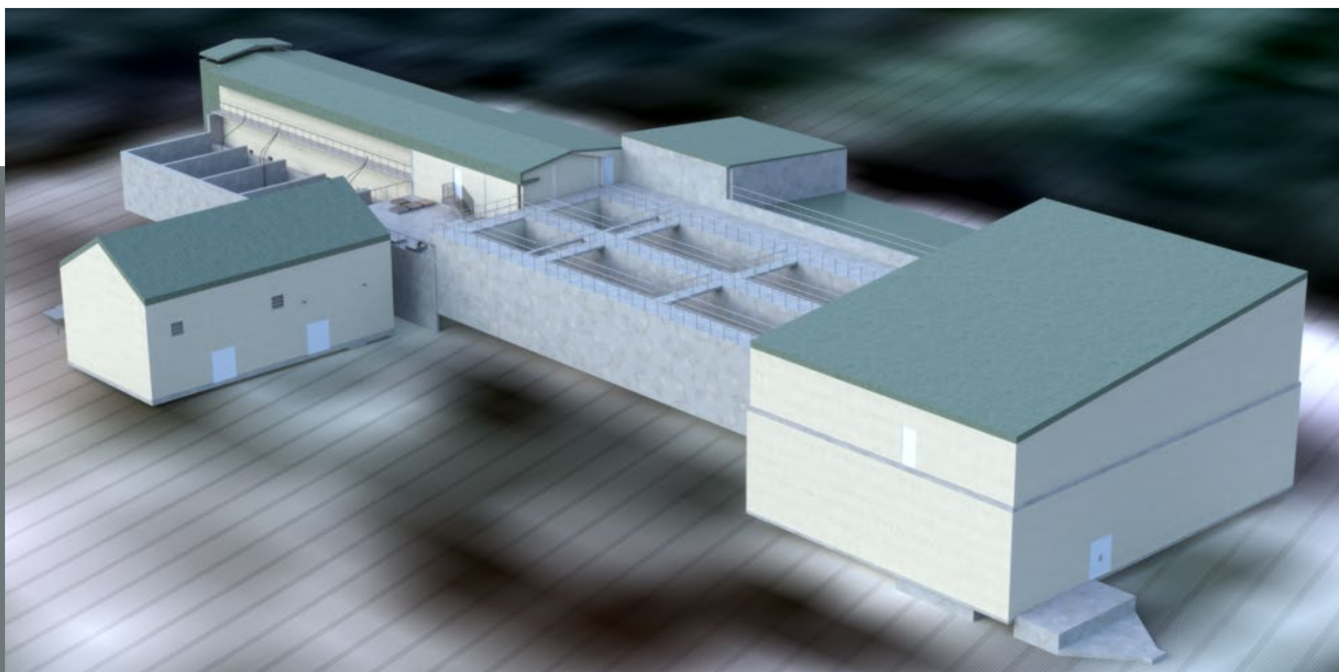
Town of Canmore

Wastewater Treatment Plant

Capacity Evaluation and Capital Upgrades



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The Association of Professional Engineers and Geoscientists of Alberta

Executive Summary

The Canmore WWTP is a two stage BAF plant for BOD removal and Nitrification. A detailed capacity assessment was conducted on all the major process units to determine bottleneck process units and aid capital planning for the 5, 15 and 25 year horizon.

The historical flow rates through the plant, loadings and effluent concentrations from 2017 through 2021 were analyzed. The historical performance was generally good with the ability to treat the wastewater within compliance limits across the challenging wet weather flows due to the spring freshet as well as the winter and max month loading scenarios. The projected flows and loads were estimated from population projections at the 5, 15 and 25 year horizon.

The Canmore WWTP is nearing its capacity for major process units and is expected to experience challenges within the next 5 years. Between the 5 and 10 year horizon, the BAF system is expected to experience difficulties to reliably achieve the effluent ammonia limits. A BioWin model was initially calibrated and aligned well with the capacity assessment demonstrating the facility will be challenged to achieve the final effluent limits by the 10 year-horizon. Process optimizations and potential stress testing are recommended to identify the empirical limitation of the nitrification process prior to reaching the capacity limit.

The water quality based effluent limits study has identified draft limits that will come into effect c. 2031. The new limits are more stringent with the inclusion of a total nitrogen limit. The existing facility will not able to be achieve these limits without a significant upgrade and potentially a change in secondary treatment technology. A separate report is being produced to identify the preferred alternative for achieving the new limits.

The capital planning identified in this report focus on the next 10 years to support the existing facility in operating prior to the large capital upgrade. The capital planning prioritized upgrades that can be re-used beyond 2031 and reduce wasted capital investment. The following table outlines the list of identified projects.

Project	Description	Project Justification	Priority	Year Required	Probable Cost
Headworks					
Inlet Lift Station Upgrade (Mechanical)	Replacement of existing lower flow pumps and discharge piping	Life Cycle Population Growth	Medium	2027	$1.17M

Inlet Lift Station Upgrade (Wetwell)	Increase the size of the Inlet LS Wetwell, Provide Actuated valving to EQ tank	Undersized Population Growth	Medium	2027	$2.9M
Odor Control Unit for EQ Tank, Headworks Bldg	Add odor control building near the Headworks to treat odors from EQ tank, Headworks	Potential Complaints, Future Regulatory Requirements	Medium	2027	$2.9M
Septage Receiving Station	Add septage receiving station with flow monitoring and payment system [Odor Control project required before]. Include EQ tank upgrades	Population Growth	Low	2032	$1.17M
Influent Piping between Inlet LS and Headworks	Piping Replacement, Actuated isolation valves at high point	Life Cycle	High	2027	$1.17M
Inlet Screen Replacement	Replacement of older inlet screen with smaller mesh, and sludge press unit	Life Cycle	High	2024	$900k
Grit Separator Replacement	Replace Existing Grit Separator	Life Cycle	Medium	2027	$720k
Grit Separator Exhaust Fan	Redesign, replace. Existing fan full of grease	Process Improvements	High	2025	$290k
Headworks Channel Valves	Add sluice gate valve at the Clarifier Distribution Channel Actuation on Clarifier sluice gates and screen inlet gates	Process Improvements	Medium	2027	$530k

Water Heating System, MUA Replacement	Replace existing boiler, piping, MUAs	Life Cycle Process Improvements	Medium	2025	$2.17M
Scum Removal Piping	Rearrange Scum Removal Piping to pump to digester instead of Headworks	Process Improvements	High	2025	$720k
Third Clarifier Addition	Add third Clarifier [high flow fluctuations]. North of ex. clarifier	Process Improvements Population Growth	Medium	2027	$10.2M
BAF, DAF					
Intermediate Transfer Pumps Upgrade	Upgrade existing pumps	Life Cycle	Medium	2026	$1M
UV					
UV 1, 2 upgrade	Replace existing UV 1, 2	Life Cycle	High	2023	700k
UV 3 addition	Add UV3	Population Growth	Medium	2028	500k

Update after the report completion (April 5. 2023)

The Town has completed the Water Quality Based Effluent Limits study and the Environment and Protected Areas (EPA) assigned the new effluent limits that includes the requirements for Total Nitrogen and deeper Phosphorus treatment.

CIMA+ has issued a separate report "Wastewater Treatment Plant Technology Evaluation" (April 5, 2023). This report provides the upgrade recommendations for Canmore WWTP to achieve the effluent limits required by EPA. The complete capital costs required for Canmore WWTP upgrade are combined in the "Wastewater Treatment Plant Technology Evaluation" (April 5, 2023) report, and include the costs noted in the "Capacity Evaluation" report.

Table of Contents

List of Figures

List of Tables

List of Appendices

1 Introduction

The Canmore Wastewater Treatment Plant (WWTP) is in the Town of Canmore in Alberta, Canada and has been in service since 1997. The plant provides secondary treatment with biological aerated filtration (BAF) and ultraviolet (UV) disinfection prior to discharging effluent to the Bow River. The wastewater treatment system consists of the following processes:

- Septage receiving
- Influent pumping
- Mechanically cleaned influent bar screening
- Flash mix chemical addition
- Enhanced primary clarification
- Primary effluent screening
- Two stage biological aerated filtration
- Ultraviolet Disinfection
- Backwash storage tank
- Dissolved air flotation (DAF)
- Two open-air holding tanks “digesters”
- Centrifuging

The following figure is the process flow diagram (PFD) of the Canmore WWTP.

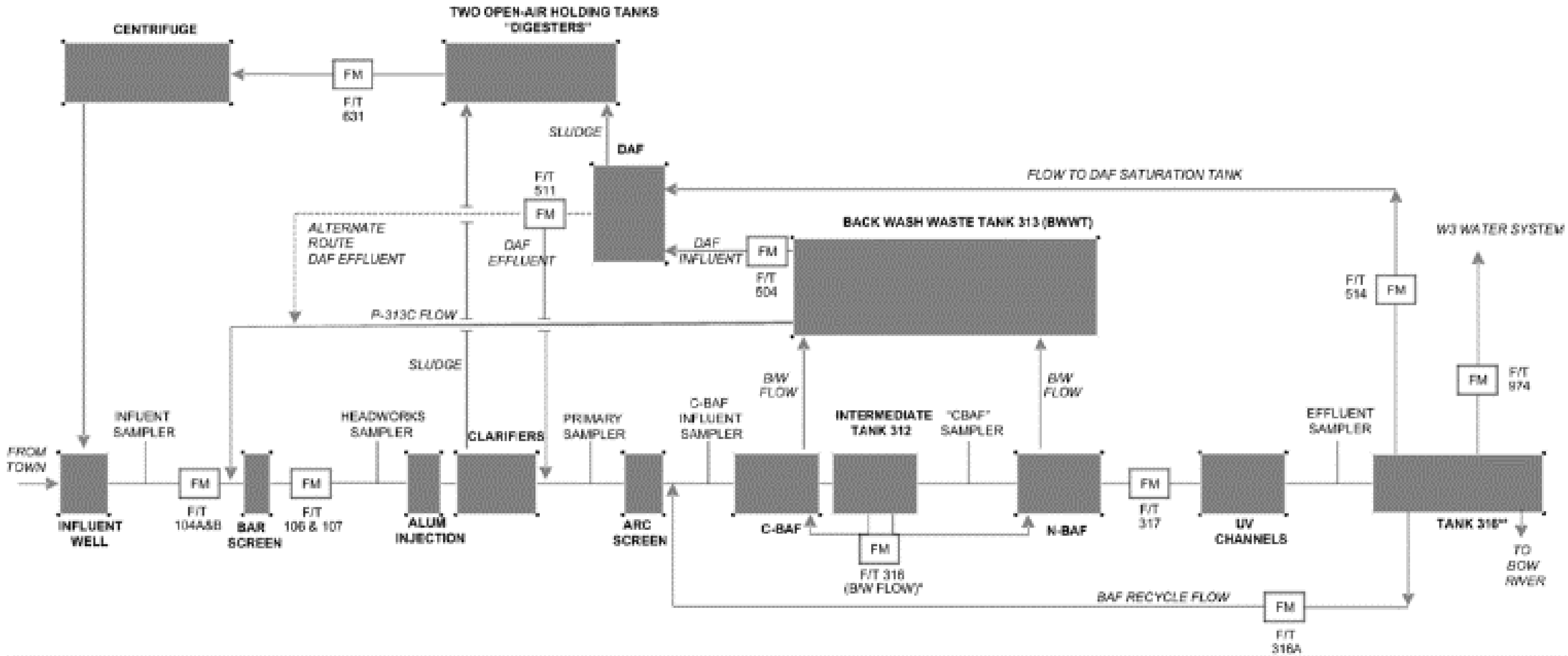

Figure 1-1 Process flow diagram (PFD) of the Canmore WWTP

The Town of Canmore is planning to increase its permanent population while maintaining significant seasonal population fluctuations due to tourism in both the summer and winter months. This growth will increase the hydraulic and constituent loading to the treatment plant. To understand the impacts of this growth, a capacity assessment of the existing wastewater plant is required to properly plan for short and long-term upgrades that may be required to maintain compliance within existing effluent limits.

1.1 Work Scope

The tasks that were completed for this report were as follows:

+ Analyze historical data to formulate trends and max flow factors
+ Assess the existing capacity of each process unit at the Canmore WWTP
+ Assess the capacity of the Canmore WWTP at 5, 15, and 25 year horizons
+ Use BioWin modelling to simulate the current and future scenarios

1.2 Objectives

The objective of this capacity assessment report is to identify bottlenecks in the existing processes at the Canmore WWTP. The findings will help steer planning for short and long-term planning of upgrades.

2 Historical Data

The capacity assessment of the process units within the Canmore WWTP relies on plant data to determine existing flows and loads entering the facility. The historical data was synthesized from 2017 to 2021 to establish various flow rates and loading scenarios to assess the plant and individual process units to determine the capacity at existing conditions. The established historical data is also used as the baseline flows and loadings for future projections at the 5, 15 and 25 year horizons.

2.1 Population

The reported population for the Town of Canmore was 19,865 in 2021 inclusive of permanent and non-permanent residents. The population for the Town of Canmore has been projected for the time intervals of 5, 15, and 25 years; the following table illustrates the projections for the population.

Table 2-1: Population Projections

Population	Current	2027	2037	2047
Permanent	15,990	20,982	25,308	27,758
Non-Permanent	3,875	5,820	10,462	16,982
Total	19,865	29,802	35,770	44,740

With these population projections, future flows and loadings entering the WWTP will be determined in the proceeding sections, and then the plant can be deemed to have sufficient capacity or require upgrades to handle the future flows and loadings.

2.2 Influent Flowrate

The influent flowrates were analyzed for the study period from 2017 to 2021 (Table 2-2). The annual average daily flow (ADF) for the study period corresponded to a per capita wastewater generation rate of 435 L/p/d. As the population of Canmore is set to increase (Table 2-1), the flows entering the WWTP will increase. The future flow projections were based on a historic per capita wastewater generation of 435 L/p/d, which is close to the general typical value of 450 L/p/d for planning purposes.

The more granular flows were analyzed for max month flow (MMF), peak daily flow (PDF) and peak hourly flow (PHF). From Figure 2-1 the peak flow occurs during June of each year. This is likely due to the large snow melt that occurs around the Canmore area that contributes to elevated extraneous flows entering the sanitary system. Extraneous flows are common for most communities and are typically challenging to identify and quantify. At Canmore, the peaks are relatively consistent over the five-year period and isn't during a noticeably high traffic time of year. This indicates the primary source of the peak flows is likely driven by the spring freshet.

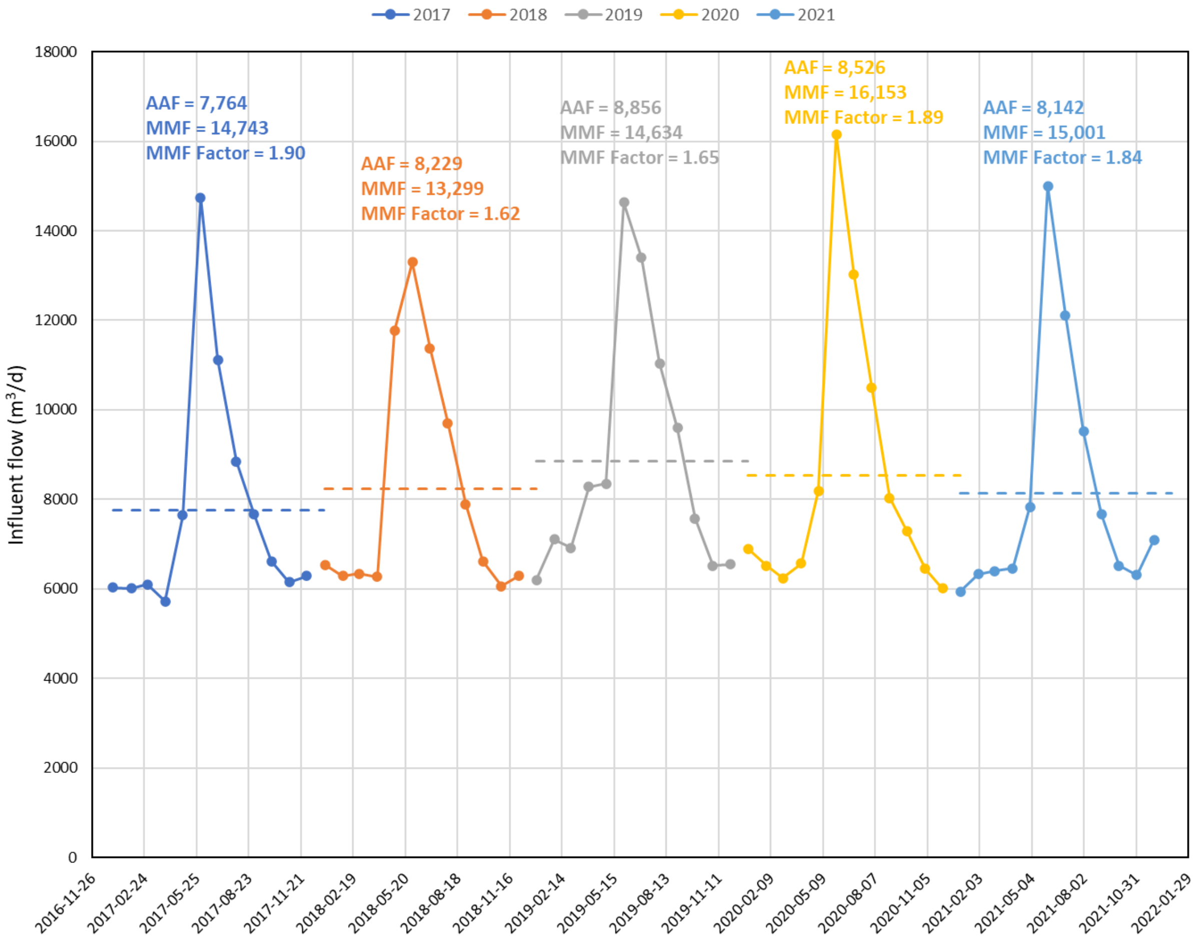

Figure 2-1: Historic Influent Flows for 2017 – 2021

The future MMF, PDF, and PHF were calculated using the historic ratios of each parameter to the ADF. The calculated peaking factors (PF) for the MMF is higher than typically observed, however, the PDF and PHF are within typical expectations for a network this sized facility. These factors were carried forward to the future projected flows as the design basis.

Table 2-2: Canmore WWTP Design Influent Flows

	2017 - 2021	2027	2037	2047
Average Annual Flow – ADF (ML/d)	8.3	11.3	15.2	19.1
Maximum Monthly Flow – MMF (ML/d)	16.2 (PF 1.95)	22.0	29.6	37.2
Peak Daily Flow – PDF (ML/d)	20.9 (PF 2.52)	28.5	38.3	48.1
Peak Hourly Flow – PHF (ML/d)	30.2 (PF 3.64)	41.1	55.4	69.5

2.3 Influent Characteristics

At the Canmore WWTP, the characteristics regularly measured in the influent water are biological oxygen demand (BOD), chemical oxygen demand (COD), total suspended solids (TSS), total ammonia nitrogen (TAN), and total phosphorus (TP). The following graphs present the historic loading of these constituents for the past 5 years (2017-2021). The constituent loadings are used to size biological process units; hence it is important to understand the monthly loadings to assess the capacity during the "worst-case scenario". For all parameters, the maximum influent loadings typically occur in the summer months (July-August) of each year, correlating with the increased tourist population during these months.

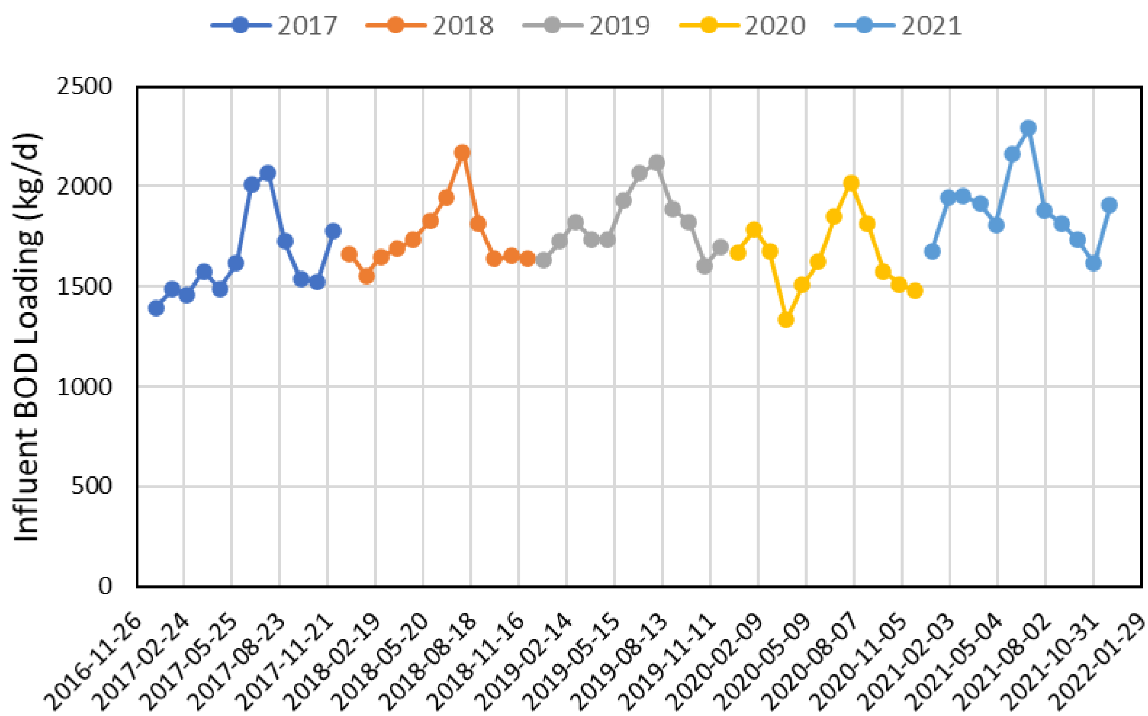

Figure 2-2: Influent BOD Loading from 2017-2021

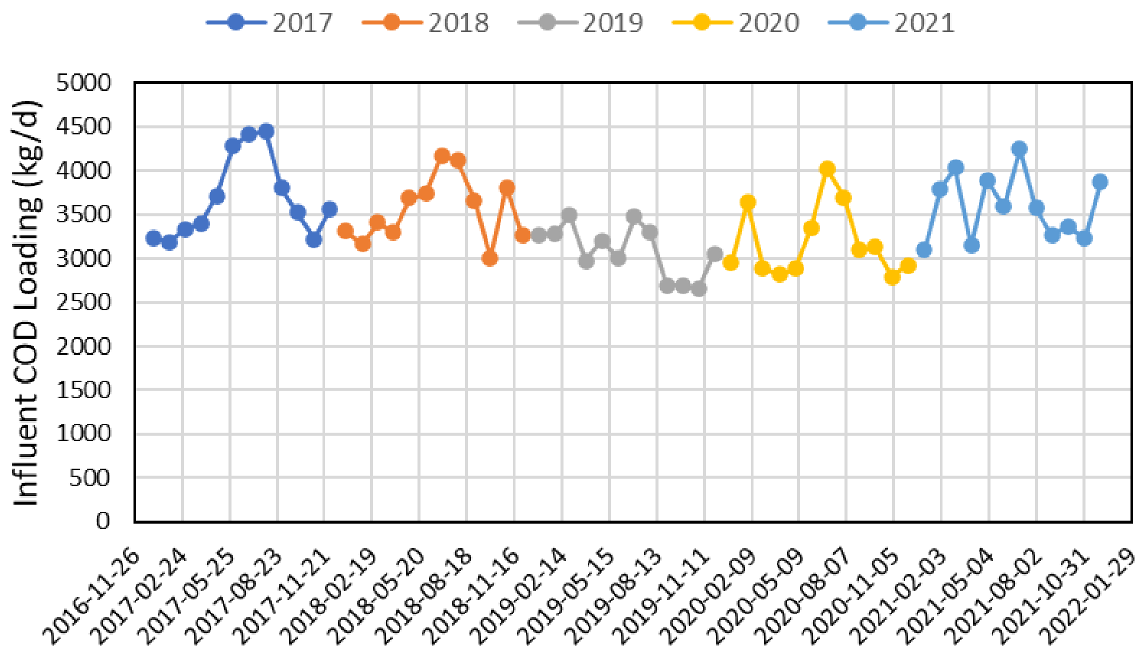

Figure 2-3: Influent COD Loading from 2017-2021

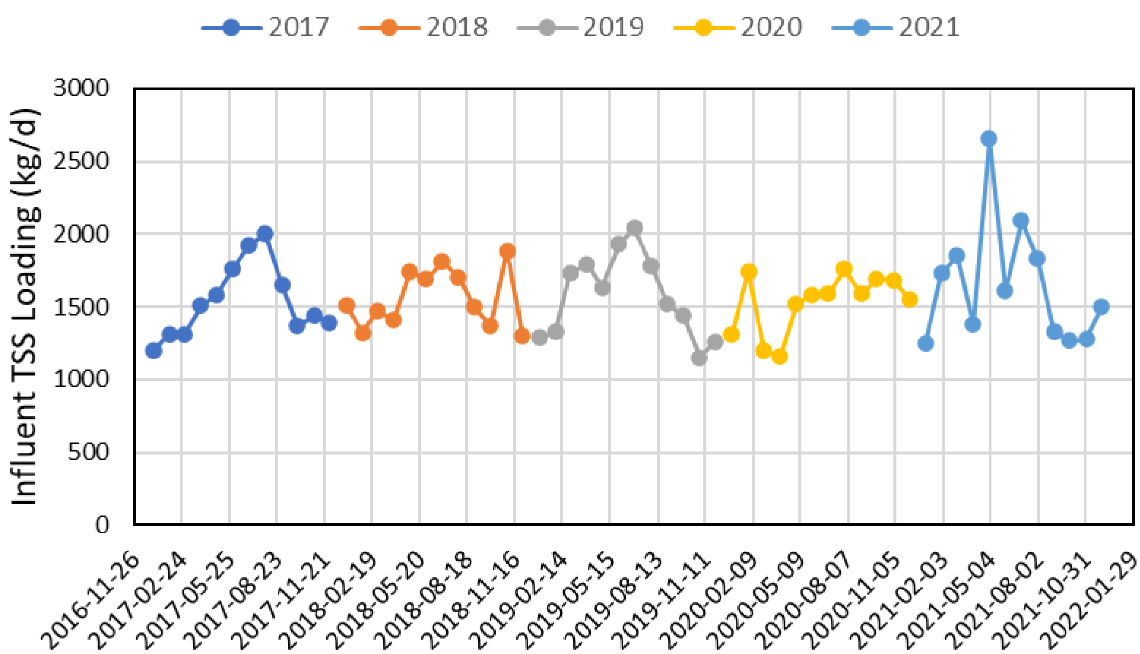

Figure 2-4: Influent TSS Loading from 2017-2021

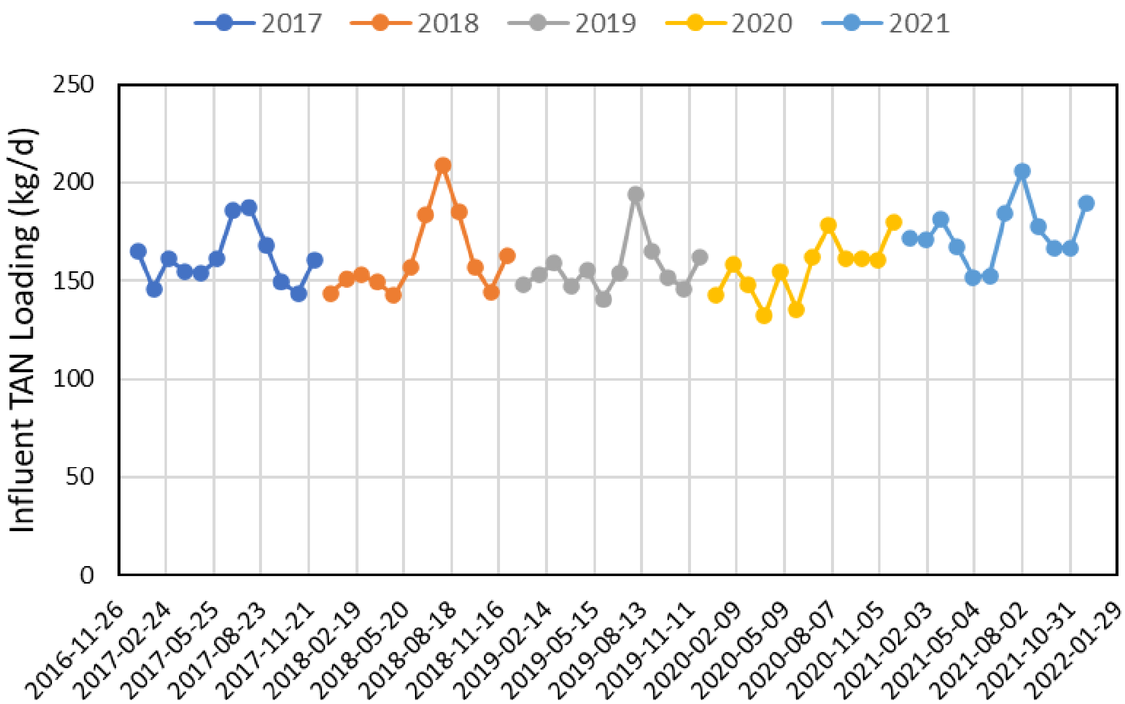

Figure 2-5: Influent TAN Loading from 2017-2021

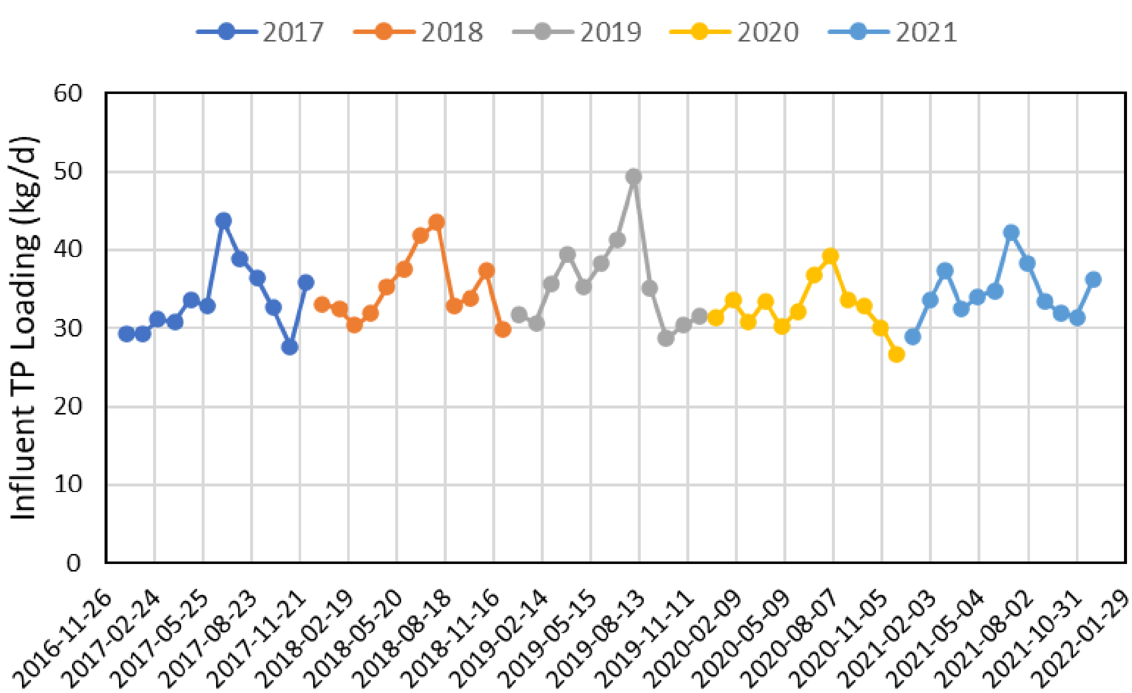

Figure 2-6: Influent TP Loading from 2017-2021

2.4 Existing Plant Performance Limits

Currently EPCOR and the Alberta Environment and Parks are in discussion about the future effluent regulations to which the Canmore WWTP must adhere. The current effluent regulations are shown below.

Table 2-3: Current Canmore WWTP Effluent Limits

Parameter	Effluent Limit
$cBOD_5$	≤ 20 mg/L (1)
TSS	≤ 20 mg/L (1)
TAN	≤ 5 mg/L (1) (Jul – Sep) ≤ 10 mg/L (1) (Oct – Jun)
TP	≤ 1.0 mg/L (1)
Faecal Coliform	≤ 200 per 100 mL(2)
Notes: (1) Monthly arithmetic mean of daily composite samples (2) Monthly geometric mean of daily grab samples	

2.5 Effluent Characteristics

The following graphs compare the Canmore WWTP effluent limits (Table 2-1) to the historic average daily effluent data for $cBOD_5$, TSS, TAN, TP, and fecal coliforms per month for the past 5 years (2017-2021).

The plant is consistently below the 20 mg/L effluent limits for BOD and TSS. Higher effluent TSS concentrations are typically recorded in spring (March-June) of each year, indicating that the plant's process units may be hydraulically strained by high flows during these months.

In 2017, the TAN effluent concentration exceeded the 10 mg TAN/L limit between January and March and exceeded the 5 mg TAN/L limit between July and September. It is important to note that in 2017 several BAFs had to be taken offline for operational maintenance and repair, artificially reducing capacity, and resulting in higher effluent TAN concentrations. Between 2018 to 2021 the plant exceeded the TAN limit of 5 mg TAN/L in summer on two occasions, July and August of 2021, with effluent concentrations of 5.04 and 6.54 mg TAN/L respectively.

The plant has historically achieved adequate TP levels in the effluent. While Figure 2-10 shows a trend of increasing TP levels in the effluent stream from the plant, the concentration of TP has not exceeded the allowable 1 mg/L over the last 5 years.

From Figure 2-16, it can be observed that the Canmore WWTP has approached or past the effluent regulation limit for fecal coliform of 200/100 mL on several occasions throughout the last 5 years. An upgrade to UV disinfection is planned to meet the allowable limit of fecal coliforms in the effluent.

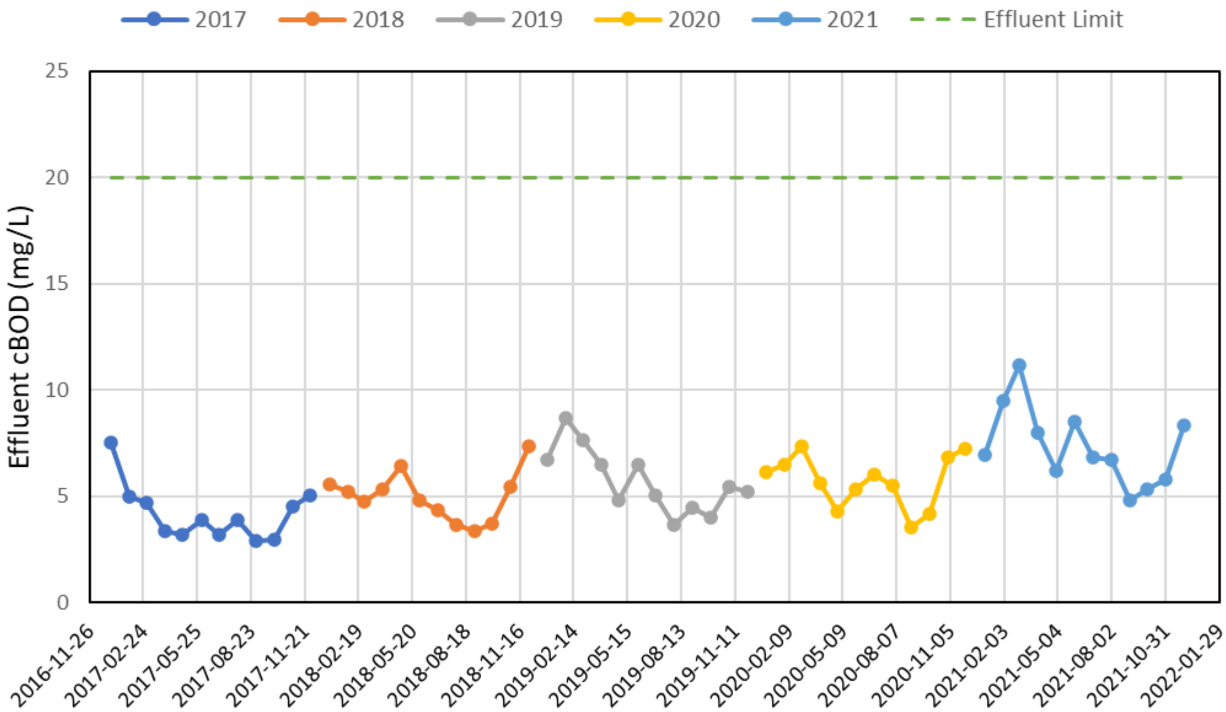

Figure 2-7: Effluent BOD level from 2017 – 2021

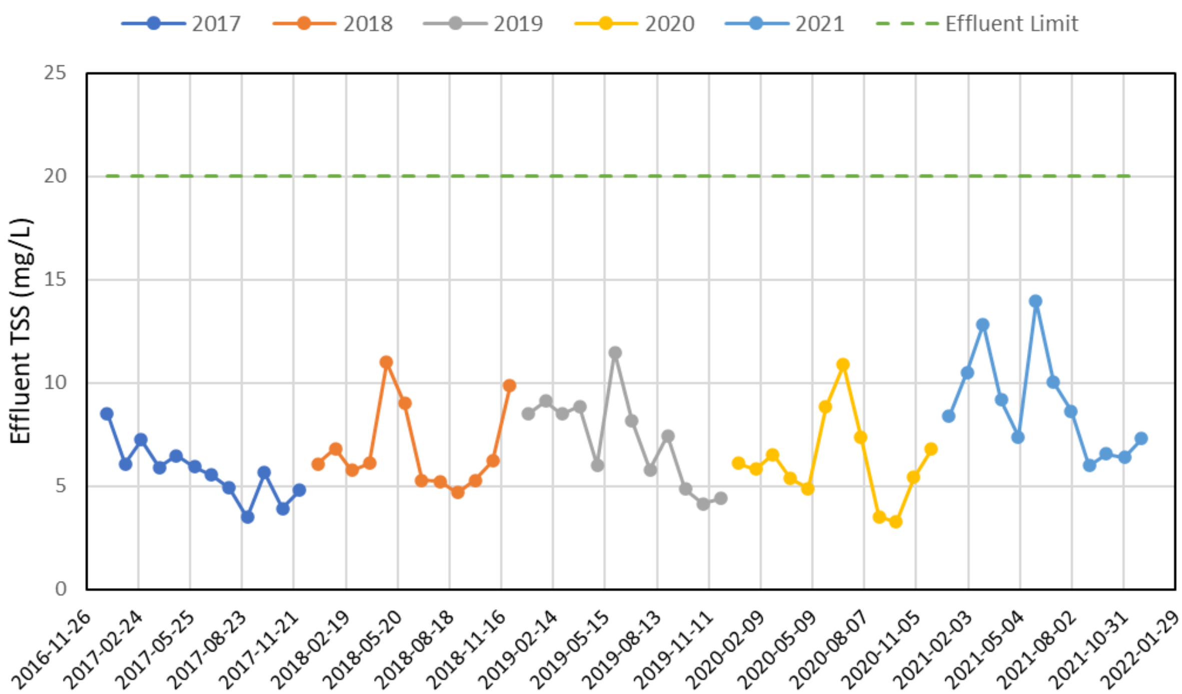

Figure 2-8: Effluent TSS level from 2017 – 2021

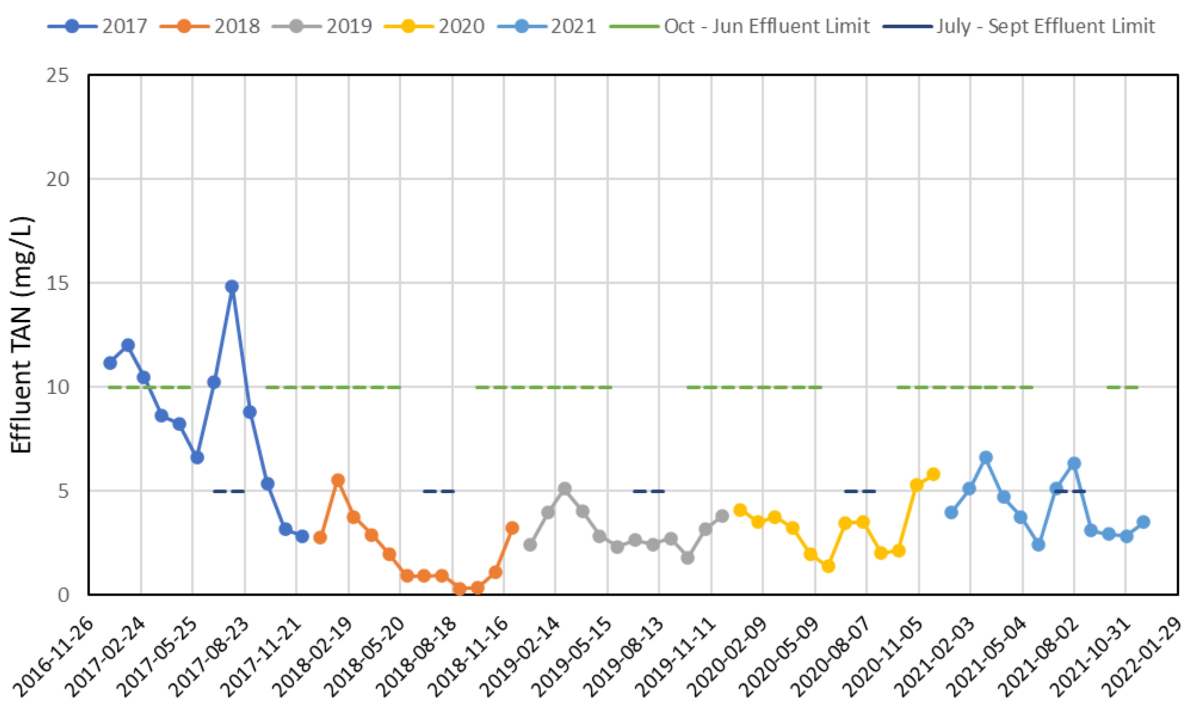

Figure 2-9: Effluent TAN level from 2017 – 2021

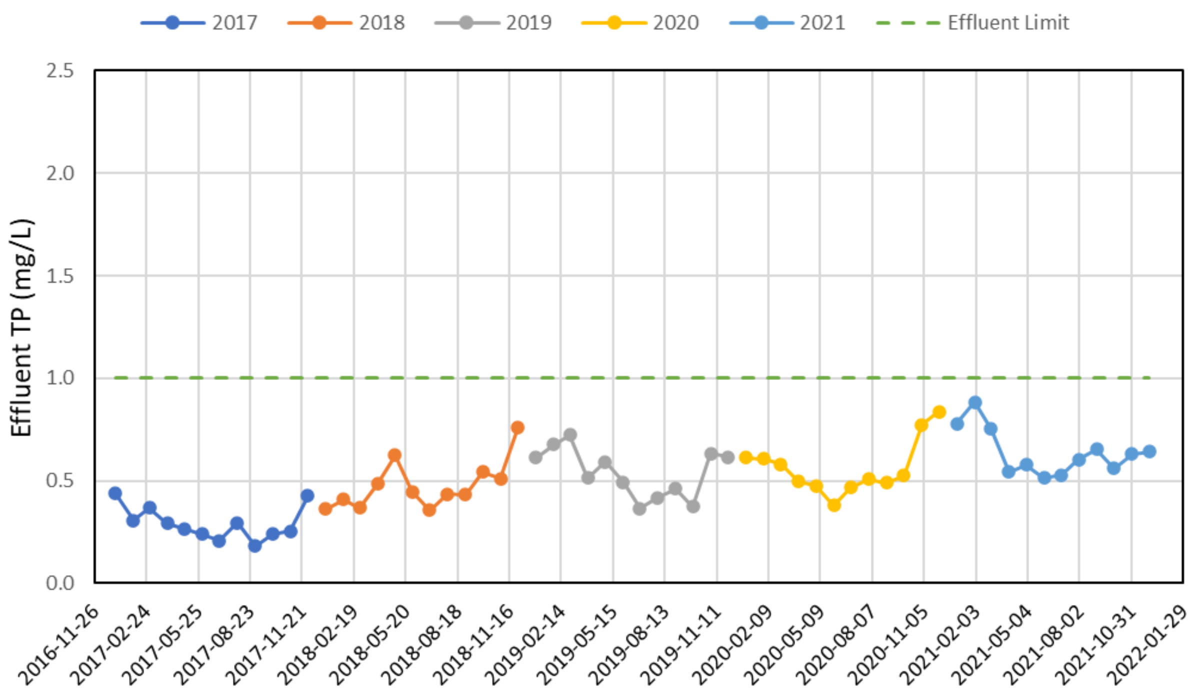

Figure 2-10: Effluent TP level from 2017 – 2021

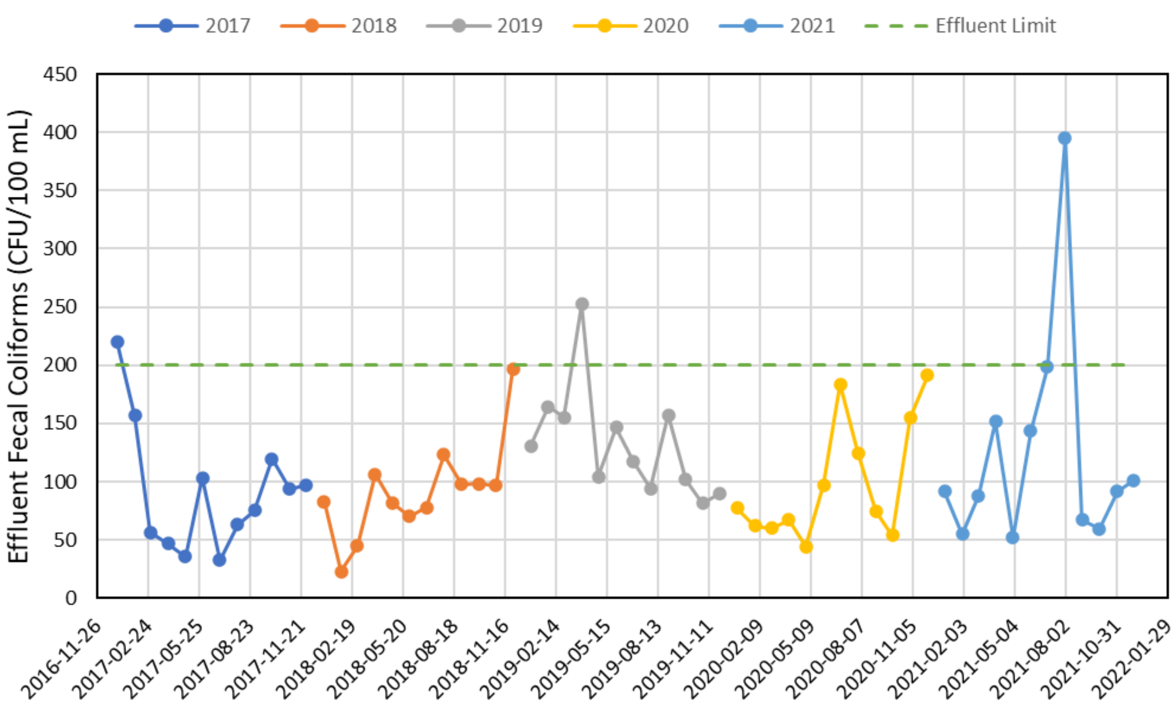

Figure 2-11: Effluent Fecal Coliform level from 2017 – 2021

3 Existing Plant Summary

Table 3-1 outlines the current equipment at the Canmore WWTP along with the rated capacities for each instrument and unit process.

Table 3-1: Canmore WWTP Design Data

Component	Current Available Capacity
Design Population	
Permanent Population	15,990
Non-Permanent Population	3,875
Total	19,865
Influent Characteristics	
Flows	
ADF, ML/d	8.3
MMF, ML/d	16.2
PDF, ML/d	20.9
PHF, ML/d	30.2
Loads	
BOD	
Average, kg/d	1,754
Maximum Month Factor	1.22
TSS	
Average, kg/d	1,574
Maximum Month Factor	1.31
TAN	
Average, kg/d	162

Component	Current Available Capacity
Maximum Month Factor	1.20
TP	
Average, kg/d	34.4
Maximum Month Factor	1.27
Equalization Tank	
Tank Cells	2
Tank Volume (total), m^3	440
Number of Overflow Pumps	2
Capacity per Pump, L/s	17.1
Power per Pump, KW	2.2
Influent Pumping	
Pump 100 C/D	
Number of Pumps	2
Capacity per Pump, L/s	105
Power per Pump, KW	18.7
Pump 100 A/B	
Number of Pumps	2
Capacity per Pump, L/s	190
Power per Pump, KW	37.3
Total Capacity, ML/d	50.9
Total Firm Capacity, ML/d	34.6

Component	Current Available Capacity
Mechanically Cleaned Bar Screens	
Number of Screens	2
Bar Opening, mm	6 (primary) 15 (bypass channel)
Width, mm	1,000
Water Depth, mm	1,050
Capacity per unit, ML/d	34.0
Grit/Scum Removal (Spiral Roll)	
Number of Chambers	1
Number of Sumps per Chamber	4
Volume, m^3	128
Capacity, ML/d	35.0
Alum Addition System	
Number of Pumps	2
Capacity per Pumps, L/min	4.0
Alum Storage, m^3	27.3
Polymer Addition System	
Number of Pumps	2
Capacity per Pumps, L/min	7.8
Primary Clarifiers	
Number	2
Dimensions	

Component	Current Available Capacity
Length, m	33.0
Width, m	6.6
Depth, m	5.0
Surface area per Clarifier, m^2	218
Volume per Clarifier, m^3	1,089
Automatic Curved Arc Screens	
Number of Screens	2
Screen Opening, mm	2.4
Width, mm	1,000
Water depth, mm	1,000
Capacity per Unit, ML/d	46.0
Biological Aerated Filters	
C-side Filters	
Number	5
Cell Dimensions	
Media Depth, m	2.5
Surface Area, m^2	40
Media Volume, m^3	100
N-side Filters	
Number	5
Cell Dimensions	
Media Depth, m	2.5

Component	Current Available Capacity
Surface area, m^2	40
Media Volume, m^3	100
Intermediate Pumps (P-312 A/B)	
Number	2
Capacity per Pump, L/s	128
Size per Pump, kW	9.5
Firm Capacity, ML/d	22.1
Aeration Blower	
Number	10
Capacity per Blower, m^3/h	440
Size per Blower, kW	18.7
Backwash Pumps (P-310 A/B & P-311)	
Number	3
Capacity per Pump, L/s	111
Size per Pump, kW	12.8
Backwash Waste Pumps (P-313 A/B/C)	
Number	3
Capacity per Pump, L/s	36.1
Size per Pump, kW	7.5
Backwash Blower	
Number	3
Capacity per Blower, m^3/h	1,400

Component	Current Available Capacity
Size per Blower, kW	45.0
Backwash Waste Tank	
Volume, m^3	460
Intermediate Transfer tank	
Volume, m^3	305
Recirculation Pumping (P-316 A/B)	
Number of Pumps	2
Capacity per Pump, L/s	56
Power per Pump, kW	7.5
Total Capacity, ML/d	9.7
UV Disinfection	
UV Transmissivity, %	65
UV Dosage, mWs/cm^2	22.4
Number of Channels	2
Number of Banks per Channels	1
Number of Lamps per Channels	42
Primary Sludge Pumping (P-134 A/B)	
Number of Pumps	2
Capacity per Pump, L/s	11.4
Grease Pumping (P-127, P-136)	
Number of Pumps	2
Capacity per Pump, L/s	11.4

Component	Current Available Capacity
Aerobic Digesters	
Sludge TS Concentration, %	2.2
Number of Reactors	2
Dimensions	
Diameter, m	14.0
SWD, m	7.5
Volume per Reactor, m^3	1,155
Aerobic Digester Blowers	
Number	2
Capacity per Blower, m^3/h	2,108
Size per Blower, kW	75
Digested Sludge Pumps (P-610 A/B)	
Number of Pumps	2
Capacity per Pump, L/s	4.1
Size per Pump, kW	3.7
Dissolved Air Flotation (DAF)	
Number	2
Dimensions	
Length, m	3.6
Width, m	2.1
Total Surface Area, m^2	7.7
Thickened sludge, %	2 - 4

Component	Current Available Capacity
Loading Rate Range, $m^3/m^2/h$	24 – 48
DAF Pumps (P-511 A/B)	
Number of Pumps	2
Capacity per Pump, L/s	50
Size per Pump, kW	11
Centrifuge	
Number	2
Operation hours per day, hr	8
Operation days per week, d	5
Minimum Solids Capture Efficiency, %	95%
Feed Sludge TS Concentration, %	1.8
Designed Firm Capacity, m^3/d	100
Flow Capacity per Centrifuge, m^3/h	29.5
Solid Loading Rate, kg/h	530
Centrifuge Sump Pumps (P-660 A/B)	
Number of Pumps	2
Capacity per Pump, L/s	13.6
Size per Pump, kW	3.7

4 Existing WWTP Capacity Assessment

4.1 Methodology

A capacity assessment aims to evaluate the capacity and performance of each unit process in isolation and in conjunction with the entire treatment train operating as a system. An assessment was conducted in the present report, to determine the capacity of each liquid and solid unit process. The assessment assists in identifying process constraints for further evaluation with operations and planning for upgrades. This capacity assessment in general was based on the current and the future design criteria, historical operational plant data during the study period of 2017 to 2021, equipment data and specifications, and typical design guidelines (MECP Design Guidelines for Sewage Works, 2008; WEF MOP 8, 2009; Metcalfe and Eddy, 2014). The Ontario Ministry of the Environment, Conservation and Parks (MECP) Design Guidelines were used as they recently underwent a comprehensive update for typical treatment performance values and a basis for benchmarking several WWTP unit processes.

4.2 Septage Receiving

The Canmore WWTP receives both wastewater from the sewer system and septage that is trucked to the plant. Historically, the plant receives approximately one truck of septage per day, however this fluctuates throughout the year. The septage that arrives at the plant is deposited upstream of the influent lift station or at the equalization tank. Septage flows and concentrations are quantified by the plant's influent sampler, thus are included in the plants' influent flows and loading values.

If the septage is received upstream of the influent lift station, the wastewater and septage enter the plant in the usual manner and go through all the plant's process units for treatment. If the septage is received at the equalization tank, it must pass through a screen to remove any large objects and flows to the influent well so that it can be pumped through the plant for treatment.

4.3 Equalization Storage

An equalization tank was installed at the Canmore WWTP to offer equalization and surge protection in the event of a power failure. The tank has a volume of 440 m^3 and is capable of storing diverted flows from the influent wet well for up to 30 minutes during high flow periods. The equalization tank contains two cells, one larger cell for overflow and surge protection, while the second cell is smaller, and acts as a septage receiving cell.

The overflow cell of the tank has two submersible pumps each with a capacity of 17.1 L/s, that return the overflow wastewater to the influent lift station so that it can be pumped through the plant for treatment.

Due to the anticipated increase in flows over the next 25 years, additional equalization capacity may be required to maintain 30 minutes of storage during high flow periods and mitigate plant bypass events. It's important to note, significant plant upgrades may negate the need for an EQ tank for buffering peak flows.

4.4 Influent Pumping

The influent lift station transfers wastewater from the collection system to the head of the WWTP. There are currently four submersible influent pumps in the influent lift station. Two of the pumps have a capacity of 105 L/s, while the other two pumps have a capacity of 190 L/s. The firm capacity (one of the larger pumps out of service) of the influent pumps is 34.6 ML/d.

The most recent upgrade to the influent pumps occurred in 2015 when two 105 L/s pumps were replaced with the current 190 L/s pumps. This upgrade ensured that the plant has sufficient influent pumping for the historical peak hourly flows of the plant, which are 30.2 ML/d (see Table 3-1). However, with the expected population increase in the Canmore area, future upgrades may be required to meet the projected peak hourly flows.

4.5 Screening

Screening is an important process of WWTPs as the screens remove large debris that could harm or clog downstream processes. At the Canmore WWTP, currently there are two screens that are used. The primary screen is a 6 mm mechanically cleaned bar screen, while the second screen, located in the bypass channel, has is manual with an opening of 15 mm. The mechanical bar screen has a rated capacity of 34 ML/d which is currently sufficient to meet plant's historical peak hourly flows.

As only the primary screen is typically in service, the Canmore WWTP will likely have insufficient screening capacity for the future flows. In future, replacement of the 15 mm bypass screen with a 6 mm mechanically cleaned bar screen may be required to provide sufficient capacity and redundancy.

4.6 Aerated Grit Chamber

From screening, the wastewater then passes into the aerated grit chamber with a rated capacity of 35 ML/d. The historical operating conditions for the grit chamber were reviewed and summarized in Table 4-1. At historical peak hourly flows of 30.2 ML/d, the hydraulic retention time (HRT) in the grit chambers is estimated to reach 6.1 minutes, which is greater than the 2 to 5 minutes recommended range by the Ministry of the Environment, Conservation and Parks (MECP) Design Guidelines (2008). The original design capacity of 35 ML/d results in an HRT of 5.26 min. The longer retention time will not degrade performance, hence the grit chambers are adequate for the existing capacity.

Table 4-1 Historical Operating Conditions of Aerated Grit Chamber (2017-2021)

Parameter	Historical Value	Typical Design Value[1]
Total Grit Chamber Volume	128 m^3	N/A
Rated Capacity	35 ML/d	N/A
Historical PHF	30.19 ML/d	N/A
Historical HRT at PHF	6.11 min	2 – 5 min
Original Design HRT	5.26 min	2 – 5 min
Note: (1) Based on MECP Design Guidelines for Sewage Works (2008).		

The grit chamber is also responsible for fat, oil, and grease (FOG) removal at the Canmore WWTP. The grit chamber achieves FOG removal through fine bubble diffusion, a process that attaches air to the FOG and then floats it to the surface so that grease pumps can transfer the FOG to sludge processing.

4.7 Chemical Injection Systems

Alum is currently added to the wastewater prior to the primary clarifiers. The injection of alum to the wastewater provides phosphorus removal, while enhancing the suspended solids removal within the primary clarifiers. Typically, the concentration of alum added to the system is between 20 – 180 mg/L, dependent on influent flow.

The current effluent regulation for phosphorus is 1 mg/L, and while the Canmore WWTP is currently meeting this regulation, if the regulation decreased to 0.5 mg/L, additional alum or a tertiary treatment process may be required to enhance removal rates.

4.8 Primary Clarifiers

The primary clarifiers each have a surface area of 218 m^2 and treat both the primary wastewater and excess filter backwash that cannot be treated by the dissolved air flotation (DAF) system. The role of these clarifiers is to remove a significant portion of both BOD and TSS from the wastewater through settling. Prior to the addition of side stream treatment for the BAF backwash, it the backwash was contributing more than 50% of the solids loading to the primary clarifiers. With the addition of the dissolved air flotation (DAF) for BAF backwash treatment, the solids loading to the primary clarifiers from the filter backwash has decreased, allowing the clarifiers to run at higher overflow rates. The effluent from the DAF can also be re-introduced to the system downstream of the clarifiers, further lessening the load on the clarifiers.

The historical operating conditions of the primary clarifiers between 2017 – 2021 are summarized in Table 4-2. The historical ADF and PDF surface overflow rate (SOR) are nearing the low end of typical MECP design guidelines for primary clarifiers not receiving waste activated sludge (WAS).

Table 4-2 Historical Operating Conditions of Primary Clarifiers (2017-2021)

Parameter	Historical Value	Typical Design Value[1]
Number of Clarifiers	2	N/A
Total Clarifier Surface Area	436 m^2	N/A
Historical Primary Influent ADF[2]	11.7 ML/D	N/A
Historical Primary Influent PDF[2]	26.1 ML/D	N/A
Historical SOR at ADF	26.9 $m^3/m^2/d$	30-40 $m^3/m^2/d$
Historical SOR at PDF	59.8 $m^3/m^2/d$	60-80 $m^3/m^2/d$

Parameter	Historical Value	Typical Design Value[1]
Note: (1) Based on MECP Design Guidelines for primary clarifiers not receiving WAS (2) Includes BAF Backwash Waste Recycle Flow		

The historical primary clarifier performance was reviewed and is summarized in Table 4-4. The primary clarifiers have produced a primary effluent of relatively good quality, with a BOD_5 and TSS removal rate of approximately 45% and 64%, respectively. In general, the performance of the primary clarifiers is between a conventional primary clarifier with no chemical addition and chemically enhanced primary treatment (CEPT).

Table 4-3 Historical Primary Clarifiers Performance (2017-2021)

Parameter	Average Concentration[1]		Removal Efficiency[1]	Typical Design Value[2]
	Influent	Effluent		
BOD_5	227 mg/L	125 mg/L	45%	Clarifiers with chemical addition: 45% to 85% BOD_5 removal 60% to 90% TSS removal Clarifiers without chemical addition: 35% to 65% BOD5 removal 40% to 70% TSS removal
TSS	201 mg/L	73 mg/L	64%	
Note: (1) Based on average historical concentration values of 2017 to 2021. (2) Based on MECP Design Guidelines for Sewage Works (2008).				

The existing primary clarifiers are limited by both ADF and PDF SOR, since the estimated values are higher than the historical average and peak flows of the plant (presented previously in Table 4-2). The historic operation of the primary clarifiers suggest that they are operating near the low end of the typical design value for both the SOR at ADF and PDF without co-thickening. This may be due to the influence of the recycle line from the DAF or the composition of the raw sewage that is not allowing the clarifiers to operate as efficiently as they could. To help alleviate some of this stress, another primary clarifier could be added to the WWTP.

The capacity of the primary clarifiers was evaluated based on an ADF SOR of 40 $m^3/m^2/d$ and a PDF SOR of 80 $m^3/m^2/d$ (MECP, 2008). The capacity assessment results are summarized in Table 4-3.

Table 4-4 Capacity Assessment for Primary Clarifiers

Parameter	Evaluation criteria[1]	Estimated Capacity
SOR at ADF	40 $m^3/m^2/d$	17.4 ML/d
SOR at PDF	80 $m^3/m^2/d$	34.9 ML/d
Note: (1) Based on MECP Design Guidelines for primary clarifiers not receiving WAS		

The clarifier inlet channel requires an isolation sluice gate valve to provide maintenance. This sluice gate can be installed as part of maintenance works or grouped into a capital upgrade.

4.9 Primary Effluent Screening

Prior to entering the BAFs, primary effluent passes through automatic arc screens, with 2 mm openings. The purpose of these screens is to remove any large objects that have made it through the clarifying process that could harm or negatively affect the biological aerated filters. The capacity of these screens is 46.0 ML/d, while providing 100% redundancy.

4.10 Biologically Activated Filters

Following the primary effluent screens, the wastewater enters the biologically activated filters. The Canmore WWTP has a two stage BAF system comprised of five (5) C – side filters, along with five (5) N – side filters. The C – side filters remove suspended solids and BOD from the water, while the N – side filters remove ammonia – nitrogen from the wastewater. The two-stage system provides reliable treatment for BOD and TSS, while any carryover of carbon inhibits the N stage process for nitrification.

All ten (10) filters are the same within the BAF system; each with a surface area of 40 m^2 and a media depth of 2.5 metres. The wastewater first enters the C – side filters through the influent channels, where it flows up through the media and then overflows to the effluent channels and is then transferred to the intermediate transfer tank. The wastewater is then pumped to the N – side filters where the process is repeated, and the effluent from the N – side filters flows to the UV reactors for disinfection.

The BAF system is not equipped with an anoxic zone and any requirement to denitrify will require downstream biological processes with carbon addition or the addition of a pre-denitrification reactor.

The primary clarifiers are precipitating phosphorus upstream of the BAF. There must be a residual P content for the biological system in the BAF to function properly. This limits the effluent TP concentration that can be reliably achieved without causing system upsets. In general, the BAF can achieve 0.5 to 0.7

mg P/L for periods, however, sustained operation below 0.5 mg P/L is not practical. It is anticipated increased flows will require the effluent phosphorus concentration to be decreased to maintain a similar loading to the river. Downstream tertiary filtration is likely required to achieve the lower effluent limits irrespective of the BAFs biological capacity.

4.10.1 General Characteristics and Main Equipment

As wastewater is filtered and treated through the BAF system, suspended solids and organic material builds up in the filters. To counteract this, the C – side BAF cells are backwashed every 14 – 24 hours, while the N – side BAF cells are backwashed every 24 – 48 hours. Performing backwashes helps remove built up organic material and helps to maintain optimal filter performance.

Losing filter media during backwashing can be a problem some WWTPs encounter. The Canmore WWTP has a media screen in each BAF cell, reducing the volume of media lost, as losing media decreases BAF performance, thus decreasing effluent quality. The media loss per year is less than 3% due to the media screens.

The equipment that is required for the operation of the BAF include the intermediate well, blowers, a backwash well, backwash blowers, and backwash waste pumps. The following subsections will discuss each of these components.

4.10.2 Design Loading and Overall Performance

The historical operating conditions for the BAF system are summarized in Table 4-10.

Table 4-5 Historical Operating Conditions of the BAF System (2017-2021)

Parameter	Historical Value	Typical Design Value
C-side BAF Filters		
Number of Filters	5	
Surface Area per Filter	40 m^2	
Media Volume Per Filter	100 m^3	
Historical Hydraulic Loading Rate at MinF	2.0 $m^3/m^2/h$	2 – 10 $m^3/m^2/h$ (1) (Average = 6 $m^3/m^2/h$)
Historical Hydraulic Loading Rate at ADF	4.4 $m^3/m^2/h$	
Historical BOD_5 Loading Rate at ADF	5.1 kg/m^3	3 – 7 kg/m^3 (2) (Average = 5 kg/m^3)
Historical BOD_5 Loading Rate at MMF	10.7 kg/m^3	
Historical TSS Loading Rate at ADF	2.9 kg/m^3	3 – 7 kg/m^3 (2)

Parameter	Historical Value	Typical Design Value
Historical TSS Loading Rate at MMF	7.6 kg/m^3	(Average = 5 kg/m^3)
N-side BAF Filters		
Number of Filters	5	
Surface Area per Filter	40 m^2	
Media Volume Per Filter	100 m^3	N/A
Historical Hydraulic Loading Rate at MinF	2.0 $m^3/m^2/h$	2 – 10 $m^3/m^2/h$ (1) (Average = 6 $m^3/m^2/h$)
Historical Hydraulic Loading Rate at ADF	4.4 $m^3/m^2/h$	
Historical TKN Loading Rate at ADF	0.8 kg/m^3	0.5 – 2 kg/m^3 (2) (Average = 1.25 kg/m^3)
Historical TKN Loading Rate at MMF	1.3 kg/m^3	

Note:

(1) Based on BAF Operation and Maintenance Manual (Degremont Technologies, 2008)
(2) Based on Degremont Technologies recommendations (Stantec, 2012)

The capacity of the BAF system was evaluated based on both the hydraulic loading rate (HLR) and the organic loading rate (OLR). These capacities were evaluated at the average HLR and OLR values recommended by the vender of the technology, Degremont Technologies, for C- and N-side BAF filters. The results are summarized in Table 4-11.

Table 4-6 Capacity Assessment of BAF Systems

Parameter	Evaluation Criteria (1,2)	Estimated Capacity
C-side BAF Filters		
Average HLR	6 $m^3/m^2/h$	28.8 ML/d
Average OLR	5 kg/m^3	20.1 ML/d
N-side BAF Filters		
Average HLR	6 $m^3/m^2/h$	28.8 ML/d

Parameter	Evaluation Criteria [1,2]	Estimated Capacity
Average OLR	1.25 kg/m^3	32.7 ML/d
Note: (1) Based on BAF Operation and Maintenance Manual (Degremont Technologies, 2008) (2) Based on Degremont Technologies recommendations (Stantec, 2012)		

The average estimated HLR capacity of the plant is higher than BAF historical combined influent ADF, 20.9 ML/d (as presented in Table 4-6). As for the OLR capacity, the estimated C-side BAF filters capacity is lower than historical ADF capacity of the BAF, while higher loading capacity for N-side BAF filters is observed. The BAF filters are therefore limited by the C-BAF average organic loading rate, which would gradually affect the overall treatment performance of the BAFs. In fact, with the population rise and the expected increase of organic loadings to the BAF, the C-side BAF would have less available capacity and thus, the untreated organic matter would pass into the N-side BAF. This could result in lower nitrification efficiency of the N-BAF and therefore, producing a higher final effluent TAN concentration.

The operations staff at the Canmore WWTP has also been experiencing issues with the BAFs during the backwash stage of the filters. Periodically during backwashing, media from the BAFs can be propelled into the air and land outside of the BAF units. This is coupled with the aeration of the BAFs being uneven in some of the tanks, leading to possible short circuiting within the BAFs. With media being propelled into the air during backwash, and uneven aeration, it is possible the BAFs are experiencing consolidation and clogging of the media. The BAF O&M manual suggests that a probable cause due to uneven process air distribution with violent local bubbling could be due to a leak in the aeration system. The course of action recommended in the manual is to empty the media from the cell and change any defective parts as required. Several of the C-Side filters were emptied and replaced within the last 5 years and the problem is persisting.

4.10.3 BAF Influent and Effluent

The BAFs is the heart of the process of the Canmore WWTP as they remove large portions of the BOD, TSS, and ammonia – nitrogen that remain in the wastewater after primary clarification. As the BAFs play a critical role in the removal of these wastewater parameters, they are important for the effluent water characteristics and ensuring that the effluent wastewater regulations are met. This section will discuss both the BAF influent and BAF effluent wastewater characteristics.

The wastewater that enters the BAF consists of primary clarified water, water from the backwash waste tank, a portion of the effluent wastewater from the DAF, and a BAF recycle line. The historical BAF combined influent flows and loadings are summarized in Table 4-6.

Table 4-7 Historical BAF Combined Influent Flows and Loadings (2017-2021)

Parameter	Historical Value
Flows (ML/d)	
ADF	20.9
MMF	28.7
PDF	42.5
PHF	46.1
MinF	9.4
Loads (kg/d)	
BOD_5 Loads	
Average Annual	2,545
Max Month	5,360
TSS Loads	
Average Annual	1,496
Max Month	3,816
TAN Loads	
Average Annual	398
Max Month	663
TKN Loads	
Average Annual	594
Max Month	989

The loadings are calculated based on the flows and the BAF influent concentration measured before C-side BAF filters. Influent TSS and COD concentrations to the BAFs are measured daily, BOD and TP are measured several days a week, while TAN influent concentrations are measured weekly. These influent concentrations, as well as final effluent concentrations and effluent limits are summarized in Table 4-7.

Table 4-8 Historical BAF System Performance (2017-2021)

Parameter	Average Concentration[1]		Removal Efficiency[1]	Effluent Limits[2]
	Influent	Effluent		
BOD_5	125 mg/L	8.2 mg/L	94%	20 mg/L
TSS	73 mg/L	6.9 mg/L	90%	20 mg/L
TP	2.7 mg/L	0.5 mg/L	81%	1 mg/L
TAN	14 mg/L	4.3 mg/L	69%	< 5/< 10 mg/L (Jul – Sept / Oct - Jun)

Note:

(1) Based on average historical concentration values of 2017 to 2021.
(2) Canmore WWTP Regulatory Effluent Limits

Based on this historical performance evaluation, BAF systems have produced a good quality effluent over the last five years. As shown previously in Section 2.5 of this report, BOD_5, TSS, and TAN average concentration effluent levels have increased slightly during the study period.

Total ammonia nitrogen removal is an important characteristic to consider especially at max month loading scenarios, both in the summer and winter. The loading rates between these two seasons illustrate the range in flow and wastewater characteristics that the Canmore WWTP receives. During the summer flows are high, peaking in June, and most parameters have their highest influent loading rate to the plant in August. During the winter (the months of January, February, March, and December) the water is cooler, the flow to the plant is less on average, while the influent concentration typically does not vary too significantly.

To evaluate the BAFs and their effectiveness, the removal rates of TAN through the BAFs at these max month loading scenarios should be investigated. The following tables highlight the average mass of TAN removed from the BAFs, the average water temperature, average effluent TAN that was discharged from the plant during these max month scenarios, along with the actual and reference removal rates.

Table 4-9: Max Month TAN / TKN Removal

	TAN Removal[1] (kg)	TKN Removal[1] (kg)	Actual Removal Rate[1] (TAN) (kg/m^3/d)	Actual Removal Rate[1] (TKN) (kg/m^3/d)	Temperature[1] (°C)	Reference Removal Rate[1] (TAN) (kg/m^3/d)	Reference Removal Rate[1] (TKN) (kg/m^3/d)	Effluent TAN[1] (mg/L)	Effluent TKN[1] (mg/L)
2017	49.97	74.59	0.10	0.15	15.89	0.14	0.20	14.83	22.13
2018	162.55	242.62	0.33	0.49	15.26	0.46	0.69	0.95	1.42
2019	174.34	260.21	0.35	0.52	14.64	0.52	0.77	2.45	3.65
2020	112.78	168.33	0.23	0.34	9.93	0.47	0.70	5.82	8.69
2021	131.22	195.84	0.26	0.39	15.39	0.37	0.55	6.34	9.46
Average	126.17	188.32	0.25	0.38	14.23	0.39	0.58	6.08	9.07

Note:

(1) **Based on Canmore WWTP data from 2017 to 2021**
(2) **TAN/TKN assumed to be 0.67**

Table 4-10: Winter Max Month TAN / TKN Removal

	TAN Removal[1] (kg)	TKN Removal[1] (kg)	Actual Removal Rate[1] (TAN) (kg/m³/d)	Actual Removal Rate[1] (TKN) (kg/m³/d)	Temperature[1] (°C)	Reference Removal Rate[1] (TAN) (kg/m³/d)	Reference Removal Rate[1] (TKN) (kg/m³/d)	Effluent TAN[1] (mg/L)	Effluent TKN[1] (mg/L)
2017	N/A[2]	N/A[2]	N/A[2]	N/A[2]	9.24	N/A[2]	N/A[2]	11.17	16.67
2018	95.68	142.80	0.19	0.29	10.33	0.39	0.58	3.24	4.84
2019	118.05	176.19	0.24	0.35	10.13	0.49	0.73	3.82	5.71
2020	112.78	168.33	0.23	0.34	9.93	0.47	0.70	5.82	8.69
2021	118.06	176.20	0.24	0.35	9.92	0.49	0.74	3.51	5.24
Average	111.14	165.88	0.22	0.33	9.91	0.46	0.69	5.51	8.23

Notes:

(3) Based on Canmore WWTP data from 2017 to 2021

(4) **Values were not recorded in historical data for Winter Max Month Loading**

From the tables above, the Actual Removal rate was calculated from the mass of TAN removed, divided by the volume of media within the biological aerated filters. The reference removal rates were normalized to 20°C to compare the influence of max month loading and winter loading, the two worst case scenarios for the biological treatment.

The max month loading included the adverse event in 2017, which skewed the removal rates lower. Removing this event and using the more typical operational conditions, the temperature corrected removals were equivalent at 0.46 kg TAN/m^3/d or 0.69 kg TKN/m^3/d. In both scenarios, the effluent ammonia concentrations were above 3 mg/L indicating the BAF units were operating near their maximum kinetics. Additional data for dissolved oxygen concentration would confirm this, however, for the purpose of operations the observed ammonia removal rate is 0.46 kg TAN/m^3/d at 15°C during the worst-case scenario operations.

4.10.4 Intermediate Well

The intermediate well is 305 m^3 with three submersible intermediate transfer pumps, two of them sized for 128 L/s and one rated at 111 L/s. The function of these pumps is to transfer C – side BAF effluent to the N – side BAF filter influent via the intermediate tank. These intermediate pumps share a standby unit with the backwash pumps (the one with 111 L/s rated capacity) and are all equipped with VFDs.

The intermediate pumps have a combined firm capacity of 22.1 ML/d. This firm capacity is currently adequate to cope with the average day flow and peak day flows expected at the Canmore WWTP. Depending on the anticipated future flows to the plant, the pumps may need to be replaced such that they have a larger firm capacity to deal with the rise in flow to the plant.

4.10.5 Aeration Blowers

There are ten (10) blowers provide oxygen to the BAF cells. The blowers utilised at the Canmore WWTP are 18.7 kW positive displacement blowers each with a rated capacity of 440 m^3/h, providing a total capacity of 4,400 m^3/d for the BAF system.

The capacity of the aeration system was assessed based on the total capacity of the blowers and an oxygen transfer efficiency (OTE) of 20%, given by the vender in the Operation and Maintenance Manual (Degremont, 2008). Average BAF influent BOD_5 and TKN concentrations were used to calculate the oxygen demand. Then a historical peak diurnal and month loading factor of 1.1 and 2.1, respectively, were applied to the calculated oxygen demand to obtain the maximum (actual) oxygen transfer required (AOTR). The aeration system capacity assessment is summarized in Table 4-5.

Table 4-11 Capacity Assessment of BAF Systems

Parameter	Estimated Capacity	Description
Oxygen Demand	220 kgO_2/h	Systems with nitrification: 1 kg O_2/kg BOD_5+4.6 kg O_2/kg TKN[1]
AOTR	503 kgO_2/h	Oxygen demand x Peak Factors

Parameter	Estimated Capacity	Description
SOTR	1,599 kgO_2/h	Standard Oxygen Transfer Required
OTE	20%[2]	
Air flow required	2,206 m^3 air/h	SOTR x OTE

Note:

(1) Based on MECP Design Guidelines for Sewage Works (2008).
(2) Based on BAF Operation and Maintenance Manual (Degremont Technologies, 2008).

The required air flow estimated is 2,206 m^3air/h, while the blower system has a total rated capacity of 4,400 m^3/h. The existing blower system therefore has adequate capacity to meet the required oxygen demand.

4.10.6 Backwash Well

The Canmore WWTP backwashes one filter at a time, such that the plant can still operate effectively when backwashing is taking place. Within the intermediate transfer tank, there are three backwash pumps, each rated at 111 L/s. One of the backwash pumps is a standby pump that can aid either the backwash pumps or the intermediate transfer pumps as needed. Along with the pumps, the backwash blowers also aid in removing built up organic material from the BAFs.

4.10.7 Backwash Blowers

The backwash blowers aid the backwash pumps in removing organic material from the BAF cells when needed. There are three blowers, each is 45 kW, and they provide air scour during the BAF backwash cycle.

4.10.8 Backwash Waste Pumps

The water that is used during backwashing goes to the backwash waste tank (460 m3) for holding prior to either side stream treatment, DAF, or re-introduction to the system at the head of the plant. The water is pumped to either the DAF or to headworks via three 36 L/s submersible backwash waste pumps. Two of these pumps are used to pump backwash waste to the DAF unit which have a total rated capacity of 6.2 ML/d. The rated capacity of the P-313 backwash return pump is 3.1 ML/d

4.10.9 BAF Recirculation Pumps

To maintain adequate flow to the BAFs, two 200 m3/hr vertical turbine pumps are installed in a wet well downstream of the UV disinfection channels, which provide a hydraulic loading rate of 4 m/hr via a recycle line to the filters. These pumps are designed to operate at variable speeds, and can provide flows between 75 m3/hr to 400 m3/hr.

4.10.10 Treated Effluent Measurement

A 600 mm wide fiberglass Parshall flume is utilised to measure BAF effluent flows. The capacity of the flume is 35.0 ML/d. To increase the future capacity of BAF effluent flow monitoring, an additional Parshall flume or alternative sensor (laser) that reduces hydraulic bottlenecks can be considered.

4.11 DAF System

The DAF system utilizes air to remove suspended matter from the surface of treated water. Dissolving air under pressure (whitewater) is introduced to the influent of the DAF tank along with BAF backwash water. The small air bubbles in the whitewater attach to suspended matter and float to the surface of the tank, where skimmers remove the solids. To encourage clustering of solids and promote removal, coagulant is typically added to the influent water. The treated water is then removed as the effluent, while the sludge that is skimmed from the surface is treated further.

Currently at Canmore WWTP, one DAF tank (AquaDAF) with a total surface area of 7.7 m2 is in operation. According to the vendor, AquaDAF can treat a high loading rate range of 24 to 48 m3/m2/h (AquaDAF* Brochure, Suez). Based on general experiences with these systems, a lower operational loading rate is anticipated, and with the addition of polymer to the DAF, the historic HLR of 9.4 m3/m2/h (Table 4-12) is within the anticipated range. The historical operating conditions for the DAF tank are summarized in Table 4-12. The operational schedule of DAF was assumed to be 24 hours per day and 7 days per week.

Table 4-12 Historical Operating Conditions of DAF (2017-2021)

Parameter	Historical Value	Typical Design Value(1)
Number of DAF tanks	1	N/A
Total Surface Area	7.7 m^2	N/A
Solid Capture	90%(2)	N/A
Historical Annual Average Waste Secondary Sludge (WSS) Feed	1,826 kg/d	N/A
Historical Max Month Waste Secondary Sludge (WSS) Feed	2,221 kg/d	N/A
Historical Average DAF Feed Flow	1.72 ML/D	N/A
Historical Average Solid Loading Rate (SLR)	9.9 $kg/m^2/h$(3)	10 $kg/m^2/h$
Historical Average Hydraulic Loading Rate (HLR)	9.4 $m^3/m^2/h$(4)	6.3 $m^3/m^2/h$

Parameter	Historical Value	Typical Design Value$^{(1)}$
Note: (1) Based on MECP Design Guidelines for Sewage works (2008). (2) **Based on AquaDAF* Brochure (Suez).** (3) Based on historical average waste sludge generated in the BAF backwash waste tank (1,826 kg/d). (4) Based on historical influent flows to the DAF (1.72 ML/d), recorded by the flow meters F/T 504 and F/T 514.		

The DAF tank has operated at SLR value less than the typical design values recommended by the MECP (2008). Though its historical HLR seems to be higher than MECP recommended typical design value (6.3 m3/m2/h), the original design HLR of AquaDAF* is anticipated to be at least 10 m3/m2/h. Hence, the DAF tank could possibly cope with historical hydraulic loading rates with polymer addition.

The capacity of the DAF tank was evaluated based on the MECP recommended SLR and HLR values and is summarized in Table 4-13.

Table 4-13 Capacity Assessment of DAF Tank

Parameter	Evaluation Criteria $^{(1)}$	Estimated Capacity
DAF Solid Loading Capacity at Average SLR	10 kg/m^3	1,845 kg/d $^{(2)}$
DAF Hydraulic Loading Capacity at Average HLR	6.3 m^3/m^2/h	1.15 ML/d
Note: (1) Based on MECP Design Guidelines for Sewage works (2008). (2) Solid Loading Capacity of the DAF.		

Based on the assessment results, it could be concluded that the solid loading capacity of the DAF tank is almost equal to the historical generated WSS fed to the DAF (1,826 kg/d). As for the hydraulic loading capacity, this unit has reached its capacity when evaluated with the considered MECP values. Stress-testing is recommended to validate the capacity assessment.

4.12 UV Disinfection

The existing ultraviolet (UV) disinfection system is housed in a separate room. There are three hydraulic channels in this room. Two channels are equipped with the banks of Trojan 3000Plus (42 lamps each) installed around the year 2000. The third channel is a bypass channel suitable for the installation of the third UV. Each of the two UV channels has a treatment capacity of 17.5 ML/d at UV transmittance of 65%. The total peak flow capacity of the installed UV system is 35 ML/d which is greater than current plant's rated peak hourly flowrate (30.2 ML/d).

Each of the two UV channels is equipped with a recirculation pump. The recirculation pumps ensure that the channels have enough water for UV lamps submergence.

The existing two ultraviolet reactors have a potential point of failure (i.e. the existing controller is operational but it does not have replacement parts due to its age). The gate valves within the existing UV channels do not allow the level control and require to run the recirculation system during low flow periods.

Additionally, the WWTP will eventually require the third UV reactor to accommodate the future flows.

The existing controller is obsolete. In the event of failure only a custom-built controller can be used for replacement. The new controllers that Trojan currently manufactures are not compatible with the existing UV system. The custom-built controller for replacement of the existing will cost around 100k (which is comparable with the cost of entire new UV bank) and would be a “throw away” cost during future addition of the third UV.

The proposed path forward is to install the new UV with automatic level control gates instead of the two existing UV banks. The new controller will be suitable to run up to three UVs and when the third UV is added in the future, it can be connected to the new controller.

4.13 Outfall

The Bow River is the receiving body of water for the effluent from the Canmore WWTP. The effluent is discharged to the river through a 660 mm outfall. A field inspection is recommended in Spring 2023 during favorable conditions.

4.14 Solids Processing

The main solids produced at the Canmore WWTP are from the clarifiers and the BAF backwash. The clarifiers produce co-thickened primary solids, while the BAF backwash has solids that are from the media that filters the wastewater. The main solids come from the primary clarifiers, however there is some FOG contribution from the aerated grit chambers.

The plant currently has capacity to treat the generated solids, however with an increase in loading to the plant expected with a rise in population, the solid processing of the plant must be evaluated to ensure adequate future capacity.

4.14.1 Primary Sludge Pumping

Primary sludge pumping occurs in two locations at the Canmore WWTP. There are sludge and scum pumps in the primary clarifiers, along with the grease pumps in the grit chamber. All these pumps are rated at 11 L/s, having a firm capacity of 985 m^3/d when one pump is out of service. The primary sludge flows were not being recorded historically. The ADF and PDF of the primary sludge leaving the primary clarifier is therefore estimated by conducting a mass balance around the clarifier. The estimated historical and projected sludge flows are summarized in Table 4-14.

Table 4-14 Historical and Projected Primary Sludge Flows and Concentrations

Parameter	Historical values[1]
Primary Sludge Flows	
ADF	55 m^3/d
PDF	124 m^3/d
Primary Sludge Loads	
Average	1,382 kg/d
Maximum Day	3,093 kg/d
Average Concentration	25000 mgTSS/L[(2)]

Note:

(1) Mass Balance estimated values based on historical ADF and PDF of the primary clarifier.
(2) **Based on 2.5% primary sludge concentration assumption.**

The primary sludge pumps have a greater firm capacity than the maximum daily primary sludge produced. These pumps have therefore adequate capacity for the historical condition of the WWTP.

4.14.2 Sludge Holding Tanks

The aerobic digesters at the Canmore WWTP act more as open-air holding tanks rather than digesters. The primary sludge and DAF Sludge are sent to two digesters for stabilization. Each of the digesters have a volume of 1,155 m^3. The historical operating conditions of the digesters are summarized in Table 4-15.

Table 4-15 Historical Operating Conditions of Digesters (2017-2021)

Parameter	Historical Value [(1)]
Number of Digesters	2
Total Volume Available	2,310 m^3

Parameter	Historical Value (1)
Annual Average Sludge Flows to the Digesters	84.9 m^3/d (1)
Max Month Sludge Flows to the Digesters	86.5 m^3/d (1)
Annual Average Sludge Mass to the Digesters	3,026 kg/d (2)
Max Month Sludge Mass to the Digesters	3,680 kg/d (2)

Note:

(1) Historical combined DAF and primary clarifiers sludge flows.
(2) Historical DAF sludge loadings + primary sludge loadings (values presented respectively in ***Table 4-12*** and ***Table 4-14***)

The historical maximum monthly total sludge flow to the digester tanks is approximately 86.5 m^3/d which provides 26.7 d of retention. The tanks are aerated to reduce odors and partially digest the solids. The solids are centrifuged and hauled to compost; hence full stabilization is not necessary.

4.14.3 Digested Sludge Pumps

To pump the digested sludge from the open-air holding tanks there are two pumps each with a capacity of 4.1 L/s. These pumps transfer sludge from the open-air holding tanks to the centrifuge such that the centrifuge has sufficient influent flow when it is operated.

4.14.4 Dewatering

The Canmore WWTP currently dewaters using a centrifuge; the capacity of the centrifuge is 27.2 L/s. The centrifuge can receive sludge with up to 4% solids and produce sludge that is between 25% – 35% solids. The centrifuge works by separating the solids from the water through centrifugal forces. Through the introduction of polymer to the influent wastewater, the centrifuge thickens the solids to allow for more efficient and effective solids separation. Centrifuging lowers the overall odor of dewatering as it is an enclosed process, while also maintaining a small footprint. The existing centrifuges operate 8 hours per day for 5 days per week. The historical operating conditions of the centrifuges are summarized in Table 4-16.

Table 4-16 Historical Operating Conditions of Centrifuges (2017-2021)

Parameter	Historical Value
Number of Centrifuges	2
Operation hours per day	8 hrs/5 days

Parameter	Historical Value
Feed Sludge TS Concentration	1.80% (1)
Flow Capacity per Centrifuge	29 m^3/h (1)
Operational Flow Capacity per centrifuge	7.02 m^3/h
Max Month Sludge Mass to the Centrifuges	3,680 kg/d (2)
Max Month Sludge Flows to the Centrifuges	204 m^3/d (2)

Note:

(1) **Centrifuges Operation and Maintenance Manual**
(2) Historical DAF sludge loadings + primary sludge loadings (values presented respectively in ***Table 4-12*** and ***Table 4-14***)

The solid loading rate of the centrifuges is 530 kg/h or 12,720 m^3/d (see Table 3-1). These dewatering units thus have sufficient capacity to receive the historical peak sludge loadings (3,680 kg/d) pumped from the digesters.

The cake that is produced from the centrifuge is loaded onto a truck and then shipped to a plant for further composting. Since the centrifuge can significantly enhance the solids content of the sludge, the number of trucks required for transportation is minimal, thus lowering transportation costs and limiting the truck traffic on the roads of the Town of Canmore.

4.14.5 Dewatering pumps

The centrifuge centrate is recycled to the influent well, using two pumps each with a rated capacity of 13.6 L/s.

4.15 Existing Capacity Assessment Summary

The assessed historical capacity of the major plant unit processes is shown Figure 4-1. The chart is colour coded based on the capacity limiting condition for each unit process as follows:

+ Unit Processes limited by peak hourly flows are shown in orange.
+ Process limited by primary clarifier average daily flows is shown in purple.
+ Process limited by primary clarifier peak daily flows is shown in green.
+ Processes limited by average day BAF Influent flows/loadings are shown in blue.

Based on the capacity analysis, the plant is already at or near capacity, with the bottleneck currently being the BAF process unit. The BAFs are currently near their capacity to handle the organic loading rate entering the facility.

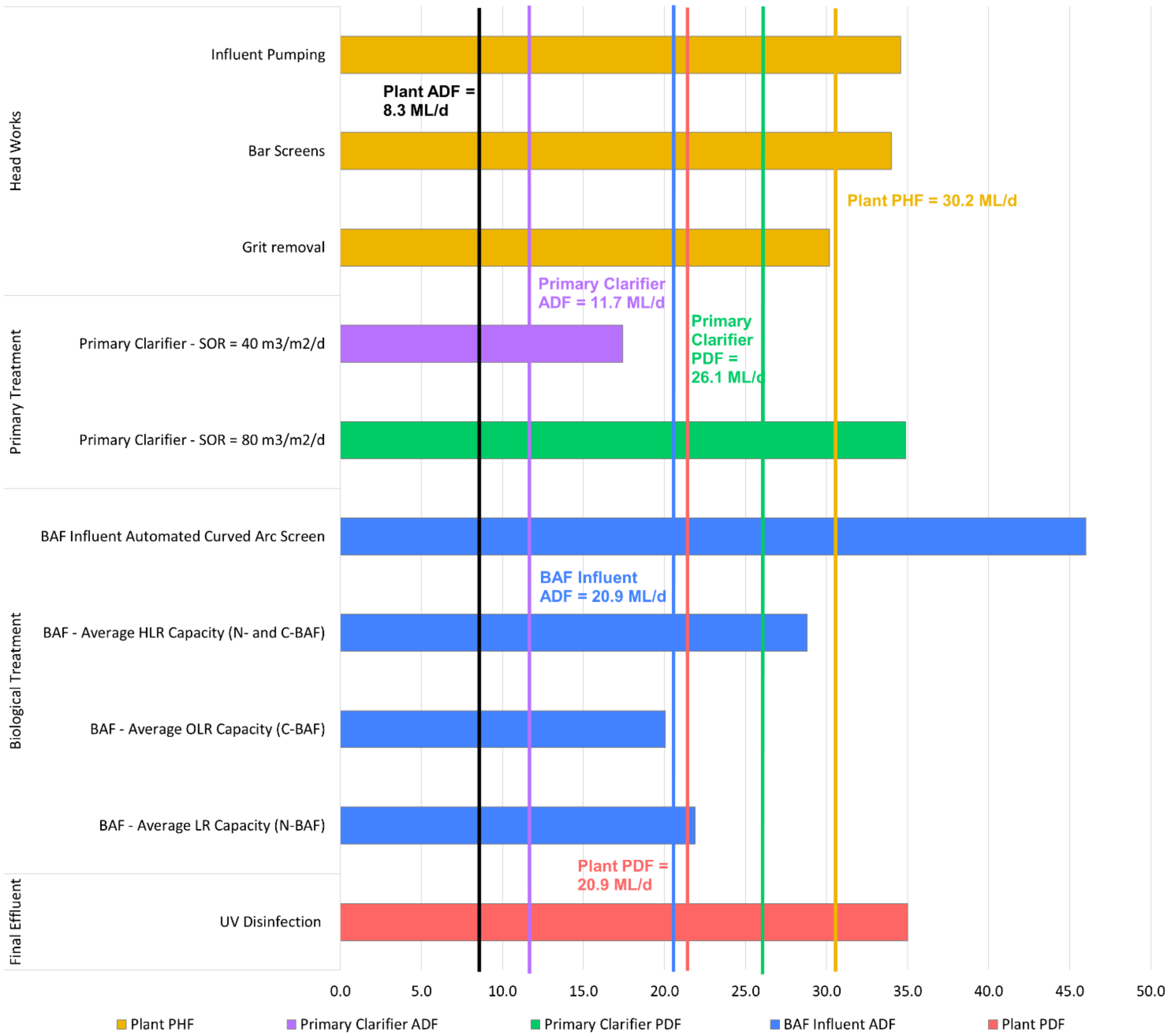

Figure 4-1: Canmore WWTP Process Capacity Assessment Summary

5 Design Basis and Future Projections

5.1 Future Loading Rates

Historic loading rates were used to project the future loading to the WWTP. Where historic rates were above or below the typical range, the future per capita loading rate was adjusted to within the range. Historic loading rates for each of the major wastewater parameters are listed in the following table, along with the assumed per capita loading rate that was considered throughout the future projection calculations.

Table 5-1: Per Capita Loading rates

Constituent Parameter	Average Per Capita Load (g/p/d)	Assumed Future Per Capita Loading (g/p/d)
BOD	88.3	90
TSS	79.2	85
TAN	8.16	8.50
TP	1.73	1.80

5.2 Future Design Basis

The design basis to evaluate the WWTP at 5, 15, and 25 year intervals is shown in Table 5-2. The max month loadings were projected using the historical max month loading factors and applied to the increased loads.

Table 5-2: Future Design Basis

Component	5 Year	15 Year	25 Year
Design Population			
Permanent Population	20,982	25,308	27,758
Non-Permanent Population	5,820	10,462	16,982
Total	26,802	35,770	44,740
Influent Characteristics			
Flows			
ADF, ML/d	11.32	15.22	19.12
MMF, ML/d	22.02	29.61	37.20
PDF, ML/d	28.46	38.26	48.07
PHF, ML/d	41.16	55.35	69.53
Loads			
BOD			

Average, kg/d	2,378	3,185	3,993
Maximum Month, kg/d	2,892	3,873	4,855
TSS			
Average, kg/d	2,163	2,925	3,688
Maximum Month, kg/d	2,846	3,849	4,852
TAN			
Average, kg/d	221.1	297.3	373.6
Maximum Month, kg/d	266.1	357.9	449.7
TP			
Average, kg/d	46.8	63.0	79.1
Maximum Month, kg/d	59.4	79.9	100.4

5.3 Future Projections

From Sections 5.1 and 5.2, a capacity assessment of the Canmore WWTP can be accomplished for the future 5, 15, and 25 year marks.

As shown in Figure 5-1, within 5 years many WWTP unit processes will have insufficient capacity, including influent pumping, screening, primary clarifiers (at PDF), BAF, and UV disinfection.

At the 15-year mark (Figure 5-2), most of the plant will exceed capacity and struggle to provide adequate effluent to the Bow River. The only unit processes that will have capacity at the projected flows will be the grit removal and the arc screens before the BAFs.

The 25-year capacity assessment (Figure 5-3) shows that the only unit process that would have capacity is the grit removal system.

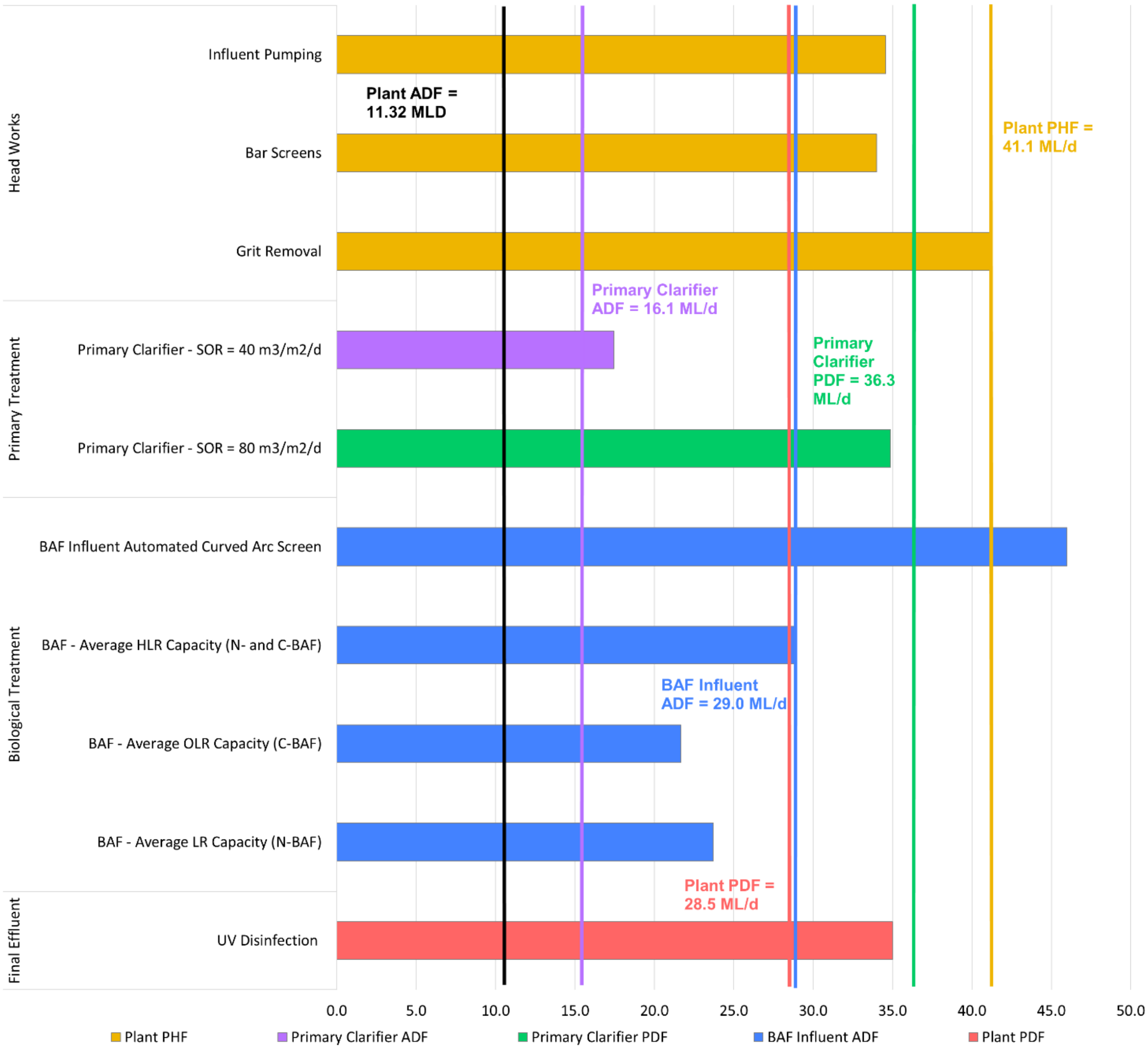

Figure 5-1: 5 Year Canmore WWTP Process Capacity Assessment

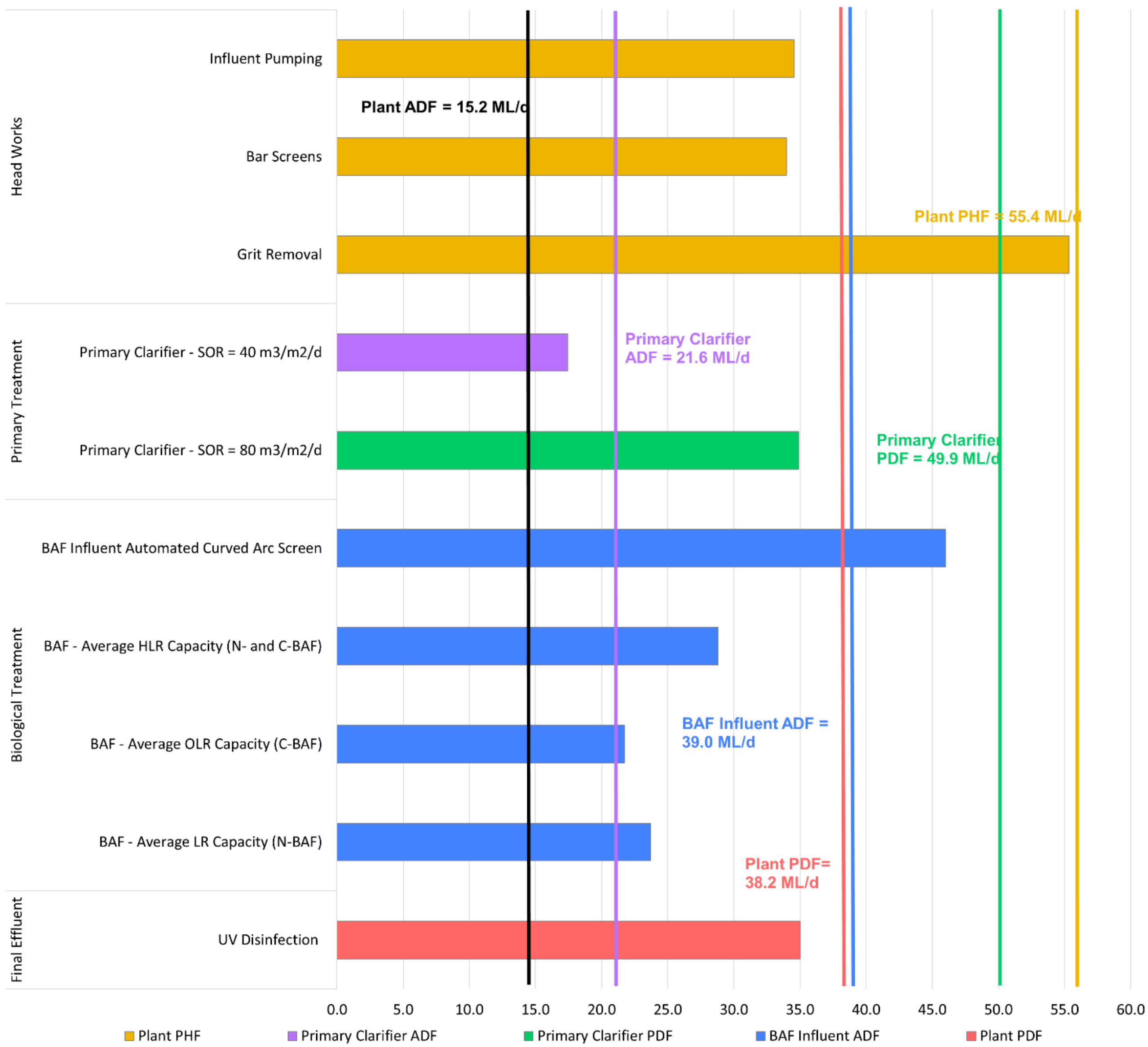

Figure 5-2: 15 Year Canmore WWTP Process Capacity Assessment

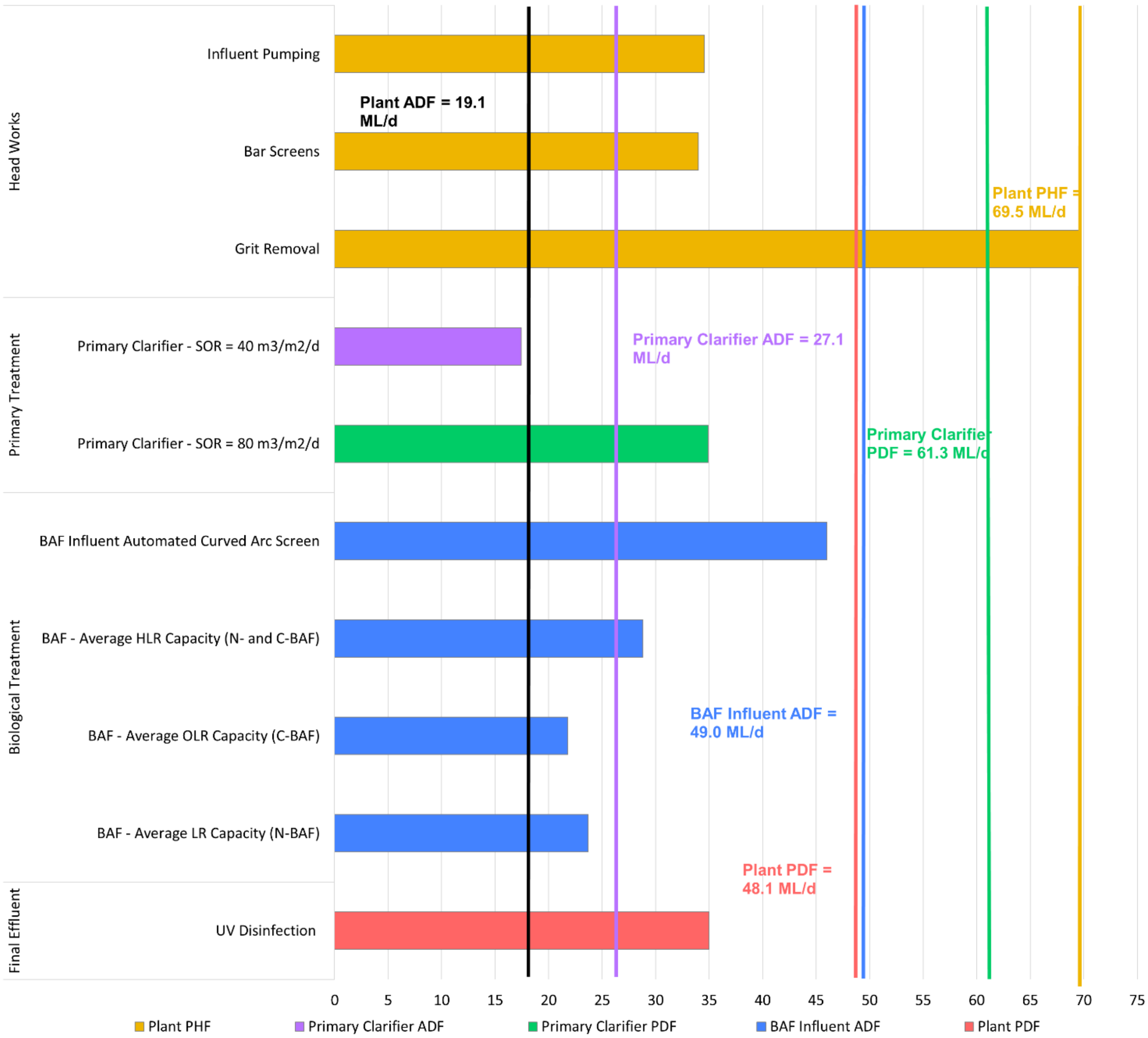

Figure 5-3: 25 Year Canmore WWTP Process Capacity Assessment

6 BioWin Modelling

A Biowin model was developed based on the existing plant process flow, sizing, historical flows (influent and recycle) and loadings. The BioWin model is most appropriate to evaluate the biological processes, which in this case in the BAF system for BOD and TAN removal. The model was partially calibrated using kinetic values from full scale two stage BAF cells on the historical performance during critical periods during the year. To fully calibrate a BioWin model for the plant, extensive sampling over extended periods would be required. This sampling program would require atypical parameters for the plant to identify influent fractionations. EPCOR provided a TKN sample to confirm the TAN/TKN fractionation was within typical ranges for municipal wastewater. The additional excess sampling was not available or required for the purpose of the modelling as a check for the capacity assessment evaluation.

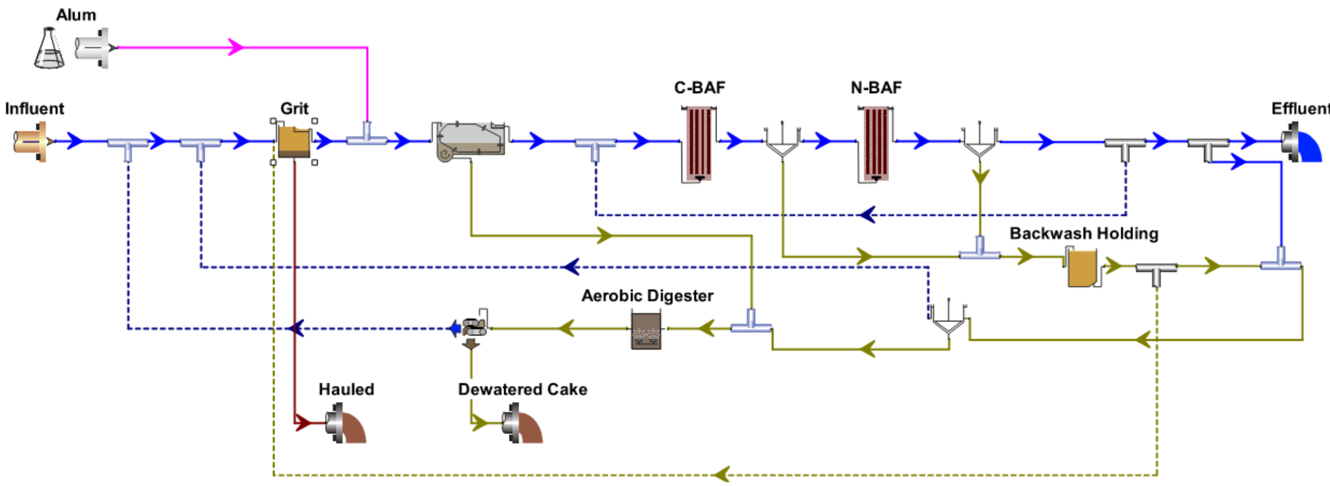

Figure 6-1: Canmore WWTP BioWin Model Process Schematic

6.1 Model Scenario Analysis

The model was calibrated on evaluated on three scenarios, winter, max month loading and max month hydraulic. The scenarios were selected as they represent the most challenging conditions given the influences at the Canmore WWTP from snow melt, tourist seasons and temperature impacts. They also represent the regulatory framework for compliance on a monthly average.

The max month hydraulic loading historically had the same mass loading as the winter scenario and occurs during the same effluent limit period. Hence, the winter case was carried forward for the future projections analysis. In this report, the effluent data presented is the ammonia concentrations. The soluble BOD removal will not be limiting in the BAF systems with the two stages and the particulate material removal is dependent on solids retention capacity/hydraulic capacity of the filters. The effluent ammonia concentrations are the limiting biological component and what is most reliable within the modeling software. Predicted effluent concentrations for all parameters is in Appendix A.

6.1.1 Winter Scenario

The winter months are present challenges for the biological process due to lower temperatures and comparable loadings to the annual average. The flowrates and loadings were pro-rated from the annual average to reflect the actual conditions present during the low temperature period. The model inputs are presented in (Table 6-1).

Table 6-1 Winter Condition

Design Parameter	Historical	5-Year	15-Year	25-Year
ADF, ML/d	6.64	9.06	12.18	15.29

Temperature, °C	8.0	8.0	8.0	8.0
BOD Loading, kg/d	1,754	2,378	3,185	3,993
TSS Loading, kg/d	1,574	2,163	2.925	3,687
TAN Loading, kg/d	162	221	297	374
TP Loading, kg/d	34.4	46.8	63.0	79

The partially calibrated effluent TAN well predicted the winter condition using the historical data. The future projections were applied to the winter condition and outputs plotted in Figure 6-2. It can be observed the effluent is projected to be at the existing limit in the 5-year horizon and exceed in the 15-year horizon. The partially calibrated model does not consider operation optimizations that may be performed (i.e. recirculation adjustment, alkalinity addition, etc.). Adjusting the kinetic rates to the theoretical maximum provides additional capacity in the five (5) year horizon (Figure 6-2). Hence, optimization efforts may gain some additional capacity, however, the trend aligns with the capacity assessment and overall plant being overloaded between the 5 and 10 year horizon.

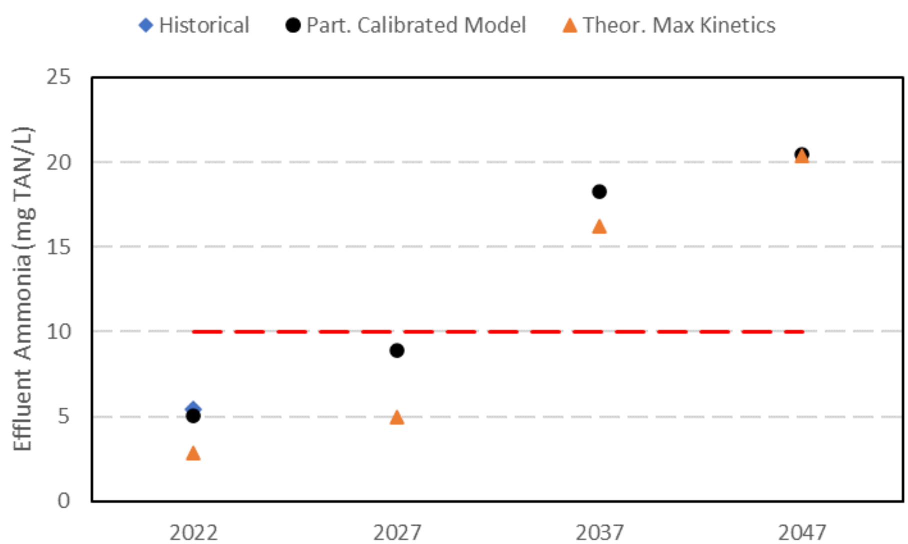

Figure 6-2: Canmore WWTP BioWin Model Winter Performance

6.1.2 Max Month Loading Scenario

The max month loading condition represents the highest mass loading the Canmore WWTP is required to remove. Historically, the max month load occurs in August for most constituents and most notably the TAN Loading. The minimum monthly average temperature observed during the study period in August was f 13.2 °C. All flows and loads were pro-rated based on historical information and the model inputs are presented in (Figure 6-2).

Table 6-2 Max Month Loading Condition

Design Parameter	Historical	5-Year	15-Year	25-Year
ADF, ML/d	8.30	11.32	15.22	19.12
Temperature, °C	13.2	13.2	13.2	13.2
BOD Loading, kg/d	2,105	2,892	3,873	4,855
TSS Loading, kg/d	1,888	2,846	3,849	4,852
TAN Loading, kg/d	195	266	358	450
TP Loading, kg/d	42.9	59.4	79.9	100

The partially calibrated effluent TAN well predicted the historical data. The BAF system generally reduces kinetics at lower effluent concentrations, hence it having a slightly lower baseline in the model compared to the historical data is not unexpected. Similar to the winter monthly loading, the existing BAF system is expected to have challenges at the five (5) year horizon with the process predicted to be beyond its capable limits between the five (5) and ten (10) year horizon.

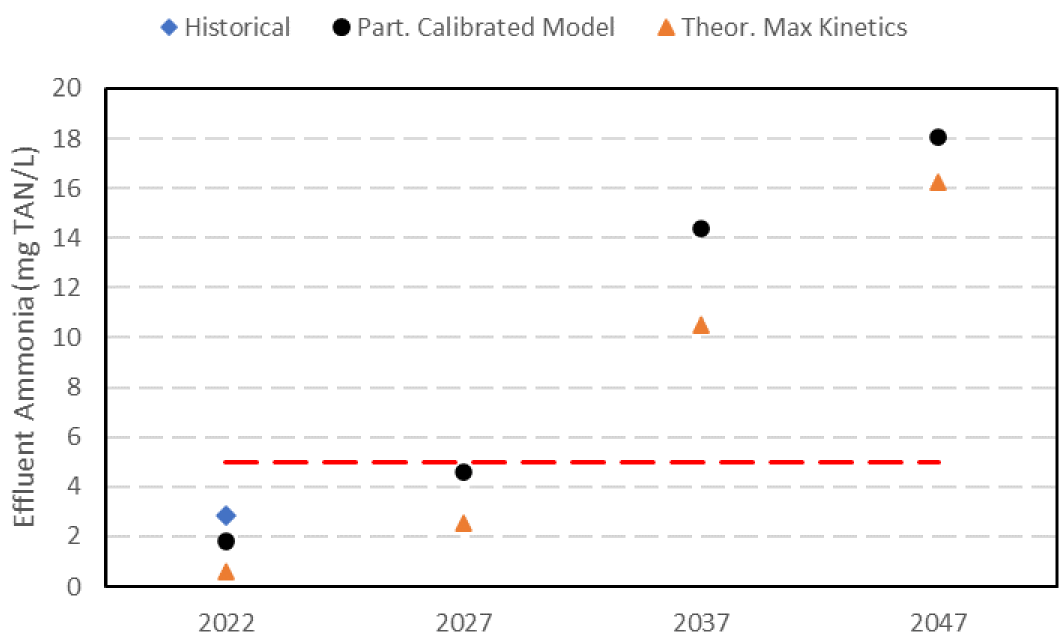

Figure 6-3: Canmore WWTP BioWin Max Month Load Performance

6.1.3 BioWin Model Summary

The BioWin modeling was well aligned with the capacity assessment outlined in Section 4 and 5. The BioWin model provides additional context to predicted effluent concentrations in the future scenarios. The predictions are to be used as a tool for planning as there are several variables that can influence the actual results in the future (i.e. population growth, load variation, plant operations, etc.). In general, the model demonstrates the operations staff are operating the facility well near the capacity limit. It is expected in the five (5) year horizon for the plant to be challenged during the worst case scenarios during the max month load and low temperature conditions.

The Canmore WWTP BAF units are projected to be pushed beyond it's limits even if the facility can operate within the maximum theoretical kinetic rates. This emphasizes the importance of planning and preparing for the large capital upgrade within the ten (10) year horizon, which also coincides with the new license and effluent limits.

7 Summary of Limits and Capacity

7.1 Existing Capacity Summary

The capacity assessment of the Canmore WWTP identified several capacity constraints for the main process units for future flows and loads. The following table illustrates the bottlenecks and challenges as the flows and loads increase with increased development.

Table 7-1 Summary of Key Process Units

Process Unit	Challenges and Recommendations
Raw Sewage Pumping	Increase pump capacity required
Bar Screens	Monitor screenings. Will operate adequately during majority of flow conditions. Bypass events will need to monitor impacts on downstream processes
Grit Removal	Monitor grit carryover. Will operated adequately during majority of flow conditions
C and N-BAF	Nearing Capacity of the filters. Capacity is expected to be challenged at the five (5) year horizon and exceeded between the five (5) and ten (10) year horizon. Optimization and stress testing of system recommended to confirm capacity assessment
UV System	Lifecycle replacement ongoing, will improve reliability from existing capacity assessment

The effluent limits achievable by the plant are limited by the installed process units. A summary of the achievable effluent limits is provided in Table 7-2.

Table 7-2 Effluent Quality Limits of Existing Process Technologies

Constituent	Limit of Installed Technologies
BOD_5	15 mg/L
TSS	15 mg/L
TAN	5 mg N/L
TN	Assimilation only (no denitrification)
TP	1.0 mg P/L

The plant is lacking the capacity to treat the influent wastewater that it will be subjected to within the next 25 years at existing limits. At the 5-year mark, 2027, the BAFs will be near their limit, along with various other process units within the plant. At the 15 and 25-year mark, 2037 and 2047 respectively, the plant does not have the capacity required to adequately treat the wastewater produced by the Town of Canmore. To cope with the future projected flows and loadings a plant upgrade will likely need to be complete by 2030 to achieve the existing limits.

7.2 Water Quality Based Effluent Limits

Alberta Environment and Parks are finalizing the limits that will be in effect on the new approval that will come into effect in 2031. The draft limits are as follows.

Table 7-3: Future Proposed Canmore WWTP Effluent Limits

Parameter	Effluent Limit
$cBOD_5$	≤ 10 mg/L (1)
TSS	≤ 10 mg/L (1)
TAN	≤ 5 mg/L (1) (Jul – Sep) ≤ 10 mg/L (1) (Oct – Jun)
TN	15 mg/L (1)
TP	≤ 0.5 mg/L (1)
Faecal Coliform	≤ 200 per 100 mL(2)

Parameter	Effluent Limit
Notes: (1) Monthly arithmetic mean of daily composite samples (2) Monthly geometric mean of daily grab samples	

The existing treatment plant is not designed to provide total nitrogen removal and will not be able to reach total phosphorus, BOD and TSS on a consistent basis. Significant upgrades will be required to meet the proposed limits.

Refer to "Wastewater Treatment Plant Technology Evaluation" (April 5, 2023) for the upgrade details.

8 Capital Projects

The projects noted in this section are required as a life cycle replacement or maintenance of the existing WWTP equipment to ensure reliable operation of the plant until the Water Quality Based Effluent Limits required upgrades come online in 2031. These upgrades and probable costs are outlined in Table 8-1.

However, the consolidated summary of upgrades and associated costs required to achieve new EPA effluent limits as well as to maintain the existing WWTP are described in a separate report "Wastewater Treatment Plant Technology Evaluation" (April 5, 2023).

Table 8-1: Capital Projects

Project	Description	Project Justification	Priority	Year Required	Engineering	Construction	Contingency	Probable Cost
Headworks								
Inlet Lift Station Upgrade (Mechanical)	Replacement of existing lower flow pumps and discharge piping	Life Cycle Population Growth	Medium	2027	$120k	$800k	$250k	$1.17M
Inlet Lift Station Upgrade (Wetwell)	Increase the size of the Inlet LS Wetwell, Provide Actuated valving to EQ tank	Undersized Population Growth	Medium	2027	$300k	$2M	$600k	$2.9M
Odor Control Unit for EQ Tank, Headworks Bldg	Add odor control building near the Headworks to treat odors from EQ tank, Headworks	Potential Complaints, Regulatory Requirements	Medium	2027	$300k	$2M	$600k	$2.9M
Septage Receiving Station	Add septage receiving station with flow monitoring and payment system [Odor Control project required before]. Include EQ tank upgrades	Population Growth	Low	2032	$120k	$800k	$250k	$1.17M

Influent Piping between Inlet LS and Headworks	Piping Replacement, Actuated isolation valves at high point	Life Cycle	High	2027	$120k	$800k	$250k	$1.17M
Inlet Screen Replacement	Replacement of older inlet screen with smaller mesh, and sludge press unit	Life Cycle	High	2024	$100k	$600k	$200k	$900k
Grit Separator Replacement	Replace Existing Grit Separator	Life Cycle	Medium	2027	$70k	$500k	$150k	$720k
Grit Separator Exhaust Fan	Redesign, replace. Existing fan full of grease	Process Improvements	High	2025	$30k	$200k	$60k	$290k
Headworks Channel Valves	Add sluice gate valve at the Clarifier Distribution Channel Actuation on Clarifier sluice gates and screen inlet gates	Process Improvements	Medium	2027	$70k	$350k	$110k	$530k
Water Heating System, MUA Replacement	Replace existing boiler, piping, MUAs	Life Cycle Process Improvements	Medium	2025	$220k	$1.5M	$450k	$2.17M

Scum Removal Piping	Rearrange Scum Removal Piping to pump to digester instead of Headworks	Process Improvements	High	2025	$70k	$500k	$150k	$720k
Third Clarifier Addition	Add third Clarifier [high flow fluctuations]. North of ex. clarifier	Process Improvements Population Growth	Medium	2027	$900k	$7M	$2.3M	$10.2M
BAF, DAF								
Intermediate Transfer Pumps Upgrade	Upgrade existing pumps	Life Cycle	Medium	2026	$150k	$650k	$200K	$1M
UV								
UV 1, 2 upgrade	Replace existing UV 1, 2	Life Cycle	High	2023	$70k	$500k	$130k	700k
UV 3 addition	Add UV3	Population Growth	Medium	2028	$50k	$350k	$100k	500k

The total estimated probable cost is $27M over the next 10 years. There are several projects that may be consolidated to generate cost efficiencies. It is important to note, with the exception of the BAF transfer pumps project, the specified projects will be utilized past 2031 and will become part of the new facility.