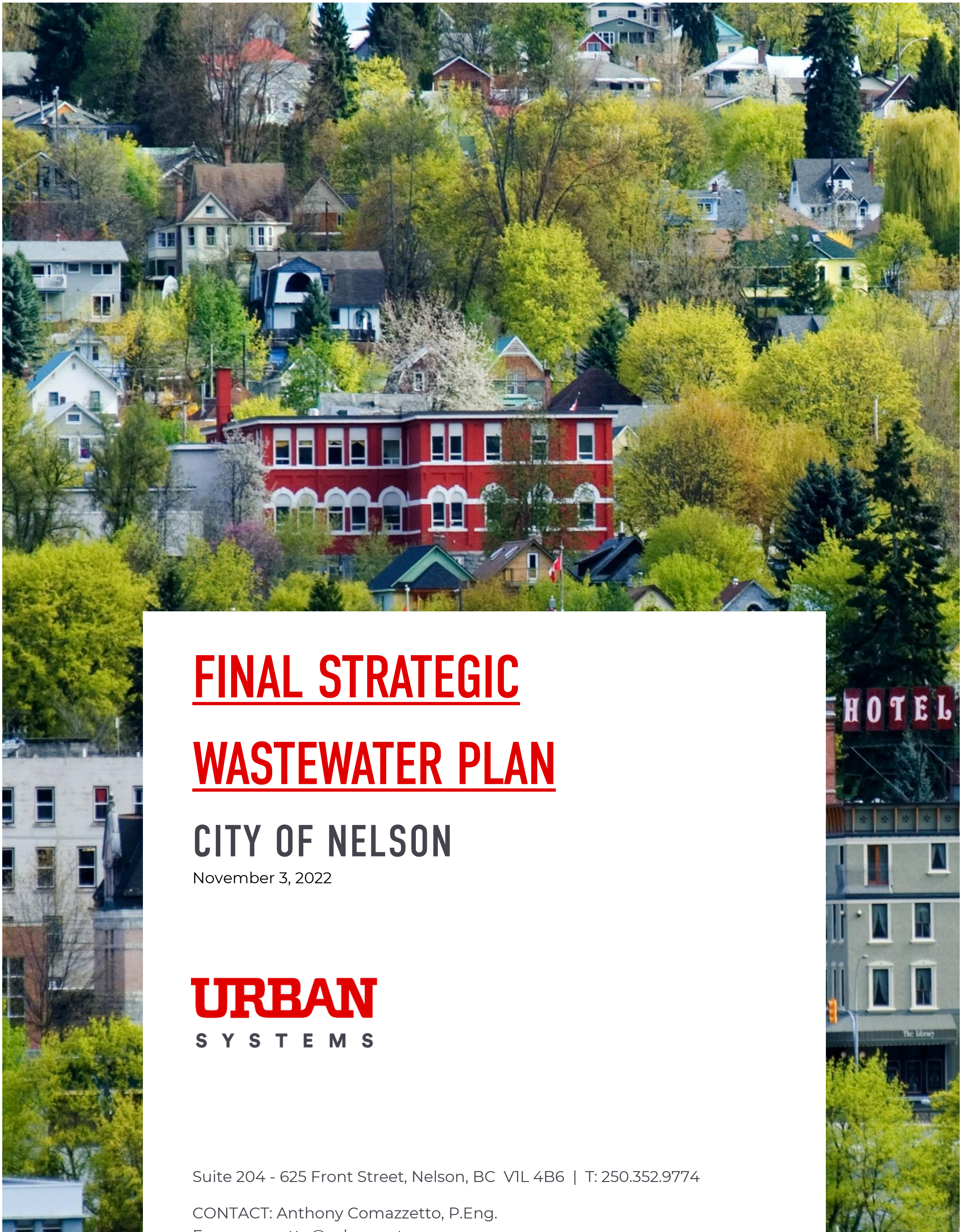

FINAL STRATEGIC WASTEWATER PLAN

CITY OF NELSON

November 3, 2022

URBAN SYSTEMS

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EXECUTIVE SUMMARY

The City of Nelson Strategic Wastewater Plan provides an inventory, condition, and capacity assessment of the City's municipal sewer system. The main components are:

- The collection / conveyance system and lift stations.
- The transmission system to the treatment plant.
- The treatment plant and liquid and biosolids disposal.

The findings can be summarized as follows:

1. The collection system consists of pipes of varying materials and ages ranging from 10 years to 100 years. Condition and capacity assessments were completed of the collection system and specific sections of gravity mains have been identified for upgrades. The City has been disconnecting storm water leads to the sewer system, however, there are remaining connection points that should be disconnected as opportunity allows.
2. The lift stations are at various stages of wear and a lift station upgrade and replacement program is recommended. The recommended short-term actions for the Airport, Lakeside Drive, and KFP lift stations are highest priority and should be implemented first.
3. The sewage transmission system from the airport lift station to the PCC has been reviewed at a high level and a concept-level review of replacing it. A more in-depth review is recommended to determine the best approach to mitigate further discharge of raw sewage to the Kootenay River.
4. The existing wastewater treatment plant is biologically overloaded with several unit processes nearing or exceeding their hydraulic capacities. A comprehensive comparison of upgrading the existing facility versus designing/constructing a new facility has been completed. It is recommended that the City complete a Liquid Waste Management Plan (LWMP) to inform the best approach moving forward.
5. Discharge to the Grohman Narrows is an acceptable and efficient form of treated effluent discharge and can be continued indefinitely.
6. Biosolids are currently hauled to the regional landfill in Ootischenia. It is recommended that the City continue to monitor the quality and treatment of its biosolids in the context of the OMRR. Once a better understanding of the future direction for the wastewater treatment options is available, a more comprehensive review of potential biosolids management options should be undertaken.

1.0 INTRODUCTION

1.1 BACKGROUND

This document presents an assessment of the City of Nelson (City) sanitary sewer system. The initiative to undertake a Strategic Wastewater Plan (SWWP) was developed by City staff considering the age of the system and in anticipation of future servicing requirements. Urban Systems Ltd. (Urban) had previously completed a Sewer Master Plan in 2010. The 2022 update makes use of the inventory and analyses completed in 2010 and provides a comprehensive assessment of the City's sewage system.

1.2 SCOPE

The SWWP is divided into four parts.

1.2.1 PART 1 – COLLECTION SYSTEM NETWORK

- Review the collection system
- Model population and flow projections
- Complete a capacity and condition assessment of the collection and conveyance system
  - Complete opinions of probable cost based on these criteria

1.2.2 PART 2 – TRANSMISSION SYSTEM

- Review the transmission system between the Airport Lift Station and the Grohman Narrows PCC

1.2.3 PART 3 – TREATMENT AND DISPOSAL SYSTEM

- Review the treatment plant's regulatory requirements
- Summarize the influent flows and biological loading to the plant
- Review the feasibility of a potential alternate sewage treatment plant
- Complete opinions of probable cost for upgrades to the existing PCC and capital costs for a potential alternate sewage treatment plant

1.2.4 PART 4 – BIOSOLIDS

- Review the biosolids management plan's regulatory framework
- Summarize qualitative and quantitative data from the treatment plant's biosolid production
- Discuss biosolids management options and recommendations

2.0 PART 1 – COLLECTION SYSTEM NETWORK

2.1 EXTENT OF SYSTEM

The existing sanitary system consists of approximately 80 km of gravity mains and manholes, 5 km of forcemains, seven (7) lift stations, Pollution Control Centre (PCC) and gravity outfall to Grohman Narrows which forms the West Arm of Kootenay Lake. Six of the lift stations (CPR, North Shore, KFP, Lakeside Park, Lakeside Drive and Tyler) pump from low points to downstream gravity sewers within the collection system. All gravity mains converge at the Airport Lift Station, which pumps through a 400 mm marine forcemain to the treatment facility/Pollution Control Centre (PCC).

The collection system spans nearly every street within the City boundary, from Granite Pointe golf course on the west, to the City cemetery on the south, the old railway right-of-way along the east and Kootenay Lake on the north. The extent of the existing collection system is shown in **Figure 1.**

2.2 POPULATION AND FLOWS

2.2.1 POPULATION PROJECTIONS

The City has undertaken an assessment of growth potential and concluded that the buildout population for the City should be increased from 15,000 persons (per previous studies) to 24,476 based on newer strategies for infill development. The City has recommended a growth rate of 1.2% per annum. **Table 1** presents the total equivalent populations used for each of the modeling scenarios (i.e., existing, 20-year growth, and buildout conditions). **Table 1** provides the key population characteristics that will guide the future conditions modeling and analysis.

Table 1: Population Projections (Growth Rate of 1.2%)

Parameter	
2016 Census Population	10,572
Estimated 2021 Population	11,222
Projected 2041 Population	14,245
20 Year Growth (equivalent population)	3,024
Buildout Population	24,476
Years to reach buildout	65
Buildout year	2086

The remainder of Section 2.2 identifies both observed flow conditions and parameters used to estimate future flows. The key parameters that are discussed include:

- Average Dry Weather Flows (ADWF) – ADWF is used to evaluate per capita flows
- Infiltration & Inflow (I/I) – I/I is an estimate of extraneous water entering the collection system
- Peak flows and peaking factor – Peak flows observed during flow monitoring and the calculated peaking factors are identified.

- Peak Wet Weather Flow (PWWF) – PWWF is the summation of the Peak Dry Weather Flow and I/I. The PWWF is the highest stressor on the collection and is used to identify residual capacity of existing pipes and size future pipes.

2.2.2 DRY WEATHER FLOWS

The City's Subdivision and Development Servicing (SDS) Bylaw suggests a per-capita sewer loading of 360 L/capita/day for residential growth (or applied to equivalent population for ICI development).

Based on the estimated 2021 population of 11,222 persons, the 2021 per capita loading under dry weather conditions was 307 L/day. The model was adjusted to reflect this rate for existing customers, while applying the 360 L/capita/day to future growth.

2.2.3 INFLOW AND INFILTRATION

The flow monitors observed minor groundwater influence under dry weather conditions. Without a detailed property-by-property evaluation of water usage records, it was assumed that approximately 50% of nighttime flows were due to dry weather I&I. Based on this assumption, it is estimated that infiltration and inflow makes up no more than 10% of the peak flow during dry weather conditions.

Peak flows into the Airport Lift Station, under existing conditions, were observed to increase by almost four times during the wet weather conditions of August 22, 2021. The rainfall event on the 22nd had a peak intensity four times greater than any other storm during the monitoring period. The existing system appears to be highly reactive in certain parts of the City (the downtown core, along Lakeside Drive and the older neighbourhoods between 5th and 10th Streets). This leads to a conclusion that these areas still have many directly connected catch basins and/or rainwater leaders, many of which the City is aware of.

For proposed pipes, the SDS bylaw values were used to estimate I/I—this includes applying 5000 L/ha/d and 8000 L/ha/d to pipes above and within the water table, respectively.

No additional I/I was allowed for infill development or new development that does not require the collection system to be extended. Minimal increase of I/I is expected as all planned growth will occur within the extents of the existing the collection system.

2.2.4 PEAKING FACTORS

The peaking factor observed from flow monitoring devices installed throughout the collection system were estimated by dividing the observed peak flow under dry and wet weather conditions by the average flow. The observed dry and wet weather peaking factors can be a helpful metrics to gauge the performance of the collection system and impact of I/I.

Peaking factors at the flow monitoring devices varied between 1.8 and 2.5 under dry weather conditions, between 1.7 and 3.7 under wet weather conditions, and between 2.3 and 9.4 during the high intensity rain event of August 22, 2021.

The peaking factor (dry weather) at the Airport lift station was approximately 1.9 for the period of 2018 – 2020. This is lower than the other lift station and makes sense as peak flows are expected to undergo some attenuation in the long, larger diameter trunk mains (for example, the Lakeside Drive trunk main).

A peaking factor was calculated using the City's SDS bylaw criteria for each sub-catchment within the sanitary model. This calculated peaking factor is applied the ADWF to estimate the peak dry weather flow. The City's SDS bylaw identifies the following formula to calculate the peaking factor:

Peaking Factor = 3.2 / (population in thousand)$^{0.105}$

2.2.5 TOTAL SYSTEM FLOWS

Table 2: Total System Flows

Scenario	Average Dry Weather Flows – ADWF (L/s)	Peak Dry Weather Flows - PDWF (L/s)	Infiltration and Inflow – I/I (L/s)	Peak Wet Weather Flow (L/s)
Existing (2021)	51	98	284	382
20 Year Growth (2041)	65	133	269	402
Buildout	108	208	279	487

2.3 EVALUATION & DESIGN CRITERIA

The criteria used to evaluate the condition and capacity of the collection system is noted below. The evaluation criteria were used to evaluate when existing infrastructure will require an upgrade based on capacity and condition considerations. The design criteria were used to size proposed infrastructure.

2.3.1 CONDITION ANALYSIS

The condition analysis reviewed gravity sewers, forcemains and lift stations. Gravity sewers and forcemains were evaluated based on their age and material. **Table 3** identifies the expected asset life for various pipe materials that was used to determine when age-based replacements are recommended.

Table 3: Expected Asset Life of Various Sewer Main Materials

Pipe material	Expected asset life
Asbestos Cement (AC)	70 years
Concrete	70 years
Corrugated Metal Pipe (CMP)	50 years
Vitreous Clay (VCT or VIT)	60 years
PIC/HDPE	80 years
In-situ Lined (CIPP program)	60 years

The lift stations were evaluated based on their performance (e.g., pump operational data was compared to pump curves published by the manufacturer) and were visually inspected for signs of deterioration and/or safety concerns. In addition, Urban Systems engaged Ready Engineering (electrical consultant) to review each station's electrical power supply capacity, the condition of electrical & instrumentation equipment and to identify if there were any items that did not comply with the Canadian Electrical Code (CEC).

2.3.2 CAPACITY ANALYSIS

The key criteria used to evaluate the capacity of the existing collection system and the size the proposed upgrades are specified below.

Table 4: Evaluation & Design Criteria

Collection System Component	Evaluation & Design Criteria
Gravity Sewer	
Min Pipe Size	200 mm for residential and 250 mm for ICI development
Min Velocity	0.6 m/s
Max Velocity	3.0 m/s
Manning's n Value	0.011 for PVC and 0.013 for concrete pipe
Depth/diameter (d/D) Max Limit	For evaluating when existing pipes are deficient, a d/D <= 1.0 limit under PWWF conditions was used. For sizing proposed pipes, the following d/D limits were used. 200 mm pipe size, use 0.5 250 mm pipe size, use 0.6 300 mm or larger pipe size, use 0.7
Min Cover	1.5 m
Forcemains	
Min Pipe Size	100 mm
Min Velocity	1.0 m/s
Max Velocity	3.5 m/s
Hazen Williams C Value	130 for PVC and 120 for ductile iron, concrete & steel
Min Cover	1.5 m
Lift Stations	
Wet Well	Active storage volume is adequate to limit pump starts to six per hour
Pumps	PWWF can be conveyed with the largest pump out of service & upgraded stations are sized to convey the projected 20-year PWWF

2.4 SANITARY MODEL INFORMATION

The hydraulic sanitary model was created with the PCSWMM software. Three models were created for this project including: 1) Existing Conditions, 2) 20 Year Growth Conditions and 3) Buildout Conditions.

The existing conditions model was updated to include all recent sewer main upgrades as identified on a map that was provided by the City – refer to **Figure 3**. Note that the pipe upgrades include mains that have been replaced or pipes that have had a cured-in-place-pipe (CIPP) liner installed.

The collection system catchments defined within the model, growth allocation and model calibration are discussed below.

2.4.1 COLLECTION SYSTEM CATCHMENTS

Catchments are defined within the collection system to aid in allocating flows in the hydraulic model. The property of each catchment is analyzed to determine the total equivalent population, peaking factor, and diurnal curve.

The City's collection system is made up of eighteen (18) separate catchments (labelled A through R) and form the basis of the manhole and sewer main asset identifiers. As part of the modeling process and review of flow monitoring data, the catchments were further subdivided to allow the assignation of sewer loads and infiltration & inflows. The forty-eight (48) subcatchments are depicted in in **Figure 2.**

A subcatchment was not created for the Tyler lift station given its very small collection boundary.

2.4.2 MODEL LOADING POINTS & GROWTH ALLOCATION

The City provided a map of known development locations that are likely to develop within the next 20 years. More detail of the specific development locations and model allocation are provided in **Appendix A** – Design Criteria for the Collection System. The larger developments and their allocation on the model are shown in **Figure 4.**

Sanitary sewer loading for the future conditions model first identified known development areas as shown on **Figure 4**. Sewer loads were assigned to the logical downstream node of each development site. The remainder of infill growth was evenly allocated across the remaining nodes in the sewer model.

2.4.3 MODEL DEVELOPMENT & CALIBRATION

Urban Systems has maintained the City's hydraulic sewer model for the past 10 years. PCSWMM is the current software used for the modeling work and shared the same engine as the free EPA-SWMM software but has an enhanced user interface. The City provided updated infrastructure data (new mains, newly lined mains, abandoned mains, etc.) for inclusion in the model. The existing conditions sewer model was then calibrated using 2021 flow monitoring data (5 stations across the network) as well as SCADA data for lift stations with flow meters. The calibration process involved allocation of units, adjustment of per-capita flows and best estimated for infiltration and inflow. Peak flows and volumes were aligned to within 5% of each other under dry and wet weather conditions. Additional details of the calibration process are provided in Technical Memorandum #2 – refer to **Appendix B.**

2.5 CAPACITY ASSESSMENT

2.5.1 GRAVITY SEWERS

As noted, the trigger for a sewer main upgrade occurs when depth of the peak flow (PWWF) exceeds the diameter of the main (depth/Diameter = 1). Four (4) instances of this condition were found in the existing conditions model and are shown in **Figure 6**. No additional surcharging observed under the 2041 scenario as shown in **Figure 8.** One additional sewer main capacity issue arose under the buildout scenario as shown on **Figure 9**. **Figures 10** and **11** identify the upgrades that are required to convey the 2041 and buildout flows, respectively.

2.5.2 LIFT STATIONS & FORCEMAINS

Table 5 lists the characteristics of the seven (7) existing lift stations in the sanitary system. The Tyler lift station was not included in the hydraulic model as detailing information was not available, nor would the small flows materially affect system performance.

Table 5: Existing Lift Station Characteristics

Data	Airport	CPR	North Shore	KFP 4th Street	Lakeside Park	Lakeside Drive	Tyler
Type	Dry pit	Dry pit	Submers.	Submers.	Submers.	Dry pit	Submers.
#Pumps	3	2	2	2	1	2	2
HP (each)	75	25	10	5	3	15	3
Power (V)	600	208	480	208	220	480	208
Genset	Yes	No	Yes	No	No	No	No
Capacity[1] (L/s)	189	30	30	18	3	20	3
TDH (m)	33.5	38	30	10	10	15	10
Peak Inflow (L/s)	217	39	3	2	< 1	15	< 1
Level Sensor	Ultra Sonic	Bubbler	Ultra Sonic	Floats	Floats	Ultra Sonic	Ultra Sonic
Controls	Multi Ranger	Relay	Relay	Relay	Relay	Relay	Miltronics
Flowmeter	Yes	Yes	No	No	No	No	No
Telemetry	Telephone	Telephone	Telephone	None	None	Telephone	None

* capacity with two pumps operating at Airport Lift Station and one duty pump operating at all other stations.

Tables 6 and **7** provide estimates of future flow and volume conditions at the lift stations (with the exception of the Tyler station) as observed in the hydraulic models.

Table 6: Expected Peak Flows into Lift Stations (20 Year Growth/2041 Horizon)

2041	Airport	CPR	Lakeside Drive	Lakeside Park	4th/KFP	North Shore
Metric						
ADWF (L/s)	65	8.7	3.7	0.03	2.7	1.6
PDWF (L/s)	133	18.6	4.8	0.1	8.2	3.1
Peaking Factor	2.1	2.1	1.3	2.9	3.1	2.0
I & I (L/s)	269	17.2	1.9	0.1	2.2	1.8
PWWF (L/s)	402	35.7	6.7	0.2	10.4	4.9

* peak flows will decrease as the City proceeds with additional storm system separation work

Table 7: Expected Peak Flows into Lift Stations (Buildout Horizon)

Buildout	Airport	CPR	Lakeside Drive	Lakeside Park	4th/KFP	North Shore
Metric						
ADWF (L/s)	108	11.6	5.6	0.05	2.9	2.8
PDWF (L/s)	208	24.6	7.7	0.1	8.7	6.0
Peaking Factor	1.9	2.1	1.4	2.2	3.1	2.0
I & I (L/s)	279	17.9	1.9	0.1	2.2	1.8
PWWF (L/s)	487	42.5	9.6	0.2	10.9	7.8

* peak flows will decrease as the City proceeds with additional storm system separation work

Comparing the future peak flow estimates to **Table 7**, only the Airport and CPR lift stations will require a capacity upgrade which are discussed below.

2.5.3 AIRPORT LIFT STATION

The existing pump station is equipped with 3 duty pumps. The lead pump runs on a VFD when wet well depths are between 0.6 m and 1.0 m meters to prevent the pump from operating off its curve. The second pump activates once the wet well reaches a depth of 1.6 meters. The third pump turns on and activates the high-level alarm when wet well depth reaches 1.9 meters. An overflow leads to Kootenay Lake when depth is approximately 1 meter above the alarm level.

All three pumps were in operation during the high intensity storm event of August 22, 2021. The peak intensity lasted for approximately 10 to 15 minutes, during which pump flows reached a combined rate of 226 L/s according to SCADA records. There was sufficient wet well capacity to avoid overflow to Kootenay Lake.

The August 22, 2021, rain event occurred in the evening and not during the morning period of 9 am to 11 am when domestic flows peak. If a similar rainfall event were to occur during the same time as peak domestic flows, the existing conditions model estimates that the peak flow into the station could reach 384 L/s. The peak wet weather flow estimates for the 20 year and buildout horizons are 402 L/s and 487 L/s, respectively.

The Airport Lift Station must be upgraded to convey the 20-year projected flow. Given the ultimate flow of 487 L/s, it is recommended that the lift station's forcemain be upsized to 600 mm to decrease friction losses. Three alignment options were considered for the replacement as identified in Section 3. The airport lift station upgrade costs assumed that a marine or the CPR alignment options will be selected by the City. The lift station upgrade will cost more if the Highway 3A forcemain alignment option is pursued.

The opinion of cost assumes a new station would be constructed near the existing facility and does not include an allowance for purchasing land. The station would consist of a buried concrete wet well with at least 40 m^3 of active storage, 2 duty pumps, 1 standby pump, and a building to house electrical and critical process valving. It is assumed the existing grit chamber will continue to be used and the new station would tie-in downstream.

2.5.4 CPR LIFT STATION

Upgrade pumps to meet future peak flow rate of 36 L/s per **Table 6**. The pump and forcemain combination should be scalable to the future peak flow rate of 43 L/s.

The existing forcemain is 150 mm in diameter. The newest section, installed last year, is comprised of HDPE 11 pipe from Ymir Road (the Highway) to Kootenay Street. The remainder of the 150 mm CI mains were installed in the 1960s and are at the limit of their service life, if not already beyond.

A hybrid forcemain, consisting of 200 mm forcemain from the station to Ymir Road and leaving the recent 150 mm from Ymir Road to Kootenay Street will reduce the friction losses in the system curve, thereby reducing required pump power and power usage. Alternately, the existing CI main could be lined and coupled with higher capacity pumps to meet the future peak flow rates.

However, we recommend dedicated infiltration and inflow testing in this catchment to confirm the peak flow into the station which could possibly defer or eliminate the need for pump upgrades.

2.6 CONDITION ASSESSMENT

2.6.1 CONDITION OF PIPES AND MANHOLES

Condition of existing mains were evaluated solely on pipe material, installation date, and estimated service life. Section 2.3.1 lists the anticipated service life of each pipe material which was then used to determine remaining service life of each sewer main. A graphic of the existing sewer main materials is provided in **Figure 5.** An estimate of the remaining service life of each main is shown in **Figure 7** and summarized in **Table 9.**

2.6.2 CONDITION OF LIFT STATIONS

The recommended actions from the lift station condition assessment are summarized in the table below. Refer to **Appendix D** for a detailed memorandum summarizing the assessment.

Table 8: Lift Station Recommendations, Priority, and Cost to Upgrade

Lift Station	Recommendations	Priority (1 high to 5 low)	Short-Term Cost ($)
General (applies to all lift stations)	Short-Term Actions: • Ensure that City has current confined space entry program for each station that complies with Part 9 of the OHS Regulation in BC • Update servicing bylaw to require all critical valving to be in above ground structures Long-Term Actions: • Eliminate confined spaces as lift stations are upgraded to increase capacity or replaced due to condition by locating critical valving and electrical controls in above ground structures.	1	Not included
Airport	Short-Term Actions: • Complete electrical upgrades (refer to **Appendix A**) • Repair or replace pressure transmitter	1	$400K

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Lift Station	Recommendations	Priority (1 high to 5 low)	Short-Term Cost ($)
	• Inspect/repair pump 2 as required due to pressure deficiency Long-Term Actions: • See General Items. No additional items.		
Lakeside Drive	Short-Term Actions: • Replace the wet-well level sensor with an alternate style that is less prone to humidity issues (e.g., radar) • Remove non-explosion proof heat lamp from wet well • Replace the broken valve to allow pump #2 to operate • Replace portable cord between dry / wet pit with permanent wiring Long-Term Actions: • Move electrical controls to an above ground kiosk	1	$20K
CP Rail	Short-Term Actions: • Investigate options to address land ownership issue • Inspect and repair pump 2 as required due to flow deficiency • Adjust float mounts per operations staff feedback • Verify that wet well is sealed to prevent migration of explosive gases Long-Term Actions: • See General Items. No additional items.	2	$25K
Tyler Lake	Short-Term Actions: • Verify that wet well is sealed to prevent migration of explosive gases • Ensure 1 m clear space in front of electrical panels is always maintained Long-Term Actions: • See General Items. No additional items.	5	$10K
Lakeside Park	Short-Term Actions: • Consider purchasing a spare pump • Verify that wet well is sealed to prevent migration of explosive gases • Consider connecting to SCADA and adding a high level float that is connected to a local audible or visual alarm Long-Term Actions: • Consider replacing with duplex packaged lift station when station is replaced due to age (provide redundancy)	3	$80K
KFP (4th Street)	Short-Term Actions: • Add one junction box for three conduits running from wet well to kiosk with appropriate seals (refer to **Appendix A**) • Investigate why genset does not turn off after utility power is restored and correct	1	$15K

Lift Station	Recommendations	Priority (1 high to 5 low)	Short-Term Cost ($)
	Long-Term Actions: • See General Items. No additional items.		
North Shore	Short-Term Actions: • Investigate why genset does not turn off after utility power is restored and correct • Add knock-out plugs in MCC door (see **Appendix A**). Long-Term Actions: • See General Items. No additional items.	5	$8K

Notes:

- Opinions of probable cost include 35% and 15% allowance for contingency and engineering, respectively
- KFP and North Shore lift stations both have an issue with the standby generator occasionally continue to run after utility power is restored. The above costs include an allowance for investigating this issue but do not include costs for replacing equipment if deemed necessary.

2.7 COLLECTION SYSTEM OPINIONS OF PROBLABLE COST

2.7.1 CONDITION BASED UPGRADES

Gravity Sewer

The expected service life of sewer mains by material listed in **Table 3** was applied to the City's sewer main dataset. **Table 9** quantifies the amount of sewer main requiring replacement due to age/condition with the prioritization of: immediately, within 10 years, within 20 years, within 50 years, and beyond 50 years.

Table 9: Sewer Main Replacement Due to Age

	Remaining Service Life				
Diameter	**0**	**1 – 10**	**11 – 20**	**21 - 50**	**50+**
50	0	0	0	72	443
100	210	164	13	195	236
150	5,996	4,693	1,878	1,508	8,333
200	1,551	8,073	9,941	4,873	17,432
225	0	0	0	0	82
250	350	843	1,976	924	2,427
300	23	55	1,360	844	1,594
375	80	234	1,227	404	647
400	3,074	0	59	69	0
450	0	0	0	236	1,587
525	0	0	0	0	186
600	0	53	0	198	0
750	0	0	405	0	149
Total	**11,284**	**14,115**	**16,860**	**9,321**	**33,114**
Replacement Cost	**$6,931,000**	**$7,994,000**	**$10,332,000**	**$5,641,000**	**$19,550,000**

Lift Stations & Forcemains

Opinions of probable cost for the recommended condition-based upgrades are included in **Table 8** and total $558k with 40% allowance being included for engineering and contingency.

2.7.2 CAPACITY BASED UPGRADES

Gravity Sewer

The existing collection system is quite robust though there are known capacity issues along Lakeside Drive, Beatty Avenue and Mill/Stanley Streets. Refer to **Figures 8** and **9** for capacity issues (d/D=1) identified under the 20 year and buildout growth horizons.

Table 10 summarizes the capacity upgrades required in the collection system to accommodate growth to 2041. Refer to **Figure 10.**

Note that the upgrades identified in **Figure 10** match the improvements identified for the buildout scenario. The upgrades required for buildout are illustrated in **Figure 11**.

Table 10: Sewer Main Upgrades

Project	Cost	Timing
Lakeside Drive	$3.9M	0 – 10 years
Beatty Avenue	$300k	0 – 10 years
Mill Street	$150k	0 – 10 years
Stanley Street	$150k	0 – 10 years
Total	**$4.5M**	**0 – 10 years**

Lift Stations & Forcemains

Table 11 summarizes the upgrades required for both condition and capacity of the existing sewage pump stations.

Table 11: Lift Station Improvements

Project	Cost	Timing
Airport Lift Station	$3.5M*	0 – 10 years
CPR Lift Station	$TBC after testing	0 – 10 years

* Does not include forcemain upgrade. Refer to Section 3 for additional costs.

2.7.3 PRIORITIZATION OF COLLECTION SYSTEM UPGRADES

Table 12 summarizes all of the proposed upgrades and their prioritization.

Table 12: Proposed Upgrades and Prioritization

Project	Order of Magnitude Cost	Priority/ Timing	Sequence for Completing Projects	Rational
Replace Airport LS (worst case)*	$3.5M	Priority 1 2021 – 2026	1	Highest consequence of failure. Does not include $400k in electrical work to the existing station that is underway and approved.
Replace Sewer Mains due to Capacity	~$2.2M	Priority 1 2021 – 2026	2	Reduce surcharging and potential flooding/odor issues
Replace CPR LS	$TBC for capacity + $25k for condition based work	Priority 2 2021 - 2031	3	Confirm I/I within catchment. May delay capacity upgrades
Replace Sewer Mains due to Age (no service life remaining)	~$7M	Priority 2 2021 - 2031	4	Highest risk mains for failure (old AC and VIT)
Upgrade Lakeside Drive LS	$20k	Priority 2 2021-2031	5.1	Refer to Table 7
Upgrade 4th Street/KFP LS	$15k	Priority 2 2021-2031	5.2	Refer to Table 7
Upgrade Lakeside Park LS	$80k	Priority 2 2021-2031	5.3	Refer to Table 7
Upgrade North Shore LS	$8k	Priority 2 2021-2031	5.4	Refer to Table 7
Upgrade Tyler LS	$10k	Priority 2 2021-2031	5.5	Refer to Table 7
Replace Sewer Mains due to Age (0 – 10 years service life remaining)	~$8M	Priority 3 2031 – 2041	6	Pipes Reaching End of Service Life
Replace Sewer Mains due to Age (11 – 20 years service life remaining)	~$10.3M	Priority 4 2041 – 2051	7	Pipes Reaching End of Service Life

* Does not include forcemain upgrade. Refer to Section 3

3.0 PART 2 – TRANSMISSION SYSTEM

The Airport lift station pumps all the City's sewage to the Grohman Narrows PCC through a 400 mm steel marine forcemain. The forcemain was installed in 1971 and has been in service for 50 years. The forcemain is aging and has had past failures that required underwater repair work. This section of the report reviews replacement options for the aging marine forcemain that must be upsized to accommodate future flows.

The replacement forcemain can be installed on land or another marine installation can be considered. Refer to the attached **Figure 12** which identifies three alignment options that have been considered. These options were originally identified in 2003 in the report titled "City of Nelson – Airport Forcemain Route Options Review" that was prepared by Urban Systems Ltd.

3.1 ALIGNMENT OPTIONS AND COSTS

The table below summarizes the alignment options that are shown in **Figure 12**. The replacement pipe must be at least 600 mm in diameter to accommodate future flows without causing excessive friction losses. The costing presented in the table below is based on installing a 600 mm pipe.

Table 13: Forcemain Alignment Options Summary

Alignment Option	Land or Marine Install?	Length (m)	High Point Elevation (m, geo.)	Land Ownership Along Alignment	Creek Crossings	Rail Crossings	Cost ($)
Existing	Marine	3,100	534	Crown	n/a	n/a	n/a
CPR Rail	Land	3,300	552	Private	1	2	$10.4M
Highway 3A	Land	3,500	578	Private	1	2	$9.8M
Kootenay River	Marine	3,100	534	Crown	n/a	n/a	$8.4M

3.2 PREFERRED ALIGNMENT

The Kootenay River marine installation is recommended. It is expected to have the lowest capital cost and will offer the lowest life cycle cost as this alignment option has the lowest high point that the Airport lift station must operate against. The plan for installing pipe supports (i.e., steel casing pipe) to bridge underwater ravines should be reviewed closely with an experienced contractor if this option advances to a preliminary design level. This is a key challenge that must be addressed through design and construction. If this alignment option is not feasible for any reason, we recommend the CPR Rail alignment option as the next choice to avoid the high point on Highway 3A alignment.

An expressed concern regarding the marine alignment is the heat lost in the sewage within the 3,100 m marine forcemain. The Kootenay River removes a lot of heat and the influent sewage at the plant can be as low as 6.3 deg-C. **Figure 13** shows the variation of temperature of influent sewage from 2019 to 2021.

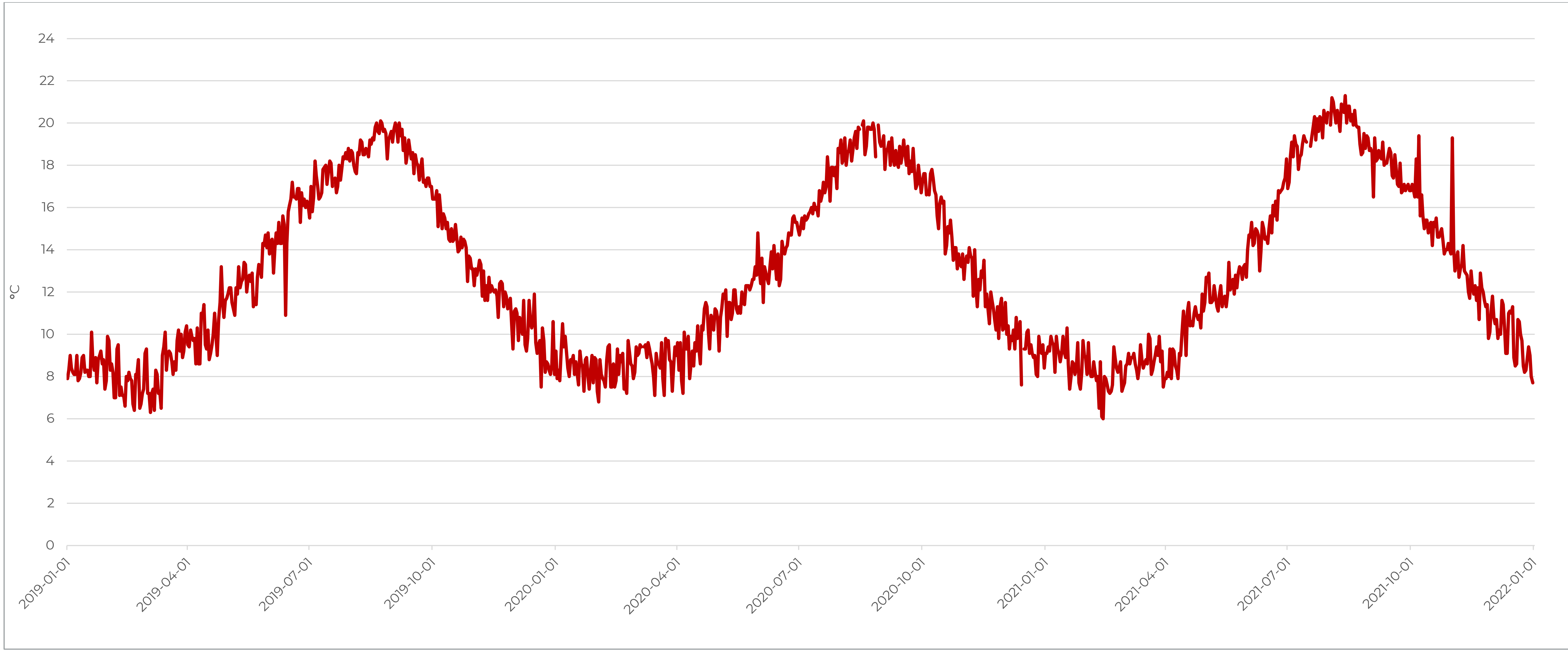


Figure 13: PCC Influent Sewage Temperature

4.0 PART 3 – TREATMENT AND DISPOSAL SYSTEM

The City of Nelson operates the Grohman Narrows Pollution Control Centre (PCC) and discharges treated effluent to the Grohman Narrows approximately 5 km west of the City of Nelson. The PCC was upgraded in 2005 to provide secondary treatment with the addition of rotating biological contactors (RBCs), secondary clarifiers and replacement of the chlorination system with an ultra-violet (UV) light disinfection system. Before 2005 the PCC was limited to primary treatment only. The 2005 upgrade included the first phase of RBCs which were anticipated to reach capacity approximately 10 years after installation. A second phase with the installation of a third RBC train was envisioned subsequently but has not been constructed.

The PCC is currently operating at, or beyond, capacity with respect to the WSER requirements and both the design and permitted flow. In 2018 the City undertook an assessment study that identified major upgrades required for both continued operation and to accommodate growth in the community (Urban Systems, 2018). Nelson is a thriving community and additional growth is planned and expected.

This section summarizes the findings of Technical Memo 4 – Sewage Treatment and Disposal which is attached to this report in **Appendix E**.

4.1 REGULATORY REQUIREMENTS

Both Provincial and Federal regulations apply to the PCC.

4.1.1 PROVINCIAL REGULATIONS

The PCC operates under BC Ministry of Environment and Climate Change Strategy (ENV) permit PE-291. This permit was last updated in March 2006 in order to recognise the upgrades to secondary treatment and replacement of the chlorination system with UV disinfection. The conditions of the permit allow the following discharge criteria:

- Flow 5,680 m^3/d, at a maximum rate of 8.5 m^3/minute;
- Five-day biochemical oxygen demand (BOD_5) ≤ 140 mg/L which is defined as total BOD_5;
- Average total suspended solids (TSS) ≤ 100 mg/L; and,
- Coliform bacteria ≤ 150,000 MPN/100 mL, which is defined as total coliforms.

The intent was to register the upgraded facility under the Municipal Wastewater Regulation (MWR). Through direction from ENV, an environmental impact study (EIS) was prepared and included an assessment of the outfall conditions and effluent dispersion/dilution in order to address the requirements of the MWR. The information presented in the EIS indicated that the estimated dilution ratio at the edge of the initial dilution zone (IDZ) is in the order of 270:1. The effluent criteria recommended in the EIS are summarised in **Table 14**, and intend not just to recognise Provincial legislation, but also to recognise the requirements of the Federal wastewater regulation.

Table 14: City of Nelson Recommended Effluent Criteria

Parameter	Criteria
Five-day carbonaceous biochemical oxygen demand ($CBOD_5$)	An average equal to or less than 25 mg/L, with a maximum of 45 mg/L.
TSS	An average equal to or less than 25 mg/L, with a maximum of 45 mg/L.
Ammonia	Treatment not required, based on the ability to meet either acute concentrations before discharge or chronic concentrations at the edge of the IDZ.
Phosphorus	Treatment not required
Disinfection	Faecal coliform concentration ≤ 200 MPN/100 mL at the edge of the IDZ. This translates to an effluent concentration of 54,000 counts/100 mL for the dry weather design flow and 42,000 counts/100 mL for the wet weather design flow.
Total chlorine residual	≤ 0.02 mg/L (if chlorine is the chosen method of disinfection)

Registration under the MWR is a multi-year process, with expected timelines for each phase outlined below.

- Preliminary application: approximately 6 months.
- Preparation of submission materials by proponent: up to 3 years.
- Final application review and approval by ENV: between 12 and 18 months depending on complexity.

In addition, it is possible that the BC Environmental Assessment Act (BCEAA) may be triggered by major upgrades to the system. The triggers under the BCEAA are:

- A new facility that is designed to serve ≥ 10,000 people.
- An existing facility that is designed to serve ≥ 10,000 people and will result in an increase of ≥ 30% of the total waste discharge.

A system upgrade could trigger the BCEAA. Undertaking a BCEAA is a multi-year process and will require community and First Nations engagement. However, should an approved Liquid Waste Management Plan (LWMP) be in place, then a facility is exempt from the BCEA process. The LWMP process will be discussed further below.

4.1.2 FEDERAL REGULATIONS

The WSER indicates the following effluent requirements. There are no requirements for maximum flow, with the flow determining the monitoring and reporting frequency. Flows above 2,500 m^3/d also trigger the requirement for undertaking toxicity testing (96-hour rainbow trout bioassay). Toxicity testing is currently undertaken once annually in order to meet the requirements of WSER.

- $CBOD_5$: 25 mg/L average.
- TSS: 25 mg/L average.

- Un-ionised ammonia: 1.25 mg/L maximum.
- Total chlorine residual: 0.02 mg/L maximum.

Unlike the MWR, there are no requirements for an authorisation approval from Environment Canada.

4.2 EFFLUENT DISPOSAL/RECLAIMED WATER USE

Once sanitary sewage is treated there are two streams that must be managed:

- Treated effluent
- Biosolids

Management of biosolids will be addressed in a different section. Treated effluent must be returned to the environment or may be reclaimed and used for purposes such as irrigation, habitat enhancement, dust control, or industrial uses. Given Nelson is situated next to Kootenay Lake, in a location with significant flow, it is assumed that Kootenay Lake will continue to be the primary option for managing the treated effluent. Additional options for management of treated effluent are:

- Disposal to ground.
- Reclaimed water use

Disposal to ground at this scale would require rapid infiltration basins (RIB) with an overall area of approximately 35,000 m^2 (assuming 75 m/y infiltrative capacity), or approximately 190 x 190 m, separated into at least 4 RIBs. The site of RIBs must be underlain with free draining, granular soils, and there are several additional requirements with respect to groundwater depth, movement, and travel time. It is our understanding that no suitable site is currently available near the Grohman Narrows facility or within the City. Consequently, for the purposes of this study it is assumed that disposal to Kootenay Lake will remain the primary option.

Reclaimed water use requires that a suitable user is located within practical distance to the source of the treated water. Irrigation is the most common use in BC for reclaimed water. There are a number of other potential uses including dust control, equipment washing, process/industrial water, wetland enhancement and stream augmentation, with the different uses requiring different levels of treatment and disinfection. Where reclaimed water use is to be implemented, the MWR still requires another disposal option to be permitted and installed, unless the release is to a wetland or is mainly industrial in nature, with the intent being that wastewater flows can be terminated if there is an issue. Given there are no suitable end users near the Grohman Narrows site, and the road is not suitable for bulk water hauling, the current site is not considered suitable for reclaimed water use. Opportunities for reclaimed water use may be possible at an alternate site. Nevertheless, because an alternate disposal method is still required by regulation, for the purposes of this study it is assumed that management of treated effluent will be disposal to Kootenay Lake in the first instance, with reclaimed water use opportunities explored as they arise.

4.3 EXISTING SYSTEMS – GROHMAN NARROWS PCC

Prior to the 2005/2006 upgrades, the City of Nelson wastewater treatment plant was a primary level facility, consisting of the primary settlement of domestic wastewater and chlorination before discharge to the Grohman Narrows, a fast-flowing section of the west arm of Kootenay Lake. The plant was upgraded to a secondary standard in 2005/2006, with the addition of the following items:

- The conversion of existing infrastructure to an aerated equalisation tank;
- The addition of four rotating biological contactors, in two separate treatment trains;
- Secondary clarifiers;
- Disinfection through the addition of two banks of ultra-violet (UV) lights, which replaced the chlorination system; and,
- An outfall to the thalweg of Grohman Narrows.

Additional upgrades for the dewatering of sludge were completed in 2011 and consisted of replacing the belt press with a centrifuge. The City's sludge is treated by mesophilic anaerobic digestion to produce a Class B biosolids before it is dewatered and transported offsite for disposal.

Since the upgrades, there have been several changes in the regulatory framework for domestic wastewater treatment. In 2012, the BC Municipal Sewage Regulation (MSR) was repealed and replaced with the BC MWR. Also in 2012, the Federal Wastewater Systems Effluent Regulations (WSER) was introduced into law.

The primary concern with the PCC is its ability to consistently meet the requirements of the Federal WSER for $CBOD_5$. The regulation requires that quarterly average $CBOD_5$ effluent concentration be 25 mg/L or less. **Table 16** and **17** summarize TSS and $CBOD_5$ compliance with the WSER, respectively. CBOD5 is almost consistently out of compliance. In 2017, this average was only met in the second quarter. However, the ability to meet the WSER effluent TSS requirement of 25 mg/L as a quarterly average was also raised as a concern. In 2017, this average was only met in the second and third quarters. In both cases, the average concentration was calculated to be 25 mg/L, which is on the threshold of non-compliance.

Table 15: Average Effluent TSS Concentration (mg/L) and WSER Compliance

Time Period	2018	2019	2020	2021	2022
1st Quarter	36.3	26.8	40.1	19.1	35.9
2nd Quarter	24.0	19.6	24.4	18.8	-
3rd Quarter	18.1	17.8	27.5	34.4	-
4th Quarter	18.4	18.9	33.2	14.7	-

Table 16: Average Effluent $CBOD_5$ Concentration (mg/L) and WSER Compliance

Time Period	2018	2019	2020	2021	2022
1st Quarter	42.1	49.9	43.8	31.7	34.2
2nd Quarter	32.9	23.8	31.4	31.0	-
3rd Quarter	26.1	24.0	41.7	54.2	-
4th Quarter	38.2	27.9	61.3	33.1	-

The City of Nelson wastewater treatment plant operates under the BC Ministry of Environment Permit Pollution Control Permit PE-291. The wastewater treatment plant is a secondary treatment facility and consists of the following processes:

- A headworks facility with a mechanical screen, manual bypass channel, aerated grit tank, and grit classifier;
- Two parallel primary clarifiers with scum removal;
- Two parallel aerated equalization tanks with two low-lift pumps (one pump per tank);
- Four rotating biological contactors (RBC), two trains of two RBCs in series;
- Two parallel secondary clarifiers with inclined plate settlers for enhanced sedimentation;
- An ultraviolet (UV) disinfection system;
- Two high-rate anaerobic digesters in series;
- A centrifuge; and,
- An emergency backup generator.

In 2018, Urban completed a Pollution Control Centre Upgrade Assessment (Urban Systems, 2018) of the secondary treatment components of the City's PCC. The assessment reviewed capacity of each individual process within the plant as well as provide recommendations for priority upgrades and estimates of their associated capital and O&M costs.

A review of the PCC determined that a number of the major treatment processes are currently at, or have exceeded, their rated capacity, which would lead to poor treatment performance and very challenging operation and maintenance conditions. The major processes that require capacity upgrades include: primary clarifiers, equalization tanks, secondary treatment (RBCs) and secondary clarifiers. Other supporting systems were also determined to be at capacity, including the electrical service, emergency backup generator and headworks screen. The UV system was also found to not meet the MWR reliability requirements and should be upgraded.

The scope of the 2018 assessment was restricted to the secondary treatment components of the PCC; it must be noted that the remaining components of the PCC have been in place for many years and will be in need of renewal/replacement in the coming years.

The PCC is biologically overloaded with several unit processes nearing or exceeding their hydraulic capacities. Influent wastewater strength and $CBOD_5$ loading is higher than what was projected at the 2006 upgrade. Due to the biological overload, there have been instances of effluent water quality exceeding the MWR discharge criteria of 45 mg/L $CBOD_5$. Lastly, the Federal Wastewater System Effluent Regulation brought into effect in 2012 set more stringent effluent quality criteria for discharges to surface water at 25 mg/L $CBOD_5$ and TSS, as quarterly averages.

The 2018 assessment recommended the following sequence of phased upgrades:

- Phase 1 – Detailed Design and Construction of Electrical and Emergency Generator Upgrades
- Phase 2 – Detailed Design and Construction of a New Headworks
- Phase 3 – Detailed Design and Construction of a New Primary Treatment Process (Mechanical Primary Screens)

- Phase 4 – Detailed Design and Construction of a New Secondary Treatment Process (MBBR) and Secondary Clarification Process (DAF)
- Phase 5 – UV Upgrades

It was also recommended that the City complete a sludge management study to remediate and upgrade the anaerobic digestion. A possible outcome of this study would be upgrading the facility's sludge management and solids dewatering system and removing digestion. A receiving bay at the plant could be constructed for the dewatered sludge to be trucked to a composting facility.

In 2021, CWMM completed a structural condition assessment of the existing PCC. No major upgrades were flagged for immediate remediation. However, given that portions of the facility are 50 years old and exceed the design life of the building, consideration should be made in decisions regarding expansions / upgrades of the existing facility versus a new facility.

4.4 ALTERNATE SEWAGE TREATMENT PLANT OPTION

Through discussions with the City, the general location of 70 Lakeside Drive, the adjacent City ROW, and portions of 80 Lakeside Drive (the City's Public Works Yard) was identified as a potential alternate sewage treatment site and assessed utilizing a set of multiple bottom-line parameters. There are additional sites that may be considered in future stages, but the purpose of this exercise was to determine whether a new site, in the vicinity of the public works yard, or the existing is preferable in the long-term.

Given the age, condition, and proposed upgrades required for the existing PCC, it is imperative that alternate solutions be critically explored to help realize a path which the City can direct their resources. As such, a new sewage treatment plant on an alternate site was considered. For the purposes of this study, a common secondary biological treatment process utilizing Sequencing Batch Reactors (SBRs) was assumed for determining the cost and footprint feasibility of the alternate site. The system used for comparison includes the following unit processes:

- Influent forcemain from existing Airport Lift Station
- Headworks including screening and grit removal
- Equalisation tank
- Sequencing batch reactors
- UV disinfection
- Sludge dewatering

A complete feasibility study would be required to select the actual preferred treatment systems for a new facility, but the system described here would work well and is useful as a representative system to understand the footprint and comparative cost of a new treatment facility.

4.5 TREATMENT PLANT OPINIONS OF PROBABLE COST

The opinion of probable capital cost to upgrade the Grohman Narrows is approximately $38.5M. The airport lift station marine forcemain upgrade cost opinion of $8,430,000 identified in **Section 3.1** is included in this estimate. This upgrade cost would be eliminated if the potential alternate location was selected. The opinion of probable capital cost to construct a new facility near the public works yard is approximately $37.2M. The costs presented here are planning level costs. More detailed cost estimates require significantly more engineering which is not appropriate or cost effective at this stage.

4.6 TREATMENT PLANT DECISION MATRIX

Five project value categories were selected based on City input and previous assessments Urban has conducted, that generally assess all parameters affecting the decision. The City assigned each category a rating range, or weight, based on their perceived importance and gave a score (positive or negative) based on the impact of the given category relative to the base case. The intent is the averaged sum of the scores will provide a positive value (further pursue the alternate site) or negative value (remain at current site).

City staff provided input to complete the decision matrix below. Based on the evaluation completed, a positive score of 9.67 indicates that the proposed site represents a possible alternate treatment facility location. The scores are summarized in **Table 17** and **Figure 13**.

Table 17: Alternate Site Decision Matrix

Project Value Category	Rating Range	Score Given	Average Score
Site Considerations	±5	-2,3,5	2
Environmental / Social	±5	-2,3,0	0.33
Capital Cost	±3	1,0,0	0.33
Operational Cost	±5	4,3,3	3.33
Operations and Maintenance	±4	3,4,4	3.67
Effect on Decision (sum)		**4,13.12**	**+9.67**

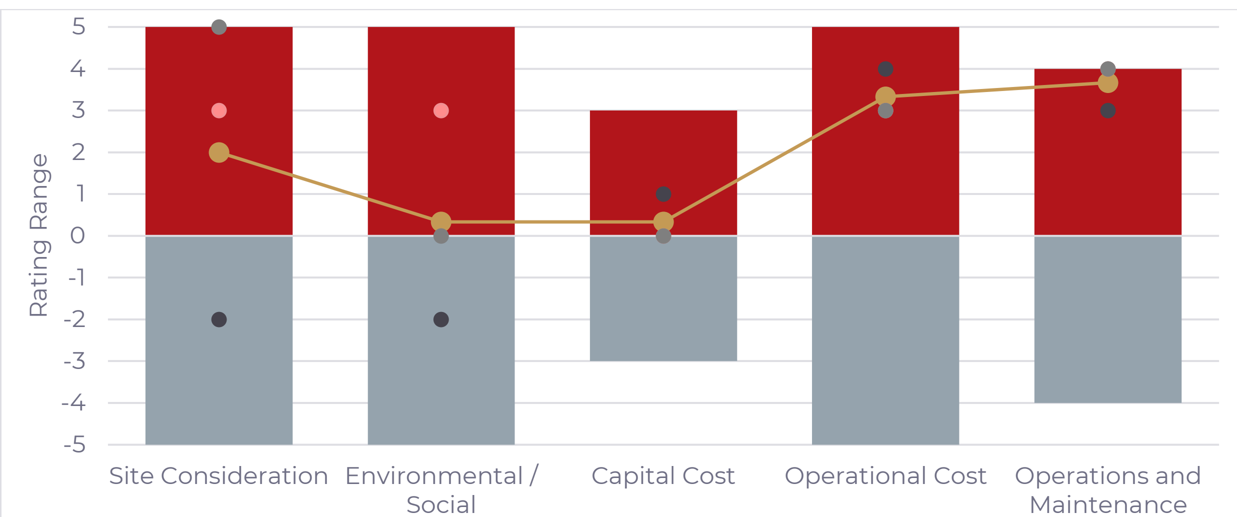


Figure 14: Alternate Site Decision Matrix Scoring

The design and use of a new treatment facility on a larger site could lend itself to a safer, more reliable plant with better functionality.

Ultimately, the new treatment facility will need to manage the City's current high strength loadings in an efficient and robust operation to meet treated effluent quality standards. The potential alternative site would allow easier access to equipment. A new facility would correct the safety deficiencies of the existing PCC by ensuring separation between occupied spaces and headworks, restricting confined spaces, and design better access for emergency vehicles.

4.7 NEXT STEPS

A new facility will require registration, monitoring, and reporting under the BC MWR. An EIS will be required as part of the registration. Registration under the Federal WSER would be required, unlike the MWR, there are no requirements for an authorization approval from Environment Canada.

Because the facility will serve more than 10,000 people a BC Environmental Assessment (EA) would be triggered *unless* the City were to complete a LWMP. A LWMP is a process that allows a community to meet the intent of the MWR over a defined period of time and incorporates the additional consultation, economic, environmental and technical studies that are required. Importantly, unlike the BC EA process, the City would be in control of the LWMP process.

This study has identified various needs, obstacles, and opportunities for long term wastewater management in Nelson. Implementation of a preferred option will require further study, engineering, permitting, public/stakeholder consultation, First Nations consultation, BC Environmental Assessment and financing. It is recommended that the City undertake a LWMP plan to address all of these issues under one project. A successful LWMP would incorporate the following:

- Technical studies
- Public/stakeholder consultation
- Environmental Impact Study (An EIS is not the same as an EA)
- Options assessment and selection of preferred pathway
- Exemption from BC EA process
- Regulatory input
- Regulatory approval
- Borrowing approval (no additional petition/counter petition process is required for borrowing with an approved LWMP)

The process of developing a LWMP emerges from the BC Environmental Management Act. The intent is to allow local governments a means to achieve community support for sanitary services. By engaging the public, decision makers, technical representatives, and the Ministry of Environment, a completed LWMP can be confidently endorsed as technically rigorous and with the support of the public. An LWMP that has undergone the public process and achieved both the local government approval and the Minister's approval effectively becomes a legal document (much like an Official Community Plan) that sets the stage for long-term management of liquid waste. By including financial considerations within the selected liquid waste management scenario, the City is able and expected to carry out projects and cost-recovery methods as developed during the plan.

5.0 PART 4 – BIOSOLIDS

5.1 INTRODUCTION

Terms "sludge" and "biosolids" are both used when describing the excess solids which are produced at a domestic wastewater treatment plant. For the purpose of this biosolids management plan, the following definitions will be used:

Sludge	The excess organic solids which are produced as a result of treating liquid wastes. These organic solids have not been treated by any recognised solids treatment process in order to produce biosolids. Therefore, the health and environmental risks associated with sludge can be high.
Biosolids	Excess organic solids which have been treated to achieve vector attraction reduction (e.g., flies, birds, rodents, etc.) and a reduction in pathogen concentrations. The treatment of sludge to produce biosolids can result in a final product which has low risks to human health and the environment.

5.2 REGULATORY FRAMEWORK

5.2.1 INTRODUCTION

The BC Organic Matter Recycling Regulation (OMRR) governs the production, quality, and land application of specific types of organic matter, including municipal wastewater sludge and biosolids. There are three aspects to the OMRR: treatment, quality and uses, which are discussed in greater detail below.

There is no Federal regulation for sludge/biosolids. However, the development of the Biosolids Management Strategy in 2012 through the Canadian Council of the Ministers of the Environment clearly indicates that the policy throughout Canada is to encourage the development and use of biosolids, rather than the disposal of what has the potential to be a valuable resource.

5.2.2 TREATMENT

The OMRR outlines treatment requirements, focusing on two aspects: pathogen reduction and vector attraction reduction. Pathogen reduction is the decrease in micro-organisms which may be present in the human gut and have the potential to cause illness or disease. Vector attraction reduction, or stabilisation, is the transformation of organic matter into a state where there is a lower potential for nuisance conditions (e.g., odour, attracting flies, etc.) to occur.

The requirements for pathogen reduction are outlined in Schedule 1 of the OMRR and are based on a temperature-time relationship for the destruction of enteric micro-organisms. The temperature-time relationship allows for either short periods of time when the material is exposed to elevated temperature or long periods of time when the material is exposed to low or ambient temperatures. The higher quality biosolids products all require a period of elevated temperature (i.e., ≥ 50 °C), while the lower quality biosolids products only require low or ambient temperature conditions. The City's current pathogen reduction process occurs under temperature condition that are lower than 50 °C.

Acceptable vector attraction reduction methods are outlined in Schedule 2 of the OMRR. There are a number of acceptable methods by which vector attraction reduction can be achieved and, unlike the pathogen reduction processes, there is little difference between the higher and lower biosolids products. The City's current vector attraction reduction process is anaerobic digestion.

5.2.3 QUALITY

Under the OMRR, organic matter is separated into five different categories:

- Class A compost;
- Class B compost;
- Class A biosolids;
- Class B biosolids; and,
- A biosolids growing medium.

Table 18 summarises the quality of the 5 organic products, as defined by the OMRR. A biosolids growing medium is the highest quality product and can be used in place of a soil. The other organic products (Class A compost, Class B compost, Class A biosolids and Class B biosolids) are all intended to be used as a soil amendment to enhance the soil nutrient content. The quality criteria for a biosolids growing medium, compost and a Class B biosolids are stated clearly in the OMRR. The quality criteria for Class A biosolids are not stated clearly in the OMRR but are indicated to be based on the Federal requirements stated in the Trade Memorandum T-4-93.

Table 18: Summary of Material Quality Under the BC OMRR

Parameter	Medium Type				
	Biosolids Growing Medium	Class A Compost	Class B Compost	Class A Biosolids	Class B Biosolids
Foreign Matter Content (% dry weight)	< 1	< 1	<1	<1	< 1
Sharp Foreign Matter	None present	None present	None present	None present	None present
Carbon to Nitrogen Ratio (C:N)	> 15:1	≥ 15:1 and ≤ 35:1	N/A	N/A	N/A
Total Kjeldahl Nitrogen (% by weight)	< 0.6	N/A	N/A	N/A	N/A
Organic Matter Content (% dry weight)	≤ 15	N/A	N/A	N/A	N/A
Faecal Coliforms (MPN/g dry weight)	< 1,000	< 1,000	< 2,000,000, with limits < 1,000 for certain land applications	< 1,000	< 2,000,000, with limits < 1,000 for certain land applications

Table 18: Summary of Material Quality Under the BC OMRR (continued…)

Parameter	Medium Type				
	Biosolids Growing Medium	Class A Compost	Class B Compost	Class A Biosolids	Class B Biosolids
Maximum Element Concentration (µg/g dry weight)					
Arsenic	13	13	75	75	75
Cadmium	1.5	3	20	20	20
Chromium	100	100	1,060	1,060	1,060
Cobalt	34	34	150	151	150
Copper	150	400	2,200	757	2,200
Lead	150	150	500	505	500
Mercury	0.8	2	15	5	15
Molybdenum	5	5	20	20	20
Nickel	62	62	180	181	180
Selenium	2	2	14	14	14
Zinc	150	500	1,850	1,850	1,850

5.2.4 USES

Under the OMRR, the intent is that organic matter can be used to enhance vegetation or plant growth. The acceptable uses include application to agricultural lands for crop growth, land reclamation and use in urban settings, which can include use of these materials in residential gardens. The quality of the organic matter provides direction on the use. There are no restrictions for the distribution and use of biosolids growing medium and a Class A compost, which are highly treated and have a high quality. However, there are restrictions for a Class A biosolids, a Class B biosolids and a Class B compost. These restrictions are outlined below.

A Class A biosolids can be distributed/used as follows:

- In volumes below 5 m^3 per vehicle per day.
- In sealed bags of < 5 m^3 for retail purposes, with there being no restrictions on how many bags can be distributed per vehicle per day.
- In volumes > 5 m^3 to composting facilities or biosolids growing medium facilities.
- In volumes > 5 m^3 per parcel of land per year in accordance with a land application plan (LAP).

A Class B biosolids can be used as follows:

- For the enhancement of vegetation under a LAP. This plan needs to consider application methods and soil substance concentrations.
- For composting, with no volume restrictions.
- To develop a biosolids growing medium as long as pathogen reduction and vector attraction reduction requirements are met. There are no restrictions on the volume that can be diverted to create a biosolids growing medium.

A Class B compost can only be used for the enhancement of vegetation under a LAP. As with a Class B biosolids, the LAP needs to consider the application methods and the soil substance concentrations.

5.2.5 ADDITIONAL REGULATORY CONSIDERATIONS

The OMRR requires permits to be issued for compost facilities that process food wastes or biosolids and have an annual design production capacity of 5,000 tonnes or more of compost. The permit is received through an application to the BC Ministry of Environment and Climate Change Strategy (ENV). Further updates to the OMRR are expected and are currently in the discussion phase. The timing of implementation of any changes is not known at the time of developing this report.

5.3 PCC BIOSOLIDS

5.3.1 PRODUCTION

Sludge is wasted from the primary and secondary clarifiers to the anaerobic digesters, where it is kept for a period of time before being dewatered by centrifuge. Once dewatered, the biosolids are stored temporarily in a container before being shipped to the Regional District of Central Kootenay (RDCK) landfill. **Table 19** summarizes the estimated annual biosolids production (as dewatered cake) from 2017 to present.

Table 19: Estimated Annual Biosolids Production

Year	Dewatered Biosolids (metric tonnes)
2017	362,120
2018	347,670
2019	360,460
2020	293,040
2021	358,900
2022*	254,460
Average	***343,578***

* Recorded until the end of September

5.3.2 QUALITY

Samples to assess the quality have been taken over the years. The available data were reviewed with respect to the following parameters:

- Solids content
- Pathogen content
- Metal content
- Hydrocarbon content

In addition to the above parameters, the OMRR also has requirements for foreign matter in terms of presence of general foreign matter (< 1% dry weight) and sharp foreign matter (non-present). There are no data available for the City's biosolids for foreign matter. For sites where the headworks allows for screening and removal of plastics, there tends to be low concern with foreign matter being present in the biosolids.

Data from 2017 and 2018 indicate that the solids content after dewatering is around 24 to 30%, which is higher than typical for domestic wastewater biosolids. The higher solids content has a number of benefits including less risk of leachate production, lower trucking costs, lower tipping fees and increased ease of management at an application or processing site. Continuing to produce a dewatered cake in a higher solids content range would be advised.

More recent data indicate that the faecal coliform concentration was in the order of 360,000 MPN/g dry weight, with historical data indicating a range between approximately 4,000 to 40,000 MPN/g dry weight. In all cases, the concentrations represent a Class B quality as defined by the OMRR (< 2,000,.000 MPN/g dry weight). Although it may be possible to achieve a lower faecal coliform concentration, should the concentration be below 1,000 MPN/g dry weight, the biosolids would only remain in the OMRR Class B category due to the lack of elevated temperature during processing.

Table 20 summarises the quality of recent samples with the metal requirements in the OMRR. Although the quality of some parameters is high and even meets the quality for a biosolids growing medium, this is not the case for all parameters. Cadmium does not meet the requirements for a biosolids growing medium. Molybdenum, selenium and zinc do not meet the requirements of a Class A compost or a biosolids growing medium. Mercury does not meet the requirements of a biosolids growing medium and is on the borderline for meeting the requirements for a Class A compost. The ability for the City's biosolids to meet criteria set for a growing medium or a compost is not overly of concern, as both of these organic products require blending with other materials and further processing. The main focus is whether the City's biosolids are able to meet a Class B biosolids quality, as these are the criteria which would be used when determining options for the biosolids and their suitability for applying to land to enhance plant growth.

Table 20: Comparison of Data with OMRR Metal Criteria

<table>
<tr><th rowspan="2">Parameter</th><th colspan="2">Concentration</th><th colspan="5">OMRR Medium Type (Lowest Quality to Highest Quality)</th></tr>
<tr><th>2017</th><th>2018</th><th>Class B Biosolids</th><th>Class B Compost</th><th>Class A Biosolids</th><th>Class A Compost</th><th>Biosolids Growing Medium</th></tr>
<tr><td colspan="8">Metal Concentration (µg/g dry weight)</td></tr>
<tr><td>Arsenic</td><td>1.93</td><td>2.16</td><td colspan="5">√</td></tr>
<tr><td>Cadmium</td><td>2.1</td><td>2.32</td><td colspan="4">√</td><td>X</td></tr>
<tr><td>Chromium</td><td>25.5</td><td>23.4</td><td colspan="5">√</td></tr>
<tr><td>Cobalt</td><td>3.95</td><td>4.84</td><td colspan="5">√</td></tr>
<tr><td>Copper</td><td>387</td><td>421</td><td colspan="5">√</td></tr>
<tr><td>Lead</td><td>45.2</td><td>47.7</td><td colspan="5">√</td></tr>
<tr><td>Mercury</td><td>1.61</td><td>2.10</td><td colspan="3">√</td><td colspan="2">X</td></tr>
<tr><td>Molybdenum</td><td>6.51</td><td>6.85</td><td colspan="3">√</td><td colspan="2">X</td></tr>
<tr><td>Nickel</td><td>16.4</td><td>17.5</td><td colspan="5">√</td></tr>
<tr><td>Selenium</td><td>3.97</td><td>4.43</td><td colspan="3">√</td><td colspan="2">X</td></tr>
<tr><td>Zinc</td><td>779</td><td>976</td><td colspan="3">√</td><td colspan="2">X</td></tr>
</table>

√ = conforms to the corresponding OMRR quality classification

X = does not conform to the corresponding OMRR quality classification

The analyses for hydrocarbons were undertaken through direction set by the RDCK, with the request being a comparison with the BC Contaminated Sites Regulation (CSR) for the industrial lands soil quality standards. Data were reviewed in 2018 for a sample taken on August 1st, 2018. The data review indicated that most of the hydrocarbon parameters were below the analytical detection limit. Where data were above the analytical detection limit, the measured concentration was below the CSR standard.

5.3.3 TREATMENT

The OMRR outlines treatment requirements, focusing on two aspects: pathogen reduction and vector attraction reduction. With respect to pathogen reduction, as indicated above, the faecal coliform concentrations for the dewatered biosolids are consistent with the requirements of a Class B biosolids quality, which would be expected given that the anaerobic digestion process does not occurs under elevated temperature conditions.

The OMRR indicates that the vector attraction reduction should be at least 38%. Data from early assessments indicate that the vector attraction reduction was in the order of 63%. These data indicate that there is a high potential for sufficient vector attraction reduction using the anaerobic digesters. However, vector attraction reduction can change based on operational conditions. It is recommended that the vector attraction reduction be confirmed at least once annually, with the potential for increased monitoring if there are seasonal variations in operations.

5.4 SUMMARY – COMPLIANCE WITH THE OMRR

From the available data, the City's biosolids meet the Class B criteria for both treatment and quality. It is reasonable to assume that this would continue in the future, however, regular sampling will confirm that this is the case. In the event that the quality changes to meet a higher criterion, the City's biosolids would still need to be managed in accordance with a Class B quality, due to the lack of elevated temperature conditions for pathogen treatment.

5.5 BIOSOLIDS MANAGEMENT OPTIONS

There are the following management options available for the City's biosolids:

- Continued trucking to the RDCK landfill. This requires on-going agreement and acceptance from the RDCK. Trucking costs and tipping fees would apply.
- Land application to enhance vegetation growth. This would need to be under the OMRR through a Land Application Plan. As the quality only meets Class B biosolids, the application is likely to be limited to lands where there are access restrictions. Enhancement of agricultural lands is the most common approach for the land application of a Class B biosolids.
- Further processing. This could result in the development of a Class A product, which has the potential to be distributed without any restrictions or authorisations under the OMRR. Further processing is commonly in the form of composting to produce a Class A compost and also provide opportunity for the amalgamation with other organic wastes.

These options are the same for both the current and future biosolids production. It should also be noted that in the case of further processing by composting, there could be options to abandon the anaerobic digesters, as digestion may have a negative effect on the ability for the organic matter to reach the elevated temperatures to prove pathogen reduction. In addition, there is pressure from both the Federal and Provincial governments with respect to the disposal of biosolids to landfill, with the direction being to encourage reuse.

The production of energy through combustion can be raised as a potential option. This approach requires a significant volume of organic matter in a combustible form (i.e. low moisture content) to be viable. With the current status on combustion, it is reasonable to assume that there would not be sufficient product from the City's wastewater treatment plant for this option to be economically viable.

5.6 RECOMMENDATIONS

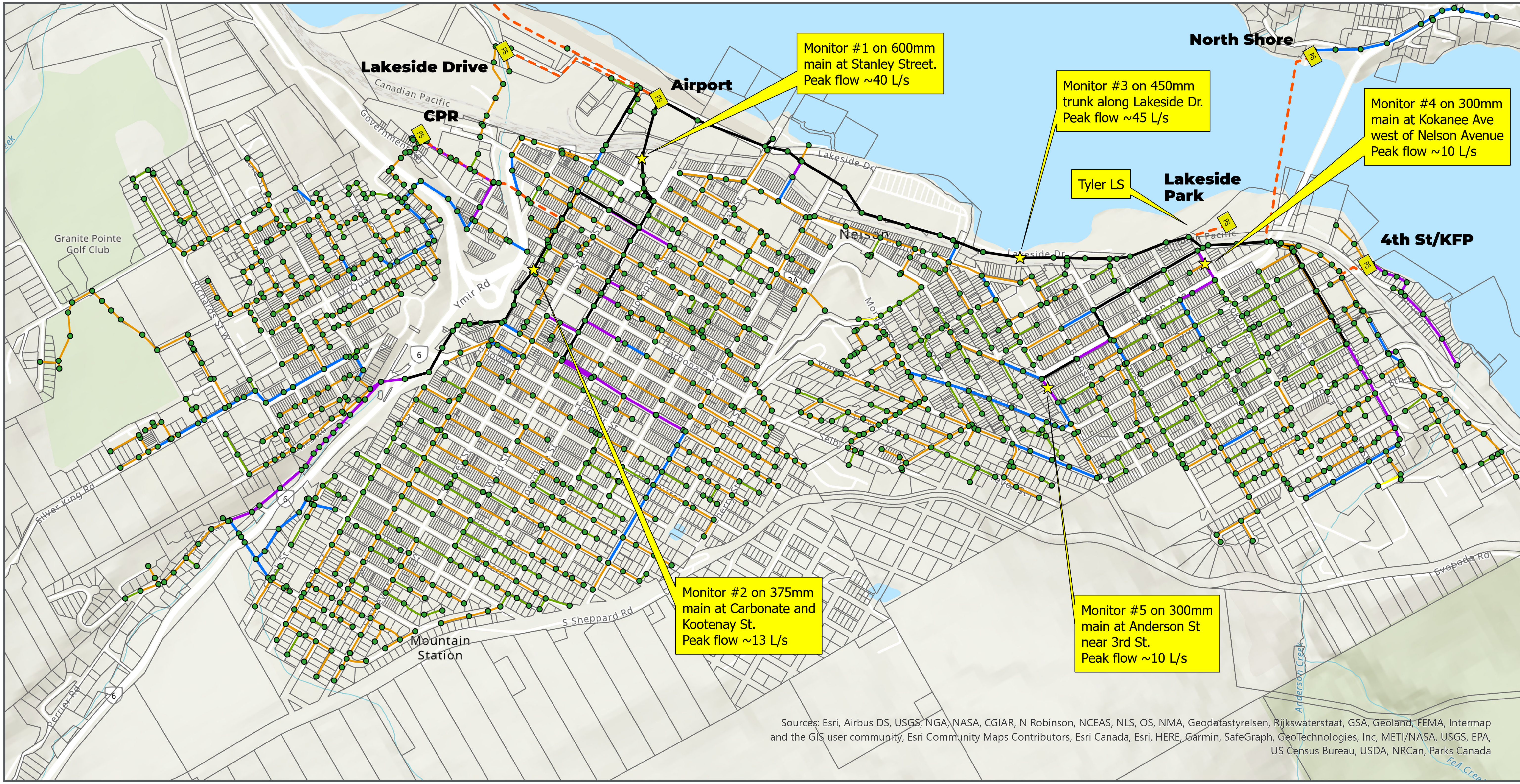
The following are recommended:

1. Undertake regular analyses to allow an on-going understanding of the status of the biosolids quality and treatment in the context of the OMRR and the requirements of the RDCK.
2. Review the updates to the OMRR once they are available to provide direction to the City on emerging options and restrictions to management approaches.
3. Once a greater understanding of the future direction for the wastewater treatment options is available, undertake a more comprehensive review of the biosolids management options which could be available.

6.0 REPORT FIGURES

This section contains Figures 1 through 12.

Figures 13 and 14 are embedded in the report in Sections 4.0 and 5.0, respectively.



URBAN systems

Project #: 0795.Clientinfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 6 / 10

200 100 0 200 400 600
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

Model - Pipes
- Gravity, 100mm
- Gravity, 150mm
- Gravity, 200mm
- Gravity, 250mm
- Gravity, 300mm
- Gravity, 375mm or larger
- Forcemains

- Flow Monitors
- Model - Manholes
- Model - Lift Stations

Notes

Peak flow observed during rainfall event on August 22, 2021.

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 01
Sanitary Sewer System and Flow Monitoring Locations

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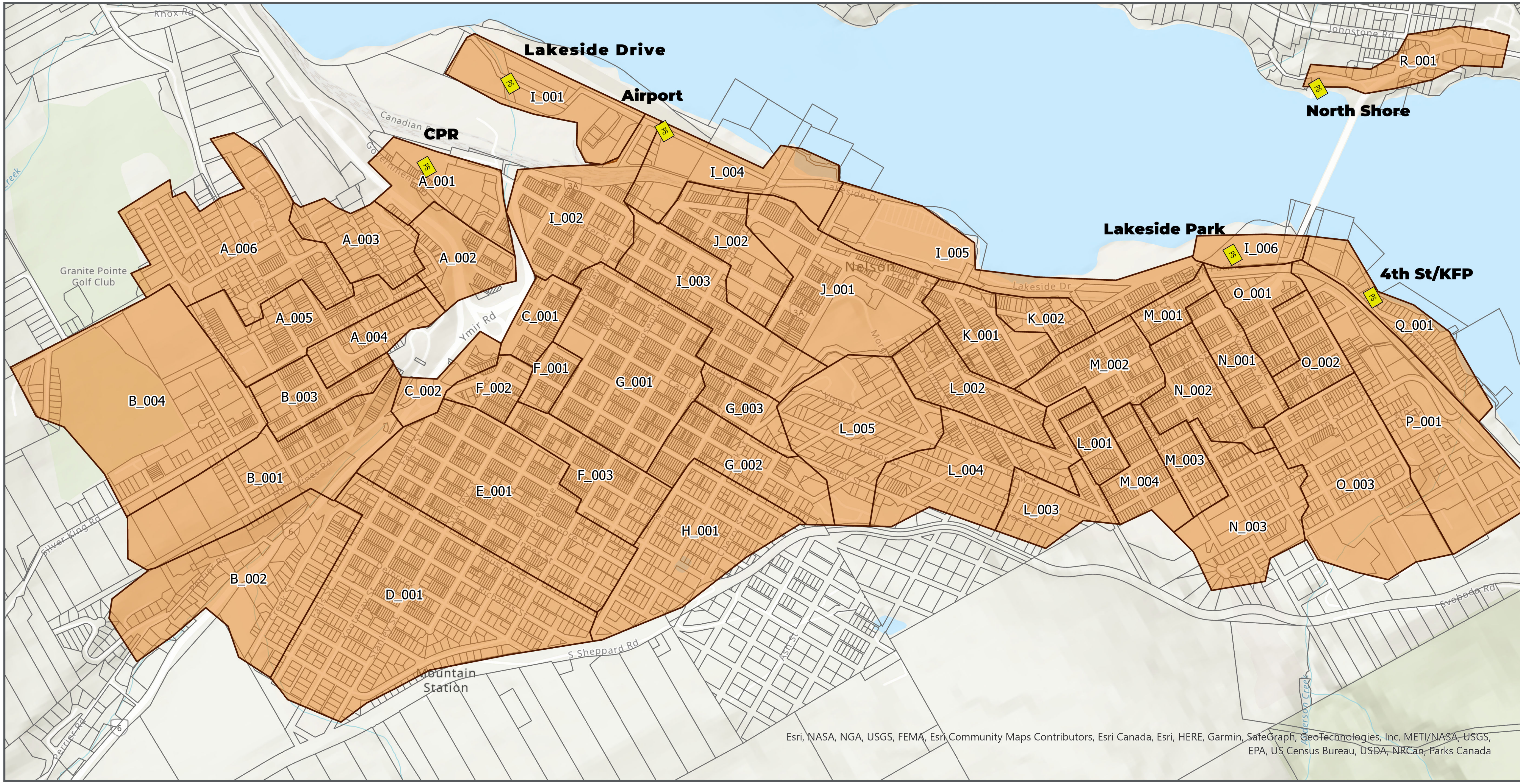

URBAN systems

200 100 0 200 400 600
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

Sanitary Sewer Catchment

Model - Lift Stations

Notes

Existing domestic sewer loading allocated to nearest manhole in the model. The sewer catchments shown are utilized for rainfall-derived infiltration & inflow and are assigned to the model node near the centroid of the catchment.
Refer to Figure 4 to view growth estimates and the allocation points for future loadings.

City of NELSON

City of Nelson
Wastewater Management Plan
FIGURE 02
Sanitary Sewer Catchments

Project #: 0795.Clientinfo
Author: JB
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Status: Draft
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URBAN systems

Project #: 0795.ClientInfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 1 / 13

200 100 0 200 400 600
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

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PS Model - Lift Stations

Sewer Main Installation Date
- 1912 - 1999
- 2000 - 2019
- 2020
- Lined Sewer Mains (2000-2020)

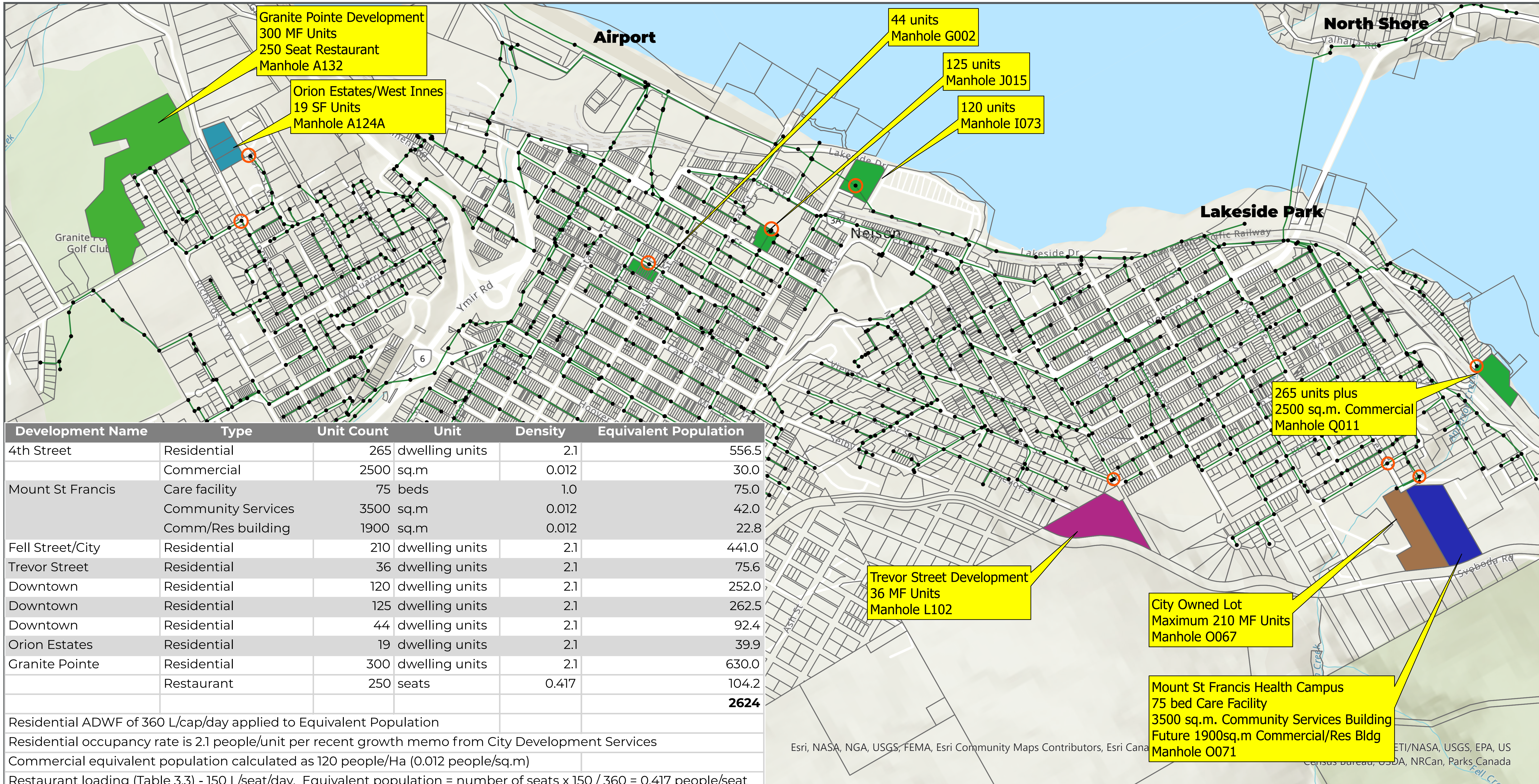
Notes

Refer to Figure 7 for estimated remaining service life of the sanitary mains

City of **NELSON**

City of Nelson
Sanitary Sewer System
FIGURE 03
Sewer Main Construction 2000 - 2020

U:\Projects_KEL\0795_ClientInfo\Models\Sanitary\GIS\GIS\Projects\Pro_Projects\Pro_Projects.aprx Last updated by jbarta on January 13, 2022 at 9:05 AM



Development Name	Type	Unit Count	Unit	Density	Equivalent Population
4th Street	Residential	265	dwelling units	2.1	556.5
	Commercial	2500	sq.m	0.012	30.0
Mount St Francis	Care facility	75	beds	1.0	75.0
	Community Services	3500	sq.m	0.012	42.0
	Comm/Res building	1900	sq.m	0.012	22.8
Fell Street/City	Residential	210	dwelling units	2.1	441.0
Trevor Street	Residential	36	dwelling units	2.1	75.6
Downtown	Residential	120	dwelling units	2.1	252.0
Downtown	Residential	125	dwelling units	2.1	262.5
Downtown	Residential	44	dwelling units	2.1	92.4
Orion Estates	Residential	19	dwelling units	2.1	39.9
Granite Pointe	Residential	300	dwelling units	2.1	630.0
	Restaurant	250	seats	0.417	104.2
					2624

Residential ADWF of 360 L/cap/day applied to Equivalent Population

Residential occupancy rate is 2.1 people/unit per recent growth memo from City Development Services

Commercial equivalent population calculated as 120 people/Ha (0.012 people/sq.m)

Restaurant loading (Table 3.3) - 150 L/seat/day. Equivalent population = number of seats x 150 / 360 = 0.417 people/seat

URBAN systems

Project #: 0795.ClientInfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 1 / 13

200 100 0 200 400 600 Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

Development loads assigned to model node as shown with red circle. Remainder of growth in each scenario (2041/Buildout) achieved by uniformly scaling existing loads of all other nodes in the model.

Population Allocation	Existing	2041	Buildout
Growth areas on Figure 4	n/a	2,624	2,624
Distributed across remainder of system	11,222	400	10,630
Total growth	n/a	3,024	13,254
Total population	11,222	14,246	24,476

Notes

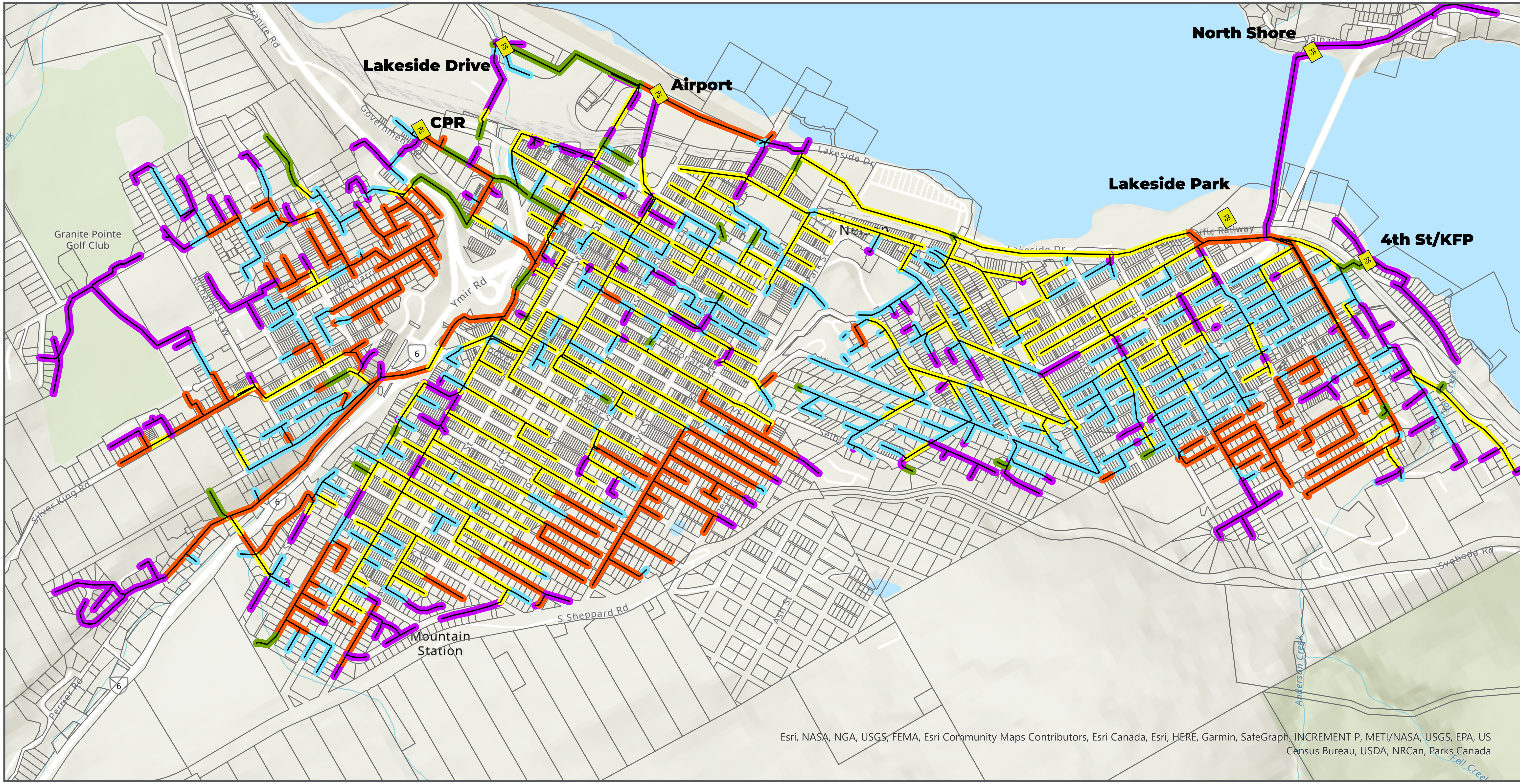
Development Areas

- City Lot near Health Campus
- Fell Street Development
- Granite Pointe
- Orion Estates West Innes
- Trevor Street
- Unnamed

City of NELSON

City of Nelson
Wastewater Management Plan
FIGURE 04
Sanitary Sewer Loading Allocation

U:\Projects_KEL\0795_ClientInfo\Models\Sanitary\GIS\GIS\Projects\Pro_Projects\Pro_Projects.aprx Last updated by jbarta on January 13, 2022 at 9:32 AM



URBAN systems

Project #: 0795.Clientinfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 1 / 13

200 100 0 200 400 600
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

PS Model - Lift Stations

Sewer Main Material/Service Life
- 50 years: CI; CMP; DI; Unknown
- 60 years: INSITU; INSTU; insitu
- 60 years: VCT; VIT
- 70 years: AC; CONC
- 80 years: HDPE V; PE; PVC

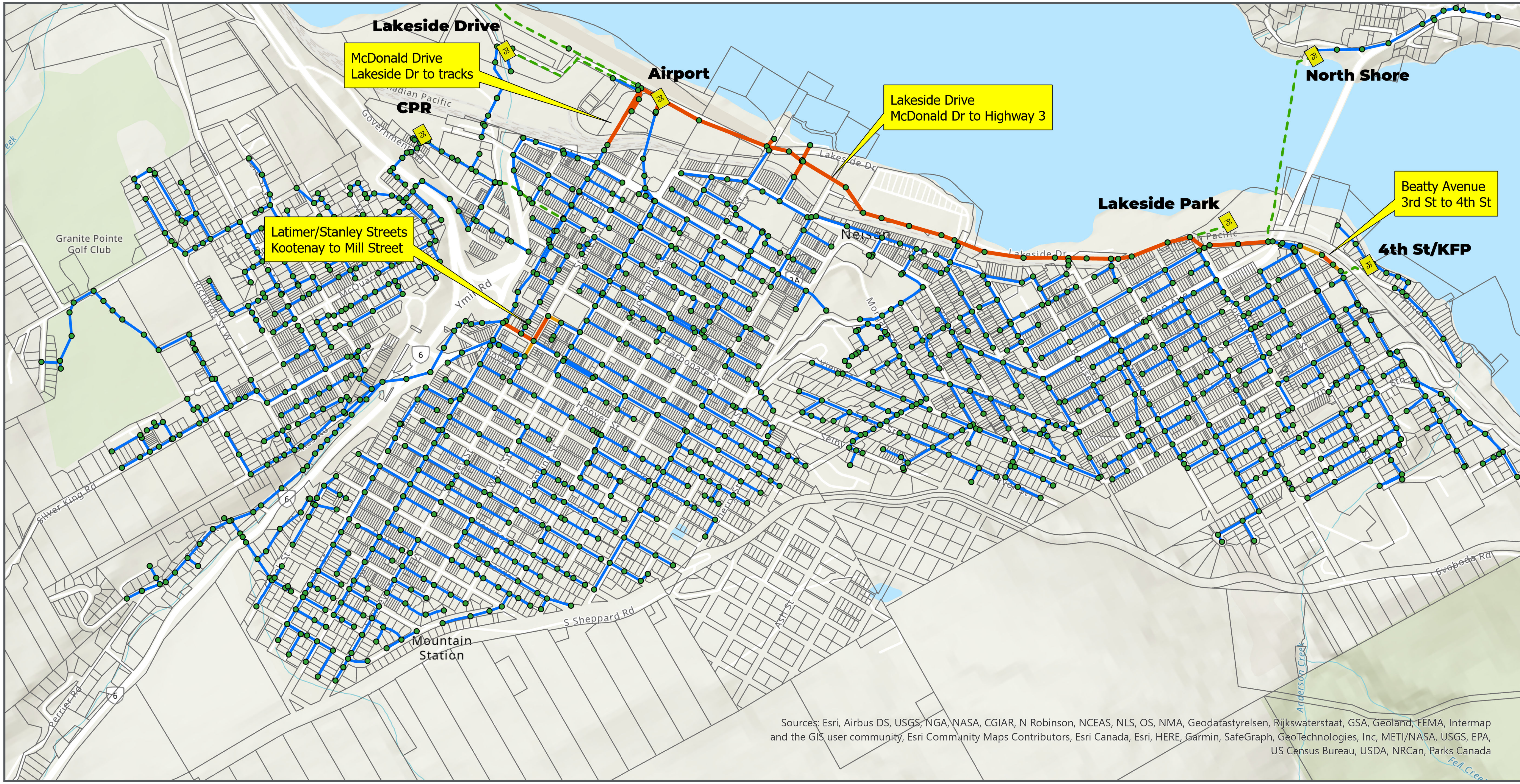
Notes

Refer to Figure 7 for estimated remaining service life of the sanitary mains

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 05
Sanitary Sewer Materials and Estimated Service Life

U:\Projects_KEL\0795_ClientInfo\Models\Sanitary\GIS\GIS\Projects\Pro_Projects\Pro_Projects.aprx Last updated by jbarta on January 13, 2022 at 8:35 AM



URBAN systems

200 100 0 200 400 600
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

Project #: 0795.Clientinfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 6 / 10

PS Lift Stations

Manholes

Pipe Capacity Results

Gravity, flowing 75% full or less

Gravity, flowing 75% to 99% full

Gravity, flowing 100% full

Forcemain

City of NELSON

City of Nelson

Sanitary Sewer System

FIGURE 06

Sewer Main Capacity Issues

Existing Conditions

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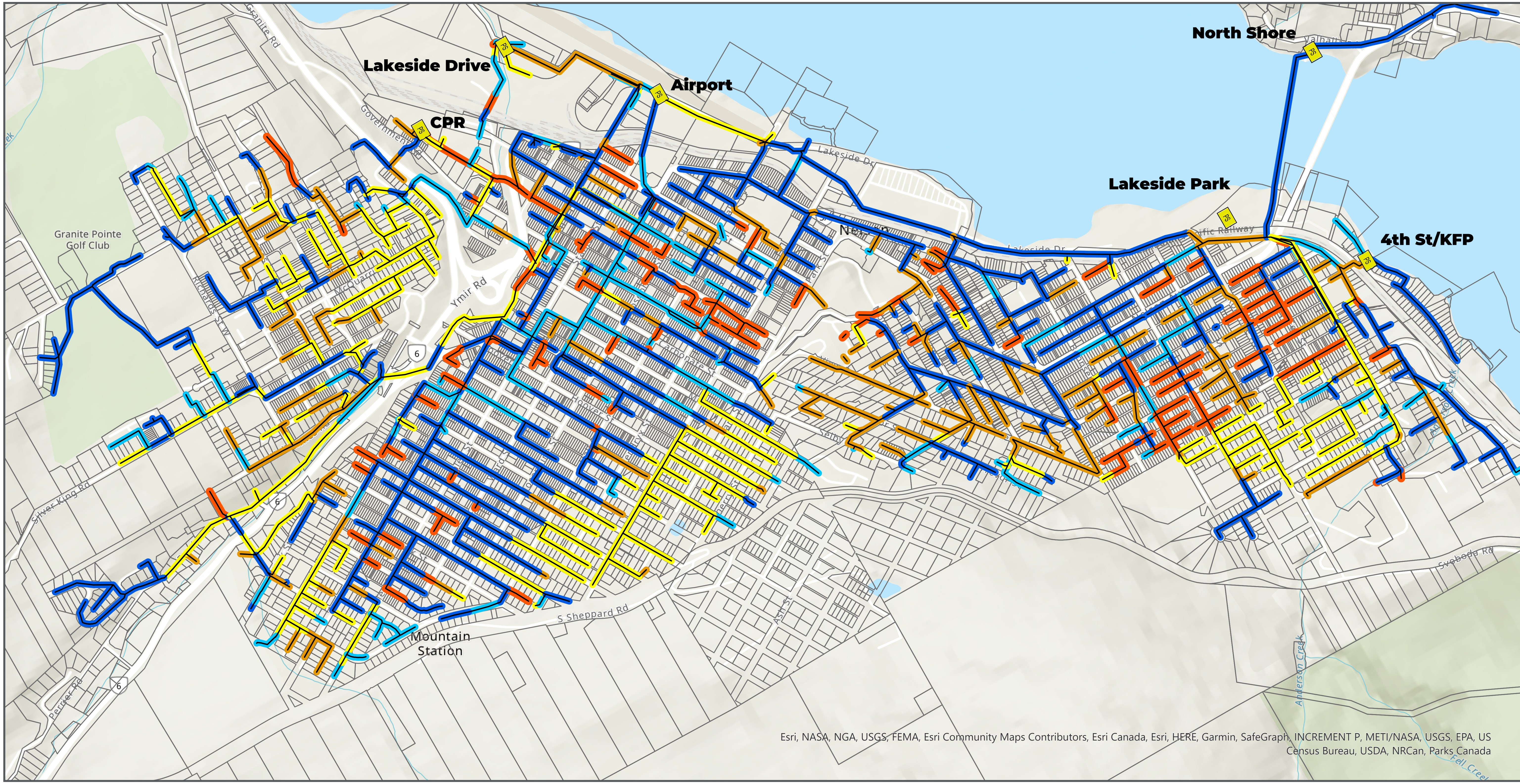


URBAN systems

Project #: 0795.Clientinfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 1 / 13

200 100 0 200 400 600
Metres

Coordinate System:
NAD 1983 UTM Zone 11N

Scale: 1:12,000
(When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

PS Lift Stations
Sewer Mains

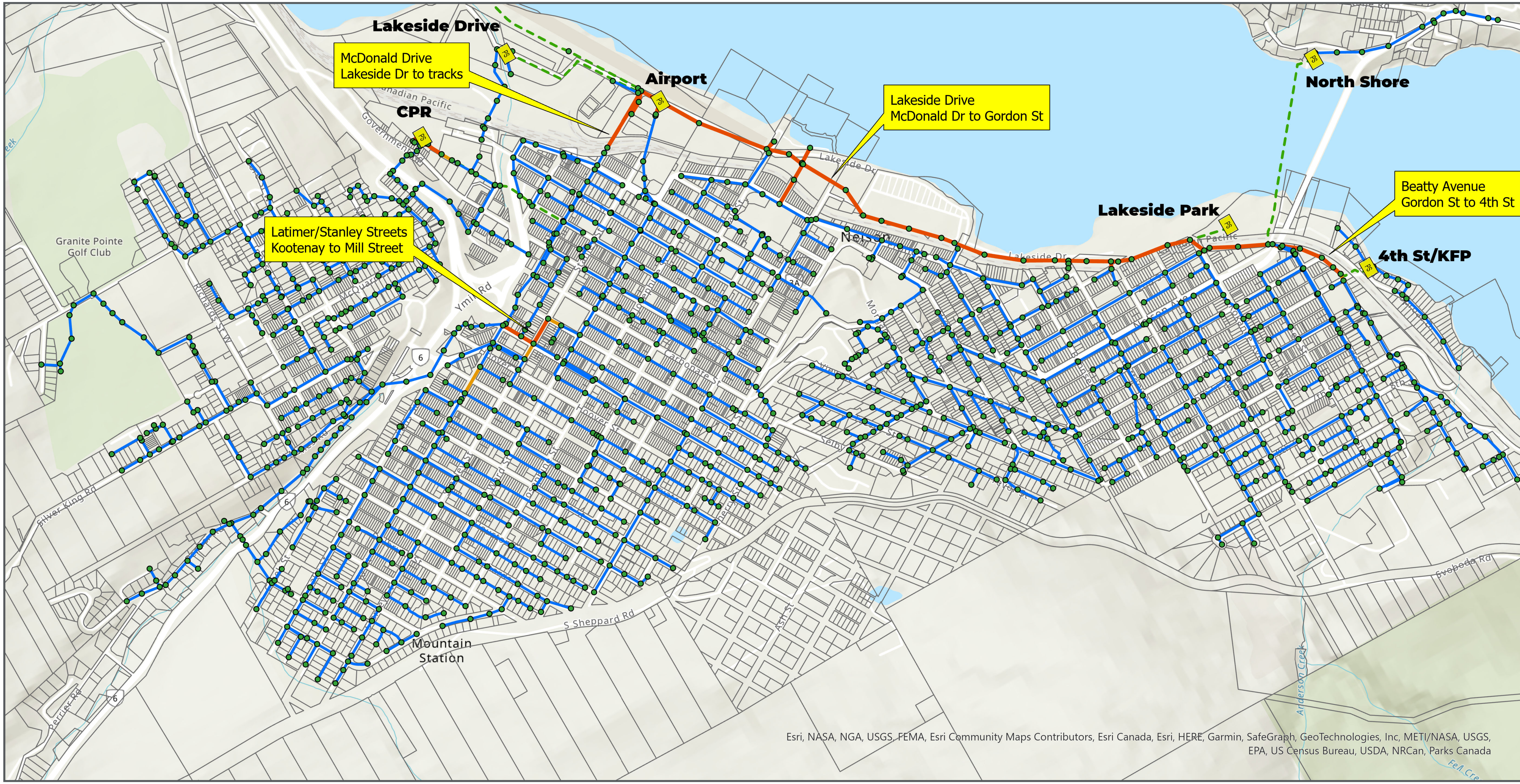
Remaining Service Life
- At end of life
- 0 to 10 years
- 11 to 20 years
- 21 to 50 years
- 50 or more years

Notes

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 07
Sanitary Sewer Estimated Remaining Service Life

U:\Projects_KEL\0795_ClientInfo\Models\Sanitary\GIS\GIS\Projects\Pro_Projects\Pro_Projects.aprx Last updated by jbarta on January 13, 2022 at 8:36 AM



URBAN systems

200 100 0 200 400 600
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

Project #: 0795.Clientinfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 6 / 10

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

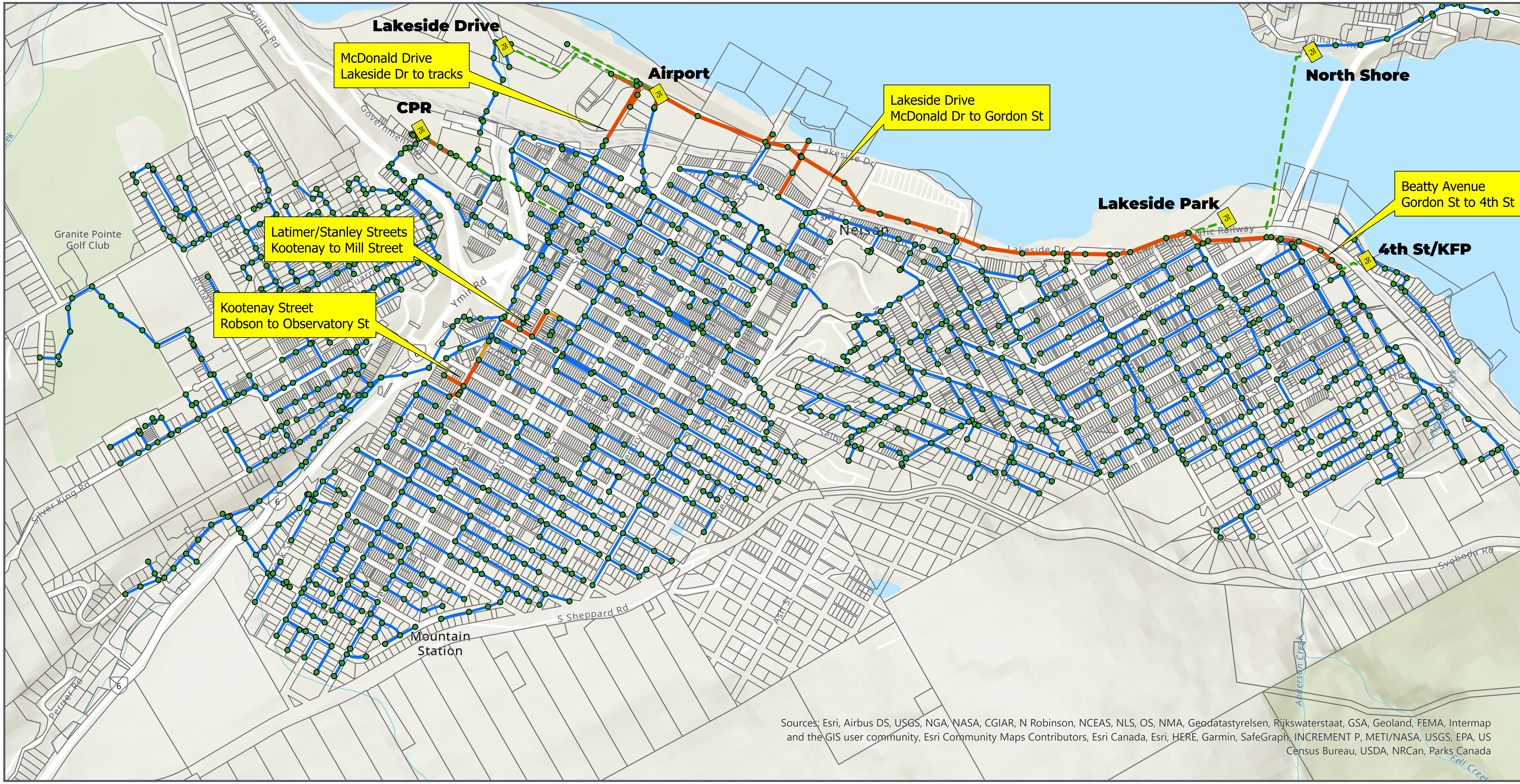
Lift Stations
Manholes

Pipe Capacity Results
- Gravity, flowing 75% full or less
- Gravity, flowing 75% to 99% full
- Gravity, flowing 100% full
- Forcemain

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 08
Sewer Main Capacity Issues
2041 Growth Scenario

U:\Projects_KEL\0795_ClientInfo\Models\Sanitary\GIS\GIS\Projects\Pro_Projects\Pro_Projects.aprx Last updated by barnold on June 10, 2022 at 12:12 PM



URBAN systems

Project #: 0795.ClientInfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 1 / 13

50 0 50 100 150
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

PS Lift Stations

Manholes

Pipe Capacity Results
- Gravity, flowing 75% full or less
- Gravity, flowing 75% to 99% full
- Gravity, flowing 100% full
- Forcemains

Notes

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 09
Sewer Main Capacity Issues
Buildout Growth Scenario

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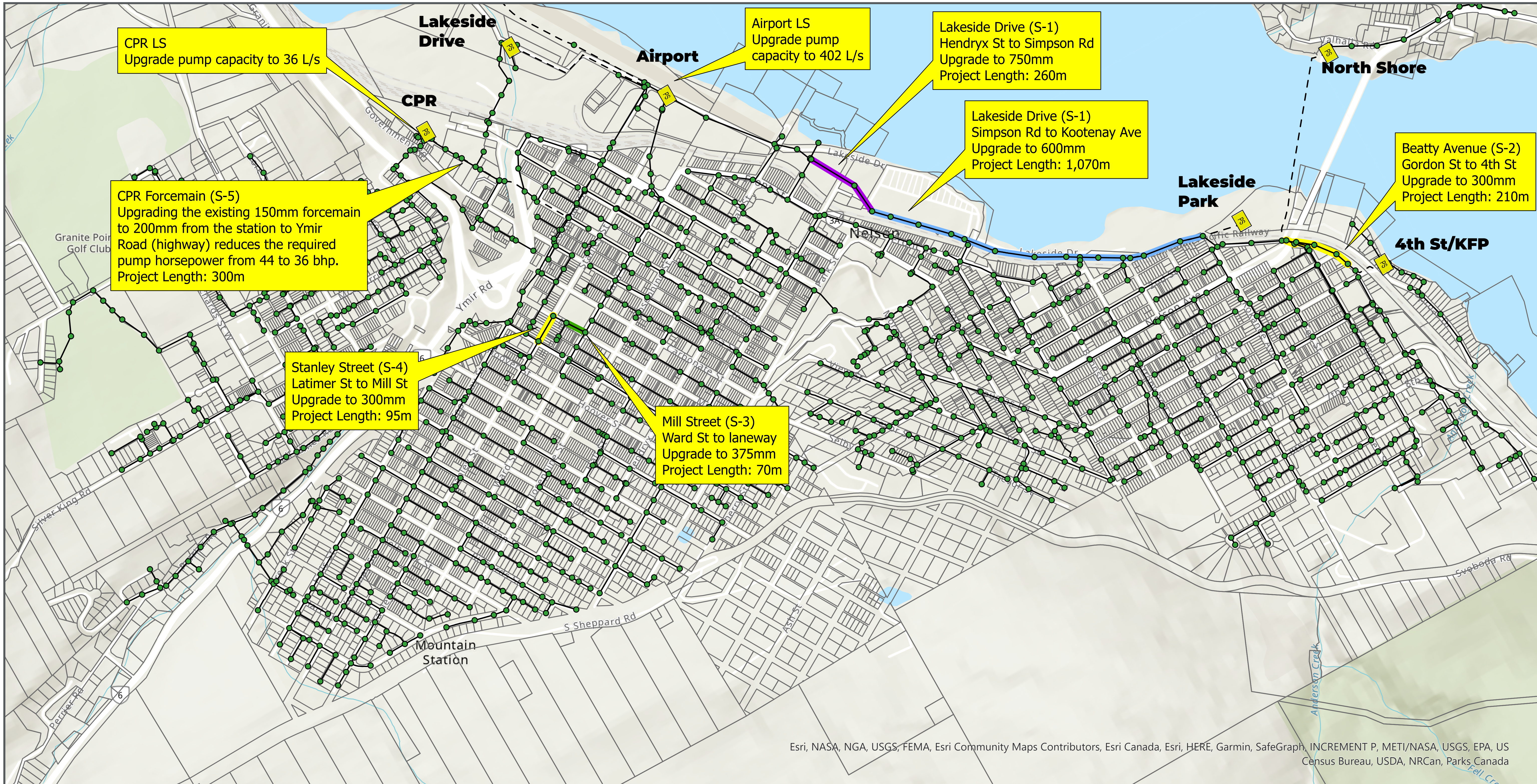

URBAN systems

50 25 0 50 100 150
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

Project #:	0795.Clientinfo
Author:	JB
Checked:	-
Status:	Draft
Revision:	A
Date:	2022 / 1 / 19

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PS Lift Stations
Manholes

Existing Sewer Mains
- Gravity
- Forcemain

Upgrades for 2041 Growth
- Upgrade to 300mm
- Upgrade to 375mm
- Upgrade to 600mm
- Upgrade to 750mm

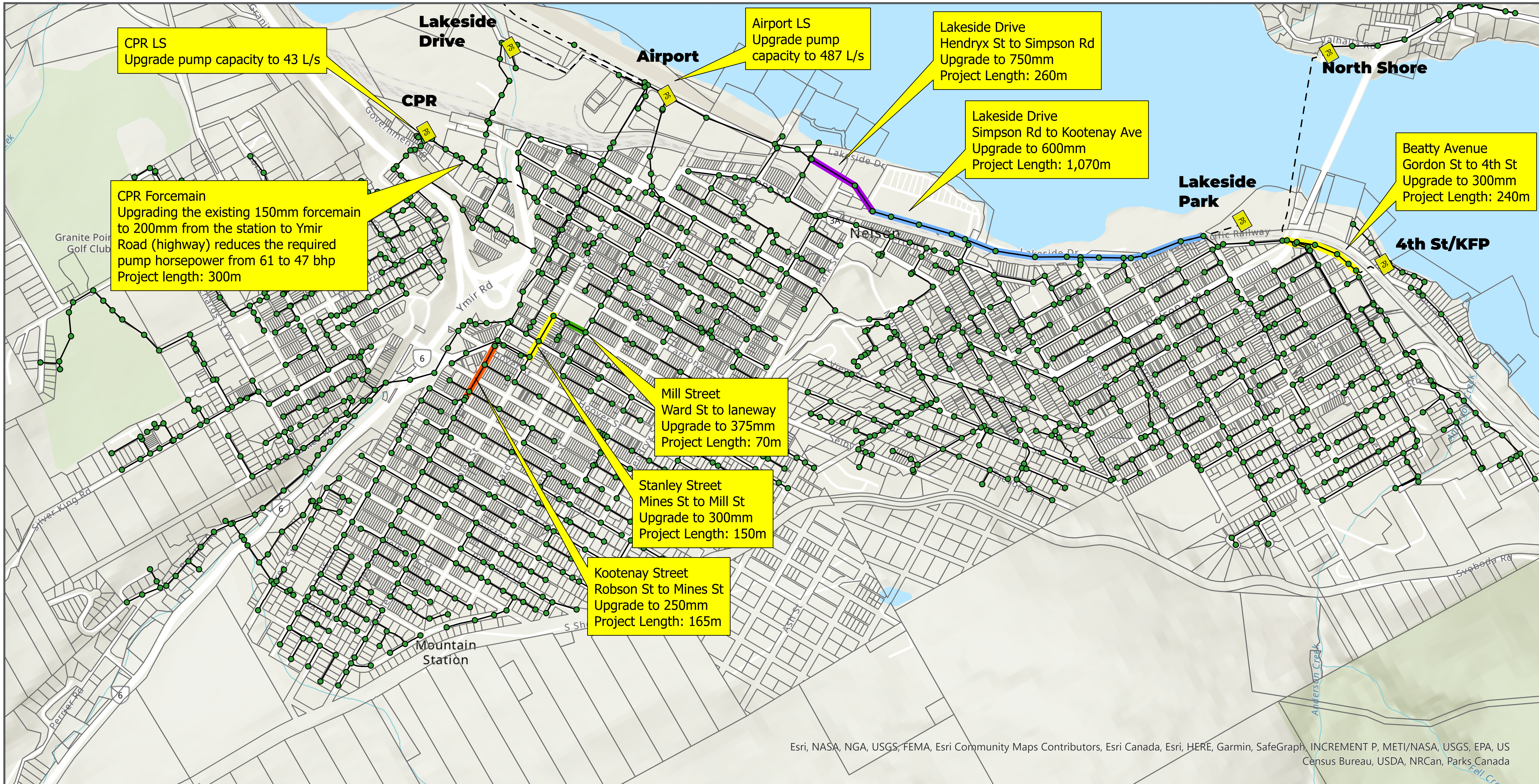
Notes

Sewer main upgrades sized to accommodate peak flows under buildout scenario.

Lift Station upgrades sized to accommodate 2041 peak flows.

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 10
Sewer Main Upgrades
2041 Growth Scenario



URBAN systems

Project #: 0795.ClientInfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 1 / 19

50 25 0 50 100 150 Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

PS Lift Stations
● Manholes

Existing Sewer Mains
Gravity
Forcemain

Upgrades for Buildout Growth
Upgrade to 250mm
Upgrade to 300mm
Upgrade to 375mm
Upgrade to 600mm
Upgrade to 750mm

Notes

Sewer main upgrades sized to accommodate peak flows under buildout scenario.

Lift Station upgrades sized to accommodate 2041 peak flows.

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 11
Sewer Main Upgrades
Buildout Growth Scenario

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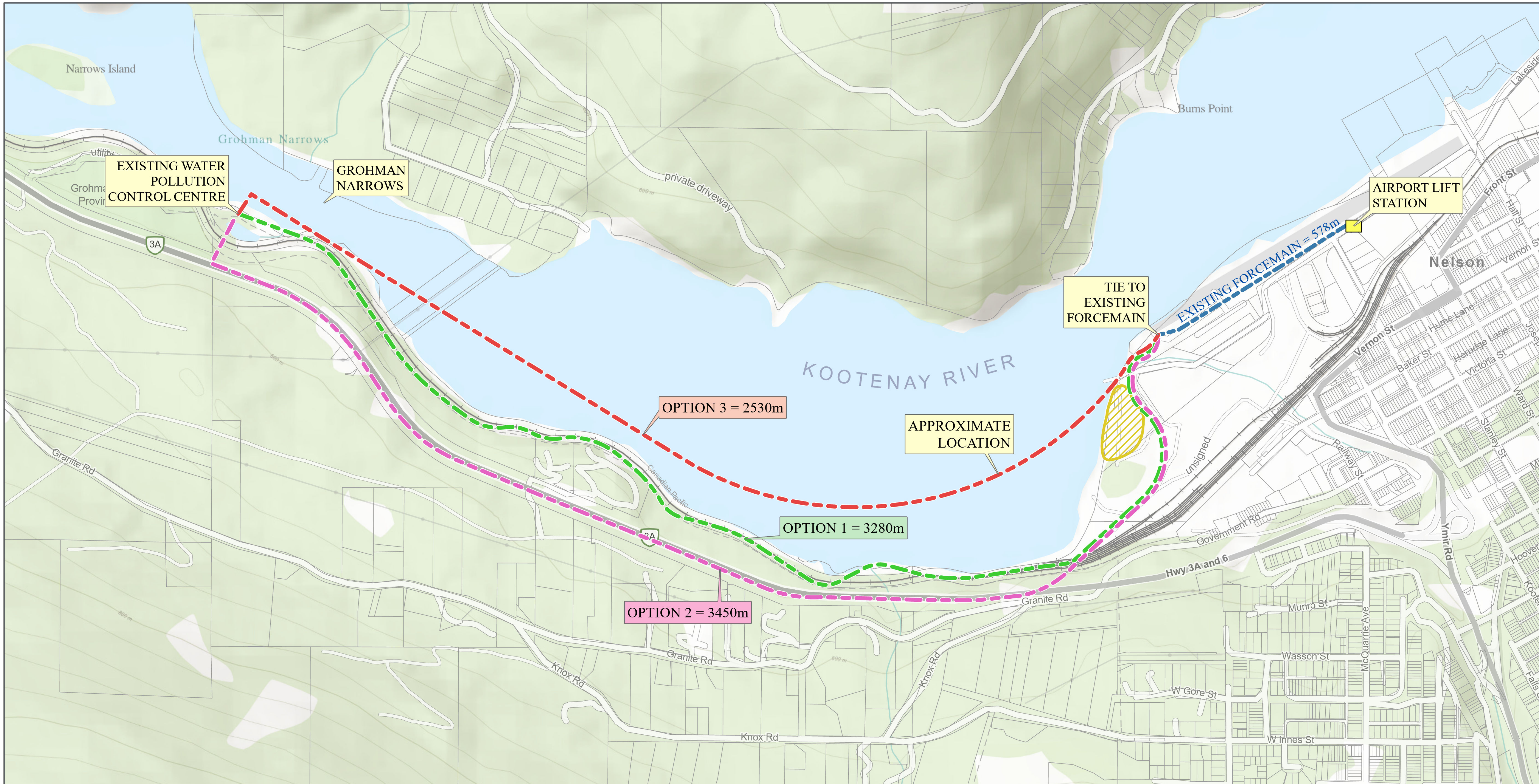

URBAN systems

Project #: 0795.0119.01
Author: BA
Checked: -
Status: Draft
Revision: A
Date: 2022 / 6 / 2

0 250 500
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:10,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model
- NRCan, Esri Canada, Canadian Community Maps Contributors

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

- Lift Station
- CPR Route (Option #1)
- Hwy. No. 3A (Route Option #2)
- HDPE Forcemain (Option #3)
- Existing Forcemain
- WPCC (Potential Site)

Notes

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 12
Airport Lift Station Sanitary Forcemain Replacement Options

\\usl.urban-systems.com\projects\Projects_NEL\0795\0119\01\D-Design\GIS\Projects\Pro_Projects\0795.0119.01.aprx Last updated by barnold on June 2, 2022 at 11:23 AM

URBAN systems

Project #: 0795.0119.01
Author: MS
Checked: -
Status: Draft
Revision: A
Date: 2022 / 6 / 16

Coordinate System:
NAD 1983 UTM Zone 11N

Scale: 1:500 (0, 10, 20m)

Data Sources:
- City's PCSWMM Model

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Notes

City of NELSON

City of Nelson
Treatment & Disposal System
FIGURE 14
Alternate Sewage Treatment Plant
Site Plan

APPENDIX A:

TECHNICAL MEMORANDUM NO. 1

DESIGN CRITERIA – COLLECTION SYSTEM

MEMORANDUM

DATE: January 13, 2022
TO: Colin Innes
CC: Rob Nystrom, Scott Eagleson
FROM: Jason Barta, Jeremy Clowes
FILE: 0795.0119.01
SUBJECT: Technical Memo No.1 – Design Criteria – Collection System, Rev. 2

1.0 INTRODUCTION

The City of Nelson (City) has requested that Urban System (Urban) review the capacity of the existing collection system as component of the Sanitary Master Plan project. Hydraulic models for existing and future conditions will be updated/created in order to estimate the peak flows throughout the system, which ultimately converge at the Airport Lift Station.

The principles set out in this memorandum can also be applied to the transmission system analysis – conveyance of sewage from the Airport Lift Station to the Pollution Control Centre (PCC).

2.0 METHODOLOGY

A hydraulic model will serve as the foundation for the capacity review. The existing conditions model should represent the existing system and flows as best as possible. This includes ensuring all manholes, gravity sewers, lift stations and forcemains are properly described in the model as that all sewer connections are also modeled. This part of the process is called validation.

We will then use flow data supplied by the City to attempt to reasonably calibrate the model. This process includes ensuring overall flow volumes and peak flows match recorded data. Some generalizations will be required to stay within project budget, while ensuring accuracy within 10% or better of recorded data.

Once a calibrated existing conditions model is created, it can be used to build the 20 year and full buildout future conditions models. Whereas the loading in the existing conditions model is based on recorded data, the growth projections used in the future models will utilize design loading from the SDS bylaw and/or existing flow data.

All three models (existing, 20 year, buildout) will then be analyzed for capacity issues in the gravity mains, forcemains and lift stations. The guiding criteria governing what constitutes capacity exceedance is described later in this memorandum.

For each model, the pipe and pump upgrades necessary to meet SDS bylaw criteria will be determined.

Lastly, a class D cost estimate for each of the three design solutions will be prepared.

3.0 EXISTING CONDITIONS MODEL

The model should be a reasonable reflection of the existing system. Validation of the model will include ensuring all infrastructure is accurately presented in the model, including:

- Sewer main and forcemain sizes, materials, and inverts
- Manhole sizes and rim elevations
- Pump station geometry (wetwell), pump curves and operational settings

The validation process will also examine sewer loading to ensure the correct volume of sewage is replicated in the model. This part of the procedure will involve reviewing available SCADA data at various lift stations, so that generalization can be made about each subcatchment (each catchment may be influenced by groundwater or storm sewer connections differently).

The calibration process will review the SCADA data and lift station operations in more detail. This part of the process focuses on pump operation, pipe roughness coefficients and establishing best-fit diurnal curves to estimate peak flows more accurately in the system.

Note that validation and calibration is an ever-ongoing process. As new infrastructure is constructed, it should be added to the model. Likewise, if water conservation efforts show a reduction in consumption, that decrease should be reflected in the model's sewer loading.

A well-calibrated/validated model also benefits the City when assessing new development on a case-by-case basis. Decisions with respect to size and timing of upgrades can be made with confidence when using a well calibrated/validated model.

3.1 EXISTING POPULATION AND FLOWS

The City has experienced a near-constant growth rate of approximately 1% between 1991 and 2019. The estimated 2021 population of the City was extrapolated from the 2016 census population of 10,572 at 1.2% to estimate 11,222 persons. Table 3.1 below presents the flow data for the Airport Lift Station, which is the sole connection to the PCC.

Table 3.1 Airport Lift Station Flows

Parameter	Unit	2014	2015	2016	2017	2018	2019	2020
Population	People	10,364	10,467	10,572	10,699	10,827	10,957	11,089
Per capita flow	L/cap/d	531	516	530	505	462	411	424
ADWF	m^3/day	5,500	5,400	5,600	5,400	5,000	4,500	4,700
AADF	m^3/day	5,400	5,300	5,500	9,200	9,000	6,400	8,300
AWWF	m^3/day	5,900	6,000	5,900	13,900	7,200	8,000	7,400
MMF	m^3/day	6,500	6,600	6,800	9,700	6,800	5,100	5,400
MDF	m^3/day	9,200	12,200	9,400	13,900	9,600	8,900	8,700
PHF	m^3/hr	688	838	767	713	700	779	700

ADWF – Average dry weather flow. Daily volume average for days with less than 1 mm precipitation

AADF – Average annual daily flow. Average of all recorded days

AWWF – Average wet weather flow – Daily volume average for days with more than 1mm precipitation

MMF – Maximum monthly flow. Rolling 30 day average

MDF – Maximum daily flow. Highest daily total from station to PCC (24 hour average)

PHF – Maximum volume in 1 hour period

As the table shows, the effects of water meters and conservation efforts are positively affecting per capita flow rates. Average flows are variable and appear to be highly influenced by groundwater and/or rainfall. The model will be calibrated to match the 2020 values.

3.2 MODEL UPDATE

New capital projects completed in 2020 will be added to the hydraulic model.

Sewer loadings and infiltration and inflow rates will be adjusted to replicate the 2020 recorded data at the Airport Lift Station. The CPR Lift Station SCADA data will also be analyzed to confirm per capita rates and for comparison against the per capita rates found at the Airport LS.

Sewer loads, infiltration estimates and diurnal curves in the model will be adjusted to align with the 2021 flow monitoring data collected by the City.

Separate diurnal curves will be created for the future conditions model that have similar shape to the existing patterns, but reflect the peaking factor set out in the SDS bylaw. Adjustments to baseline (I&I) flows in the model will be made to reflect the removal of catch basin cross connections and newly lined (in situ) sewer mains.

4.0 FUTURE CONDITIONS MODEL

The future conditions models will be built upon the existing conditions models. A separate model will be created for the 20 year and ultimate build-out design horizons. In each model, the loading for existing customers will remain unchanged. New development will utilize SDS bylaw criteria for Average Dry Weather Flow (ADWF), peaking factors and Infiltration and Inflow (I&I).

4.1 AVERAGE DRY WEATHER FLOW (ADWF)

The SDS bylaw provides an average per capita flow of 360 Litres per day per capita. The average per capita flow rate will be multiplied by the equivalent population to yield the Average Dry Weather Flow.

4.1.1 Residential Equivalent Population

Future growth shall be expressed in terms of equivalent population. Residential equivalent population will be calculated using projected unit counts and densities. Where unit counts are not available, units will be estimated using development type and land area. Equivalent population will be calculated using the criteria in Table 4.1 below.

Table 4.1 Residential Equivalent Population Criteria

Land Use	People/Gross Hectare	People/Unit
Single Family	24 to 30	3
Multi-Family: Low Density	65	2
Multi-Family: Medium Density	120 (up to 3 storey)	2
Multi-family: High Density	320-960 (4-12 storey)	2
Mobile Home	40	2

4.1.2 Non-Residential Equivalent Population

Non-residential flows shall be specific to the actual use prescribed for the development area. In many cases, the actual design flow is not known, and an estimate of non-residential equivalent population must be made using approximate land densities for commercial, institutional, and industrial (ICI) lands types. Table 4.2 below lists the equivalent population factors based on land use.

Table 4.2 Non-Residential Equivalent Population Factors

Land Use	Equivalent Population/Gross Hectare
Commercial	120
Institutional	200
Industrial	200

Table 3.3 of the SDS bylaw provides type ADWF rates for specific ICI land uses (such as restaurants and care facilities) that were applied where applicable.

Non-residential loadings utilizing MMCD design guidelines (25,000 L/Ha/day as compared to 43,000 to 72,000 L/Ha/day using the SDS Bylaw) were also reviewed during the preliminary stage of the work. The City directed Urban to use the SDS Bylaw values for non-residential development.

4.2 PEAKING FACTOR (PF)

The Peak Dry Weather Flow (PDWF) is calculated by summing the ADWF for all parcels within a sewer subcatchment and multiplying by a Peaking Factor. The Peaking Factor is directly related to the total equivalent population of the subcatchment and expressed by the following formula:

Pf = 3.2 x (Equivalent Population in thousands) ^ -0.105

4.3 INFILTRATION AND INFLOW (I&I)

The SDS bylaw provides infiltration (groundwater) and inflow (catch basins, storm services, etc.) rates of 5000 L/ha/day and 8,000 L/ha/day for new mains not in the water table and in the water table, respectively.

Where infill development occurs, it is assumed that no additional sanitary sewer will be constructed and as such, there will be no infiltration & inflow in these circumstances.

4.4 PEAK WET WEATHER FLOW (PWWF)

The Peak Wet Weather Flow is the summation of the Peak Dry Weather Flow and Infiltration & Inflow. The PWWF will influence the system at least once during the calendar year and will be the highest stressor of the collection system. The PWWF will help determine residual capacity of the system and where upgrades are required.

4.5 GROWTH PROJECTIONS

The 2017 Water Master Plan update cited a buildout population of approximately 15,000 people. The WMP update estimated the growth rate at just over 0.6% per annum. A recently supplied population memo from the City's Development Services team provided updated growth rates ranging from 1.4% to 1.7% and a revised buildout population of 24,476 based on newer strategies for infill development.

In a December 9th, 2021, email to Urban Systems, the City, in consultation with its Planning Department, confirmed that the growth rate shall be 1.2% for this work. The growth rate will govern trigger points for infrastructure upgrades.

Table 4.3 provides details of the design horizons with the previous and confirmed growth rates.

Table 4.3 Population Projections

Parameter	1.2% Growth Rate
2016 Census Population	10,572
Estimated 2021 Population	11,222
Projected 2041 Population	14,245
20 year growth in equivalent pop.	3,024
Buildout Population (new 2021)	24,476
Years to reach buildout	65
Buildout Year	2086

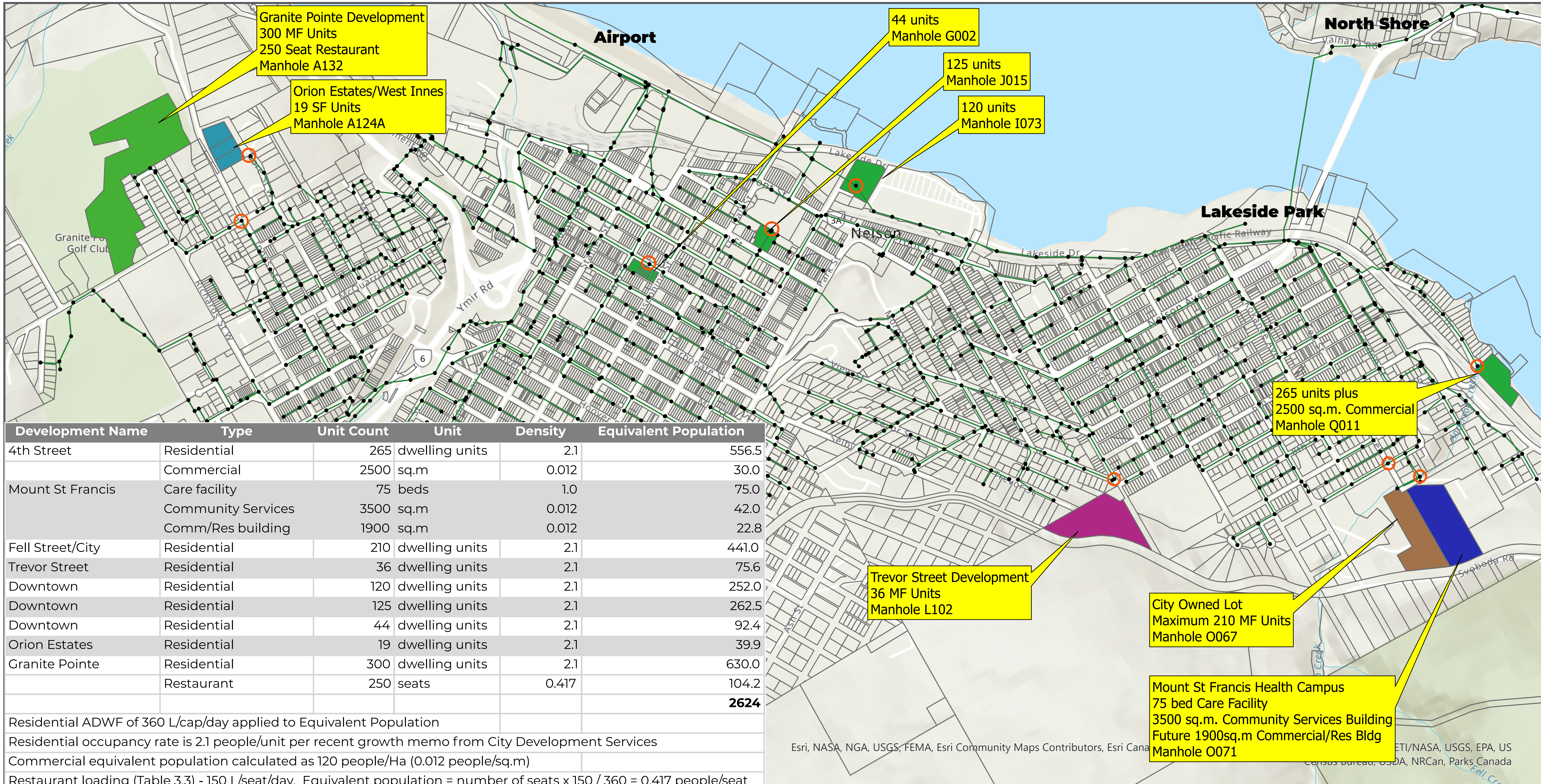
Figure 4, attached, provides details of the known development locations within the City. It is assumed that these developments will reach full build-out within the next 20 years. The total equivalent population of the developments in the figure is 2,624 persons.

The 20 year future conditions model will be populated as follows:

- The developments (2,624) from Figure 4 will be added to specific nodes in the model.
- The remaining growth (400 persons) will be allocated evenly across the remaining nodes.

The buildout future conditions model will be populated similarly:

- The developments (2,624) from Figure 4 will be added to specific nodes in the model.
- The remaining growth (10,630 persons) will be allocated evenly across the remaining nodes.



Development Name	Type	Unit Count	Unit	Density	Equivalent Population
4th Street	Residential	265	dwelling units	2.1	556.5
	Commercial	2500	sq.m	0.012	30.0
Mount St Francis	Care facility	75	beds	1.0	75.0
	Community Services	3500	sq.m	0.012	42.0
	Comm/Res building	1900	sq.m	0.012	22.8
Fell Street/City	Residential	210	dwelling units	2.1	441.0
Trevor Street	Residential	36	dwelling units	2.1	75.6
Downtown	Residential	120	dwelling units	2.1	252.0
Downtown	Residential	125	dwelling units	2.1	262.5
Downtown	Residential	44	dwelling units	2.1	92.4
Orion Estates	Residential	19	dwelling units	2.1	39.9
Granite Pointe	Residential	300	dwelling units	2.1	630.0
	Restaurant	250	seats	0.417	104.2
					2624

Residential ADWF of 360 L/cap/day applied to Equivalent Population

Residential occupancy rate is 2.1 people/unit per recent growth memo from City Development Services

Commercial equivalent population calculated as 120 people/Ha (0.012 people/sq.m)

Restaurant loading (Table 3.3) - 150 L/seat/day. Equivalent population = number of seats x 150 / 360 = 0.417 people/seat

URBAN systems

Project #: 0795.Clientinfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 1 / 13

200 100 0 200 400 600
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

Development loads assigned to model node as shown with red circle. Remainder of growth in each scenario (2041/Buildout) achieved by uniformly scaling existing loads of all other nodes in the model.

Population Allocation	Existing	2041	Buildout
Growth areas on Figure 4	n/a	2,624	2,624
Distributed across remainder of system	11,222	400	10,630
Total growth	n/a	3,024	13,254
Total population	11,222	14,246	24,476

Notes

Development Areas
- City Lot near Health Campus
- Fell Street Development
- Granite Pointe
- Orion Estates West Innes
- Trevor Street
- Unnamed

City of NELSON

City of Nelson
Wastewater Management Plan
FIGURE 04
Sanitary Sewer Loading Allocation

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5.0 SYSTEM EVALUATION

This section describes the criteria used to evaluate deficient infrastructure and the guidelines for sizing the replacement infrastructure.

5.1 REPLACEMENT TRIGGERS

5.1.1 Gravity Sewers

Any main where flow depth matches or exceeds pipe diameter (d/D >= 1) shall be deemed capacity limiting and subject to upsizing per the criteria in Section 5.2.1. Note that there may be instances where a flagged pipe is influenced by a downstream bottleneck, and thus, no upsizing will be required.

Conversely, there may be instances where peak flows cause flooding in the model. Under these circumstances, some of the design flow is "lost" from the system. Once local upgrades are identified to avoid the flooding condition, the "lost" flow can then be conveyed further downstream and impact other pipes not previously identified.

Remaining asset life – based on pipe age and material – will be reviewed to determine whether replacement is warranted. There are approximately 55 km of main installed between 1912 and 2004, 30 km between 2005 and 2019 and a final 580 m installed in 2020. Table 5.1 lists the expected asset life for various pipe materials.

Table 5.1 Expected Life of Various Sewer Main Materials

Material	Expected Asset Life (Years)	Values used in Analysis
Asbestos Cement (AC)	40-70	70
Concrete (Conc)	50-75	70
Corrugated Metal Pipe (CMP)	25-50	50
Vitreous Clay (VCT)	50-60	60
PVC	60-100	80
In-situ Lined Sewer	50-60	60

5.1.2 Lift Stations and Forcemains

Pump replacement will be required where peak design flow into the station exceeds the capacity of the pump(s) with the largest flow pump being out of service.

Wetwell size shall be reviewed to ensure no more than six starts per pump per hour.

5.1.3 Forcemains

Where forcemain velocity exceed 3.5 m/sec under design peak flow conditions, the main shall be replaced or twinned, depending on condition and remaining asset life.

5.2 REPLACEMENT SIZING

Replacement sizing will be governed by the City's Subdivision and Development Servicing (SDS) Bylaw.

5.2.1 Gravity Sewers

The minimum velocity under peak flow conditions shall be 0.6 m/sec. Consideration shall be given where maximum velocity exceeds 3.0 m/sec.

Replacement mains will be assumed to have the same grade as the existing main. The exception to this assumption is the design of the transmission system between the Airport LS and WWTP.

The roughness coefficient for new mains will be 0.011 for PVC and 0.013 for concrete pipe.

The minimum gravity main size shall be 200 mm and 250 mm for residential and ICI development, respectively. New 200 mm mains shall flow no more than 50% full under peak flow conditions. New 250 mm mains shall flow no more than 60% full under peak flow conditions. New 300 mm or larger mains shall flow no more than 70% full under peak flow conditions.

5.2.2 Lift Station Pumps

Typical station arrangement is for a duplex pump configuration with the station able to meet maximum flow conditions with one pump in failure mode. Triplex stations or greater can be used if the peak flow exceeds capacity of a single, commonly available pump.

Pumps shall be able to operate alternately and independently of each other.

5.2.3 Forcemains

Forcemains must be reviewed in tandem with lift station pumps.

Maximum velocity shall not exceed 3.5 m/sec and must also achieve a minimum cleansing velocity of 1.0 m/sec at least once per day.

5.3 LONEGEVITY OF UPGRADES

5.3.1 Gravity Sewer and Forcemains

The existing, 20 year and ultimate buildout models may experience different peak design flows in each sewer reach and thus may require different upgrade diameter to meet the design criteria in Section 5.2.

Because the design life for new PVC and HDPE mains may well exceed 60 or 70 years, it makes financial sense to incorporate the buildout upgrades in the 20 year program to avoid costly road replacements and the need to replace infrastructure before it reaches the end of its service life.

5.3.2 Lift Stations

The design life for pump station components is typically 20 years. As such, the peak design flows from the 20 year future conditions model will be used to size the pump upgrades necessary for the 20 year capital plan.

A condition assessment of each station's mechanical and electrical components has been completed and will be included in the scope of the report. The review also considered the age of the asset, maintenance records and visual inspections. Wet wells were visually inspected (where feasible) for signs of deterioration as well. Structural reviews were not undertaken.

5.4 UPGRADE TRIGGER POINTS

Growth rates are often tied to market conditions, which can be volatile.

The trigger point for infrastructure upgrades can be expressed in terms of residual capacity - full flow capacity minus design peak flow for that element. This residual capacity can also be expressed as development units.

If a phasing plan is desired, linear growth can be assumed between the existing and 20 year and Build-out horizons. This will allow an estimate of the trigger year for each replacement project identified in the plan.

6.0 COST ESTIMATES

A Class D cost estimate will be prepared for the recommended upgrades which are needed to address deficiencies that are identified in the existing and 20 year growth models. The estimate will carry a 35% contingency and 15% engineering allowance. This estimate classification is typical for high level planning exercises where investment into geotechnical investigations and detailed utility alignments have not been made. Proposed upgrades will be presented in a prioritized list that is based on remaining residual capacity.

Sincerely,

URBAN SYSTEMS LTD.

Jason Barta, B.Sc.
Municipal Infrastructure Analyst

Jeremy Clowes, P.Eng.
Principal, Water & Wastewater Engineer

cc: Anthony Comazzetto, P.Eng.

/jb
Enclosure

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APPENDIX B:

TECHNICAL MEMORANDUM NO. 2

2021 SANITARY SEWER MODEL CALIBRATION

URBAN SYSTEMS MEMORANDUM

DATE: January 13, 2022
TO: Colin Innes
CC: Rob Nystrom, Scott Eagleson
FROM: Jason Barta
FILE: 0795.0119.01
SUBJECT: Technical Memorandum No.2 - 2021 Sanitary Sewer Model Calibration

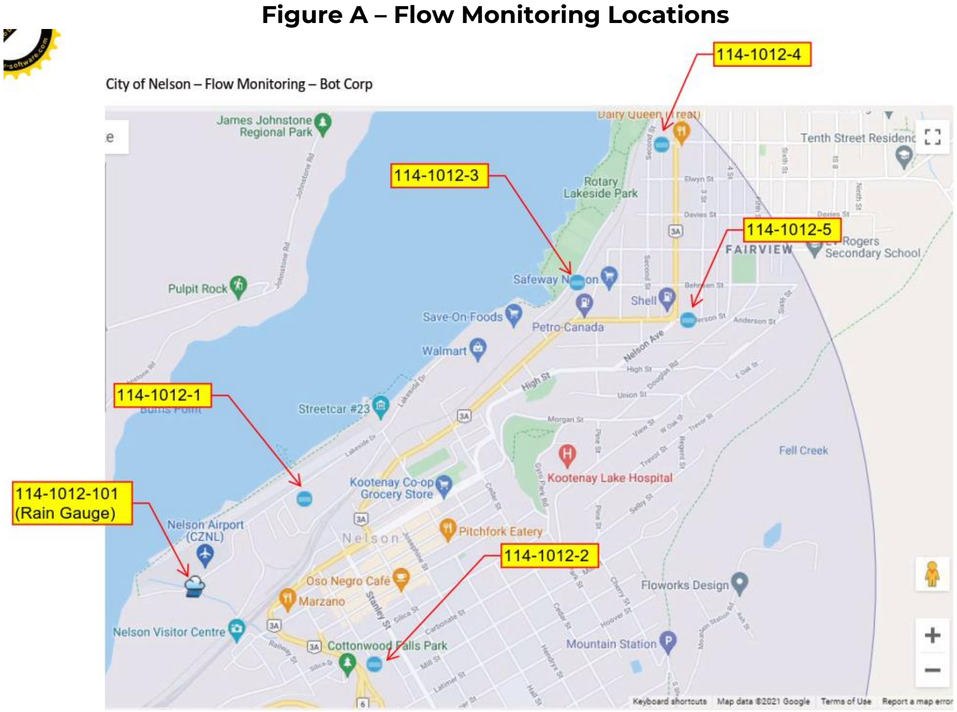
1.0 BACKGROUND

The purpose of calibration is to demonstrate that the sanitary sewer model accurately reflects flow measurements taken throughout the collection system, both in terms of peak flows and volumes. This memorandum summarizes the calibration procedure that was followed in the development of the City's sanitary model. A graphic of the current system is provided in **Appendix 1**. The model has been created and maintained using PCSWMM software by Computational Hydraulics.

2.0 FLOW MONTITORING DATA

Brian Bot of Botcorp provided flow monitoring services in Nelson from June 07 to Sept 09, 2021. Monitoring was undertaken at five manholes in different catchments as shown in **Figure A** below. A rain gauge was also installed for the duration of the monitoring.

Figure A – Flow Monitoring Locations



Each dataset will be imported into the model as "observed data" and used to calibrate the model. **Table 1** lists some of the key data from each monitoring location. Graphs of each monitor are provided in **Appendix 2**.

Table 1 – Flow Monitoring Data

Monitor ID	Manhole ID in model	Downstream Main Size	Peak Dry Weather Flow (PDWF) L/s	Peak Wet Weather Flow (PWWF) L/s
1	I-032	600mm	39	197
2	C-002	375mm	3	13
3	I-079	450mm	17	120
4	N-001	300mm	5	20
5	L-008b	300mm	5	912

2.1 GAUGE #1 (114-1012-1)

The gauge placed along Stanley Street, north of Vernon Street to measure the majority of the downtown catchment. The gauge recordings were hydraulically stable throughout entire monitoring period. The gauge was highly responsive to wet weather conditions, indicating the likelihood of inflow sources such as directly connected catch basins and/or rainwater leaders.

2.2 GAUGE #2 (114-1012-2)

The gauge was located along Kootenay Street and measured trunk sewer flow from the area west of Highway 6 and south of the CPR lift station catchment. The gauge experienced silt accumulation starting on August 15, affecting the calculated volume. There is sufficient good data between June 07 and August 14 to utilize in the calibration process. The gauge did not register significant inflow or infiltration during any of the storm events during the monitoring period with the exception of the high intensity rainfall event on August 22, 2021.

2.3 GAUGE #3 (114-1012-3)

The gauge was placed along Lakeside Drive, roughly 250 meters north of Poplar Street. The main purpose of this gauge was to monitor groundwater and infiltration impacts along the riverfront trunk. The gauge was highly responsive to wet weather conditions, indicating the likelihood of inflow sources such as directly connected catch basins and/or rainwater leaders.

2.4 GAUGE #4 (114-1012-4)

The gauge was situated along Kokanee Avenue, just east of 2nd Street, and provided a measurement of the majority of the northeast catchment of the City. Hydraulics at the site were not optimal. Botcorp installed a weir to improve flow recordings on July 7th. The calibration dataset will be limited to July 07 through September 09. The gauge did not register significant inflow or infiltration during any of the storm events during the monitoring period with the exception of the high intensity rainfall event on August 22, 2021.

2.5 GAUGE #5 (114-1012-5)

The gauge was installed along Anderson Street, just west of 3rd Street and measured a catchment that was primarily residential in nature with the exception of the hospital. This gauge ended up displaying the least consistent pattern (i.e. not suitable for establishing a diurnal curve). The gauge was highly responsive to wet weather conditions, indicating the likelihood of inflow sources such as directly connected catch basins and/or rainwater leaders.

3.0 CALIBRATION PROCESS AND RESULTS

The flow monitoring data will be used to aid in the calibration process. Key variables examined were:

- Average daily volume of sewage
- Flow pattern (diurnal curve)
- Peak flow
- Infiltration and Inflow during dry periods (groundwater)
- Infiltration and Inflow during wet periods (directly connected storm sewers)

In addition, lift station characteristics and SCADA were used to confirm pump rates and wetwell volumes in the model, as well as estimate per capita average dry weather flow. SCADA data was provided by the City for the years 2014 through 2020.

3.1 SCADA DATA

SCADA data for the Airport lift station was provided by the City for 2014 to 2020. Population data was based on the 2016 census population and Urban estimated the 2020 population using a 1.2% assumed growth rate as directed by the City. **Table 2** provides a snapshot of the SCADA data.

Table 2 – SCADA Data at Airport Lift Station

Parameter	Unit	2014	2015	2016	2017	2018	2019	2020
Population	People	10,364	10,467	10,572	10,699	10,827	10,957	11,089
Per capita	L/cap/day	531	516	530	505	462	411	424
ADWF	m3/day	5,500	5,400	5,600	5,400	5,000	4,500	4,700
AADF	m3/day	5,400	5,300	5,500	9,200	9,000	6,400	8,300
AWWF	m3/day	5,900	6,000	5,900	13,900	7,200	8,000	7,400
MMF	m3/day	6,500	6,600	6,800	9,700	6,800	5,100	5,400
MDF	m3/day	9,200	12,200	9,400	13,900	9,600	8,900	8,700
PHF	m3/day	16,500	20,100	18,400	17,100	16,800	18,700	16,800

Notes:

Per capita	Average flow per capita	(calculated as ADWF divided by population)
ADWF	Average dry weather flow	(only days with less than 1mm precipitation included)
AADF	Average annual daily flow	(all days included)
AWWF	Average wet weather flow	(only days with more than 1mm precipitation included)
MMF	Monthly maximum flow	(running 30 day average)
MDF	Monthly daily flow	(largest volume in a single day/24 hour average)
PHF	Peak hour flow	(maximum 1 hour value)

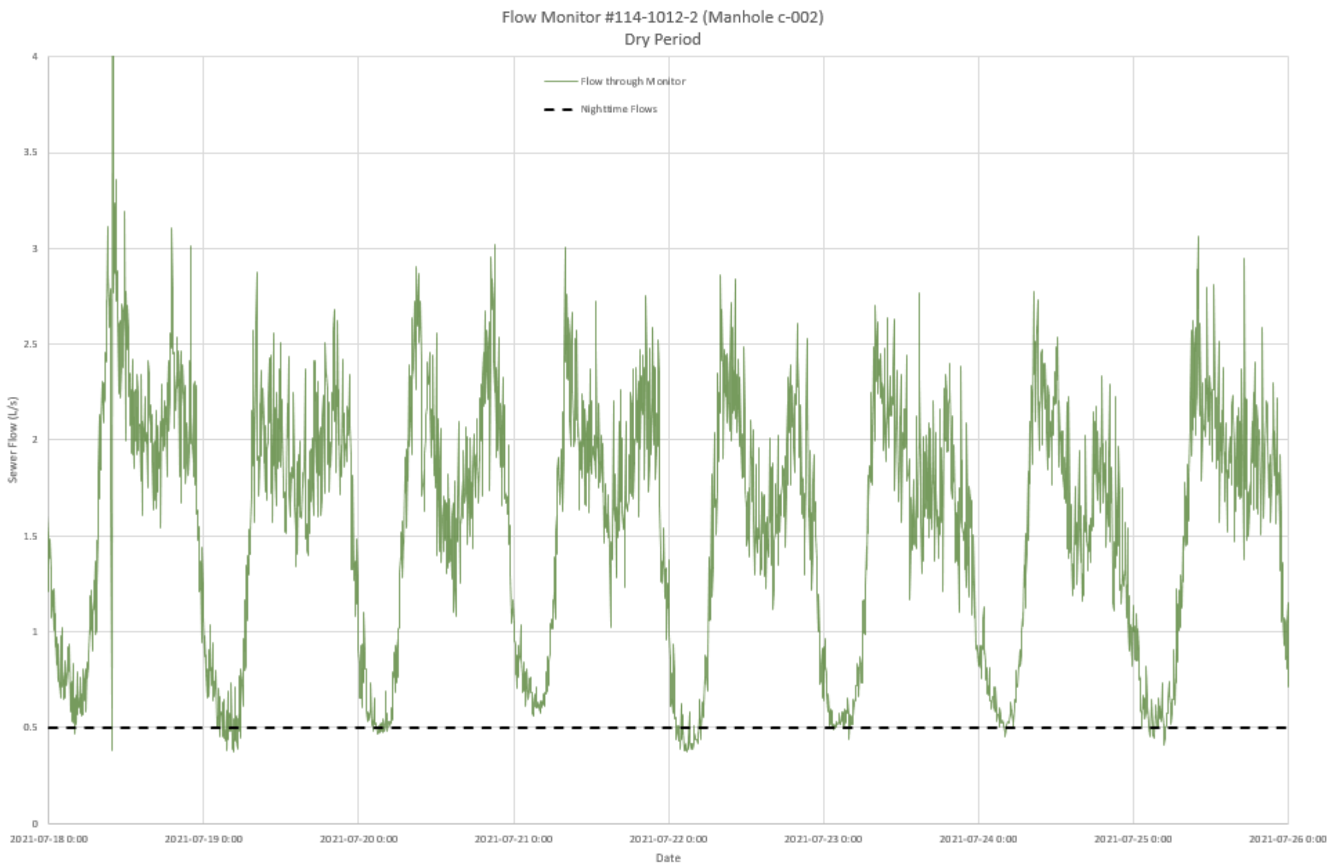
The per-capita ADWF shows a downward trend, likely the result of water conservation efforts, education, and low flow fixtures. The average from 2018 through 2020 is approximately 420 L/capita/day.

Interestingly, the unit ADWF for the City of Nelson is significantly higher than values for the Okanagan Valley (250-300 L/capita/day). It may be that groundwater infiltration has a significant impact on the Nelson system. Nighttime flows within the flow monitoring data will be reviewed to test this supposition.

3.2 PEAK DRY WEATHER FLOW (PDWF) AND DIURNAL PATTERN

The period from July 18, 2021, through July 25, 2021, will be used to extract dry weather flow data from the model as there was no precipitation during this period and all monitors were without issue during this time frame. Upon review of the data, Gauge #2 provided the most consistent, repeatable pattern on a day by day basis. It was assumed that 50% of the nighttime flows were due to infiltration/inflow and the remainder was domestic flow. This assumption was applied to all five monitors. The shape of this pattern was normalized (unit area under graph = 1) to be used as the residential diurnal for the project.

Figure B – Gauge #2 – Dry Period (representative of residential daily pattern)



Tables 3 and 4 summarize the key flow data from each gauge during dry and wet weather conditions, respectively.

Table 3 – Flow Monitoring Data (July 13, 2021: Dry weather period)

Monitor ID	Peak Flow L/s	Average Flow L/s	Peaking Factor	Nighttime Flows (L/s)	Assumed I&I (50% of nighttime)	Daily Volume (cu.m)
114-1012-1	38.5	17.6	2.2	6.5	3.25	1,525
114-1012-2	3.2	1.8	1.8	0.5	0.25	155
114-1012-3	16.6	8.1	2.0	3.0	1.5	700
114-1012-4	5.1	2.4	2.1	0.3	0.15	205
114-1012-5	4.9	2.0	2.5	1.5	0.75	175

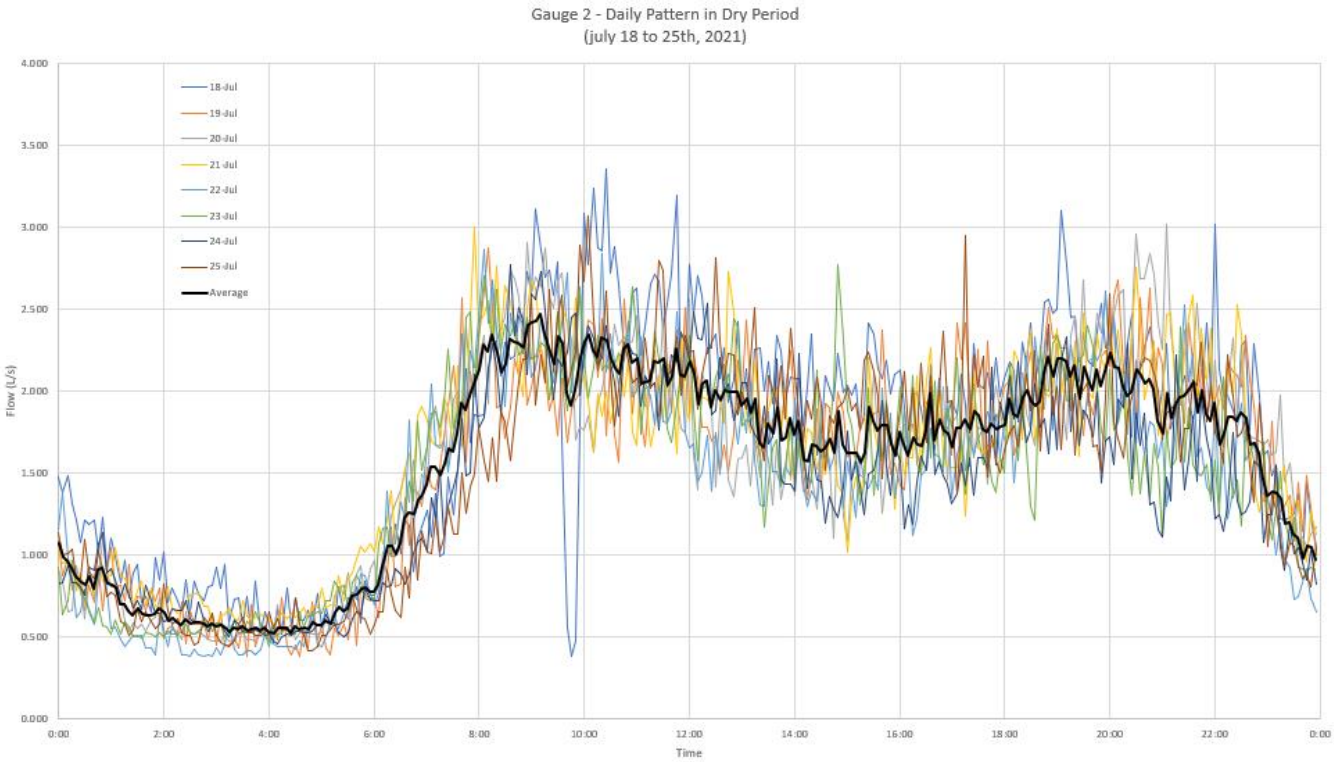
Table 4 – Flow Monitoring Data (Aug 22, 2021: Major Rain Event)

Monitor ID	Peak Flow L/s	Average Flow L/s	Peaking Factor	Nighttime Flows (L/s)	Assumed I&I (50% of nighttime)	Daily Volume (cu.m)
114-1012-1	196.5	32.3	6.1	6.5	3.25	2,790
114-1012-2	12.9	5.0	2.6	0.5	0.25	435
114-1012-3	119.8	12.8	9.4	3.0	1.5	1,100
114-1012-4	19.7	4.7	4.2	0.3	0.15	405
114-1012-5	12.0	5.1	2.3	1.5	0.75	445

The average peaking factor (MDF/ADWF) is 1.9 at the Airport lift station for 2018 through 2020 data in Table 2. As shown in Table 3, we would expect higher peaking factors in the upstream catchments which would be partially attenuated in the long trunks leading to the Airport lift station

Figure C below illustrates the residential diurnal pattern to be applied in the model. Small modifications to the pattern will be made to adjust the peaking factor as required in each catchment, however, the overall shape will remain consistent.

Figure C – Residential Pattern



3.3 MODEL ADJUSTMENTS

The model adjustments were done in two phases. Part one included adjustments under dry weather conditions. Part two examined flows under rainfall events.

3.3.1 Part 1 – Dry Weather Periods

For each catchment upstream of the flow monitors, adjustments were made to the Baseline (Infiltration & Inflow), Average Flow (ADWF), and Time Pattern (Diurnal curve) values at each node.

For each monitor, we have assumed that 50% of the night-time flow rate was attributable to infiltration and inflow – the remainder from domestic usage. That value of flow was allocated evenly across all model nodes in the catchment.

Significant effort was spent during the 2010 Master Plan modeling to allocate sewer loads. Loads were scaled such that the overall dry period volume matched within 5% at each monitor.

Figure D1 through D5 show the alignment between the model and the monitoring gauges.

Figure D1 – Model and Field Data Comparison
Gauge #1 (Dry Weather)

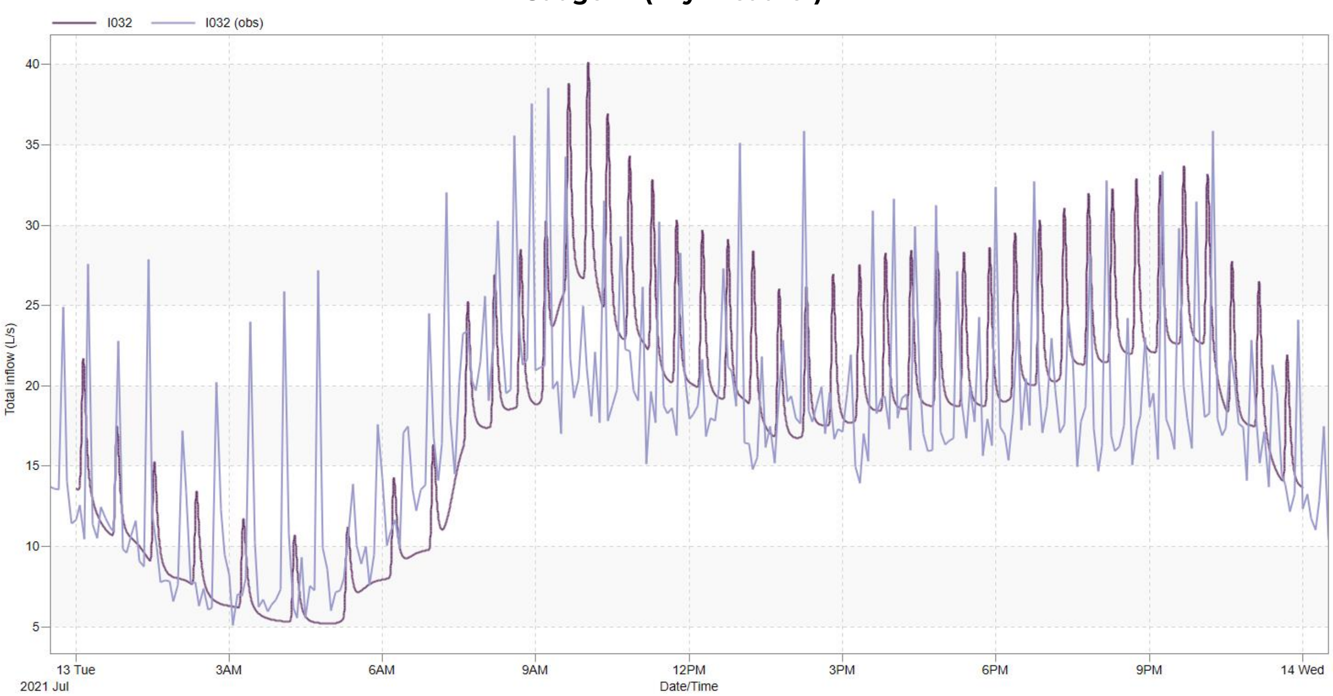


Figure D2 – Model and Field Data Comparison
Gauge #2 (Dry Weather)

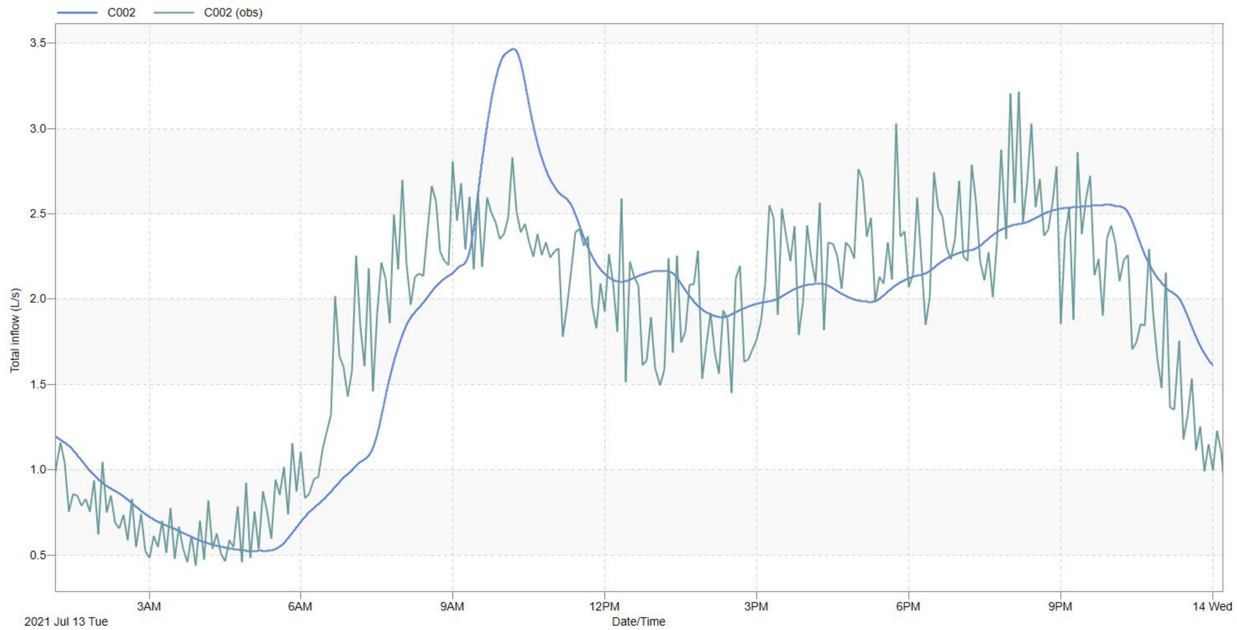


Figure D3 – Model and Field Data Comparison
Gauge #3 (Dry Weather)

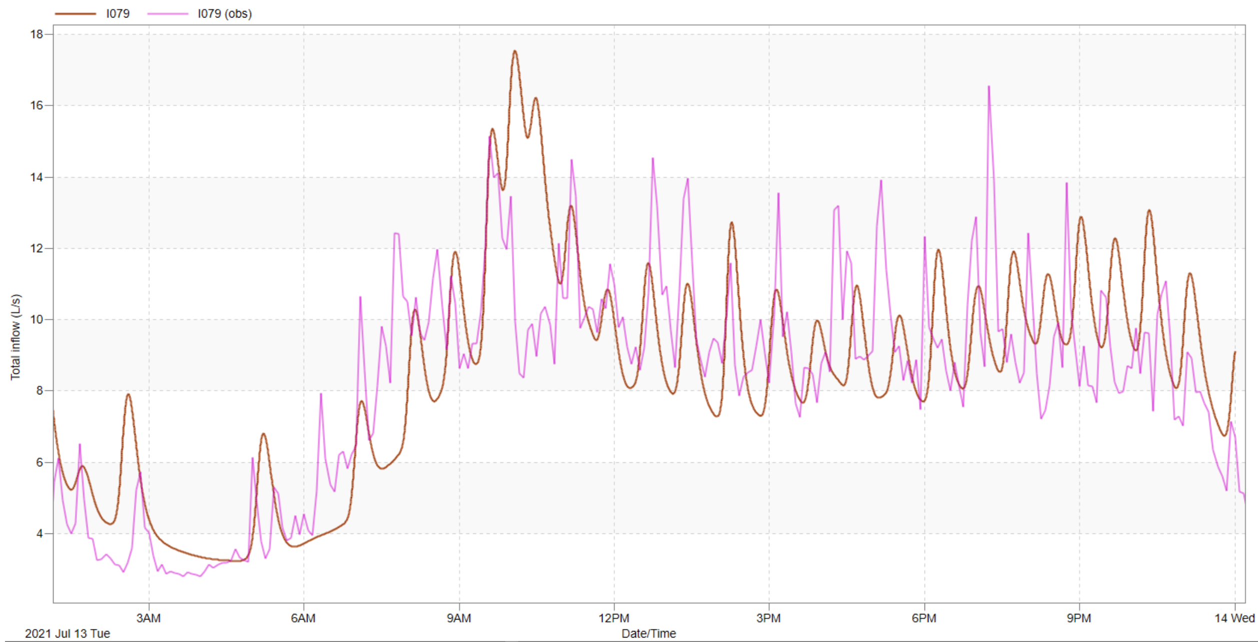


Figure D4 – Model and Field Data Comparison
Gauge #4 (Dry Weather)

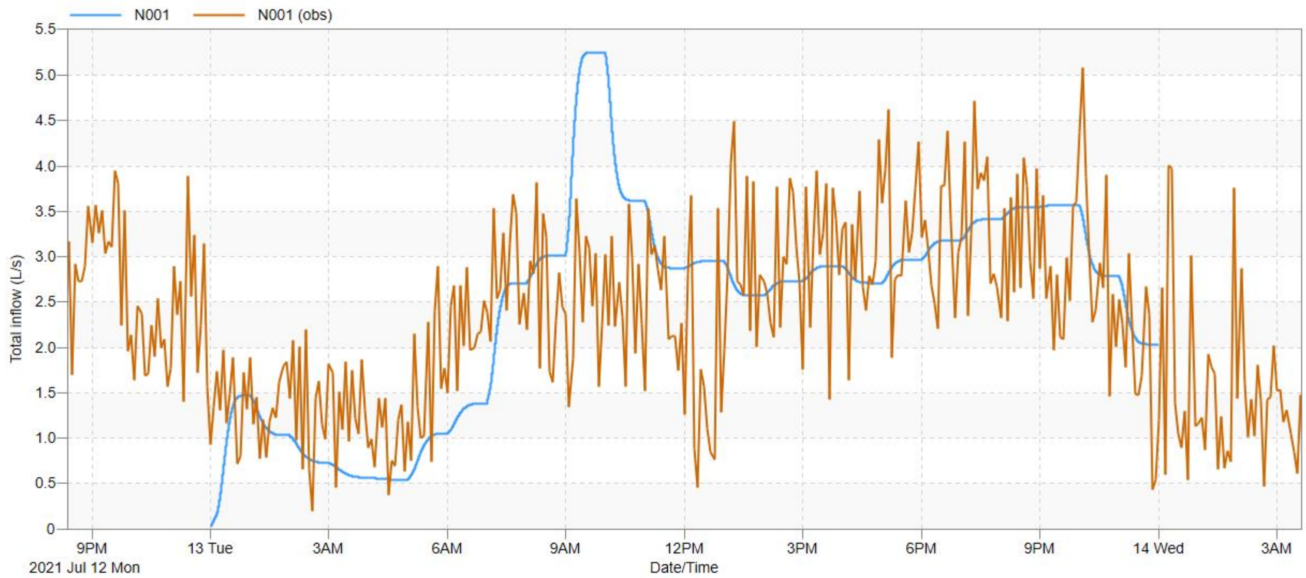
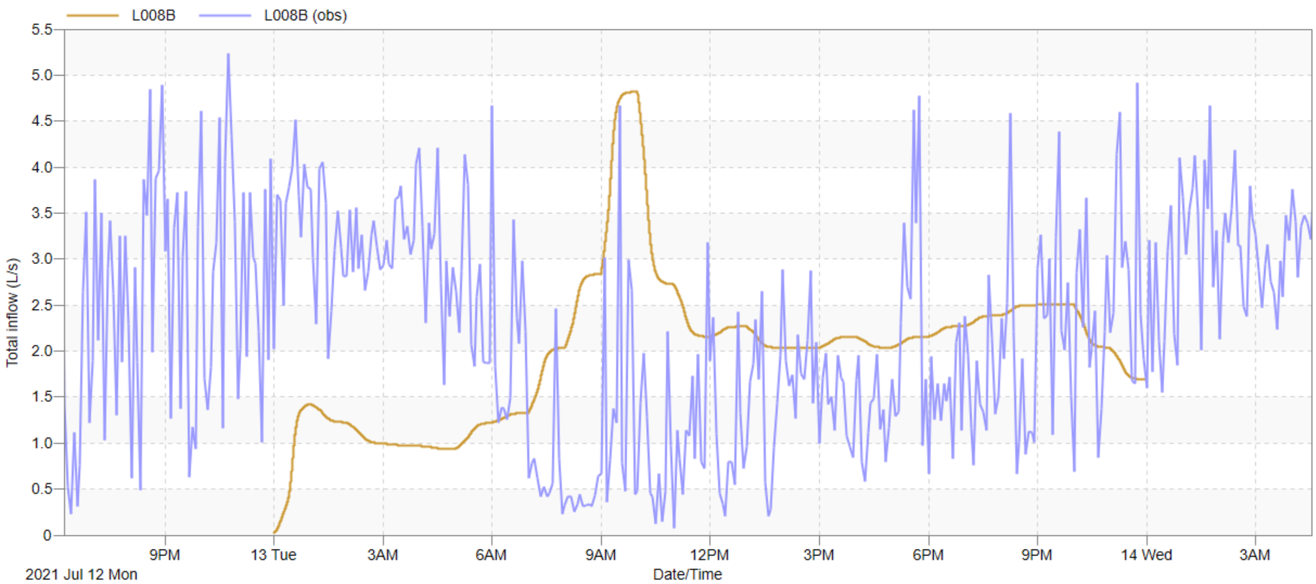


Figure D5 – Model and Field Data Comparison
Gauge #5 (Dry Weather)



The patterns align fairly well at each gauge, with the exception of gauge #5 which captures flow from the hospital which has completely different peaks than the typical residential pattern. At each location, the peak flows and volumes between the model and gauge align within 5%, so even though the model may be slightly conservative, there is strong confidence that the model will not overestimate future sewer main replacement sizing.

Note that we used our best estimate for operation of the Northshore, Lakeside Park and 4th Street lift stations. Exact timing of peaks slightly differs between the model and field recording. However, the overall peak flows and volumes match between the two. Additional time to exactly replicate the field data in the model was not felt to be warranted, given that the capacity analysis will be solely focused on peak flows.

3.3.2 Peak Wet weather flow (PWWF)

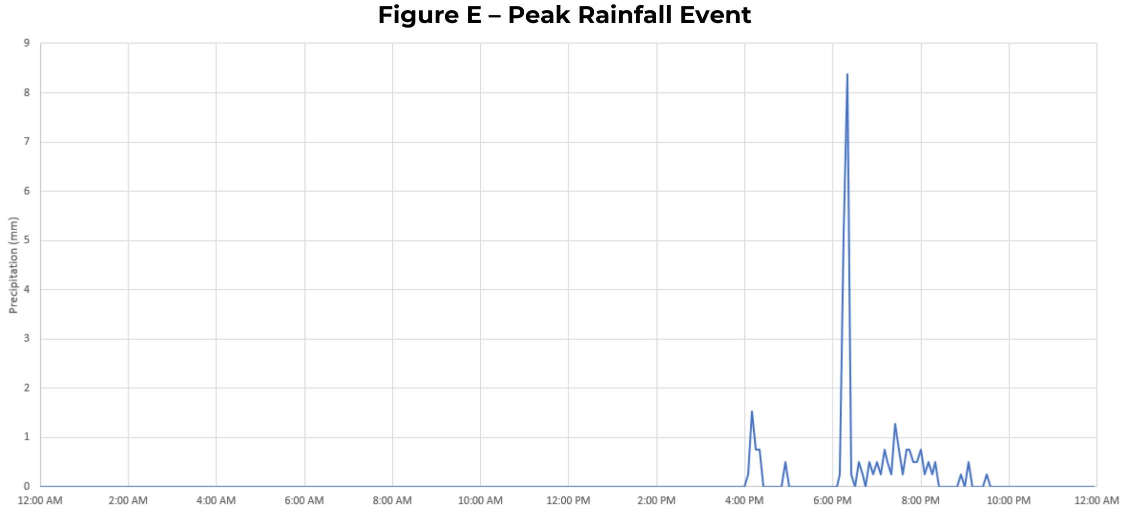
Peak wet weather flows – the difference between the values in Table 3 and Table 4 – are the result of directly connected catch basins and rainwater leaders to the sewer system and sheet flow entering through manhole lid openings. The City has been disconnecting catch basins as opportunity allows. Table 5 lists the remaining catch basins still directing rainfall to the sanitary sewer.

Table 5 – Directly Connected Catch Basins

ID	Model Node	Basin Area (m2)	Location
1	O-032	1,400	5th Street and Gordon Street
2	K-013	630	200 block Behnsen Street (Safeway/Front Street)
3	I-023	1,000	300 block Baker Street (Rear lane heading to Kootenay St)
4	L-006	1,300	400 block of 4th Street (Behnsen Street)
5	M-015	800	Lane west of 400 block of 4th Street
6	M-023	1,200	400 block of 6th Street
7	M-019	1,000	Lane west of 400 block of 6th Street
8	B-082	900	Lane near 1400 Slocan/Vancouver Street
9	J-010A	700	Lane north of 800 block of Vernon Street

The rain gauge provided precipitation data for the peak rainfall event on August 22, 2021, as shown below.

Figure E – Peak Rainfall Event



Runoff into the sanitary system during rainfall events was simulated in the model by adding a constant flow to each node in the catchment. The intent is to align peak flow in the model at each flow monitoring location during a peak wet weather event. The peak flows in the model were adjusted to be within 5% of the observed data on August 22, 2021. Note that volume alignment was not considered as part of the wet weather calibration as sewer main and lift station upgrades only require peak flows to properly size the improvements.

The City's known list of directly connected catch basins does not include any within the downtown core. However, the monitoring data reflects increased volume and peak flows from this catchment (gauge #1) during rain events, likely the result of rainwater leaders connected to the sanitary sewer. Smoke testing could potentially identify connect rainwater leaders within this zone.

4.0 CAPACITY ANALYSIS – EXISTING CONDITONS

The calibrated wet weather model suggests four areas of surcharging (flow above pipe crown) within the system, but no flooding, as shown in Figure 6:

- Lakeside Drive – from McDonald Drive to Nelson Avenue (Highway 3)
- McDonald Drive – from Lakeside Drive to the rail tracks
- Beatty Avenue – from 3rd Street to 4th Street
- Latimer/Stanley Streets – from Kootenay Street to Mill Street

Table 6 lists the characteristics of the sewer system lift stations as well as the anticipated peak flows under existing wet weather conditions.

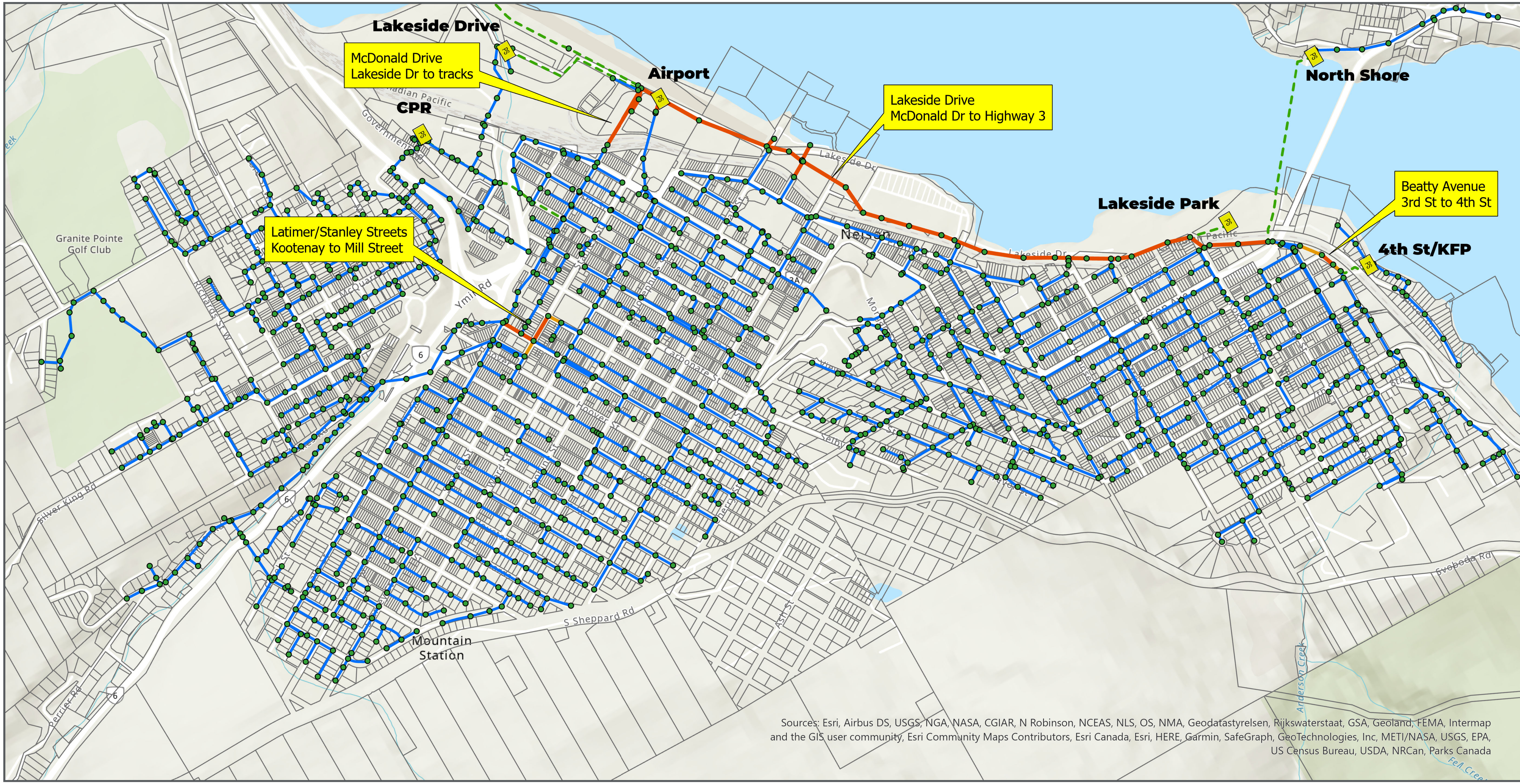
Table 6 – Peak Flows into Lift Stations (Existing Wet Weather Conditions)

Station	Estimated PWWF (L/s)	Number of Pumps	Pump Capacity (L/s)	Forcemain Size (mm)	Pump TDH (m)
Airport	384	3	189/226 (3)	375	35
CPR	26	2	30	150	38
Lakeside Drive	6 (2)	2	20	150	15
North Shore	5 (2)	2	30	150	30
Lakeside Park	1 (2)	1	3	100	10
4th Street	3 (2)	2	18	100	10
Tylar (1)	n/a (2)	2	3	unknown	10

(1) Station not modeled, very small catchment, limited potential for growth

(2) No calibration data available

(3) Capacity with two/three pumps operating, as observed August 22 when high intensity rainfall event occurred.



URBAN systems

200 100 0 200 400 600
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

Lift Stations
Manholes

Pipe Capacity Results
- Gravity, flowing 75% full or less
- Gravity, flowing 75% to 99% full
- Gravity, flowing 100% full
- Forcemain

Project #: 0795.Clientinfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 6 / 10

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 06
Sewer Main Capacity Issues
Existing Conditions

5.0 GROWTH PROJECTIONS

The City confirmed that a growth rate of 1.2% shall be utilized for the purposes of the WWMP update. The rate was applied to the 2016 census data to estimate the 2021 and 2041 populations of 11,222 and 14,245 respectively. In addition, the City provided a buildout population of 24,476 persons, which would be reached in 65 years, or 2086, at a sustained growth rate of 1.2%.

The City has provided the locations and unit counts for several development projects. The projects are assumed to be built-out by the 20 year horizon. The remainder of growth across the City will be uniformly applied to nodes in the model for the future conditions (2041 and buildout) models. Technical Memorandum No 1 – Design Criteria – Collection System Revision 1, dated January 13, 2022, provides the full details of the growth projects, including a map of their locations. The memorandum is included as an appendix to the WMP Update document.

6.0 CAPACITY ANALYSIS – FUTURE CONDITIONS

Future growth created additional capacity issues in the collection system. The growth associated with the 2041 design horizon expands on the Lakeside Drive capacity deficiency, extending furth north towards 4th Street, as shown in Figure 8. The future buildout scenario also increases capacity issues along Lakeside Drive, similar to the 2041 scenario, and triggers additional capacity upgrades along Kootenay Street between Robson and Observatory Streets, as shown in Figure 9. Table 9 provides the anticipated peak flows into the modeled lift stations under both future conditions.

Table 9 – Future Peak Flows into Lift Stations

Station	Estimated PWWF (L/s) 2041	Estimated PWWF (L/s) Buildout	Number of Pumps	Pump Capacity (L/s)	Forcemain Size (mm)	Pump TDH (m)
Airport	406	426	3	189/226	375	35
CPR	36	43	2	30	150	38
Lakeside Drive	7	10	2	20	150	15
North Shore	5	8	2	30	150	30
Lakeside Park	1	1	1	3	100	10
4th Street	10	11	2	18	100	10

Jason Barta, B.Sc.
Municipal Infrastructure Analyst
/jb

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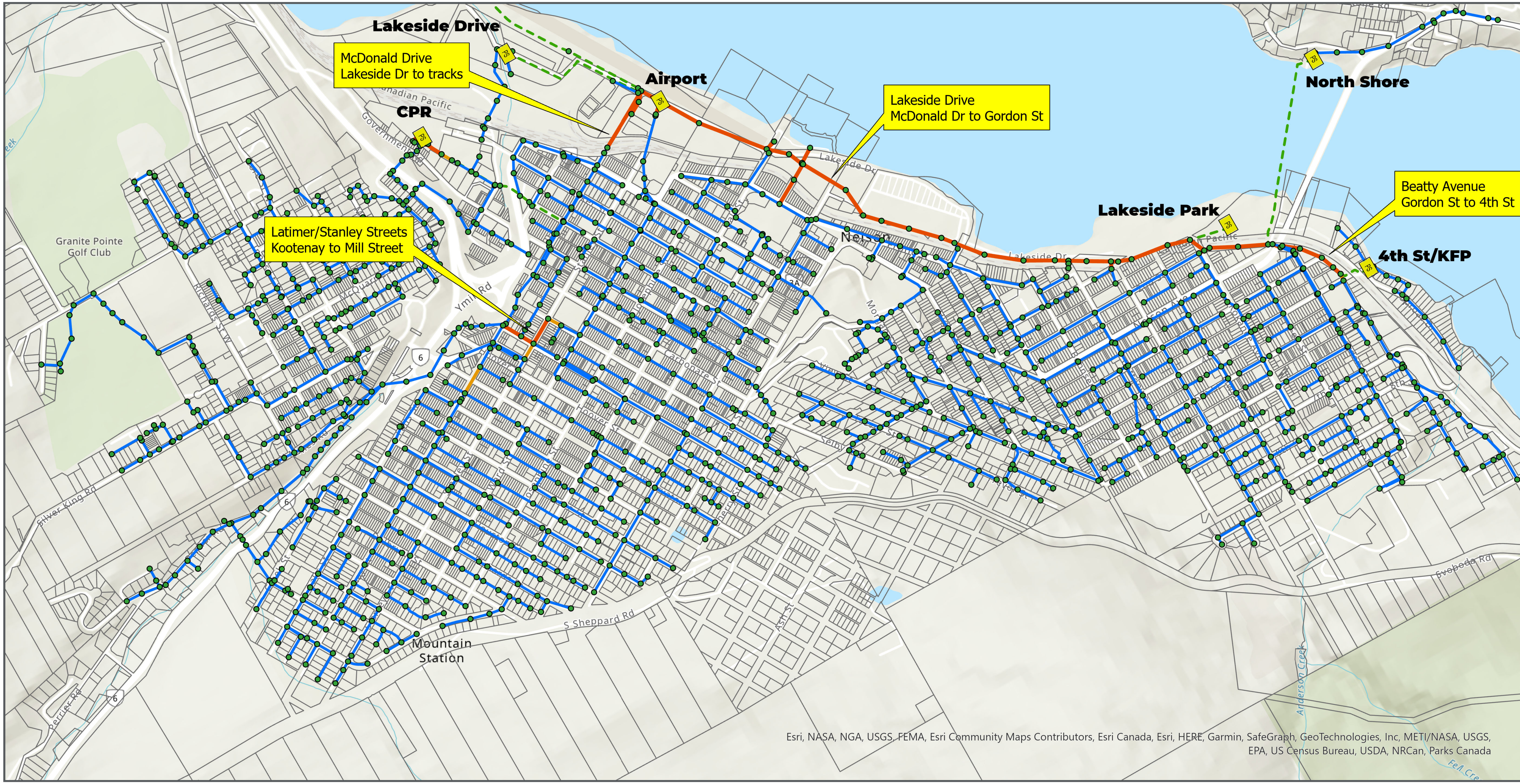


URBAN systems

Project #: 0795.Clientinfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 6 / 10

200 100 0 200 400 600
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

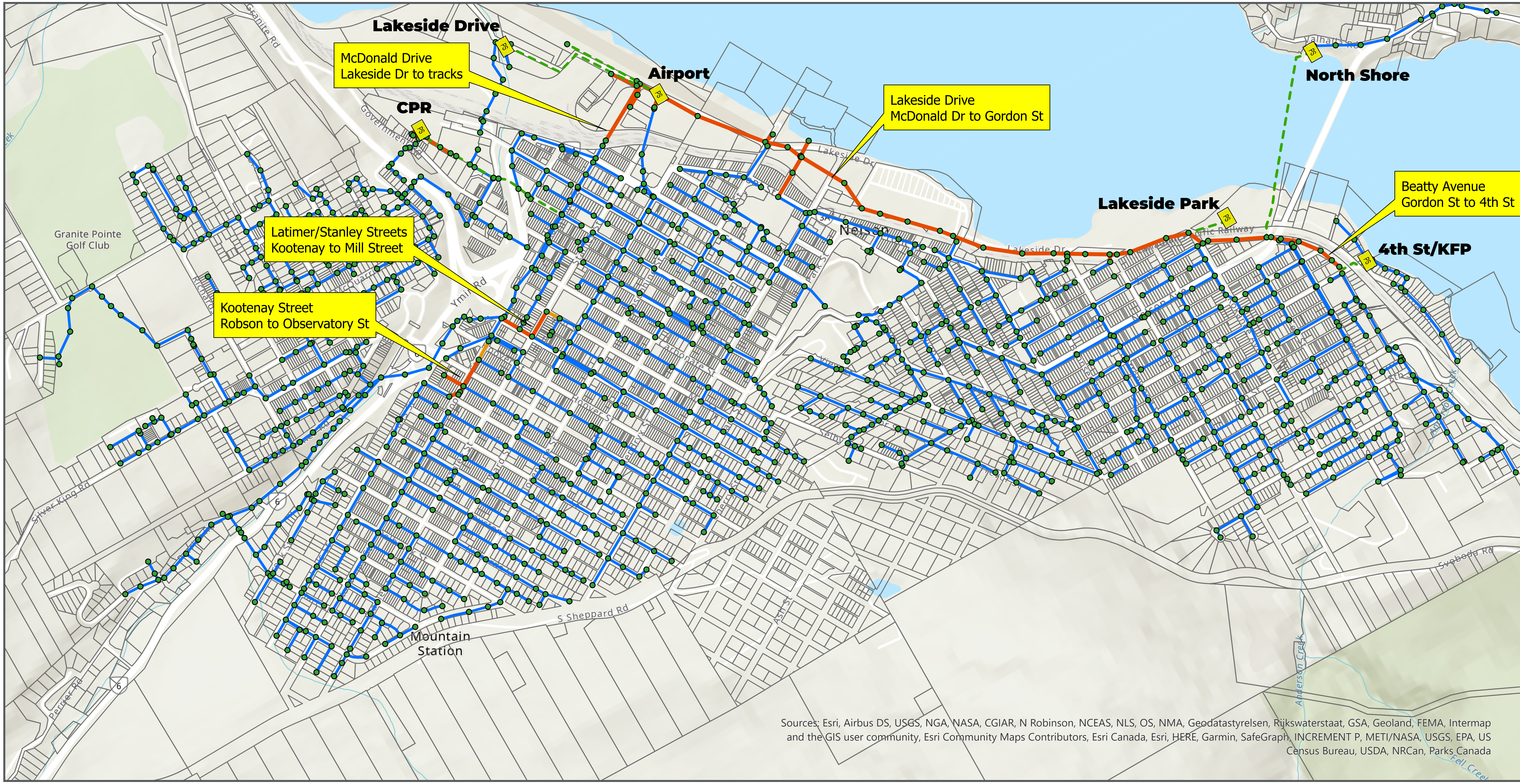
The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

- Lift Stations
- Manholes

Pipe Capacity Results
- Gravity, flowing 75% full or less
- Gravity, flowing 75% to 99% full
- Gravity, flowing 100% full
- Forcemain

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 08
Sewer Main Capacity Issues
2041 Growth Scenario



URBAN systems

50 25 0 50 100 150
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

- PS Lift Stations
- Manholes

Pipe Capacity Results
- Gravity, flowing 75% full or less
- Gravity, flowing 75% to 99% full
- Gravity, flowing 100% full
- Forcemains

Notes

Project #: 0795.ClientInfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 1 / 13

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 09
Sewer Main Capacity Issues
Buildout Growth Scenario

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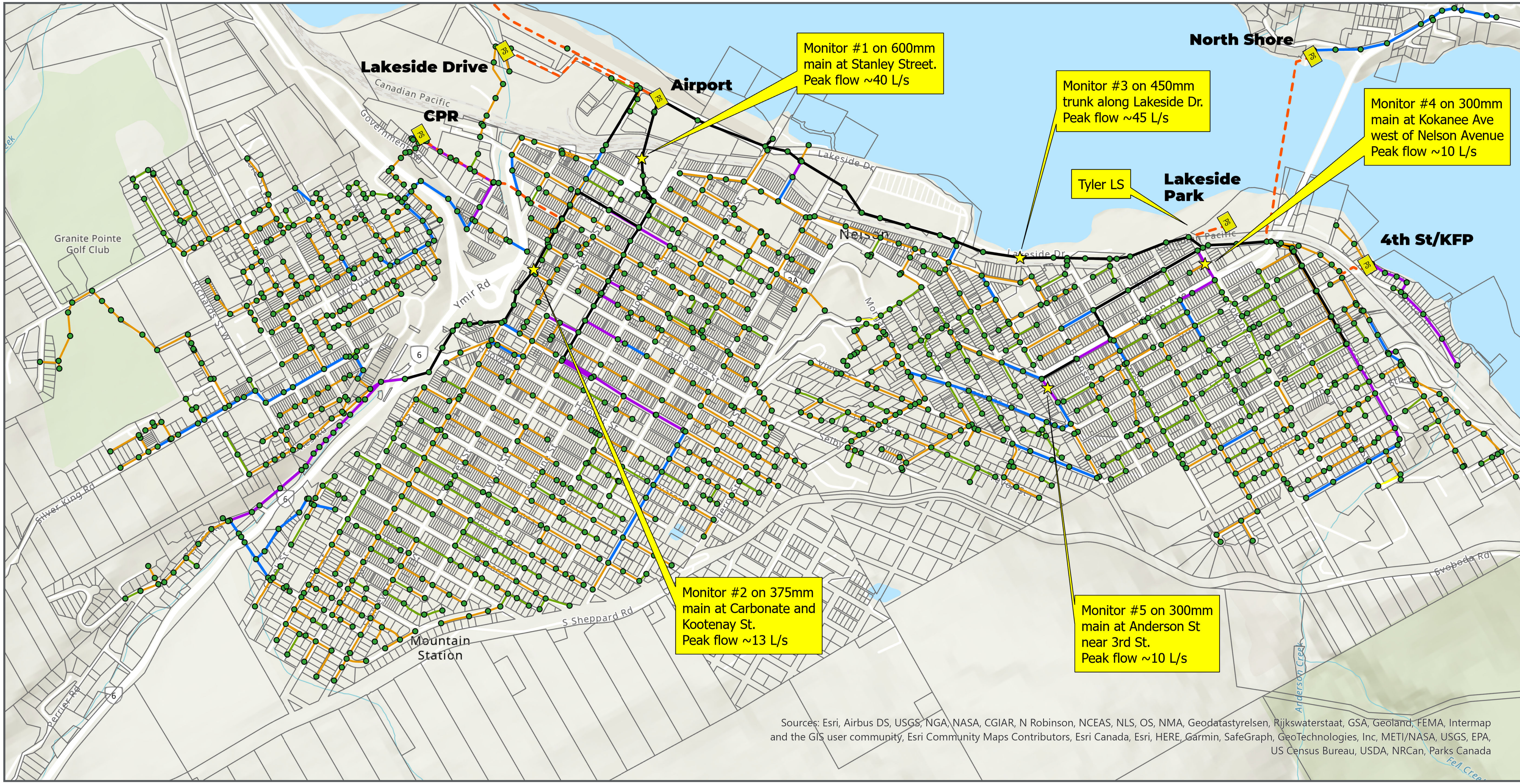
Appendix 1

Sanitary Sewer System



URBAN systems

Project #: 0795.ClientInfo
Author: JB
Checked: -
Status: Draft
Revision: A
Date: 2022 / 6 / 10

200 100 0 200 400 600
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:12,000 (When plotted at 11"x17")

Data Sources:
- City's PCSWMM Model

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

Model - Pipes
- Gravity, 100mm
- Gravity, 150mm
- Gravity, 200mm
- Gravity, 250mm
- Gravity, 300mm
- Gravity, 375mm or larger
- Forcemains

- Flow Monitors
- Model - Manholes
- Model - Lift Stations

Notes

Peak flow observed during rainfall event on August 22, 2021.

City of NELSON

City of Nelson
Sanitary Sewer System
FIGURE 01
Sanitary Sewer System and Flow Monitoring Locations

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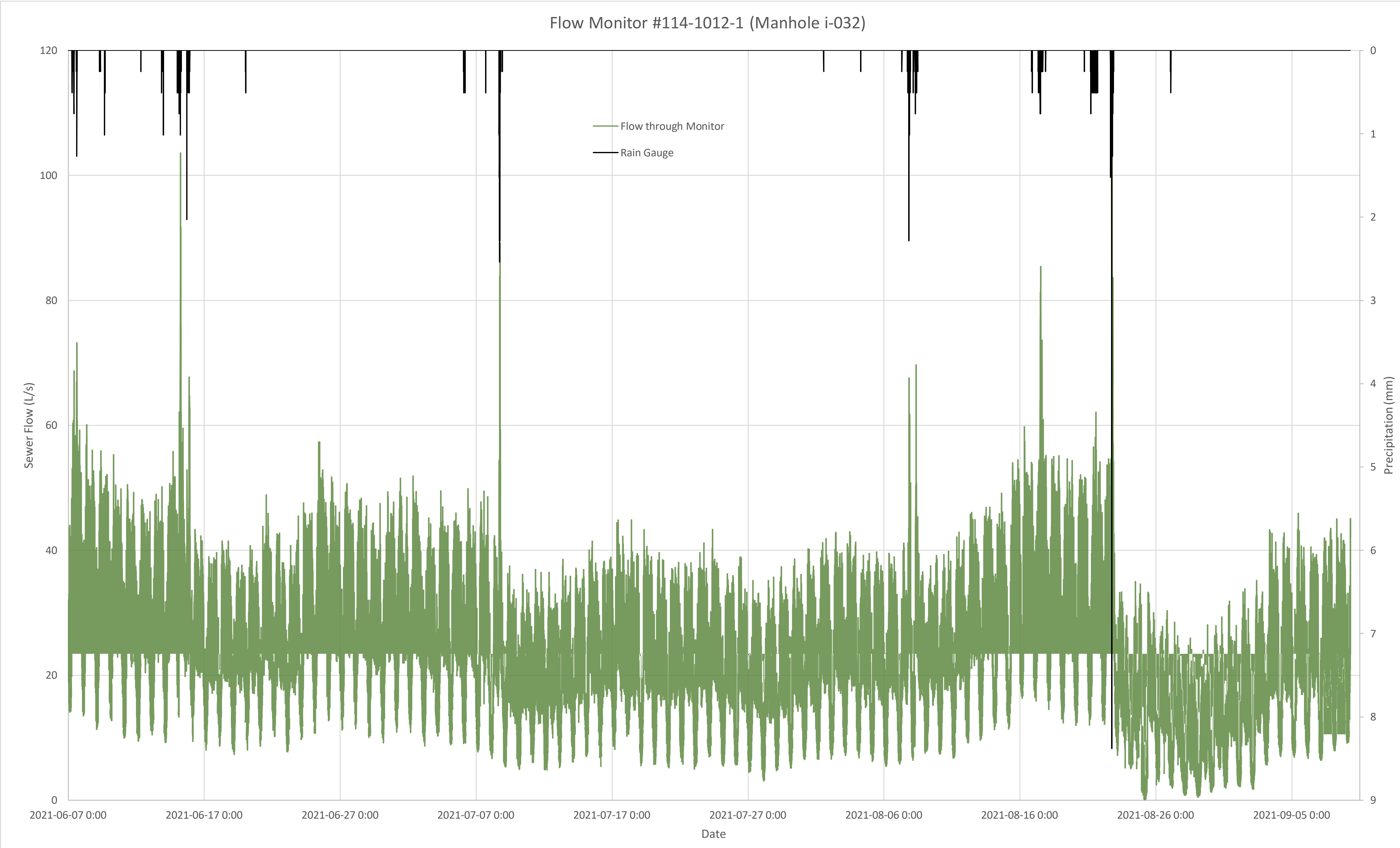
Appendix 2

Flow Monitoring Data – Graphs



Flow Monitor #114-1012-1 (Manhole i-032)
Flow through Monitor
Rain Gauge
Sewer Flow (L/s)
Precipitation (mm)
Date
0
20
40
60
80
100
120
0
1
2
3
4
5
6
7
8
9
2021-06-07 0:00
2021-06-17 0:00
2021-06-27 0:00
2021-07-07 0:00
2021-07-17 0:00
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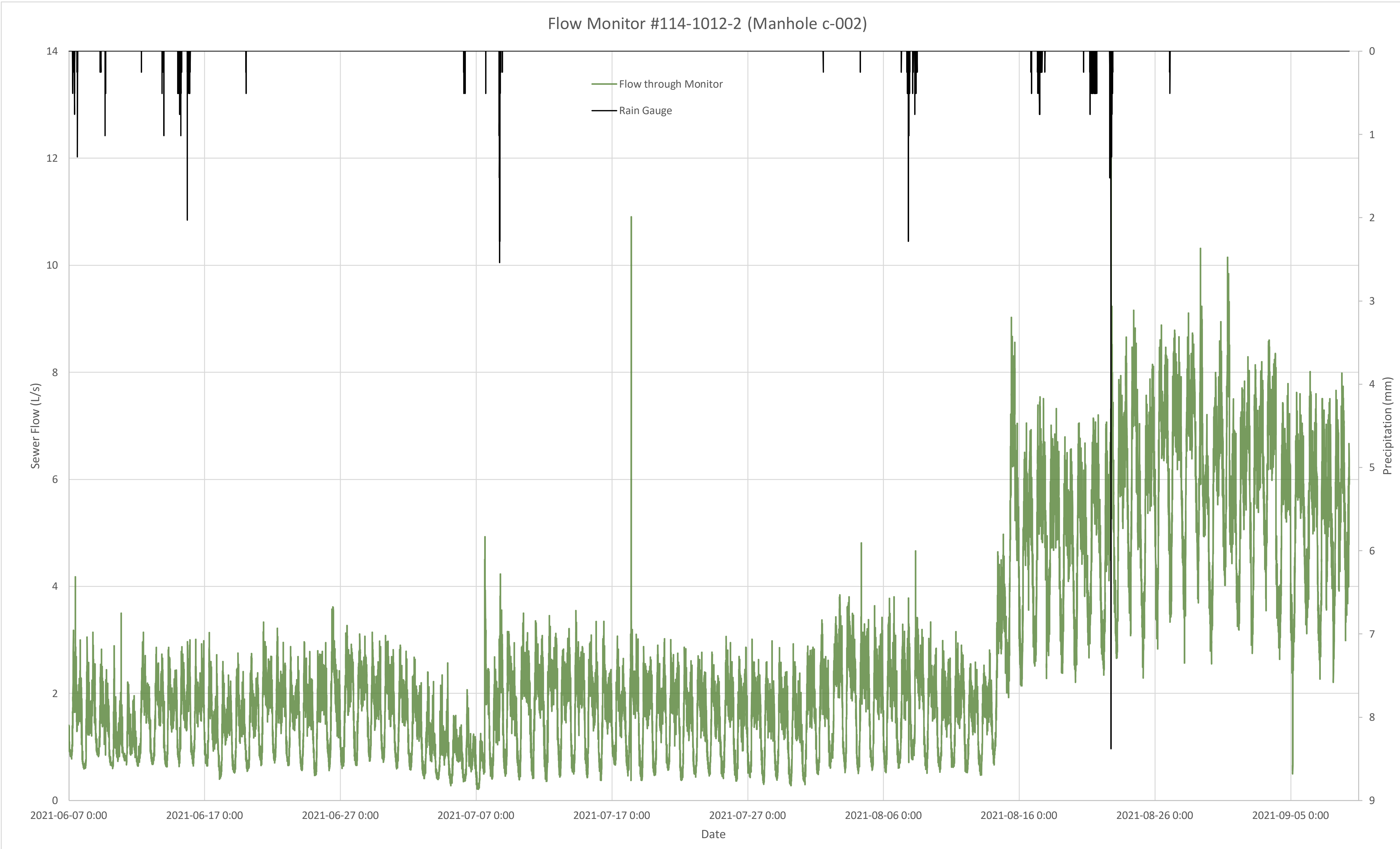
2021-08-16 0:00
2021-08-26 0:00
2021-09-05 0:00



Flow Monitor #114-1012-2 (Manhole c-002)
Flow through Monitor
Rain Gauge
Sewer Flow (L/s)
Precipitation (mm)
Date
0
2
4
6
8
10
12
14
0
1
2
3
4
5
6
7
8
9
2021-06-07 0:00
2021-06-17 0:00
2021-06-27 0:00
2021-07-07 0:00
2021-07-17 0:00
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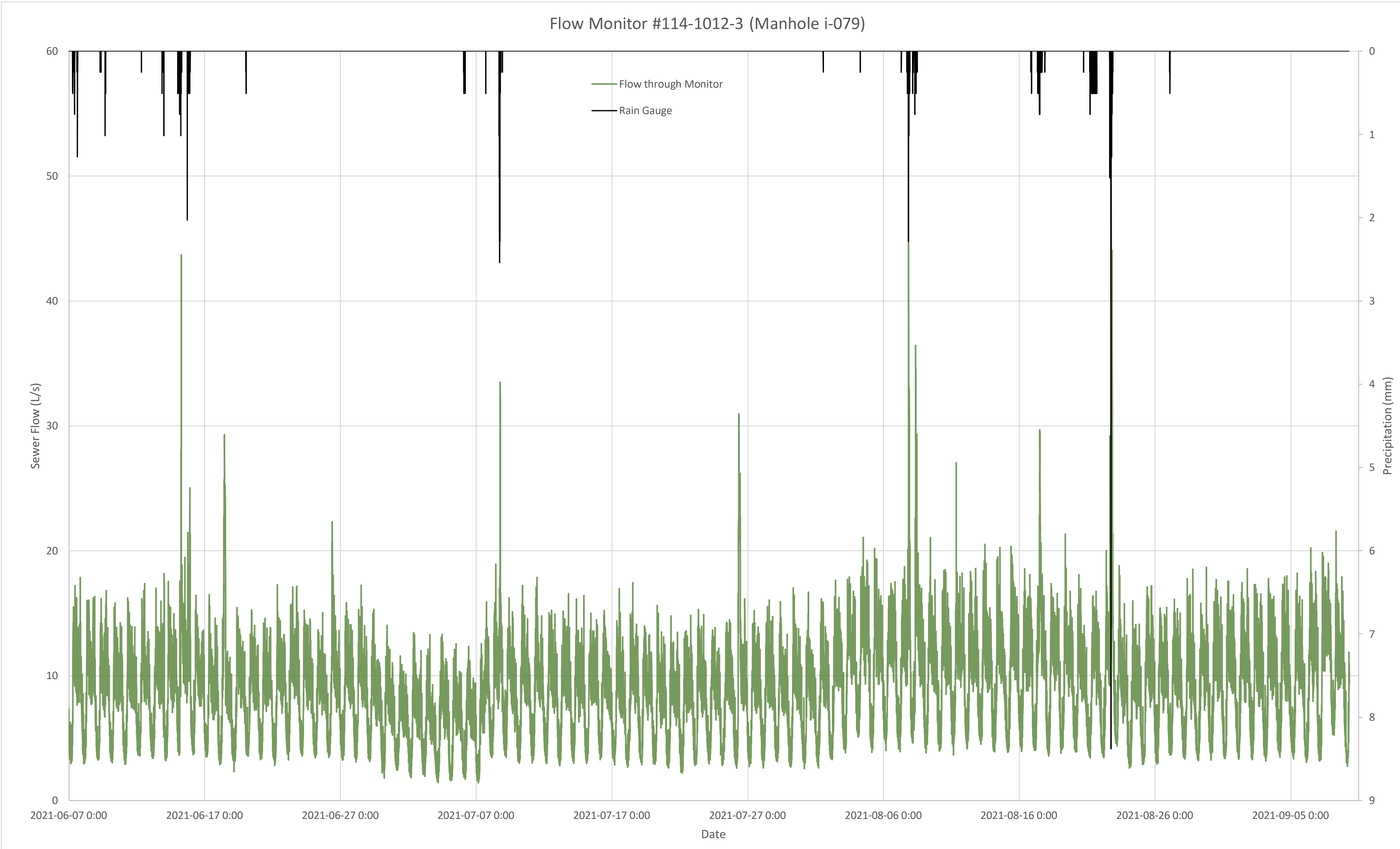
2021-09-05 0:00



Flow Monitor #114-1012-3 (Manhole i-079)
Flow through Monitor
Rain Gauge
Sewer Flow (L/s)
Precipitation (mm)
Date
0
10
20
30
40
50
60
0
1
2
3
4
5
6
7
8
9
2021-06-07 0:00
2021-06-17 0:00
2021-06-27 0:00
2021-07-07 0:00
2021-07-17 0:00
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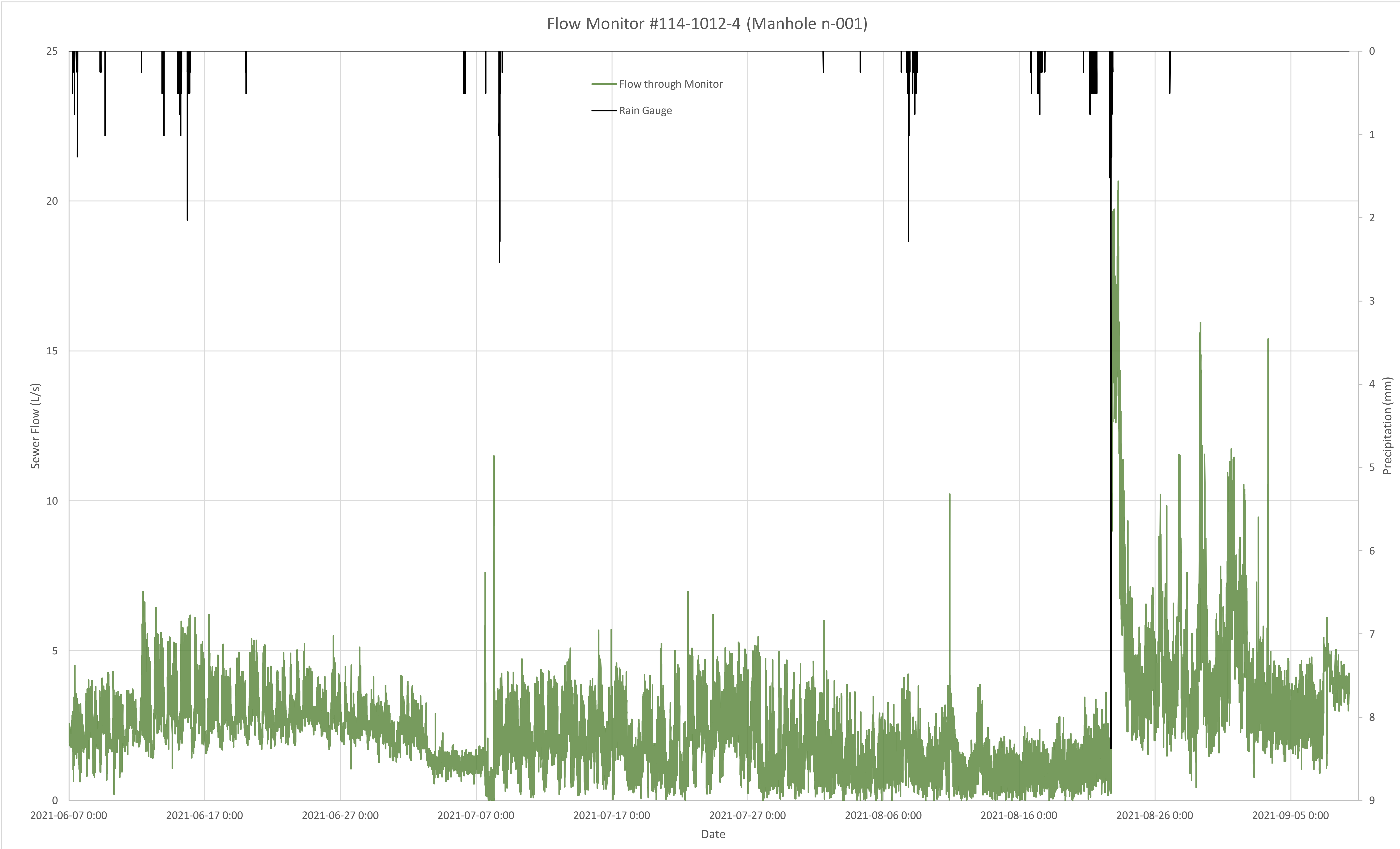
2021-09-05 0:00


Flow Monitor #114-1012-4 (Manhole n-001)
Flow through Monitor
Rain Gauge
Sewer Flow (L/s)
Precipitation (mm)
Date
0
5
10
15
20
25
0
1
2
3
4
5
6
7
8
9
2021-06-07 0:00
2021-06-17 0:00
2021-06-27 0:00
2021-07-07 0:00
2021-07-17 0:00
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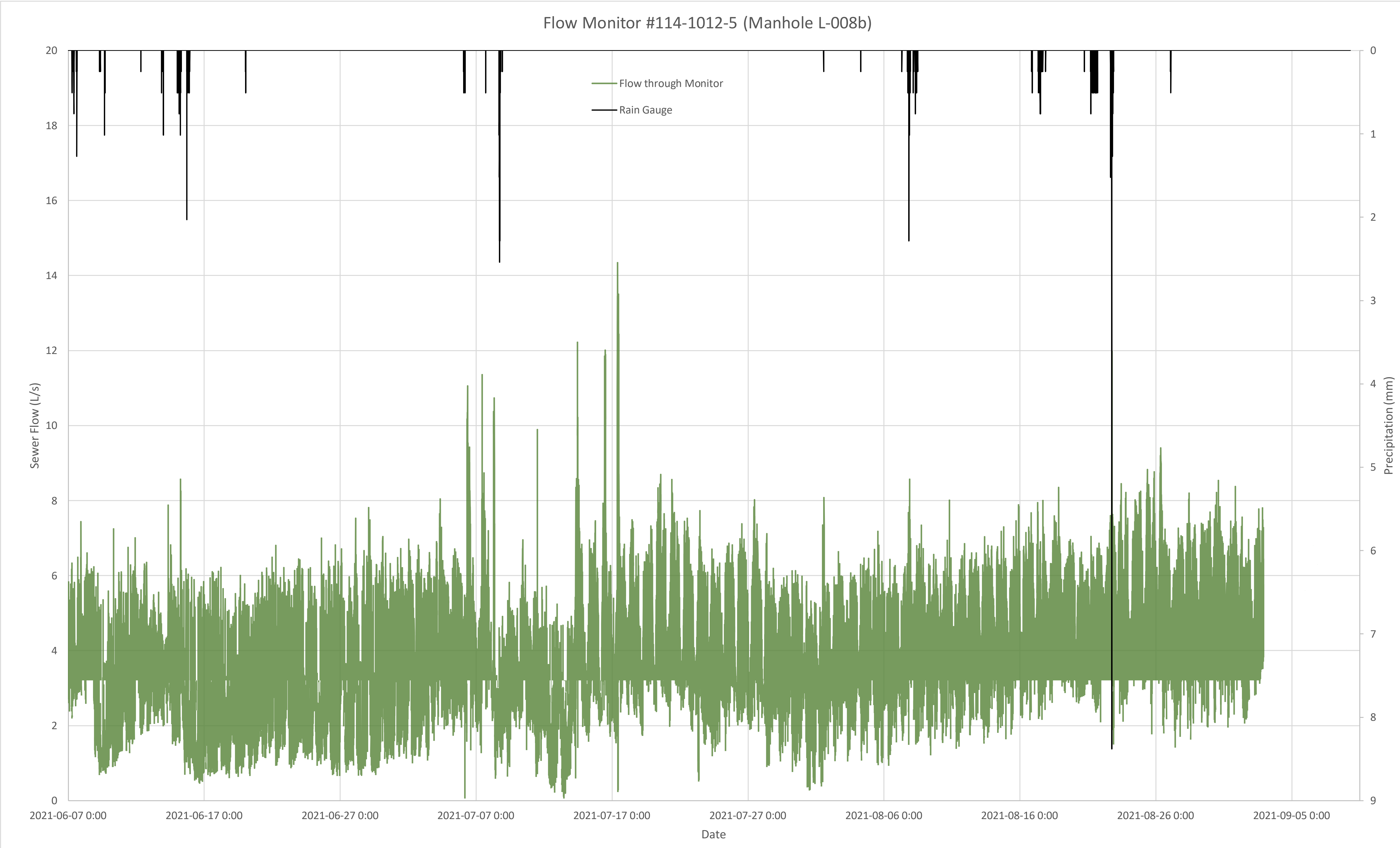
2021-09-05 0:00



Flow Monitor #114-1012-5 (Manhole L-008b)
Flow through Monitor
Rain Gauge
Sewer Flow (L/s)
Precipitation (mm)
Date
0
2
4
6
8
10
12
14
16
18
20
0
1
2
3
4
5
6
7
8
9
2021-06-07 0:00
2021-06-17 0:00
2021-06-27 0:00
2021-07-07 0:00
2021-07-17 0:00
2021-07-27 0:00
2021-08-06 0:00
2021-08-16 0:00
2021-08-26 0:00
2021-09-05 0:00

APPENDIX C:

COLLECTION SYSTEM OPINIONS OF PROBABLE COST

City of Nelson - Sewer Main Capacity Upgrades
Opinion of Probable Cost

URBAN SYSTEMS

Job No: 0795.0119.01
Date: 19-Jan-22
Prepared by: J. Barta
Checked by: J. Clowes

Project #	DESCRIPTION	UNIT	TOTAL QUANTITY	UNIT PRICE	TOTAL
S-1	**Lakeside Drive from Hendryx Street to Kootenay Avenue**				
	750mm ø pipe, PVC SDR35	m	260	$ 938	$ 244,000
	600mm ø pipe, PVC SDR35	m	1070	$ 750	$ 803,000
	Tie-in to existing manhole or new manhole	ea	18	$ 7,000	$ 126,000
	Reconnect existing sewer services	ea	5	$ 3,500	$ 18,000
	Import trench backfill (assume 50% replacement required)	cu.m	8189	$ 45	$ 369,000
	Remove and dispose existing asphalt	sq.m	9643	$ 7	$ 67,000
	Pavement restoration (75mm/100mm/300mm)	sq.m	9643	$ 66	$ 636,000
	Utility conflict allowance	LS	1	$ 100,000	$ 100,000
	Dewatering	lm	998	$ 150	$ 149,625
				Subtotal	$ 2,512,625
				Contingency (35%)	$ 879,419
				Engineering (15%)	$ 508,807
				Total	**$ 3,900,850**
S-2	**Beatty Avenue from Gordon Street to 4th Street**				
	300mm ø pipe, PVC SDR35	m	210	$ 375	$ 79,000
	Tie-in to existing manhole or new manhole	ea	6	$ 5,500	$ 33,000
	Reconnect existing sewer services	ea	4	$ 3,500	$ 14,000
	Import trench backfill (assume 50% replacement required)	cu.m	159	$ 45	$ 7,000
	Remove and dispose existing asphalt	sq.m	840	$ 7	$ 6,000
	Pavement restoration (75mm/100mm/300mm)	sq.m	840	$ 66	$ 55,000
				Subtotal	$ 194,000
				Contingency (35%)	$ 67,900
				Engineering (15%)	$ 39,285
				Total	**$ 301,185**
S-3	**Mill Street from Ward Street to laneway**				
	375mm ø pipe, PVC SDR35	m	70	$ 469	$ 33,000
	Tie-in to existing manhole or new manhole	ea	2	$ 5,500	$ 11,000
	Reconnect existing sewer services	ea	3	$ 3,500	$ 11,000
	Import trench backfill (assume 50% replacement required)	cu.m	94	$ 45	$ 4,000
	Remove and dispose existing asphalt	sq.m	280	$ 7	$ 2,000
	Pavement restoration (75mm/100mm/300mm)	sq.m	280	$ 66	$ 18,000
				Subtotal	$ 79,000
				Contingency (35%)	$ 27,650
				Engineering (15%)	$ 15,998
				Total	**$ 122,648**
S-4	**Stanley Street from Latimer Street to Mill Street**				
	300mm ø pipe, PVC SDR35	m	95	$ 375	$ 36,000
	Tie-in to existing manhole or new manhole	ea	2	$ 5,500	$ 11,000
	Reconnect existing sewer services	ea	1	$ 3,500	$ 4,000
	Import trench backfill (assume 50% replacement required)	cu.m	72	$ 45	$ 3,000
	Remove and dispose existing asphalt	sq.m	380	$ 7	$ 3,000
	Pavement restoration (75mm/100mm/300mm)	sq.m	380	$ 66	$ 25,000
				Subtotal	$ 82,000
				Contingency (35%)	$ 28,700
				Engineering (15%)	$ 16,605
				Total	**$ 127,305**
S-5	**CPR forcemain from Lift Station to Hwy/Ymir Road**				
	200mm ø pipe, HDPE	m	95	$ 300	$ 29,000
	Import trench backfill (assume 50% replacement required)	cu.m	72	$ 50	$ 4,000
	Remove and dispose existing asphalt	sq.m	380	$ 7	$ 3,000
	Pavement restoration (75mm/100mm/300mm)	sq.m	380	$ 66	$ 25,000
				Subtotal	$ 61,000
				Contingency (35%)	$ 21,350
				Engineering (15%)	$ 12,353
				Total	**$ 94,703**
				All Projects - Subtotal	$ 2,928,625
				Contingency (35%)	$ 1,025,019
				Engineering (15%)	$ 593,047
				All Projects - Total	**$ 4,546,690**

Airport Lift Station Replacement
Opinion of Probable Cost

URBAN SYSTEMS

Job No: 0795.0119.01
Date: 01-Jun-22
Prepared by: J.Clowes
Checked by: S.Johnson

ITEM	DESCRIPTION	UNIT	TOTAL QUANTITY	UNIT PRICE	TOTAL
1.0	**General**				
	Mobilization/demobilization	LS	1	$ 50,000.00	$ 50,000.00
	Insurance and bonding	LS	1	$ 8,000.00	$ 8,000.00
	Bypass pumping	LS	1	$ 50,000.00	$ 50,000.00
	Dewatering	LS	1	$ 200,000.00	$ 200,000.00
2.0	**Removals**				
	Existing lift station and piping	LS	1	$ 10,000.00	$ 10,000.00
3.0	**Site Works**				
	600 mm C900 PVC piping	l.m.	600	$ 800.00	$ 480,000.00
	600 mm plug valve	ea.	1	$ 30,000.00	$ 30,000.00
	Connect to existing piping (d/s of grit chamber)	LS	1	$ 50,000.00	$ 50,000.00
	Misc site works (extended gravel area, seeding)	LS	1	$ 10,000.00	$ 10,000.00
4.0	**Lift Station**				
	Concrete wet well (4.2 length x 3.0 width x 7.8 depth, 40 m^3 active storage)	cu.m.	98	$ 1,000.00	$ 98,280.00
	Lift station building (electrical and process rooms)	sq.m.	60	$ 4,000.00	$ 240,000.00
	Pumps (250 L/s at XX TDH m)	ea.	3	$ 105,000.00	$ 315,000.00
	400 mm std wall SS 304 pipe	l.m.	25	$ 3,000.00	$ 75,000.00
	400 mm plug valve	ea.	4	$ 25,000.00	$ 100,000.00
	400 mm check valve	ea.	4	$ 40,000.00	$ 160,000.00
	75 mm air release	ea.	2	$ 10,000.00	$ 20,000.00
	Clamp on flow meter	ea.	1	$ 20,000.00	$ 20,000.00
	Electrical and instrumentation (includes genset)	LS	1	$ 600,000.00	$ 600,000.00

Subtotal	$ 2,516,280.00
Contingency (35%)	$ 880,698.00
Engineering (15%)	$ 509,546.70
Total	$ 3,906,524.70
Rounded Total	$ 3,907,000.00

City of Nelson - Lift Station Condition Based Upgrades
Opinion of Probable Cost

URBAN SYSTEMS

Job No: 0795.0119.01
Date: 01-Jun-22
Prepared by: J.Clowes
Checked by: S.Johnson

ITEM	DESCRIPTION	UNIT	TOTAL QUANTITY	UNIT PRICE	TOTAL
1.0	**Airport Lift Station**				
	Replacement of electrical equipment and standby generator (in progress)	LS	1	$ 250,000.00	$ 250,000.00
	Repair or replace pressure transmitter	ea.	1	$ 3,000.00	$ 3,000.00
	Pump 2 repair	ea.	1	$ 20,000.00	$ 20,000.00
				Subtotal	$ 273,000.00
				Contingency (35%)	$ 95,550.00
				Engineering (15%)	$ 55,282.50
				Total	**$ 423,832.50**
2.0	**Lakeside Drive Lift Station**				
	Add alternate style level transmitter to deal with humidity issue	ea.	1	$ 3,000.00	$ 3,000.00
	Remove heat lamp from wet well	ea.	1	$ -	$ -
	Repair valve for pump 2	ea.	1	$ 10,000.00	$ 10,000.00
	Replace temporary cord with permanent wiring	ea.	1	$ 500.00	$ 500.00
				Subtotal	$ 13,500.00
				Contingency (35%)	$ 4,725.00
				Engineering (15%)	$ 2,733.75
				Total	**$ 20,958.75**
3.0	**CP Rail Lift Station**				
	Pump 2 repair	LS	1	$ 10,000.00	$ 10,000.00
	Adjust float mounting braket	LS	1	$ 2,000.00	$ 2,000.00
	Verify wet well is sealed	LS	1	$ 6,000.00	$ 6,000.00
				Subtotal	$ 18,000.00
				Contingency (35%)	$ 6,300.00
				Engineering (15%)	$ 3,645.00
				Total	**$ 27,945.00**
4.0	**Tyler Lift Station**				
	Verify wet well is sealed to prevent migration of explosive gas to electrical equipment	LS	1	$ 6,000.00	$ 6,000.00
				Subtotal	$ 6,000.00
				Contingency (35%)	$ 2,100.00
				Engineering (15%)	$ 1,215.00
				Total	**$ 9,315.00**
5.0	**Lakeside Park Lift Station**				
	Verify wet well is sealed to prevent migration of explosive gas to electrical equipment	LS	1	$ 10,000.00	$ 10,000.00
	Shelf spare pump	ea.	1	$ 5,000.00	$ 5,000.00
	Connect to SCADA	ea.	1	$ 40,000.00	$ 40,000.00
				Subtotal	$ 55,000.00
				Contingency (35%)	$ 19,250.00
				Engineering (15%)	$ 11,137.50
				Total	**$ 85,387.50**
6.0	**KFP/4th Street Lift Station**				
	Add junction box with seals on 3 conduits that extend from wet well to electrical equipment	LS	1	$ 6,000.00	$ 6,000.00
	Investigate why genset does not turn off when utility power is restored	LS	1	$ 5,000.00	$ 5,000.00
				Subtotal	$ 11,000.00
				Contingency (35%)	$ 3,850.00
				Engineering (15%)	$ 2,227.50
				Total	**$ 17,077.50**
7.0	**North Shore Lift Station**				
	Investigate why genset does not turn off when utility power is restored	LS	1	$ 5,000.00	$ 5,000.00
	Add plugs in MCC door	LS	1	$ 500.00	$ 500.00
				Subtotal	$ 5,500.00
				Contingency (35%)	$ 1,925.00
				Engineering (15%)	$ 1,113.75
				Total	**$ 8,538.75**
				All Sites - Subtotal	$ 382,000.00
				Contingency (35%)	$ 133,700.00
				Engineering (15%)	$ 77,355.00
				All Sites - Total	**$ 593,055.00**

City of Nelson - Airport Lift Station Forcemain Replacement Options
Opinion of Probable Cost

URBAN SYSTEMS

Job No: 0795.0119.01
Date: 01-Jun-22
Prepared by: J.Clowes
Checked by: S.Johnson

ITEM	DESCRIPTION	UNIT	TOTAL QUANTITY	UNIT PRICE	TOTAL
1.0	**CPR Alignment Option**				
	Clearing and Grubbing	LS	1	$ 100,000.00	$ 100,000.00
	600 mm PVC C900	l.m.	3,300	$ 900.00	$ 2,970,000.00
	600 mm plug valve	ea.	9	$ 30,000.00	$ 270,000.00
	75 mm air release - direct bury style	ea.	8	$ 40,000.00	$ 320,000.00
	Rock blasting - assume 50% of length	l.m.	1,650	$ 600.00	$ 990,000.00
	Import backfill - assume 50% remove and replace	cu.m.	5,115	$ 50.00	$ 255,750.00
	Pavement Structure Restoration	sq.m.	0	$ 85.00	$ -
	Creek Crossing using steel casing pipe	LS	1	$ 450,000.00	$ 450,000.00
	CPR Crossing using steel casing pipe	LS	2	$ 450,000.00	$ 900,000.00
	Connect to Existing	ea.	2	$ 50,000.00	$ 100,000.00
	Dewatering - assume 50% of length	l.m.	1,650	$ 200.00	$ 330,000.00
				Subtotal	$ 6,685,750.00
				Contingency (35%)	$ 2,340,012.50
				Engineering (15%)	$ 1,353,864.38
				Total	**$ 10,379,626.88**
				Rounded Total	**$ 10,380,000.00**
2.0	**Highway 3A Alignment Option**				
	Clearing and Grubbing	LS	1	$ 100,000.00	$ 100,000.00
	600 mm PVC C900	l.m.	3,500	$ 900.00	$ 3,150,000.00
	600 mm plug valve	ea.	9	$ 30,000.00	$ 270,000.00
	75 mm air release - direct bury style	ea.	6	$ 40,000.00	$ 240,000.00
	Rock blasting - assume 25% of length	l.m.	875	$ 600.00	$ 525,000.00
	Import backfill - assume 50% remove and replace	cu.m.	5,425	$ 50.00	$ 271,250.00
	Pavement Structure Restoration - 15% of alignment	sq.m.	2,100	$ 85.00	$ 178,500.00
	Creek Crossing using steel casing pipe	LS	1	$ 450,000.00	$ 450,000.00
	CPR Crossing using steel casing pipe	LS	2	$ 450,000.00	$ 900,000.00
	Connect to Existing	ea.	2	$ 50,000.00	$ 100,000.00
	Dewatering - assume 20% of length	l.m.	700	$ 200.00	$ 140,000.00
				Subtotal	$ 6,324,750.00
				Contingency (35%)	$ 2,213,662.50
				Engineering (15%)	$ 1,280,761.88
				Total	**$ 9,819,174.38**
				Rounded Total	**$ 9,820,000.00**
3.0	**Kootenay River Alignment Option**				
	600 mm HDPE pipe c/w concrete ballast	l.m.	3,100	$ 1,000.00	$ 3,100,000.00
	600 mm plug valve	ea.	2	$ 30,000.00	$ 60,000.00
	75 mm air release - direct bury style	ea.	3	$ 40,000.00	$ 120,000.00
	Allowance for steel casing for underwater ravines (methodology for install TBC with contractor, quantity to be confirmed through bathametric survey)	l.m.	200	$ 10,000.00	$ 2,000,000.00
	Connect to Existing	ea.	2	$ 50,000.00	$ 100,000.00
	Dewatering - at each end of main	LS	1	$ 50,000.00	$ 50,000.00
				Subtotal	$ 5,430,000.00
				Contingency (35%)	$ 1,900,500.00
				Engineering (15%)	$ 1,099,575.00
				Total	**$ 8,430,075.00**
				Rounded Total	**$ 8,431,000.00**

APPENDIX D:

TECHNICAL MEMORANDUM NO. 3

LIFT STATION CONDITION ASSESSMENT

URBAN SYSTEMS
MEMORANDUM

DATE: June 6, 2022
TO: Colin Innes
CC: Rob Nystrom, Scott Eagleson
FROM: Shiloh Johnson, Jeremy Clowes
FILE: 0795.0119.01
SUBJECT: Technical Memo No.03 – Lift Station Condition Assessment, Rev.4

1.0 INTRODUCTION

The City of Nelson (City) currently operates seven sewage lift stations as part of their sanitary collection / conveyance systems. Urban Systems Ltd. along with Ready Engineering completed a condition assessment of each lift station that is summarized in this technical memorandum. This condition assessment is being completed as part of the City's update of their sanitary master plan and will be appended to the final master plan report. The following table provides a summary of each station.

Table 1.1 Lift Station Summary

Lift Station	Station Type	Pumping Configuration*	Pump Size	Confined Space	Standby Power	Grit Chamber	Conform to SDS Bylaw
Airport	Concrete wet-well (4.3 m x 3.0 m) / dry-well w/ control building	Triplex	75 HP	Yes	Yes	Yes	Yes
Lakeside Drive	Buried steel wet-well / dry-well	Duplex**	15 HP	Yes	No	No	No
CP Rail	Buried steel dry-well and concrete wet-well (1.8 m x 1.8 m)	Duplex	25 HP	Yes	Yes	No	Yes
Tyler Lake	Buried concrete wet-well w/ submersibles	Duplex	3 HP	Yes	No	No	No
Lakeside Park	Buried concrete wet-well (1.2 m x 1.2 m) w/ submersible	Simplex	3 HP	Yes	No	Yes	No
KFP (4th Street)	Buried brick and mortar wet-well (2.8 m dia.) w/ submersibles	Duplex	5 HP	Yes	Yes	No	No
North Shore	FRP wet well (2.4 m dia.) w/ submersibles and adjacent control and genset building	Duplex	10 HP	Yes	Yes	No	Yes

* Simplex = one pump, duplex = two pumps, triplex = 3 pumps
** Lakeside Drive LS is currently operating with one pump

2.0 LIFT STATION CONDITION ASSESSMENT

On September 22, 2021, Shiloh Johnson of Urban Systems Ltd. and Dave McIntosh from Ready Engineering were accompanied by City Staff to complete visual condition assessment of each lift station. Ready Engineering's electrical condition assessment is provided in **Appendix 1**.

2.1 AIRPORT LIFT STATION

All sewage in the City's collection system is conveyed to the Airport Lift Station where it is pumped through a 400 mm diameter steel marine forcemain to the Grohman Narrows Pollution Control Centre (PCC).

The Airport Lift Station is comprised of an upstream grit chamber, concrete wet-well/dry-well, and a triplex pump configuration. The pumps are Hayward Gordon Model VDP-XCS6A size 6x8x14 and are all operated with 75 HP motors. Pumps 1 and 2 operate on variable frequency drives (VFDs) while Pump 3 uses a soft starter. Upgrades were made to the pump discharge header, inlet, and outlet piping in 2007 and to the SCADA system in 2019.

Figure 2.1 Airport Lift Station

Given that this lift station pumps all of the City's sewage, the consequences are significant if a failure occurs. There are significant electrical deficiencies at this site which are documented in Ready's memorandum (refer to **Appendix 1**) and work is underway to correct these deficiencies. The following electrical equipment is in the process of being replaced including: 1) standby generator, 2) switchgear and MCC and 3) lighting.

A drawdown pump test was completed for each of the three pumps to determine if they have any deficiencies. The test included ramping up each pump to full power (60 Hz) and recording the flow, discharge pressure, and wet well level for a brief duration (between 2 and 3 minutes) to determine an average operating point for each pump. The following table summarizes the results from the drawdown test and their skewness from the pump manufacturer's published pump curve. From this drawdown test, it was determined that the pressure transmitter is faulty and should be replaced.

Table 2.1 Airport Lift Station Drawdown Pump Test

Pump	Drawdown Pump Test		Pump Curve**	TDH Deficiency (%)
	Total Flow (L/s)	TDH* (m)	TDH (m) at Test Flow	
1	143	21.4	25.0	-14%
2	138	19.3	25.7	-25%
3	134	21.4	26.5	-19%

* Drawdown Pump Test TDH was measured using a pressure gauge and pressure transmitter on the station's discharge header. There was a discrepancy between the pressure gauge and transmitter, and we expect the pressure transmitter to be inaccurate (i.e., recorded pressures from the transmitter were much lower than anticipated based on the manufacturer's pump curve). Above numbers are based on readings taken from the pressure gauge.

** The published curve for the pump indicates that the total dynamic head (TDH) should be higher than what was observed during flow test for each of the three pumps using the pressure gauge. It is common to observe lower than expected flow and/or pressures on older pumps due to wear occurring on impellers. A large performance deficiency warrants further investigation and could indicate the need for refurbishment work.

Given the large discrepancies between the drawdown test points and the published pump curve, further investigation is recommended. We'd suggest confirming the values noted in Table 2.1 with third test using a recently calibrated pressure gauge. If the results are similar, we recommend inspecting the condition of pump 2 and repairing as required. Repairing the pressure transmitter in the station is also recommended as having accurate flow and pressure data allows for regular pump performance monitoring and can help with determining proactive maintenance steps to take in advance of equipment failing.

In summary, the following observations were made requiring action:

- Short-term Recommended Actions
  - Electrical upgrades are in progress (refer to **Appendix 1**) and to be completed
  - The station's discharge pressure transmitter is inaccurate and requires repair.
  - Pump 2 is underperforming and may require refurbishment. Inspection and repair as required is recommended and if desired, additional performance testing can be completed in advance to verify results presented in Table 2.1.
  - Station's gravity inlet pipe does have not include any isolation valves. The City's confined space entry program should consider this and identify an isolation procedure that complies with Part 9 of the OHS Regulation in BC. Permanent valving or use of pneumatic plugs can be considered for isolating the station's wet well from the upstream collection system.
- Long-term Recommended Actions
  - Critical valving (pump discharge isolation and check valves) is in a confined space and difficult to access. Locate in above ground structure when station is upgraded to increase capacity or due to condition.

2.2 LAKESIDE DRIVE LIFT STATION

The Lakeside Drive lift station is a buried wet-well/dry-well duplex lift station. It could not be entered for assessment as the dry-well is a confined space. No HMI or SCADA has been installed on this lift station.

The following observations were made requiring action:

- Short-term Recommended Actions
  - Upstream gate valve stem broke for pump #2 in May 2019 and is stuck closed; only one pump is operating.
  - Level transmitter prone to issues caused by condensation
  - A heat lamp was installed in the wet-well to keep condensation off the level sensor and its not rated as explosion-proof.
  - Portable cord between dry and wet pit should be replaced with permanent wiring
- Long-term Recommended Actions
  - Steel wet well used which are prone to corrosion issues. Consider replacing with insert style FRP wet well when replacement is required due to condition. In addition, move critical valving to an above ground structure when station is upgraded or replaced.
  - Electrical controls are located below ground. Move to above ground kiosk.

Figure 2.2 Lakeside Drive Lift Station – Wet Well Hatch and Heat Lamp

2.3 CP RAIL LIFT STATION

Similar to the Lakeside Drive Station, the CP Rail lift station is a buried wet-well/dry-well configuration and could not be entered as it is a confined space. This station is located on private property which could significantly complicate any future repair or upgrades to the station.

Figure 2.3 CP Rail Lift Station

The lift station's programmable logic controller (PLC) has a human-machine interface (HMI) located in the electrical kiosk. There is a flowmeter on the station's discharge header that records and displays flow. Drawdown tests were completed for both pumps.

Table 2.2 CP Rail Lift Station Drawdown Pump Test

Pump	Draw Down Test Flow (L/s)	Rated Pump Flow (L/s)	Flow Deficiency
1	28.2	30	-6%
2	24.9	30	-17%

Pump one is performing reasonably close to the manufacturer's pump curve. Pump two has larger flow deficiency of 17% (or 5 l/s) and warrants further investigation. It is recommended that pump 2 be inspected and repaired as required. The performance test could be repeated with a clamp-on flow meter, in advance of pump inspection, to verify results if the City has any concerns with the accuracy of the station's flow meter.

In summary, the following observations were made that requiring action:

- Short-term Recommended Actions
    - The station is entirely trespassing on private property and the City should review options for addressing this land ownership issue.

- Pump 2 is underperforming and may require refurbishment. Inspection and repair as required is recommended and if desired, additional performance testing can be completed in advance to verify results presented in Table 2.2.

- Station's gravity inlet pipe does have not include any isolation valves. The City's confined space entry program should consider this and identify an isolation procedure that complies with Part 9 of the OHS Regulation in BC. Permanent valving or use of pneumatic plugs can be considered for isolating the station's wet well from the upstream collection system.
- Adjust mounting bracket for wet well level floats to provide easier access per operations staff feedback (refer to **Appendix 1**)
- Verify that wet well is sealed to prevent migration of explosive gases (refer to **Appendix 1**)

- Long-term Recommended Actions
    - Critical valving (pump discharge isolation and check valves) is in a confined space and difficult to access. Locate in above ground structure when station is upgraded to increase capacity or due to condition.

2.4 TYLER LAKE LIFT STATION

Tyler Lake LS is a small lift station that services the restroom / facilities building adjacent to the soccer fields at Lakeside Park. The lift station is comprised of a buried concrete wet-well with two submersible pumps and a level sensor. All pump controls are situated in a utility room in the building it services.

Figure 2.4 Tyler Lake Lift Station

The Tyler Lake Lift Station is privately owned, growth within its catchment is not anticipated, and no City investments to repair or upgrade the station are planned at this time. In summary, the following observations were made requiring action:

- Short-term Recommended Actions
    - Station's gravity inlet pipe does have not include any isolation valves. The City's confined space entry program should consider this and identify an isolation procedure that complies with Part 9 of the OHS Regulation in BC. Permanent valving or use of pneumatic plugs can be considered for isolating the station's wet well from the upstream collection system.
    - Verify that wet well is sealed to prevent migration of explosive gases (refer to **Appendix 1**)
    - Ensure 1 m clear space is always maintained in front electrical panels (refer to **Appendix 1**)
- Long-term Recommended Actions
    - Critical valving (pump discharge isolation and check valves) is in a confined space and difficult to access. Locate in above ground structure when station is upgraded due to condition.

2.5 LAKESIDE PARK LIFT STATION

Similar to Tyler Lake LS, the Lakeside Park LS is a small buried concrete wet-well / submersible pump that services adjacent public washrooms, café/canteen, and Lakeside Park facilities. The lift station is equipped with a 3 HP Flygt 3085.183 pump and float level sensors. No growth is anticipated within this station's catchment. It does not comply with Subdivision and Development Bylaw standards due to the following concerns:

- Only float level switches, no backup ultrasonic sensor;
- Requires manual check to ensure it is operating;
- No alarm or communication to the City's SCADA; and,
- Single pump, no redundancy.

Figure 2.5 Lakeside Park Lift Station

A gravity channel style grit chamber is located just upstream of the wet well. It is pumped out approximately once per year by City staff.

In summary, the following observations were made requiring action:

- Short-term Recommended Actions
  - Consider having a shelf spare pump for emergencies
  - Consider connecting to SCADA system to allow for remote monitoring and add high level float that is connected to a local audible or visual alarm
  - Station's gravity inlet pipe does have not include any isolation valves. The City's confined space entry program should consider this and identify an isolation procedure that complies with Part 9 of the OHS Regulation in BC. Permanent valving or use of pneumatic plugs can be considered for isolating the station's wet well from the upstream collection system.
  - Verify that wet well is sealed to prevent migration of explosive gases (refer to **Appendix 1**)
- Long-term Recommended Actions
  - Consider replacing station with a duplex packaged lift station to provide pumping redundancy
  - Pump's check valve is in a confined space and difficult to access. Move critical valving to above ground structure.

2.6 KFP (4TH STREET) LIFT STATION

KFP LS is configured with a buried brick and mortar structure wet-well with duplex submersible pumps. The station is equipped with two 5 HP Flygt 3101.180 pumps and float level sensors. The pump controls and SCADA were updated in 2014 and are in an above-ground kiosk.

Figure 2.6 KFP (4th Street) Lift Station

In summary, the following observations were made requiring action:

- Short-term Recommended Actions
    - Three conduits between wet well and electrical kiosk could allow for H_2S gas to enter the kiosk. Add junction box with seals to correct issue as described in **Appendix 1**.
    - Genset occasionally does not shutdown when utility power is restored. Investigate what is causing this issue and correct (refer to **Appendix 1**).
    - Station's gravity inlet pipe does have not include any isolation valves. The City's confined space entry program should consider this and identify an isolation procedure that complies with Part 9 of the OHS Regulation in BC. Permanent valving or use of pneumatic plugs can be considered for isolating the station's wet well from the upstream collection system.
- Long-term Recommended Actions
    - Critical valving (pump discharge isolation and check valves) is in a confined space and difficult to access

2.7 NORTH SHORE LIFT STATION

The North Shore LS is a buried FRP wet-well with duplex submersible pumps and an adjacent control and genset building. The station is equipped with two 10 HP Flygt 3127.160 pumps and float level sensors.

The lift station is in good condition. Odour control had been a previous concern with the lift station; the City poured the concrete slab around the wet-well lid as well as adding foam around the collar of the opening to better seal the hatch when it's closed.

Figure 2.7 North Shore Lift Station

In summary, the following observations were made requiring action:

- Short-term Recommended Actions
  - Genset occasionally does not shutdown when utility power is restored. Investigate what is causing this issue and correct (refer to **Appendix 1**).
  - Station's gravity inlet pipe does have not include any isolation valves. The City's confined space entry program should consider this and identify an isolation procedure that complies with Part 9 of the OHS Regulation in BC. Permanent valving or use of pneumatic plugs can be considered for isolating the station's wet well from the upstream collection system.
  - Plugs must be added to MCC door per CEC (refer to **Appendix 1**).
- Long-term Recommended Actions
  - Critical valving (pump discharge isolation and check valves) is in a confined space and difficult to access.

3.0 CONCLUSIONS AND RECOMMENDATIONS

The following table provides a summary of the recommendations and their associated priority.

Table 3.1 Lift Station Recommendations and Priority

Lift Station	Recommendations	Priority (1 high to 5 low)
General (applies to all lift stations)	Short-Term Actions: • Ensure that City has current confined space entry program for each station that complies with Part 9 of the OHS Regulation in BC • Update servicing bylaw to require all critical valving to be in above ground structures Long-Term Actions: • Eliminate confined spaces as lift stations are upgraded to increase capacity or replaced due to condition by locating critical valving and electrical controls in above ground structures.	1
Airport	Short-Term Actions: • Complete electrical upgrades (refer to **Appendix 1**) • Repair or replace pressure transmitter • Inspect and repair pump 2 as required due to pressure deficiency Long-Term Actions: • See General Items. No additional items.	1
Lakeside Drive	Short-Term Actions: • Remove non-explosion proof heat lamp from wet well • Replace the wet-well level sensor with an alternate style that is less prone to humidity issues (e.g., radar) • Replace the broken valve to allow pump #2 to operate • Replace portable cord between dry/wet pit with permanent wiring Long-Term Actions: • Move electrical controls to an above ground kiosk	1
CP Rail	Short-Term Actions: • Investigate options to address land ownership issue • Inspect and repair pump 2 as required due to flow deficiency • Adjust float mounts per operations staff feedback • Verify that wet well is sealed to prevent migration of explosive gases Long-Term Actions: • See General Items. No additional items.	2
Tyler Lake	Short-Term Actions: • Verify that wet well is sealed to prevent migration of explosive gases • Ensure 1 m clear space in front of electrical panels is always maintained	5

Lift Station	Recommendations	Priority (1 high to 5 low)
	Long-Term Actions: • See General Items. No additional items.	
Lakeside Park	Short-Term Actions: • Consider purchasing a shelf spare pump • Verify that wet well is sealed to prevent migration of explosive gases • Consider connecting to SCADA and adding a high level float that is connected to a local audible or visual alarm Long-Term Actions: • Consider replacing with duplex packaged lift station when station is replaced due to age (provide redundancy)	3
KFP (4th Street)	Short-Term Actions: • Add one junction box for three conduits running from wet well to kiosk with appropriate seals (refer to **Appendix 1**) • Investigate why genset does not turn off occasionally after utility power is restored and correct Long-Term Actions: • See General Items. No additional items.	1
North Shore	Short-Term Actions: • Investigate why genset does not turn off occasionally after utility power is restored and correct • Add knock-out plugs in MCC door (refer to **Appendix 1** for details). Long-Term Actions: • See General Items. No additional items.	5

The above findings will be incorporated into the City's sanitary master plan and be prioritized with other capacity-based recommendations for the City's lift stations. Estimated costs for the recommended works will be identified in the City's Sanitary Master Plan. Please do not hesitate to contact the undersigned if you have questions or comments regarding the above subject matter.

Sincerely,

URBAN SYSTEMS LTD.

Shiloh Johnson, EIT
Wastewater Engineer

Jeremy Clowes, P.Eng.
Wastewater Engineer / Principal

cc: Anthony Comazzetto, P.Eng.

/sj/jc
Enclosure

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Appendix 1

City of Nelson
Lift Station Condition Assessment
Electrical and Instrumentation Review
Ready Engineering

CITY OF NELSON

City of Nelson – Sanitary Master Plan – Lift Station Condition Assessment

PREPARED BY:

Dave McIntosh

Ready Engineering, A Division of Shermco Industries Canada, Inc.

1402 Columbia Avenue

Castlegar, BC V1N 3K3

PREPARED FOR:

Shiloh Johnson, EIT

Urban Systems

204 – 625 Front Street

Nelson, BC V1L 4B6

June 06, 2022

Subject: City of Nelson – Sanitary Master Plan – Lift Stations Condition Assessment

Attention Shiloh:

The following document provides an electrical review of seven lifts stations in the City of Nelson, BC.

We have attended all seven of the lift stations, some of which we have worked on the electrical systems in the distant past. Interestingly, some codes and regulations have either changed or become enforced, compared to when some of these stations were built, making them non-compliant in current times. In the report we will point out specific situations where stations are currently non-compliant or have evolved to be very difficult to operate and maintain.

Each of the seven stations has from minor to major electrical issues, some very minor and non-urgent, to significant and if not "urgent", then to be considered "pressing in nature". Combined with your civil and mechanical recommendations, the City should be able to chart a course towards remedying non-compliances and modernizing their sewerage lift stations. In general, sewerage lift stations regardless of whose design or manufacture, have been robust and reliable, as they must be to do the quiet and necessary work for a community, without failures. The seven lift stations are presently, doing their work, although several are approaching what might be considered a normal "end of life".

Our report regarding the electrical condition at the seven City of Nelson sewerage lift stations follows and includes "order of magnitude" cost estimates for the electrical remediations we have suggested / recommended.

If you have any questions or concerns, please contact me at (250) 365-8455 ext. 2101 or via email at dave.mcintosh@readyengineering.com .

Sincerely,

Dave McIntosh, AScT | Senior Technologist, Castlegar
Ready Engineering
A Division of **Shermco Industries Canada, Inc.**

CC: Rae Landry, P.Eng.

Table of Contents

1.0 Background

1.1 Site Visit – August 18, 2021

The City of Nelson, BC Has seven (7) sanitary sewer lift stations scattered along the lower elevations of the City. We visited all seven on September 22, 2021, and Ready Engineering, a Division of Shermco Industries Canada, Inc. (Ready) reviewed the electrical installations at each of the stations.

The site reviews were carried out with Urban System's, Mr. Shiloh Johnson and two City Personnel.

Below are some general comments regarding all of the stations and then site-specific comments for each of the sites; in the order that we visited the sites.

As we visited each site, we enquired regarding past failures or site history and although there had been situations, none were considered recurring or catastrophic.

Ready Engineering has been working directly with the City of Nelson, first to renew an aged and end of life genset at the Airport Lift Station. And as this lift station is considered Nelson's most important lift station, we have also begun evaluating a plan to update the electrical switchgear and genset at this station. We have kept Urban Systems informed regarding our work to ensure we are coordinated in our reporting at this sewerage lift station.

2.0 General

This report is being prepared as an evaluation of the electrical systems at the seven sewerage lift stations within the City of Nelson, BC and is to be considered as a part of the overall report prepared by Urban Systems. Most of the stations have electrical non-compliances or situations such as confined spaces that have evolved over the years to make the stations cumbersome and costly to operate. The best example is Lakeside Drive lift station where all of the electrical distribution, electrical controls and operating system are below grade in the dry well, now considered a "confined space"; very difficult to access. In the past operators could simply climb down the station ladder to enter the space, but now "confined space" procedures must be followed using more resources and time before safe entry is possible. Some of the stations have issues of explosive gas migration, that has always been a Canadian Electrical Code (CEC) rule but in the past relaxed by Technical Safety BC (TSBC – the electrical inspection department).

3.0 Airport Lift Station (Sometimes referred to as Main Lift Station)

3.1 Existing Condition

The Airport Lift Station has 3 x 75Hp dry well centrifugal pumps, 2 VFDs and one solid state reduced voltage (SSRV) starter, an end-of-life Klockner-Mueller MCC complete with a transfer switch and end-of-life natural gas genset. The electrical service to the building is 400 amps, 347 / 600vac, 3 phase from an adjacent pad mount transformer. The transfer switch had the capability to synchronize onto the utility system to "peak shave" FortisBC imported energy at one time, but that system is non-functional. The lights are slowly being changed to LED, however as they are typically off, energy savings will be small.

There is a lot of heating and ventilation equipment in this building, once the genset is changed from a natural gas, water cooled genset, the HVAC system should be revisited, and in-turn electrical connections reviewed. There is freshwater connections to the existing genset to cool the engine when it is running and in the past the building got too cool and a line froze causing water issues, so heat was added potentially more than required but another broken line couldn't be tolerated. Also, there is significant heat build up in the electrical room especially in the summer if the genset is running, so a large exhaust fan and intake louvers were added to ensure plenty of cooling and combustion air for the genset. This ventilation will not be required when the genset is moved outdoors.

We note that there is electrical drawings available for this station, although a few minor changes may not be documented.

Electrically, this station has major issues with end-of-life switchgear that has no replacement parts, and a genset that is also at end of life. And this lift station is the most critical in the Nelson sewerage system, it pumps to the Wastewater Treatment Plant (WWTP).

Both the interior and exterior light fixtures should be replaced with modern LED fixtures as ballasts or bulbs need to be replaced. Or they could be changed as a small non-urgent project. The interior lights are fluorescent and the exterior may be high pressure sodium.

3.2 Proposed Work

Some work is already in the planning stages to mitigate the electrical issues at this station.

Below is the electrical scope of work Ready is aware of:

- Phase 1 – Connect and test the new genset to the existing switchgear through a new Automatic Transfer Switch.
- Phase 2 – Remove the existing genset, after a trial period to ensure the new genset is functional.
- Phase 3 – Plan and budget for the new switchgear required to replace the end-of-life switchgear.
- Phase 4 – Implement the installation of the new switchgear and removal of the existing.
- We also understand the City intends to install 3 new variable frequency drives (VFDs) for the 3 pumps.

Phase 1 – Connect the New Genset

Ready has proposed the following items as the scope coordinated with Logan Lynn:

1. We have already reviewed the genset shop drawings and made a couple of site visits, we have issued a site layout drawing for placement of the genset and fuel tank plus fuel piping and cables.
2. We have 6 electrical drawings partially marked up indicating the revisions to be made to connect the new genset and controls to the existing equipment. We'll finish marking up these drawings to include a new automatic transfer switch (ATS) and issue for Logan's review and use during the changeover to the new genset and ATS. (Some updates / revisions are required as initially we had though we might reconnect the existing ATS. Logan pointed out the ATS is truly a weak link at this station). A purchase specification will be generated for a new Thomson Power Systems / Marathon ATS.

3. We have presumed notes on the drawings, not a formal specification for tendering for the installation work.
4. We have assumed lots of pre-wiring in preparation for the cut over to the new genset from the existing genset. As the intent is, in the near future, to upgrade the switchgear the cable routing will have extra length to be able to relocate the cables later, so the "appearance" of the installation doesn't need to be prefect.
5. Coordinate to have the Cummins genset commissioning team on site at the correct time. Ideally, they should be on site to pre-run the genset and confirm it is set to be the standby power source. Also, ideally, they are on site for the day of the final cut over and testing with actual station loads.
6. We are assuming most of the coordination of crews (both electrical & operations), preferred outage day, time or season will be by Nelson. We want to have the least exposure for a Nelson Hydro power failure as possible.

Phase 2 – Remove the Existing Genset

Ready plans to provide what we might call a "work package" or "task checklist" for the removal of the existing natural gas genset. Actual, physical, removal of the genset could take place after a couple of automatic operations of the new genset, either from real power outages or simulated situations. Ready plans to complete the following tasks for this phase:

1. Confirm the new standby genset is functioning properly, by discussing its operation with the City.
2. Complete a high-level task list or checklist of equipment to remove. The value of having Ready involved is twofold, it helps us ensure we don't remove anything that might be reused and if we identify patching to accommodate new equipment, we can get that done during this early stage. Generally, the list could include:
    a. Disconnect the natural gas supply to the building and have the meter removed by FortisBC Gas.
    b. Completely remove the natural gas piping from the building.
    c. Safety shut off and cap off the cooling water supply to the genset.
    d. Remove exhaust piping.
    e. Remove the obsolete power & control wiring to the genset.
    f. Potentially adjust the wet well vent pipe just outside the doorway to accommodate genset removal.
    g. Remove genset and send for salvage or as directed by the City.
    h. Consider ventilation settings with genset removed.
3. Attend site to take stock of the empty space and how best to utilize it for the next phases.

Phase 3 – Design and Budgeting for New Switchgear

Phase 3 can begin immediately and, some of the high-level planning has already begun with input from Logan and being advised 3 new VFDs are being considered for the station. During this phase Ready will complete a design for new switchgear and other electrical components as necessary. Once a design is formulated a capital cost budget will be provided for the electrical portion of the work to the City. Then we will complete the detailed design, so it is "shelf ready" when the City has funds to proceed.

Ready will complete the following items as the scope of electrical design and budgeting work:

1. Select replacement equipment and placement of such equipment within the room in a manner to accommodate a practical transition to the new equipment.
2. Presently, we are estimating 6 or 7 drawings may be required. Some may be demolition drawings or modifications to existing drawings indicating connections to the new equipment which may be slightly different than the existing equipment.
3. Attend site twice, to confirm design details. We will endeavour to coordinate these visits with other work ongoing in Nelson; with the renovation style of project for a critical operation we are certain, we'll need to confirm details as the plan is developed.
4. Provide a budget for the electrical portion of the work.
5. We'll want to confirm how this project will be executed, either by City crews or a Contractor or combination; regardless, we assume Ready will provide the electrical portion of a specification or detailed bill of materials and a work package.

Phase 4 - Installation of New Switchgear

This will be the construction services phase of work, where we would provide the following while a Contractor and the City install equipment:

1. Assist with tendering as necessary.
2. Review the Contractor's schedule.
3. Review shop drawings provided by the Contractor.
4. Monitor the Contractor's progress during construction and finally see the finished station.
5. Complete record drawings and final sign off of the project with authorities.

3.3 Recommendations

There is an electrical work program underway at this station as outlined above. Due to the critical nature of this station completing the work has a fairly high priority, but each step must be well planned.

A very high-level cost estimate of the electrical work would be:

- Final placement of genset, and reconnection of power to existing MCC - $60,000
- Removal of the existing genset - $15,000
- Electrical design of the work to renew the switchgear - $45,000
- A complex swap over to the new MCC and removal of existing - $120,000
- Change all of the light fixtures to LED as a non-urgent project - $2,000

Note that these are very high-level estimates at this time until further detailed design is completed. And the schedule, is dependent on successful completion of each step and a trial period to ensure each step is working reliably.

4.0 Lakeside Drive Lift Station

4.1 Existing Condition

We did not enter this station during our tour due to confined space regulations. And we didn't have any drawings to review. We understand generally the station functions satisfactorily. It does seem logical to modernize the station.

As we didn't enter this station and we haven't seen any drawings; we assume the service is likely 200 amp, 120 / 208vac, 3 phase (to be confirmed), all underground to power the 2 x 15Hp dry well centrifugal pumps. We were advised there is no SCADA at this station and there is a high-level alarm transmitted to the Works Yard via an old style phone line hardwired connection, that will be obsolete soon.

4.2 Proposed Work

The City should provide an above grade kiosk complete with all controls and communications to the works yard. There is plenty of available space at the site and this would permit most electrical maintenance and monitoring to be done from grade without any special gear. This work is non-urgent as the station is presently functioning well. There are a few non-compliances of the Canadian Electrical Code, such as some electrical equipment is not sealed off from the wet well and potentially explosive atmospheres. There is a non-rated heat lamp at the top of the wet well which is not permitted. There is a portable cord that seems somewhat permanent into the dry wet which should be replaced with permanent wiring methods.

4.3 Recommendations

As noted above the CEC violations should be cleaned up which could be inexpensive maybe $2,000 unless a rated blower heater were installed to heat the wet well which could then be $10,000.

Then a kiosk mounted at grade with SCADA and the necessary interconnections to existing equipment and removal of the existing controls could cost $85,000 plus design work at $30,000.

Non-Compliant heat lamp.

5.0 CPR Lift Station

5.1 Existing Condition

We did not enter this station during our tour due to confined space regulations. We understand generally the station functions satisfactorily. It does seem logical to modernize the station at a convenient time by migrating the controls to grade located at the genset skid. We do note that the City has had minor access issues as the property is owned by Maglios.

Again, we have no electrical drawings in hand for this station.

The electrical service seems to be 200 amp. 240vac, 3 phase, delta (an older style of voltage configuration) to a pole by the back of the adjacent food warehouse, then run underground to the station. The service probably needed to stay at 240vac as it also feeds the food warehouse which must have older style 240vac, 3 phase equipment. There is a step-down transformer on the genset skid to convert the voltage from 240vac to 120 / 208vac, 3 phase.

There are 2 x 25Hp pumps at this station.

The skid mounted genset and automatic transfer switch (ATS) are relatively new and in good condition. The genset seems to be rated 115KVA continuous with a 250-amp circuit breaker, this is plenty adequate for the standby power supply at this station.

We noticed that the float switches in the wet well are not handy to reach, however the operators must make do, but they may wish to consider adapting the mounting arrangement just to make it easier to test the floats if ever necessary. The installation is electrically compliant though.

5.2 Proposed Work

The City should non-urgently consider migrating the controls to an above grade control panel.

Ready couldn't confirm if the wet well was sealed to prevent the migration of gases into electrical panels, ideally if drawings were available this could be checked. And as there is no drawings for the ATS and genset connection to the station, an update of any existing drawings would be helpful.

5.3 Recommendations

Other than checking the sealing to prevent the migration of wet well gases and remedying if necessary and updating the drawings, this station seems to function well.

6.0 Tyler Lake Lift Station (Soccer Field Washroom)

6.1 Existing Condition

The lift station is located about 20 meters from the washroom facility that the station services. The 2 x 3Hp submersible pumps and level control in the wet well are feed underground from the utility room in the washroom building. The electrical subservice is 60 amps, 120 / 240vac, single phase, which is fed from a larger building service fed underground from a power pole across the parking lot. We took photos of the paper copies of drawings in the building and we have them for reference.

Level control is by a Milltronics Multiranger ultrasonic level transmitter. There is a high level float that sounds a local alarm horn.

This is a very low flow station that reportedly functions well.

We could not see any sealing to mitigate the flow of explosive gases from the wet well into the electrical room.

6.2 Proposed Work

During maintenance, an electrician could check if there is sealing in the ducts from the wet well to the building.

6.3 Recommendations

Add sealing, if necessary, in the ducts to the wet well to become CEC compliant. There is supposed to be 1 meter of clear footing in front of all electrical panels at all times, some housekeeping could be done in the electrical room.

7.0 Lakeside Park Lift Station (At the Greenhouses)

7.1 Existing Condition

The lift station's wet well is approximately 10 meters away from the control panel mounted on an exterior wall of the adjacent greenhouse. The single submersible pump is a 2.2KW unit controlled by float switches. The power to the control panel is a 30 amp, 120 / 208vac, 3 phase sub-feed from the main greenhouse power. The pump runs at around 9 to 10 amps in normal conditions. The overload relay (O/L) is set at 12 amps, and apparently occasionally trips, shutting down the station. The electricians have been monitoring this issue to resolve the problem.

Again, we do not have any drawings of this station for reference.

7.2 Proposed Work

Again, at this station, we could not see any seals, such as EYS condulets or rated Teck cable connectors, to prevent explosive gases from the wet well entering the control panel.

7.3 Recommendation

This low usage station doesn't require any electrical capital upgrades to become compliant, other than confirming if seals are installed and if not, they can be added. As the float and pump cables transition from dedicated cables in the wet well to Teck cables arriving at the Control Panel, it seems very unlikely gases could migrate into the control panel.

Possibly, a high level float could be added to the controls to initiate a local alarm such as a horn or strobe light.

However, if as suggested in the body of the report, a new duplex station is installed, then new controls and power supply would need to be designed and installed. An opinion of cost for a new duplex kiosk installed would be $125,000.

8.0 KFP Lift Station (4th Street - North of the Orange Bridge)

8.1 Existing Condition

There are 2 x 5Hp submersible pumps, rated for 230 / 460 vac, 3 phase operation in the wet well which has an above ground control panel mounted on the top of the wet well and a 31 KW, 240vac, 3 phase standby diesel genset adjacent this equipment. There is a note in the control panel that a pump runs at 13.4 amps, which is not fully loaded for a 5Hp pump. The electrical service comes from a power pole up on the road with the main circuit breaker or fused switch in a locked cabinet on that pole. We do not have drawings of this station; however, we assume the service is 100 amp, 120 / 208vac, 3 phase, underground down to the station. (It could be 240vac, 3 phase – could confirm by measurement). The existing genset is in good condition, although it was reported that on occasion the genset does not shut off after a return to normal utility power. The ATS is a Marathon / Thomson Technology switch rated 100 amps, 240vac and dated 2012.

We noticed the physical mounting of the level sensors may be difficult to reach.

We noticed there were no seals, such as EYS condulets, in the 3 conduits between the wet well and control panel direct above. This is a violation of the CEC, and apparently a Technical Safety BC electrical inspector had also noticed this situation and asked for it to be remedied.

8.2 Proposed Work

The 3 conduits between the wet well and the control panel should be replaced with short runs of conduit into the wet well, a junction box (600 x 200 x 200mm, approximately) complete with terminals for all control and pump wires and then conduits with EYS seals from the junction box (JB) into the control panel. (The sealed run into the control panel could be Teck cables with rated connectors). This allows the pumps or controls to be disconnected at the JB and pulled out of the wet well for maintenance or replacement; and any potentially explosive gases cannot migrate up into the control panel where electrical arcing and sparking devices such as motor starters or relays could ignite the gases.

Trouble shoot and remedy the issue of the genset not automatically shutting down.

Consider a better mounting for the level sensors for ease of maintenance.

Unsealed conduits from the wet well below into the control panel.

8.3 Recommendation

The genset seems to continue running at times, either the Nelson electrical crews need to do a thorough check of this situation or the manufacturer's representative should be brought in to review the installation. As a temporary measure to assist in identifying and troubleshooting when this condition happens an alarm point could be added to the SCADA system where if Nelson Hydro power is on and the genset is running after a 2 minute or so time delay, an alarm is sent. (If this temporary alarm is added, be sure to confirm the time delay to alarm is longer than the cool-down run time of the genset).

Install a JB and seals to prevent the migration of explosive gases into the control panel. This could be completed by City crews or a contractor. An estimate to complete this work would be approximately $6,000.

9.0 North Shore Lift Station

9.1 Existing Condition

The station currently has 2 x 10Hp submersible pumps installed in the wet well located adjacent to the control building that houses the electrical equipment and a diesel genset. The drawings, which we took photos of, indicate that the station may ultimately be upgraded to 2 x 47Hp pumps. The electrical service is 150 amp, 347 / 600vac, 3 phase fed underground from an adjacent power pole.

The 31.5 KVA genset appears to be in good condition, as does all of the equipment in the building. The operator mentioned this genset has on occasion not shutdown and continues to run after the power supply returns to normal.

The MCC was manufactured by Furnas and there has been some changes to the controls, as a couple of starter doors have holes where push buttons or switches were removed.

The lights in the room are older style fluorescent and look aged, as does the exterior light at the door.

9.2 Proposed Work

Trouble shoot and remedy the issue of the genset not automatically shutting down.

Install knockout plugs in the holes in the MCC door, to become CEC compliant.

Change the room lighting to LED fixtures when the existing fixtures fail or need relamping. Typically, LED fixtures are used for energy savings and their long life / reduced maintenance; at this location, the interior lights are rarely on so neither of these scenarios warrant urgent action.

Change the exterior light at the door to a modern LED fixture. The existing fixture's lens is discoloured and likely high pressure sodium and probably controlled with a photocontroller. If a different means of controlling the exterior light is desired, we recommend adding an astronomic timer that can be used as dusk to dawn or programmed for various on – off cycles.

9.3 Recommendation

The 31.5KVA genset occasionally has issues where it doesn't shutdown when utility power is returned to the station, either the Nelson electrical crews need to do a thorough check of this situation or the manufacturer's representative should be brought in to review the installation. Similar to our suggestion for the KFP lift station; as a temporary measure to assist in identifying and troubleshooting when this condition happens an alarm point could be added to the SCADA system where if Nelson Hydro power is on and the genset is running after a 2 minute or so time delay, an alarm is sent. (If this temporary alarm is added, be sure to confirm the time delay to alarm is longer than the cool-down run time of the genset).

A new exterior light installed complete with a new astronomic switch may cost $1,000.

10.0 Overall Comments (Electrical)

Sewage lift stations are installed to do, as their name states, gather sewage at low points and lift or pump it to a higher point to again let gravity do its job and run to a lower elevation or possibly into a treatment facility. Typically, lift stations are classified as hazardous areas where explosive gas atmospheres can exist. The explosive gas atmospheres can appear from methane developing from the sewage or other fluids such as gasoline or certain cleaning products may be poured into the sewerage system and make their way to a lift station.

The Canadian Electrical Code (CEC) has specific rules for electrical installations in or about hazardous areas. Some of the specific rules for sewage lift stations are defined in the CEC Sections 18 and 22. The general concept is to eliminate the possibility of an arc or spark anywhere that explosive gases may develop which is generally considered in the wet well. And secondly, to prevent the explosive gases from migrating to other areas, such through electrical conduits into control panels where arcing and sparking devices could be located.

The CEC Section 22 rules deal with the harsh environment typically found in wet wells, where there is excess moisture and sometimes a corrosive atmosphere.

Basically, wiring and equipment installed in wet wells is specially rated for use in these atmospheres and standard equipment is not to be used in wet wells.

The 7 lift stations we visited were likely all designed to be compliant with the CEC rules, which have not significantly changed for many years. We did however spot a few non-compliances such as a non-rated heat lamp in a wet well and some unsealed conduits.

Four of the stations have standby gensets to provide power when Nelson Hydro has an outage. Generally, all of these gensets are in good condition with no reported serious issues. The exception is the Airport lift station, however a new diesel genset is in place there waiting to be connected. And oddly, two of the stations with gensets seem to have a issue where the genset does not automatically shutdown after being called to run. (Possibly the automatic transfer switch is providing a false signal to the genset, or maybe the cool-down run timer at the genset has malfunctioned or the genset is getting a false run signal from another source).

The Airport Lift Station is the most critical station for Nelson as it pumps to the wastewater treatment plant, and it is in the worst condition electrically. Nelson is aware of the situation and has been working with Ready Engineering to resolve the issues with a phased approach to renewing all of the equipment in the station. The work plan is outlined above.

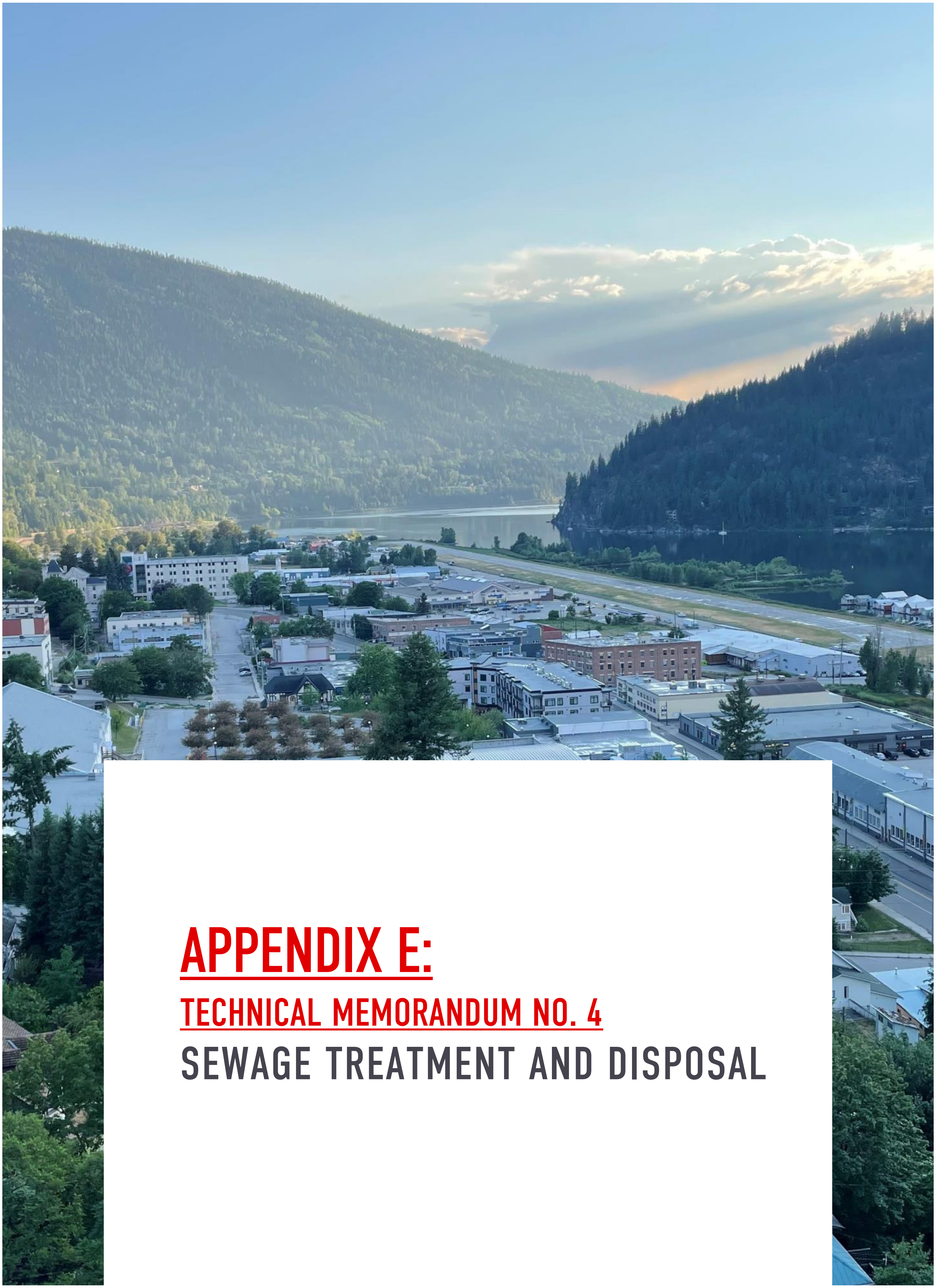
11.0 Conclusion and Final Recommendations (Electrical)

The only lift station that requires urgent attention to the electrical equipment is Airport Lift Station and that work has begun.

Then there is a couple of stations that warrant some non-urgent attention to be become CEC compliant. The issues are mainly concerning the explosive gas atmospheres that could develop in the wet wells. Nelson should consider the sealing to prevent migration of explosive gases as very high importance. They may want to place “Duct Seal” putty in unsealed conduits as a temporary measure to try keep explosive gases out of control panels where arcing or sparking devices are installed until proper seals can be installed.

Finally, maintenance should be done to automatically shutdown the gensets that are malfunctioning. Light fixtures can be updated at the City’s leisure.

End of report.



APPENDIX E:

TECHNICAL MEMORANDUM NO. 4

SEWAGE TREATMENT AND DISPOSAL

URBAN SYSTEMS

MEMORANDUM

DATE: May 31, 2022
TO: Colin Innes
CC: Scott Eagleson, Rob Nystrom
FROM: Matt Smith, Shiloh Johnson
FILE: 0795.0119.01
SUBJECT: Technical Memo No.04 – Sewage Treatment and Disposal, Rev.3

1.0 INTRODUCTION

The City of Nelson (City) is updating its Wastewater Strategic Plan: the plan update includes a review of treatment and disposal of sanitary sewage. This memorandum provides a critical comparison between upgrading the existing pollution control centre (PCC) and replacing it with a new sewage treatment plant at an alternate location near the public works yard.

2.0 BACKGROUND

The City of Nelson operates the Grohman Narrows Pollution Control Centre (PCC) and discharges treated effluent to the Grohman Narrows approximately 5 km west of the City of Nelson. The PCC was upgraded in 2005 to provide secondary treatment with the addition of rotating biological contactors (RBCs), secondary clarifiers and replacement of the chlorination system with an ultra-violet (UV) light disinfection system. Before 2005 the PCC was limited to primary treatment only. The 2005 upgrade included the first phase of RBCs which were anticipated to reach capacity approximately 10 years after installation. A second phase with the installation of a third RBC train was envisioned subsequently but has not been constructed.

The PCC is currently operating at, or beyond, capacity with respect to the WSER requirements and both the design and permitted flow. In 2018 the City undertook an assessment study that identified major upgrades required for both continued operation and to accommodate growth in the community (Urban Systems, 2018). Nelson is a thriving community and additional growth is planned and expected.

3.0 REGULATORY REQUIREMENTS

The City of Nelson wastewater treatment plant operates under the BC Ministry of Environment Permit Pollution Control Permit PE-291. In addition, as the releases are > 100 m^3/d and to a fisheries stream, the Federal Wastewater Systems Effluent Regulations (WSER) also apply. This regulation is administered by Environment Canada.

3.1 PROVINCIAL REQUIREMENTS

The PCC operates under BC Ministry of Environment and Climate Change Strategy (ENV) permit PE-291. This permit was last updated in March 2006 in order to recognise the upgrades to secondary treatment and replacement of the chlorination system with UV disinfection. The conditions of the permit allow the following discharge criteria:

- Flow 5,680 m^3/d, at a maximum rate of 8.5 m^3/minute;
- Five-day biochemical oxygen demand (BOD_5) ≤ 140 mg/L which is defined as total BOD_5;
- Average total suspended solids (TSS) ≤ 100 mg/L; and,
- Coliform bacteria ≤ 150,000 MPN/100 mL, which is defined as total coliforms.

A copy of the permit can be found in **Appendix 1**.

The intent was to register the upgraded facility under the Municipal Wastewater Regulation (MWR). Through direction from ENV, an environmental impact study (EIS) was prepared and included an assessment of the outfall conditions and effluent dispersion/dilution in order to address the requirements of the MWR. The information presented in the EIS indicated that the estimated dilution ratio at the edge of the initial dilution zone (IDZ) is in the order of 270:1. The effluent criteria recommended in the EIS are summarised in Table 1.1, and intend not just to recognise Provincial legislation, but also to recognise the requirements of the Federal wastewater regulation.

Table 3.1 City of Nelson Recommended Effluent Criteria

Parameter	Criteria
Five-day carbonaceous biochemical oxygen demand ($CBOD_5$)	An average equal to or less than 25 mg/L, with a maximum of 45 mg/L.
Total suspended solids (TSS)	An average equal to or less than 25 mg/L, with a maximum of 45 mg/L.
Ammonia	Treatment not required, based on the ability to meet either acute concentrations before discharge or chronic concentrations at the edge of the IDZ.
Phosphorus	Treatment not required
Disinfection	Faecal coliform concentration ≤ 200 MPN/100 mL at the edge of the IDZ. This translates to an effluent concentration of 54,000 counts/100 mL for the dry weather design flow and 42,000 counts/100 mL for the wet weather design flow.
Total chlorine residual	≤ 0.02 mg/L (if chlorine is the chosen method of disinfection)

Registration under the MWR is a multi-year process, with expected timelines for each phase outlined below.

- Preliminary application: approximately 6 months.
- Preparation of submission materials by proponent: up to 3 years.
- Final application review and approval by ENV: between 12 and 18 months depending on complexity.

In addition, it is possible that the BC Environmental Assessment Act (BCEAA) may be triggered by major upgrades to the system. The triggers under the BCEAA are:

- A new facility that is designed to serve ≥ 10,000 people.
- An existing facility that is designed to serve ≥ 10,000 people and will result in an increase of ≥ 30% of the total waste discharge.

A system upgrade could trigger the BCEAA. Undertaking a BCEAA is a multi-year process and will require community and First Nations engagement. However, should an approved Liquid Waste Management Plan be in place, then a facility is exempt from the BCEA process. The LWMP process will be discussed further below.

3.2 FEDERAL REQUIREMENTS

The WSER indicates the following effluent requirements. There are no requirements for maximum flow, with the flow determining the monitoring and reporting frequency. Flows above 2,500 m^3/d also trigger the requirement for undertaking toxicity testing (LC50 96-hour rainbow trout bioassay). It is reasonable to expect that the toxicity testing requirement will be triggered should the two plants be consolidated.

- $CBOD_5$: 25 mg/L average.
- TSS: 25 mg/L average.
- Un-ionised ammonia: 1.25 mg/L maximum.
- Total chlorine residual: 0.02 mg/L maximum.

Unlike the MWR, there are no requirements for an authorisation approval from Environment Canada.

4.0 DESIGN CRITERIA

Technical memo 1 identified the design population and flows for the strategic planning horizons. Statistics Canada recently published the 2021 census of population, which was recorded at 11,106 people.

In addition, the City provided a buildout population of 24,476 persons, which would be reached in 67 years, or 2088, at an estimated growth rate of 1.2%.

Two design scenarios were considered for sizing the alternate site: existing (2021) and future design horizon (governed by population growth to 15,000). The following tables summarizes the flow and loading assumptions, respectively:

Table 4.1 Design Flows

Parameter	Units	Existing	Design Horizon
Population		11,106	15,000
Average Day Flow (ADF)	m^3/d	4,900	7,200
Maximum Month Peaking Factor		1.7	
Maximum Month Flow (MMF)	m^3/d	5,600	12,250
Maximum Day Peaking Factor		2.3	
Maximum Day Flow (MDF)	m^3/d	8,900	16,550
Peak Hour Peaking Factor		3.8	
Peak Hour Flow (PHF)	L/s	216	406

5.0 SEWAGE TREATMENT AND DISPOSAL

5.1 DISPOSAL/REUSE

Once sanitary sewage is treated there are two streams that must be managed:

1. Treated effluent
2. Biosolids

Management of biosolids will be addressed under separate cover. Treated effluent must be returned to the environment or may be reclaimed and reused for purposes such as irrigation, habitat enhancement, dust control or industrial uses. Given Nelson is situated next to Kootenay lake, in a location with significant flow, it is assumed that Kootenay Lake will continue to be the primary option for disposal of treated effluent. Additional options for management of treated effluent are:

1. Disposal to ground.
2. Effluent reuse.

Disposal to ground at this scale would require rapid infiltration basins (RIB) with an overall area of approximately 35,000 m^2 (assuming 75 m/y infiltrative capacity), or approximately 190 x 190 m, separated into at least 4 RIBs. The site of RIBs must be underlain with free draining, granular soils, and several additional requirements with respect to groundwater depth, movement, and travel time. It is our understanding that no suitable site is currently available near either the Grohman Narrows PCC, or the potential alternative site. Consequently, for the purposes of this study it is assumed that disposal to Kootenay Lake will remain the primary option.

Reuse of treated effluent requires that a suitable user of the reclaimed water is located within practical distance to the source of the water. Typical uses include irrigation, dust control, equipment washing and stream augmentation, with different uses requiring different levels of treatment and disinfection. Where effluent reuse is to be implemented the MWR still requires another disposal option to be permitted and installed. Because there are no suitable end users near the Grohman Narrows site, and the road is not suitable for bulk water hauling, that site is not considered suitable for effluent reuse. Reuse opportunities may be easier to find at the alternate site. The alternate site is closer to the City where there may be greater opportunity for irrigation, light industrial use, bulk hauling, or equipment washdown. Nevertheless, because an alternate disposal method is still required by regulation, for the purposes of this study it is assumed that management of treated effluent will be disposal to Kootenay Lake in the first instance, with effluent reuse opportunities explored as they arise.

5.2 GROHMAN NARROWS PCC – CURRENT CONDITION

Prior to the 2005/2006 upgrades, the City of Nelson wastewater treatment plant was a primary level facility, consisting of the primary settlement of domestic wastewater and chlorination before discharge to the Grohman Narrows, a fast-flowing section of the west arm of Kootenay Lake. The plant was upgraded to a secondary standard in 2005/2006, with the addition of the following items:

- The conversion of existing infrastructure to an aerated equalisation tank;
- The addition of four rotating biological contactors, in two separate treatment trains;
- Secondary clarifiers;
- Disinfection through the addition of two banks of ultra-violet (UV) lights, which replaced the chlorination system; and,
- An outfall to the thalweg of Grohman Narrows.

Additional upgrades for the dewatering of sludge were completed in 2011 and consisted of replacing the belt press with a centrifuge. The City's sludge is treated by mesophilic anaerobic digestion to produce a Class B biosolids before it is dewatered and transported offsite for disposal.

Since the upgrades, there have been several changes in the regulatory framework for domestic wastewater treatment. In 2012, the BC Municipal Sewage Regulation (MSR) was repealed and replaced with the BC Municipal Wastewater Regulation (MWR). Also in 2012, the Federal Wastewater Systems Effluent Regulations (WSER) was introduced into law.

The primary concern with the PCC is its ability to consistently meet the requirements of the Federal WSER for 5-day carbonaceous biochemical oxygen demand ($CBOD_5$). The regulation requires that quarterly average $CBOD_5$ effluent concentration be 25 mg/L or less. In 2017, this average was only met in the second quarter. However, the ability to meet the WSER effluent TSS requirement of 25 mg/L as a quarterly average was also raised as a concern. In 2017, this average was only met in the second and third quarters. In both cases, the average concentration was calculated to be 25 mg/L, which is on the threshold of non-compliance.

The City of Nelson wastewater treatment plant operates under the BC Ministry of Environment Permit Pollution Control Permit PE-291. The wastewater treatment plant is a secondary treatment facility and consists of the following processes:

- A headworks facility with a mechanical screen, manual bypass channel, aerated grit tank, and grit classifier.
- Two parallel primary clarifiers with scum removal.
- Two parallel aerated equalization tanks with two low-lift pumps (one pump per tank).
- Four rotating biological contactors (RBC), two trains of two RBCs in series.
- Two parallel secondary clarifiers with inclined plate settlers for enhanced sedimentation.
- An ultraviolet (UV) disinfection system.
- Two high-rate anaerobic digesters in series.
- A centrifuge.
- An emergency backup generator.

In 2018, Urban completed a Pollution Control Centre Upgrade Assessment (Urban Systems, 2018) of the secondary treatment components of the City's PCC. This assessment (attached in **Appendix 2**) reviewed the capacity of each individual process within the plant as well as provide recommendations for priority upgrades and estimates of their associated capital and O&M costs.

A review of the PCC determined that a number of the major treatment processes are currently at, or have exceeded, their rated capacity, which would lead to poor treatment performance and very challenging operation and maintenance conditions. The major processes that require capacity upgrades include: primary clarifiers, equalization tanks, secondary treatment (RBCs) and secondary clarifiers. Other supporting systems were also determined to be at capacity, including the electrical service, emergency backup generator and headworks screen. The UV system was also found to not meet the Municipal Wastewater Regulation (MWR) reliability requirements and should be upgraded.

The scope of the 2018 assessment was restricted to the secondary treatment components of the PCC; it must be noted that the remaining components of the PCC have been in place for many years and will be in need of renewal/replacement in the coming years.

5.3 UPDATED PCC ASSESSMENT SUMMARY

The PCC is biologically overloaded with several unit processes nearing or exceeding their hydraulic capacities. Influent wastewater strength and $CBOD_5$ loading is higher than what was projected at the 2006 upgrade. Due to the biological overload, there have been instances of effluent water quality exceeding the MWR discharge criteria of 45 mg/L $CBOD_5$. Lastly, the Federal Wastewater System Effluent Regulation brought into effect in 2012 set more stringent effluent quality criteria for discharges to surface water at 25 mg/L $CBOD_5$ and TSS, as quarterly averages.

High strength wastewater can often be attributed to industrial wastewater contributors, such as:

- Breweries;
- Coffee roasteries;
- Meat and animal processing facilities;
- Bakeries; and,
- Food product manufacturing facilities.

The City is working with Kootenay-Columbia Environmental Innovations Co. to provide *in situ* monitoring for select businesses to characterize wastewater loadings generated from high strength contributors.

The 2018 assessment recommended the following sequence of phased upgrades:

- Phase 1 – Detailed Design & Construction of Electrical and Emergency Generator Upgrades
- Phase 2 – Detailed Design & Construction of a New Headworks
- Phase 3 – Detailed Design & Construction of New Primary Treatment Process (Mechanical Primary Screens)
- Phase 4 – Detailed Design & Construction of a New Secondary Treatment Process (MBBR) and Secondary Clarification Process (DAF)
- Phase 5 – UV Upgrades

It was also recommended that the City complete a sludge management study to remediate and upgrade the anaerobic digestion. A possible outcome of this study would be upgrading the facility's sludge management and solids dewatering system and removing digestion. A receiving bay at the plant could be constructed for the dewatered sludge to be trucked to a composting facility. The RDCK has a stated long-term goal of diverting organic waste from landfills through the development of two proposed regional-scale composting facilities, one located at the Creston Landfill and one at the closed Central Landfill site near Salmo. The Salmo location is the closest proximity to the City of Nelson.

In spring 2021, CWMM completed a structural condition assessment (see **Appendix 3**) of the existing PCC. No major upgrades were flagged for immediate remediation. However, given that portions of the facility are 50 years old and exceed the design life of the building, consideration should be made in decisions regarding expansions / upgrades of the existing facility versus a new facility.

6.0 ALTERNATE SEWAGE TREATMENT PLANT OPTION

Through discussions with the City, 70 Lakeside Drive was identified and assessed utilizing a set of multiple bottom-line parameters. There are additional sites that may be considered in future stages, but the purpose of this exercise was to determine if an alternative site is potentially feasible in the long-term. The alternate site that has been identified as a potential location for a treatment facility is shown in **Figure 6.1**.

City of Nelson

Wastewater Master Plan Update

Alternate Site for the Treatment Facility

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

0 50 100 150
Metres

Coordinate System: NAD 1983 UTM Zone 11N

Scale: 1:4,000 (When plotted at 11"x17")

Data Sources:

- Imagery: Esri, Maxar.
- Cadastral: GeoBC.
- Railway: NRCan.

Project #:	0795.0119.01
Author:	BA
Checked:	SJ
Status:	
Revision:	A
Date:	2022 / 5 / 5

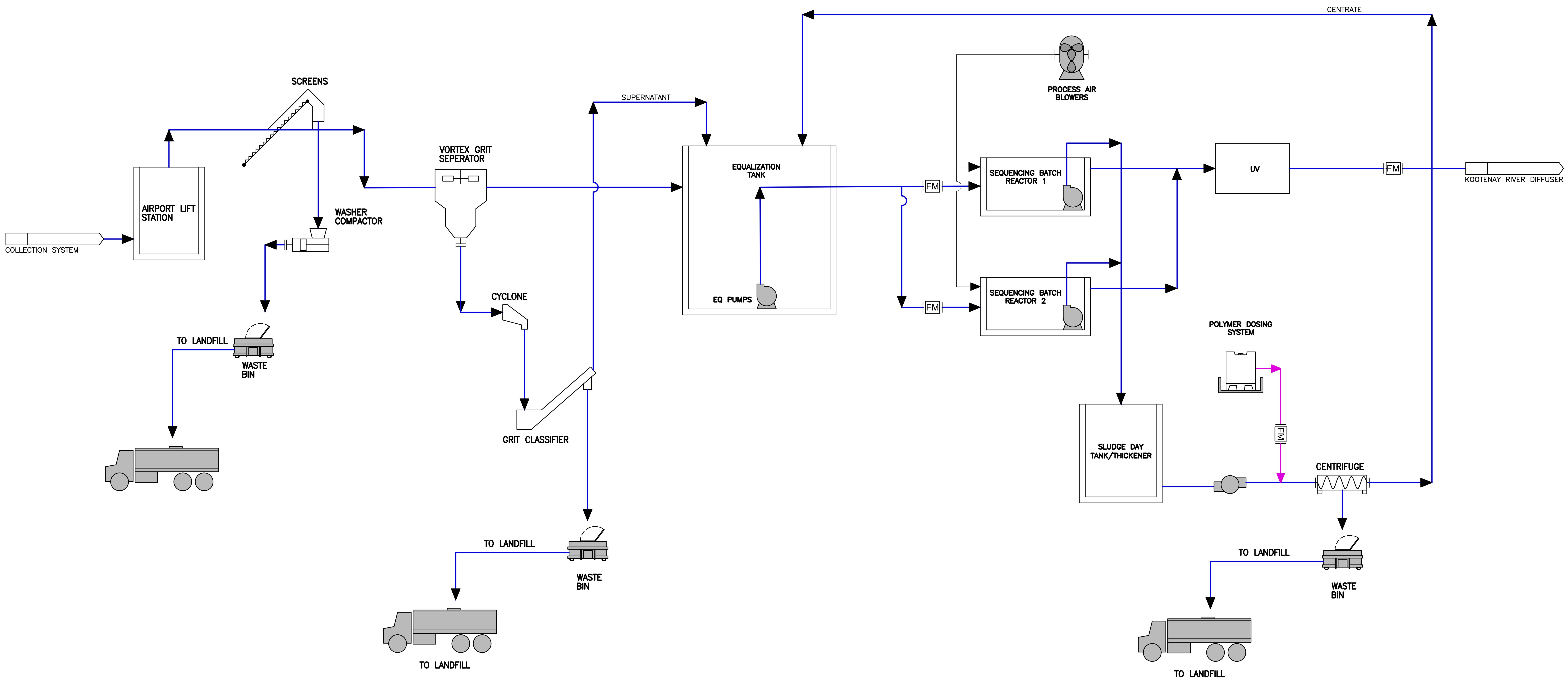
URBAN SYSTEMS

FIGURE 6.1

Given the age, condition, and proposed upgrades required for the existing PCC, it is imperative that alternate solutions be critically explored to help realize a path which the City can direct their resources. As such, a new sewage treatment plant on an alternate site was considered. For the purposes of this study, a common secondary biological treatment process utilizing Sequencing Batch Reactors (SBRs) was assumed for determining the cost and footprint feasibility of the alternate site. The following **Figure 6.2** is the process flow diagram of the alternate sewage treatment plant which was also the basis of the feasibility study. The system used for comparison includes the following unit processes:

- Influent forcemain from existing Airport Lift Station
- Headworks including screening and grit removal
- Equalisation tank
- Sequencing batch reactors
- UV disinfection
- Sludge dewatering

A complete feasibility study would be required to select the actual preferred treatment systems for a new facility, but the system described here would work well and is useful as a representative system to understand the footprint and comparative cost of a new treatment facility.



City of Nelson

Alternate Sewage Treatment Plant

Process Flow Diagram

Issued For Review
May 05, 2022
urbansystems.ca

The accuracy and completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate and establish the precise location of all existing information whether shown or not.

Coordinate System: N/A

Scale: Not to Scale

Data Sources: N/A

Project #: 0795.0119.01
Author: M.Smith
Checked: S.Johnson
Status: N/A
Revision: A
Date: 2022-05-05

URBAN SYSTEMS

Figure 6.2

U:\Projects_NEL\0795\0119\01\D-Design\CAD\10_DES\PFD-0795011901_recover.dwg, GIS FIG 22x34 LANDSCAPE, 2022-05-05 12:37 pm musmith

7.0 CAPITAL COST AND LIFE-CYCLE ANALYSIS

Cost is often the deciding factor when comparing options. Order-of-magnitude cost estimates were developed for both scenarios to provide further insight into deciding which option is better suited for the City.

7.1 PCC PROCESS UPGRADES CAPITAL COSTS

The 2018 PCC assessment report included a Class D cost estimated for the proposed upgrades. These have been adjusted to 2021 CAD and include the additional components discussed in Section 5.3; see **Table 7.1**.

Table 7.1 PCC Upgrades Opinion of Probable Capital Cost

Phase	Process Upgrade	Capital Cost
Phase 1	Emergency Generator	$ 290,000
	Electrical Upgrades	$ 360,000
Phase 2	Headworks	$ 3,900,000
Phase 3	Primary Filtration	$ 5,600,000
Phase 4	MBBR	$ 6,100,000
	DAF	$ 6,300,000
Phase 5	UV Upgrade	$ 540,000
Digester Upgrades	Decommission Digesters, Upgrade Solids Dewatering Equipment, and Install Solids Pick-Up Bay	$ 7,000,000
Forcemain Upgrades	Replace Forcemain with New Submarine HDPE Forcemain	$ 8,430,000
	TOTAL	**$ 38,520,000**

7.2 ALTERNATE SITE CAPITAL COSTS

The cost of a project frequently governs its feasibility. At this stage of considering the capital costs for design and construction of a new sewage treatment facility on an alternate site, only an order-of-magnitude cost estimate can be developed. As such, it is important to use a range of methods to determine what the order-of-magnitude cost estimate is.

7.2.1 Burnside Study Review

An order-of-magnitude cost estimate was developed using the Burnside study, "Water and Wastewater Asset Cost Study" (R.J. Burnside & Associates Limited, 2005). This study was completed to generate cost replacement curves for water and wastewater infrastructure and provides equations that can be used to estimate the cost of infrastructure projects based on capacity.

Burnside provides a cost equation, in 2004 dollars, for a complete tertiary treatment plant based on capacity in m^3/d. For the City of Nelson, the capacity is the MDF of 8,300 m^3/d. The basic cost equation is:

$$y = 1445.3x + 4x10^6$$

where: y = cost ($) and x = capacity in m^3/d

A combined general multiplier of 1.33 is applied to account for items such as controls, programming, and engineering.

For the Nelson WWTP this yields a 2004 cost of:

$$y = [1445.3(8300) + 4x10^6]\ 1.33 = \$21,274,667$$

To bring the estimate to a 2018 cost for comparison with the predesign cost estimate, we apply an average annual inflation of 3.3% to give a 2021 cost of **$36.9M**.

7.2.2 Comparison to Similar Facilities

An alternate approach to generating an order-of-magnitude cost estimate is using actual costs for similar facilities constructed in the past few years. These costs can be scaled using the following, commonly used, equation called the "rule of six-tenths":

$$C_B = C_A \left(\frac{B}{A}\right)^n$$

Where:

- C_A = cost of facility A
- C_B = cost of facility B
- A = capacity of facility A
- B = capacity of facility B
- n = economy of scale exponent = 0.6

A facility of similar capacity, producing a similar effluent quality but using Sequencing Batch Reactors (SBR) was constructed by design-build in 2013/14. The cost facility, adjusted to 2021 costs is **$42.1M**, and when the rule of six-tenths is applied to bring the cost up to the existing MDF capacity of the Nelson.

7.2.3 Suppliers Opinion of Probable Cost

Finally, supplier costs were compiled in accordance with the treatment process selected in Section 2.2 and an opinion of probable cost was developed. The capital costs are estimated at **$32.7M**, see **Appendix 4** for the break-down of the probable cost.

Equipment suppliers were engaged to provide sizing and current budgetary costing for major components of the alternate treatment facility and included:

- JWC Environmental (Mequipco) — Headworks Screen, Compactor, and Washer
- Veolia — Vortex Grit Separator and Grit Classifier
- Premier Tech — SBRs
- Trojan (Ramtech) — UV Disinfection

An average was taken of the three costing methodologies and summarized in the table below:

Table 7.2 Average Alternate Treatment Plant Capital Cost

Cost Approach	Capital Cost
Burnside Study	$36.9M
Comparison of Similar Facilities	$42.1M
Opinion of Probable Cost	$32.7M
Average	**$37.2M**

8.0 DECISION MATRIX

A workshop review meeting was held with City staff to review and evaluate the preferred option for long-term wastewater management. A decision matrix was utilized to guide decision making to determine whether an alternate wastewater treatment facility is potentially feasible. The matrix compares the base case – the existing Grohman Narrows site, with the alternate case – the potential new site.

8.1 ASSESSMENT PARAMETERS

Five project value categories were selected based on City input and previous assessments Urban has conducted, that generally assess all parameters affecting the decision. The City assigned each category a rating range, or weight, based on their perceived importance and gave a score (positive or negative) based on the impact of the given category relative to the base case. The intent is that the averaged sum of the scores will provide a positive value (further pursue the alternate site) or negative value (remain at current site).

City staff provided input to complete the decision matrix below. Based on the evaluation completed, positive 9.67 was the effect on decision, which means there is preference to proceed with an alternate treatment facility. The scores are summarized in **Table 8.1** and **Figure 8.1**.

Table 8.1 Alternate Site Decision Matrix

Project Value Category	Rating Range	Score Given	Average Score
Site Considerations	±5	-2,3,5	2
Environmental / Social	±5	-2,3,0	0.33
Capital Cost	±3	1,0,0	0.33
Operational Cost	±5	4,3,3	3.33
Operations and Maintenance	±4	3,4,4	3.67
Effect on Decision (sum)		**4,13,12**	**+9.67**

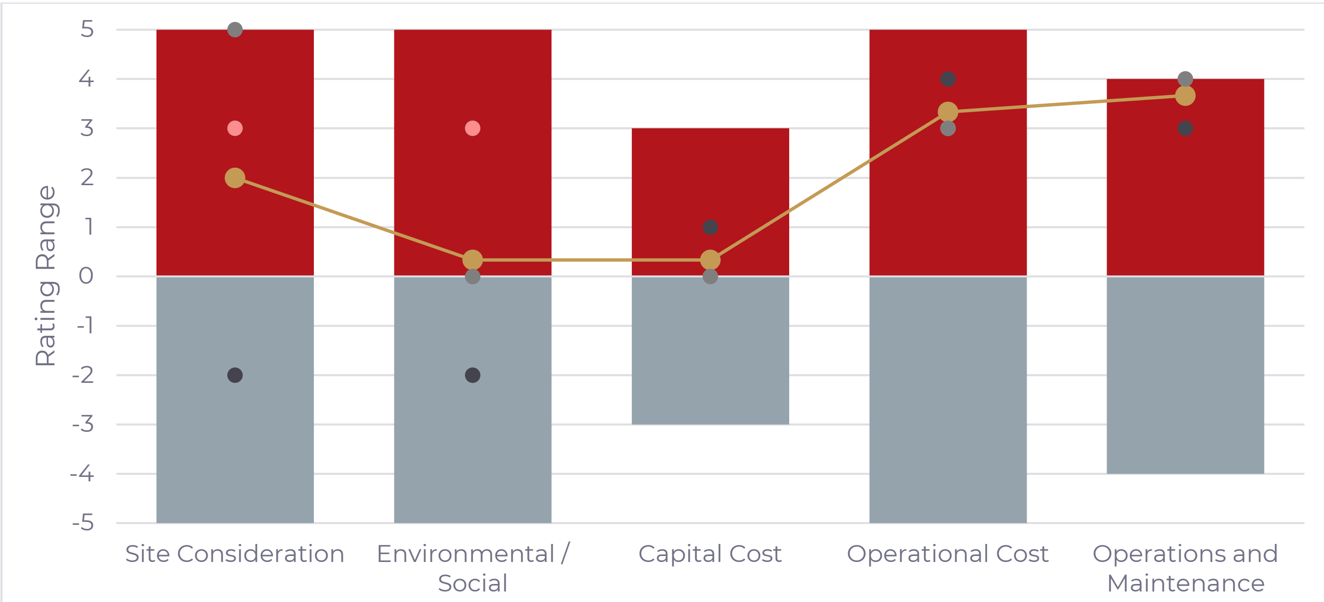


Figure 8.1 Alternate Site Decision Matrix Scoring

8.2 ASSESSMENT CATEGORY REVIEW

8.2.1 Site Considerations (+2)

The existing PCC lot is less than 0.5 ha and further expansion to the facility footprint is not feasible. The site was constructed to old standards and does not conform to modern post-disaster or setback requirements. While it is generally out of sight from the public, it is roughly 7 km from the Public Works yard and has an insecure power supply.

The proposed alternate site of 70 Lakeside Drive is larger with approximately 1.4 ha of useable space. The lot is adjacent to the City's Public Works yard; there is potential to incorporate shared facilities between the two. The alternate site has historically been used as a waste transfer station and landfill. The landfill was closed and the site re-zoned to P1 – Parks. Rezoning and a contaminated sites and geotechnical investigation would be required to confirm the suitability of this particular site. These would determine the best approach to manage potential on-site contaminated soils and groundwater quality as well as how to best address soil stability for structural considerations of the sewage treatment plant. Additionally, there is potential of additional flow-through contamination from the adjacent CP Rail site. These challenges pose cost and feasibility risk as an alternate site because the required remediation is unknown.

8.2.2 Environmental / Social (+0.33)

Treatment / Construction: As described in **Section 2.1**, the existing PCC requires almost complete redevelopment to be adapted to future needs. Redevelopment of operating facilities is challenging and costly, as complicated sequencing with multiple commissioning episodes is required to maintain treatment operations during construction. In contrast, a new facility would be designed for future needs from the ground up and could be constructed and commissioned with minimal effect on the existing systems. A new treated effluent outfall for the alternate site could pose a potential treatment risk as these generally tend to release to environmentally sensitive ecosystems. Additionally, public input has historically suggested "natural treatment" such as wetlands. This could be further explored with a new facility.

Environmental Risk: The City currently conveys all the sewage to the PCC through an approximately 3,100 m long 400 mm diameter submarine force main that routinely leaks. Major restoration of this forcemain will have to be completed, either through replacing the forcemain, re-lining it, or installing a replacement along an alternate alignment. The proposed site would eliminate this force main as the Airport Lift Station is roughly 800 m away with a negligible amount of elevation gain between the two.

The greatest environmental risk of the alternate site is risk of contaminant release from redeveloping the site. Additionally, there is some concern for increased bird activity in the area given the site is adjacent to the airport.

Permitting Risk: Upgrades to the existing PCC poses low to moderate permitting risk. There is a potential challenge with obtaining permitting to bypass treatment if the existing plant cannot be kept in operation during upgrades.

There is moderate risk with obtaining a permit for a new treatment facility. Namely, a new outfall will have to be confirmed and rezoning will be required.

Social Risk: The alternate site is likely to have social challenges due to NIMBY thinking, especially considering the site was designated as park land use in the 2013 Official Community Plan. Assumption is it may be difficult to gain public acceptance of this infrastructure "coming into the City".

Odour: The existing PCC receives seasonal odour completes for that portion of the highway and adjacent parks users. A new facility, while in the industrial area of the City, would incorporate odour management and treatment features to mitigate it.

Appearance: The existing the site is well shielded from the public along Highway 3A and generally is only visible from boaters on the Kootenay River. The alternate site is reasonably well shielded from public visibility behind the existing City's current Public Works Yard in an existing brownfield and industrial area. The site would be visible by boaters on Kootenay River or hikers at Pulpit Rock. A new facility could be used to enhance the area, or landscaping could be used to hide its appearance.

Education / Opportunity: Wastewater treatment facilities are critical pieces of infrastructure for cities. Incorporating educational opportunities into their design can break down a community's misconceptions about their role. The existing facility is currently not suitable for visitors. While upgrades could improve this, there are considerable challenges that exist. A new facility could be designed from the outset to incorporate educational opportunity. 70 Lakeside Drive could provide limited public access via trails on the north end of the site and keeping the Public Works Yard border of the property open to only operators. Finally, there is potential for resource recovery by reusing treated effluent at an adjacent industrial site instead of discharging to the Kootenay River.

8.2.3 Capital Cost (+0.33)

Cost Risk: Preliminary capital cost estimations have been summarized in **Section 3.0**; however, there is still great volatility in these values – these estimates should be used for comparison and high-level planning only. Fundamental engineering design ensures the life of equipment is expected to last at least 20 years and up to 50 years for structures. A complex retrofit would be required to complete the recommended equipment upgrades described in **Section 2.1**. While proper maintenance can prolong the service life of equipment and structures, as of this year the existing structure has reached its service life of 50 years. Appropriate inspection is required to confirm its stability for 50 more years of use. Additional expansion beyond available space may also be required to meet the treatment requirements of the future growth.

Alternatively, constructing a new facility on a new site while continually operating the existing facility is much simpler and could be more cost-effective than upgrading the existing facility while bypassing treatment through temporary works. As a capital project, the service life of the new equipment and structures will at a minimum last for 20 years and 50 years, respectively. Lastly, it is unknown if geotechnical and environmental challenges at 70 Lakeside Drive would result in cost escalation.

Demolition / Remediation: Upgrading the PCC would not require major demolition or rehabilitation. This, however, is contingent on the tanks being sufficient for 50 more years of use. Otherwise, the existing tanks would need to be replaced.

In considering relocating the wastewater treatment facility to a new location, there is added cost and complexity of decommissioning the PCC after the new facility is commissioned.

8.2.4 Operational Cost (+3.33)

The existing PCC incurs staffing costs for travelling to the site. Additionally, there is a need to maintain the road and power lines through the park to the PCC.

The proposed alternate site could reduce staffing and maintenance costs with having the facility integrated into the Public Works Yard. Not being constricted by site footprint like the current facility could allow for a less costly treatment approach to be selected. Additionally, there would be lower power consumption at the Airport Lift Station when pumping to the proposed alternate site.

8.2.5 Operations and Maintenance (+3.67)

While the existing PCC is a familiar site, any retrofits to upgrade it would include operational compromises. This is exacerbated by the fact that almost all the developable footprint of the site has been utilized. Although upgrades to the PCC would improve safety, there are numerous components that cannot be addressed. These include improved access for emergency vehicles, removal of confined spaces, and proximity of workspace to headworks.

The design and siting of a new treatment facility on a larger site could lend itself to a safer, more reliable plant with ease of functionality for operators. The current operators can provide valuable insight of existing deficiencies to ensure the new facility is optimized for the City's needs. A new treatment facility would be tailored to manage the City's current high strength loadings in an efficient and robust operation to meet treated effluent quality standards. A larger site would allow easier access to equipment. A new facility will need to correct the safety deficiencies of the existing PCC by ensuring separation between occupied spaces and headworks, restricting confined spaces, and designing better access for emergency vehicles. Finally, a new facility could be incorporated into the City's Public Works Yard, sharing many facilities, and improving operational efficiencies.

9.0 NEXT STEPS

A new facility will require registration, monitoring, and reporting under the BC Municipal Wastewater Regulation (MWR). An environmental impact study (EIS) will be required as part of the registration. Registration under the Federal WSER would be required, unlike the MWR, there are no requirements for an authorization approval from Environment Canada.

Because the facility will serve more than 10,000 people a BC Environmental Assessment (EA) would be triggered *unless* the City were to complete a Liquid Waste Management Plan (LWMP). An LWMP is a process that results in a MWR registration, but incorporates the additional consultation, economic, environmental, and technical studies that are required. Importantly, unlike the BC EA process, the City would be in control of the LWMP process.

This study has identified various needs, obstacles, and opportunities for long term wastewater management in Nelson. Implementation of a preferred option will require further study, engineering, permitting, public/stakeholder consultation, First Nations consultation, BC Environmental Assessment and financing. It is recommended that the City undertake a LWMP plan to address all of these issues under one project. A successful LWMP would incorporate the following:

- Technical studies
- Public/stakeholder consultation
- Environmental Impact Study (EIS – this is not the same as an Environmental Assessment (EA))
- Options assessment and selection of preferred pathway
- Exemption from BC EA process
- Regulatory input
- Regulatory approval
- Borrowing approval (no additional petition/counter petition process is required for borrowing with an approved LWMP)

The process of developing a LWMP emerges from the BC *Environmental Management Act*. The intent is to allow local governments a mean to achieve community support for sanitary services. By engaging the public, decision makers, technical representatives, and the Ministry of Environment, a completed LWMP can be confidently endorsed as technically rigorous and with the support of the public. An LWMP that has undergone the public process and achieved both the local government approval and the Minister's approval effectively becomes a legal document (much like an Official Community Plan) that sets the stage for long-term management of liquid waste. By including financial considerations within the selected liquid waste management scenario, the City is able and expected to carry out projects and cost-recovery methods as developed during the plan.

Sincerely,

URBAN SYSTEMS LTD.

Matt Smith, P.Eng.
Environmental Engineer / Principal

Shiloh Johnson, EIT
Wastewater Engineer

cc: Anthony Comazzetto, P.Eng.

/sj/ms

Enclosure

U:\Projects_NEL\0795\0119\01\R-Reports-Studies-Documents\R1-Reports\Memos\Memo 04 Treatment Plant Options\2022-05-31 - Memo 04 - Comparison of Sewage Treatment Plant Options r3.docx

Appendix 1

City of Nelson 1977 Pollution Control Permit and 2006 Amendment of Permit file 0795.0065.01

DEPARTMENT OF ENVIRONMENT
WATER RESOURCES SERVICE
POLLUTION CONTROL BRANCH

PERMIT

Under the Provisions of the Pollution Control Act, XXXX

The Corporation of the City of Nelson

502 Vernon Street, Nelson, British Columbia, V1L 4E8

is hereby authorized to discharge effluent

from a municipal sewerage system

located at Nelson, British Columbia

to the Kootenay River

This permit has been issued under the terms and conditions prescribed in the attached appendices 01 and A

Assistant Director of Pollution Control

Permit No. PE-291

Date issued July 31, 1969.
Amendments dated OCT 25 1977, 19..
, 19....
, 19....

COPY

FAXED

Peter G
Kelowna Office
Joanne
(250-374-5334
7 DATE: April 22/03.

Post-it Notes

DEPARTMENT OF ENVIRONMENT
WATER RESOURCES SERVICE
POLLUTION CONTROL BRANCH

APPENDIX No. PE-291

to Pollution Control Permit No. 01

(a) The discharge of effluent applicable to this appendix is from a municipal sewerage system

(Source of operation)

as shown on attached Appendix A

(b) The quantity of effluent which may be discharged is 1,250,000 gallons (5 680 m^3) per day at a maximum rate of 1,875 gallons (8.5 m^3) per minute. 2.7 MGD

(c) The characteristics of the effluent shall be equivalent to or better than an average of total suspended solids - 100 mg/L; BOD_5 - 140 mg/L; coliform bacteria MPN - 150,000 per 100 mL

(d) The works authorized are a primary treatment plant with comminution, flow measurement, and chlorination facilities, outfall and related appurtenances approximately as shown on the attached Appendix A.

(e) The land from which the effluent originates and to which this appendix is appurtenant is the City of Nelson

~~XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX~~

(f) ~~XXXX~~The authority to discharge is contingent upon the authorized works having been constructed as per~~XXXX~~ ~~XXXXXXXXXX~~ plans approved in accordance with the *Pollution Control Act*, ~~XXXX~~.

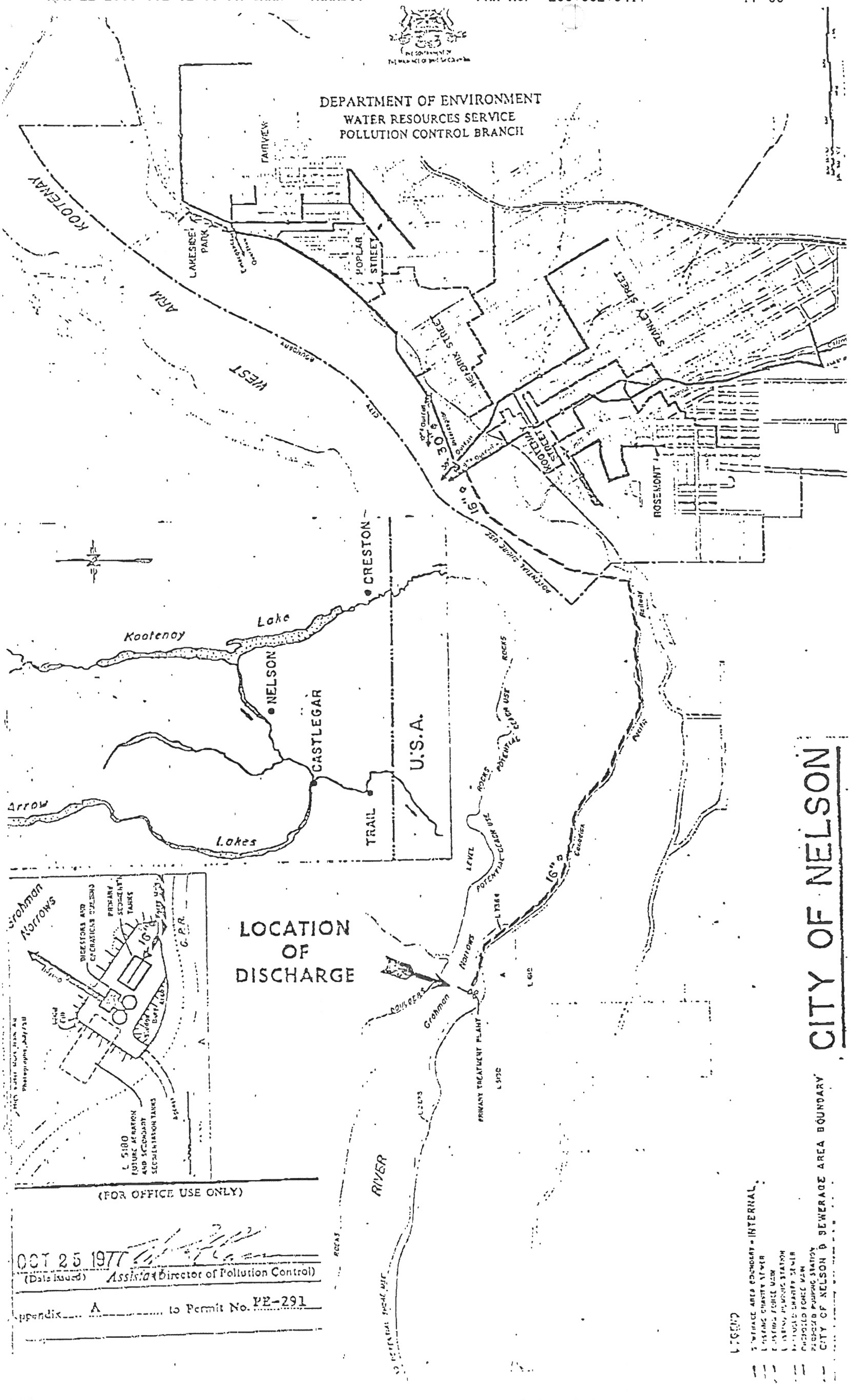
Date issued July 31, 19 69

Date amended OCT 25 1977, 19

, 19

Assistant Director of Pollution Control

PCB 11-0


DEPARTMENT OF ENVIRONMENT
WATER RESOURCES SERVICE
POLLUTION CONTROL BRANCH
FAIRVIEW
LAKESIDE PARK
POPLAR STREET
KOOTENAY
WEST ARM
STANLEY STREET
KOOTENAY STREET
ROSEMONT
Kootenay Lake
CRESTON
NELSON
CASTLEGAR
TRAIL
U.S.A.
Arrow Lakes
LOCATION OF DISCHARGE
Grohman Narrows
PRIMARY TREATMENT PLANT
RIVER
G.P.R.
CITY OF NELSON
LEGEND
CITY OF NELSON & SEWERAGE AREA BOUNDARY
(FOR OFFICE USE ONLY)
OCT 25 1977
(Date Issued) Assistant Director of Pollution Control
Appendix A to Permit No. PE-291

Province of British Columbia
Ministry of The Environment
POLLUTION CONTROL BRANCH
Victoria British Columbia V8V 1X4
Phone. 387-5321

YOUR FILE ....
OUR FILE 0262100-PE-291

OCT 25 1977

The Corporation of the City of Nelson
502 Vernon Street
Nelson, British Columbia
V1L 4E8

Gentlemen:

Amendment to Pollution Control Permit PE-291
The Corporation of the City of Nelson.

Following receipt of your letter dated August 26, 1977 further consideration was given to the items raised and our recommendations together with your letter were then forwarded to the Director for final consideration. You should have, or will be, receiving notification of that decision shortly.

With regard to the monitoring program you proposed, our recommendations to the Director represented a reduced program overall by omitting the analysis of both digester sludge and supernatant monitoring but increasing effluent monitoring to four times per month instead of three. We are in agreement on the frequency of plant influent monitoring at once per month and flow measurements daily. We concur nevertheless that monitoring of the digester operation is important but such analysis will not be a requirement in conjunction with the Permit amendment.

Yours very truly,

R. Schurr

R.C. Schurr, P.Eng.,
Head - Interior Section
Municipal Division

RCS/tm

cc: Regional Manager - Kootenay

RECEIVED OCT 28 1977

British Columbia
The Environment
POLLUTION CONTROL BRANCH
British Columbia
V8V 1X4
Phone: 387-5321

YOUR FILE ....
OUR FILE 0262100-PE-291

OCT 25 1977

DOUBLE REGISTERED

The Corporation of the City of Nelson
502 Vernon Street
Nelson, British Columbia
V1L 4E8

Gentlemen:

LETTER OF TRANSMITTAL

Enclosed is a copy of Pollution Control Permit No. PE-291 in the name of the Corporation of the City of Nelson, amended as of this date. The terms and conditions of this amended Permit replace and supersede those of Permit No. PE-291 dated July 31, 1969 as of and from the date of this Amendment.

Your attention is respectfully directed to the terms and conditions now outlined in the amended Permit. It should be noted that the expiry date has been deleted and the Permit updated to the format currently in use.

In addition, as of and from the date of this Amendment, this Letter of Transmittal replaces and supersedes the Letter of Transmittal dated July 31, 1969 and the letter of January 31, 1972 which outlined monitoring requirements for PE-291.

In conjunction with this amended Permit, you are directed to comply with the following requirements:

A. CHLORINATION

Maintain a chlorine residual between 0.1 and 1.0 mg/L at all times and provide not less than one hour chlorine contact time at average flow rates.

B. REDUCTION OF NON SANITARY FLOWS IN SEWERAGE SYSTEM

In issuing this permit due cognizance was taken of your commitment to significantly reduce non-sewage flows from and carry out necessary improvements to the sewerage system by December 31, 1979.

It should also be noted that a review of Permit criteria and the capabilities of the treatment plant under normal operating conditions must await assembling adequate monitoring information for an assessment as corrective measures are completed.

...2

C. MONITORING

1. Sampling

Install suitable sampling facilities and obtain a composite sample of the effluent weekly. In addition, obtain a composite sample of the raw influent once per month, coincident with one of the effluent samples. These samples are to be composited in proportion to flow over 8 hours in daytime.

2. Analyses

Obtain analyses of the samples as follows:

a) total suspended solids;
b) 5-day biochemical oxygen demand;
c) fecal coliforms - monthly on a grab sample of treated effluent only.

Analyses are to be carried out in accordance with procedures described in the second edition (February 1976) of "A Laboratory Manual for the Chemical Analysis of Waters, Wastewaters, Sediments and Biological Materials", and/or by suitable alternative procedures as approved by the Director.

Copies of the above mentioned manual are available from the Environmental Laboratory, 3650 Wesbrook Crescent, Vancouver, British Columbia, V6S 2L2 at a cost of $10.00, and are also available for inspection at all Pollution Control Branch offices.

3. Flow Measurement

Install a flow measuring device acceptable to our Regional Manager and measure the effluent volume discharged during each 24 hour period.

4. Reporting

Maintain data of analyses and flow measurements for inspection and annually submit the data suitably tabulated to the Director, for the previous year's monitoring, the first report to be submitted by March 1, 1978. The need for subsequent increased or decreased monitoring will be based on this data and any other data obtained by the Pollution Control Branch in connection with this discharge.

...3

D. BYPASSES

Discharges of dry weather effluent flows which have bypassed any portion of the designated treatment works under Permit is prohibited.

E. PROCESS MODIFICATIONS

The Permittee shall notify the Director prior to implementing any changes to the treatment works that may affect the quality and/or quantity of the effluent.

F. MAINTENANCE PROCEDURE

Inspect regularly the pollution control works and maintain them in good working order.

Notify the Director of significant malfunction of these works.

You will note that values have been expressed in the International System of Units (SI). You are encouraged to use these units in submitting monitoring results and any other information in connection with this Permit.

The administration of this Permit will be carried out by staff from our Regional Office located at 310 Ward Street, Nelson, British Columbia, V1L 5S4 (telephone 352-2211, local 273). Plans, data and reports pertinent to the Permit are to be submitted to the Director through the Regional Manager at this address.

This Permit does not authorize entry upon, crossing over, or use for any purpose, of private or Crown lands or works, unless and except as authorized by the owner of such lands or works. The responsibility for obtaining such authority shall rest with the Permittee.

For your reference, enclosed is a copy of the Pollution Control Act, as amended, and the Regulations thereto. Also enclosed is a copy of the Metric Practice Guide.

Yours very truly,

H. P. Klassen, P. Eng.
Assistant Director (Services)
Pollution Control Branch

Enclosure

PCB 11— 0

BRITISH COLUMBIA

March 2, 2006 File: PE-00291

City of Nelson
Suite 101, 310 Ward Street
Nelson, BC V1L 5S4

Attention: Gil Bogaard, Utilities & Arena Supervisor

Dear Gil Bogaard:

Re: Amendment of Permit PE-00291

This letter amends Permit PE-00291 issued July 31, 1969 and amended November 2, 1997 by eliminating the need to comply with Section A. Chlorination, subject to all of the following conditions:

1. Chlorination of the final effluent be stopped and the discharge be diverted through the existing ultra violet system.
2. The primary effluent be directed through the secondary treatment system's 4 Rotating Biological Contactors.
3. During high storm events, if the plant exceeds its maximum capacity and needs to bypass the secondary system;
   a. the permittee must notify this office by fax immediately when the bypass begins and ends;
   b. all discharges to the Kootenay River must continue to pass through the ultra violet disinfection;
   c. a report must be submitted to this office, as soon as is reasonably possible, which includes all data collected during the bypass, including:
      i. the total flow bypassing the secondary;
      ii. daily Faecal Coliform, TSS and BOD reading of the discharge (to the Kootenay River);
      iii. the turbidity and the ultraviolet light transmittance readings during the above sampling.

Yours truly,

Ric W Baker

Ric Baker, P.Eng.
For Director, *Environmental Management Act*

RB:lkm

Ministry of Environment
Environmental Protection
Kootenay and Okanagan Regions
Mailing/Location Address:
401 - 333 Victoria Street
Nelson BC V1L 4K3
Telephone: 250 354-6355
Facsimile: 250 354-6332

March 1, 2006

Technical Assessment for a Bypass of the Nelson STP (PE-00291)

HISTORY

The City of Nelson has applied for a bypass of their effluent permit from the Sewage Treatment Plant. The permit, which was issued in 1969 and amended in 1977 has not been changed for almost 30 years. An upgrade to the works for secondary treatment has now been completed and commissioning of the 4 new Rotating Biological Contactors (RBCs) is starting. In order for the RBCs to grow enough biomass to reduce the bacteria, Biochemical Oxygen Demand (BOD) and Total Suspended Solids (TSS) to Municipal Sewage Regulation levels, 2 to 3 months may be required.

Since this growth media is a living mass, chlorine would kill the entire colony and no secondary treatment could occur. It is for this reason that the hypochlorite must be shut off and the effluent diverted through the ultraviolet banks. Enough TSS reduction may not be achieved for 3 to 4 weeks in order for the ultraviolet transmittance to be totally effective during high inflow events. Since this time of year is typically when storm events cause overflows from the plant, added risk to the environment is present. However, storm events also increase River flows such that the dilution factor of 1500:1 to 2000:1 is maintained.

The current permit requires that a chlorine residual be maintained at all times to protect human health. In order for growth to occur on the RBCs and for secondary treatment to begin I recommend the following:

1. Chlorination of the final effluent be stopped and the discharge be diverted through the ultra-violet system.
2. The primary effluent be directed through the 4 RBCs in the secondary treatment system.
3. During high storm events when the plant exceeds its maximum capacity and bypasses the secondary system;
   a. all discharges must still pass through the ultraviolet disinfection;
   b. the total flow bypassing the secondary must be recorded;
   c. the discharge to the river must be sampled daily for Faecal Coliform, TSS and BOD;
   d. the turbidity and the ultraviolet light transmittance must be recorded during the sampling.

Terri Kinrade
Environmental Protection Officer

Agreed.
[signature]
Mar 02/06

PROVINCE OF
BRITISH COLUMBIA

MINISTRY OF
ENVIRONMENT,
LANDS AND PARKS

Environmental Protection
617 Vernon Street
Nelson, BC, V1L 4E9
Telephone: (604) 354-6355
Fax: (604) 354-6367

APR 15 1993

File: PE-00291

REGISTERED MAIL

The Corporation of the City of Nelson
502 Vernon Street
Nelson, British Columbia
V1L 4E8

Gentlemen:

**NOTICE OF MINOR AMENDMENT TO PERMIT PE-00291
ISSUED UNDER THE PROVISIONS OF
THE WASTE MANAGEMENT ACT, S.B.C. 1982, c.41,
IN THE NAME OF THE CORPORATION OF THE CITY OF NELSON**

Please insert the attached amendment in your Permit package.

If you have any questions regarding Facility Classification or Operator Certification please contact Barry Wood, Environmental Protection, Nelson, B.C. at 354-6355 or Gary Lawrence, Environmental Protection, Cranbrook, B.C. at 426-1475.

Yours truly,

R.J. Crozier, R.P. Bio.
Regional Waste Manager

/sw

Attachment

FACILITY CLASSIFICATION

The wastewater treatment facility must be classified and the classification maintained with the "British Columbia Water and Wastewater Operators Certification Program Society" (BCWWOCPS). An application to classify the facility must be received by the BCWWOCPS by August 1, 1993.

OPERATOR CERTIFICATION

December 1, 1994

All operators employed at municipal wastewater treatment facilities classified by the B.C. Operators Certification Program at Level II or higher shall be certified by the Program to a Class I level at a minimum.

Operators in Training (OIT)

Operators in training at municipal wastewater treatment facilities classified by the B.C. Operators Certification Program Level II or higher will be required to successfully pass an OIT examination within three (3) months of commencement of employment.

The OIT Certificate will be valid for fifteen (15) months from the date of issue.

Prior to the expire date of the OIT Certificate, but not sooner than twelve (12) months from the date of hire, the operator shall successfully complete a Class I certification examination in order to remain in the wastewater treatment field.

Date issued: July 31, 1969

Amendment date: APR 15 1993
(most recent)

R.J. Crozier, R.P. Bio.
Regional Waste Manager

Permit Number: PE-00291

December 1, 1996

All owners of municipal wastewater treatment facilities classified by the B.C. Operators Certification Program at Level II or higher shall designate at least one operator to be the Chief Operator of the facility.

All municipal wastewater treatment facilities classified at Level II or higher shall have a "Chief Operator" certified at a Class II level at a minimum.

After this date, no person shall have Direct Responsible Charge of a municipal wastewater treatment facility classified at Level II or higher unless they possess a valid operator's certificate not more than one level below the level of the facility (i.e. a Class I operator may not have even temporary control of a Level III or Level IV facility).

December 1, 1998

All municipal facilities classified by the B.C. Operators Certification Program at Level III shall have a "Chief Operator" certified at a Class III level at a minimum.

All municipal facilities classified by the B.C. Operators Certification Program at Level IV shall have a "Chief Operator" certified at a Class IV level.

All operators are encouraged to seek certification at the highest possible level. The structure of the Certification Program allows an operator to be certified to a level one higher than the level at which his/her facility is classified (i.e. an operator at a Level II facility could certify to the Class III level).

Date issued: July 31, 1969

Amendment date: (most recent) APR 15 1993

R.J. Crozier, R.P. Bio.
Regional Waste Manager

Permit Number: PE-00291

Appendix 2

2018 Pollution Control Centre Upgrade Assessment

City of
NELSON
BRITISH COLUMBIA

Draft Report

City of Nelson

Pollution Control Centre Upgrade Assessment

URBAN systems

625 Front Street
Nelson BC, V1L 4B6

August 2018
File: 0795.0108.01

Dr. Joanne Quarmby, R.P.Bio.
T: 250.374.8311
jquarmby@urbansystems.ca
urbansystems.ca

Prepared for:

City of NELSON BRITISH COLUMBIA

101 – 310 Ward Street
Nelson, BC V1L 5S4

Attention: Chris Gainham and Scott Eagleson

Prepared by:

URBAN systems

625 Front Street
Nelson BC, V1L 4B6

Contact: Matt Smith, P.Eng.
Environmental Engineer/Principal
T: 250.374.8311
msmith@urbansystems.ca

File: 0795.0108.01 / August 2018

Table of Contents

Appendices

1 EXECUTIVE SUMMARY

The City of Nelson operates the Grohman Narrows Pollution Control Centre (PCC) and discharges treated effluent to the Grohman Narrows. The PCC was last upgraded in 2005 to provide secondary treatment. The primary concern with the PCC is its ability to consistently meet the requirements of the Federal Wastewater Systems Effluent Regulation (WSER) for 5-day carbonaceous biochemical oxygen demand ($CBOD_5$). The regulation requires that quarterly average $CBOD_5$ effluent concentration be 25 mg/L or less. In 2017, this average was only met in the second quarter.

Given the inconsistency in meet the regulatory requirements that are now in place, the City retained Urban Systems to perform a review of the treatment processes and recommend upgrade options.

This assessment reviews the historic hydraulic and organic loading on the PCC and assesses the capacity of the major treatment processes. During the review of the historic loading data, it was found that the organic loading on the PCC has increased substantially in recent years independent of population growth. A high strength wastewater study has been initiated by the City at the recommendation of Urban Systems to determine the cause of the increasing load on the PCC.

A review of the PCC determined that a number of the major treatment processes are currently at, or have exceeded, their rated capacity, which would lead to poor treatment performance. The major processes that require capacity upgrades includes: primary clarifiers, equalization tanks, secondary treatment (RBCs) and secondary clarifiers. Other supporting systems were also determined to be at capacity, including the electrical service, emergency backup generator and headworks screen. The UV system was also found to not meet the Municipal Sewage Regulations (MSR) reliability requirements and should be upgraded.

To achieve the effluent discharge requirements prescribed under the WSER and MSR, the future hydraulic and organic loads have been estimated. The loads are estimated to 2037 at the population growth rate of 1.5% from the Official Community Plan (OCP).

Four secondary treatment technologies are assessed at the 2037 projected loads: rotating biological contactor (RBC), moving bed bioreactors (MBBR), sequencing batch reactors (SBR) and complete mixed activated sludge (CMAS). The major constraint at the PCC site is the limited available space for expansion. The CMAS and RBC technologies physically could not fit within the available space while SBR could, but could not be expanded beyond the 20-year design horizon. The MBBR process is the most promising technology assessed because it can be easily integrated into the existing plant while still allow for future expansion on the site.

In order to integrate MBBR into the existing PCC, the electrical service, headworks, primary clarifiers and secondary clarifiers require upgrading. It is recommended that the existing equalization, primary clarifier and RBC tanks be retrofitted to MBBR. New mechanical primary filters are proposed to replace the primary clarifier tanks. To protect the mechanical primary filters and MBBR processes a new headworks building is required. Also, the secondary clarifiers are proposed to be replaced with two new dissolved air flotation (DAF) units.

It is assumed in the assessment that the anaerobic digesters will remain in service. This means that the primary clarification process is to be maintained; this is accomplished by providing mechanical primary filters. It is outside the scope of this assessment to assess changing the anaerobic digestion process and it is recommended that a condition assessment of the anaerobic digesters be performed. The condition assessment will assist decision makers in determining if a second study is required to consider a change in the current sludge management process (i.e. a changes to digestion processes).

It is also recommended that the UV disinfection system be upgraded to provide the prescribed reliability requirements under the MWR.

A phased approach to upgrading the PCC is proposed with the following estimated capital and operating costs per phase:

Phase	Process Upgrade	Capital Cost	O&M
Phase 1	Emergency Generator	$240,000.00	-
	Electrical Service Upgrades	$300,000.00	-
Phase 2	Headworks	$3,300,000.00	$2,000.00
Phase 3	Primary Filtration	$4,700,000.00	$47,000.00
Phase 4	MBBR	$5,100,000.00	$22,000.00
	DAF	$5,300,000.00	$58,000.00
Phase 5	UV Upgrade	$450,000.00	$7,000.00
Total	**All Phases**	**$19,390,000.00**	**$136,000.00**

The following next steps are recommended:

1. Secure financing to undertake detailed design and construction of Phase 1 and Phase 2.
2. Perform a condition assessment of the anaerobic digesters to estimate the remaining life and renewal requirements for the process.
3. Re-assess Phase 3 after the anaerobic condition assessment has been completed and determine if an additional study is required that would review the cost implications of a change in sludge management.
4. Complete the remaining phases. Note, an environmental impact assessment (EIS) and MWR registration update may be required at Phase 4.

2 BACKGROUND

2.1 Introduction

The City of Nelson operates the Grohman Narrows Pollution Control Centre (PCC) and discharges treated effluent to the Grohman Narrows approximately 5 km west of the City of Nelson. The PCC was upgraded in 2005 to provide secondary treatment with the addition of rotating biological contactors (RBCs), secondary clarifiers and replacement of the chlorination system with an ultra-violet (UV) light disinfection system. Before 2005 the PCC was limited to primary treatment only. The 2005 upgrade included the first phase of RBCs which were anticipated to reach capacity approximately 10 years after installation. A second phase with the installation of a third RBC train was envisioned subsequently, but has not been constructed.

Since the upgrades, there have been several changes in the regulatory framework for domestic wastewater treatment. In 2012, the BC Municipal Sewage Regulation (MSR) was repealed and replaced with the BC Municipal Wastewater Regulation (MWR). Also in 2012, the Federal Wastewater Systems Effluent Regulations (WSER) was introduced into law.

The primary concern with the PCC is its ability to consistently meet the requirements of the Federal WSER for 5-day carbonaceous biochemical oxygen demand ($CBOD_5$). The regulation requires that quarterly average $CBOD_5$ effluent concentration be 25 mg/L or less. In 2017, this average was only met in the second quarter. However, the ability to meet the WSER effluent TSS requirement of 25 mg/L as a quarterly average was also raised as a concern. In 2017, this average was only met in the second and third quarters. In both cases, the average concentration was calculated to be 25 mg/L, which is on the threshold of non-compliance.

The effluent water quality has been highly variable and has exceeded the WSER limit on many occasions; thereby, prompting a closer examination of the process treatment and upgrade options.

Given the lack of ability to consistently meet the regulatory requirements that are now in place, a review of the process treatment and upgrade options was recommended to enable the City to make an informed decision as to how to proceed with the wastewater treatment plant into the future.

2.2 Scope of Work

The scope of the wastewater treatment plant assessment and options analysis is:

1. Collate and summarise process and effluent data and compare these data to Federal and Provincial regulatory requirements.
2. Summarise current treatment, operational and performance issues and concerns.
3. Summarise future treatment or performance concerns, including health concerns that may have been raised during the operator discussions.
4. Assess options with respect to the optimisation of the current treatment processes and alternative process approaches.

3 CURRENT LOADING

3.1 Current Hydraulic Loading

The PCC receives 100% of its influent from the City's main lift station, also referred to as the airport lift station. Detailed flow records from the airport lift station's flow meter were provided by the City, and data for January 2014 to May 2017 were analysed. The PCC hydraulic loading conditions in this time period were calculated and are summarized in Table 1.

Table 1 - Summary of Hydraulic Flows from the Airport Lift Station 2014 to 2017

Parameter	Units	2014	2015	2016	2017
Average Day Dry Weather Flow (ADWF)	m^3/d	5,400	5,300	5,500	-
Average Annual Daily Flow (AADF)	m^3/d	5,500	5,400	5,600	-
Average Day Wet Weather Flow (AWWF)	m^3/d	5,900	6,000	5,900	-
Maximum Month Flow (MMF)	m^3/d	6,500	6,600	6,800	9,900
Maximum Day Flow (MDF)	m^3/d	9,200	12,200	9,400	14,600
Peak Hour Flow (PHF)	m^3/d	16,500	20,100	18,400	17,100

AADF: Average of the total flow in each year.

ADWF: Average flow rate each day where precipitation as reported by Environment Canada was less than 1 mm.

AWWF: Average flow rate each day where precipitation as reported by Environment Canada was greater than 1 mm.

MMF: The maximum value in each year of a 30-day running average.

MDF: The maximum value in each year of a 24-hour running average.

PHF: The maximum 1-hour value.

From the 2014 to 2017 flow data reviewed, the values in Table 2 were selected to represent the current hydraulic condition. For those values that are averages, the average of the years from 2014 to 2016 were used and the values that are maximums or peaks, the highest recorded value from 2014 to 2017 was selected.

Table 2 - Current Hydraulic Conditions

Parameter	Units	Current
Average Dry Weather Flow (ADWF)	m^3/d	5,400
Average Annual Daily Flow (AADF)	m^3/d	5,500
Average Wet Weather Flow (AWWF)	m^3/d	6,000
Maximum Month Flow (MMF)	m^3/d	9,900

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Parameter	Units	Current
Maximum Day Flow (MDF)	m^3/d	14,600
Peak Hour Flow (PHF)	m^3/d	20,100

3.2 Current Biological Loading

The available influent water quality records for $CBOD_5$ and TSS were assessed from 2014 to 2016. The influent concentration and mass load for both $CBOD_5$ and TSS have increased annually over this period. The calculated results are summarized in Table 3.

Table 3 - Calculated Average $CBOD_5$ and TSS Loads

Year	Average Annual Daily Flow (m^3/d)	Average Daily $CBOD_5$ Load (kg/d)	Average $CBOD_5$ Concentration (mg/L)	Average Daily TSS Load (kg/d)	Average TSS Concentration (mg/L)
2014	5,500	1,206	219	1,351	246
2015	5,400	1,460	270	1,544	286
2016	5,600	1,553	277	1,551	277

Medium and high strength wastewaters are defined as having concentrations of 200 mg/L and 400 mg/L BOD_5 respectively (Metcalf & Eddy), with the City's medium strength wastewater trending upwards in concentration.

The typical literature and design per capita loading rate for North American communities is 70 g $CBOD_5$/c/d. The calculated values for the City have risen from 106 g/c/d to 151 g/c/d from 2013 to 2016. The calculated per capita rates are summarized in Table 4.

Table 4 - Calculated Annual Average $CBOD_5$ per Capita Loading Rates

Year	Units	Values
2013	g/c/d	106
2014	g/c/d	114
2015	g/c/d	137
2016	g/c/d	151

The per capita $CBOD_5$ loading in 2015 and 2016 would be approximately equivalent to twice the current population of the City. This suggests that there are additional biodegradable loads being discharged to the sanitary sewage collection system. The biological load coming into the PCC is higher than would be expected from the population alone. It is likely that the additional load is coming from one or more of the following sources:

1. “Ghost population” – tourism or seasonal worker populations
2. Food processing
3. Brewing/distilling/wineries

Ghost populations normally cause an increase in flow and load, but do not cause high concentration sewage. Because Nelson has higher wastewater strength than expected, it is likely that some type of industrial process is contributing high strength waste to the system.

Nelson is home to successful breweries including the Nelson Brewing Company and Torch brewery, each of which is known to have increased production in recent years. Breweries are known to produce high strength wastewater that has been demonstrated to have a significant impact on domestic wastewater facilities in other communities. For the purposes of this report, it is assumed that there are two concurrent loads: a population load and an institutional, commercial, industry (ICI) load.

To determine the current loading for 2017, the typical North American CBOD5 and TSS per capita loading rates of 70 g/c/d along with the projected 2017 population were used. The ICI load was broken out and projected forward into 2017 assuming the same rate of change that was observed between 2015 and 2016 (a 22% increase). The calculated 2017 biological load and influent concertation is summarized in Table 5.

Table 5 - Current Influent Biological Concentrations and Loads

Parameter	Units	Current
$cBOD_5$*	mg/L	336
$cBOD_5$	kg/d	1,850
TSS*	mg/L	289
TSS	kg/d	1,590

** Calculated using average annual day flow*

4 EXISTING FACILITY

4.1 Overview

The City of Nelson wastewater treatment plant operates under the BC Ministry of Environment Permit Pollution Control Permit PE-291. The wastewater treatment plant is a secondary treatment facility and consists of the following processes:

- A headworks facility with a mechanical screen, manual bypass channel, aerated grit tank, and grit classifier.
- Two parallel primary clarifiers with scum removal.
- Two parallel aerated equalization tanks with two low-lift pumps (one pump per tank).
- Four rotating biological contactors (RBC), two trains of two RBCs in series.
- Two parallel secondary clarifiers with inclined plate settlers for enhanced sedimentation.
- An ultraviolet (UV) disinfection system.
- Two high-rate anaerobic digesters in series.
- A centrifuge.
- An emergency backup generator.

4.1.1 *Reliability Category*

The most recent environmental impact assessment (EIS) was prepared October 2014 as requested by the BC Ministry of Environment under the MSR. That report to did not discuss the reliability category for the PCC. However, the 2004 Predesign Report did indicate that the facility is in Reliability Category II under the MSR. The MWR defines three categories as follows:

> *"(a) Category I - Treatment works for reclaimed water or that discharge to waters or land that could be permanently or unacceptably damaged by effluent that is degraded in quality for even a few hours (for example, discharges near drinking water sources, shellfish waters or waters used for contact sports where "shellfish waters" means water bodies that have or could have sufficient shellfish quantities that recreational or commercial harvesting would take place or water for which commercial shellfish leases have been issued);*
>
> *(b) Category II - Treatment works that discharge to waters or land that would not be permanently or unacceptably damaged by short term effluent degradation, but would*

URBAN systems

be damaged by continued (several days) effluent quality degradation (for example discharges to recreational land and waters);

(c) Category III - Treatment works not otherwise designated as Category I or II."

The reliability category prescribes the minimum level of redundancy required in different treatment processes to treat a specified portion of the design flow. Treatment process redundancy is required to allow for process models to be taken offline for servicing (or in case one should fail) and still provide a margin of treatment efficiency. For the purposes of this report, the reliability category for the PCC is assumed to be Reliability Category II under the MWR.

4.2 Secondary Treatment Upgrade Design Parameters (2004)

The hydraulic design criteria used in the predesign report by Urban Systems 2004, are summarized in Table 6. Two per capita flow rates of 420 L/c/d and 560 L/c/d were used to calculate the average annual daily flow and average wet weather daily flow respectively, using a 2028 projected population of 13,000.

Table 6 - 2004 Design Hydraulic Loading Rates

Parameter	Units	Value
Average Annual Daily Flow (AADF)	m^3/d	6,000
Average Wet Weather Daily Flows (February to June)	m^3/d	7,300
Wet Weather Infiltration Allowance	m^3/d	2,000
Maximum Day Flow (MDF)	m^3/d	9,300
Peak Hourly Flow (PHF)	L/s	180

From the 2004 predesign report, the design concentrations used to calculate the design loads are summarized in Table 7. The influent total phosphorus and ammonia concentrations were determined from one 8-hour composite sample.

Table 7 - 2004 Design Concentrations

Parameter	Units	Value
Average BOD_5	mg/L	200
Peak BOD_5	mg/L	280
Average TSS	mg/L	170
Peak TSS	mg/L	350
Total Phosphorus	mg/L	2.65
Ammonia	mg/L	16.3

The primary clarifier effluent was assumed to have a 30% to 40% removal of BOD_5 and 60% to 70% removal of TSS. These values correspond with literature values in Metcalf & Eddy and were used to design the downstream biological loading on the assessed secondary treatment processes in the 2004 report.

From the concentration reported in Table 7 and the primary removal efficiencies discussed, the design load on the facility and on the secondary treatment process is summarized in Table 8.

Table 8 – 2004 Design Loads

Parameter	Concentration (mg/L)	PCC Load (kg/d)	Secondary Treatment Load (kg/d)
Average BOD_5	200	1,200	720
Peak BOD_5	280	1,680	1,100
Average TSS	170	1,020	310
Peak TSS	350	2,100	740
Total Phosphorous	2.65	15.9	-
Ammonia	16.3	97.8	-

Load calculated using AADF

4.3 Headworks

The headworks consists of screening and grit removal. A mechanical coarse bar screen is installed in the main channel, and a static bar screen is installed in an emergency bypass channel. Should the mechanical screen fail, or have its capacity exceeded, influent backs up and flows through the static bar screen into the bypass channel. The operators report that the mechanical screen is at, or beyond, capacity and is hydraulically overloaded during peak flows. The bypass channel retains water after use and requires that operators manually pump out and clean the channel to prevent septic conditions.

The aerated grit removal tank facilitates the removal of grit and scum. The settled grit is continuously pushed to one end by a submerged rake and chain system. This grit is drawn from the bottom of the grit tank by a single pump (no backup) and is transferred to a grit classifier for disposal in a solids bin. Any floating scum is pushed to a scum collection trough by a continuously flowing nozzle system. The scum is then collected and pumped to the anaerobic digesters.

The capacity of the aerated grit tank was conservatively estimated using the maximum suggested detention value of 5-minutes, as suggested by Metcalf & Eddy 5th Edition. This calculation determined the maximum hydraulic capacity of the aerated grit tank and is summarized in Table 9.

Table 9 - Aerated Grit Tank

Parameters	Units	Value
Width	m	3.6
Length	m	16
Depth	m	3.5
Volume	m^3	202
Target HRT	min	5
Max flow	m^3/d	58,000

The existing aerated grit tank should have sufficient capacity for the foreseeable future. The aeration rate was not assessed and would be an important consideration in determining the capacity. Aeration rates are recommended to be 0.2 to 0.5 $m^3/m/min$ to achieve the performance calculated above. Process redundancy under the MWR is not required for preliminary treatment.

4.4 Primary Clarifiers

After preliminary treatment at the headworks there are two parallel, rectangular primary clarifiers with a total surface area of 169.5 m^2. The primary clarifier process removes scum and a portion of the TSS and $CBOD_5$. The primary clarifiers help to reduce the load on downstream processes. Secondary sludge from the secondary clarifiers is returned to the headworks and is removed along with the primary sludge in the primary clarifiers. Settled primary sludge, secondary sludge and scum is collected and pumped to the anaerobic digesters for treatment. Finally, primary effluent is collected by v-notch weirs and flows by gravity to the adjacent equalization tanks.

The capacity of the two existing primary clarifiers was estimated using typical design values for the surface area loading rate and overflow rate of a non-activated sludge influent from WEF Manual of Practice No. 8, 5th Edition. The estimated hydraulic capacities are reported in Table 10.

Table 10 - Estimated Primary Clarifier Hydraulic Capacity

Parameter	Units	Average Daily Flow	Peak Hour Flow
Area per Tank	m^2	85	85
Surface Overflow Rate	$m^3/m^2/d$	40	100
Capacity - One Tank	m^3/d	3,390	8,500
Capacity- Two Tanks	m^3/d	6,780	17,000
Current Hydraulic Loading	m^3/d	5,600	20,100

The review of the primary clarifiers indicates that under current average annual flow rates there is sufficient hydraulic capacity; however, the primary clarifiers are hydraulically overloaded at the peak

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hour flow which may result in carryover or resuspension of settled solids. It should also be noted that in the event that one primary clarify must be taken offline for servicing, or due to mechanical failure, there is insufficient capacity in the remaining clarifier to treat the average annual flow rates. This situation may lead to overloading of the downstream secondary treatment process.

Under the MWR, sufficient primary clarifier capacity must be available to provide effective treatment of 50% of the design maximum flow with the largest clarifier offline. The design flow for the primary clarifiers would be 50% of the peak hour demand (10,050 m^3/d) which cannot be achieved by one primary clarifier (8,500 m^3/d).

4.5 Equalization Tanks

The equalization (EQ) tanks are intended to attenuate the peak instantaneous flow so that the loading to the secondary treatment system more constant. The existing EQ tanks provide a total storage volume of 155 m^3 and two pumps transfer the primary effluent to the secondary treatment process. At the time of commissioning in 2005, the EQ pumps had a recorded flow rate of approximately 100 L/s, but that may have changed over time as the pumps wear.

If the inflow to the tanks exceeds the pumping and storage capacity then an emergency overflow weir allows excess flow to spill over where it mixes with treated effluent before being discharged to the outfall in Grohman narrows. The main characteristics of the equalization tank are summarized in Table 11.

Table 11 - Equalization Tank Capacity Before Overflow

Parameter	Units	Values
Tank Volume	m^3	155
Pump Capacity (2 pumps)	L/s	140
Pump Capacity	m^3/d	12,100
Max Day Flow	m^3/d	14,600
Peak Hour Flow	m^3/d	15,820
Peak Hour Volume	m^3	659
Pump Volume (1 hour, 2 pumps)	m^3	504
Difference in Volume	m^3	155

The equalization tank pump capacity is exceeded on the current max day flow condition. Additional pumping capacity is required to prevent annual emergency overflows during the maximum day flow. However, increasing pumping capacity may result in hydraulically overloading downstream treatment processes (this will be assessed further in the report).

The equalization tank can attenuate the current peak hour (155 m^3) with both pumps operating. However, this statement is only true if the flow following the peak hour immediately drops to, or less than, the pump capacity of both pumps operating. Given that the peak hour flow may occur on the same day as the maximum day, it can be presumed that the existing equalisation tank system is over capacity at current loading conditions.

The MWR does not prescribe redundancy requirements for equalization basins. However, it is best practice to have sufficient redundancy in pump capacity should the largest pump be offline. The equalization basins currently do not have a backup pump installed.

4.5.1 *Overflow Events*

Peak flow rates can occur during periods of intense rainfall, or rapid snow melt, resulting in emergency overflow events. The PCC reported 22 overflow events from 2013 to 2017. When possible, samples were taken and delivered to an accredited laboratory to assess the effluent water quality discharged to the receiving environment. These events are summarized in Table 12.

Table 12 - Overflow Events from 2013 to 2017

Date (yyyy-mm-dd)	Overflow Volume (m^3)	Duration (min)	TSS (mg/L)	$CBOD_5$ (mg/L)	Explanation
2013-03-18	33	10			Cleaning lift station, pump became air locked
2013-05-21*	449	310	84	72	Heavy rain
2013-05-22*	69	47	76	68	Heavy rain
2013-06-19*	1,064	205	112	53	Heavy rain
2013-06-19	384	170			Heavy rain
2013-06-20	830	342			Heavy rain
2013-07-17*	121	40	43	122	Heavy rain
2013-09-06	7	10			Heavy rain
2013-09-21	68	28			Heavy rain
2014-05-26*	32	20	141	110	Heavy rain
2014-06-27	22	30			Heavy rain
2014-09-03*	86	20	93	<10	Heavy rain

Date (yyyy-mm-dd)	Overflow Volume (m^3)	Duration (min)	TSS (mg/L)	$CBOD_5$ (mg/L)	Explanation
2014-12-11	41	65	102	86	Heavy rain and pump failure
2015-02-06	583	300			Storm event
2015-02-20	293	35	24	32	Power outage
2015-03-28	274	102			Storm event
2015-06-29	199	61			Storm event
2015-07-13	30	16	130	190	Storm event
2015-07-26	21	25	106	125	Storm event
2015-09-20	230	59	90	167	Storm event
2016-08-23*	160	58	126	235	Storm event
2016-09-17	32	23			Storm event
2017-03-18	1,692	80			Heavy rain and snow melt
2017-03-24	44	80	90	88	Heavy rain and snow melt
2017-04-07	28	28	111	92	Heavy rain
2017-08-13	87	25			Heavy rain

**Dates where values were measured and reported as total BOD_5*

The City of Nelson has an inflow and infiltration (I&I) management plan to reduce I&I in the sewer collection system.

4.6 Rotating Biological Contactors

Secondary treatment is provided by four RBCs installed in two parallel trains of two. These systems are simple and have media discs attached to a slowly rotating drum. A biofilm layer grows on the media discs which go through a cycle of being wetted, to provide nutrients to the microbes, and being exposed to air to provide oxygen for respiration. Accumulated biological growth gradually sloughs-off the media discs and is removed in the secondary clarifiers.

The RBCs have a rated hydraulic and organic loading at each stage of the RBC train. Each RBC train is comprised of two stages (i.e. two RBCs in series). The predesign contemplated that a third parallel RBC train would be required, to accommodate increased loads, after approximately ten years of operation. The RBC design values are from the 2004 predesign report (Urban Systems) are summarized in Table 13.

Table 13 - Existing RBC Design Capacity

Parameter	Units	Stage 1	Stage 2
Total Surface Area	m^2/d	30,000	36,000
Rated Hydraulic Capacity	m^3/d	7,300	7,300
Surface Loading Design Capacity	$m^3/m^2/d$	0.24	0.20
Organic Loading Design Capacity	kg/1000 m^2/d	29	12
Influent BOD_5 Concentration	mg/L	120	60
BOD_5 Removal	%	50%	55%
BOD_5 Load	kg/d	876	438
Effluent BOD_5 Concentration	mg/L	60	27

The comparison of the 2004 design loading of the RBC's is compared to the current loading in XXX

Table 14 - Summary of Current Loading on the RBCs

Parameter	Units	Design Capacity	Current Loading
Average Annual Daily Flow	m^3/d	7,300	5,500
Maximum Month Flow	m^3/d		9,900
Maximum Day Flow	m^3/d		14,600
Average $CBOD_5$ Load	kg/d	1,314	1,203

The RBCs are hydraulically overloaded during the current maximum month and maximum day flow (the maximum day flow is limited by the total EQ tank pump capacity of 12,100 m^3/d). It was assumed that the current $CBOD_5$ load of 1,850 kg/d is reduced 35% (1,203 kg/d) by the upstream primary clarifiers. The average $CBOD_5$ load is approaching the rated capacity of the RBCs. However, the summary in Table 14 does not reflect the variability in $CBOD_5$ loading from the collection system or from the upstream primary clarify performance (it was noted previously that there are instances where the primary clarifiers are hydraulically overloaded that would impact $CBOD_5$ loading on the RBCs).

The MWR does not specifically refer to RBCs but infers secondary treatment redundancy requirements of providing treatment for 75% of the design flow with the largest treatment model offline. The existing system is designed to attenuate the peak hour flow; therefore, the 75% of the maximum day flow apples (10,950 m^3/d). With one RBC train offline (3,650 m^3/d) there is insufficient capacity to meet the reliability category.

4.7 Secondary Clarifiers

The secondary clarifiers remove suspended solids from the secondary effluent before disinfection with UV, and discharge to the environment. There are two parallel rectangular secondary clarifiers with inclined plate settlers. The inclined plate settlers improve settling capacity by inducing a more laminar, upward plug-flow which allows for improved settling. Treated effluent passes upwards through the settling plates and over an effluent collection weir at the surface.

Settled solids are collected by a travelling cable-vac at the bottom of the tank. The original design intention was to pump the settled solids to either the equalization tanks or to the anaerobic digesters. However, the plant operators reported that this system stopped operating correctly shortly after commissioning and have modified the system so that almost all settled secondary solids are pumped to the headworks.

Scum that may float to the surface of the clarifiers is collected by a manually operated scum collection pipe. A small scum pump and holding tank collects the scum and pumps it to the headworks.

The plate settlers can accumulate a biological film and are equipped with an air sparge system to assist with cleaning during routine maintenance.

Inclined plate settlers can also be referred to as modified gravity clarifiers. From the 2004 predesign report, the surface loading rates for these types of systems range from 100 - 150 $m^3/m^2/d$. The average of 125 $m^3/m^2/d$ was used to estimate the capacity of the clarifiers and the results are summarized in Table 15.

Table 15 - Hydraulic Capacity of Existing Secondary Clarifiers

Parameter	Units	Value
Surface Area (per tank)	m^2	49
Surface Area Loading Rate	$m^3/m^2/d$	125
Hydraulic Capacity (per tank)	m^3/d	6,130
Number of Clarifiers		2
Total Clarifier Capacity	m^3/d	12,260

Table 16 summarizes the current hydraulic loading on the existing secondary clarifiers.

Table 16 - Secondary Clarifier Loading

Parameter	Units	Total Clarifier Capacity	Current Load
Average Annual Daily Flow	m^3/d	12,260	5,500
Maximum Month Flow	m^3/d		9,900
Maximum Day Flow	m^3/d		14,600

The secondary clarifiers are rated for the maximum EQ tank pump flow (with both pumps operating). It was previously noted that the existing EQ pump capacity is exceeded during the maximum day and their capacity should be increased. Therefore, the secondary clarifier capacity is exceeded at the maximum day flow.

During the site visit, it was reported by operators that at high flows they have difficulty achieving good settling in the secondary clarifiers. One explanation may be that water temperature affects the performance of the secondary clarifiers and their effective capacity may be less than the theoretical value calculated above when operating at during peak spring snow melt (I&I) when the effluent water temperature is cold.

4.8 UV Disinfection System

The UV system is installed in a concrete channel with two modules in series and disinfects the effluent stream prior to discharge. The process is designed so that one of the two UV models can treat a peak flow of 500 m^3/hr (12,000 m^3/d) with suspended solids less than 30 mg/L and the UV transmittance of the water greater than 60%. Each bank contains 48 UV lamps, each inside a quartz sleeve. These sleeves require frequent manual cleaning by the operators. The current maximum day flow is 14,600 m^3/d.

Under the MWR, the reliability category of the disinfection system must be able to treat 50% of the design flow (maximum day flow) with the largest reactor offline. Therefore, the current system does not meet the reliability category.

4.9 Anaerobic Digesters

Two high-rate, mesophilic anaerobic digesters with floating roofs are installed in series, each with a tank volume of 755 m^3. The first digester is heated and mechanically mixed. The second digester provides settling, decant of supernatant and removal of settled sludge. Because the second tank is used for settling it is not heated. Gas is collected at the top of both tanks and is used for heating with any excess gas being flared off.

Both digesters are equipped with mechanical mixers. Each mixer can be operated in both forward and reverse. The mixer in the first tank operates continuously while the mixer in the second digester is normally off.

Waste sludge from the primary clarifiers is pumped for approximately 8 hours during weekdays into the first digester. The mixed solids from the first tank are pumped to the second tank for settling. After settling, the thickened, digested solids are pumped to dewatering and disposal and the supernatant is pumped back to the equalization tanks. Figure 1 shows a typical process flow diagram for the anaerobic digesters.

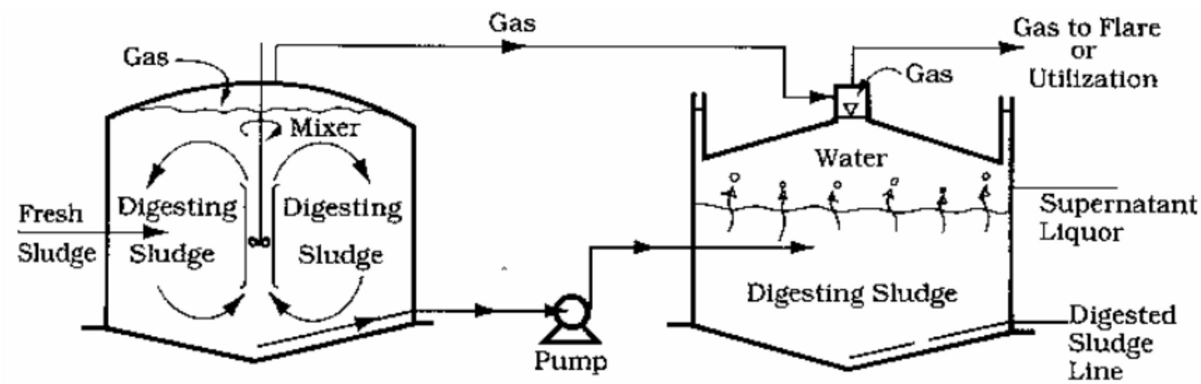


Figure 1 - Two-Stage Mesophilic Anaerobic Digester Process Flow Diagram

The digestion process is mesophilic with an operating temperature below 40ºC and is completely mixed. To estimate the process capacity, typical values for solids loading rates of 3.2 kg/m^3/d for the mixed and 1.1 kg/m^3/d for the unmixed digesters with a hydraulic retention of 21 days were selected. It was assumed that there was a 50% reduction of sludge volume in the second digester during settling. The dry solids loading differs between Tank 1 and Tank 2 due to two main factors: the first tank is completely mixed thereby improving the effective load, while tank two is not mixed and operates at reduced volume for solids separation. The calculations are summarized in Table 17.

Table 17 - Anaerobic Digester Capacity

Parameter	Units	Tank 1	Tank 2
Tank Volume	m^3	755	755
Solids Loading	kg/m^3/d	3.2	1.1
Dry Solids Load	kg/d	2420	790
Hydraulic Retention	d	21	21
Hydraulic Capacity	m^3/d	36	18

The calculated capacity is the sum of the two tanks for both hydraulic and solids loading capacity, which is 54 m^3/d and 3,210 kg/d respectively. The 2016 calculated average annual dry solids loading was 1,020 kg/d and a volumetric load of 25.5 m^3/d at 4% solids. The constraining parameter is the hydraulic loading rate which can be controlled by the feed solids concentration. Therefore, solids feed concentration should be monitored and not allowed to drop below 2.2% based on current average loading conditions.

The reliability requirement for anaerobic digesters under the MWR requires that with the largest reactor offline (tank 1) that the system be capable of treating 50% of the design flow (12.8 m^3/d at 4% solids). Based on a solid loading concentration of 4% and the current estimated loading, the existing system can meet the reliability category

4.10 Electrical Supply

From the 2005 secondary treatment upgrades, it was reported that the electrical service may be approaching its capacity. The report indicated that upgrades would be required in the future to accommodate additional electrical loads. Nelson Hydro spot tested the electrical amperage draw with a clamp-on amp meter on May 25th, 2017. They reported that there was a 126 A draw on the 200 A service under normal, spring-time operating conditions. It can be expected that the average amp draw would increase in winter when electrical heaters are operating. The maximum instantaneous amp draw upon start-up after a power outage is not known but could be close to the panel's rated capacity.

The backup generator is rated for 150 A and is a pinch-point during power outages. The operators reported that power outages occur frequently. Start-up of the plant would likely see instantaneous peak amp draws that would exceed the rated capacity of backup generator, to prevent this, equipment start-up is controlled sequentially following a power outage.

4.10.1 *Emergency Backup Generator*

The PCC has an emergency backup generator with an automatic transfer switch. During a power outage the generator will start automatically. A seven second delay allows the generator to reach full speed, then the following systems power up automatically:

- Heat and lights
- Equalization pump
- UV system
- Water pump
- Unit water pump
- Heat exchange pump
- Boilers
- One RBC
- Grinder
- Grinder auger

After the automatic starting systems are online, the operators can selectively start any of the following pieces of equipment that were deemed not essential for continuous operation during an outage:

- Recirculation pump
- Blower
- Grit washer
- Grit tank collector

The PCC operational software, called the human machine interface (HMI), will indicate what can and cannot be run. The operator may choose to turn off some equipment to free up additional capacity for other equipment. The existing generator is a Model 100DGDB that is 3 phase, 100 kW, 125 kVA, 469 volts and 150 amps.

5 STUDY DESIGN CRITERIA

A review of available data was undertaken to assess the current loading conditions and develop study design criteria. Data reviewed included: census population, hydraulic flows from the airport lift station, biological mass loading of $CBOD_5$ and TSS and effluent discharge requirements. For the purposes of this study a 20 year design horizon has been assumed. A 20 year design horizon is commonly used for municipal wastewater projects because growth beyond that horizon is difficult to predict and plan for, facilities are often amortized over twenty years, and much of the equipment in the facilities has approximately a 20 year life span.

5.1 Population

Population census data from Statistic Canada was acquired for 2006, 2011 and 2016. The City of Nelson's Official Community Plan (OCP) was referenced to assess the predicted population growth. The OCP suggests a population growth range of 1 to 2%. From that range, the middle population growth rate of 1.5% was selected to project the future population. The historic and forecasted growth rates are summarized in Table 18.

Table 18 - City of Nelson Population Summary

Year	Growth Rate	Population	Source
2006	-0.6%	9,258	Census
2011	2.0%	10,230	Census
2016	1.0%	10,752	Census
2017	1.5%	11,000	Projected
2027	1.5%	12,800	Projected
2037	1.5%	14,900	Projected

5.2 Hydraulic Loading

The average day dry weather flow and corresponding per capita flow rate of 510 L/c/d was calculated. The per capita flow rate is reasonable and is slightly under the Canadian averages as reported by Environment Canada's 2010 and 2011 Municipal Water Use Reports. Communities in British Columbia on average generate 629 L/c/d of sewage while communities in the 5,000 to 50,000 population range generate 704 L/c/d. The existing condition flows and corresponding peak factors are summarized in Table 19.

Table 19 - Design Flows and Peaking Factors

Flow Parameter	Design Flows (m^3/d)	Peaking Factor
Average Dry Weather Flow (ADWF)	5,400	-
Average Annual Daily Flow (AADF)	5,500	1.02
Average Day Wet Weather Flow (AWWF)	6,000	1.1
Maximum Month Flow (MMF)	9,900	1.8
Maximum Day Flow (MDF)	14,600	2.7
Peak Hour Flow (PHF)	20,100	3.7

The peak hour peaking factor corresponds well with typical range of 3.5 to 4 for North American communities of a similar size.

The per capita sewage flow rate and peaking factors were assumed to remain constant for future loads. The projected future ADWF was calculated using the per capita flow rate and a population growth of 1.5%. The other flow rates were derived from the projected ADWF multiplied by the peaking factors. The calculated current and future design flows are summarized in Table 20.

Table 20 - Hydraulic Design Parameters

Parameter	Units	Current 2017	Design 2037
Population	pop.	11,000	14,900
Per Capita Flow Rate	L/c/d	510	510
Average Dry Weather Flow (ADWF)	m^3/d	5,400	7,600
Average Annual Daily Flow (AADF)	m^3/d	5,500	7,800
Average Wet Weather Flow (AWWF)	m^3/d	6,000	8,500
Maximum Month Flow (MMF)	m^3/d	9,900	14,000
Maximum Day Flow (MDF)	m^3/d	14,600	20,600
Peak Hour Flow (PHF)	m^3/d	20,100	28,300

5.3 Biological Loading

To project future biological loading, the typical literature values of 70 g/c/d for both TSS and $CBOD_5$ were used to estimate the population contribution at a 1.5 % population growth rate. The ICI portion of $CBOD_5$ and TSS were projected forward at a 0%, 1.5% and 3% growth rate. The 20-year TSS and CBOD5 projections are summarized in Figures 2 and 3.

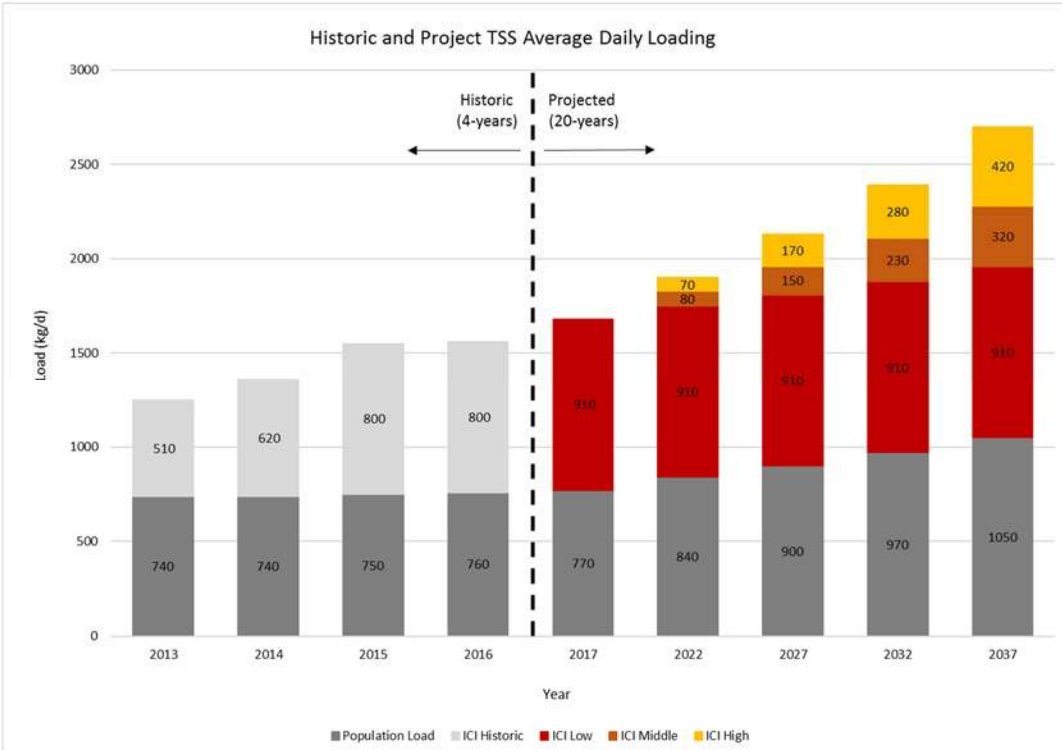


Figure 2 – Historic and Projected TSS Influent Load

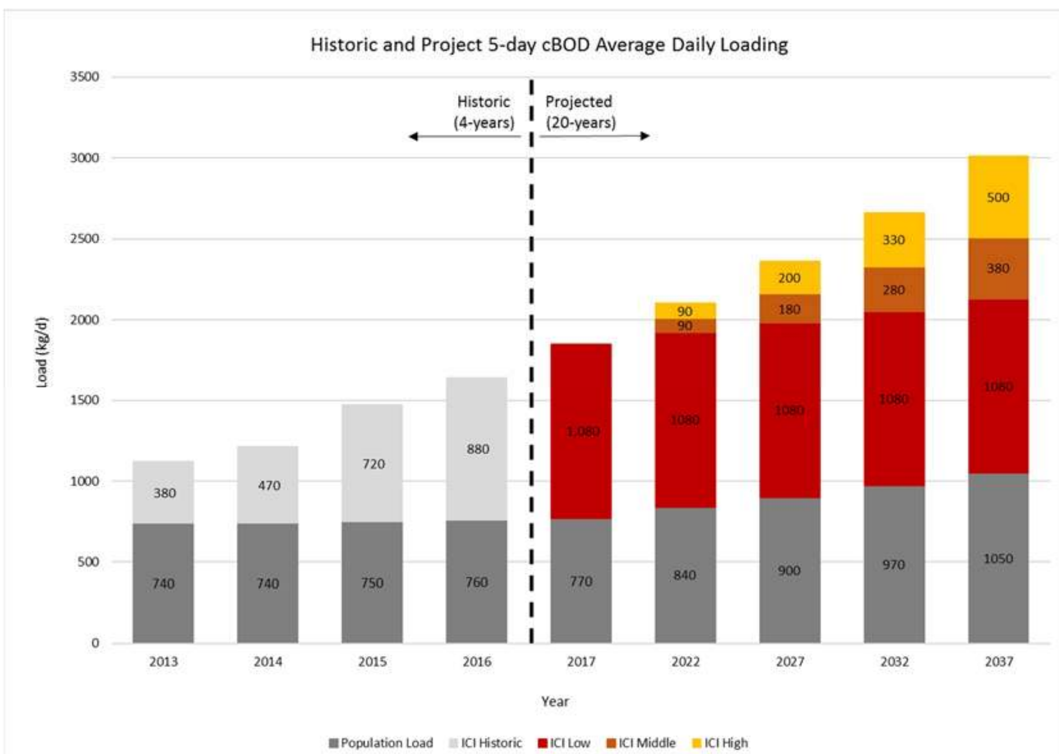


Figure 3 - Historic and Projected $CBOD_5$ Load

The mid-range load projections were selected for design which corresponds with a growth rate of 1.5% for the ICI load.

Two samples were collected on April 26 and May 10, 2017 for soluble BOD_5. From these samples, the soluble proportion of $CBOD_5$ was calculated as 35%. The design influent criteria calculated above are summarized in Table 21.

Table 21 - Design Load Criteria

Parameter	Units	Current	Design
		2017	2037
Population		11,000	14,900
$cBOD_5$	mg/L	336	322
$cBOD_5$*	kg/d	1,850	2,510
TSS	mg/L	305	292
TSS*	kg/d	1,680	2,280
Influent Temperature	°C	6	6
Soluble to $CBOD_5$ Ratio		0.35	0.35

**Calculated using average annual day flows*

5.4 Effluent Criteria

The Federal WSER applies to surface water discharge and is applicable for the Grohman Narrows outfall. The effluent discharge requirements under the WSER are currently more stringent than the MWR and therefore govern design criteria. Based on the information assessed in the 2014 environmental impact study, nutrient removal was not required and it is assumed that this will also continue to be the case in the future. However, this should be re-assessed at the time of preliminary design with a revised environmental assessment study (EIS). The effluent design criteria are summarized in
Table 22.

Table 22 - Effluent Design Criteria

Parameter	Units	Value
TSS (average)	mg/L	25
TSS (maximum)	mg/L	45
$CBOD_5$ (average)	mg/L	25
$CBOD_5$ (maximum)	mg/L	45
Ammonia (un-ionized)	mg/L	1.25

6 OPTIONS ASSESSMENT

Four secondary treatment technologies were assessed as potential options for upgrading the PCC. The four options considered are:

- Rotating Biological Contactors (RBC)
- Moving Bed Biofilm Reactors (MBBR)
- Complete Mixed Activated Sludge (CMAS)
- Sequencing Batch Reactors (SBR)

For the intent of this report, it has been assumed that the PCC would fall under reliability category 2 as defined in the MWR.

6.1 Preliminary Screening of the Options

The following will summarize the four processes selected, their footprint, capital cost, operating costs, and their ability to expand in the future.

6.1.1 *Rotating Biological Contactors*

The existing PCC has four RBCs that operate in two parallel trains of two. RBCs are composed of circular media disks attached to a rotating shaft. The media is designed to maximize the surface area for the growth of micro-organisms. The attached biological growth is cyclically rotated in-and-out of the wastewater influent stream; this keeps the biofilm saturated with nutrient rich primary effluent while providing oxygen exchange when in the ambient air.

The RBC technology has been around a long time, have a small footprint, are simple to operate, have low operating cost and low energy consumption. RBCs and other fixed film processes grow slowly and, therefore, do not respond quickly to rapid variations in load. Fixed film processes like RBCs are self-regulating with respect to biomass inventory because excess biomass sloughs off the disks naturally, however, the sludge that is produced is generally difficult to settle.

The design of the existing facility in 2004 contemplated a phased expansion of the RBC process to meet the 2025 capacity. Space is available north of the existing RBC building to add a third train to the existing RBC system.

Hannah Environmental Equipment, the supplier of the existing RBC system at the PCC, was contacted to assist in sizing an expanded RBC process (Appendix A). The initial calculations received from

Hannah determined the number of RBC trains that would be required at the design average day flow. Table 23 summarizes the design values from Hannah's calculations.

Table 23 - RBC Design Criteria

Parameter	Units	Value
Average Day Flow	m^3/d	8,500
$CBOD_5$ Load	mg/L	225
Influent Temperature	ºC	6
Target $CBOD_5$ Effluent	mg/L	15
Total Required Media	m^2	158,039
Number of Trains Required		6
Hydraulic Retention Time*	h	1.7

**Hydraulic retention time calculated to meet the MSR reliability category with the largest RBC train down at 75% of the design flow.*

From the Hannah proposal, six RBC trains (two RBCs per train) would be required to achieve the design effluent quality using the 2037 average day design loads. Further calculations were not performed to assess the design maximum day load because there is insufficient space at the PCC site to construct even the four RBC trains necessary for the average day load. Because there is insufficient space at the PCC to construct the necessary RBCs, this option was not pursued further.

6.1.2 *Moving Bed Biofilm Reactors*

Moving bed biofilm reactors (MBBR) are a fixed film process in which biomass is grown on small, lightweight plastic media wafers. Different manufacturers offer proprietary media with varying shapes and density. The biomass growing on the wafers self-regulate according to the amount of food available to the organisms. As more food is available, the biomass will increase to the practical limit that the media can carry. Excess solids slough off the media and are expelled with the treated effluent.

Coarse bubble diffusers provide the oxygen required for biological growth and treatment of the wastewater. Sizing of the aeration system is typically mixing limited because the media must be kept in suspension.

Fixed film processes, including MBBR, grow biomass slowly and do not respond quickly to rapid changes in biological loading, preferring steady, uniform conditions. However, they do respond well to changes in flow. Because fixed film processes self-regulate their biomass inventory, process control is simple, but there is no operational ability to manage the sludge age or solids inventory. The solids produced are generally light and difficult to settle in standard clarifiers, consequently other methods such as dissolved air flotation thickeners (DAFT) are preferred for phase separation with MBBRs.

MBBR technology was developed to be a compact biological treatment process that can be constructed indoors to reduce heat loss in cold weather climates or used to retrofit existing facilities. While the technology can be constructed indoors, there are many installations worldwide that are installed outside in uncovered tanks. The high surface area of the wafer media allows for a greater mass of microbes as compared to conventional activated sludge. More microbes in a smaller space allows for greater treatment capacity per unit of volume.

Veolia was approached to submit a proposal of their MBBR technology. The proposal from Veolia suggested using a process combination of their AnoxKaldnes MBBR and Spidflo® dissolved air floatation (DAF) technologies. The proposal assessed two possible retrofit scenarios as summarized in Table 24.

Table 24 - MBBR Scenarios

Parameter	Scenario 1	Scenario 2
Construct Additional Equalization Tanks	No	No
Provide Primary Clarification Provided Using New Mechanical Primary Screens	No	Yes
Convert Existing Primary Clarifiers to MBBR	Yes	Yes
Convert Existing EQ Tanks to MBBR	Yes	Yes
Convert Existing RBC Tanks to MBBR	No	Yes
Construct Additional Tankage for MBBR	Yes	No

In scenario 2, Veolia proposed using one Hydrotech drum filter to provide mechanical primary filtration. Without the use of polymer, the drum filter could remove 50% of TSS and 25% of BOD_5. With polymer the removal of both TSS and BOD_5 can be increased significantly.

The MBBR tanks were sized to meet the MWR reliability criteria of one tank down at 75% of the design flow. The design values, hydraulic retention time, media fill and tank sizes for the two different scenarios are summarized in Table 25.

Table 25 - MBBR Design Criteria

Parameter	Units	Scenario 1	Scenario 2
Maximum Monthly Flow (MMF)	m^3/d	14,000	14,000
Design Flow (MMF x 75%)	m^3/d	10,500	10,500

URBAN systems

Parameter	Units	Scenario 1	Scenario 2
Total Reactor Volume	m^3	1,720	1,140
HRT (at average daily flow)	h	5.3	3.5
Number of trains		2	2
Media fill	%	60	60
Equipment Cost	$	$3,200,000	$3,160,000

Under scenario 1, the calculated total volume available in the primary clarifiers and equalization tanks is 806 m^3; therefore, an additional 914 m^3 of new tankage would be required. Any new tankage could be constructed adjacent to the existing RBC facility. To convey screened sewage to this location, a pump station would be required after the aerated grit tank.

Under scenario 2, the RBC tanks could also be retrofitted to MBBR and no new tankage would be required; however, a new primary filter building and pump station would be required. No new tankage is required because the mechanical primary filters remove $CBOD_5$ from the process reducing load on the MBBRs.

Both of Veolia's proposed options include their proprietary Spidflo® process: a dissolved air floatation (DAF) process. The Spidflo® process differs from traditional DAF technologies by using pumps and venturis to induce entrained air, instead of compressors. Other similar technologies are available that would be comparable.

DAF processes work by providing micro air bubbles that capture and rapidly transport suspended solids to the water surface. Influent water enters the Spidflo® and flows upward with the injected air in the dispersion water injection zone. The floating scum, that looks like a thick brown foam, is then removed by a mechanical skimmer for disposal. The percent solids concentration from the DAF is expected to be 3-4% on average, which is a good concentration for feeding to the anaerobic digester. Clarified effluent is collected in an underdrain system and directed to UV disinfection. Figure 4 shows the Spidflo® process and is from the Veolia's proposal attached in Appendix B.

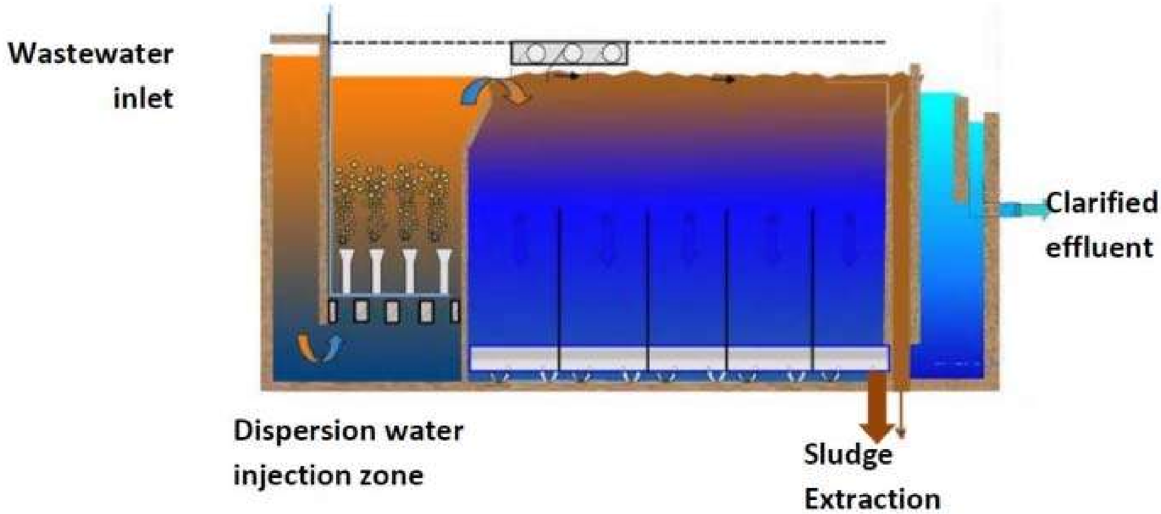


Figure 4 - Spidflo® Process Diagram from Veolia

The benefits of a DAF process include a small foot print and a higher sludge concentration appropriate for feeding to the anaerobic digester. However, the downsides include higher capital costs, increased operational requirements and higher electricity consumption.

The combination of difficult to settle solids, small available footprint and ability to retrofit into the existing building, supports the use of this process.
The supply of equipment from Veolia for both scenarios and their costs are:

1. Retrofit existing tanks (no primary treatment or equalization)......................$3,200,000
    a. Aeration system including blowers
    b. Media and retaining screens
    c. DAF with pumps and polymer dosing system

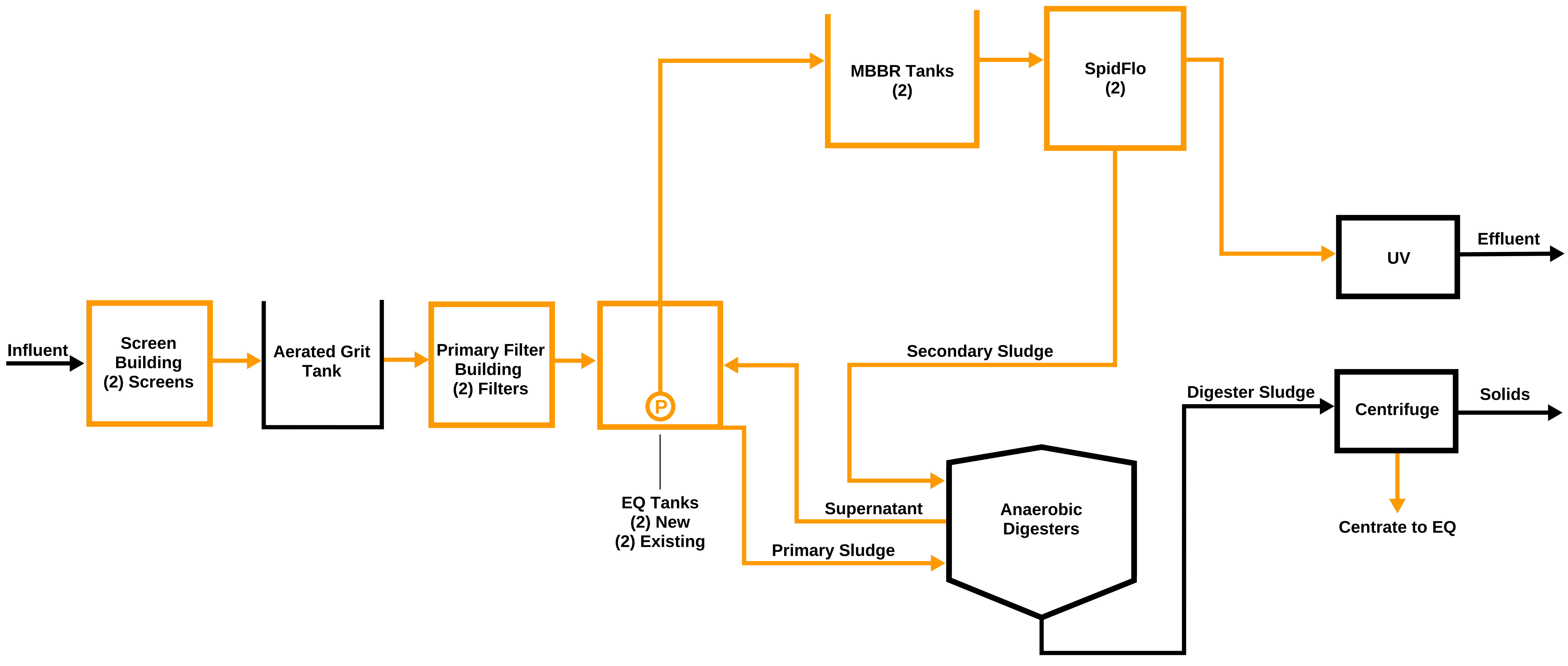
2. New tankage and new primary treatment system $3,160,000
    a. Aeration system including blowers
    b. Media and retaining screens
    c. DAF with pumps and polymer dosing system
    d. One primary filter

The price for primary filtration equipment is offset by the reduction in equipment required for the MBBR process. Both MBBR options would also require 6 mm fine screens installed in the headworks. This is to prevent the accumulation of inorganic materials such as plastics, grit and sand that will cause damage aeration system and media.

Continuing to provide primary treatment to feed the anaerobic digesters simplifies the change in processes. Anaerobic digesters are intended to treat raw primary sludge, and are capable of treating mixed sludge that contains secondary, biological solids (Metcalf & Eddy). Without primary treatment, the anaerobic digesters may not operate as intended and would likely need to be decommissioned or converted to aerobic digesters. Without digesting the solids, there would be an increase in solids production that would impact the existing dewatering facility's capacity and the costs associated with solids disposal. Assessing the impacts of removing primary clarification on the anaerobic digesters is outside the scope of this report. Therefore, given that the price of equipment is approximately the same for both scenarios and that there are added complications with managing sludge should the anaerobic digesters be decommissioned, the ability to provide primary treatment and maintain the operation of the digesters under scenario 2 makes this the preferred option.

A conceptual layout is shown in Figure 5. Based on the conceptual layout, MBBR technology is anticipated to be relatively straight forward to integrate into the existing PCC. Adequate space is available to construct the necessary process facilities while maintaining the RBC process during construction. MBBR media can be added in stages as loads change over time, this allows some financial flexibility. A conceptual process flow diagram is also shown in Figure 6.

Drawbacks to an MBBR process include expected high capital cost, a significant increase in electrical usage and a modest increase in operating requirements.



Influent
Screen Building (2) Screens
Aerated Grit Tank
Primary Filter Building (2) Filters
P
EQ Tanks (2) New (2) Existing
MBBR Tanks (2)
SpidFlo (2)
UV
Effluent
Secondary Sludge
Digester Sludge
Centrifuge
Solids
Supernatant
Primary Sludge
Anaerobic Digesters
Centrate to EQ

6.1.3 Complete Mixed Activated Sludge (CMAS)

CMAS is a common treatment process at many municipal wastewater treatment plants. In a CMAS process, aeration is provided to homogeneously mix and aerate tanks with mixed liquor suspended solids (MLSS) to consume BOD. This process requires a separate solids separation phase, often completed in circular secondary clarifiers. Activated sludge is recycled in the process to maintain a high biomass concentration relative to the food entering the system. The high biomass consumes the sewage and oxygen rapidly to provide a high rate system.

Two CMAS scenarios were assessed:

1. Extended aeration with primary clarification.
2. Conventional complete mixed aeration with primary clarification.

The two options were assessed at using the 2037 maximum month flow rate and the MWR reliability requirement of 75% flow with the largest reactor off-line. The primary clarifiers were assumed to remove 30% of the total BOD_5, in particulate form. Cold water temperatures control the biological metabolic process and was assessed at the design value of 6 ºC. A hydraulic retention time between 20 and 30 hours was targeted for the extended aeration process and 3 to 6 hours for the conventional process. A sludge retention time was targeted between 20 to 40 days for the extended aeration process and 3 to 15 days for the conventional system. Table 26 summarizes and compares the design values for both the extended aeration and conventional activated sludge process calculations.

Table 26 - CMAS Design Criteria

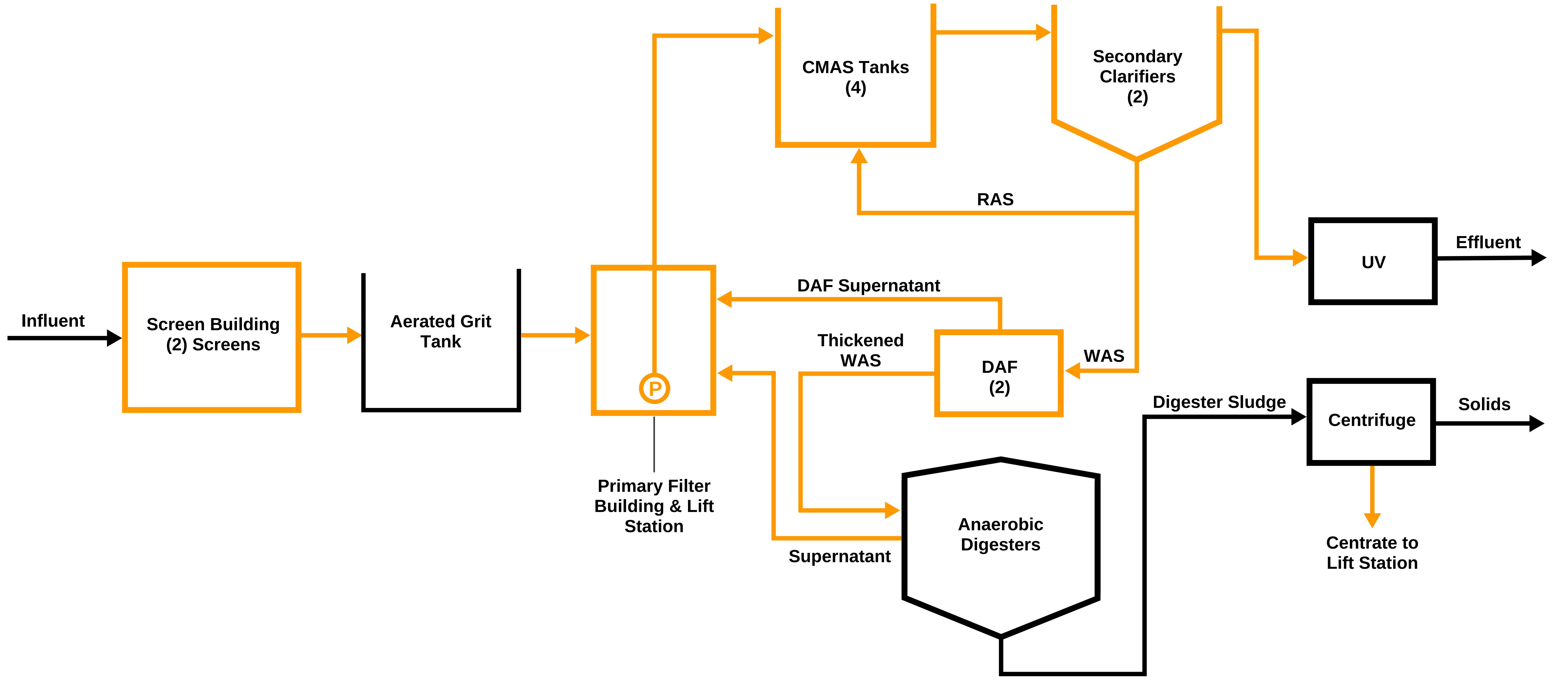
Parameter	Units	Extended Aeration	Conventional
Max Month Flow*	m^3/d	10,500	10,500
BOD_5 Load	kg/d	1,577	1,577
Temperature	ºC	6	6
HRT	h	21	3.7
SRT	d	14	3
MLSS	mg/L	2,680	2,920
Number of tanks		3	3
Tank Volume (ea)	m^3	3,060	540
Tank Depth	m	6	6

**Hydraulic loading assessed using the MWR reliability of 75% flow with largest reactor done*

The extended aeration scenario tank size would exceed the space available at the site, as depicted in Figure 8. Because of this, the scenario was determined to not be feasible.

The conventional CMAS process with primary treatment and equalization is potentially feasible. However, the overall footprint required allows for little to no expansion beyond the 20-year design horizon. Comparison between the extended aeration and conventional activated sludge was done at the maximum month loading.

This process type would require a significant increase in both labour and electricity. Labour costs are higher for this process because of the control requirements for sludge age and MLSS. Due to the significant footprint required, integration into the existing PCC would be complicated with higher anticipated capital cost; this is demonstrated in the conceptual layout shown in Figure 8. A conventional CMAS process flow diagram is shown in Figure 7.



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Scale	**Date**	**Figure**
NTS	2017-10-14	8
Title		
CMAS Process Flow Diagram		

6.1.4 *Sequencing Batch Reactors (SBR)*

There are several types of SBR processes. For the purposes of this report we have selected the intermittent cycle extended aeration system (ICEAS) from Sanitaire, who submitted a proposal (Appendix C). This process allows for the potential re-use and retrofit of the existing primary clarifiers and equalization tanks. The SBR process is similar to activated sludge process, but by cycling the processes the treatment and settling processes are accomplished in the same tanks and no separate clarifiers are required.

The ICEAS process is a batch process that combines secondary treatment with clarification in a common tank. Flow of influent occurs continuously while decanting of treated effluent occurs sequentially from each individual basin. In the ICEAS process there are three distinct process time periods with the following typical process durations:

- Filling and aeration (120 minutes)
- Settling (60 minutes)
- Decant (60 minutes) combined with sludge removal (2-5 minutes)

Each basin operates in staggered 60-minute intervals, when there are an uneven number of basins (more or less than a multiple of four) then downstream flows fluctuate significantly. Due to this batch decant process, downstream equalization is required to attenuate flows from multiple basins. The equalization allows for the controlled, continuous flow to the UV system which reduces the size of the UV system, reduces on/off cycles and undesirable surges.

The SBRs were sized using the 2037 maximum month flow with the MWR reliability requirement of 75% of the design flow with the largest reactor down. The SBR option was assessed under two scenarios:

1. No primary treatment or equalization and converting the existing tanks to SBR.
2. Providing primary treatment with equalization and constructing new SBR process tanks.

Upon review of the first option, it was clear that it would not be feasible as there was insufficient space for new basins to meet the 20-year design horizon.

The second option provides a more compact process layout that may be marginally feasible. The process footprint is reduced 30% by providing primary treatment. The SBR tank design information is summarized in Table 27.

Table 27 - SBR Design

Parameter	Units	Scenario 1	Scenario 2
Number of Basins		5*	6
Water Depth	m	5	6.5
Length	m	26	21
Width	m	9	7.6
Surface Area (per basin)	m^2	234	160
Total Surface Area	m^2	1,170	960
Process Air (per basin)	m^3/h	1,910	1,080
Equipment Cost	$	$1,962,000	$1,365,000

**Retrofit of primary and EQ tanks not included in tank count*

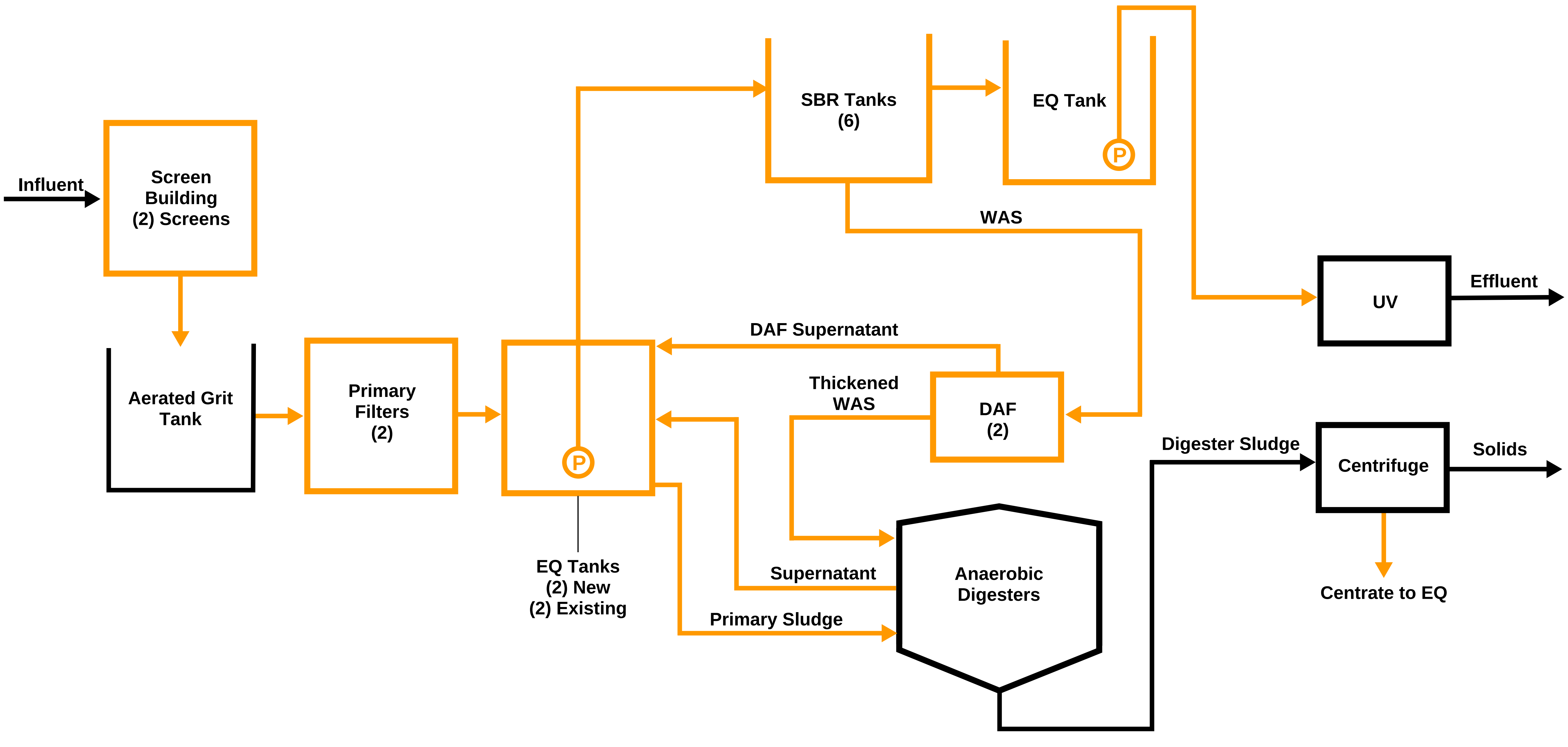
From the SBR proposal, four process tanks must be constructed immediately to meet the current design load and two additional tanks would be required to meet the 20-year design. Constructing an SBR process would be complicated because the space occupied by the existing RBC building is required for new tankage. This would make demolishing the RBC building while maintaining effluent discharge quality during construction challenging and costly, if feasible at all.

UV systems prefer continuous, steady flow rate to prevent problems such as on/off cycling and hydraulic surges that would otherwise reduce the life expectancy of the equipment. Since the SBR process operates on 60-minute cycles and there are 6 tanks, there will be two cycles with a flow of 6 m^3/minute and two cycles with 12 m^3/minute. The equalization tank would be sized to accommodate a storm event with two SBRs decanting. A cost analysis could be performed at preliminary design to determine the most cost-effective solution, but for the purposes of this report, and to be conservative, it will be assumed that an equalization tank will be constructed.

While the SBR process has a built-in solids separation phase, the concentration of secondary solids is anticipated to be thin (approximately 1%) and would require thickening to prevent the anaerobic digesters from being hydraulically overloaded. A DAF is suggested to thicken the waste activated sludge (WAS) to 4% for digester feeding. DAFs require a small footprint and could be retrofitted into the existing secondary clarifier building. The existing clarifier tank could also be retrofitted to equalize WAS for a constant feed. The downside of DAFs includes high energy use and chemical makeup and dosing systems during peak loads.

The SBR scenario process flow diagram and conceptual layout are shown in Figure 9 and Figure 10.

The benefits of an SBR process include relatively simple operation and a combined settling process. The downsides include complicated and costly integration due to the large footprint, no ability to expand beyond the 20-year design horizon, a moderate increase in labour and a significant increase in electrical consumption.



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Title

SBR Process Flow Diagram

6.1.5 *Process Screening*

A simple decision matrix was developed (Table 28) to summarize the preliminary assessments of the four options. The purpose of the matrix is to provide an assessment of the four options that consider other decision-making parameters that cannot be immediately quantified in dollars. A qualitative value corresponding to a 1, 2 or 3 was assigned to each of five assessment criteria. The higher the value assigned, the more favourable it is.

Table 28 - Process Comparison Matrix

Parameter	RBC		MBBR		CMAS		SBR	
Footprint	Excessive	0	Small	3	Excessive	0	Large	1
Capital Cost	Low	3	High	1	High	1	Moderate	2
O&M Cost	Low	3	Moderate	2	High	1	Moderate	2
Process Integration	Poor	1	Good	3	Poor	1	Moderate	1
Meets Design Loading	No	0	Yes	3	No	0	Yes	3
Ability to Expand	Unable	0	Good	3	Unable	0	Unable	0
Score		**7**		**15**		**3**		**9**

This assessment suggests that the MBBR process is the best solution. The SBR process is the second-best option, but when directly compared to the MBBR, it is clear that that the physical footprint and retrofit requirements would be capitally intensive. Therefore, the SBR, RBC and CMAS options are ruled out at this stage and the MBBR option is carried forward for further assessment.

6.2 Retrofit Using Moving Bed Biofilm Reactors

The MBBR process with primary treatment was selected for further concept development. Due to site constraints, this was the only feasible option that had been reviewed. To implement MBBR, several supporting process upgrades are required. The conceptual layout presented in Figure 5 includes the following retrofit and supporting process upgrades, listed in order of required construction:

1. Upgrade electrical service and emergency backup generator.
2. Construct a new headworks with 6 mm fine screens.
3. Construct a new mechanical primary filter building with wet well and pumps.
4. Converting the existing primary clarifiers, EQ tanks and RBC tanks to MBBR.
5. Convert the secondary clarifiers to the Spidflo® DAF process, or equivalent.

Each of the proposed upgrades can be completed individually as funding is available, beginning with the electrical upgrades and headworks.

6.2.1 Electrical Service Upgrade and Emergency Power

The existing PCC currently has a 200 amp, 480 volt, 3-phase service with a 150 amp emergency backup generator. Increasing the plant treatment capacity will result in additional electrical consumption that will require both an upgrade of the existing service from 200 amps to 300 or 400 amps and upgrading the emergency backup generator accordingly.

The original motor control centres, commonly referred to as MCCs, are old and were previously exposed to chlorine gas, which has resulted in ongoing failures and repairs. The replacement of the original MCCs is strongly recommended to ensure reliability of the process.

Modern electrical codes in Canada, such as the Canadian Standards Association (CSA), have standardized electrical services as being 600 volts. Equipment would have to be obtained from the United States to continue to use 480 volts. Emergency repairs of 480 volt rated equipment that is not kept in inventory by the City would likely take longer than equipment stocked in Canada.

It is recommended that the building's electrical service and backup generator be upgraded to increase its capacity and provide 600-volts, 3-phase. A 460 volt stepdown transfer will be added to service the existing 480 volt equipment. This would begin the conversion of modernization of the electrical equipment from 480 volt to 600 volt in a gradual manner. This upgrade would be required at the same time as the secondary treatment process upgrade.

Operations and Maintenance Costs

No additional O&M costs are expected and that the existing cost to service the current backup generator are expected to be the similar.

Class "D" Cost Estimate

A Class "D" Cost Estimate to purchase and install a new generator and replace the MCCs was assessed with allowances for 15% Engineering and 35% Contingency. The capital cost includes the supply, install and construction of the following major items:

- Emergency Generator..$240,000
- Electrical Upgrades..$300,000
    - Upgrade electrical service and transformers to the PCC
    - Supply and install replacement MCCs

- Supply and install a 480 volt step transformer

6.2.2 *Headworks*

Fine screening and grit removal is required for the mechanical primary filters and the MBBR process. The existing grit removal system was assessed and should meet the long-term needs of the PCC. However, the existing mechanical screen is hydraulically overloaded, subject to freezing and additional reliability is strongly recommended to protect future downstream equipment (i.e. mechanical primary screen and MBBR). Upgrades to the headworks is required prior to the implementation of mechanical primary clarification and MBBR.

Influent to the PCC is pumped from the airport lift station. Because of this, the size of solids received at the headworks would be less than 75 mm; therefore, coarse screening to protect the fine screens is not be required.

A budgetary proposal to provide two 6 mm perforated plate automatic screens with a common wash presses was obtained. There are many types of screening technologies available on the market and the 6 mm perforated plate was selected for its high solids capture rate. Each screen is rated for the 20-year design peak hour flow thereby providing 100% screening redundancy. Screen redundancy is not required by regulation; however, screening is important for proper operation of MBBR's so redundancy is strongly recommended. The addition of the wash-press to the mechanical screen will generate a screened solids product that is cleaner, has reduced odours and easier to for the operators to handle.

Conceptual implementation would see the forcemain re-aligned and discharge into the new headworks building, located adjacent to, and east, of the existing headworks (currently where the screenings and grit solids bins are located). This location should allow for the construction of the new screening building while maintaining influent flows, it would also allow for a relatively easy transition of a re-aligned forcemain.

The building would contain two concrete channels that would split to isolate the flow between the two mechanical screens. Solids bins would be located in the room with access to the north for truck pick-up and disposal at the landfill. Screened sewage would then flow by gravity to the existing aerated grit tank by modifying the existing headworks channel.

Operations and Maintenance Costs

This proposed upgrade would provide one additional mechanical screen and one wash-press. Labour costs for major annual maintenance, such as inspection, cleaning and lubrication were estimated at 8 person-hours per piece of additional equipment. This results in an additional $2,000 per year in labour.

Class "D" Cost Estimate

A Class "D" Cost Estimate for a new headworks with allowances for 15% Engineering and 35% Contingency was assessed. The capital cost includes the supply, install and construction of the following major items:

- Headworks Building..$3,300,000
  - Construct new headworks building
  - Supply and install two 6 mm mechanical screens
  - Install one common wash-press
  - Re-align forcemain
  - Modify and connect to existing headworks channels

6.2.3 *Primary Treatment*

The existing primary clarifiers capacity was calculated earlier in the report.
Table 29 compares the design flows to the primary clarifier capacity.

Table 29 - Comparison of Primary Clarify Capacity and Design Flows

Parameter	Units	Primary Clarifier Capacity	Current Flows	Design Flows
Average Annual Daily Flow	m^3/d	6,780	5,600	7,800
Average Day Dry Weather Flow	m^3/d	-	5,500	7,600
Average Day Wet Weather Flow	m^3/d	-	5,900	8,200
Maximum Month Flow	m^3/d	-	9,900	13,700
Maximum Day Flow	m^3/d	-	14,600	20,200
Peak Hour Flow	m^3/d	17,000	20,100	27,800

The comparison indicates that the primary clarifiers are overloaded at the current peak hour flow and their process performance will likely deteriorate as hydraulic loads increase in the future.

Primary treatment is recommended to reduce the overall MBBR process footprint which helps to maintain space on the site for expansion beyond the 20-year design horizon. The existing primary clarifiers also provide the necessary feed sludge for the operation of the anaerobic digesters. Anaerobic digestion operates optimally with primary sludge only, and process efficiency decreases when secondary, biological solids are added. If primary clarification is not provided, the anaerobic digesters may no longer operate properly and may be better suited as aerobic digesters to treat secondary, biological solids (Metcalf & Eddy). However, assessing aerobic digestion is outside the scope of this report.

Because of their small footprint, it is recommended that mechanical primary filtration be implemented. However, before this process can be implemented, the headworks upgrade is required to protect the filter equipment. Primary filters require good, reliable screening and grit removal upstream to protect the equipment.

A new primary treatment building would be constructed adjacent to anaerobic digester #2. Veolia proposed using the Hydrotech HDF2010 to meet the 20-year design but there are a number of other manufacturers of this type of technology that could also be used. The estimated space requirements for two, parallel Hydrotech filters with 100% redundancy at peak hour flows is 100 m^2.

The aerated grit tank effluent channel would be modified to re-direct flow to the new primary filter building. Flow would be conveyed by concrete channel to the filters and polymer could be added upstream to assist with TSS and $CBOD_5$ removal. Veolia's proposal anticipated 50% removal of TSS and 25% removal of $CBOD_5$ without polymer addition.

The filters use an integrated pump-wash system that uses filtered primary sewage to wash itself. The solids are collected at a concentration of 3-4%, suitable for loading of the anaerobic digester, and would be pumped to the digesters for treatment.

Operations and Maintenance Costs

The daily operational time required for the primary filters was estimated to be half an additional person-hour per day to perform checks and process control. Other person-hour increases are associated with major maintenance of mechanical equipment such as four-hours per pump and motor per year and a full day annual inspection, cleaning, lubricating and testing.

Electrical consumption is increased with the addition of two 15 hp backwash pumps, two 5 hp filter motors, both estimated to operate eight hours per day, and three 10 hp low-lift submersible pumps in a two-duty, one-standby configuration.

Chemical use to assist with filter operation is estimated assuming continuous dosing of 2 g of polymer per kg of solids.

Table 30 summarizes the calculated annual increase in operating and maintenance costs.

Table 30 - Primary Filter Major O&M

Operations and Maintenance	Annual Cost
Labour[1]	$17,000.00
Electricity[2]	$22,000.00
Chemical[3]	$8,000.00
TOTAL	**$47,000.00**

[1] *Labour costs were assessed at $65/h*
[2] *Electricity costs were assessed at $0.0973/kW.*
[3] *Chemical dose was calculated at 2 g polymer per kg of solids*

Class "D" Cost Estimate

A Class "D" Cost Estimate for a new primary filter building with allowances for 15% Engineering and 35% Contingency was assessed. The capital cost includes the supply, install and construction of the following major items:

- Primary Filter Building..$4,700,000
    - Filter building with wet well
    - Two primary filters
    - Polymer dosing system
    - Sludge pumps
    - Primary sewage pumps
    - Modifications to existing channels

6.2.4 *Moving Bed Biofilm Reactor*

To convert to the MBBR process, the existing two primary clarifiers, both EQ tanks and RBC tanks will need to be retrofitted. This retrofit would include the supply and installation of aeration systems, media retaining screens, MBBR media, blowers and additional pumps. The process air is proposed to be by two 30 hp blowers in a duty/standby configuration.

Previous process upgrades would have made the primary clarifier tanks obsolete and these tanks would be empty and unused. The chain and scraper system, effluent trough and other miscellaneous mechanical components from the primary clarifiers would be removed. An effluent wall would be constructed at the end and used to fasten media retaining screens and create an effluent wet well. It is assumed that the two existing EQ pumps would be reused and additional pumps added as needed. Once these two MBBRs are commissioned, along with at least one DAF, the EQ tank and RBCs can be decommissioned and retrofitted to MBBR in the same manner.

The media percent fill is an important phasing consideration. The proposal has a media fill of 60% at 2037 design loading. The maximum media fill volume is 60% to 65%, this indicates that at the 20-year design the basins will be close to, or at, the maximum fill volume. However, at the onset of construction a lower percent fill can be installed with additional media added as in stages as needed. The cost of the media is significant and phasing it can reduce the initial capital costs. Detailed phasing options could be looked at in more detail at preliminary design.

Operations and Maintenance Costs

It is anticipated that there will be some increases in O&M costs when switching to an MBBR process. Pumping to the DAF from the retrofitted primary clarifiers and EQ tank will be required; however, the existing facility already uses pumps and no significant increase in electrical usage is anticipated. The largest increase to operating costs will be electricity for the aeration system. Annual electric costs were estimated for one 30 hp blower operating continuously. Also, there will be some additional labour to maintain the blowers and additional effluent pumps, this was estimated at 4 person-hours per piece of equipment to inspect, clean and lubricate annually.

Table 31 - MBBR O&M Costs

Operations and Maintenance	Annual Cost
Labour[1]	\$2,000.00
Electricity[2]	\$20,000.00
TOTAL	**\$22,000.00**

[1] Labour costs were assessed at \$65/h

[2] Electricity costs were assessed at \$0.0973/kW.

Class "D" Cost Estimate

A Class "D" Cost Estimate to retrofit the existing primary clarifiers, EQ tank and RBCs was assessed with allowances for 15% Engineering and 35% Contingency. The capital cost includes the supply, install and construction of the following major items:

- Retrofit to MBBR...\$5,000,000
  - Remove and dispose of RBC, EQ and primary clarifier equipment.
  - Modify existing EQ and primary tanks for effluent pumping.
  - Reuse and install existing EQ pumps and purchase and install additional pumps.

- Supply and install aeration grids and blowers (one-duty and one-standby).
- Supply and install all of the MBBR media.

6.2.5 *Secondary Clarification*

The existing secondary clarifier capacity was calculated earlier in the report. Table 32 compares the design flows to the primary clarifier capacity.

Table 32 - Comparison of Secondary Clarifier Capacity with Design Flows

Parameter	Units	Secondary Clarifier Capacity	Current Flows	Design Flows
Average Annual Daily Flow	m^3/d	12,260	5,600	7,800
Average Day Dry Weather Flow	m^3/d	-	5,500	7,600
Average Day Wet Weather Flow	m^3/d	-	5,900	8,200
Maximum Month Flow	m^3/d	-	9,900	13,700
Maximum Day Flow	m^3/d	-	14,600	20,200
Peak Hour Flow	m^3/d	-	20,100	27,800

The existing inclined-plate, secondary clarifiers were calculated to be overloaded at current maximum day flows, and effluent quality will deteriorate as hydraulic loads increase. The operators have also indicated that they are having difficulty maintaining effluent water quality at elevated flows and are expecting to have to use chemical polymer to assist the process. The sludge from the MBBR will be a fixed film that is anticipated to be difficult to settle. The additional mechanical shear of the process aeration and physical collisions of the media in the MBBR tanks is anticipated to produce biomass particles that are finer than the existing RBC process. These smaller particles will likely result in further settling difficulties during secondary clarification.

The loading rate of the DAF, relative to the existing secondary clarifiers, is 720 m/d compared to 125 m/d respectively, this results in a much more compact process. **Table 33** summarizes the proposed Spidflo® design criteria.

Table 33 – Spidflo® Design

Parameter	Units	Value
Dry Solids Load	kg/d	1,890
Dry Solids Concentration	%	4%
Surface Loading Rate	m/d	720
Peak Loading Rate	m/d	1,200

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The DAF is required to be installed at the same time as the MBBRs. One secondary clarifier would be taken offline and retrofitted at a time. With one secondary clarifier down, it would be recommended to perform this work during the low flow season. Coordination would be required with the operators as chemical conditioning for the one clarifier will be required to improve settling performance.

Once the DAF is commissioned it can begin receiving treated effluent from the first two MBBRs. Once the new process has stabilized, the second clarifier can be decommissioned and retrofitted with a second DAF to provide redundancy.

Operations and Maintenance Costs

The daily operational time required for the Spidflo® would be comparable to other DAF technologies, and was estimated to be half a person-hour per day to perform checks and process control. Other person-hour increases are associated with major maintenance of mechanical equipment such as four-hours per pump and motor per year and a full day annual inspection, cleaning, lubricating and testing.

Electricity increases are the result of two 15 hp pumps, one dedicated to each DAF. It was estimated that one DAF would operate continuously while the second operated during spring time peak flows.

Chemical use to assist with solids separation is estimated conservatively by assuming continuous dosing of 2 g of polymer per kg of solids. Chemical use may not be required initially and the equipment and chemicals could be added at a later date. Table 34 summarizes the calculated annual increase in operating and maintenance costs.

Table 34 – Spidflo® O&M Costs

Operations and Maintenance	Annual Cost
Labour[1]	$23,000.00
Electricity[2]	$20,000.00
Chemical[3]	$15,000.00
TOTAL	**$58,000.00**

[1] Labour costs were assessed at $65/h

[2] Electricity costs were assessed at $0.0973/kW.

[3] Chemical dose was calculated at 2 g polymer per kg of solids

Class "D" Cost Estimate

A Class "D" Cost Estimate to retrofit the secondary clarifiers to the Spidflo® process with allowances for 15% Engineering and 35% Contingency was assessed. The capital cost includes the supply, install and construction of the following major items:

- Retrofit Secondary Clarifiers..$5,300,000
  - Create opening in existing secondary clarifier building wall.
  - Remove existing secondary clarifier equipment.
  - Supply and install Spidflo® equipment in existing clarifier tanks.
  - Modify and connect existing piping and channels.

6.2.6 *UV System*

The existing UV system has a rated capacity of 12,000 m^3/d with one of the two modules down and does not meet the MWR reliability requirements at the current maximum day flow of 14,600 m^3/day. An upgrade to the UV system is recommended in the future to increase its capacity to meet the 20-year design max day flow of 20,600 m^3/d. During the site investigation operators reported that the existing system requires frequent, manual cleaning.

Trojan submitted a proposal to upgrade the UV system to meet the design maximum day flow. For the reliability category II in the MWR, the redundancy requirement is 50% of the peak design flow with the largest UV bank offline. The proposal suggested replacing the existing system with their TrojanUV3000plus which would include two new banks (and removing the old UV units) and would fit into the existing channel. The revised system will have flow paced dosing, a higher intensity and would operate at a UV transmittance of 55%. For reference, the current system was designed with a transmittance of 60% or greater and lowering this to 55% will improve performance. The system would also have an automatic cleaning system that would reduce labour costs and improve reliability and operations.

Operations and Maintenance Costs

With the addition of the automatic wiping system we estimated that there would be a labour savings of two person-hours per month. The existing UV system does not have flow pacing and operates at 100% intensity and uses 8.4kW per hour. The proposed UV system is flow paced and can operate within a 60% to 100% intensity range. At 100% intensity, the proposed UV system would use 18 kW per hour. Table 35 summarizes the anticipated labour savings and the maximum additional electrical cost should the system be operated at 100% full-time.

Table 35 - UV O&M Costs

Operations and Maintenance	Annual Cost
Labour[1]	-$2,000.00
Electricity[2,3]	$9,000.00
TOTAL	**$7,000.00**

1 Labour costs were assessed at $65/h
2 Electricity costs were assessed at $0.0973/kW.
3 Difference between the existing and proposed system, both operating at 100%.

Class “D” Cost Estimate

A Class “D” Cost Estimate to upgrade the existing UV system for the 20-year design with allowances for 15% Engineering and 35% Contingency was assessed. The capital cost includes the supply, install and construction of a retrofitted system that would replace the existing modules with two new modules.

- Retrofit UV Disinfection System $450,000

7 RECOMMENDATIONS

The existing PCC is biologically overloaded and at high flows some of the individual processes are at or nearing their hydraulic capacities. Influent wastewater strength and $CBOD_5$ loading is significantly higher than what was projected when the PCC was upgraded in 2005. Due to the PCC being organically overloaded, it has resulted in instances where the effluent water quality has exceeded the MWR discharge criteria of 45 mg/L $CBOD_5$. Also, the Federal Wastewater Systems Effluent Regulations was brought into effect in 2012 which has set more stringent effluent quality criteria for discharges to surface waters (25 mg/L for $CBOD_5$ and TSS, as quarterly averages). The change in the Federal regulation was contemplate in this assessment as it should be addressed with any upgrade to the PCC.

This report assessed four secondary treatment technologies: rotating biological contactors (RBC), sequencing batch reactors (SBR), complete mix activated sludge (CMAS) and moving bed biofilm reactors (MBBR). Of these four only the MBBR was determined to be a feasible solution. MBBR is therefore the recommend process upgrade of the four technologies assessed.

In order to transition the secondary treatment process from RBCs to MBBR a number of process upgrades are required. Table 36 lists the process upgrade and suggests a phased approach to their implementation. The table also summarizes the capital and annual O&M cost implications for each upgrade.

Table 36 - Summary of MBBR Process Upgrades. Capital and O&M Costs

Phase	Process Upgrade	Capital Cost	O&M
Phase 1	Emergency Generator	$240,000.00	-
	Electrical Service Upgrades	$300,000.00	-
Phase 2	Headworks	$3,300,000.00	$2,000.00
Phase 3	Primary Filtration	$4,700,000.00	$47,000.00
Phase 4	MBBR	$5,100,000.00	$22,000.00
	DAF	$5,300,000.00	$58,000.00
Phase 5	UV Upgrade	$450,000.00	$7,000.00
Total	**All Phases**	**$19,390,000.00**	**$136,000.00**

The order in which the emergency generator is upgraded could be deferred until after the headworks; however, additional assessment by an electrical engineer should be conducted. The UV is currently listed as the last upgrade item, but this may need to be revised depending on schedule, process performance and regulatory compliance, this item could be advanced as required.

A high strength wastewater study is currently underway. Sampling of the influent using automated samplers to identify the sources and confirm the characteristics of the high strength wastewater is expected to start in the near future.

The impact on the anaerobic digesters by removing primary treatment was outside the scope of this report. Because of this, it was recommended that a primary treatment process continue to be utilized (mechanical primary screen). However, should the City wish to consider removing the primary treatment process, it is recommended that an additional sludge management study be completed.

Next Steps

The outcomes of this assessment identified a number of processes that are at, or have exceeded, their rated capacity and has recommended a phased approach to upgrading the facility. The recommended sequence of the phased upgrades is as followings:

Phase 1 – Detailed Design and Construction of Electrical and Emergency Generator Upgrades

Phase 2 – Detailed Design and Construction of a New Headworks

Optional Sludge Management Study

Phase 3 – Detailed Design and Construction of a New Primary Treatment Process (Mechanical Primary Screens)

Phase 4 – Detailed Design and Construction of a New Secondary Treatment Process (MBBR) and Secondary Clarification Process (DAF)

Phase 5 – UV Upgrades

It is recommended that the first step be for the City to plan for and secure financing to undertake detailed design and construction of Phases 1 and 2 as they are able.

The City should then review the condition of the anaerobic digesters to determine the feasibility of their long-term use into the future. This will assist decision makers in deciding if a sludge management study should be conducted for the continued use of the anaerobic digesters or if a switch to an alternative means of sludge management (i.e. aerobic digestion). If the condition assessment is good, then a sludge management study may not be required and the subsequent phases can be implemented as outlined. If the condition assessment indicates that renewal of the anaerobic digesters is required, a sludge management study will assist decision makers with identifying the most cost-effective solution for sludge management. The outcome of the sludge management study could influence Phase 3 with additional costs to renew the anaerobic digesters (implement primary treatment) or change Phase 3 altogether to an aerobic digestion upgrade instead.

The cost estimate of Phase 4 is directly influenced by Phase 3 and is valid so long as primary treatment is continued to be provided. Should this change, the recommendation for MBBR stays the same, but new tankage would need be added that would increase the cost. Because an entirely new treatment process will be integrated, fundamentally changing the existing PCC, a revised EIS and MWR registration is anticipated at the time of construction.

Appendix A

Hannah Environmental RBC Proposal

Hannah Environmental Equipment Inc.
144 Wescar Lane, Suite 200, Carp, Ontario, K0A 1L0
(613) 254-7475
www.HannahEquipment.com

BUDGET QUOTATION

Urban Systems
Kelowna

14 May 2017
Our Reference # H077

Michael Schaad

Subject: Supply of (2) 4.5 M Rotating Biological Contactor

The **Hannah Environmental Equipment Inc NuDisc ® R 4.5** sewage treatment system which we propose to supply is described herein. The rotors are a mirrored image of one of the two existing trains. It will be a direct drop in to concrete biozones.

R 4.5

The following is a summary of the scope of our supply.

- **(2) Hannah Environmental Equipment Inc NuDisc ®** R 4.5
- (4) New bearings
- (2) New gear box and torque arm
- Installation assistance
- O&M Manual
- Commissioning

DESIGN CRITREA

The (2) 4.5m rotors are the same as the existing trains.

PLANT DESCRIPTION

The sewage treatment plant which we propose to supply will consist of a prefabricated treatment plant consisting of a rotor, bearings and drive.

The rotor is a "Second Generation" design. "Second Generation" RBC's have some key Features:

1. Removable pie shaped sections of biological support media.
2. Bio support media supported on structural steel frames.
3. Gaps between the bio support media banks for drainage.
4. A gap between the bio support media and the main rotor shaft. This reduces the possibility of plugging of spent biomass in the media around the shaft thus reduces the possibility of shaft overload and shaft breakage.

The above rotors are designed for placement in the concrete tanks.

The **Hannah Environmental Equipment Inc NuDisc ®** RBC has a main rotor shaft and media support frame system designed for a 20-year operating life.

ELECTRICAL SERVICE

The 7.5 hp RBC motor to suit the following existing electrical service.

ELECTRICAL CONTROLS

The existing RBC electrical controls will be used.

Installation

The following is a summary of the scope of our field work. Using our factory trained crew.

- Assist in placing the supplied rotor in the concrete tank.
- Assist in setting the bearings.

2 site day are allowed to complete the above work.

Additional services of a site work are not included but are available for a per diem rate of $950.00, plus living and travelling expenses per man. This is for an 8 hour day. We reserve the right to charge extra for over 8 hours. The trip starts and ends at our facility. Travelling time is charged.

Commissioning

2 day on site for commissioning. This will be conducted directly after the installation.

Insurances & Safety

We comply with all of the insurance, WSIB and safety requirements.

WARRANTY

Our warranty is one year from date of shipment. This warranty covers:

- All parts and components
- Labour by our crew.

PRICE

Price including all quoted items: $471,420.00

ALL TAXES EXTRA

The above quoted price is net and firm for purchase within 30 days of the date of this quotation.

INVOICING & PAYMENT TERMS

To be reviewed.

F.O.B. POINT:

Carp Ontario

DELIVERY

The normal shipping schedule from the factory will be approximately 12 - 16 working weeks from the day Hannah Environmental Equipment receives back the final approved drawings signed by the client or his designated representative, and Hannah Environmental Equipment receives of the signed contract along with down payment cheque. If the approval of drawings is waived, then this must be so stated by the Owner or his designated representative. The normal delivery time quoted is based on work loads of Hannah Environmental Equipment's Engineering and Production Departments at the time of submitting this price quotation. The actual delivery time can be finalized when Hannah Environmental Equipment receives approved drawings back or when we are notified that the approval of drawings is waived.

Drawings for review/approval can be supplied in approximately 2 working weeks from receipt of a signed contract, finalization of the detailed scope of supply and finalizing of on-site information required such as the existing concrete tanks details. The normal drawing preparation time quoted is based on work loads of Hannah Environmental Equipment's

Engineering Department at the time of submitting this price quotation. The actual drawing preparation time can be finalized when Hannah Environmental Equipment receives the signed contract along with down payment cheque and necessary on site information.

INVOICING

Invoices will be issued when the goods are ready for shipment. If the goods are held in storage at the customer's request, the invoice will still be issued when the goods are ready for shipment and will become due for payment as it would if the goods had been shipped when ready.

EXCLUSIONS

Unless otherwise noted herein, the price shown does not include installation, field work, concrete work, concrete design work, anchor bolts, site work, excavation, influent and effluent piping, plumbing, electrical wiring/components or work, flumes, pumps, lift stations, surveys, permits, agency approvals, sampling, testing or other products and services not specifically presented in this proposal.

TERMS AND CONDITIONS

Back charges to Hannah Environmental Equipment Inc. will be allowed only with our preauthorized written permission.

With over 40,000 installations worldwide and 45 years of experience building RBC units, we thank you for your interest in our products. The requisite summary of the qualifications and experience including references will be provided with the submittal package. If you require further information please feel free to contact us, or your local representative:

Yours very truly,
Hannah Environmental Equipment Inc.

Simon Hannah. President

Appendix B

Veolia MBBR and Spidflo® Proposal

URBAN SYSTEMS

Attention: Michael Schaad

BUDGET PROPOSAL

NELSON WWTP, BC

2017-09-25

PREPARED BY:

ROBERT LAFOND, ING, SENIOR PROCESS ENGINEER

CHRIS HOWORTH, SALES REPRESENTATIVE

REF: TM-87775

Veolia Water Technologies Canada

ISO 9001: 2008

4105 Sartelon, St-Laurent (QC) H4S 2B3

Tél: 514 334-7230 ● Fax: 514 334-5070

www.veoliawatertechnologies.ca

WATER TECHNOLOGIES

Michael Schaad
Urban Systems
Kamloops, BC

Chris Howorth
3138 Brookridge Drive
Vancouver, BC, V7R 3A8

September 25th , 2017

by email

NELSON Wastewater Treatment Plant – Budgetary Proposal

Dear Michael,

Thank you for the opportunity to support your evaluation of waste water treatment for Nelson. Please find attached our budget proposal to aid in your evaluation.

Since inventing the MBBR process in the 1980s we have now provided it for over 900 installations globally, including over 20 in Canada. Our proposed MBBR and DAF combination for Nelson is very compact, making best use of available space and existing infrastructure. This makes it a cost effective solution for the City. Its simple flow-through nature makes it easy to operate and maintain, and provides flexibility/robustness to deal with varying conditions. As RBCs are also a flow-through process it will be easy for operators to adapt to the new system. The main differences are that MBBRs are far more efficient (delivering more than twice the capacity in the same volume), and require no in-tank maintenance.

I hope our proposal helps you with your evaluations, and look forward to speaking with you in due course.

Yours respectfully,

Chris Howorth

Sales Representative

Veolia Water TechnologiesCanada Inc. www.veoliawatertechnologies.ca

Head office: John Meunier Inc. 4105 Sartelon St., Saint-Laurent, QC, H4S 2B3

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1. INTRODUCTION

The City of Nelson's WWTP faces challenges that include:

- Insufficient treatment capacity
- Failing mechanical equipment (in particular the RBCs)
- Inability to meet stringent effluent quality limits
- Operational impediments (e.g. poor layout and lack of odour control in the headworks screening area)

This proposal addresses these challenges and will enable the plant to meet treatment goals well into the future, while improving operational efficiency and safety.

Our proposal presents two different treatment solutions:

- Scenario 1 comprises fine screening, MBBR and DAF. New MBBR tanks are proposed to augment the capacity of existing tanks.
- Scenario 2 adds primary filters to reduce loading. This reduces the size of the MBBRs and avoids the need for new MBBR tanks (though new primary filter basins are needed).

The three common components provide various benefits, including:

1. Escalator fine screens:
    a. Performance: Independently proven to deliver best in class performance (averaging 79% solids capture under both wet and dry flow conditions)
    b. Robustness: Tolerates (and removes) large solids, high trash loads, and wide flow ranges
    c. Complete solution: Integration with Rotopac washer-compactor to produce inoffensive screenings ready for disposal, which are dewatered and low in organic content.
2. AnoxKaldnes MBBR:
    a. Compactness: Our proposed "K5" media provides 800 m2 of protected surface area per m3 of volume, meaning a vast amount of biomass is retained in a small basin, and hence large loads can be treated with short hydraulic retention times. This results in compact solutions, reducing civil costs and footprint requirements.
    b. O&M simplicity: The MBBR process is a simple, flow-through technology (like RBCs and lagoons). The only mechanical component is aeration blowers, which are controlled automatically using DO measurement.
    c. Resiliency: MBBR's attached biomass (fixed film) tackles wide load variations, resists shocks, and cannot wash out under high flow conditions. It automatically adapts to changing conditions – operators do not need to worry about sludge age, microorganism types etc.
3. Spidflow DAF:
    a. Compactness: Operates at average rise rates of 30 m/h, delivering very compact solutions that can be retrofitted into existing clarifiers.

b. Performance: Produces high solids content sludge (averaging 4% DS content), avoiding the need for thickeners. Routinely operates without any chemicals, only requiring polymer to tackle high flow/load events.
c. Easy O&M: Uses an innovative method of white water production that is more flexible, efficient and reliable compared to conventional DAFs.

Our Hydrotech primary filters proposed for Scenario 2 provide the following benefits:

- Simplicity: The equipment starts and stops automatically using a level sensor, needs no automatic valves, and uses just two motors (one to turn the filter drum, the other in the backwash pump).
- Ease of O&M: Our filters have very long media life, avoiding the need to replace filter panels (although when required this is very quick to do). The few components used in the system are all easily accessible.
- Flexible performance: The filters achieve equivalent removal performance to conventional primary treatment (sedimentation) without the need for chemicals. Polymers can be used to enhance performance, achieving TSS removal as high as 90%. This can be done only when needed, e.g. to respond to peak events.

Veolia Water Technologies Canada employs approximately 220 professionals across the country, making us a leader in the industry. Our team encompasses every skill needed to develop, design, deliver and support wastewater treatment solutions, from R&D to ongoing O&M support services (and everything in between).

2. DESIGN BASIS

Our design is based on parameters provided by Urban Systems, as outlined below.

Table 1 Design Flow and Load

Flow Category	Current (m^3/d)	20 Year Design (m^3/d)
Average Day Dry Weather Flow (ADWF)	5 400	7 600
Average Day Wet Weather Flow (AWWF)	6 000	8 500
Average Annual Daily Flow (AADF)	5 500	7 800
Maximum Month Flow (MMF)	9 900	14 000
Maximum Day Flow (MDF)	14 600	20 600
Peak (Hour) Wet Weather Flow (PHF)	20 100	28 300

Loading Parameter	Units	2 files 75% MMF 20 YEAR *each file*
Flow	m3/d	14 000
$cBOD_5$	mg/L	161
$cBOD_5$*	kg/d	2 259
TSS	mg/L	150
TSS*	kg/d	2 106
Influent Temperature	°C	6
$sBOD_5$:$cBOD_5$	%	35
Phosphorus removal	kg/d	N/A
Nitrogen removal**	kg/d	N/A

BOD loading is increse by 20% for MMF condition
TSS loading is increase by 30% for MMF condition

Peaking Factors	Value
$CBOD_5$ Max Month	1,2
TSS Max Month	1,3

Our proposed fine screening is designed to provide 2 X 100% of the 20 year PHF. The remaining processes are designed to provide 2 X 75% of the 20 year MMF. Each of the two trains can be operated independently of the other.

Table 2 presents effluent quality objectives:

Table 2 Effluent quality objectives

Effluent Parameters	Value	Units
$CBOD_5$ (Regulation)	25	mg/L
$CBOD_5$ (Target)	15	mg/L
TSS (Regulation)	25	mg/L
TSS (Target)	15	mg/L
Un-ionized NH_3-N (Regulation)	1,25	mg/L

Assumptions:

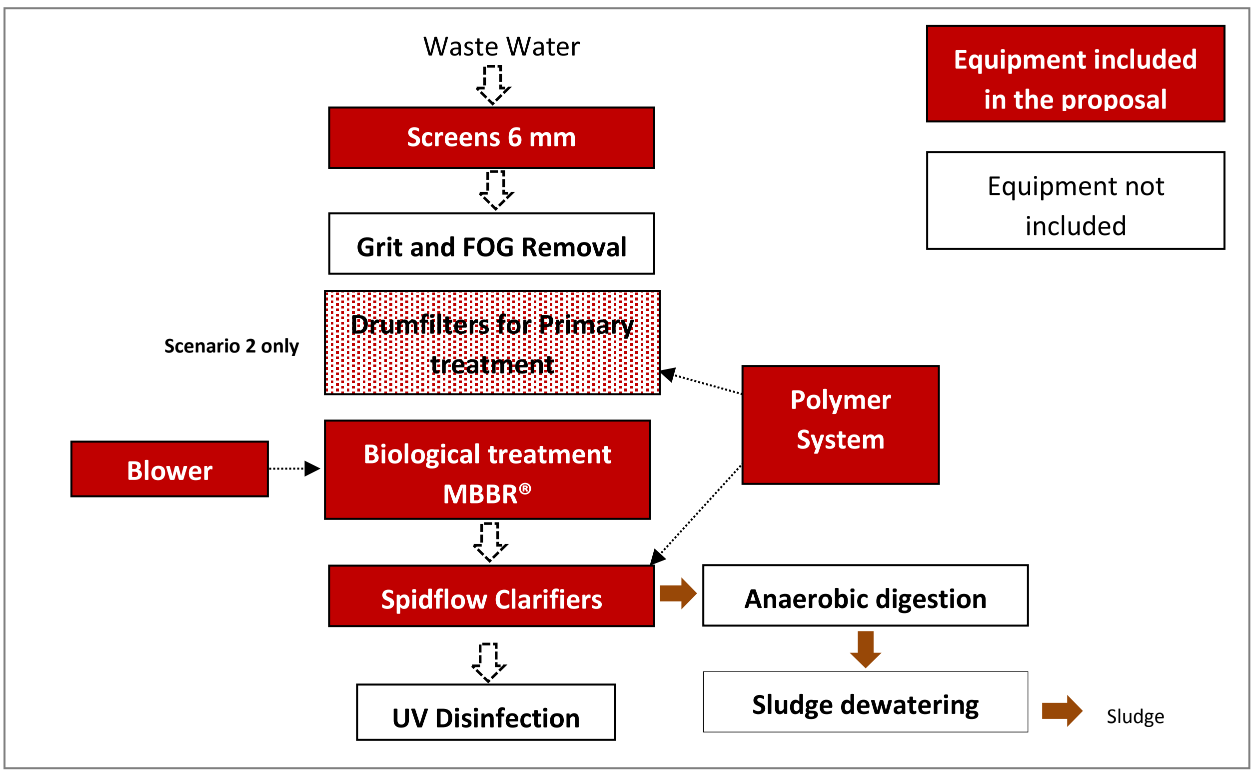
- $cBOD_5$ soluble is 35% of the total $cBOD_5$.
- Oil and Grease (O&G) concentration is less than 50 mg/L downstream of the headworks
- Site elevation is 530 m.
- Nitrification is not required to achieve 1.25 mg/L un-ionized NH3-N

3. TREATMENT CHAIN

Our proposal provides two alternative treatment options:

- Scenario 1 comprises:
  - Headworks screening - new 6 mm Escalator perforated plate screens with Rotopac washer compactors
  - No Primary treatment
  - MBBR retrofitted into existing tanks (primaries, EQ) and installed into new tanks
  - Dissolved Air Flotation retrofitted into existing secondary clarifier tanks
- Scenario 2 comprises:
  - Headworks screening - new 6 mm Escalator perforated plate screens with Rotopac washer compactors
  - Primary treatment installed into new concrete channels
  - MBBR retrofitted into existing tanks (primaries, EQ and RBCs)
  - Dissolved Air Flotation retrofitted into existing secondary clarifier tanks

In both scenarios the plant's existing grit and FOG removal systems will be retained.



3.1. Escalator® Fine Screens and Rotopac Washer Compactors

MBBR influent will be screened using ESCALATOR® 6 mm perforated plate screens. Escalators provide highly efficient fine screening in any direction, unlike bar and step type screens, which pass approximately twice the amount of material at the same aperture spacing. Escalators provide consistently high capture under both wet and dry weather conditions, with an independently verified average solids capture ratio of 79% (UKWIR study on the 6 mm version, installed with self-adjusting brush mechanism). Escalator can be installed in a new or existing channel with a minimum of civil works. With over 1100 units in service, ESCALATOR® is a well proven, reliable technology.

Escalator screens are a type of continuous belt screen, using multiple panels with drilled perforations mounted on heavy duty chains on each side of the equipment. The chains are carried on driven sprockets at the top and idler sprockets at the base. The panels are shaped with a “shelf” on top, which provides rigidity, and lifts larger unmattable solids out of the flow. The equipment is delivered fully assembled and tested following manufacture in our Canadian fabrication facility.

Under dry conditions the screen operates in “stepping mode”, whereby screenings are allowed to accumulate on the panels, creating a filtering mat that maximises capture. On reaching a differential level set point, or after a predetermined time, dirty panels are lifted just clear of the flow (a “step”), presenting clean panels. After multiple steps the screen enters cleaning mode, where panels are rotated around the unit and cleaned using both a rapidly rotating, self-adjusting brush, and with water jets spraying from inside the unit. Under wet weather conditions the screen rotates and is cleaned continuously, ensuring flow is reliably passed. High and low screen speeds are available to accommodate all flow rates under the wet weather mode. These methods of operation and cleaning maximise performance and minimise wear. Unlike most other screen types (e.g. bar, step, inclined auger types), Escalator lifts captured material out of the channel before removing it. This avoids material being broken up on the screen face, reduces the risk of flow backing up, and improves overall performance.

Figure 1 Escalator ® - 6 mm screen

The screenings that have been retained by the ESCALATOR® will be dewatered, washed and compacted using Rotopac RPW washer compactors. The Rotopac RPW washer compactor eases material handling, reduces disposal costs and avoids nuisance (e.g. odours, sanitary risks to staff etc.) associated with screenings. ROTOPAC ® is a proven technology with more than 150 units in operation in North America. This simple device is delivered pre-assembled and pre-tested, and is designed to integrate seamlessly with Veolia's wide range of John Meunier fine and coarse screens.

In operation, screenings are introduced into the slowly rotating spiral auger zone by a hopper. The screenings are washed in wash water (e.g. plant effluent), returning organics to the main process stream to be treated. The screw auger then conveys screenings up the discharge tube where they are dewatered and compacted. At the point of discharge the screenings are inoffensive and dewatered to a non-dripping state. The discharge tube is available with a bagging option to enhance handling and nuisance control still further.

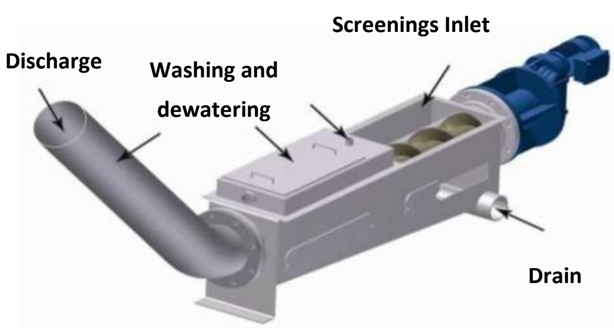
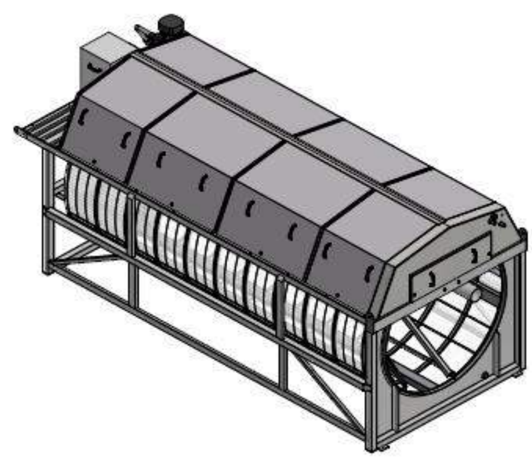
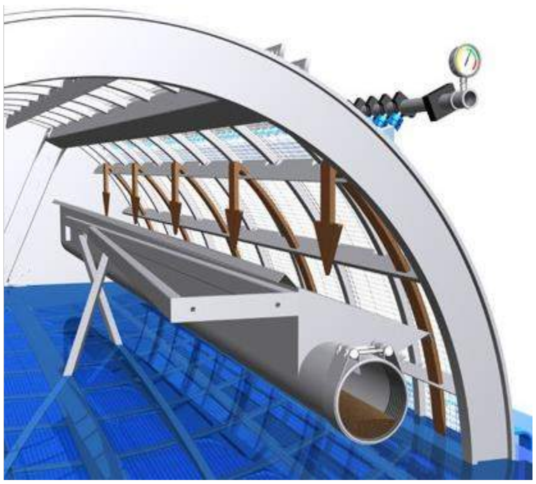


Figure 6 ROTOPAC RPW pretreatment (dewatering, washing and compacting)

3.2. Hydrotech Drumfilter Primary Treatment (Scenario 2 only)

Hydrotech microscreens provide a very compact and flexible primary treatment alternative compared to conventional primary clarification. The filters are installed downstream of the headworks, and are primarily targeted at TSS removal. Our proposed Hydrotech Drumfilters utilize a 40 micron mesh size polyester filter cloth mounted on filter panels installed around the outside of a drum, which is mounted in a stainless steel frame for installation into a concrete channel/basin. Pre-treated water enters into the drum and is filtered in inside-out mode, flowing through the polyester cloth before reaching the filtered water side. Solids and particles present in the inlet water are retained by the filter media inside the drum, leading to a build-up of captured solids on the filter media, and raising the water level at the inlet of the unit. At a predetermined level set point a backwash cycle is initiated.





When backwashing, the drum rotates to expose clean filter media to the water flow path, while dirty filter panels are cleaned by spray nozzles. Filtered water is pumped to a series of spray nozzles strategically installed on the backwash system. This serves to clean the entire surface of the filter media while limiting the use of filtered water in the process. An outlet weir integral to the drum filter basin is used to maintain a reservoir of filtered water for backwashing purposes. Excess filtered water flows over the weir and out of the unit. The collected solids are washed off the filter panels into the solids collection trough as the drum slowly rotates. The removed solids flow together with the backwash water out of the filter by gravity.

The Hydrotech primary drumfilter routinely operates without any chemicals, achieving approximately 50% TSS removal (which typically equates to approximately 25% BOD removal). For Nelson we propose to provide polymer preparation and dosing – polymer addition improves TSS removal up to approximately 90%. This reduces TSS and BOD loads on downstream treatment processes (MBBR and DAF), and also maximises energy recovery in anaerobic digestion. Further, polymer addition enables the filters to run at higher loading rates, allowing the high PHF flows to be reliably treated. We recommend a flocculation time of 3-4 minutes is provided upstream of Drumfilter following polymer addition.

Table 3 Drumfilter Design

Item	Unit	Value
Drumfilter unit	---	HDF-2010
Unit footprint	m^2	24
Design Capacity		
Number of units	---	1 per train
Number of drum per unit	---	1
Filtration area per unit	m^2	22
Mesh size	microns	40

3.3. AnoxKaldnes MBBR secondary treatment

We propose AnoxKaldnes™ Moving Bed Biofilm Reactor (MBBR) technology for secondary biological treatment, using AnoxKaldnes™ K5 media. The microorganisms treating the wastewater grow on the surfaces of the AnoxKaldnes™ media (or carriers) in an aerated reactor. The K5 media are approximately 25 mm in diameter, as seen in Figure 2, and provide a protected environment in which bacterial populations and protozoa can grow very effectively.

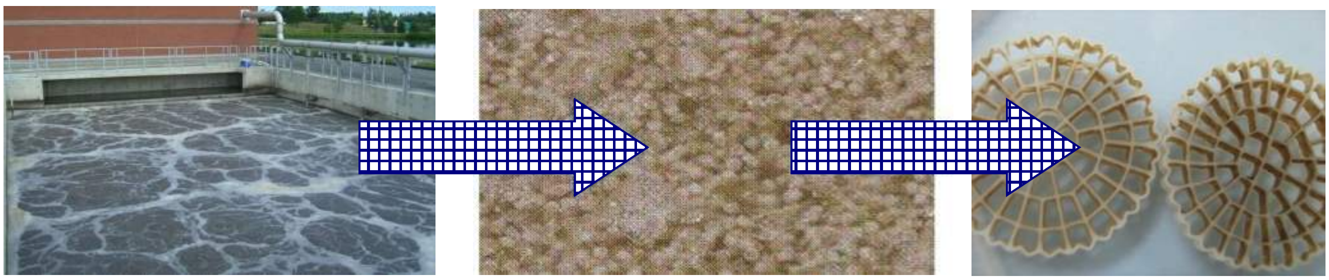


Figure 2 Biofilm Growth in MBBR Media

The carriers are retained in the tanks by sieves (see Figure 3) which allow the treated water to pass to downstream units for further processing. Stainless steel laterals and diffusers provide air to the system for bacterial growth and mixing.

Figure 3 Aeration grids & MBBR Sieves

One important feature of the process is that biofilm thickness is automatically controlled by the movement of the media, so that oxygen diffusion through the biofilm is efficient. Detached biofilm is suspended within the reactor and leaves the reactor with the treated wastewater. Sloughed biofilm is removed in a downstream clarifier, together with other particulates (in Scenario 1 these particulates include the material that was formerly removed by primary clarification).

Figure 4 MMBR system in operation

MBBR systems are among the most simple technologies available for secondary treatment, employing the same flow-through basis as RBCs and lagoons. There is no need to control activated sludge wasting/recycling rates, sludge age or worry about F/M ratios. The biomass adapts automatically to the feeding conditions, temperature etc. Our AnoxKaldnes media does not require backwashing, and has a proven longevity of decades in operation.

MBBR Design Parameters:
Table 3 shows the characteristics of the MBBR process proposed for each of the scenarios. The MBBR system is designed for 2 x 75% of MMF load at the lowest winter temperature (6°C).

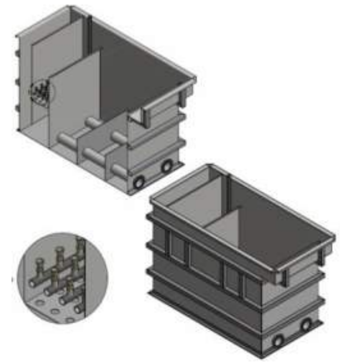
Table 4 MBBR design parameters for each scernario

Parameter	Unit	Scenario 1 (no primary treatment)	Scenario 2 (with primary treatment)
Average Daily Flow (ADF)	m^3/d	7 800	7 800
Maximum Monthly Flow (MMF)	m^3/d	14 000	14 000
Design scenario (2 x 75% MMF)	m^3/d	2 x 10 500	2 x 10 500
Site altitude	m	530	530
Total reactor volume	m^3	1720	1140
Total Hydraulic retention time (average)	h	5.3	3.5
Number of trains	---	2	2
Number of reactors per train	---	Primaries tanks + EQ tanks + New tanks *	Primaries tanks + EQ tanks + RBC tanks *
% fill of media in reactors	%	60	60

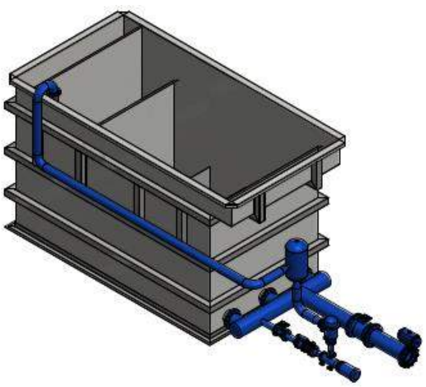
* In Scenario 1, 914 m^3 of additional MBBR tank volume is required, divided equally between two trains. The dimensions of these tanks is flexible to fit site constraints. A side water depth of approximately 5 m is typical, and we recommend a freeboard of 750 mm. In Scenario 2, the RBC tanks have sufficient volume to provide the remaining volume required (330 m^3).

3.4. SPIDFLOW® Dissolved Air Flotation Secondary Clarification

Dissolved Air Flotation (DAF) is a process whereby micron-size air bubbles coming from the addition of pressurized air-saturated water (white water) attach themselves to higher-density particles, causing suspended particulate matter to float to the surface of a vessel, achieving liquid/solids separation. These floated particles form a dense foam/sludge mixture that is removed by mechanical skimming.



Our proposed Spidflow DAF technology uses an innovative system to produce air saturated white water without the need for an air compressor or any pressurized tanks, which often lead to reliability issues with conventional DAF technology. Our system uses centrifugal pumps which draw in air using the Venturi principle, resulting in a far simpler and more reliable system. This design also allows for multiple pumps to adapt to varying flow conditions, improving hydraulic flexibility and reducing energy costs.



Spidflow's unique hydraulic design allows for a fast dispersion of the micron-size air bubbles by means of white water injection nozzles, and controls the velocity and direction of the liquid so that suspended solids are rapidly separated at the surface of the separation zone. This allows for high nominal loading rates of up to 30 m/h, and hydraulic capacity up to approximately 50 m/h, resulting in very compact designs. It also produces a thick sludge with average dry solids of 4%, avoiding the need for thickening prior to digestion. A key feature of Spidflow DAF is that it routinely operates without any chemical addition. Our proposal includes a polymer preparation and dosing system however, which we recommend for maintaining performance during high flow/load events.

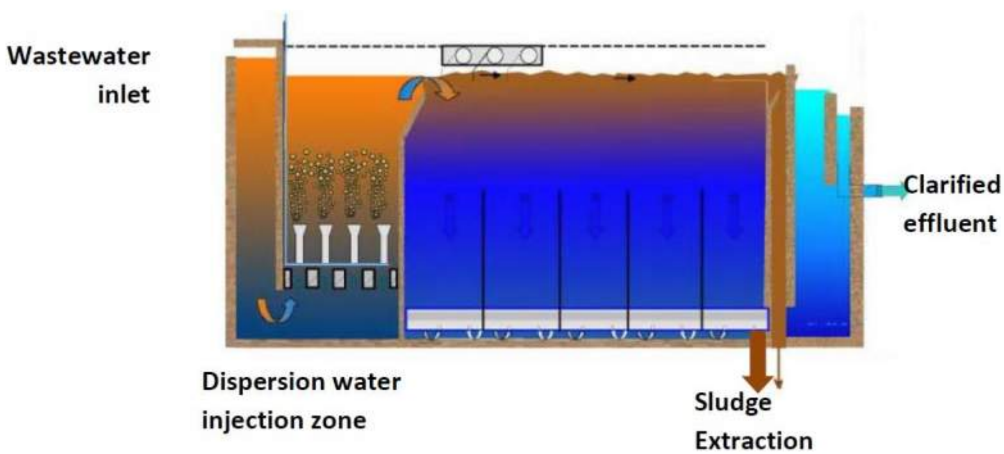


Figure 5 Spidflow Secondary clarifier

3.5. HydraPol Polymer Preparation and dosing

Our HydraPol automatic dry polymer preparation system is included in our proposal, to provide the polymer needed for primary filtration and DAF. Dry polymers contain between 90% and 100% active product, and hence incur the lowest transportation (hence operating) costs. They can also be stored for longer periods than emulsion polymers.

Dry polymer systems broadly comprise three components:
1. Dry material storage/handling/feeding
2. Polymer wetting and mixing

3. Polymer maturation

Dry polymers are typically procured either in small bags or bulk bags (holding up to 1000 kg), which should be stored in dry conditions. The material is transferred to a hopper before feeding to the wetting process. For this project, small bags are expected to be used and polymer transfer can be done using a vacuum system, which is included in our proposed equipment scope. Dry material is metered from the base of the hopper to the wetting cone automatically to provide the correct polymer quantity. Veolia offers screw based systems for this purpose, with screws turning at a constant feed rate for a controlled time period. When feeding is complete a valve closes to isolate the dry material from the wetting cone, avoiding moisture impacts on the dry material side of the equipment.

Effective wetting is critical to introduce dry material into solution without forming clumps or "fish eyes", which can lead to inconsistent polymer batches and O&M challenges. Wetting cone design also needs to allow easy access for maintenance. After wetting, the material needs to be rigorously mixed to produce a dilute, homogenous solution. Our HydraPol technology addresses these various O&M challenges, and uses an educator based system (working on the Venturi principle) to impart high shear forces using water pressure as the motive energy source. Polymer solution is transferred to a maturation tank, where it is gently mixed to encourage polymerisation and polymer chain entanglement. Mixing duration guidelines are provided by polymer suppliers. When fully mature the batch is transferred to a storage tank ("day tank") from where it is dosed.

Our proposed polymer dosing system employs progressive cavity type pumps (screw pumps) manufactured by Seepex, but we can (and do) work with other types/manufacturers when required. Our package dosing systems include all necessary piping, valves, gauges, calibration columns, controls, pump redundancy, etc.

3.6. Equalisation Considerations

Our proposed screens are each designed to pass the PHF (2 X 100% capacity). Our remaining unit processes are designed to treat 2 X 75% of the MMF load. They are also designed to pass the PHF hydraulically. The filters and DAFs will require polymer addition to maximise performance under peak conditions, hence our proposal includes a Hydrapol dry polymer preparation system and polymer dosing skid. All of these design considerations avoids the requirement for equalisation, and hence eases footprint challenges on the constrained site.

We recommend further discussion on the subject of PHF management as the project develops, for example to explore PHF duration, frequency and loads/dilution, and to assess different approaches to handling PHF conditions. The relatively high length to width ratio of some existing basins is one particular consideration in regard to MBBR retrofitting. This might influence our recommendations for basin partitioning.

4. SCOPE OF SUPPLY

Scope per train:

Pre-treatment	
ESCALATOR pretreatment unit (6 mm perforated stainless screen panels): - Structural frame with pivoting system, guides, outlet chute and covers; - Filter elements belt with shafts and sprockets, c/w dual-speed motor* and gear drive*; - Zinc coated extra-strong carbon steel chain; - Self-adjusting rotating cleaning brush with carbon steel shaft, c/w motor and gear drive; - Elements washing system, c/w spray nozzles, solenoid and manual valves, NEMA-7 enclosure; - Differential level control system, ultrasonic bubbler type, NEMA-7 enclosure; - High water level start float switch; - Fasteners & anchors in stainless steel AISI 304. **ROTOPAC RPW pretreatment unit (screenings compaction)** - Trough, hopper with floor mounted support (AISI 304); - Shafted high abrasion resistant steel screw and bearings box, c/w motor and gear drive; - Solids washing system and dewatering zone washing system; - Set of solenoid and manual valves for washing systems, NEMA-7 enclosure; - Fasteners & anchors in stainless steel AISI 304.	One (1)
Primary Drumfilter	
Hydrotech Drumfilter HDF 2101 (internal frame for installation into concrete channel) • **One (1) 304 SS Filter frame** • **One (1) 304 SS center drum** • **One (1) Motor with gearbox** • **One (1) Grundfos backwash pump** • **Lot of woven polyester filter media (40 um pore) panels** • **Polymer dosing system**	One (1)
MBBR equipment	
K5 Media	Lot
Medium Bubble aeration systems in 304L stainless steel, including header and lateral piping within the reactors. Vertical down comers are included	Lot

Sieves assembly in 304L stainless steel to retain the carrier elements and to minimize head loss.	Lot
Positive displacement blower (1 per train) including silencers and soundproof enclosures	One (1)
Instrumentation - Level switch - DO probe - Level float	 Three (3) Three (3) Three (3)
Spidflow Secondary Clarifier	
Pre-fabricated Spidflow tank in carbon steel; **Surface sludge skimming system** **White Water production system (skid mounted)** **White Water pumps (1 in duty + 1 in stand-by)** **Polymer dosing system** **Lots - Instrumentation; including** **- Spidflow influent flowmeter** **- White water flowmeter** **- Spidflow level probe** **- Effluent TSS probe**	One (1)
FOR BOTH TRAINS: **Positive displacement blower** (backup) including silencers and soundproof enclosures	 One (1)
Control panel (NEMA 12) for the operation of equipment included in this proposal. Interface to allow equipment operation. - One (1) PLC Compac Logic, NEMA 12 - One (1) HMI - Control system engineering - Programming (PLC and HMI) - Testing at Veolia's shop	One (1)
Hydrapol dry polymer preparation system	One

These elements are included in the proposal:

- Services:
  - Process engineering and drawings showing outline tank requirements and equipment location
  - Maintenance manuals.

These elements are not included in this proposal:

- Permits, including certificate of authorization, necessary construction permits and licences.
- Unloading, storage, maintenance preservation and protection of all equipment and materials on-site.
- All site preparation, grading, finding foundation placement and excavation for foundation, underground piping, conduits and drains.
- Foundations, buildings, sumps, trenches and similar concrete works, site interferences, fencing and landscaping (including asphalt or paving).
- Supply and installation of interconnecting piping between the client's installations and the treatment system, and between the various unit processes that are part of the treatment system.
- All labor, material and utilities required to install the supplied equipment.
- Supply and installation of all electrical power and conduit to the treatment system main control panel plus interconnection between the treatment system main control panel and ancillary equipment as required, including wire, cable, junction boxes, fittings, conduit, etc.
- Equipment transportation to Nelson, BC.
- Equipment Commissioning and Start-up.
- Chemicals for commissioning
- All concrete tanks
- SCADA

5. BUDGET PRICE AND TERMS OF PAYMENT

Estimated cost

The estimated budgetary cost for equipment supply is **(**Currency: Canadian dollars & All taxes extra):

Without Primary Drumfilter (2 x 10500 m3/d)………………………………………………………..$ 3,200,000 CDN

With Primary Drumfilters (2 x 10500 m3/d)………………………………………………………..$ 3,160,000 CDN

Terms of payment

Our proposed terms of payment are as follows:

- 10% on receipt of fully executed contract
- 15% on submittal of shop drawings
- 70% on the delivery of equipment to the site
- 5% after commissioning
- All payment terms are net 30 days from the date of invoice.

Suggested schedule

The projected schedule is shown in the following table:

Table 5 Schedule

ITEM	TIMELINE	CONDITIONS
Shop drawings	6-8 weeks	Submission within designated timeline following receipt of a contract executed by all parties
Complete Equipment Delivery	18-24 weeks	After receipt of written approval of shop drawings

APPENDIX A: EQUIPMENT INFORMATION

John Meunier Escalator® Fine Screen

WATER TECHNOLOGIES

John Meunier Escalator®
Fine Screen

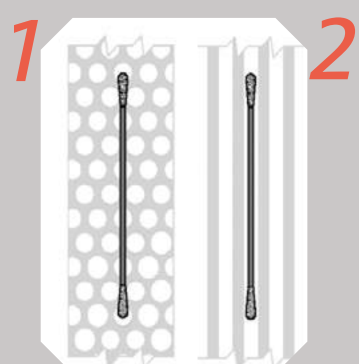
The John Meunier Escalator gives continuous fine screening for channel type applications with superior efficiency to slotted and bar screens. This highly versatile screen is successfully employed in wastewater, stormwater and potable water applications. Perforated stainless steel screen panels are carried on heavy-duty chains and incorporate holes of 1 / 4” (6mm) diameter or less, giving fine screening in any direction.

The screen panels are specially formed to create shelves giving the ability to remove larger screenings and to increase the effective screening area. Flow capacity is dependent on channel width, water level and perforation size.

The Escalator can readily be installed in new or existing channels with a minimum of civil alterations. With over 1100 units in service, the Escalator Fine Screen is a proven, reliable component of John Meunier pretreatment products.

Screening removal efficiency comparison

1. **John Meunier** Escalator Fine Screen
2. **Bar & Step Type** Fine Screen

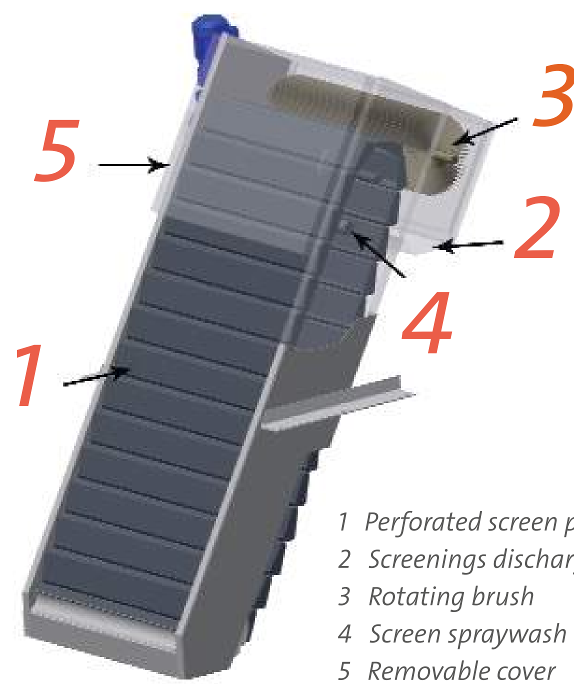


79% 35%

Percentage of Solid Capture Ratio

Features

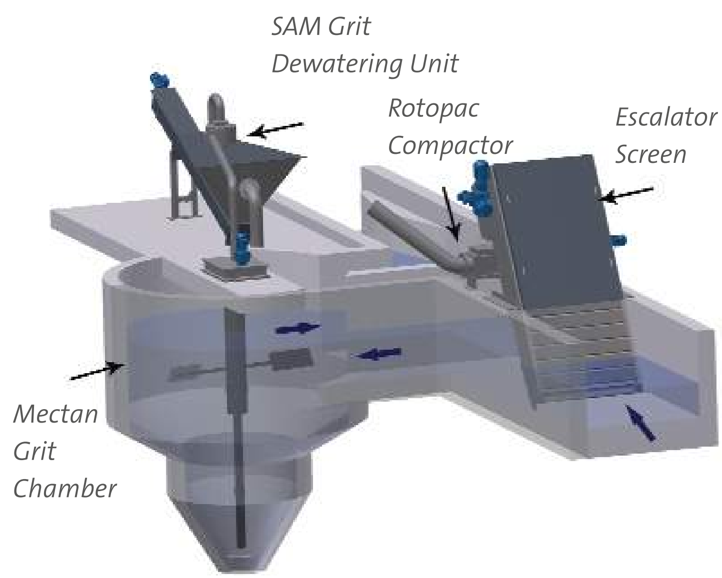
- Top performer in UK study.
- Versatile for wastewater, stormwater and surface water applications.
- Heavy duty, stainless steel construction.
- Intermediate internal grid supports.
- Industry leading tight-tolerance construction.
- Positive sealing at foot via double full-width brushes and neoprene sealing flap.
- Superior cleaning with combined spray wash and mechanical high-speed brush.
- All maintenance from operating floor.
- Available pivoting design.
- John Meunier products’ superior applications engineering, support and service.



1 Perforated screen panel
2 Screenings discharge
3 Rotating brush
4 Screen spraywash
5 Removable cover

Principle of operation

- Flow enters screen at foot and solids are captured by perforated panels.
- Perforated panels convey screenings to operating floor discharge on downstream side of screen.
- Structural shelf on perforated panels lifts larger "unmattable" items.
- High-speed rotating brush and spray wash clean screening from perforated panels.
- Screenings pass through discharge chute into dewatering compactor.



Proven Performance

The screen performance impacts the overall operation and maintenance of the subsequent treatment processes. Between 1998 and 2000, the Escalator screen was evaluated along with other manufacturers' units at the Chester-Le-Street STP, Co Durham UK.

These tests demonstrated that for all inclined screens tested, the Escalator Fine Screen has the highest SCR (Solids Capture Ratio) in the static mode (Off- mode).

Other tests also determined that the Escalator screen is over 97% efficient in limiting solids "Carry Over", thus reducing significantly the amount of traceable solids in the effluent.

Your Pretreatment Specialist

A complete line of John Meunier headworks solutions

Resourcing the world

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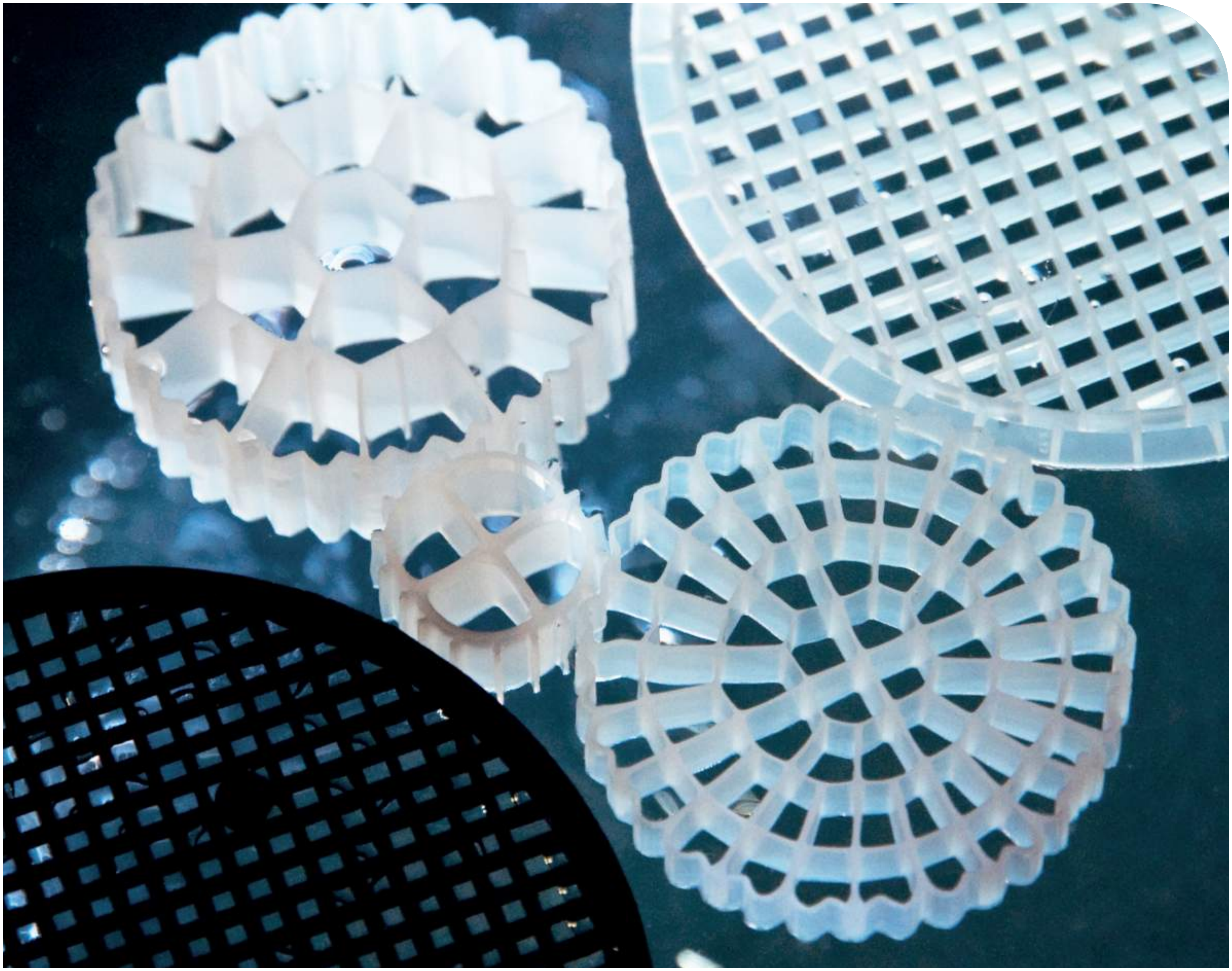


VEOLIA

AnoxKaldnes™ MBBR

WATER TECHNOLOGIES

AnoxKaldnes™ MBBR

Description

The patented MBBR process is based on the biofilm principle and utilizes the advantages of activated sludges and other biofilm systems without being restrained by their disadvantages. The core of the process is the biofilm carrier elements that are made from polyethylene with a density slightly below that of water.

The elements are designed to provide a large protected surface area for the biofilm and optimal conditions for the bacteria culture when the elements are suspended in water. AnoxKaldnes developed carriers of different shapes and sizes which gives us the flexibility to use the best suitable carrier depending on wastewater characteristics, pre-treatment, discharge standards and available volumes.

Features

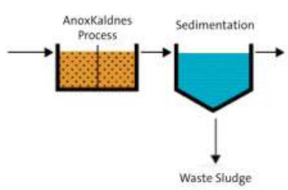
The MBBR Process is feasible for both industrial and municipal wastewater and is used for organic removal, nitrification and denitrification. The flexibility of AnoxKaldnes Moving Bed™ process makes it an ideal solution for new plants or the upgrade of old plants. The process can be delivered as a pure biofilm treatment system or combined with activated sludge to meet nitrification requirements.

- Compact
- Robust Biofilm
- Flexible Reactor Design
- Easy Upgrade for Existing Plants
- Easy to Operate and Control
- No Clogging of Biofilm Carriers
- No Sludge Return
- Low Load on Particle Separation

AnoxKaldnes Stand Alone MBBR Solutions

This process set-up will ensure a compact plant with all the biofilm features and is typically used for:

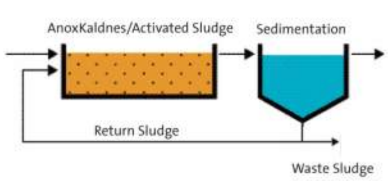
- New BOD/COD removal plants
- New nitrogen removal plant
- Upgrade of existing plants



AnoxKaldnes Moving Bed™ and Activated Sludge in the same reactor (HYBAS™)

This solution combines the benefits of a conventional activated sludge process with biofilm process in the same reactor. This process set-up can be suitable for:

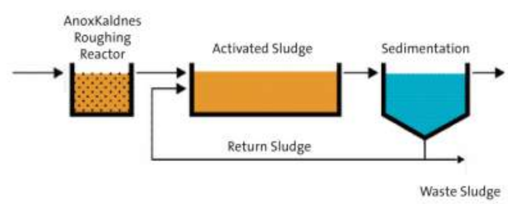
- Upgrading of existing activated sludge to achieve nitrification or higher BOD/COD capacity
- Upgrading of existing activated sludge to achieve nitrogen removal and phosphorus removal



AnoxKaldnes Moving Bed™ followed by Activated Sludge (BAS™)

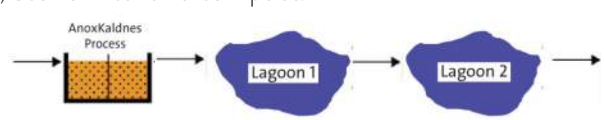
In this combination the biofilm process will work as a pre-treatment "roughing reactor" to reduce the load on the activated sludge reactor. It is typically used for:

- New plants where the biofilm process works as pre-treatment
- Upgrading of existing activated sludge to achieve higher BOD/COD capacity or nitrification



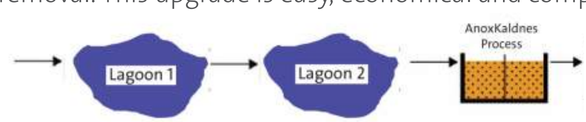
Lagoon Guard (Carbon)

This biofilm solution installed before an aerated lagoon, will handle more additional BOD/COD removal. This upgrade is easy, economical and compact.



Lagoon Guard (Nitrification)

This biofilm solution installed after an aerated lagoon, will handle the ammonium and even some more additional BOD/COD removal. This upgrade is easy, economical and compact.



Flexibility: the key to success!

The flexibility of the AnoxKaldnes Moving Bed™ process has given us more than 500 satisfied customers in over 50 countries. The process is excellent for BOD/COD removal nitrification/denitrification in all types of wastewater. Our reference list includes but is not limited to:

Municipal wastewater

- BOD/COD removal
- Nitrification/denitrification

Industrial wastewater

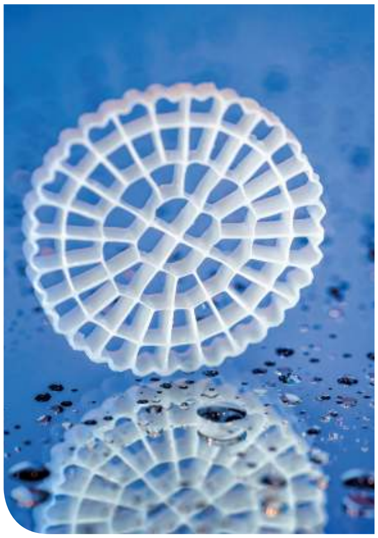
- Food & Dairy
- Pulp & Paper
- Chemical & Pharmaceutical
- Distilleries & Breweries
- Textile & Machinery
- Leachate

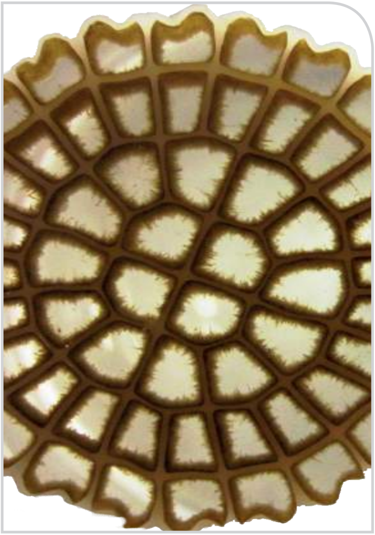
Fish farming

- Water treatment

Why Choose AnoxKaldnes Moving Bed™ Biofilm Reactor Process (MBBR)?

- Increase solids inventory of existing activated sludge system using the AnoxKaldnes "carrier" in a "hybrid plant" to meet ammonia limits
- Excellent for new plants, especially those requiring a small footprint and easy operation, for BOD/COD and nitrogen removal
- Perfect as a high loaded system in front of existing biological treatment - "roughing reactor"
- Utilization of almost any available existing volumes
- Easy implementation of pre- and post treatment to existing plants for process improvements
- Up to 500% increase in capacity of the organic load treated within existing biological volumes
- Ability to use diffused air or pure oxygen for BOD/COD and nitrification applications





Ste. Julie, Quebec : Our first Canadian reference

In the case of Ste. Julie, a full-scale MBBR™ was installed and commissioned in 2007. Even with temperatures as low as 3°C, the removal efficiency of the MBBR was in accordance with the results of the pilot study done 2 years before.

The main objective of the pilot study was to demonstrate the capability of the MBBR process to eliminate organic matter in a separate stage and to reach a level of BOD required during coldest months of winter. The study determined that the MBBR™ solution not only eliminated COD and BOD, but also nitrified the water coming in at low temperature. The average ammonia effluent concentration was 3.9 mg/L, down from an average influent concentration of 16.2 mg/L.

Resourcing the world

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VEOLIA

Hydrotech® Drumfilter

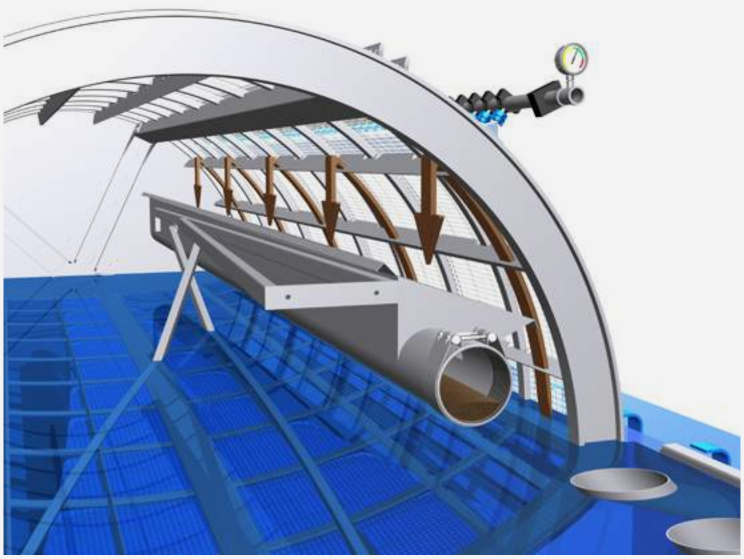
WATER TECHNOLOGIES

Hydrotech Drumfilter

Microscreening

Microscreening is an efficient and reliable technique for separation of particles from all kinds of liquids. Hydrotech develop and manufacture high performance microscreen filters for water purification and product recovery. The Hydrotech Drumfilter is a mechanical and self-cleaning filter specially designed with a view to achieving high performance in systems where it is essential to prevent the particles from fragmentation. The filter works without pressure and is robustly designed with few moving parts to ensure long life and low maintenance costs.

Over 2000 Hydrotech Drumfilters have been installed worldwide.



The liquid is filtered through the periphery of the slowly rotating drum. Assisted by the filter panels special cell structure, the particles are carefully separated from the liquid. Separated solids are rinsed off the filter cloth into the solids collection tray and discharged. Careful handling of the solids to prevent fragmentation is in many applications essential to achieve high filtration efficiency. Hydrotech's unique design of the filter panels makes this possible.

Hydrotech's Unique Construction

ConstructionHydrotech's unique filter panels greatly simplify both the replacement and change out of the filter opening size. The drum is constructed in sections, each with up to 6 filter panels depending on diameter. This facilitates maintenance and makes it easy to adapt the filter to actual need of flow capacity and performance requirements of filtration.Drum and tank are made of stainless (AISI304) or acidproof steel (AISI316L). For use in extremely corrosive surroundings, special alloys, GRP or titanium is used. The filter panels structure, bearing wheels and main shaft bearing are made of resistant plastics. The filter cloth is made of polyester or stainless steel. The operation of the drumfilter can be continuous or automatically controlled. Different types of automatic control systems are available.

Flow capacity: Up to 1000 l/s per filter
Filter opening: 10-1000 µm.

Modular design with very high flexibility

The modular design incorporates 6 different drum diameters from 0.5 m to 2.4 m with filter areas from 0.35 m^2 for the HDF501 to 21.6 m^2 for the HDF2408. In total there are 20 different drum filter sizes. The standard modular filter panel measures 1.2 x 0.4 m. As an example, the 1.6 m drum diameter series has from 1 up to 8 sections, with from 4 up to 32 filter panels.

Available in three drive systems

The Hydrotech Drumfilter is available in two drive system versions; direct drive and chain transmission. The direct drive is used on the two smallest drum sizes with 0.5 and 0.8 m drum diameter. The chain drive version is Hydrotech's well proven solution with filters in operation for more than 10 years.

Picture showing a part of Hydrotech's wide drumfilter product line.

Fields of application

- **Filtering intake water** from streams and lakes for municipal and industrial water supply systems.
- **Polishing effluent** from municipal waste water treatment works. The Hydrotech Drumfilter can also replace e.g. primary clarifiers.
- **Food processing industries** are using the Hydrotech Drumfilter for treatment of waste water and process water.
- **The Hydrotech Drumfilter is suitable in fish farming systems** where it is essential to prevent the particles from fragmentation. This is important specially in recirculated systems and in open systems for intake and outlet water.
- **Other examples** are filtering transport water in plastic industries and purification of scrubber water in power plants.

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JOHN MEUNIER

Hydra-Pol®

Preparation Systems for Polymer Powders

VEOLIA
WATER
Solutions & Technologies

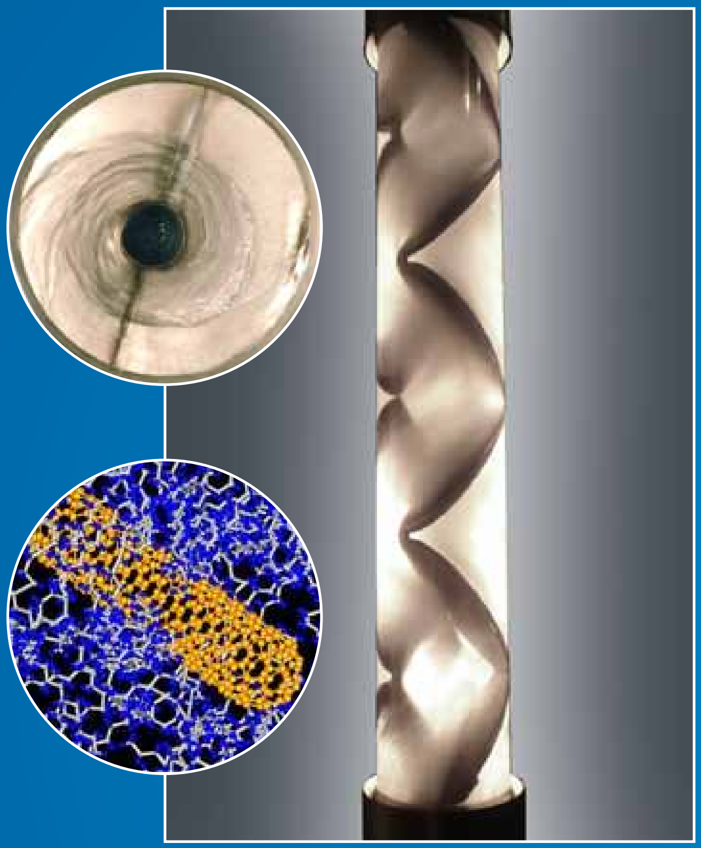
Hydra-Pol®

John Meunier's polymer preparation and dosing system is designed to prepare and activate our many types of dry polymer. The Hapman Posiportion™ volumetric feeder technology introduces dry polymer (from 25 kg bags or super sacks) into the dampening system. The volumetric feeder includes a flexible hopper with external agitators to reduce the formation of arches and associated maintenance. The output of the volumetric feeder is equipped with an automatic shut-off valve which prevents any contact between moisture and dust.

The activation of dry polymers is initiated through an effective high shear pre-wetting stage to enhance the reaction of polymer chains avoiding formation of polymer lumps and clogging. The pre-wetting stage consists of a cone shaped stainless steel vortex for instantaneous dry polymer dispersion in water. Pre-wetted particles of polymer are then transported via an injector, to the mixing tank through a stainless steel transport pipe. The pipeline includes a high efficiency static mixer to ensure complete wetting of polymer before entering the mixing tanks.

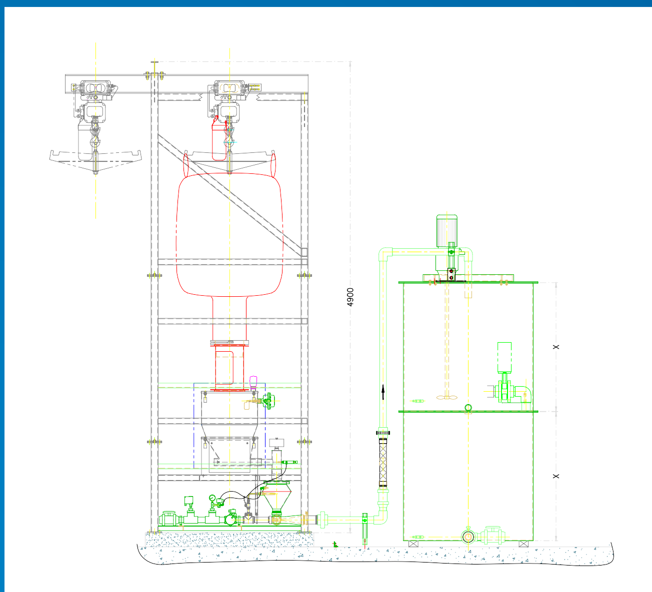
In the mixing tank the polymer is continuously activated with a low shear agitator. JMI customizes the speed of the agitator and the diameter of the propeller, based on the geometry of the tank to ensure optimal activation of polymer. Once the polymer is properly mixed, the solution is transferred to the storage tank. The mixing and storage tanks are cylindrical in design and constructed of stainless steel. They are also mounted on top of each other to reduce total footprint. All operations are fully automated through a customizable control panel for easy management of polymer preparation.

Complete Hydration of the Polymer



Features

- Complete and autonomous system, designed for operators
- Accurate and consistent dosing, providing constant and repeatable concentrations
- Customizable control panel to govern operational sequences
- Handling system adapted to customer needs: vacuum or super-bag unloader
- Efficient design reducing footprint
- Multiple shearing zones ensuring optimal activation of dry polymer
- Easy maintenance



System capacities at a concentration polymer of 0.2% and 0.5% with a time maturation of 90 and 45 minutes.

Capacity (kg/h) :

Systems	0,5% solution		0,2% solution	
Maturation time	90 min	45 min	90 min	45 min
Hydra-Pol 250	0,77	1,46	0,31	0,59
Hydra-Pol 500	1,47	2,74	0,59	1,12
Hydra-Pol 750	2,18	4,02	0,89	1,67
Hydra-Pol 1000	2,81	5,03	1,15	2,11
Hydra-Pol 1250	3,39	6,04	1,40	2,56
Hydra-Pol 1500	4,12	7,35	1,69	3,08
Hydra-Pol 1750	4,66	8,34	1,92	3,52
Hydra-Pol 2000	5,51	9,87	2,25	4,09
Hydra-Pol 2500	6,81	12,17	2,79	5,09
Hydra-Pol 3000	8,52	15,57	3,47	6,43
Hydra-Pol 3500	9,75	17,73	3,98	7,35
Hydra-Pol 4000	10,73	18,92	4,39	7,86
Hydra-Pol 9000	22,17	36,66	9,27	15,80
Hydra-Pol 15000	33,86	51,87	14,43	23,10

New probe for the activation of polymer

- Allows real-time monitoring activation of the polymer
- Confirms the concentration of polymer
- Promotes the preparation optimization of polymer

Patent Pending 13/186,722

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VEOLIA
WATER

Solutions & Technologies

REV2012-07-18

APPENDIX B: TERMS AND CONDITIONS

GENERAL TERMS AND CONDITIONS FOR SALE

The present General Terms and Conditions of Sale of Products ('Terms and Conditions') govern the supply of Products and Services (collectively the "Products") by Veolia Water Technologies Canada Inc. herein defined as "Veolia Canada". These terms and Conditions shall prevail over the Customer's terms and conditions of purchase whether or not provided to Veolia Canada. Neither commencement of performance nor delivery by Veolia Canada shall be construed as or constitute acceptance of Customer's terms and conditions of purchase. The present Terms and Conditions shall not be amended without Veolia Canada's prior consent in writing.

1. Definition and interpretation

1.1 In the present Terms and Conditions:

'Customer' means a person to whom an Offer is made or to whom Products are supplied; Veolia Canada means Veolia Water Technologies Canada Inc. Veolia Canada and Customer shall be defined hereinafter individually or collectively as Party or Parties; 'Delivery Date' means the date set for delivery in the Offer or the Order, and if such Offer and Order conflict in such respect, then the date set out in the Offer unless agreed in writing by the parties; 'Intellectual Property' means all forms of intellectual property rights including patents, designs, copyright, trademarks, trade names, trade secrets or any other intellectual or industrial property right, whether registered or unregistered related to the Products; 'Offer' means an offer by Veolia Canada to supply Products; 'Order' means an effective contract to supply Products as per article 3 to which these Terms and Conditions apply; 'Products' means goods, spare parts, consumables, equipment or materials, and services as the case may be supplied by Veolia Canada to the Customer pursuant to an Order; 'Work' means the delivery of Products to the agreed point of delivery, and any installation or other related activities included in the Order. 1.2 In the present Terms and Conditions: a) clause headings and bold characters are for convenience only and shall not affect interpretation thereof; b) words importing the singular include the plural and vice versa; and c) words importing a gender include any gender.

2. Offer

2.1 Veolia Canada may vary the content of the Offer at any time before its acceptance.2.2 Unless otherwise stated in the Offer; the Offer remains open for acceptance for thirty (30) days after its date, but may be withdrawn by Veolia Canada at any time before acceptance.

3. Effective date

3.1 The Order shall become effective upon Veolia Canada's written acceptance of the Customer's Order, unless otherwise agreed between the Parties.

4. Cancellation

The Customer may not cancel any Order unless the Customer: a) obtains Veolia Canada's prior written approval; and b) pays Veolia Canada all costs incurred or damages suffered by Veolia Canada in connection with the cancellation of the Order (including without limitation any charges, termination costs, duties, taxes, expenses, design costs, expected profits, purchasing costs or other outgoings paid or incurred in expectation of the completion of the Order). Products returned without Veolia Canada prior written consent will not be accepted for credit.

5. Variations and Change in Law

5.1 If the Customer requests in writing a variation to an Order: a) Veolia Canada will use its reasonable efforts to comply with the request; and b) if Veolia Canada can comply with the request: i) the Customer shall pay Veolia Canada the costs reasonably invoiced for the variation; ii) Veolia Canada will advise the Customer of any delivery delay resulting from complying with the request; and iii) Veolia Canada will advise the Customer of any impact on the warranties given in respect of the Products. 5.2 Any attempt by the Customer to unilaterally vary the content of an Order (including these Terms and Conditions), whether orally or in writing, is void. Veolia Canada shall begin work related to the Variation unless agreement is reached between the Parties. Veolia Canada shall be entitled to compensation for any change in law having effect on the performance of the Order.

6. Price and payment

6.1 The price of Products shall be specified in the Offer to the Customer. Except as may be otherwise provided in an Offer, the price does not include any goods and services or consumer sales tax, and/or other similar taxes, excise and custom duties, required by law in the jurisdiction of delivery of the Products or otherwise. The Customer shall bear sole responsibility for the payment of any such tax or duty. 6.2 The price shall be subject to adjustment upon an increase in the cost of raw materials and/or wages according to the formula determined by Veolia Canada in its sole discretion, and upon written notice to the Customer. 6.3 Unless specified otherwise in writing, terms of payment are 100%, net 30 days. 6.4 Customer shall be charged 2% interest per month (24% per year) of any unpaid balance, and Customer shall pay all of Veolia Canada's reasonable costs (including attorneys' fees) of collecting amounts due but unpaid. All orders are subject to credit approval. 6.5 All above prices are in Canadian Dollars; 6.6 Nothing in the provisions of clause 6.4 above shall limit any right Veolia Canada may otherwise have to recover payment of amounts due and/or damages.

7. Delivery and risks

7.1 Unless otherwise stated in an Order: a) Veolia Canada shall deliver the Products Ex Works; and b) the Customer must arrange to pick up the Products immediately upon the Delivery Date; and c) all risks including risk of loss or damage and care and custody to the Products shall pass to the Customer upon delivery as per a) above. Any use of the Products before acceptance other than at the time of the tests carried out in the presence of the Contractor shall be deemed to be Provisional Acceptance of the Work and shall automatically result in the immediate transfer of risk and the beginning of the warranty period.

8. Ownership of the products

8.1 Subject to clause 8.2 below, Veolia Canada shall provide full and unrestricted title to the Customer for the Products free and clear of all liens, restrictions, reservations, security interests and encumbrances (save as for the intellectual property rights associated with the Products). 8.2 Ownership of the Products only passes to the Customer when all of the Products under the said Order are paid for in full. Until then: a) ownership of the Products remains with Veolia Canada; b) the Customer holds the Products as bailer for Veolia Canada; and c) the Customer shall maintain Veolia Canada's identification property signs on the Products.

9. Warranty

9.1 Unless otherwise stated in the Offer: Veolia Canada Products shall be guaranteed to be free from faulty materials, workmanship or defects for a fixed period of eighteen (18) months from the Delivery Date or (12) months from the date of substantial performance, whichever period expires the earliest. 9.2 The present warranty is subject to prior notification by the Customer to Veolia Canada within ten (10) business days after the discovery of the defect. 9.3 During the warranty period Veolia Canada will, at its sole discretion, either: a) repair or replace Ex-Works, or b) pay to the Customer the cost of replacing or repairing, at Customer's risk, that part or all of the Products which are reasonably found to be defective. Repair and/or replacement of Products shall not constitute an extension of the warranty period. 9.4 Customer's failure to notify Veolia Canada pursuant to clause 9.2 above shall constitute acknowledgement of compliance of the Products with the Order and the Customer shall then be deemed to have waived any such claim in relation to the Products. 9.5 Save and except for warranties expressly stated in the Offer, THE WARRANTIES EXPRESSLY SET FORTH IN THESE TERMS AND CONDITIONS ARE THE SOLE AND EXCLUSIVE WARRANTIES OF VEOLIA CANADA. VEOLIA CANADA MAKES NO OTHER WARRANTIES OF ANY KIND WHATSOEVER, EXPRESS, IMPLIED, ORAL, WRITTEN, STATUTORY OR OTHERWISE, INCLUDING, WITHOUT LIMITATION, THE IMPLIED WARRANTIES OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE, OR WARRANTIES ARISING BY CUSTOM, TRADE USAGE, PROMISE, EXAMPLE OR DESCRIPTION, ALL OF WHICH WARRANTIES ARE EXPRESSLY DISCLAIMED BY VEOLIA CANADA AND WAIVED BY THE CUSTOMER. The warranty provided for in the present clause shall not be extended, altered or varied except by a written instrument signed by Veolia Canada and the Customer.

10. Exclusions from warranty

10.1 The foregoing warranty shall only apply in respect of claims as a result of defects in the Products or parts thereof which become apparent within the applicable warranty period. 10.2 Veolia Canada shall not be liable in any way, whether in contract, tort, under statute or otherwise, for any failure of the Products to comply with the warranties given under clause 9 and, (if applicable) under the express terms of the Offer: a) unless the Customer can prove, to Veolia Canada' satisfaction, that the Customer stored, installed, used and operated the Products strictly in accordance with Veolia Canada' instructions (which the Customer will receive, or must request and receive before installation –if not performed by Veolia Canada- and initial use of the Products); or b) if the failure is caused by: i) normal wear and tear, impact, improper use, or mishandling; or ii) repair, alteration or use beyond their specifications, iii) repair or modification in any way by any person other than Veolia Canada; iv) a force majeure event. For the purposes of clarification, the warranty provided by Veolia Canada in respect of the Products or the Work does not cover normal wear and tear. 10.3 The Customer acknowledges that: a) in order to comply with its warranty obligations, Veolia Canada shall not be obliged to make any change in the design and/or specifications of the delivered Product so as to render the said Product equivalent to any other new similar Product, or new model of the Product, supplied by Veolia Canada (but the Customer agrees to accept such new model of the Product or replacement for the Product if offered by Veolia Canada); and b) Veolia Canada shall not be responsible for the replacement of consumable and spare parts items used in operation of the Products.

11. Exclusions and limitation of liability

11.1. The total and aggregate liability of Veolia Canada to the Customer, whether in contract, tort (including negligence), statute or under any other legal theory whatsoever shall in no event exceed twenty-five percent (25%) of the Order price. 11.2 Veolia Canada shall in no event be held liable to Customer for any indirect, special, punitive or consequential damages whatsoever arising under the Order, including any loss of profits, loss of revenues, loss of opportunities, loss of use, loss of production, loss of contracts.. 11.3 The present clause 11 shall apply notwithstanding any other provision of any Order.

12. Purpose of products

12.1 The Customer acknowledges it relies solely on its own skill and judgment in all respects and in particular: a) in its decision to purchase the Products; and b) that the Products are fit for the purpose for which they are being acquired. 12.2 It is the Customer's sole responsibility to ensure that the Products are used for the purposes for which they were intended to be used.

13. Force Majeure

13.1 Veolia Canada shall not be held liable for any delay or failure in performance of any part of the Order to the extent that such delay or failure is caused by an event of force majeure, being an occurrence (other than in respect of the financial capability of a party) which prevents or delays a party from performing its obligations and which is beyond the reasonable control of such party; and which shall include, without limitation: accidental damage to its equipment or machinery; acts of God or of public enemy; blockade, rebellion, insurrection, riot or other civil unrest or violence or sabotage; weather conditions, fire, storm, flood, earthquake, or other natural disaster; terrorism, bomb or explosion; war; illness or epidemic; quarantine restrictions; industrial or labor dispute, labor shortage; transportation embargo; act or omission (including laws, regulations, disapprovals or failures to approve) of any other person (including a government, government agency, a supplier or a sub-contractor). 13.2 If any such event occurs, and Veolia Canada is delayed or unable to perform, Veolia Canada shall give notice to the Customer, and shall be automatically relieved from performance of the Order for the entire duration of such event. 13.3 If the said event lasts for more than thirty (30) days, Veolia Canada shall have the right to terminate the Order with immediate effect by giving written notice to the Customer. 13.4 If Veolia Canada terminates an Order under this clause 13.3 due to a Force Majeure event as described in 13.1 affecting the Customer; the Customer shall pay Veolia Canada all costs incurred or damages suffered by Veolia Canada in connection with the Order (including without limitation any charges, duties, taxes, expenses, design costs, purchasing costs or other outgoings paid or incurred in the expectancy of completion of the Order).

14. Export control

Unless otherwise agreed by the parties in writing, and to the extent applicable to the Work, the Customer is responsible for compliance with all laws and regulations applicable to the storage, use, handling, installation, maintenance, removal, registration and labeling of all Products from and after Customer's receipt of the Products, as well as for the proper management and disposal of all wastes and residues associated with the Products (including but not limited to containers, excess or off-spec product, testing wastes, e.g., spent or expired lab reagents and test kits). Customer agrees to ensure that all Products provided to Customer for export are exported only in compliance with applicable export control laws and regulations. Any permits and licenses which are required to operate or to use the Products shall be procured by Customer at Customer's sole expense.

15. Intellectual property

The Customer acknowledges that Veolia Canada preserves all the intellectual property rights on all Products of the Order. Accordingly, the plans, technical drawings and specifications supplied by Veolia Canada and more generally any documents or information communicated in conformance with the Order remain the full and whole property of Veolia Canada and can in no way be used by the Customer for any other purpose other than that set out in the Order. As such, Veolia Canada grants to the Customer a non-exclusive license to use such documents exclusively for the purpose of installing, maintaining and repairing the Products. During the execution and for five years following the termination date of the Order, the Customer commits not to reveal to any third party, officially or not, directly or indirectly, in writing or by other means, all or any of the information which would have been communicated to the Customer by Veolia Canada within the framework of the Order, except if the Customer obtains Veolia Canada' prior written approval. The term "information" includes, without limitation, the knowledge, the plans and the worksheets, and generally, all the technical, financial or commercial information that was exchanged or communicated in relation to the Order.

16. Customer's default

16.1 If: a) the Customer fails to make any payment required under the Order, including interests and any other amount owing to Veolia Canada, on the date or dates due; b) the Customer breaches any other provision of the Order, Offer or of the present Terms and Conditions and fails to remedy the breach within seven (7) days after receiving a written notice requiring it to do so; or c) any step is taken to appoint a receiver, a receiver and manager, a trustee in bankruptcy, a liquidator, a provisional liquidator, an administrator or other like person in respect of part or all of the Customer's assets or business, Veolia Canada may: i) declare the entire sum remaining unpaid under the Order to immediately become due and payable; or ii) require the Customer to pay in advance of delivery or completion; or ii) suspend or cease performance until all amounts owing to Veolia Canada are paid in full; or iii) request the Customer to immediately return to Veolia Canada any Product for which full payment has not been received by Veolia Canada; or iv) enter the premises in which the Products are stored and retake possession of them; and/or v) resell all or part of the Products without notice. 16.2 This clause shall not limit any other right Veolia Canada may have to recover damages for breach of contract or any other claim under statute or at common law. For greater certainty, no failure or partial exercise of any remedy or delay in exercising any remedy, shall operate as a waiver thereof; the rights and remedies herein provided are cumulative and may be exercised singly or concurrently, and are not exclusive of any rights or remedies provided by law. 16.3 Further to the foregoing, in the event of any one of the occurrences described in 16.1a) to c), Veolia Canada may also elect to terminate the contract in relation to the Order without prejudice to its right to claim all payment owed under the Order and under the present terms and conditions.

17. Early Termination

The Customer shall pay Veolia Canada, at the latest within 30 calendar days following the effective date of termination of the Order, the value of the Work conducted, performed or delivered on the Site in accordance with the Order and all the amounts remaining due to Veolia Canada on the date of termination and any early termination costs incurred or expected by Veolia Canada.

18. Applicable law

Veolia Canada and the Customer agree that the Offer, the Order and these Terms and Conditions shall be governed in accordance with Canadian federal laws and the applicable provincial laws in which delivery occurs (the "Province"). For any delivery outside of Canada, the laws of the province of Québec shall apply. All disputes arising between the parties in respect of such Offer, Order or Terms and Conditions shall be settled by arbitration, in the city of Montreal, Québec unless otherwise agreed to by the Parties.

20. Notices

20.1 All notices required to be given under the Order must be sent to the address of the recipient as set out in the Order (or any other address notified in writing by the recipient in accordance with the present clause). 20.2 Any notice will be deemed to have been duly given, if sent by mail, five (5) business days after posting, if delivered by hand, on signature of receipt acknowledging delivery and, if sent by facsimile transmission, on generation of an acknowledgment that the transmission has been successfully completed.

Appendix C

Sanitaire SBR Proposal

DESIGN PROPOSAL

Nelson, BC Sanitaire #28079-17ac

		Max Month*		20 Year	
Operating Mode		Max Month*		20 Year	
Normal Cycle Flow	M^3/DAY	5,500		7,800	
Max Normal Cycle Flow	M^3/DAY	7,300		10,300	
Minimum Cycle Flow	M^3/DAY	14,600		20,600	
		mg/l	kg/day	mg/l	kg/day
BOD_5 (20°C)		252	1387	241	1882
Suspended Solids		145	794.493	138	1079
TKN		33	183	32	248
Total Phosphorus		6	33	6	47
Max Wastewater Temperature	°C	20		20	
Min Wastewater Temperature	°C	6		6	
Ambient Air Temperature	°C	-6 - 32		-6 - 32	
Site Elevation	M	535		535	

*** - Maximum 30 day period mass flow**

Table B: ICEAS® EFFLUENT QUALITY (MONTHLY AVERAGE)

BOD_5 (20°C)	mg/l	15	15
Suspended Solids	mg/l	15	15
NH_3-N	mg/l	1.25	1.25

Table C: ICEAS PROCESS DESIGN CRITERIA

Operating Basins		4	6
Operating Top Water Level	M	6.50	6.41
F / M	BOD5/DAY/MLSS	0.108	0.108
SVI (after 30 minutes settling)	ml/g	150	150
MLSS at Bottom Water Level	mg/l	4,050	3,671
Waste Sludge Produced (Approx.)	kg/day	917	1,244
Volume of Sludge Produced (Approx., 0.85% solids)	M^3/DAY	108	146
Normal Decant Rate	M^3/min	5.07	4.99**
Peak Decant Rate	M^3/min	10.14	9.54**
Hydraulic Retention Time	Days	0.70	0.74
Sludge Age	Days	13.3	13.3
Alkalinity	mg/l	137	134

Bold, italicized text indicate assumptions made by Sanitaire

**Note that this flow is coming from two basins simultaneously 50% of the time due to overlapping decant cycles.

Cycle Timing

		Max Month*		20 Year	
		Normal	Min	Normal	Min
Air-On	min	120	60	120	60
Settle	min	60	30	60	30
Decant	min	60	30	60	30
Total	min	240	120	240	120

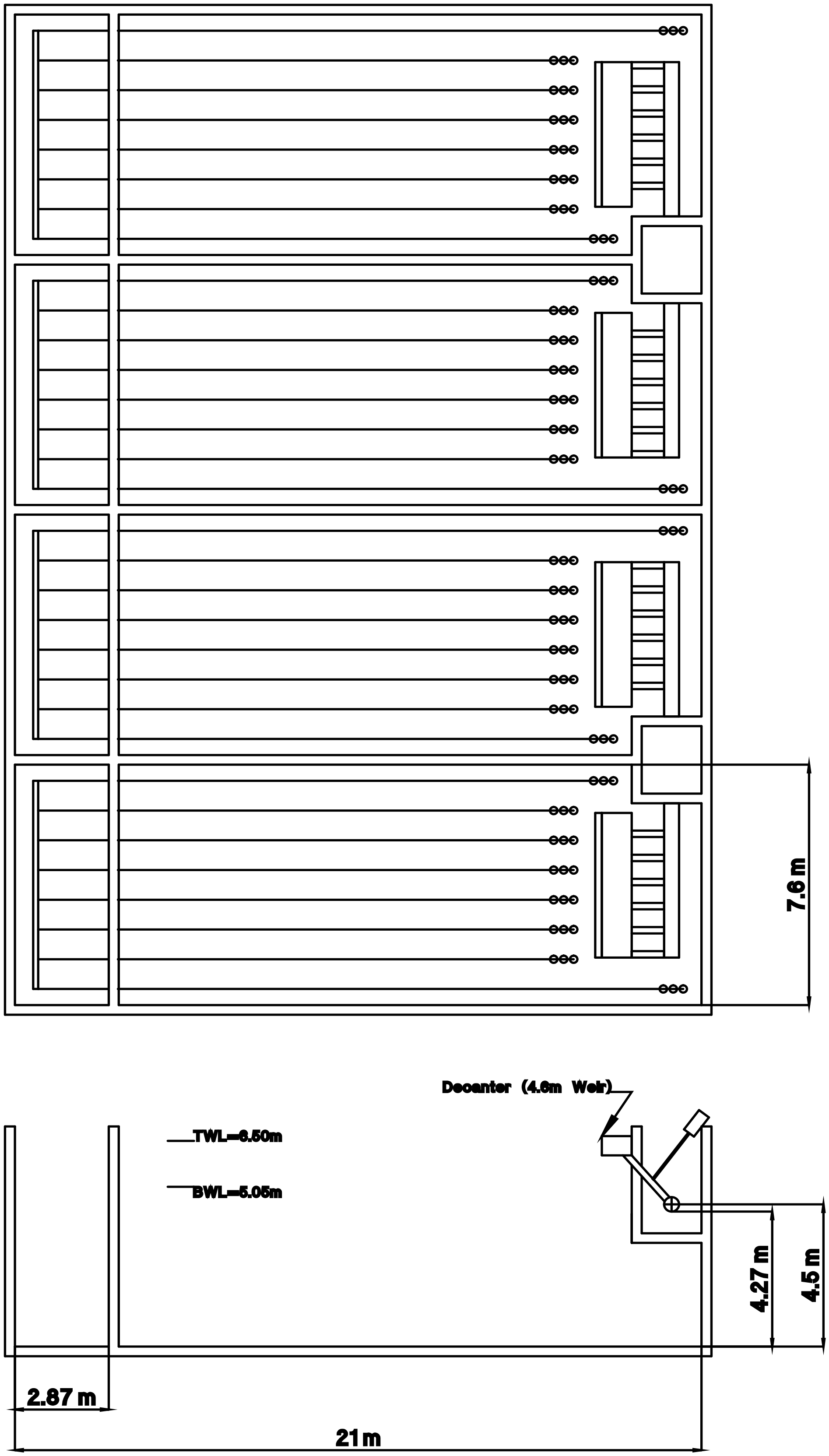
Table D: KEY ICEAS DESIGN DETAILS

Top Water Level	M	6.50
Basin Width (Inside)	M	7.6
Basin Length (Inside)	M	21.0
Bottom Water Level	M	5.05

ICEAS EQUIPMENT(Base Design)						Motor HP	No. Req.
Decanter Mechanism	4.57	m Weir length					4
Decanter Drive Unit						1/2	4
ICEAS Blower	1,190	M^3/HR	66.2	KPAG		60	3
ICEAS Fine Bubble Aeration System	496	Disc Diffusers/Basin					4
Air Control Valve	200	mm					4
Waste Sludge Pump	416	L/min				2.4	4
ICEAS Controls (including main panel, HMI, VFDs, motor starters, and DO Control)							1

ICEAS POWER REQUIREMENTS	Max Month		(At Average Aeration Depth)					Kwh/Day
Decant Drive Unit	0.4	BHP	4	run	@	6	Hrs/day	7.2
ICEAS Air Blowers	43.2	BHP	2	run*	@	24	Hrs/day	1,548.5
Waste Sludge Pump	1.9	BHP	4	run	@	1.1	Hrs/day	6.2
						KWH/DAY		1,561.9
					AVERAGE	KWH/HR		65.08

* Shared ICEAS Blowers



PRELIMINARY - THIS DRAWING IS NOT INTENDED FOR CONTRACT DOCUMENTS, SUBMITTALS, OR CONSTRUCTION

Sanitaire a xylem brand BROWN DEER, WISCONSIN 53223	CUST NO. DWG NO.	THIS DRAWING IS THE PROPERTY OF XYLEM AND IS SUBMITTED IN CONFIDENCE. IT IS NOT TO BE DISCLOSED, USED OR DUPLICATED WITHOUT PERMISSION OF XYLEM.	Nelson, BC PRELIMINARY LAYOUT ICEAS System	DRAWN BY DM CHKD BY APPVD BY	DATE 10/3/17 DATE DATE	JOB 28079-17ao SHEET

SANITAIRE ICEAS Detailed Design Calculations
BOD Removal and Nitrification Process

SANITAIRE Project #28079-17ac
Nelson, BC

Design Parameters

A. Flow

Max Month	5,500	m^3/day
Max 4.0hr Cycle Flow	7,300	m^3/day
Max 2.0hr Cycle Flow	14,600	m^3/day

B. Treatment

	Influent Quality	Effluent Requirement
BOD_5 (20°C), mg/l	252.3636364	10
Suspended Solids, mg/l	144.5454545	10
TKN, mg/l	33.27272727	
NH_3-N, mg/l		2
TN, mg/l		
Phosphorus	6	

C. Environment

Alkalinity (Minimum Requirement)	140	mg/l
Max Wastewater Temperature	20	°C
Min Wastewater Temperature	6	°C
Ambient Air Temperature	-6.7 - 32.2	°C
Site Elevation	535	m

D. ICEAS Process Design Criteria

F / M	0.108	BOD_5 / MLSS / day
SVI (after 30 minutes settling)	150	ml/g
Number of ICEAS Basins	4	
Top Water Level	6.5	m

E. Cycle Timing

		Normal	Storm
Air-On	min	120	60
Air-Off	min		
Settle	min	60	30
Decant	min	60	30
Total	hrs	4	2

F. Detailed Calculations

Mass of BOD

$$BODL = \frac{Q \times BODin}{1{,}000} = \frac{1{,}375 \times 252}{1{,}000} = \mathbf{347\ kg/day/basin}$$

where: BODL = BOD Load (kg/day/basin)
Q = Average Dry Weather Flow per basin (m^3/day)
BODin = Influent BOD concentration (mg/l)
1,000 = Conversion (l/m^3)

Mass of Biomass

$$BMOB = \frac{BOD_L}{F / M} = \frac{347}{0.1078} = \mathbf{3{,}216\ kg/basin}$$

where: BMOB = Mass of Biomass (kg/day/basin)
F / M = Food to Microorganism ratio (day^{-1})

Volume of Biomass

$$Vbio = BMOB \times SVI = 3{,}216 \times 0.15 = \mathbf{482\ m^3/basin}$$

where: Vbio = Volume of Biomass (m^3/basin)
SVI = Sludge Volume Index (m^3/kg)

Maximum Volume Above Bottom Water Level

Peak Dry Weather Flow:

$$Vbwld = \frac{PDWF \times (NCT - NDT)}{24} = \frac{1{,}825 \times (4.0 - 1.00)}{24} = \mathbf{228\ m^3/basin}$$

where: Vbwld = Maximum Volume Above BWL at Peak Dry Weather Flow (m^3/basin)
PDWF = Peak Dry Weather Flow (m^3/day)
NCT = Normal Cycle Time (hr/cycle)
NDT = Decant Time (hr/cycle)

Peak Wet Weather Flow:

$$Vbwls = \frac{PWWF \times (SCT - SDT)}{24} = \frac{3{,}650 \times (2.0 - 0.50)}{24} = \mathbf{228\ m^3/basin}$$

where: Vbwls = Maximum Volume Above BWL at Peak Wet Weather (Storm) Flow (m^3/basin)
PWWF = Peak Wet Weather Flow (m^3/day)
SCT = Storm Cycle Time (hr/cycle)
SDT = Storm Decant Time (hr/cycle)

MVAB (Maximum Volume Above Bottom Water Level) is larger of Peak Dry Weather and Peak Wet Weather Calculation

$$MVAB = \mathbf{228\ m^3/basin}$$

Decant Rates

Peak Dry Weather Flow:

$$PDR = \frac{MVAB}{NDT} + \frac{PDWF}{1{,}440} = \frac{228}{60.0} + \frac{1{,}825}{1{,}440} = \mathbf{5.07\ m^3/min}$$

where: PDR = Normal Decant Rate (m^3/min)
NDT = Normal Decant Time (min/cycle)
1440 = Conversion (min/day)

Peak Wet Weather Flow:

$$PWR = \frac{MVAB}{SDT} + \frac{PWWF}{1{,}440} = \frac{228}{30.0} + \frac{3{,}650}{1{,}440} = \mathbf{10.14\ m^3/min}$$

where: PWR = Peak Decant Rate (m^3/min)
SDT = Storm Decant Time (min/cycle)

Decanter Sizing

Peak Dry Weather Flow:

$$DLa = \frac{PDR}{\text{Weir Loading Rate}} = \frac{5.07}{1.86} = \mathbf{2.73\ m}$$

where: DLa = Decanter Length for Average Dry Weather Flow (m)
1.86 = Weir Loading Rate (m^3/min/m of decanter weir)

Peak Wet Weather Flow:

$$DLp = \frac{PWR}{\text{Weir Loading Rate}} = \frac{10.14}{2.23} = \mathbf{4.55\ m}$$

where: DLp = Decanter Length for Peak Wet Weather (Storm) Flow (m)
2.23 = Weir Loading Rate (m^3/min/m of decanter weir)

Design Decanter Length = **4.6 m**

Basin Working Volume

$$BWV = MVAB + Vbio = 228 + 482 = \mathbf{710\ m^3/basin}$$

where: BWV = Basin Working Volume (m^3/basin)

Basin Area

$$BA = \frac{BWV}{TWL - BZ} = \frac{710}{6.5 - 2.0} = \mathbf{157\ m^2/basin}$$

where: BA = Basin Area (m^2)
TWL = Top Water Level (m)
BZ = Buffer Zone (m) (Safety Factor)

Sludge Depth

$$SD = \frac{Vbio}{BA} = \frac{482}{157} = \mathbf{3.06\ m}$$

where: SD = Sludge Depth (m)

Decanter Draw Down

$$DD = \frac{MVAB}{BA} = \frac{228}{157} = \mathbf{1.45\ m}$$

where: DD = Draw Down (m)

Bottom Water Level

$$BWL = SD + BZ = 3.06 + 1.99 = \mathbf{5.05\ m}$$

where: BWL = Bottom Water Level (m)
Vd = Depth of Chemical Sludge for Phosporus precipitation (m)

Top Water Level

$$TWL = BWL + DD = 5.05 + 1.45 = \mathbf{6.50\ m}$$

where: TWL = Top Water Level (m)

Hydraulic Retention Time

$$HRT = \frac{BA \times MAFD}{QT}$$

where: HRT = Hydraulic Retention Time (days)
MAFD = Maximum Average Flow Depth (m)
QT = Fill Rate at Average Dry Weather Flow (m^3/day)

$$MAFD = \frac{Q \times [(NCT \times 60) - NDT]}{BA \times 1{,}440} + BWL = \frac{1{,}375 \times [(4.0 \times 60) - 60.0]}{157 \times 1{,}440} + 5.05 = \mathbf{6.14\ m}$$

$$HRT = \frac{157 \times 6.14}{1{,}375} = \mathbf{0.70\ days}$$

MLSS Concentration at Bottom Water Level

$$MLSS = \frac{Mbio \times 1{,}000}{BWL \times BA} = \frac{3{,}216 \times 1{,}000}{5.05 \times 157} = \mathbf{4{,}050\ mg/l}$$

where: MLSS = Mixed Liquor Suspended Solids concentration at Bottom Water Level (mg/l)
1E+03 = Conversion (g/kg)

Mass of Sludge Produced

$$\Delta M = \left(\frac{Y \times (BOD_{in} - BOD_{out})}{1 + (B \times \theta^{(T-20)} \times SRT)} + Zio + Zno \right) \times \frac{Q}{1{,}000}$$

$$\Delta M = \left(\frac{0.6 \times (252 - 10.0)}{1 + (0.07 \times 1.04^{(6-20)} \times 13.3)} + 28.9 + 43.4 \right) \times \frac{1.4E{+}03}{1{,}000} = \mathbf{229\ kg/day/basin}$$

(Lawrence-McCarty Equation as presented in WEF MOP/8 4th Edition, pg 11-11, Eqn. 11.7)

where: ΔM = Mass of Sludge Produced (kg/day/basin)
Y = Volatile cell yield (VSS/BOD removed)
q = Arrhenius Temperature Correction Factor
B = Decay Rate (day^{-1})
BOD_{out} = Anticipated Effluent BOD (mg/l)
SRT = Solids Retention Time (days)
Zio = Nonvolatile Influent suspended solids (mg/l)
Zno = Volatile Non-Biodegradable solids (mg/l)
T = Minimum Wastewater Temperature (°C)

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Volume of Sludge Produced

$$V_{ws} = \frac{\Delta M}{SFws \times 1{,}000} = \frac{229}{0.0085 \times 1{,}000} = \textbf{27 m}^3\textbf{/day/basin}$$

where: Vws = Volume of Waste Sludge (m^3/day/basin)
SFws = Solids Fraction in Waste Sludge
8.34 = Density (kg/m^3)

Observed Yield Factor

$$Yobs = \frac{\Delta M}{BOD_L} = \frac{229}{347} = \textbf{0.66} \; \frac{\textbf{MLSS}}{\textbf{BOD}}$$

where: Yobs = Observed Yield Factor (kg/day MLSS/kg/day BODremoved)

Mean Cell Residence Time

$$MCRT = \frac{Mbio}{\Delta M + ((Q - Vws) \times TESS / 1E{+}03)}$$

$$MCRT = \frac{3{,}216}{229 + ((1{,}375 - 27) \times 10.0 / 1{,}000)} = \textbf{13.3 days}$$

where: MCRT = Mean Cell Residence Time (days)
TESS = Anticipated Effluent Total Suspended Solids (mg/l)
3.79E-03 = Conversion (lb/mg x l/gal)

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Sludge Age for Nitrification

Refer to Metcalf and Eddy, Edition IV pages 614 and 705

Constants and Temperature Corrections:

Coefficient	Base Value	Theta	Temperature Corrected	Symbol
Maximum Specific Growth Rate of Nitrifying bacteria, g VSS/g VSS.day	0.75	1.07	0.291	$\mu_{nm}(T)$
Half-Velocity constant for nitrifiers	0.74	1.053	0.359	Kn(T)
Nitrifier decay rate	0.08	1.04	0.046	Kdn(T)
Dissolved Oxygen, mg/l	2		2	DO
Half-Velocity Constant for Dissolved Oxygen, mg/l	0.5		0.5	Ko
Minimum Water Temperature, °C	6		6	T
Safety Factor	1.0		1.0	SF

Calculations:

$$\mu_n = \left(\mu_{nm}(T) \times \frac{TENH_3}{TENH_3 + Kn(T)} \times \frac{DO}{DO + Ko} \right) - Kdn(T)$$

$$\mu_n = \left(0.291 \times \frac{2.0}{2.0 + 0.359} \times \frac{2.0}{2.0 + 0.5} \right) - 0.046 = \mathbf{0.151\ days^{-1}}$$

$$SRTmin = \frac{1}{\mu_n} = \frac{1}{0.151} = \mathbf{6.6\ days}$$

$$SRTaerobic = SRTmin \times SF = 6.6 \times 1.0 = \mathbf{6.6\ days}$$

$$SRToverall = \frac{SRTaerobic \times 24}{TA} = \frac{6.6 \times 24}{12.0} = \mathbf{13.2\ days}$$

Design sludge age adequate for nitrification.

where: $\mu nm(T)$ = Maximum Temperature Corrected Nitrifier Growth Rate ($days^{-1}$)

μ_n = Specific Nitrifier Growth Rate at Temperature, DO, and Effluent NH_3 (g/g-days)

SRTmin = Minimum Sludge age required for Nitrification (days)

SRTaerobic = Design Aerobic Sludge Age (days)

SF = Safety Factor

SRToverall = Sludge Age accounting for entire ICEAS cycle (days)

TA = Aeration Time (hrs/day)

$TENH_3$ = Anticipated Effluent Ammonia (mg/l)

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Waste Sludge Pump Capacity

$$WSP = \frac{Vws \times 1{,}000 \times NCT}{24 \times SPT} = \frac{27 \times 1{,}000 \times 4.0}{24 \times 10.80} = \mathbf{416\ l/min}$$

where: WSP = Waste Sludge Pump Capacity(l/min)
SPT = Sludge Pumping Time (min/cycle)

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SANITAIRE ICEAS Aeration Design Calculations
BOD Removal and Nitrification Process

SANITAIRE Project #28079-17ac
Nelson, BC

Carbonaceous Oxygen Demand

$$AOR1 = A \times \frac{Q \times BODin}{1{,}000} = 1.20 \times \frac{1{,}375 \times 252}{1{,}000} = \mathbf{416\ kg/day/basin}$$

where AOR1 = Actual Oxygen Required for BOD oxidation (kg/day/basin)
A = O2 / BOD
Q = Average flow (m^3/day/basin)
BODin = Influent BOD received (mg/l)
1,000 = Conversion (l x m^3)

Nitrification Oxygen Demand

$$AOR2 = TKNox \times 4.60 = 24.7 \times 4.60 = \mathbf{114\ kg/day/basin}$$

where AOR2 = Actual Oxygen required for Ammonia Oxidation (kg/day/basin)
TKNox = Nitrogen available for oxidation(kg/day/basin)

Constants

Coefficient	Value	Symbol
VSS/TSS	0.8362	
Sludge N	0.07	Ns
Effluent Dissolved Organic Nitrogen, mg/l	1	EDON
Expected Effluent Ammonium concentration	2	$TENH_3$

$$TKN_{ox} = (TKN - EDON - TENH_3 - N_{assim} - N_{part}) \times Q \div 1{,}000$$

$$TKN_{ox} = (33.2727272727273 - 1 - 2 - 11.68 - 0.59) \times 1{,}375 \div 1{,}000 = \mathbf{24.7\ kg/day/basin}$$

where N_{assim} = Nitrogen assimilated into biomass, (mg/l)

$$N_{assim} = BOD_{in} \times N_s \times Y_{obs} = 252.363636363636 \times 0.07 \times 0.661 = \mathbf{11.68\ mg/l}$$

where Y_{obs} = Observed Sludge Yield, (MLSS produced / BOD removed)

$$N_{part} = TESS \times N_S \times VSS/TSS = 10 \times 0.07 \times 0.84 = \mathbf{0.59\ mg/l}$$

where N_{part} = Nitrogen bound to VSS portion of effluent TSS (mg/l)
TESS = Anticipated Effluent Total Suspended Solids (mg/l)

Denitrification Oxygen Credit

$$O_{2denit} = 2.9 \times NO3\text{-}Ndenit = 2.9 \times 16 = \mathbf{46\ kg/day/basin}$$

where O_{2denit} = Oxygen mass credit from denitrification (kg/day/basin)

NO3-Ndenit = Mass of NO3-N denitrified (kg/day/basin)

$$NO_3\text{-}N_{denit} = \mu_{DN} \times VSS/TSS \times BMOB \times ART = 0.00075 \times 0.84 \times 3{,}216 \times 7.92 = \mathbf{16\ kg/day/basin}$$

where μ_{DN} = Denitrification rate at 6°C (NO3/MLVSS/hr)

BMOB = Basin biomass (kg/basin)

ART = Anoxic Retention Time, (hrs/day)

Total Actual Oxygen Transfer

$$AOR = AOR1 + AOR2 - O_{2denit} = 416 + 114 - 46 = \mathbf{484\ kg/day/basin}$$

where AOR = Total Actual Oxygen Required (kg/day/basin)

Total Standard Oxygen Transfer

$$SOR = \frac{AOR}{AOR\ /\ SOR} = \frac{484}{0.3556} = \mathbf{1,360\ kg/day/basin}$$

$$\frac{AOR}{SOR} = \frac{\alpha \times \theta^{(TSite - 20)} \times (\beta \times C^{*}sat_{20} \times Psite\ /\ Pstd \times Csurf_{T}\ /\ Csurf_{20} - D.O.)}{C^{*}sat_{20}}$$

$$\frac{AOR}{SOR} = \frac{0.50 \times 1.024^{(20 - 20)} \times (0.95 \times 10.84 \times 95.54\ /\ 101.36 \times 9.07\ /\ 9.07 - 2.0)}{10.84} = \mathbf{0.3556}$$

where SOR = Standard Condition Oxygen Requirement (kg/day/basin)
α = Alpha factor
θ = Temperature coefficient
Tsite = Water temperature (°C)
β = Beta factor
Psite = Site Atmospheric Pressure
Pstd = Standard atmospheric pressure (kpag)
$C^{*}sat_{20}$ = Dissolved oxygen solubility at standard conditions (mg/l)
$Csurf_{T}$ = Dissolved oxygen solubility at site water temperature (mg/l)
$Csurf_{20}$ = Dissolved oxygen solubility at 20°C (mg/l)
D.O. = Residual dissolved oxygen concentration (mg/l)

Aeration System Standard Oxygen Transfer Rate

$$SOTR = \frac{SOR}{TA} = \frac{1,360}{12} = \mathbf{113\ kg/hr/basin}$$

where SOTR = Standard oxygen transfer rate (kg/hr/basin)
TA = Aeration Time, (hrs/day)

Aeration Depth

Average Aeration Depth

$$AADad = \frac{Q \times [(NCT \times 60) - (NDT + NST)]}{2 \times 1{,}440 \times BA} + BWL$$

$$AADad = \frac{1{,}380 \times [(4.0 \times 60) - (60 + 60)]}{2 \times 1{,}440 \times 157} + 5.05 = \mathbf{5.41\ m}$$

where AADad = Average Aeration Depth at Average Dry Weather Flow (m^3/day)
Q = Average Dry Weather Flow (m^3/day/basin)
NCT = Normal Cycle Time (hr)
NDT = Normal Decant Time (min)
NST = Normal Settling Time (min)
BA = Basin Area (m^2)
1440 = Conversion (min/day)
2 = Calculate Aeration Depth at Middle of Normal Reaction Phase (NCT - NST - NDT)

Maximum Aeration Depth

$$MADpw = \frac{PWWF \times [(SCT \times 60) - (SDT + SST)]}{1{,}440 \times BA} + BWL$$

$$MADpw = \frac{3{,}650 \times [(2.0 \times 60) - (30 + 30)]}{1{,}440 \times 157} + 5.05 = \mathbf{6.02\ m}$$

where MADpw = Maximum Aeration Depth at Peak Wet Weather Flow (m^3/day)
PWWF Peak Wet Weather Flow (m^3/day/basin)
SCT = Storm Cycle Time (hr)
SDT = Storm Decant Time (min)
SST = Storm Settle time (min)
MAD = Maximum Aeration Depth (m)

MAD is larger of MADad and MADpw

MAD = **6.02 m**

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Air Flow Requirement

$$\text{Process Air} = \frac{\text{SOTR} \times 10{,}000}{\rho \times \text{SOTE} \times \text{Opw}} = \frac{113 \times 10{,}000}{1.201 \times 34.61 \times 23.2} = \mathbf{1{,}180\ m^3/hr}$$

where Process Air = Process air flow requirement (m^3/hr)
ρ = Air density (1.201 kg/day/m^3)
SOTE = Standard Oxygen Transfer Efficiency @ Submergence of 5.11 m
Opw = Fraction of Oxygen in air by Weight
10,000 = Conversion (100% * 100%)
60 = Conversion (min/hr)

$$\text{Mixing Air} = \text{MI} \times \text{BA} = 2.3 \times 157 = \mathbf{360\ m^3/hr}$$

where Mixing Air = Mixing air flow requirement (m^3/hr)
MI = recommended air flow per unit area of basin ($m^3/hr/m^2$)

Blower Unit Capacity

Blower unit capacity (BUC) is the larger of the process air requirement and the mixing air requirement.

Process Air 1,180 m^3/hr

Mixing Air 360 m^3/hr

Use 1 blower per tank

$$\text{BUC} = \mathbf{1{,}190\ m^3/hr}$$

Blower Pressure

$$\text{kpag} = \text{MAD} \times 9.772 + H_L = 6.02 \times 9.772 + 6.90 = \mathbf{66.2\ kpag}$$

where kpag = blower pressure (rounded to next kpag)
9.772 = water density (kpa/m)
H_L = Cumulative piping and diffuser headloss (kpag)

Average Blower Power

Blower power based on vendor curves, BUC, and Average Aeration Depth (5.11 m)

$$\text{Power}_{avg} = \mathbf{32.3\ kW}$$

SANITAIRE ICEAS Detailed Design Calculations
BOD Removal and Nitrification Process

SANITAIRE Project #28079-17ac
Nelson, BC

Design Parameters

A. Flow

Max Month	7,800	m^3/day
Max 4.0hr Cycle Flow	10,300	m^3/day
Max 2.0hr Cycle Flow	20,600	m^3/day

B. Treatment

	Influent Quality	Effluent Requirement
BOD_5 (20°C), mg/l	241.4230769	10
Suspended Solids, mg/l	138.474359	10
TKN, mg/l	31.79487179	
NH_3-N, mg/l		2
TN, mg/l		
Phosphorus	6.025641026	

C. Environment

Alkalinity (Minimum Requirement)	135	mg/l
Max Wastewater Temperature	20	°C
Min Wastewater Temperature	6	°C
Ambient Air Temperature	-6.7 - 32.2	°C
Site Elevation	535	m

D. ICEAS Process Design Criteria

F / M	0.108	BOD_5 / MLSS / day
SVI (after 30 minutes settling)	150	ml/g
Number of ICEAS Basins	6	
Top Water Level	6.4138	m

E. Cycle Timing

		Normal	Storm
Air-On	min	120	60
Air-Off	min		
Settle	min	60	30
Decant	min	60	30
Total	hrs	4	2

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F. Detailed Calculations

Mass of BOD

$$BODL = \frac{Q \times BODin}{1{,}000} = \frac{1{,}375 \times 252}{1{,}000} = \mathbf{347\ kg/day/basin}$$

where: BODL = BOD Load (kg/day/basin)
Q = Average Dry Weather Flow per basin (m^3/day)
BODin = Influent BOD concentration (mg/l)
1,000 = Conversion (l/m^3)

Mass of Biomass

$$BMOB = \frac{BOD_L}{F / M} = \frac{314}{0.1076} = \mathbf{2{,}915\ kg/basin}$$

where: BMOB = Mass of Biomass (kg/day/basin)
F / M = Food to Microorganism ratio (day^{-1})

Volume of Biomass

$$Vbio = BMOB \times SVI = 2{,}915 \times 0.15 = \mathbf{437\ m^3/basin}$$

where: Vbio = Volume of Biomass (m^3/basin)
SVI = Sludge Volume Index (m^3/kg)

Maximum Volume Above Bottom Water Level

Peak Dry Weather Flow:

$$Vbwld = \frac{PDWF \times (NCT - NDT)}{24} = \frac{1{,}717 \times (4.0 - 1.00)}{24} = \mathbf{215\ m^3/basin}$$

where: Vbwld = Maximum Volume Above BWL at Peak Dry Weather Flow (m^3/basin)
PDWF = Peak Dry Weather Flow (m^3/day)
NCT = Normal Cycle Time (hr/cycle)
NDT = Decant Time (hr/cycle)

Peak Wet Weather Flow:

$$Vbwls = \frac{PWWF \times (SCT - SDT)}{24} = \frac{3{,}433 \times (2.0 - 0.50)}{24} = \mathbf{215\ m^3/basin}$$

where: Vbwls = Maximum Volume Above BWL at Peak Wet Weather (Storm) Flow (m^3/basin)
PWWF = Peak Wet Weather Flow (m^3/day)
SCT = Storm Cycle Time (hr/cycle)
SDT = Storm Decant Time (hr/cycle)

MVAB (Maximum Volume Above Bottom Water Level) is larger of Peak Dry Weather and Peak Wet Weather Calculation

$$MVAB = \mathbf{215\ m^3/basin}$$

Decant Rates

Peak Dry Weather Flow:

$$PDR = \frac{MVAB}{NDT} + \frac{PDWF}{1{,}440} = \frac{215}{60.0} + \frac{1{,}717}{1{,}440} = \mathbf{4.99\ m^3/min}$$

where: PDR = Normal Decant Rate (m^3/min)
NDT = Normal Decant Time (min/cycle)
1440 = Conversion (min/day)

Peak Wet Weather Flow:

$$PWR = \frac{MVAB}{SDT} + \frac{PWWF}{1{,}440} = \frac{215}{30.0} + \frac{3{,}433}{1{,}440} = \mathbf{9.54\ m^3/min}$$

where: PWR = Peak Decant Rate (m^3/min)
SDT = Storm Decant Time (min/cycle)

Decanter Sizing

Peak Dry Weather Flow:

$$DLa = \frac{PDR}{\text{Weir Loading Rate}} = \frac{4.99}{1.86} = \mathbf{2.69\ m}$$

where: DLa = Decanter Length for Average Dry Weather Flow (m)
1.86 = Weir Loading Rate (m^3/min/m of decanter weir)

Peak Wet Weather Flow:

$$DLp = \frac{PWR}{\text{Weir Loading Rate}} = \frac{9.54}{2.23} = \mathbf{4.28\ m}$$

where: DLp = Decanter Length for Peak Wet Weather (Storm) Flow (m)
2.23 = Weir Loading Rate (m^3/min/m of decanter weir)

Design Decanter Length = **4.6 m**

Basin Working Volume

$$BWV = MVAB + Vbio = 215 + 437 = \mathbf{651\ m^3/basin}$$

where: BWV = Basin Working Volume (m^3/basin)

Basin Area

$$BA = \frac{BWV}{TWL - BZ} = \frac{651}{6.5 - 2.3} = \mathbf{157\ m^2/basin}$$

where: BA = Basin Area (m^2)
TWL = Top Water Level (m)
BZ = Buffer Zone (m) (Safety Factor)

Sludge Depth

$$SD = \frac{Vbio}{BA} = \frac{437}{157} = \mathbf{2.78\ m}$$

where: SD = Sludge Depth (m)

Decanter Draw Down

$$DD = \frac{MVAB}{BA} = \frac{215}{157} = \mathbf{1.36\ m}$$

where: DD = Draw Down (m)

Bottom Water Level

$$BWL = SD + BZ = 2.78 + 2.27 = \mathbf{5.05\ m}$$

where: BWL = Bottom Water Level (m)
Vd = Depth of Chemical Sludge for Phosporus precipitation (m)

Top Water Level

$$TWL = BWL + DD = 5.05 + 1.36 = \mathbf{6.41\ m}$$

where: TWL = Top Water Level (m)

Hydraulic Retention Time

$$HRT = \frac{BA \times MAFD}{QT}$$

where: HRT = Hydraulic Retention Time (days)
MAFD = Maximum Average Flow Depth (m)
QT = Fill Rate at Average Dry Weather Flow (m^3/day)

$$MAFD = \frac{Q \times [(NCT \times 60) - NDT]}{BA \times 1{,}440} + BWL = \frac{1{,}300 \times [(4.0 \times 60) - 60.0]}{157 \times 1{,}440} + 5.05 = \mathbf{6.08\ m}$$

$$HRT = \frac{157 \times 6.08}{1{,}300} = \mathbf{0.74\ days}$$

MLSS Concentration at Bottom Water Level

$$MLSS = \frac{Mbio \times 1,000}{BWL \times BA} = \frac{2,915 \times 1,000}{5.05 \times 157} = \mathbf{3,671\ mg/l}$$

where: MLSS = Mixed Liquor Suspended Solids concentration at Bottom Water Level (mg/l)
1E+03 = Conversion (g/kg)

Mass of Sludge Produced

$$\Delta M = \left(\frac{Y \times (BOD_{in} - BOD_{out})}{1 + (B \times \theta^{(T-20)} \times SRT)} + Zio + Zno \right) \times \frac{Q}{1,000}$$

$$\Delta M = \left(\frac{0.6 \times (241 - 10.0)}{1 + (0.07 \times 1.04^{(6-20)} \times 13.3)} + 27.7 + 41.5 \right) \times \frac{1.3E+03}{1,000} = \mathbf{207\ kg/day/basin}$$

(Lawrence-McCarty Equation as presented in WEF MOP/8 4th Edition, pg 11-11, Eqn. 11.7)

where: ΔM = Mass of Sludge Produced (kg/day/basin)
Y = Volatile cell yield (VSS/BOD removed)
q = Arrhenius Temperature Correction Factor
B = Decay Rate (day^{-1})
BOD_{out} = Anticipated Effluent BOD (mg/l)
SRT = Solids Retention Time (days)
Zio = Nonvolatile Influent suspended solids (mg/l)
Zno = Volatile Non-Biodegradable solids (mg/l)
T = Minimum Wastewater Temperature (°C)

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Volume of Sludge Produced

$$V_{ws} = \frac{\Delta M}{SFws \times 1{,}000} = \frac{207}{0.0085 \times 1{,}000} = \mathbf{24\ m^3/day/basin}$$

where:
Vws = Volume of Waste Sludge (m^3/day/basin)
SFws = Solids Fraction in Waste Sludge
8.34 = Density (kg/m^3)

Observed Yield Factor

$$Yobs = \frac{\Delta M}{BOD_L} = \frac{207}{314} = \mathbf{0.66}\ \frac{\mathbf{MLSS}}{\mathbf{BOD}}$$

where: Yobs = Observed Yield Factor (kg/day MLSS/kg/day BODremoved)

Mean Cell Residence Time

$$MCRT = \frac{Mbio}{\Delta M + ((Q - Vws) \times TESS / 1E+03)}$$

$$MCRT = \frac{2{,}915}{207 + ((1{,}300 - 24) \times 10.0 / 1{,}000)} = \mathbf{13.3\ days}$$

where:
MCRT = Mean Cell Residence Time (days)
TESS = Anticipated Effluent Total Suspended Solids (mg/l)
3.79E-03 = Conversion (lb/mg x l/gal)

Sludge Age for Nitrification

Refer to Metcalf and Eddy, Edition IV pages 614 and 705

Constants and Temperature Corrections:

Coefficient	Base Value	Theta	Temperature Corrected	Symbol
Maximum Specific Growth Rate of Nitrifying bacteria, g VSS/g VSS.day	0.75	1.07	0.291	μ_{nm}(T)
Half-Velocity constant for nitrifiers	0.74	1.053	0.359	Kn(T)
Nitrifier decay rate	0.08	1.04	0.046	Kdn(T)
Dissolved Oxygen, mg/l	2		2	DO
Half-Velocity Constant for Dissolved Oxygen, mg/l	0.5		0.5	Ko
Minimum Water Temperature, °C	6		6	T
Safety Factor	1.0		1.0	SF

Calculations:

$$\mu_n = \left(\mu_{nm}(T) \times \frac{TENH_3}{TENH_3 + Kn(T)} \times \frac{DO}{DO + Ko} \right) - Kdn(T)$$

$$\mu_n = \left(0.291 \times \frac{2.0}{2.0 + 0.359} \times \frac{2.0}{2.0 + 0.5} \right) - 0.046 = \mathbf{0.151\ days^{-1}}$$

$$SRTmin = \frac{1}{\mu_n} = \frac{1}{0.151} = \mathbf{6.6\ days}$$

$$SRTaerobic = SRTmin \times SF = 6.6 \times 1.0 = \mathbf{6.6\ days}$$

$$SRToverall = \frac{SRTaerobic \times 24}{TA} = \frac{6.6 \times 24}{12.0} = \mathbf{13.2\ days}$$

Design sludge age adequate for nitrification.

where: $\mu nm(T)$ = Maximum Temperature Corrected Nitrifier Growth Rate ($days^{-1}$)

μ_n = Specific Nitrifier Growth Rate at Temperature, DO, and Effluent NH_3 (g/g-days)

SRTmin = Minimum Sludge age required for Nitrification (days)

SRTaerobic = Design Aerobic Sludge Age (days)

SF = Safety Factor

SRToverall = Sludge Age accounting for entire ICEAS cycle (days)

TA = Aeration Time (hrs/day)

$TENH_3$ = Anticipated Effluent Ammonia (mg/l)

Waste Sludge Pump Capacity

$$WSP = \frac{Vws \times 1{,}000 \times NCT}{24 \times SPT} = \frac{24 \times 1{,}000 \times 4.0}{24 \times 9.77} = \mathbf{416\ l/min}$$

where: WSP = Waste Sludge Pump Capacity(l/min)
SPT = Sludge Pumping Time (min/cycle)

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SANITAIRE ICEAS Aeration Design Calculations
BOD Removal and Nitrification Process

SANITAIRE Project #28079-17ac
Nelson, BC

Carbonaceous Oxygen Demand

$$AOR1 = A \times \frac{Q \times BODin}{1,000} = 1.20 \times \frac{1,300 \times 241}{1,000} = \mathbf{376\ kg/day/basin}$$

where AOR1 = Actual Oxygen Required for BOD oxidation (kg/day/basin)
A = O2 / BOD
Q = Average flow (m^3/day/basin)
BODin = Influent BOD received (mg/l)
1,000 = Conversion (l x m^3)

Nitrification Oxygen Demand

$$AOR2 = TKNox \times 4.60 = 22.1 \times 4.60 = \mathbf{102\ kg/day/basin}$$

where AOR2 = Actual Oxygen required for Ammonia Oxidation (kg/day/basin)
TKNox = Nitrogen available for oxidation(kg/day/basin)

Constants

Coefficient	Value	Symbol
VSS/TSS	0.8364	
Sludge N	0.07	Ns
Effluent Dissolved Organic Nitrogen, mg/l	1	EDON
Expected Effluent Ammonium concentration	2	$TENH_3$

$$TKN_{ox} = (TKN - EDON - TENH_3 - N_{assim} - N_{part}) \times Q \div 1,000$$

$$TKN_{ox} = (31.7948717948718 - 1 - 2 - 11.17 - 0.59) \times 1,300 \div 1,000 = \mathbf{22.1\ kg/day/basin}$$

where N_{assim} = Nitrogen assimilated into biomass, (mg/l)

$$N_{assim} = BOD_{in} \times N_s \times Y_{obs} = 241.423076923077 \times 0.07 \times 0.661 = \mathbf{11.17\ mg/l}$$

where Y_{obs} = Observed Sludge Yield, (MLSS produced / BOD removed)

$$N_{part} = TESS \times N_S \times VSS/TSS = 10 \times 0.07 \times 0.84 = \mathbf{0.59\ mg/l}$$

where N_{part} = Nitrogen bound to VSS portion of effluent TSS (mg/l)
TESS = Anticipated Effluent Total Suspended Solids (mg/l)

Denitrification Oxygen Credit

$$O_{2denit} = 2.9 \times NO3\text{-}Ndenit = 2.9 \times 14 = \mathbf{41\ kg/day/basin}$$

where O_{2denit} = Oxygen mass credit from denitrification (kg/day/basin)

NO3-Ndenit = Mass of NO3-N denitrified (kg/day/basin)

$$NO_3\text{-}N_{denit} = \mu_{DN} \times VSS/TSS \times BMOB \times ART = 0.00075 \times 0.84 \times 2{,}915 \times 7.57 = \mathbf{14\ kg/day/basin}$$

where μ_{DN} = Denitrification rate at 6°C (NO3/MLVSS/hr)

BMOB = Basin biomass (kg/basin)

ART = Anoxic Retention Time, (hrs/day)

Total Actual Oxygen Transfer

$$AOR = AOR1 + AOR2 - O_{2denit} = 376 + 102 - 41 = \mathbf{438\ kg/day/basin}$$

where AOR = Total Actual Oxygen Required (kg/day/basin)

Total Standard Oxygen Transfer

$$SOR = \frac{AOR}{AOR / SOR} = \frac{438}{0.3556} = \mathbf{1{,}232\ kg/day/basin}$$

$$\frac{AOR}{SOR} = \frac{\alpha \times \theta^{(TSite - 20)} \times (\beta \times C^*sat_{20} \times Psite / Pstd \times Csurf_T / Csurf_{20} - D.O.)}{C^*sat_{20}}$$

$$\frac{AOR}{SOR} = \frac{0.50 \times 1.024^{(20 - 20)} \times (0.95 \times 10.84 \times 95.54 / 101.36 \times 9.07 / 9.07 - 2.0)}{10.84} = \mathbf{0.3556}$$

where SOR = Standard Condition Oxygen Requirement (kg/day/basin)

α = Alpha factor

θ = Temperature coefficient

Tsite = Water temperature (°C)

β = Beta factor

Psite = Site Atmospheric Pressure

Pstd = Standard atmospheric pressure (kpag)

C^*sat_{20} = Dissolved oxygen solubility at standard conditions (mg/l)

$Csurf_T$ = Dissolved oxygen solubility at site water temperature (mg/l)

$Csurf_{20}$ = Dissolved oxygen solubility at 20°C (mg/l)

D.O. = Residual dissolved oxygen concentration (mg/l)

Aeration System Standard Oxygen Transfer Rate

$$SOTR = \frac{SOR}{TA} = \frac{1,232}{12} = \mathbf{103\ kg/hr/basin}$$

where SOTR = Standard oxygen transfer rate (kg/hr/basin)
TA = Aeration Time, (hrs/day)

Aeration Depth

Average Aeration Depth

$$AADad = \frac{Q \times [(NCT \times 60) - (NDT + NST)]}{2 \times 1,440 \times BA} + BWL$$

$$AADad = \frac{1,300 \times [(4.0 \times 60) - (60 + 60)]}{2 \times 1,440 \times 157} + 5.05 = \mathbf{5.39\ m}$$

where AADad = Average Aeration Depth at Average Dry Weather Flow (m^3/day)
Q = Average Dry Weather Flow (m^3/day/basin)
NCT = Normal Cycle Time (hr)
NDT = Normal Decant Time (min)
NST = Normal Settling Time (min)
BA = Basin Area (m^2)
1440 = Conversion (min/day)
2 = Calculate Aeration Depth at Middle of Normal Reaction Phase (NCT - NST - NDT)

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Maximum Aeration Depth

$$MADpw = \frac{PWWF \times [(SCT \times 60) - (SDT + SST)]}{1{,}440 \times BA} + BWL$$

$$MADpw = \frac{3{,}430 \times [(2.0 \times 60) - (30 + 30)]}{1{,}440 \times 157} + 5.05 = \mathbf{5.96\ m}$$

where MADpw = Maximum Aeration Depth at Peak Wet Weather Flow (m^3/day)
PWWF Peak Wet Weather Flow (m^3/day/basin)
SCT = Storm Cycle Time (hr)
SDT = Storm Decant Time (min)
SST = Storm Settle time (min)
MAD = Maximum Aeration Depth (m)

MAD is larger of MADad and MADpw

MAD = **5.96 m**

Air Flow Requirement

$$\text{Process Air} = \frac{SOTR \times 10{,}000}{\rho \times SOTE \times Opw} = \frac{103 \times 10{,}000}{1.201 \times 34.17 \times 23.2} = \mathbf{1{,}080\ m^3/hr}$$

where Process Air = Process air flow requirement (m^3/hr)
ρ = Air density (1.201 kg/day/m^3)
SOTE = Standard Oxygen Transfer Efficiency @ Submergence of 5.09 m
Opw = Fraction of Oxygen in air by Weight
10,000 = Conversion (100% * 100%)
60 = Conversion (min/hr)

Mixing Air = MI x BA = 2.3 x 157 = **360 m^3/hr**

where Mixing Air = Mixing air flow requirement (m^3/hr)
MI = recommended air flow per unit area of basin ($m^3/hr/m^2$)

Blower Unit Capacity

Blower unit capacity (BUC) is the larger of the process air requirement and the mixing air requirement.

Process Air	1,080	m^3/hr
Mixing Air	360	m^3/hr

Use 1 blower per tank

BUC = **1,090 m^3/hr**

Blower Pressure

kpag = MAD x 9.772 + H_L = 5.96 x 9.772 + 6.90 = **65.5 kpag**

where kpag = blower pressure (rounded to next kpag)
9.772 = water density (kpa/m)
H_L = Cumulative piping and diffuser headloss (kpag)

Average Blower Power

Blower power based on vendor curves, BUC, and Average Aeration Depth (5.09 m)

$Power_{avg}$ = **29.9 kW**

Appendix 3

Structural Condition Assessment Sewage Treatment Plant

CONDITION ASSESSMENT
SEWAGE TREATMENT PLANT
NELSON, BC

Prepared by:

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Attention: Peter Gigliotti, P.Eng.

CWMM Project No. K6141 **May 13, 2021**

1.0 Introduction & Scope

CWMM Consulting Engineers Ltd. has been retained to provide a structural condition assessment of the existing sewage treatment plant in Nelson, BC. This assessment is a follow-up to the 2010 inspection and assessment completed by CWMM. The purpose of this assessment is to determine the general condition and potential remaining service life of the existing facility from a structural standpoint, and to provide recommendations for upgrading or replacement as required. A Class C cost estimate of the proposed work is provided for specific items. The inspection was limited to a visual examination of those components which could be observed directly. No tanks were drained, no building finishes were removed, and no non-destructive testing was carried out. A full analytical assessment of the existing structure is beyond the scope of this report.

2.0 Site Description and Inspection

An inspection of the existing facility was carried out on March 19, 2021 by Eric Densmore, P.Eng. and Jonathon Smith, Senior Technologist with CWMM. The inspection consisted of a general walk through, accompanied by maintenance staff, and a visual examination of the structural components that could be observed directly.

The sewage treatment plant is located on Highway 3A approximately 4 km west of Nelson. The original primary treatment facility was built in 1972. The cast-in-place reinforced concrete structure consists of two large digesters, a service building and sedimentation tanks.

In 1996 an addition was added to the east side of the sedimentation tanks. The addition consists of a slab-on-grade basement with reinforced concrete foundation walls and main floor suspended slab with load-bearing masonry walls supporting an I-joists roof structure.

In 2005 the plant was upgraded to achieve secondary treatment with the addition of rotating biological contactors (RBC), secondary clarifiers and UV treatment. The addition consists of two connected buildings for the clarifiers and RBC. The substructure consists of cast-in-place reinforced concrete and reinforced concrete suspended slabs. Load-bearing masonry walls supporting an I-joists roof structure was used for the clarifier building. A fiber-reinforced plastic roof structure was used as a covering over the RBC tanks. A wood-framed office building addition located at the front for the plant was also included in the 2005 upgrade.

A partial set of the original drawings were provided for review. CWMM Consulting Engineers Ltd. provided the structural design for the 1996 and 2005 additions.

3.0 Condition Assessment

Original Structure (1972):

In general, the original 1972 structure was found to be in good condition given its age. No major changes noted from previous inspection.

Similar rust was noted on the guardrails, access stairs, and ladders. On going monitoring and maintenance is required. Standing water and moss buildup was noted on the roof over the original service building (see photos 7 and 8). Signs of leaks or water damage to building finishes was noted in this area (see photos 11 and 12).

Moss was noted on the concrete roof structure over the sedimentation tanks. The moss allows excess moisture to accumulate on the concrete and should be removed (see photo 10).

No changes were noted in the area of prior settlement and subsequent excavation and underpinning at the north east corner of the original building. No signs of follow up repair work were noted. The existing undermined slabs have not been replaced. Cracking at the base of a column supporting the roof structure in this area remains largely unchanged (see photos 15 and 16). No changes noted at the construction joint between the grit tank and sedimentation tank 1 (see photos 17 and 18) however a small sinkhole has developed beneath the asphalt at the west joint location which indicates likely leakage of the tank. The brick veneer has separated from the concrete column at the north east corner.

Additions (1996 and 2005):

The 1996 and 2005 additions are generally in good condition. No major changes noted from previous inspection. The amount of efflorescence and staining on the exterior concrete surface of the RBC walls has increased. Exterior walls should be cleaned, and moss buildup removed. Existing control joint seals should be inspected and resealed as needed. Increased rusting and deterioration of the column base plate welds in RBC building was noted (see photo 21).

5.0 Conclusions and Recommendations

Based on our observations, we feel that the existing structure is generally in good condition. There were no obvious signs of new deterioration that would affect the structure. On this basis, we do not foresee a requirement for major upgrading to the structure in the immediate future.

Given the amount of deterioration that has occurred on the exterior guardrails and post baseplates, it is recommended that these areas be monitored and maintained as required on a regular basis.

The existing concrete tanks generally appear to be in good condition. No tanks were drained, and a detailed review of the existing concrete surfaces was not possible. The west construction joint between the grit tank and sedimentation tank 1 appears to be leaking. The condition of the concrete joint on the inside face of the wall should be reviewed by draining the tank and remedying the cause of liquid loss. There were no other obvious indications of excessive cracking or concrete deterioration that would limit the tanks' current continued use. We understand a retrofit/possible expansion of the existing facility is being explored. A detailed review of the concrete, including draining the tanks for interior inspection, should be done prior to any planning work.

The increased rusting and deterioration of the column base welds in the RBC building need addressing. The source of rusting should be determined which could be as simple as a conventional galvanized paint application may not have been applied over the original welds of the galvanized steel members. The welds should be cleaned and touched up with a galvanized paint application if the loss of weld from rusting is minimal. A preliminary cost of $5,000 is estimated for the investigation and repairs.

No additional movement was noted in the area of prior settlement and subsequent excavation and underpinning at the north east corner of the original building. Further investigation should be conducted to determine the severity and extent of any differential settlement including a geotechnical review. The undermined slabs in this area could be a potential safety hazard and likely would be found to need replacing. The replacement recommendation for the undermined slabs would likely be with reinforced concrete suspended slabs or steel platform structures. Despite the cracking noted in the adjacent column and foundation walls being largely unchanged, there does appear to be rusting of rebar occurring and bleeding through the cracks so we would recommend the column base be chipped out to expose the condition of the rebar and repair as necessary with new concrete patching. The repair work done in these areas will extend the lifespan and reduce the severity of any future repairs. A preliminary cost of $90,000 is estimated for the investigation and repairs noted above. Further excavation or underpinning of existing foundations may be required based on the outcome of the investigation.

Architectural finishes and roofing in the original building are in fair to poor condition and in need of maintenance or replacement.

As noted, a full analytical assessment of the existing structure is beyond the scope of this report.

It should be noted that portions of this facility are 50 years old which is the design life of buildings specified in the building code. While many buildings are able to exceed this design life by a large margin, structures under severe service conditions can be limited. The construction quality and oversight in the 1970's when the original facility was constructed can further limit lifespan. These factors should be considered in decisions regarding expansions/upgrades of the existing facility versus a new facility. Additionally, portions of this facility would not be able to satisfy more stringent current code requirements for minimum structural performance such as for seismic performance. Therefore, portions of this facility, with its diminished remaining lifespan, would perform at a level that is far below current code requirements.

We trust this is satisfactory to you. Should you have any questions or comments, please do not hesitate to call.

Report Prepared by:

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PROVINCE OF BRITISH COLUMBIA
E. M. DENSMORE
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2021-05-13

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Appendix A – Photos

Photo 1: West elevation, original building and 2005 office addition


Photo 2: North elevation, original structure

Photo 3: North elevation, 2005 addition and 1996 addition

Photo 4: South elevation, 2005 addition


Photo 5: Looking west, original structure


Photo 6: North digester

Photo 7: Looking east, original structure

Photo 8: Original building roof


Photo 9: Original structure, equalization tanks


Photo 10: Original structure, suspended concrete roof

Photo 11: Leaking from roof in original control building

Photo 12: Leaking from roof in original control building


Photo 13: Excavation under existing slab


Photo 14: Excavation under existing foundation


Photo 15: Cracking at column between grit tank and sedimentation tank 1


Photo 16: Cracking at column between grit tank and sedimentation tank 1

Photo 17: Cracking at west construction joint between grit tank and sedimentation tank 1

Photo 18: Cracking at east construction joint between the inlet and sedimentation tank 1

Photo 19: Efflorescence at north RBC wall


Photo 20: RBC building


Photo 21: Rust and deterioration at column baseplates

Appendix 4

Alternate Site Wastewater Treatment Facility – Opinion of Probable Cost

Alternate Site Wastewater Treatment Facility - City of Nelson
Opinion of Probable Cost

URBAN SYSTEMS

Job No: 0795.0119.01
Date: 06-May-22
Prepared by: S. Johnson
Checked by: M. Smith

ITEM	DESCRIPTION	UNIT	TOTAL QUANTITY	UNIT PRICE	TOTAL
1.0	**General**				
	Mobilization/Demobilization	LS	1	$ 220,000.00	$ 220,000.00
	Insurance and Bonding	LS	1	$ 330,000.00	$ 330,000.00
	Commissioning	LS	1	$ 220,000.00	$ 220,000.00
	Excavation Dewatering	LS	1	$ 100,000.00	$ 100,000.00
	Contaminated Sites Investigation/Remediation/Soil Stability	LS	1	$ 2,000,000.00	$ 2,000,000.00
2.0	**Removals**				
	Remove and Replace unsuitable material - Treatment Building and Tanks	cu.m.	2000	$ 100.00	$ 200,000.00
3.0	**Site Works**				
	Yard Piping	LS	1	$ 300,000.00	$ 300,000.00
	Influent Forcemain	l.m.	750	$ 800.00	$ 600,000.00
	Valves	LS	1	$ 75,000.00	$ 75,000.00
	Gravel Access Roading and Parking	sq.m.	250	$ 50.00	$ 12,500.00
	Fencing	l.m.	500	$ 125.00	$ 62,500.00
	Landscaping	LS	1	$ 50,000.00	$ 50,000.00
4.0	**Treatment Building**				
	Main Floor (Headworks, Blowers, Chemical Storage, Odour Control, Electrical)	sq.m.	500	$ 4,500.00	$ 2,250,000.00
	Upper Floor (Office)	sq.m.	250	$ 1,000.00	$ 250,000.00
	Monorails (Screens, Centrifuge, Fans)	ea.	3	$ 20,000.00	$ 60,000.00
	Bridge Crane (Blowers)	ea.	1	$ 30,000.00	$ 30,000.00
	Overhead Doors	ea.	5	$ 20,000.00	$ 100,000.00
	Misc. Metals	LS	1	$ 300,000.00	$ 300,000.00
5.0	**Treatment Tankage, Process Piping and Equipment**				
5.1	**Headworks**				
	JWC Environmental Finescreen Monster (c/w screen, washer, compactor, Freight to Site, Start-Up Service)	LS	1	$ 600,000.00	$ 600,000.00
	John Meunier Self-Standing Grit Removal System (c/w MECTAN Vortex Grit Removal, Gorman-Rupp Grit Pumps, SAM Type GDS Grit Dewatering Screw, PLC/HMI Control, Freight to Site, Start-Up service, 12 month warranty	LS	1	$ 470,000.00	$ 470,000.00
5.2	**EQ Tanks**				
	Cast-in-Place Concrete Tanks	LS	1	$ 420,000.00	$ 420,000.00
	Mixing System	LS	1	$ 25,000.00	$ 25,000.00
	Transfer Pumps (EQ to SBR)	LS	1	$ 25,000.00	$ 25,000.00
5.3	**SBRs**				
	Cast-in-Place Concrete Tanks	LS	1	$ 1,340,000.00	$ 1,340,000.00
	PremierTech SBR Equipment (c/w decanters w/ motorized valves and air compressor, WAS pumps, fine bubble aeration, blowers, electrical motoroized valves, instrumentation for DO, level and float)	LS	1	$ 1,265,000.00	$ 1,265,000.00
5.4	**Process Piping**				
	Process Piping	LS	1	$ 1,400,000.00	$ 1,400,000.00
5.5	**Chemical Systems**				
	Chemical Storage, Pumping, Process Piping/Valves, Emergency Shower, Containment Sump, Epoxy	LS	1	$ 250,000.00	$ 250,000.00

ITEM	DESCRIPTION	UNIT	TOTAL QUANTITY	UNIT PRICE	TOTAL
5.6	**Sludge Dewatering**				
	Centrifuge and polymer feed system	LS	1	$ 1,150,000.00	$ 1,150,000.00
	Sludge Feed Pumps	LS	1	$ 17,000.00	$ 17,000.00
	Sludge Day Tank	LS	1	$ 340,000.00	$ 340,000.00
	Sludge Tank Mixer	LS	1	$ 50,000.00	$ 50,000.00
5.7	**UV Disinfection**				
	Trojan UV Signa disinfection system	LS	1	$ 900,000.00	$ 900,000.00
	Building	sq.m.	150	$ 4,500.00	$ 675,000.00
5.8	**Outfall**				
	Outfall piping and diffusers	LS	1	$ 300,000.00	$ 300,000.00
5.9	**Odour Control**				
	Granular Activated Carbon Odour Control System	LS	1	$ 410,000.00	$ 410,000.00
5.10	**Miscellaneous Process Items**				
	Onsite Reclaimed Water System	LS	1	$ 150,000.00	$ 150,000.00
	Flow Meter	LS	1	$ 10,000.00	$ 10,000.00
6.0	**Electrical**				
	Electrical Service	LS	1	$ 30,000.00	$ 30,000.00
	Electrical Controls and Instrumentation	LS	1	$ 2,000,000.00	$ 2,000,000.00
	Emergency Generator	LS	1	$ 420,000.00	$ 420,000.00
	Programming	LS	1	$ 175,000.00	$ 175,000.00
	SCADA System	LS	1	$ 235,000.00	$ 235,000.00
7.0	**HVAC**				
	General HVAC Servicing	LS	1	$ 1,000,000.00	$ 1,000,000.00
8.0	**Miscellaneous**				
	Water service connection	ea.	1	$ 10,000.00	$ 10,000.00
	Lab/office equipment and furniture	LS	1	$ 130,000.00	$ 130,000.00
	Other	LS	1	$ 100,000.00	$ 100,000.00
	Summary				
1.0	**General**				$ 2,870,000.00
2.0	**Removals**				$ 200,000.00
3.0	**Site Works**				$ 1,100,000.00
4.0	**Treatment Building**				$ 2,990,000.00
5.0	**Treatment Tankage, Process Piping and Equipment**				$ 9,797,000.00
6.0	**Electrical**				$ 2,860,000.00
7.0	**HVAC**				$ 1,000,000.00
8.0	**Miscellaneous**				$ 240,000.00
				Subtotal	**$ 21,057,000.00**
				Contingency (35%)	**$ 7,369,950.00**
				Engineering (15%)	**$ 4,264,042.50**
				Total	**$ 32,690,992.50**
				Rounded Total	**$ 32,700,000.00**

Notes:

1. Plant Capacity

a) Existing	AADF	4,900	m^3/d
	MMF	5,600	m^3/d
	MDF	8,900	m^3/d
	PHF	18,700	m^3/d
b) Buildout	AADF	7,200	m^3/d
	MMF	12,250	m^3/d
	MDF	16,550	m^3/d
	PHF	27,350	m^3/d

2. Treatment Process Includes:
   a) Liquid Process - Fine Screens, Grit Removal, pH Adjustment, SBRs, UV Disinfection.
   b) Solids Process - Aeration, Gravity Thickening, Dewatering via Centrifuge. Composting off-site by others.
   c) Odour Process - Granular Activated Carbon

3. Electrical cost estimates to be checked by Electrical Consultant

4. Geotechnical / Structural investigation to be completed to determine foundation requirements