky Raw Water Supply Equipment

The supply of river water to Husky has been reviewed and included within this assessment. While there is no financial burden on the City to supply river water to Husky, the amount of river water supplied each year accounts of approximately 55% of the volume pumped from the North Saskatchewan River each year.

This amount of water accounts for a significant percentage of the pumping and treatment capacity of the installed infrastructure used to move water from the River to the Husky Raw Water Pumphouse. It is understood that the water supplied to Husky is subjected to treatment at their facility and used as cooling water, wash down water and boiler feed water. For some of these applications, it would be possible to use an alternative source of water, such as wastewater effluent.

From the Husky perspective moving to wastewater effluent would provide them with a stable and somewhat predictable water quality which would not be susceptible to seasonal variations, and at a likely reduced cost. From the City's perspective releasing Husky portion of the river water pumping and treatment capacity would:

- Adjust the raw water demands such that the operation of two river intake low lift pumps would not be required until after 2040. This would delay the urgent need to address sand and sediment build up in the intake,
- Increase the stand-by capacity at the River Intake, thus delaying future upgrades or expansions significantly,
- Reduce the operation and wear on equipment, thus extending the frequency of overhaul or replacement of equipment, and
- Reduce the amount of power used at the river intake, reducing the carbon foot print of the facility.

In addition to the above benefits, moving Husky to wastewater effluent will provide advantages to the wastewater system operated by the City, as it will:

- Reduce the costs of pumping wastewater effluent 29 km to the North Saskatchewan River, and
- Off set the need to address the hydraulic capacity limitation of the effluent forcemain, which is predicted to be required between 2030 and 2035, at a potentially significant cost to the City.

While moving Husky to an alternative water source is a possibility and has potential benefits, the recommendations that are made and updated in the following pages, are provided on the basis that river water will be supplied to Husky for the foreseeable future on the basis established within this report.

8.3 Previous Waterworks System Assessment Recommendations Update

Within the previous Sections of this report, the City of Lloydminster's existing waterworks has been assessed in terms of conditions and treatment capacity. In undertaking the assessment, ISL has reviewed the previous WSA reports and ascertained the status and the actions performed with regards to previous recommendations. Within Tables 8.1 to 8.3 (following this page) any previous outstanding recommendations have been summarized and their status updated. In addition, ISL has made further comments to either clarify or assist the City in moving the recommendation forward.

Table 8.1: Previous WSA Recommendations for Raw Water System

Component	Comments/Recommendations provided by past WSAs	Actions Undertaken	Current Status	ISL Comments/Recommendations
Presence of Pathogens in Raw Water Supply	Due to the risk of delivering pathogens to raw water customers, the 2005 and 2010 WSA recommended increasing testing for pathogenic microorganisms in raw water.	Bi-monthly testing for *Giardia* and *Cryptosporidium* and Bi-weekly testing for Total Coliform and E. Coli in the raw water reservoir was implemented by the City in 2010.	The City continues to conduct routine sampling and analysis for monitoring pathogenic levels in the raw water reservoir.	Continue sampling and monitoring for pathogens within the raw water supply system
Leak in Raw Water Vault	A leak in one of the raw water service connection valves was identified in the 2005 WSA.	Leak was repaired.	No further leaks have developed since repair.	No further actions required.
Safety and Security at River Intake Pumphouse	The 2005 WSA recommended improving the security measures at the river intake pumphouse.	Pumphouse is now protected by a fence and a locked gate. Warning signs have been posted on site with emergency contact information.	No further concerns have been flagged since implementation of security measures.	No further actions required.
Sediment Intrusion at River Intake and Pumphouse	As part of the strategy to minimize/eliminate sediment intrusion at the intake structure, the 2010 WSA recommended undertaking a detailed inspection and review of the intake structure, including the establishment of a design that will reduce/eliminate sand entrainment and accumulation at the intake.	As per the recommendation, an assessment of the intake was conducted by Stantec Inc. and a recommendation to add a wing dam near the intake structure to increase the velocity of flow across the face of the structure was made. This study incorporated modelling and simulation of a wing dam structure at the existing intake structure and a design was developed.	No modifications to the intake structure has been made. The City has been routinely monitoring the sediment levels in the intake structure and flushing the sediment back into the river through the intake pipe, as required. An internal inspection of the structure conducted in 2011 using an inspection camera, showed that the intake structure does not demonstrate any signs of physical damage. ISL has been informed by the City that they do not run both low lift pumps at the same time as the resulting high velocities cause sand/sediment to be drawn into the intake structure. This increases the wear on the mechanical equipment within the intake, and increases the risk of a flow restriction occurring.	Continue the current measures and procedures in place to control the level of sediment in any around the intake. As a priority move forward with updating, tendering and constructing the wing dam, as the limitation of running one low lift pump will soon start to limit the supply of raw water to the system.
Increasing High Lift Pumping Capacity	The 2010 WSA recommended replacing one of the existing larger pumps (capacity of 20,000 m^3/day) pump with a larger pump (capacity of 30,000 m^3/day) to address increasing raw water demand projections.	No upgrades in pumping capacity have been undertaken as historical demands have been met using only two of the three pumps.	The operation one 20,000 m^3/d and one 10,000 m^3/d provides the City with sufficient capacity for current operations. The third pump (20,000 m^3/day) is used instead of the smaller high lift pump during emergencies and maximum day demands.	Revised projections developed with City indicated that upgrade of these pumps is not required for at least another 10 to 15 years (Refer to Table 6.5). City confirmed that if the high lift pumping capacity were to be increased, the smaller pump would be replaced first.
Sludge and Scale Accumulation in Raw Water Pipeline	The 2005 WSA identified sludge and scale build up in the raw water pipeline as a potential concern. It was recommended to perform regular swabbing/flushing of the raw water pipeline to prevent any build up. As such, the City obtained a quotation from a consultant to conduct routine flushing and swabbing of the raw water pipeline, which the City determined not to be cost effective.	Following the 2010 WSA, the pipeline was drained and a small area was inspected. Minimal sludge and scale build up was discovered in the inspected area by the City.	Inspection of the raw water pipeline occurs when the opportunity arises, and no issues have been identified with regard to maintaining the flowrate, pressure and water quality.	Continue with inspections along the length of the pipeline to ensure uniform condition throughout the pipeline, when the opportunity arises.
Raw Water Pipeline Condition	The 2010 WSA indicated buckled of a section of the raw water pipe on the east side of the road crossing as a major issue. A variety of options were provided to the City for eradicating/minimizing this issue. The previous WSA recommended leaving the pipe section as it is and inspect the interior of the pipe as no leaks has developed.	The affected section was excavated in 2010 to allow for an internal and external assessment. The pipe wall thickness at this location was determined to remain unchanged since installation, and therefore no deterioration of pipe wall thickness was identified at this location. The pipeline was also externally assessed by a third party to identify and investigate any corrosion issues on the pipeline. Only surface rust was discovered on the exterior surface of the pipe at different locations, which was addressed at the time of identification.	The Raw Water Pipeline is inspected internally (at some locations) and externally annually. With internal inspections, only certain locations along the length of the pipeline have been inspected as the cost of conducting a full internal analysis is significant. Annual inspections reveal no significant change to the condition of the pipeline.	Continue with annual visual inspections.

Component	Comments/Recommendations provided by past WSAs	Actions Undertaken	Current Status	ISL Comments/Recommendations
Rodent Activity in Raw Water Reservoir	The 2005 WSA identified rodent activity as a possible issue and a recommendation was made to implement a rodent control program to control this issue. As such a gopher control program was put into place. The 2010 WSA recommended that the City evaluate the effectiveness of this program and develop the program further, if required.	A further review was undertaken as recommended.	The City contracted a pest control company in 2016 to inject spray foam down gopher holes. The non-toxic foam acts as a respiratory irritant, blocking oxygen transfer in their lungs, thus suffocating them. During this WSA the city clarified that the historical concern was the presence of Muskrats within the raw water reservoir. This issued has been address for some time and is no longer an issue.	ISL noted an increase in E.coli concentrations across the raw water reservoir as part of our analysis. It was concluded that this is the result of bird activity around the reservoir in summer. Continue monitoring E.coli and gopher activity, and continue to work with the local conservation office to address the presence of birds.
Soil Movement at Meridian Bridge	The 2005 WSA recommended that the electronic monitoring system be re-instated to measure any pipe movement at Meridian Bridge. The 2010 WSA indicated that this issue was investigated by the City.	City investigated as recommended. Ministry of Transportation is using the monitoring system, and as such has been flagged as not an immediate concern.	No further actions required as investigation confirmed system was still in place and used by the Ministry of Transportation.	No further actions required.

Table 8.2: Previous Recommendations for Water Treatment System

Component	Comments/Recommendations provided by past WSAs	Actions Undertaken	Current Status	ISL Comments/Recommendations
Increasing WTP Throughput – WTP Operation	Both the 2005 and 2010 WSAs recommended increasing the WTP operating time to 24 hours to provide increased treatment capacity without any major infrastructure upgrades.	Recommendation of 2005 and 2010 WSA has not been implemented.	The WTP is meeting the required demands based upon the current regime of operating the WTP for part of the day at a well-established flowrate.	The City is aware that the WTP will have to be eventually operated on a 24-hour basis when the WTP is faced with higher demands. When required the operation of the WTP will be adjusted accordingly.
Increasing WTP Throughput – Treatment Capacity	The 2005 WSA recommended that the City's WTP be expanded in 2013 to meet the treated water demands. Using the clarifier's loading rate of 4.2 m/hr, the solids contact clarifier was identified the bottleneck for the City's Waterworks system. The 2010 WSA also identified this limitation and recommended that the City upgrade the WTP in the near future to address maximum demands.	Recommendation of 2005 and 2010 WSA has not been implemented.	Under the current operating conditions, the WTP has not yet reached its maximum capacity with regards to actual average day and maximum day demands. The WTP continues to operate for part of the day (about 16 hours per day) at a well-established flowrate. Revised projections included within this WSA forecast capacity upgrade in 2026.	Develop concept design in the next 5 years to expand the water treatment facility ahead of 2026. Consider options for • Optimization of media filtration to increase combined clarifier / filtration treatment capacity • Duplication of second solids contact clarifier and media filtration with additional disinfection capacity. • Application of membrane filtration in place of media filtration which will provide 3-log reduction in Cryptosporidium and Giardia, independently of the clarifier's availability.
Rainwater Runoff in Clarifier	The 2005 WSA indicated that a portion of the rainwater runoff from the roof is piped into the mixing zone of the clarifier. This was flagged by the 2005 WSA as long durations of heavy rainfall periods can impact treatment performance, specifically pH adjustment as Lime is added to the mixing zone of the clarifier and it is fed based on the raw water flow rates. The 2010 WSA commented that as clarifier performance has been optimized by modifying the mixing speed and no significant changes in the treated water quality have been observed, this is no longer a priority issue.	No actions were identified as part of the previous WSAs.	No modification to the operation of the clarifier has been made. The rainwater runoff is still directed to the mixing zone of the clarifier. Based on a review of historical data by ISL, no evidence of impact of rainwater on treatment performance is seen on the process.	Continue with their current mode of operation of the clarifier and management of rain water. The rain water is effectively being discharged to the front of the process.
Disposal of Filter Backwash Wastewater	The 2005 WSA recommended implementing a water quality monitoring program to determine the effects of the backwash wastewater on the natural stream and surrounding watershed. A downstream impact study was conducted by the City to measure the chlorine residual level within the stream downstream of the discharge. No significant effects of the wastewater on the stream were identified from the study.	Impact assessment was completed and no concerns of issues were raised.	The City continues to dispose backwash wastewater into the natural stream.	No further actions required.
Manual Loading of PAC	The 2005 WSA identified manual loading of powdered activated carbon into the carbon hopper as a potential hazard. The 2010 WSA indicated that the current mode of loading will be maintained until the plant undergoes an expansion, during which the PAC loading process will be modified.	No changes have been made.	The City continues to manually load powdered activated carbon into the hopper. City has been looking at upgrading powdered activated carbon system.	This WSA recommends upgrading the powdered activated carbon system in the next five years as spare parts are no longer available for the system.

Component	Comments/Recommendations provided by past WSAs	Actions Undertaken	Current Status	ISL Comments/Recommendations
Access to Chlorine Room	The 2005 WSA recommended upgrading the Chlorine room to meet the access and HVAC guidelines for chlorine gas use as per the Saskatchewan Ministry of Environment (SOME).	The 2010 WSA stated that the chlorine room's HVAC system has been upgraded to meet the SOME requirement.	Interior access to the existing chlorine storage room is through a vestibule. City staff have concerns to only external access to the Chlorine room, due to operational practicalities in winter (i.e. temperature ice, slips & trips).	No actions on access to be undertaken until the WTP undergoes a significant upgrade.
Upgrade to Disinfection System	The 2005 and 2010 WSA both recommended the incorporation of a UV disinfection system into the treatment process to accomplish a 3 log removal of Cryptosporidium under the conditions when the clarifier is, and is not in service.	Recommendation of 2005 and 2010 WSA has not been implemented.	Within EPB-501 (November 2015), Table 3.2 states that a 3.0-log reduction credit in Cryptosporidium can be recognized for conventional sedimentation / filtration (i.e. clarifier in service). As demonstrated within this WSA the City's WTP meets the turbidity requirements set out within the Permit to Operate, which act as a surrogate for the 3-log reduction above.	When the clarifier is out of service, direction filtration is practiced which is recognized in EPB-501, Table 3.2 as providing 2.5-log reduction in both Cryptosporidium and Giardia and a 1-log reduction in viruses. As demonstrated within this WSA, the current disinfection process is capable of providing the required 0.5-log reduction in Giardia and 3-log reduction in viruses, such that the requirements of SWSA are achieved. In the event the clarifier is out of service additional monitoring, timing and boil water order notices can be used to address risk presented by the remaining 0.5-log reduction requirement for Cryptosporidium. Within the future WTP upgrade the measures that can be included to mitigate this risk include application of: • A second solids contact clarifier (treated water can continue to be produced but at a reduced flowrate) • A UV disinfection stage to provide 0.5 log reduction in Cryptosporidium and Giardia, • Membrane filtration, which will provide 3-log reduction in Cryptosporidium and Giardia, independently of the clarifier's availability.

Table 8.3: Previous Recommendations for Distribution System Water System

Component	Comments/Recommendations provided by past WSAs	Actions Undertaken	Current Status	ISL Comments/Recommendations
Treated Water Reservoir Condition (West End Reservoir)	The 2005 WSA recommended conducting a complete internal and external inspection of the treated water reservoir in place since 1974. Inspection of this reservoir was completed in 2009, however the condition of the reservoir was not commented on in the 2010 WSA due to lack of information.	Recommendation of 2005 completed.	2009 inspection report noted the condition of the above ground West End Reservoir to be in poor to fair condition, with a remaining service life of 10 years in its assessed condition. The above ground reservoir still in service and no other internal inspections or work has been done to the reservoir since the 2009 inspection.	The condition of the 1974 above ground reservoir is a concern. ISL recommends that within the next five years the City add a further 9,850 m^3 to the existing below ground structure and then subsequently demolish the above ground reservoir. Once complete this would provide the City with 30,051 m^3 of storage at the West End Reservoir, which based upon the projections used within this WSA would be sufficient until 2031. In addition the demolishing of the reservoir would provide space for a further reservoir expansion.
Treated Water Reservoir Capacity	The 2010 WSA recommended that the City immediately design and construct an additional storage volume of 10,125 m^3 to their treated water storage system to comply with the SMOE EPB 201 guideline requiring a minimum storage capacity of twice the average daily demand for systems requiring fire protection.	Recommendation of 2010 WSA has not been implemented.	The City continued to operate with the same treated water volume at the West End Reservoir that was in place during the 2010 WSA. No reports have been made with regards to issues of a shortfall in the treated water storage volume.	Projections completed within this WSA, which are based upon historical information, has predicted that the current storage volume at the West End Reservoir is sufficient to meet SMOE requirements until approximately 2022 (Section 6.3.2). This prediction contradicts the assessment completed within the 2016 Distribution Water Master Plan, as the Distribution Master Plan uses land use and standard rates to developed water demands rather than historical information.
Restriction in Treated Water Distribution	The 2010 WSA identified that there is a restriction in the distribution system between the WTP and the West End reservoir that has reduced the treated water transfer rate from 26,400 to 21,600 m^3/day. As such, it was recommended to conduct a hydraulic modelling study to isolate and rectify the restriction, if possible.	Further investigations have been completed, but no specific modeling has been completed.	The City has been investigating this issue for several years and their investigation is still ongoing. As part of the recent Water Master Plan, the City investigated specific branches for restrictions within the distribution system. The City did not have any success in identifying the restriction in the system and will continue with their investigations in 2016.	City to continue with investigations and consider possibility of more intrusive investigations (i.e. camera investigations)

8.4 2015 Waterworks System Assessment Conclusions and Recommendations

This report has provided information and data on the historical performance of the City of Lloydminster waterworks system, and demonstrated that since 2010 the waterworks system has provided potable (or treated) water that meets the requirements of the City's Permit to Operate. In using this historical information, future treated water demands have been projected, which in turn have been used to anticipate the future performance of the waterworks system, and when upgrades or expansions will be required. The specifics of those upgrades have been provided within Sections 6 and 7.

Based upon the performance of the waterworks system and the projected future demands developed in Section 5 the following recommendations can be made with regards to each stage of the waterworks system:

- Raw Water Supply System
  - Continue with the overhaul and replacement of mechanical equipment on a regular basis as scheduled within Section 7, to ensure reliability.
  - As soon as it is reasonably practicable, install measures to prevent excessive sand and sediment from entering the intake structure on an ongoing basis (i.e. wing dam). The introduction of sand and sediment into the intake structure results in excessive wear of mechanical equipment and build up of material within the intake structure.
  - By 2020 use the knowledge and experience of the City staff to developed the operation of the raw water system (i.e. desilting pond, raw water reservoir etc.) to shave the peak raw water demands, thus minimising the use of the low lift pump.
  - Prior to 2020, review and establish the purpose of the raw water reservoir and identify the trigger for its expansion.
- Water Treatment Plant
  - Continue with the overhaul and replacement of mechanical equipment on a regular basis as schedule within Section 7, to ensure reliability.
  - Prior to 2020,
    - Assess the combined operation of the solids contact clarifier and the media filtration stages to refine the combined site specific treatment capacity of these stages, thus allowing the refinement of the WTP upgrade schedule.
    - Develop the concept design for the future expansion of the WTP addressing all of the needs identified within this report
  - Design and execute WTP expansion for completion before 2026, such that the expansion provides additional treatment capacity and fully address requirements for Cryptosporidium and Giardia reduction.
  - Plan for an electrical upgrade of the WTP between 2025 and 2030.
- Distribution System
  - Continue with the overhaul and replacement of mechanical equipment on a regular basis as schedule within Section 7, to ensure reliability.
  - Prior to 2020,
    - Provide additional storage at West End Reservoir and demolish 1974 above ground reservoir.
    - Design, tender and construct the dedicated fill line from the WTP to the West End Reservoir, which will also require some modifications to the pumphouse at the West End Reservoir.
    - Undertake 50% of the existing distribution system upgrades identified within the 2016 Distribution Master Plan.
    - Undertake the future distribution system (network) upgrades identified within the 2016 Distribution Master Plan.

- Between 2021 and 2025
  - Undertake remaining 50% of the existing distribution system upgrades identified within the 2016 Distribution Master Plan.
  - Undertake the future distribution system (network) upgrades identified within the 2016 Distribution Master Plan for this time period
- Review the expansion of the West End Reservoir between 2026 and 2030 to meet peak hour demands as recommended by the 2016 Distribution Water Master Plan.
- Review the implementation of a new West End Reservoir and pump house in a separate location between 2030 and 2035, as recommended by the 2016 Distribution Water Master Plan.

Whilst the list of recommendations above is long and the budget commitment to complete these actions is significant, it is essential that the overhaul and replacement of equipment continues. In assessing this list on the basis of urgency and the need for future planning, the top five recommendations that should be addressed are summarized below.

Priority 1 – Address the buildup of sand and sediment in and around the intake structure.

The movement of sand and sediment in response to the rate of withdrawal not only reduces the service life of the installed screen and low lift pumps, but also results in a real risk of significant deposition of material in the intake structure, thus reducing the rate at which water can be withdrawn from the river. At this time the operations staff take measures not to run two low lift pumps together, such that sand and sediment are not drawn into the intake. As demands for raw water climb, the operation of only one low lift pump will no longer be sufficient, which if left unaddressed will result in an increase in cost of equipment maintenance and risk of flow restrictions.

Priority 2 – Dedicated Fill Line to West End Reservoir Planning.

The implementation of a dedicated fill line to the West End Reservoir has been planned for a number of years. This report has identified that in addition to the fill line, some modifications to the pumphouse at the West End Reservoir are required. In order to plan the capital programmer for the next 10 years the City needs to understand the year by which the dedicated fill line is needed (as this is what the 2016 Distribution Master Plan was based upon) and the cost implications. With a number of other significant projects planned within the City a true understanding of the need and timing of this project is required.

Priority 3 – Water Treatment Plant Concept Design

The imminent expansion of the water treatment plant that has been recommended since the 2005 Waterworks System Assessment, has been explored and revised within this assessment to 2026 using historical data (see Section 5 and 6). To support this 2026 prediction further, the City's operations team should undertake onsite testing of the clarifier and media filters together to quantify their true treatment capacities of this process stages.

Once completed and the expansion schedule refined, the City should engage a Consultant, who can use this report as a basis and work with the City to decide what the future expansion of the WTP needs to address. Once this list of requirements has been developed, processes and technologies should be assessed, and a detailed and thorough concept design and budget should be developed that will allow the City to move immediately into design when the decision is made to proceed with detailed design and construction. As noted before with a number of other significant projects planned within the City, a true understanding of the need and timing of this project is required.

Priority 4 – Existing Distribution System Upgrades

The 2016 Distribution Water Master Plan identified over $35 million of upgrades that were required to ensure sufficient fire flows within the existing distribution system. Recognizing the significance of the budget commitment required to address these issues, the City should develop a plan and schedule by which these 25 upgrade projects can be executed.

Priority 5 – Demolishing of 1974 West End Reservoir.

In 2010 the assessment of the above ground West End Reservoir noted that the reservoir was in a fair to poor condition and had a remaining service life of 10 years. At this time the reservoir remains in service and this report has recommended that stage 2 of the West End Reservoir expansion is completed, allowing the 1974 above ground reservoir to be taken off line and demolished with the next five years. While it might not be possible to implement this work, the City should proceed with the design and tender preparation of the stage 2 expansion and subsequent demolishing of the above ground reservoir, such that in the event that the 1974 reservoir has to be removed from service, the implementation of its replacement is ready to go.

Appendix A
Existing System Upgrades

Appendix A1 - Proposed Existing Distribution System Upgrades

Priority No.	Address	Existing size/Material	Upgrade size/Material	Pipe Upgrade Length (m)	Total Cost ($)	Comments
1	49 Ave from 50 St to 44 St	150 mm, CI	250 mm, PVC	593	$1,720,000	Upgrade - to improve fire flow at several locations east of 49 Ave for highway commerical, institutional, multi-family and single family fire flows (see also Upgrade No. 11 for further improvements)
	49 Ave from 41 St to 40 St	150 mm, CI	250 mm, PVC	103		
2	50 Ave from 18 St to 12 St	150 - 200 mm, AC	250 mm, PVC	645	$1,590,000	Local Upgrade - replace existing 150 mm AC pipe to improve fire flow in the area to the east of 50 Ave.
3	46 St, from 52 Ave along to 51 Ave	150 mm, CI	200 mm, PVC	175	$4,950,000	Local Upgrade - this would replace a section of CI pipe and help improve fire flow here.
	47 St, from 50 Ave to 53 Ave	150 mm, CI	200 mm, PVC	523		Local Upgrade - this would replace a section of CI pipe and help improve fire flow here.
	48 St, from 53 Ave to 52 Ave	150 mm, CI	200 mm, PVC	177		Local Upgrade - this would replace a section of CI pipe and help improve fire flow at node J-1240
	50 St, from 51 Ave to 55 Ave	150 mm, CI	200 mm, PVC	762		Local Upgrade - this would replace a section of CI pipe and help improve fire flow here.
	51 St, from 51 Ave along 51 St to 56 Ave	150 mm, CI	200 mm, PVC	670		Local Upgrade - this would replace a section of CI pipe and help improve fire flow here.
4	50 Ave, from 56B St to 54 St	150 mm, CI	250 mm, PVC	619	$3,060,000	Upgrade - this would replace a section of CI pipe and help improve fire flow here.
	50 Ave west along 56 A St	150 mm, CI	200 mm, PVC	153		Upgrade - this would replace a section of CI pipe and help improve fire flow here.
	55 St, from 51 Ave to 52 Ave	150 mm, CI	300 mm, PVC	174		Upgrade - this would replace a section of CI pipe and help improve fire flow here.
	54A St, from 50 Ave to 51 Ave	150 mm, CI	200 mm, PVC	342		Upgrade - this would replace a section of CI pipe and help improve fire flow here.
5	49 Ave, from 56B St to 54 St	150 mm, CI	250 mm, PVC	610	$1,530,000	Local Upgrade as part of the 49 Ave future upgrades - to meet Industrial, multi-family and single family fire flow
	54 St, from 49 Ave to 48 Ave	150 mm, AC	150 mm, PVC	16		Local Upgrade as part of the 49 Ave future upgrades - to meet Industrial, multi-family and single family fire flow
6	50 Ave, from 60 St to 57 St	150 mm, AC	250 mm, PVC	591	$1,850,000	Upgrade - existing pipe is 150 mm AC and this helps improve fire flow at node J-3805.
	52 Ave, West on 57 St	150 mm, AC	250 mm, PVC	159		Local Upgrade - replace existing 150 mm AC line to improve fire flow here.
7	50 Ave, from 36 St to 29 St	150 mm, AC	250 mm, PVC	823	$3,580,000	Upgrade existing 150 mm AC to help improve fire flow for HWY commercial
	Crosses 50 Ave at 29 St	N/A	250 mm, PVC	157		Upgrade - new 250 mm PVC to improve looping and increase fire flow for HWY commercial and local area to the east
	Crosses 50 Ave at about 32 St	N/A	250 mm, PVC	45		Upgrade - new 250 mm PVC to improve looping and increase fire flow for HWY commercial
	From 36 St along 50 Ave east side to about 35 St	150 mm, AC	250 mm, PVC	124		Upgrade - the existing 150 mm AC to help improve fire flow for HWY commercial.
	31 St, from 50 Ave to 51 Ave	N/A	250 mm, PVC	124		Upgrade - new 250 mm PVC to improve fire flow for the local area
	52 Ave, from 35 St to 34 St	150 mm, AC	200 mm, PVC	204		Local Upgrade - replace existing 150 mm AC pipe to improve fire flow to meet instituttional & Single family fire flow
8	50 Ave, from 42 St to 40 St	150 mm, AC	250 mm, PVC	209	$890,000	Local Upgrade - replaced existing 150 mm AC pipe to improve fire flow for HWY commercial.
	West on 41 St	150 mm, AC	200 mm, PVC	175		Local Upgrade - replaced existing 150 mm AC pipe to improve fire flow for HWY commercial.
9	41 St, from 59 Ave to 57 Ave	150 mm, AC	200 mm, PVC	331	$1,490,000	Local Upgrade - replaced existing 150 mm AC pipe to improve fire flow for HWY commercial.
	43 St, from 56 Ave to 57 Ave	150 mm, AC	200 mm, PVC	364		Local Upgrade - replace existing 150 mm AC pipe to improve fire flow for HWY commercial.
10	62 Ave west on 48 St	200 mm, AC	250 mm, PVC	215	$530,000	Local Upgrade - replaced existing 200 mm AC pipe to improve fire flow for Industrial fire flow.
11	48 St, from 49 Ave to 47 Ave	150 mm, CI	200 mm, PVC	460	$5,330,000	Local Upgrade - this would replace a section of CI pipe and help improve fire flow here.
	48 Ave, from 49 St to 47 St	N/A	250 mm, PVC	196		Local Upgrade - this is a new pipe proposed here that runs north-south and improves looping.
	45 St, from 49 Ave to 48 Ave	N/A	200 mm, PVC	93		Local Upgrade - new 250 mm PVC pipe to go north south to meet the required FF
	46 Ave, from 50 St to 49 St	N/A	250 mm, PVC	101		Local Upgrade - this is a new pipe proposed here that runs north-south and improves looping.
	45 Ave east along 49 St	150 mm, AC	200 mm, PVC	82		Local Upgrade - existing pipe is 150 mm AC.
	45 Ave east along 46 St	150 mm, AC	200 mm, PVC	94		Local Upgrade - existing pipe is 150 mm AC.
	45 Ave east along 45 St	200 mm, PVC	300 mm, PVC	576		Local Upgrade - existing pipe is 200 mm PVC, but would need to upsize to 300 mm PVC in order to meet required fire flow for industrial area.
	52 St, from 45 Ave to about 43 Ave	200 mm, AC	250 mm, PVC	213		Local Upgrade - the existing 200 mm AC main to 250 mm PVC main helps improve flows along 52 St.
	47 Ave, from 49 St to 47 St	N/A	250 mm, PVC	191		Local Upgrade - propose to install new pipe here to provide looping to improve fire flow for industrial area here.
	47 Ave, from 46 St to 45 St	N/A	250 mm, PVC	91		Local Upgrade - new 250 mm PVC pipe to go north south to meet the required FF
	47 Ave, from 47 St to 46 St	N/A	250 mm, PVC	103		Local Upgrade - new 250 mm PVC pipe to go north south to meet the required FF

Appendix A1 - Proposed Existing Distribution System Upgrades

Priority No.	Address	Existing size/Material	Upgrade size/Material	Pipe Upgrade Length (m)	Total Cost ($)	Comments
12	48 Ave, from 27 St to 26 St	150 mm, AC	250 mm, PVC	102	$440,000	Local Upgrade - existing pipe is 150 mm AC and this helps meet the required fire flow in the area.
	27 St, south on 47A Ave	150 mm, AC	200 mm, PVC	88		Local Upgrade - existing pipe is 150 mm AC and this helps improve fire flow in the area.
13	57 Ave, from 51 St to 48 St	150 mm, PVC	200 mm, PVC	291	$1,200,000	Local Upgrade this would replace a section of 150 mm PVC pipe and help meet the required fire flow here.
	57 Ave, west on 51 St	150 mm, PVC	250 mm, PVC	233		Local Upgrade - to meet the fire flow requirements for the institutional area
14	44 St, from 66 Ave to 62 Ave	150 mm, AC	250 mm, PVC	422	$1,040,000	Local Upgrade - to meet Highway Commercial fire flow
15	32 St, from 49 Ave to 48 Ave	150 mm, AC	200 mm, PVC	98	$210,000	Local upgrade to meet required FF
16	62 Ave along 56 St to 59 Ave	N/A	250 mm, PVC	296	$2,000,000	Local Upgrade - propose to install new pipe here to provide looping to improve fire flow for industrial area here.
	59 Ave north to 62 St	N/A	250 mm, PVC	517		Local Upgrade - propose to install new pipe here to provide looping to improve fire flow for industrial area here.
17	West on 65 St and north on 52 Ave	200 mm, AC	250 mm, PVC	490	$1,210,000	Local Upgrade - replace existing 200 mm AC pipe with 250 mm PVC to meet the required fire flow for industrial area.
18	62 St, south on 52 Ave	200 mm, AC	250 mm, PVC	232	$870,000	Local upgrade to meet required FF
	53 Ave from 60 St to 59 St	250 mm, AC	300 mm, PVC	112		City can consider the local upgrade here in order to meet required FF at node J-51. This location is fairly close to the WTP and given the calibrated C value for AC pipe is 90, the model may be underestimating the flows here. City may want to consider doing local fire flow testing here.
19	29A St, north on the PUL between 58 Ave and 57B Ave	150 mm, AC	200 mm, PVC	96	$210,000	Local Upgrade - new and replace existing 150 mm AC pipe to improve fire flow to meet SF residential flow.
20	46 Ave West on 35 St	150 mm, AC	200 mm, PVC	125	$270,000	Local Upgrade - replaced existing 150 mm AC pipe to improve fire flow to meet low density residential fire flow.
21	North of 36 St	N/A	300 mm, PVC	125	$110,000	Upgrade - new 300 mm line to residential area to meet the required fire flow for medium and single family residential.
22	30 St and 55A Ave	150 mm, AC	200 mm, PVC	210	$780,000	Local upgrade to meet required FF
	30 St , south of 55 Ave	150 mm, AC	200 mm, PVC	152		Local upgrade to meet required FF
23	46A Ave, along 23 St	150 mm, AC	200 mm, PVC	84	$180,000	Local upgrade to meet required FF
24	35 St, south on 45A Ave	150 mm, AC	200 mm, PVC	89	$190,000	Local upgrade to meet required FF
25	50 Ave south to 44 St	N/A	250 mm, PVC	32	$80,000	Local upgrade to provide looping to improve the level of service here

Notes:

1. AC = Asbestos Cement
   CI = Cast Iron
   FF = Fire flow
   HWY = Highway
   WTP = Water Treatment Plant
2. Upgrades are grouped by location.
4. Cost Estimates are conceputal.
5. Costs are in 2015 dollars and are based on historical costs as provided by the City of Llodyminster.
7. The total cost is the cost for the entire upgrade group.

Appendix B

Future System Upgrades

Appendix B - Proposed Future Distribution System Upgrades (Near Term to 20 Year Horizon)

Upgrade Type		Address	Upgrade size/Material	Pipe Upgrade Length (m)	Total Cost ($)	Comments
NEAR FUTURE						
FUTURE 3 Year						
Network Upgrade		44 St, from 70 Ave to 75 Ave and from 75 Ave at 44 St to 52 St.	500 mm, PVC	1360	$1,774,800	Extend the 500 mm main on 44 St, in order to meet minimum system pressures during peak hour demand in the northwest industrial area.
Fire Flow Upgrade		48 Ave, from 54 St to 53 St	200 mm, PVC	111	$238,220	New 200 mm pipe, running north-south is proposed in order to meet the required FF. Increases fire flow from 75 L/s to 146 L/s (100 L/s is required).
FUTURE 5 Year						
FUTURE 10 Year						
Network Upgrade		At the WR discharge	750 mm, PVC	160	$698,320	Upgrade the 500 main leading out of the WR with 750 mm (or equivalent pipe twinning)
Network Upgrade		43 St, from about 62 Ave to 66 Ave	750 mm, PVC	207	$903,470	Upgrading in the vicinity of the WR is required to minimize headloss and ensure minimum system pressures are met during peak hour demand.
FUTURE 20 Year						
J-78	J-77	73 Ave, from 43 St to 44 St	600 mm, PVC	162	$641,280	Twin the existing 400 mm PVC with 600 mm PVC to minimize headloss and ensure minimum system pressures are met during peak hour demand.
J-77	J-1196	44 St, from 66 Ave to 70 Ave	500 mm, PVC	520	$1,869,920	Twin the existing 500 mm PVC with 500 mm PVC to minimize headloss and ensure minimum system pressures are met during peak hour demand.
J-290	J-2322	48 Ave, from 42 St to 41 St	200 mm, PVC	99	$212,460	New 200 mm pipe, running north-south is proposed in order to meet the required FF. Increases fire flow from 224 L/s to 289 L/s (225 L/s is required).

Notes:

1. FF = Fire flow
   WR = West End Reservoir
2. Pipe length is based on the model scaled length.
3. Cost estimates are conceputal and are in 2015 dollars. Where possible, rates are based on historical costs provided by the City of Lloydminster.