

**Arthur Water and  
Wastewater Servicing**  
In the Township of Wellington North

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**Summary Report**

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MARCH 2025

**Report Prepared For:**  
TRIBUTE/SORBARA ARTHUR HOLDINGS INC

**Report Prepared By:**



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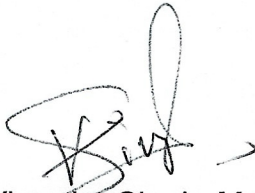
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## REVISION LOG

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## SIGNATURE PAGE

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## 1.1 INTRODUCTION

DLW Engineering Services Limited was retained by Tribute/Sorbara Arthur Holdings Inc. to provide consulting engineering services related to the conceptual design of the water and wastewater servicing of the Tribute Arthur Development in the Township of Wellington North. This memorandum has been prepared to establish the viability of expanding the existing Arthur Water Supply and Wastewater Treatment Plant (WWTP) to accommodate the Tribute Arthur Development. This memorandum will show that there are viable water and wastewater servicing options for the proposed Tribute Arthur Development.

## 1.2 WATER SERVICING

According to the September 2020 Water and Sanitary Systems Technical Study – Arthur, completed by Triton Engineering Services Limited for the Township of Wellington North, the Arthur water system is a single-pressure zone water main distribution network that is pressurized by two elevated towers. Water is supplied to the system from three bedrock wells. The system currently provides service to 939 homes and 109 Industrial/Commercial/Institutional (ICI) properties, according to Township records (2020). The system also provides fire protection to the entire service area.

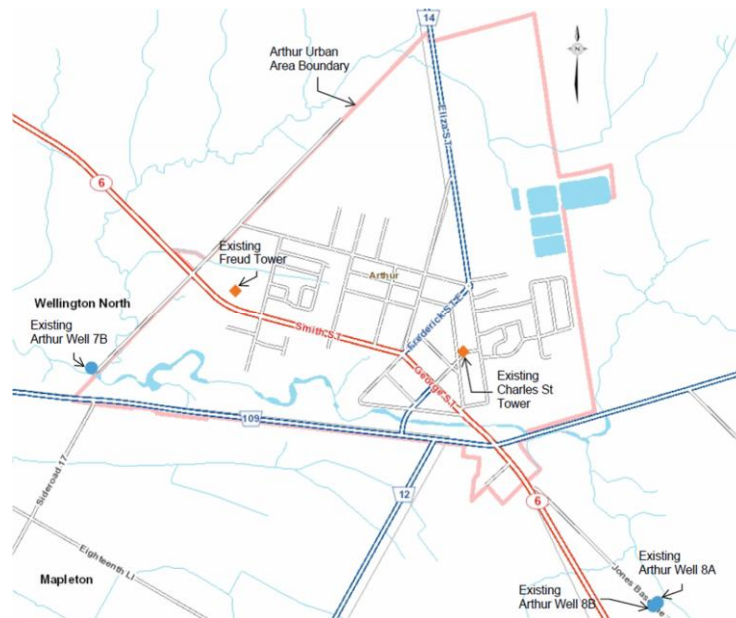


Figure 1: Existing water works location plan

### 1.2.1 Water Demands

A review of the historical water usage rates within Arthur indicates that the maximum day demand for the existing system is 652 Lpcd as presented in the Reserve Capacity Calculation (RCC), and this per person RCC value includes the ICI demands. The Triton 2020 study states that a maximum day-per-person flow rate of 500 Lpcd will be applied for future demand

calculations. Based on Table 3.1 in that report, the total population (equivalent residential units) is multiplied by 500 Lpcd to determine the future flows.

At this preliminary stage, the proposed Tribute Arthur development will be based on either 823 or 872 residential units. Based on a residential density of 2.7 persons per unit this will equate to 2,222 persons for 823 units, and 2,354 persons for 872 units (Table 1).

As noted in the November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards, the existing 2023 Arthur Water supply system services an equivalent population of approximately 3,195 persons. With the addition of the Tribute Arthur development, the water supply service population will increase the total population to the following:

Table 1: Water Service Population

Development	Total Existing and Proposed Population
Existing 2023 Development (Nov 2024 PIC Boards)	3,195
Existing plus 823 Unit Tribute Development	5,417
Existing plus 872 Unit Tribute Development	5,549

### 1.2.2 Water Wells

Water supply is provided by three bedrock wells, named Well No. 7B, Well No. 8A, and Well No. 8B as shown in the figure above and which are described as follows:

#### 1.2.2.1 Well No. 7B

- Located at 109 Wells Street West near the Conestoga River.
- Commissioned in 1998.
- 46 m deep drilled.
- Well pump is a submersible type complete with a 30kW (40 hp) motor that discharges directly to the distribution system (i.e., no high lift pumps).
- Rated capacity is 22.7 L/s (1,961 m<sup>3</sup>/day).
- Disinfection using sodium hypochlorite. Contact time is provided by the oversized discharge main.
- Iron sequestering treatment provided.

#### 1.2.2.2 Well No. 8A/8B

- Located on Part of Lots 20 and 21, Concession A, approximately 1.15 km south of County Road 109 and 235 m east of Highway 6.
- Commissioned in 2005.
- Depth of wells is 61.9 m and 62.2 m for 8A and 8B, respectively.
- Well pumps are submersible type complete with a 30 kW (40hp) motor which discharges directly to the distribution system (i.e., no high lift pumps).

- Rated capacities are 26.1 L/s (2,255 m<sup>3</sup>/day) each; however, the Permit to Take Water (PTTW) allows for the operator to pump either Well 8A or Well 8B, but not both wells concurrently. Therefore, total production from this facility is limited to 2,255 m<sup>3</sup>/day.
- Disinfection using sodium hypochlorite. Contact time is provided by the oversized discharge main.
- Treatment for high manganese is provided.
- Standby power provided by diesel generator.

MECP lists the firm capacity of infrastructure as the capacity with the largest unit out of operation. Thus, based on this definition and the above, the total production capability (i.e. source capacity) of the Arthur water system is 4,216 m<sup>3</sup>/day. Because of the duty/standby nature of Wells 8A and 8B which is mandated by MECP, this is also the firm production capacity for the well system. Notwithstanding this, in the 4 April 2022 Triton reserve capacity analysis report for Arthur, Triton states that the Arthur firm capacity is 2,255 m<sup>3</sup>/day.

### 1.2.3 Reserve Capacity Analysis

As per the reserve capacity analysis report submitted by Triton Engineering Services Limited (dated April 4, 2022) on water supply and sewerage systems in Arthur, the Maximum Day Demand flow of the 3 years (2019, 2020, & 2021) was 1,531 m<sup>3</sup>/day. Given Triton’s assertion that the firm capacity of the water supply system is 2,255 m<sup>3</sup>/day this results in a reserve capacity of 724 m<sup>3</sup>/day. This translates to 489 uncommitted reserve capacity equivalent residential units (Table 2).

Table 2: Reserve Capacity Analysis – Arthur Water Supply

Item	Description	Based on Firm Capacity in 2022 Triton Report
1	Firm Capacity (m <sup>3</sup> /day)	2,255
2	Three-Year Maximum Day Demand (m <sup>3</sup> /day)	1,531
3	Three-Year Average Day Demand (m <sup>3</sup> /day)	1,036
4	Available reserve capacity (1-2) (m <sup>3</sup> /day)	724
5	Three-Year Max/Avg Day Peaking Factor (2/3)	1.48
6	Three-Year Average Residential Demand (m <sup>3</sup> /d)	604
7	Peaked Max Day Residential Flow (5x6) (m <sup>3</sup> /day)	893
8	Occupied Serviced households	968
9	PPU (2016 census data)	2.4
10	Population Served (8x9)	2,323
11	Maximum Residential Day Demand per Capita (7/10) (m <sup>3</sup> /day)	0.384
12	Additional Population that can be served (4/11)	1,885
13	Persons per new Equivalent Residential Unit (ERU) (2018 Growth Management Plan)	2.7
14	Additional Equivalent Residential Units that can be served (12/13)	701
15	Committed Equivalent Residential Units	212
<b>16</b>	<b>Uncommitted Reserve Capacity Equivalent Residential Units (14-15)</b>	<b>489</b>

It is worth noting that in the November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards the current water system now has an uncommitted reserve capacity of 710 m<sup>3</sup>/d. This translates to 480 uncommitted reserve capacity equivalent residential units.

The November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards also states that a new well will be provided that will supply at least 864 m<sup>3</sup>/d but more likely that a new 2,332 m<sup>3</sup>/d well will be provided. This well was shown to be located on the Tribute Arthur lands. We presume the larger capacity well is typical of the well capacities seen at Wells 7B, 8A and 8B. When this well is constructed the uncommitted reserve capacity will increase to 1,574 m<sup>3</sup>/d to 3,042 m<sup>3</sup>/d, which even on the lower end is more than enough flow for the proposed Tribute Arthur development.

#### 1.2.4 Water Treatment

As noted above, each well is treated with iron or manganese removal plus disinfected with chlorine to control bacteria, algae, and viruses by the provincial Procedures for Disinfection for Drinking Water in Ontario guidelines. We assume that any new water supply well will provide water of similar quality as the existing supply wells and will require either iron or manganese treatment as well as chlorine disinfection. The treatment plant for the proposed well(s) will require a small building structure to house the discharge piping, valves, and flowmeter, iron or manganese treatment system, chlorination system, MCC's, instrumentation system to monitor water quality, a chlorine contact tank to ensure that the water receives a sufficient amount of chlorine contact before reaching the first consumer. The treatment plant will also include a generator to provide uninterrupted power for the pumps to ensure uninterrupted water supply to the distribution system.

While we believe that there is sufficient well and associated water treatment capacity for the proposed Tribute Arthur development, if additional well supplies are required, the appropriate treatment system can be provided to treat the new well supplies and will likely include iron and manganese treatment plus disinfection.

#### 1.2.5 Water Storage

According to the September 2020 Triton Water and Sanitary Systems Technical Study, the storage for the Arthur water system is provided by two elevated facilities. These facilities are as follows:

##### 1.2.5.1 Charles Street Tower

- Located near the intersection of Charles Street East and Isabella Street, in the southeast part of the system (195 Isabella Street East).
- Multi-legged steel tank
- Commissioned in 1932
- Volume is 227 m<sup>3</sup>

- Operation range: 494.2 m – 499.6 m

The November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards states that this tower has reached the end of its useful life and that the Township will be decommissioning this tower when the new elevated water storage tank is constructed.

#### 1.2.5.2 Freud (Spheroid) Tower

- Located just north of Smith Street between Preston and Wells Streets in the northwest
- part of the system (460 Smith Street).
- All steel spheroid tank
- Commissioned in 1967
- Volume is 1,137 m<sup>3</sup>
- Operation range: 494.0 m – 499.2 m

According to Triton discussions with Township staff, these facilities have been recently inspected and no significant deficiencies were noted. The total system storage volume currently available is 1,364 m<sup>3</sup>.

#### 1.2.5.3 New Elevated Water Storage Tank proposed in 2024 Class EA

According to the November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards, a new 900m<sup>3</sup> elevated water storage tank is proposed to address proposed development to 2051. The additional elevated water storage tank was identified as being located adjacent to the proposed new well on the Tribute Arthur lands. The Class EA states that this additional capacity is required to accommodate a 2051 population of 4,800 persons. This new storage tank was proposed to accommodate an additional population of 1,605 persons (or 639 additional units based on 2.6 persons per unit). It is worth noting that this amount of storage will not accommodate the full Tribute Arthur development which on the top end will result in a total population of 5,549 persons.

#### 1.2.5.4 Proposed Storage Requirement

As per the reserve capacity analysis report submitted by Triton Engineering Services Limited (dated April 4, 2022) on water supply and sewerage systems in Arthur, the existing system storage in Arthur is 1,137m<sup>3</sup> and the required storage in 2021 was 994 m<sup>3</sup> based on MECP design guidelines. Thus, the existing available surplus storage in the system was 143 m<sup>3</sup>. Triton's existing storage assumption is based on the fact that the Charles Street Tower is reaching the end of its useful life and thus excluded from their existing storage calculation. According to the November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards Triton now projects a storage deficit of 118m<sup>3</sup>.

We have assumed the MECP formulation for the development of system storage and used the Triton April 2022 recommendation for maximum day flows based on 500 Lpcd. Based on the existing serviced and committed population in combination with the proposed Tribute Arthur development we estimate the total storage requirement for Arthur will increase to the following:

- 2,875 m<sup>3</sup> for 823 units (Tribute development requires 1,359 m<sup>3</sup>)
- 2,922 m<sup>3</sup> for 872 units, and (Tribute development requires 1,406 m<sup>3</sup>)

The above Tribute storage requirements are based on the fact that the total system fire flows will grow from the existing 100 L/s for 2 hours (based on Triton 2020 report) to the proposed 150 L/s for 3 hours with the addition of 823 units, or 152 L/s for 3 hours for 872 units, based on the increase in service population.

The Tribute Arthur development will require a new reservoir to meet their storage demands. Assuming the MECF formulation for the development of system storage and using the Triton April 2022 recommendations, if we assume that the Tribute Arthur development storage is sized on the basis of just its own population, the water storage tank for the Tribute Arthur development will be as follows:

- 1,232 m<sup>3</sup> for 823 units and
- 1,271 m<sup>3</sup> for 872 units.

Thus, the storage for the Tribute development will be based on the whether the storage is based on servicing strictly the Tribute Arthur development or on the impact to the full Arthur development will be as follows:

- For 823 development units, storage will be between 1,232 to 1,359 m<sup>3</sup>, and
- For 872 development units 1,271 to 1,406 m<sup>3</sup>.

**Based on the November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards, the proposed elevated water storage tank should increase from its currently proposed 900 m<sup>3</sup> to at least 1,800 m<sup>3</sup> to accommodate the existing development and the full Tribute Arthur development.**

If the proposed elevated water storage tank in the Class EA is not expanded to accommodate the Tribute Arthur development, the reservoir configuration at the Tribute Arthur development could be either of the following:

- Inground Concrete Reservoir with associated Booster Pumping Station, all located on the Tribute Arthur development site with jockey, duty, and fire pumps.
- Elevated Storage Tank located on the Tribute Arthur development site fed directly from the wells

### 1.2.6 Proposed Water Supply and Storage System Layout

Based on the preliminary concept plan, the proposed well, pumphouse, and elevated water storage tank will be located on proposed Block 67 at the intersection of Wells Street and Macauley Street. Figure 2 below illustrates a plan showing the proposed groundwater well, well pumphouse/water treatment plant and elevated water storage tank. This plan shows that an area of approximate size 0.2 ha is required to house these facilities.



## 1.3 WASTEWATER SERVICING

### 1.3.1 Project Background

Further to our preliminary site servicing investigation dated March 18, 2022, SCS and DLW held a conference call on April 25, 2022, with Matthew Aston, who is the Director of Operations at the Township of Wellington North.

Mr. Aston provided us with a Memorandum Re: Review of the 2016 Arthur WWTP Class EA – Summary of the Basis of Development of the Design ADF of 2,300 m<sup>3</sup>/d (EA Review). According to the EA Review *“Existing (base) 2012 service population was reported to be 2,596 persons and residential growth projections were based on 103 lots from the planned Phase 3 Eastridge contribution (284 persons) and other anticipated growth within the service area (714 persons), for a total residential growth of 998 persons to 2031. The residential growth forecast for the Village of Arthur (excluding Eastridge) was taken from the Wellington County Official Plan 1999, revised in February 2011 (Part 3, Wellington Growth Strategy, Table 2, p. 10). The total projected residential population to 2031 was, therefore, estimated to be 3,594 persons.”* The Class EA used a design per capita sewage generation flow rate (for residential growth) of 370 L/cap/d (dry weather) plus an Inflow/Infiltration allowance of 90 L/cap/d, for an overall value of 460 L/cap/d.

The 2016 Arthur WWTP Class EA identified a Phase 1 expansion of the plant from 1,465 m<sup>3</sup>/d to 1,860 m<sup>3</sup>/d and a Phase 2 expansion from 1,860 m<sup>3</sup>/d to 2,300 m<sup>3</sup>/d. Mr. Aston also provided a Staff Report on water and wastewater technical updates for the Mayor and Members of Council Special for a Meeting held on June 2, 2021. In this staff report, the shovel-ready Phase 2 wastewater treatment plant expansion was identified to cost approximately \$8.3 million based on 2018 pricing.

The EA Review memorandum stated that the Conestoga River was determined to be a Policy 2 receiver for total phosphorus (TP), E. coli, and dissolved oxygen (July and August only). Further, the MOE (now MECP) approved an effluent Total Phosphorus limit of 0.25 mg/L, equivalent to an annual TP loading limit of 210 kg/year at an ADF of 2,300 m<sup>3</sup>/d. This is critical because with different technology that provides a higher level of treatment, phosphorus can be reliably reduced to as low as 0.05mg/L. The WWTP appears to have sufficient capacity to accommodate this expansion, but this will be addressed in design.

### 1.3.2 Existing WWTP Information and Sewerage Pumping Stations

#### 1.3.2.1 Existing WWTP

The Arthur Wastewater Treatment Plant (WWTP), constructed in approximately 1990, services the community of Arthur in the Township of Wellington North (the Township). The WWTP is an extended aeration plant with tertiary filtration, with offsite effluent storage lagoons, and seasonal discharge of final effluent to the Conestoga River. Per the ECA, discharge is permitted between September 16 to April 30 and limited to no more than 6,500 m<sup>3</sup>/d. It is important to note that no discharge is allowed into the river from June to September

due to the low flow conditions in the river. The allowable effluent flow rate is dependent on the effluent TAN concentration and the flow rate in the Conestoga River. In 2016 when the ESR was prepared, the rated average day flow (ADF) capacity of the works was 1,465 m<sup>3</sup>/d. Flow to the WWTP is conveyed via two sewage pumping stations (Wells St. Sewage Pumping Station (SPS) and Frederick St. SPS) and gravity sewers. The unit process unit details of the 2016 WWTP are given in Table 3.

Table 3: Unit Sizing and Process Details of the existing WWTP based on ADF of 1,465 m<sup>3</sup>/day

Parameter	Process Design
<b>Grit Removal</b> Type Number Dimensions Capacity	Manually cleaned Grit Channels 2 5.4 m x 0.75 m x 0.5 m SWD 5,045 m <sup>3</sup> /d
<b>Comminution</b> Capacity	5,045 m <sup>3</sup> /d
<b>Screening</b> Type Capacity	Manually cleaned 5,045 m <sup>3</sup> /d
<b>Aeration Tank</b> Number Dimensions Volume (each cell) Volume (total) Diffuser Type	2 cell annular ring-type aeration tank 27.95 m Equivalent Length x 4.65 m x 4.18 m SWD - Cell 1 27.26 m Equivalent Length x 4.65 m x 4.18 m SWD - Cell 2 543 m <sup>3</sup> - Cell 1 530 m <sup>3</sup> - Cell 2 1,073 m <sup>3</sup> Coarse Bubble
<b>Blowers</b> Number Capacity	2 (1 duty, 1 standby) 486 L/s, each
<b>RAS/WAS Pumps</b> Number Capacity Storage Volume	2 34 L/s, each 50 m <sup>3</sup>
<b>Secondary Clarifier</b> Type Number Dimensions Surface Area	Circular inlet clarifier 1 13.5 m diameter x 3.8 m SWD 143 m <sup>2</sup>
<b>Chemical Pumps (Alum)</b> Number Capacity Chemical Storage Volume	2 (1 duty, 1 standby) 250 L/d, each 23 m <sup>3</sup> storage tank & 450 L/day tank

Parameter	Process Design
<b>Tertiary Filtration</b> Type Number of Modules Total Filtration Area Backwash pumps	Continuous backwash, upflow, deep bed granular media (1 m depth) 6 27.9 m <sup>3</sup> 2 wash water reject pumps (1 duty, 1 standby) Each rated at 6.1 L/s at 3.5 m TDH
<b>UV Disinfection</b> No. of bank Modules Per bank Lamps per module Channel dimension Capacity	2 banks in a series 4 7.9 m long x 0.5 m wide x 0.9 m SWD 6,500 m <sup>3</sup> /d

### 1.3.2.2 Pumping stations

The Arthur wastewater collection system consists of a gravity sewer network with two SPSs:

#### 1. The Wells St. SPS.

- Two submersible pumps (one duty, one standby), each with a rated capacity of 16L/s (1,382 m<sup>3</sup>/d).
- One wet well with a liquid volume of approximately 120 m<sup>3</sup>.
- One standby diesel generator.

#### 2. The Frederick St. SPS.

- Two submersible pumps (one duty, one standby), each with a rated capacity of 58.4 L/s (approximately 5,045 m<sup>3</sup>/d).
- One reinforced wet well, measuring approximately 5.3 m x 5.3 m x 6.2 m (deep), providing a total storage volume of approximately 174 m<sup>3</sup>.
- One 60 kW standby diesel generator with 450 L fuel tank.

Each pumping station has a force main that discharges near the treatment plant. Over the review period (2007 – 2015), bypasses of the Frederick St. SPS have occurred during peak flow periods. As such, the Township wishes to evaluate the existing capacity of both pumping stations and determine if any upgrades and/or expansions are required for the study’s design year of 2031.

### 1.3.2.3 Population growth and flow projections for Phase-II expansion

The future servicing needs for the Town of Arthur are based on the historic flows from the existing service area, plus the projected flows from the future residential and industrial/commercial/institutional (ICI) development.

The Existing (base) 2012 service population was reported to be 2,596 persons and residential growth projections were based on 103 lots from the planned Phase 3 Eastridge contribution

(284 persons) and other anticipated growth within the service area (714 persons), for a total residential growth of 998 persons to 2031 (Table 4). The total projected residential population in 2031 was estimated to be 3,594 persons including future Eastridge contribution (Arthur WWTP Class EA, 2016). Industrial / commercial / institutional (ICI) flows were also considered. Based on the 2012 Master Plan Study for Water Supply and Sanitary Sewage (Triton Engineering, 2012), a total of 28.7 ha of developable non-residential land would be serviced by 2031, including a vacant designated industrial parcel with 25.1 ha of developable land, and 3.6 ha of developable Highway Commercial parcels. Future growth contribution associated with Golden Valley Farms was based on an equivalent of 9 residential units which was reported to “reflect the remaining unused portion of the Golden Valley Farms allocation” as included in the 2012 Reserve Capacity Calculations for the Arthur WWTP (Triton Engineering, 2012).

Table 4: Population growth and flow projections

Parameter	Value
<b>Residential Flow Projections</b>	
2012 Service Population	2,596
Historical ADF	1,171 m <sup>3</sup> /d
Future Population Contribution from Eastridge	284
Other anticipated population growth	714
2031 Projected Service Population	3,594
Design Per Capita Flow	370 Lpcd
Design Per Capita Average Inflow and Infiltration	90 Lpcd
2031 Residential Flow	1,653 m <sup>3</sup> /d
<b>ICI Flow Projections</b>	
Industrial - Golden Valley Historic Flows	171 m <sup>3</sup> /d
Industrial - Golden Valley Growth Flows to 2031	11 m <sup>3</sup> /d
Commercial/Industrial Land Flows	488 m <sup>3</sup> /d
Total ICI Flow Projections	670 m <sup>3</sup> /d
<b>TOTAL 2031 FLOW PROJECTION</b>	<b>2,300 m<sup>3</sup>/d</b>

### 1.3.2.4

#### Proposed Class EA developmental phases, Influent concentration, loadings, and effluent criteria

The Class EA proposed a staged approach to the capacity increase as follows:

**Phase 1:** Design ADF of 1,860 m<sup>3</sup>/d, based on providing additional servicing capacity for the short-term via minor upgrades to the WWTP; and,

**Phase 2:** Design ADF of 2,300 m<sup>3</sup>/d, based on providing servicing capacity required for the design year of 2031.

The flow and loadings for BOD<sub>5</sub>, TSS, TKN, and TP with respect to the phases that were provided in the Class EA are given in Table 3 below:

Table 5: Influent concentrations and organic loadings for the phase-I and II expansions

Parameter	Phase-I (completed)	Phase-II (to year 2031)
ADF	1,860 m <sup>3</sup> /d	2,300 m <sup>3</sup> /d
MDF	7,853 m <sup>3</sup> /d	8,784 m <sup>3</sup> /d
PIF	11,592 m <sup>3</sup> /d	12,887 m <sup>3</sup> /d
BOD <sub>5</sub> concentration	154 mg/l	154 mg/l
Average Loading	287 kg/d	322 kg/d
Maximum Month Loading <sup>(1)</sup>	430 kg/d	483 kg/d
TSS concentration	151 mg/L	151 mg/L
Average Loading	281 kg/d	348 kg/d
Maximum Month Loading <sup>(1)</sup>	421 kg/d	521 kg/d
TKN concentration	32.9 mg/l	32.9 mg/l
Average Loading	61 kg/d	76 kg/d
Maximum Month Loading <sup>(1)</sup>	92 kg/d	114 kg/d
TP concentration	4.76 mg/l	4.76 mg/l
Average Loading	9.0 kg/d	11 kg/d
Maximum Month Loading <sup>(1)</sup>	13.5 kg/d	16.5 kg/d
Influent Concentrations: Refer to Table 3.2 Class EA (page 149 of the pdf)		
Typical maximum month loading = 1.5 x average loading <sup>1</sup>		
**Reference: Arthur WWTP class EA		

The effluent objectives and compliance limits for the WWTP are provided in Table 6 below:

Table 6: Effluent Criteria for the Proposed ESR Phases

Parameter	Phase 1 Capacity (1,860 m <sup>3</sup> /d)		Phase 2 Capacity (2,300 m <sup>3</sup> /d)	
	Objective Concentration	Compliance Limit	Objective Concentration	Compliance Limit
BOD <sub>5</sub>	6 mg/L	10 mg/L	5 mg/L	10 mg/L
TSS	6 mg/L	10 mg/L	5 mg/L	10 mg/L
TP	0.21 mg/L	0.25 mg/L	0.17 mg/L	0.25 mg/L
		3.5 mg/L <sup>(1)</sup>		3.5 mg/L <sup>(1)</sup>
TAN	0.6 mg/L	2.8 mg/L <sup>(2)</sup>	0.5 mg/L	2.8 mg/L <sup>(2)</sup>
E. coli <sup>(3)</sup>	-	100 cfu/100 mL	-	100 cfu/100mL
pH	-	6 - 8	-	6 - 8
Notes: For discharge during all months except October. For discharge during October. <sup>2</sup> Based on a monthly geometric mean.				

It is worth noting that the 2019-2020 Annual Performance Report for the Arthur WWTP states that the annual design flow was 1,329 m<sup>3</sup>/d and the maximum monthly average flow occurred in March 2020 at an ADF of 2,088 m<sup>3</sup>/d. Interestingly, the annual report lists the plant capacity at 1,465 m<sup>3</sup>/d and the Total Phosphorus effluent concentration limit at 1.0 mg/L which we understand should have been 0.25 mg/L in both Phases 1 and 2 as identified in the Class EA.

The biosolids management strategy identified in the Class EA for the plant is as follows:

Phase 1 – No biosolids work is required. Liquid biosolids are shipped to the Lystek regional processing facility located in Dundalk, Ontario.

Phase 2 - Onsite aerobic digestion followed by dewatering of digested sludge using geotextile tubes. The 2 existing (total 4) sludge storage tanks, each of 150 m<sup>3</sup> volume will be converted into anaerobic digester which will reduce the sludge storage capacity to 300 m<sup>3</sup>. Hence, an additional sludge storage volume of 120 m<sup>3</sup> is required (In phase-II expansion, a total of 420 m<sup>3</sup> of sludge storage volume is required based on 240 days of onsite storage capacity, Table 3.14 of Class EA). Upgrades include increased blower capacity and increased sludge transfer pump size. The dewatered cake will be land-applied seasonally.

### 1.3.2.5 Projected cost estimates identified in the Class EA

The project liquid stream costs identified in the Class EA were as follows:

Table 7: Project Cost Estimates for WWTP Phase-I and Phase-II Liquid Stream Expansions

Treatment Process	Phase-I	Phase-II
General/Miscellaneous	\$340,000	\$569,000
Headworks	\$0	\$3,020,000
Storage Lagoon Conveyance Upgrades	\$695,000	\$1,275,000
Blowers, Standby Power, and Other Common Upgrades	\$681,000	\$0 (2)
Equalization Tank	\$1,674,000	\$0
Secondary Treatment	\$0	\$809,000
Sub Total	\$3,390,000	\$5,673,000
Contingency (30%)	\$1,017,000	\$1,702,900
Engineering (12%)	\$407,000	\$681,000
Liquid Treatment Train Total	\$4,814,000	\$8,056,000
Notes: All costs are conceptual-level opinions of probable costs and are considered to be accurate to within -25 to +40percent and are exclusive of HST. Assumed increase in blower capacity provided in Phase 1 is adequate for Phase 2 demand.		

- 7.1 Referenced Reports and Guidelines
- Arthur WWTP Environmental Study Report, by XCG Consultants Ltd., dated 17 August 2016.
- MOE (now MECP) Design Guidelines for Sewage Works, 2008
- Municipal Comprehensive Review Municipal Servicing Analysis, by WSP, dated 31 January 2022
- 2019-2020 Arthur WWTP Annual Performance Report

The Class EA also lists the biosolids costs as presented in Table 8 below:

Table 8: Project Cost Estimates for WWTP Phase-I and Phase-II Solid Stream Expansions

Treatment Process	Phase-I	Phase-II
Digester	\$0	\$900,000
Biosolids Storage	\$0	\$1,400,000
Solid Treatment Train Total	\$0	\$2,300,000
Notes: All above costs include 12% engineering and 30% contingency.		

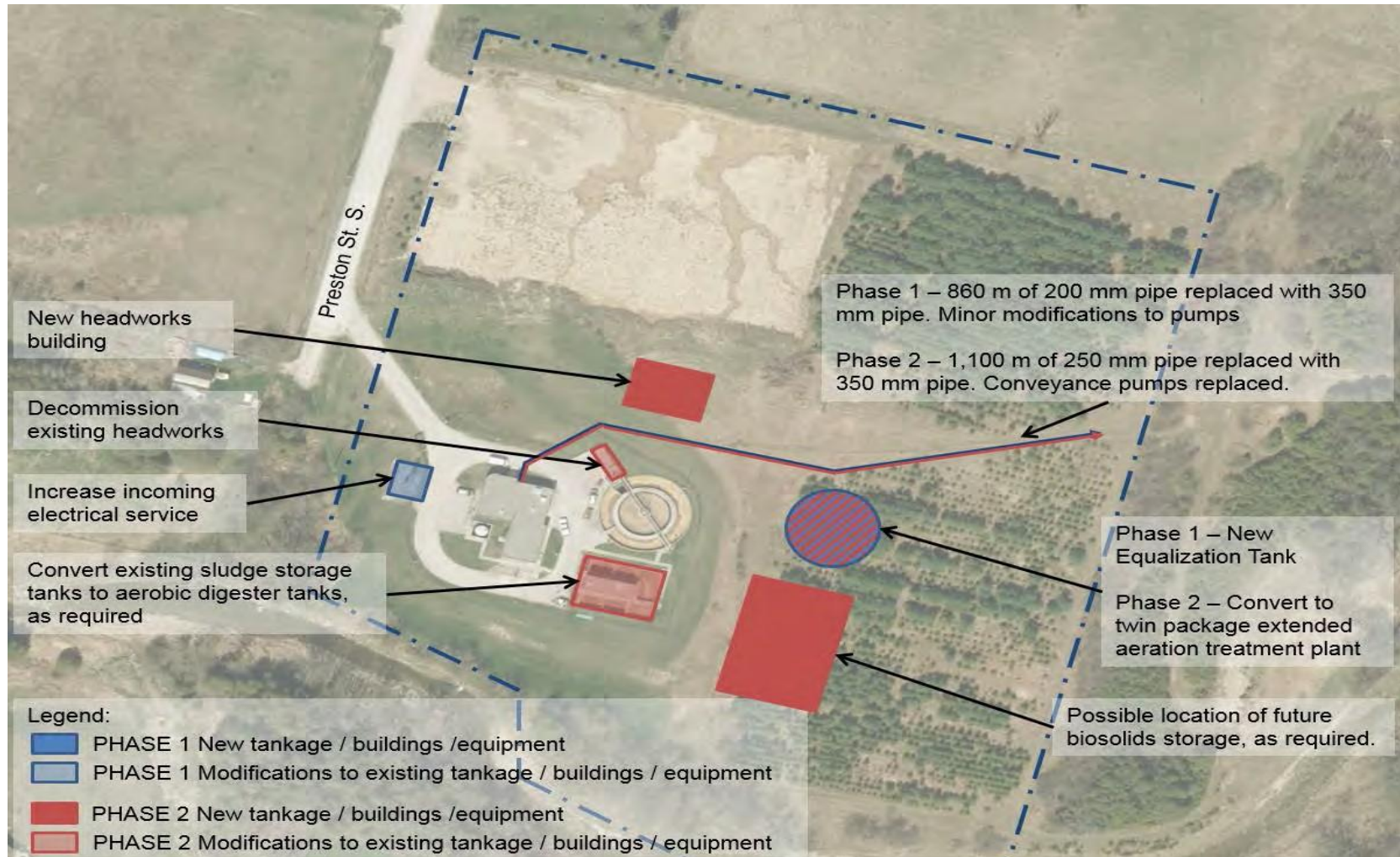


Figure 3: Design concept layout for Phase-I and II expansions as identified in Class

### 1.3.3 Progress to Date on Wastewater Treatment Plant (Completed)

The proposed Phase-I components in the Class EA report are already completed, resulting in an updated capacity of 1,860 m<sup>3</sup>/day from 1,465 m<sup>3</sup>/day (Annual Performance Report 2022). The key components covered under the phase-I expansion as identified in the Class EA are given below:

- a. Primary treatment:** A new 2,100m<sup>3</sup> equalization tank is added to restrict the flow at 6,450 m<sup>3</sup>/d (design capacity of the headworks building). Hence, no changes in the headworks building are required. *This is confusing as we understand the existing treated capacity of the headworks system before the Phase I expansion was listed at 5,045 m<sup>3</sup>/d.*
- b. Secondary treatment:** It is important to mention that based on the influent BOD concentrations reported in the 2021 annual performance report, the required aeration tank volume for the Phase-I rated capacity of 1869 m<sup>3</sup>/day at F/M ratio, MLSS concentrations of 0.1 and 4000 mg/l is 1612 m<sup>3</sup>. Notably, the existing aeration tank volume is only 1,073 m<sup>3</sup> and needs an additional volume of 612 m<sup>3</sup>. In contrast, there is no provision for additional volume considered for phase-I expansion. Consequently, WWTP is operated at a higher volumetric loading of 0.378 kg/m<sup>3</sup>/day and F/M ratio of 0.15 than that specified in the MECP guidelines i.e., 0.17 to 0.24 kg/m<sup>3</sup>/day and 0.05 to 0.15. Moreover, the existing surface area for clarification is only 143 m<sup>2</sup> whereas the minimum required additional surface area for clarification for Phase-I expansion i.e., 1860 m<sup>3</sup>/day is 143 m<sup>2</sup> based on a SOR of 37 m<sup>3</sup>/m<sup>2</sup>/day at a design peak flow of 7,069 m<sup>3</sup>/d plus 10% (10% of ADF) recycled flow from the tertiary filters (Table 4.2, Class EA). Thus, there seems to be insufficient clarification tankage available in Phase-I, which could be the reason for the current poor plant performance. It is also worth noting that the historical data from the class EA suggests that the peak SOR was 59 m<sup>3</sup>/m<sup>2</sup>/day rather than the MECP specified maximum value of  $\leq 37$  m<sup>3</sup>/m<sup>2</sup>/day which would exacerbate this issue for Phase-I.
- c. Tertiary Filtration / Chemical Addition / UV Disinfection:** The MDF to existing tertiary filters will be 7,068 m<sup>3</sup>/day, resulting in a need for a surface area of 25.32 m<sup>2</sup> which is less than the existing surface area of 27.2 m<sup>2</sup>. Hence, existing systems have a sufficient capacity for Phase-I expansion.
- d. Effluent Storage and Conveyance:** The existing conveyance system consists of a 2.56 km forcemain and transfer pumps. The existing forcemain consists of 860 m of 200 mm diameter pipe, 1,100 m of 250 mm diameter pipe, and 600 m of 300 mm diameter pipe. There are 2 pumps each rated at 58.5 L/s (5,054 m<sup>3</sup>/d) *which closely matches the peak flow of the existing headworks facility.* Phase-I peak flows would be restricted to approximately 6,450 m<sup>3</sup>/d through the operation of an equalization tank. There is insufficient capacity in the conveyance system to transfer projected equalized peak flows. Additional conveyance capacity can be provided by replacing the entire 860 m length of the existing 200 mm diameter pipe with 350 mm diameter pipe. It is anticipated that the existing transfer pumps may have sufficient capacity required via minor modifications (i.e. new impellers). The projected storage requirement at the Phase-I ADF capacity is approximately 246,000 m<sup>3</sup>,

and the existing capacity of the treated effluent holding ponds is approximately 340,000 m<sup>3</sup> which is sufficient for Phase-I.

- e. **Sludge Management:** No upgrade is required. Liquid biosolids will be hauled to the Lystek regional processing facility located in Dundalk, Ontario.



Figure 4: Google Earth Aerial View of Arthur WWTP December 2023

### 1.3.4 Reserve Capacity Analysis Post Phase-I Expansion

As per the reserve capacity analysis report submitted by Triton Engineering Services Limited (dated April 4, 2022) on water supply and sewerage systems in Arthur, the ADF of the 3 years (2019, 2020, & 2021) was 1,293 m<sup>3</sup>/day, resulting in a reserve capacity of 567 m<sup>3</sup>/day (Table 9). For the reserve capacity analysis, the adopted per capita flow in the Triton report was 350 Lpcd. On the contrary, the ESR report recommended a per capita flow of 460 Lpcd (370 Lpcd + I/I of 90 Lpcd). Therefore, in our assessment, we adapted the ESR recommended value and revised the reserve capacity assessment. Considering the per capita flow of 460 Lpcd, an additional population of 1,232 can be further served using this available reserve capacity. Considering a residential

density of 2.7 ppu (based on the 2018 growth management plan), 456 additional new units can be serviced. Per Table 3 of the Triton report, there are 212 committed development residential units, resulting in 244 uncommitted development residential units. These available uncommitted development residential units are equivalent to a population of 658 and a flow capacity of 303 m<sup>3</sup>/day.

Table 9: Reserve Capacity Analysis Post Phase-I Execution

Item	Description	2021
1	Updated capacity of the WWTP (m <sup>3</sup> /day)	1,860
2	Average ADF (m <sup>3</sup> /day)	1,293
3	Available reserve capacity (1-2) (m <sup>3</sup> /day)	567
4	Future development per capita flow (m <sup>3</sup> /day)	0.460
5	The additional population that can be served (3/4)	1,232
6	PPU	2.7
7	New additional residential units that can be served (5/6)	456
8	Committed residential developmental units	212
9	Available uncommitted residential developmental units (7-8)	244
10	Available uncommitted flow capacity (9 x 6 x4) (m <sup>3</sup> /day)	<b>303</b>

### 1.3.5 Future Development Scenarios

#### 1.3.5.1 Phase-II expansion: rated capacity of 2,300 m<sup>3</sup>/day

We believe that the Township is now focusing on expanding the WWTP plant capacity to 2,300 m<sup>3</sup>/day (Phase-II development identified in the class EA). The components proposed under the Phase-II expansion are given below:

- a. Primary treatment:** A new headworks building is required because the existing system has reached the end of its useful life, hence, the old headworks system will be decommissioned. The headworks components should be designed at a Phase-II PIF i.e., 12,887 m<sup>3</sup>/day. However, this proposed capacity of the equipment is not going to be sufficient for the future growth. Hence, it is vital to add the provisions for the future growth. *Moreover, it is typical to provide equalization after the headworks system to mitigate the potential for solids deposition in the equalization tankage.*
- b. Secondary treatment:** The proposed solution to achieve Phase-II plant secondary treatment capacity is to twin the existing extended aeration package plant. In this context, it is proposed that the equalization tanks constructed in Phase-I will be going to be converted into an extended aeration tank and secondary clarifier. *However, this proposed approach results in directing the Peak Instantaneous Flows to the downstream processes such as secondary treatment system, clarifiers, tertiary filters, and UV disinfection system, consequently, disturbing the biological process and need to design secondary clarification and tertiary filtration process units to accept the peak instantaneous flows.*

- c. Tertiary Filtration / Chemical Addition / UV Disinfection System:** The capacity of existing tertiary filtration, UV disinfection system, and chemical addition processes is sufficient to treat the Phase-II ADF capacity of 2,300 m<sup>3</sup>/d. *While the chemical addition processes may be sufficient, removing the equalization tankage from the process train as proposed in the Phase-II expansion plan (Class EA) requires additional filters and UV disinfection system capacity. The existing tertiary filters have an ECA-rated capacity of 6,500 m<sup>3</sup>/d at a filtration rate of 2.7 L/m<sup>2</sup>/sec. They could potentially treat a maximum inflow of 7,955 m<sup>3</sup>/day (based on the MECF design criteria of 3.3 L/m<sup>2</sup>/sec and an existing total filter area of 27.9 m<sup>2</sup>). In contrast, considering design based on the MDF of 8828 m<sup>3</sup>/day resulted in a need for additional filters having a minimum surface area of 4.65 m<sup>2</sup>. Likewise, the UV disinfection system has an ECA-rated capacity of 6,500 m<sup>3</sup>/d and needs an expansion at MDF flow. However, without the equalization tankage, the filters and UV disinfection system will see peak flows of as high as 12,887 m<sup>3</sup>/d. Thus, this strategy would require either adding additional filtration and UV disinfection capacity or adding new equalization tankage to buffer out the peak flows.*
- d. Effluent Storage and Conveyance:** For Phase-II capacity, additional conveyance capacity would be added by upgrading the remaining 1,100 m of 250 mm diameter pipe to 350 mm diameter pipe, and through the installation of new conveyance pumps. For Phase-II, approximately 304,000 m<sup>3</sup> of storage will be required which is less than the existing lagoon storage of 340,000 m<sup>3</sup>. Thus, there is no additional storage capacity required for the Phase-II expansion.
- e. Sludge Management:** The sludge management options offered in the Class EA, onsite aerobic digestion, with onsite storage of biosolids using geotextile tubes are proposed. This involves the conversion of two of the four existing sludge holding tanks (each is sized for 150 m<sup>3</sup>) to aerobic digesters. However, based on our calculations the sludge production will range between 50 to 55 m<sup>3</sup>/day. This resulted in a need for a total aerobic digestion volume of 1573 m<sup>3</sup>. Considering the existing digestion volume of 468 m<sup>3</sup> and converting all sludge storage tanks into aerobic digesters, still 505 m<sup>3</sup> of additional volume is required. Upgrades include increased blower capacity and increased sludge transfer pump size. Moreover, an additional sludge storage tank having a total volume of 400 m<sup>3</sup> is required for the dewatered sludge storage. The dewatered cake will be land-applied seasonally.

### 1.3.5.2 Tribute/Sorbara Arthur Holdings Inc. Developments

Tribute/Sorbara Arthur Holdings Inc. is assessing the potential for further expanding the Arthur WWTP to service an additional 823 or 872 residential units. Consequently, the plant capacity will be further developed to 3,346 m<sup>3</sup>/day or 3,406 m<sup>3</sup>/d serving the range of development considered by Tribute/Sorbara Arthur Holdings Inc.. In this report, the flow projected with 823 and 872 lots is denoted as Scenario-I and II expansions as presented in Table 10 below:

Table 10: Design flows for Arthur WWTP including Phase-II expansion flow plus proposed Tribute/Sorbara Arthur Holdings Inc. Developments

		<b>Scenario-1 (Phase-II Flow + 823 Lots)</b>	<b>Scenario-2 (Phase-II Flow + 872 Lots)</b>
Population		7,379	7,379
Peaking Factor <sup>1</sup>		5.6	5.6
Maximum Day Factor <sup>2</sup>		3.8	3.8
Average Dry Weather Flow	m <sup>3</sup> /d	2,680	2,730
	L/s	31.03	31.60
Average Wet Weather Flow or Average Day Flow (ADF)	m <sup>3</sup> /d	<b>3,333</b>	<b>3,395</b>
	L/s	38.60	39.30
Maximum Day Flow	m <sup>3</sup> /d	12,665	12,899
	L/s	146.60	149.30
Peak Dry Weather Flow	m <sup>3</sup> /d	15,014	15,289
	L/s	174	177
Peak Wet Weather Flow	m <sup>3</sup> /d	16,658	16,965
	L/s	192.8	196.35
Peak Instantaneous Flow	m <sup>3</sup> /d	18,509	18,849
	L/s	214.23	218
<sup>1</sup> Peaking Factor and Maximum day Factor <sup>2</sup> are based on the class EA report (page 349)			

### 1.3.5.3 Sewage Pumping Station

Based on the current Tribute concept plan, it has been determined that a new sewage pumping station will be required in the Tribute Arthur development for the eastern portion of the proposed development. The pumping station will be located on a 0.05 ha parcel of land on Block 70 at the intersection of Street R and Eliza Street. The pumping station will pump through a forcemain to the sanitary collection system at the intersection of Street A and Eliza Street. The station is estimated to have a capacity of approximately 20 L/s.

Figure 5 below illustrates a plan showing the proposed sewage pumping station. This plan shows that an area of approximate size 0.05 ha is required to house these facilities.

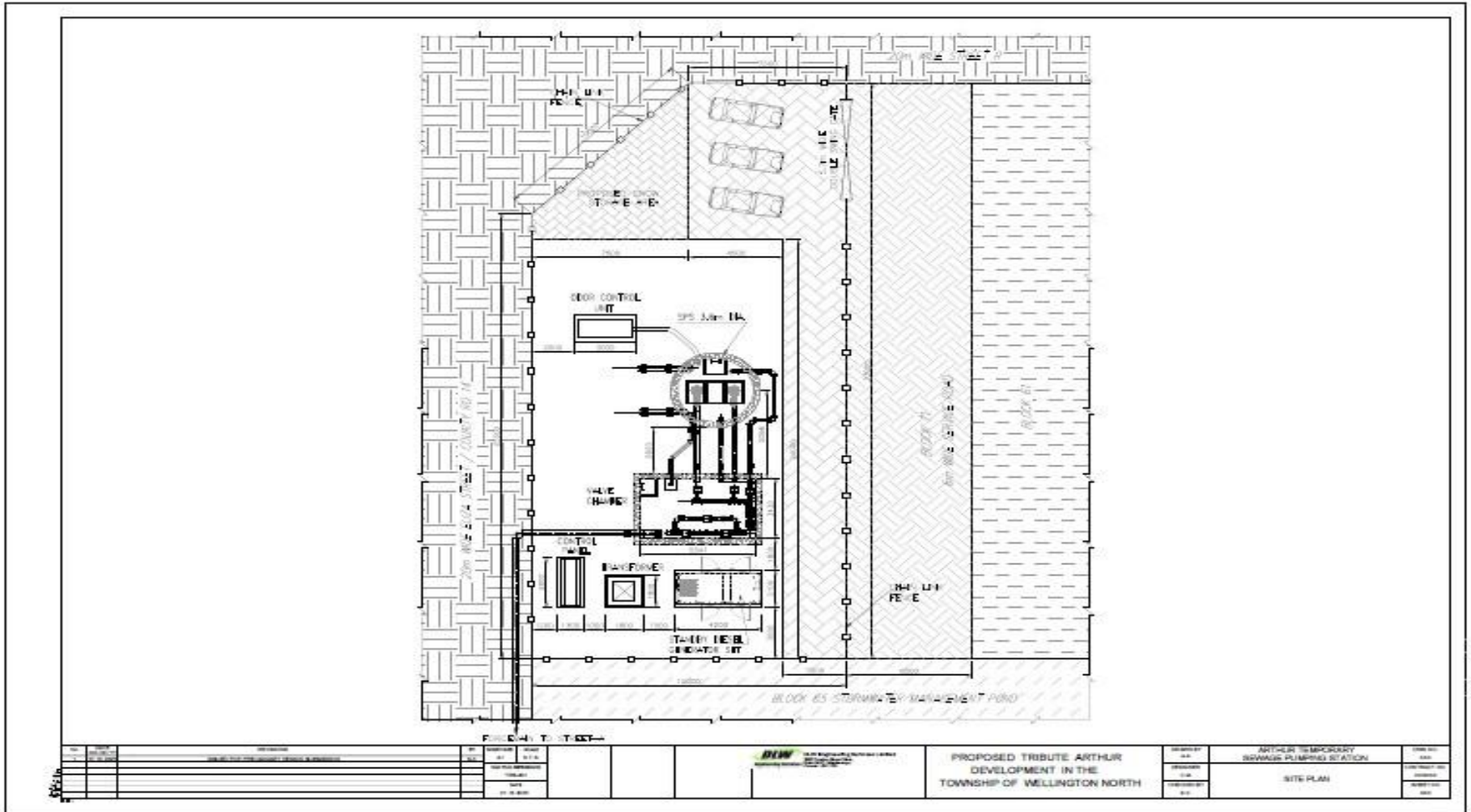


Figure 5: Proposed Tribute Arthur Sewage Pumping Station

### 1.3.5.4 Influent and effluent wastewater characteristics

The design concentration of the key parameters adapted for the Phase-I and Phase-II expansions as stated in the class EA, 2016 report are lower than that observed in the Arthur WWTP annual performance reports for the years 2019, 2020, and 2021 (Figure 5). Hence, it seems logical to design the WWTP at influent concentrations based on the APRs rather than Class EA-specified concentrations to avoid the secondary treatment plant operation at a higher organic loadings rate. While, the effluent criteria are based on the approved compliance limit stated in the class EA, 2016 report (Table 2.9, Future effluent limits). The maximum month loading factor of 1.5 is the same as adapted to design phase-I and phase-II expansions identified in the Class EA, 2016. The proposed design influent loading rates for the various plant expansion options are presented in Table 11 below:

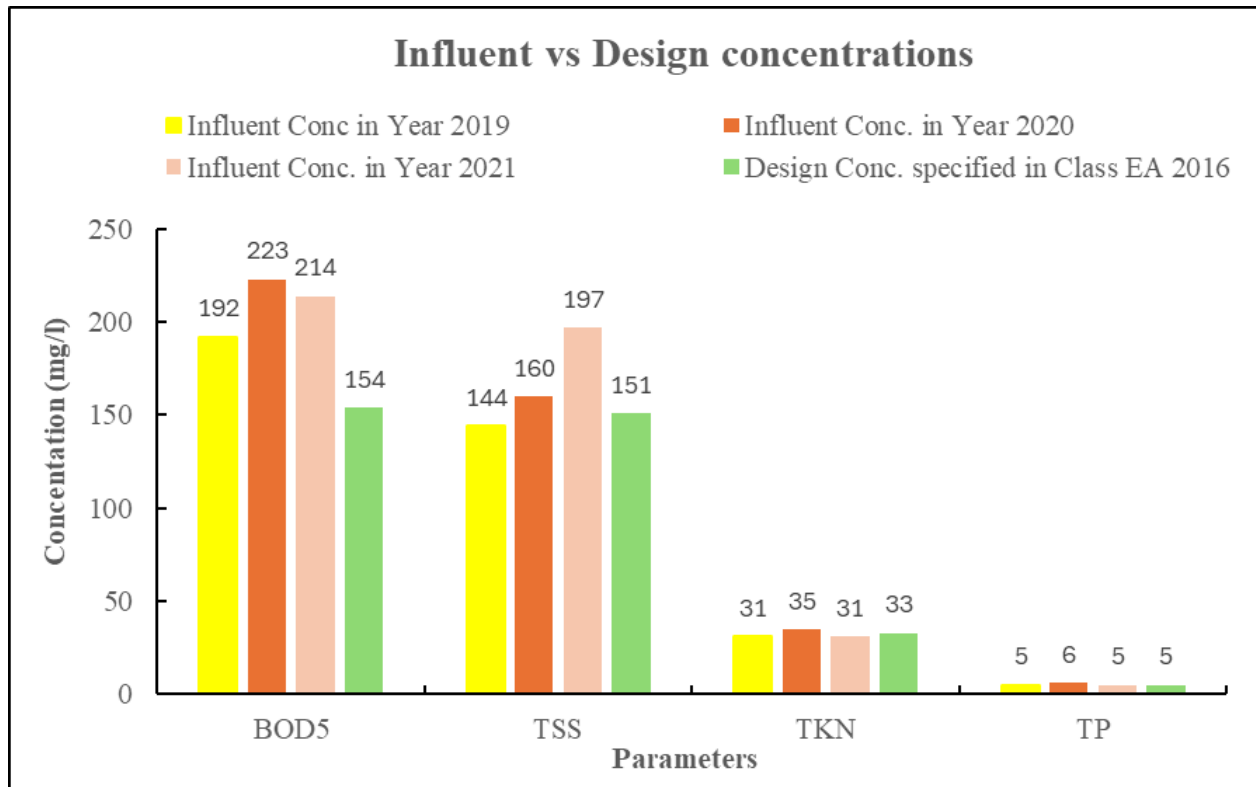


Figure 6: The influent vs design concentrations based on the class EA

Table 11: Proposed Design Influent Loadings for Arthur WWTP based on the APR

Parameter	Design Conc. (mg/l) <sup>1</sup>	APR based influent Conc. (mg/l) <sup>2</sup>	Average Day Loading (kg/d) <sup>3</sup>		Maximum Month Loading (kg/d) <sup>4</sup>	
			Scenario-1	Scenario-2	Scenario-1	Scenario-2
Biological Oxygen Demand (BOD <sub>5</sub> )	154	213	709.93	723.14	1064.89	1084.70
Total Suspended Solids (TSS)	151	197	656.60	668.82	984.90	1003.22
Total Kjeldal Nitrogen (TKN) <sup>5</sup>	33	40	133.32	135.80	199.98	203.70
Total Ammonia Nitrogen (TAN) <sup>6</sup>	26	32	106.66	108.64	159.98	162.96
Total Phosphorous (TP)	4.76	5.1	17.00	17.31	25.50	25.97
Alkalinity	250 – 350	250 – 350				

<sup>1</sup>Design concentration as per the class EA

<sup>2</sup>Design concentration as per the annual performance report 2022

<sup>3</sup>Average Day Loading = <sup>2</sup>APR based Design Concentration × ADF

<sup>4</sup>Maximum Month Loading = Average Day Loading × 1.5

<sup>5</sup>Aeration with nitrification process (Single sludge system) should be designed at Peak daily TKN loading. Hence, the maximum TKN concentration of the year 2022 is selected.

<sup>6</sup>Assumed that TAN is approximately 80% of the influent TKN

The proposed effluent objectives and compliance limits for the various development scenarios are presented in Table 12, below:

Table 12: Projected compliance limits based on the scenarios I and II flows

Parameter	Compliance limits (mg/L) at Phase-II flow (2,300 m <sup>3</sup> /day)	Kg loadings (kg/day) <sup>1</sup> based on the phase-2 flow of 2,300 m <sup>3</sup> /day	Revised compliance limits (mg/L) based on Scenario-I flow (3,333 m <sup>3</sup> /day) <sup>2</sup>	Revised compliance limits (mg/L) based on Scenario-II flow (3,395 m <sup>3</sup> /day)	Proposed Effluent Objective conc. (mg/L)
Biological Oxygen Demand (BOD <sub>5</sub> )	10	23	6.8	6.7	≤5
Total Suspended Solids (TSS)	10	23	6.8	6.7	≤5
Total Ammonia Nitrogen (TAN)	3.5 2.8	8.05 6.5	2.4 1.9	2.3 1.85	≤1.5 ≤1
Total Phosphorous (TP)	0.25	0.57	0.17	0.168	≤0.1
pH	6-8	-	6-8	6-8	6-8
<i>E. Coli</i>	100 cfus/100 ml	-	100 cfus/100 ml	100 cfus/100 ml	100 cfus/100 ml
<sup>1</sup> Kg compliance effluent loading = effluent conc. X (Flow/1000) <sup>2</sup> Revised effluent concentration = (Kg loadings <sup>1</sup> X 1000)/Flow (m <sup>3</sup> /day)					

### 1.3.6 Proposed Wastewater Treatment Process Overview

The proposed WWTP technology for future developments (Phase-II, Scenario-I, and II) will be the same as the existing one i.e., extended aeration technology followed by clarification and tertiary filtration. However, it seems sustainable to retain the flow equalization in the process train for any future expansions (post Phase-I execution) to avoid sizing of the downstream treatment units for peak flows. The design criteria and sizing for the components such as the aeration tank, secondary clarifier, tertiary filters, aerobic digestion, sludge holding units, and UV system will be based on flow equalization. To reduce the sludge hauling cost, we propose aerobic digestion followed by dewatering and storage, which seems more viable than direct hauling of the WAS to the centralized handling facility.

### 1.3.6.1 Headworks building

The wastewater from the existing and new sewage pumping stations (dedicated to the south Fergus area) will be pumped to the WWTP site. After that, it will be pumped to the new headworks building which is going to be equipped with a new rotary drum screen followed by the new vortex grit removal system. The design PIF in the phase-II, Scenario-I, and Scenario-II developments are 12,887 m<sup>3</sup>/day (537 m<sup>3</sup>/hr), 18,509 m<sup>3</sup>/day (771 m<sup>3</sup>/hr), and 18,849 m<sup>3</sup>/day (785 m<sup>3</sup>/hr). The Class EA recommended providing headworks equipment with design capacities at the phase-II PIF. However, this capacity is not sufficient for future tribute developmental flows. Hence, we propose providing a single train of drum screen sized for a PIF of 785 m<sup>3</sup>/hr and the grit removal system will be in two trains, each designed at half of the peak flow i.e., 394 m<sup>3</sup>/hr. Moreover, the grit removal system is also equipped with grit slurry pumps and classifiers with 1 working and 1 standby arrangement. The wastewater post-screening and grit removal will be channeled to the equalization tank.

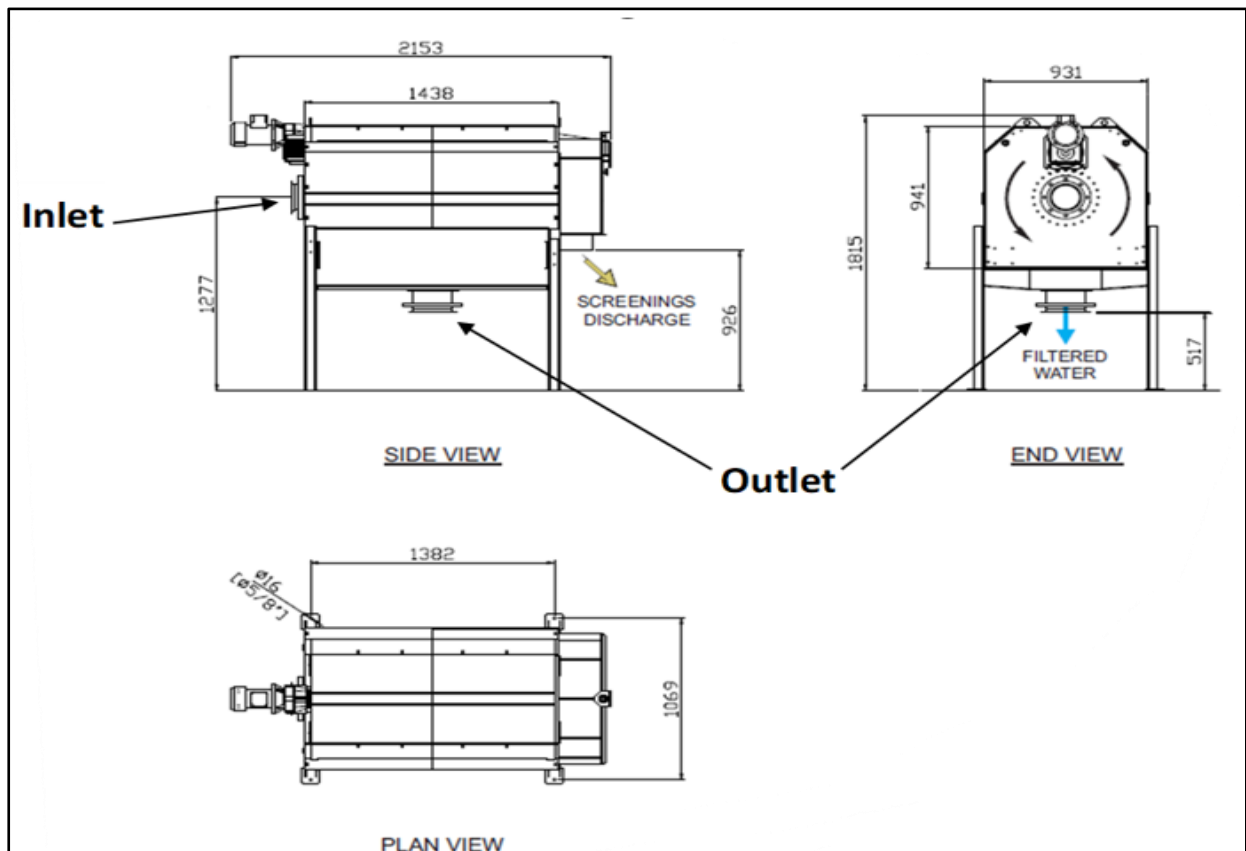


Figure 7: Rotary drum screen

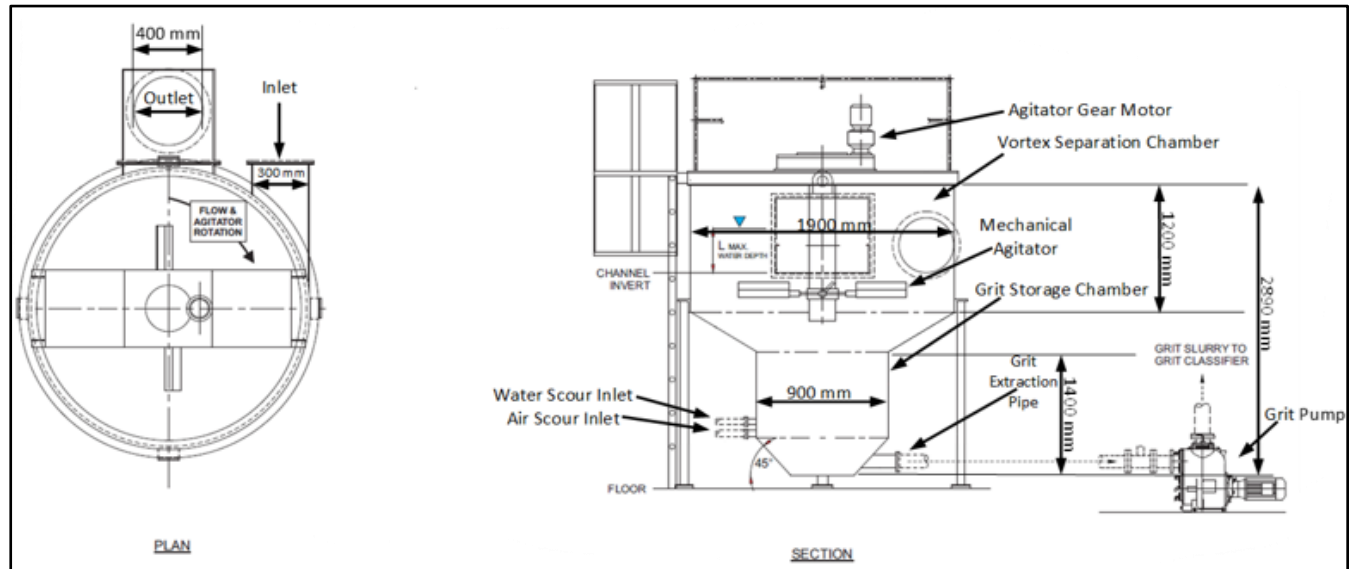


Figure 8: Vortex Grit Separator

### 1.3.6.2 Equalization volume

The equalization of the peak flows will overcome the issue of process disturbances and fewer treatment units for the downstream processes. In accordance with the Class EA, the addition of an equalization tank (2,100 m<sup>3</sup>) in the phase-I expansion limits 6,500 m<sup>3</sup>/day of flow to the tertiary filters which is well within the filter system’s actual design capacity of 7,950 m<sup>3</sup>/day. However, the proposed conversion of the equalization tank to a secondary treatment system in Phase-II (as proposed in the class EA) will completely remove the equalization capacity. Consequently, there is a need to design the downstream units to handle future peak flows. Furthermore, the higher PIF and MDF factors of 5.6 and 3.8 as given in the class EA report also highlighted the importance of equalization volumes. In this context, it is proposed to provide sufficient equalization volumes based on the PIF minus ADF. In addition, all the downstream process units will be designed on MDF rather than PIF. The design hydraulic retention time of the equalization volume at PIF minus ADF will be 3 hours. The required equalization volume for each flow condition is given in Table 13 below:

Table 13: Required equalization volume for different flow conditions

ADF	PIF-ADF	HRT (hours) at (PIF-ADF) flow	Equalization volume (m <sup>3</sup> )	HRT (hours) at ADF
2,300 m <sup>3</sup> /day (Phase-II)	10,577 m <sup>3</sup> /day	3	1,323	13.6
3,333 m <sup>3</sup> /day (Scenario-I)	15,176 m <sup>3</sup> /day	3	1,897	13.6
3,395 m <sup>3</sup> /day (Scenario-II)	15,455 m <sup>3</sup> /day	3	1,932	13.6

### 1.3.6.3 Secondary treatment system

The secondary biological treatment system is based on extended aeration technology to enhance the influent COD and TKN removal efficiency. The extended aeration technology is a modified activated sludge process that is operated at a high MLSS, low F/M, high DO concentration, longer HRT, and SRT. Such operating conditions enable the endogenous respiration process, resulting in a lower quantity of sludge production. The process design criteria are given in Table 14. The major components of the extended aeration system would include aeration tanks with valves and appurtenances, flowmeters, dissolved oxygen (DO) probes, blower systems, sensors, and other ancillary equipment. All exterior equipment associated with the system will be provided in weatherized, prefabricated enclosures complete with covers and doors for access and equipment maintenance. The system will be supplied with programmable logic controllers, which will be tied to a centralized SCADA system controlling all components of the WWTP. Critical process equipment, such as pumps, blowers, etc. will be provided in a duty-standby configuration. It is worth noting that for the Phase-II expansion, it is proposed to convert the equalization tank (constructed in the phase-I expansion) to the secondary treatment system. This conversion results in an inflow of the peak flows to the aeration tank, reducing the HRT and possibly carrying excessive TSS to the secondary clarifiers which may increase the sludge production volumes.

Table 14: Aeration Design Criteria are based on the flow conditions

Developmental Phases	Design
Phase-II (ADF: 2,300 m <sup>3</sup> /day)	<ul style="list-style-type: none"> <li>• Total required volume: 2130 m<sup>3</sup></li> <li>• Nos of existing tanks (2) with a total volume of 1,073 m<sup>3</sup></li> <li>• Required additional aeration tank volume of 1,134 m<sup>3</sup></li> <li>• Design SRT: 21 days (&gt;15 days)</li> <li>• F/M: 0.1 (0.05-0.15)</li> <li>• HRT: 22 hours (Minimum 15 hours)</li> <li>• MLSS: 3,800 mg/l (3000-5000 mg/l)</li> </ul>
Scenario-I (ADF: 3,333 m <sup>3</sup> /day)	<ul style="list-style-type: none"> <li>• Total required volume: 3,227 m<sup>3</sup></li> <li>• Existing tanks volume post phase-II expansion will be 1,073 m<sup>3</sup> + 1,134 = 2207 m<sup>3</sup></li> <li>• Required additional aeration tank volume of 1,020 m<sup>3</sup></li> <li>• F/M: 0.095 (0.05-0.15)</li> <li>• Design SRT: 22 days (&gt;15 days)</li> <li>• HRT: 23 hours (Minimum 15 hours)</li> <li>• MLSS: 3,800 mg/l (3,000-5,000 mg/l)</li> </ul>
Scenario-II (ADF: 3,395 m <sup>3</sup> /day)	<ul style="list-style-type: none"> <li>• Total required volume: 3,227 m<sup>3</sup></li> <li>• Existing tank volume post-scenario-I expansion will be 3,227 m<sup>3</sup>, which is sufficient for Scenario-II.</li> <li>• F/M: 0.1 (0.05-0.15)</li> <li>• Design SRT: 21 days (&gt;15 days)</li> <li>• HRT: 22 hours (Minimum 15 hours)</li> <li>• MLSS: 3,800 mg/l (3,000-5,000 mg/l)</li> </ul>

#### 1.3.6.4 Chemical Phosphorus Removal

A chemical means is used for phosphorus removal. A required dose of the chemical i.e., Alum is added downstream of the aeration tank, where it reacts with the phosphorus to form precipitates. These precipitates settled in the secondary clarifier along with WAS. The required chemical dose rate for the influent flow conditions is given in Table 15 below.

Table 15: Required Alum Dosing Rate at Various Developmental Phases

Influent flow	Kg AlSO <sub>4</sub> required	Alum solution concentration	Required dosing rate
Phase-II (ADF: 2,300 m <sup>3</sup> /day)	190	50%	290 L/day
Scenario-I (ADF: 3,333 m <sup>3</sup> /day)	263	50%	400 L/day

Scenario-II (ADF: 3,395 m <sup>3</sup> /day)	268	50%	410 L/day
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Moreover, for liquid chemical storage, existing WWTP has a storage capacity of 23 m<sup>3</sup>. This capacity is equivalent to 56 days of storage at Scenario-II flow and sufficient. The required dosing pump capacity will be 268 L/day with 1 working and 1 standby arrangement.

### 1.3.6.5 Secondary clarifiers

The treated wastewater will be conveyed to the clarifiers to remove suspended solids. The part of the settled solids will be recycled (Return Activated Sludge or RAS) to the aeration tank to maintain the required mixed liquor suspended solids (MLSS). In contrast, excess solids will be wasted as waste-activated sludge (WAS). The secondary clarifier's design is based on the MECF guidelines. The required surface area of the clarifier is designed against the peak flow and solid loading rate at maximum day flow. However, adding equalization volume will shave the PIF to the clarifiers, resulting in the clarifiers' design at maximum day flow. The secondary clarifiers are center inlet and equipped with a sludge collection mechanism and scum skimming mechanism.

Table 16: Required secondary clarification based on various flow conditions

Design Flow condition: MDF	Design criteria	Clarifiers details
12,776 m <sup>3</sup> /day (Phase-II)	Solid loading rate (SLR) is 170 kg/m <sup>2</sup> /day (at MDF +200% RAS+ 10% recycled flow from tertiary filters)	<ul style="list-style-type: none"> <li>Required total surface area for clarification is 285 m<sup>2</sup></li> <li>The existing area of the clarifier is 143 m<sup>2</sup> + 271 m<sup>2</sup> (after converting part of equalization volume into clarifier) = 414 m<sup>2</sup></li> <li>Its surface area is sufficient</li> </ul>
18,331 m <sup>3</sup> /day (Scenario-I)	Solid loading rate (SLR) is 170 kg/m <sup>2</sup> /day (at MDF +150% RAS+ 10% recycled flow from tertiary filters)	<ul style="list-style-type: none"> <li>Required total surface area for clarification is 409 m<sup>2</sup></li> <li>The existing area of the clarifier is 414 m<sup>2</sup></li> <li>Its surface area is sufficient</li> </ul>
18,669 m <sup>3</sup> /day (Scenario -II)	Solid loading rate (SLR) is 170 kg/m <sup>2</sup> /day (at MDF +150% RAS+ 10% recycled flow from tertiary filters)	<ul style="list-style-type: none"> <li>Required total surface area for clarification is 417 m<sup>2</sup></li> <li>The existing area of the clarifier is 414 m<sup>2</sup></li> <li>Its surface area is sufficient</li> </ul>

### 1.3.7 Secondary Treatment Design Evaluation using BIOWIN modeling tool

The existing Phase-I and future developments' secondary treatment system design was evaluated using the BIOWIN modeling tool. It is important to mention that this proposed secondary design treatment intends to maintain TSS concentration  $\leq 20$  mg/l, BOD<sub>5</sub>  $\leq 10$  mg/l, TP  $\leq 1$  mg/l, and TAN  $\leq 1$  mg/l to meet the targeted objective concentrations as specified in the class EA report.

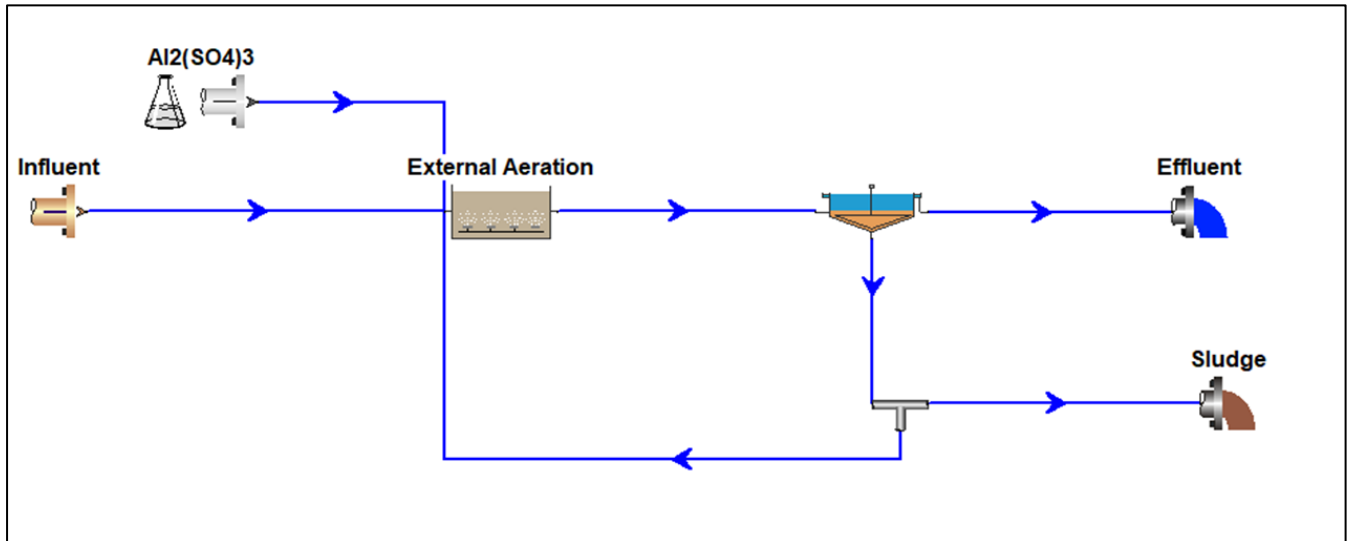


Figure 9: General Process Scheme for the Secondary Treatment System

Table 17: Shows the performance of the secondary treatment system based on the BIOWIN modeling tool

Design Parameters	Conc. (mg/l)	Secondary effluent concentrations				
		Phase-I: 1,860 m <sup>3</sup> /day (Operation at higher MLSS conc.)	Phase-I: 1,860 m <sup>3</sup> /day (based on plant operation at low MLSS conc.)	Phase-II: 2,300 m <sup>3</sup> /day	Scenario-I: 3,333 m <sup>3</sup> /day	Scenario-II: 3,395 m <sup>3</sup> /day
BOD <sub>5</sub>	223	6.2	6.0	4.6	5.6	5.1
TSS	197	26	20	18	19	20
TP	5.5	0.6	0.44	0.45	0.64	0.6
TKN	45	NA	NA	NA	NA	NA
N-NH <sub>3</sub>	36	1.0	1.2	0.71	0.58	0.6
N-NO <sub>3</sub>	NA	26	27	28	29	28.1
MLSS	NA	5,342	4,400	3,900	4,005	4,000
RAS	NA	1.3	1.15	1.2	1.5	1.5
WAS (m <sup>3</sup> /day)	NA	41	51	64	80	89

Table 17 shows that the updated Arthur WWTP plant is operated at an MLSS of 5,342 mg/l, which is higher than the MECAP design limits of 5,000 mg/l. Class EA also reported the WWTP operation at a higher MLSS concentration of 5,221 mg/l. This results in a high TSS concentration of  $>26$

mg/l in the secondary effluent as found in the BIOWIN modeling results. To reduce the effluent TSS concentration and MLSS in the aeration tank it is suggested to increase the WAS flow rate from 41 to 51 m<sup>3</sup>/day which resulted in a lowered TSS and MLSS concentration to 19 mg/l and 4400 mg/l. However, this increase in WAS flow rate also needs a large capacity of the sludge treatment and storage units. Apart from this, the proposed Phase-II secondary treatment system design can meet the effluent criteria but removing equalization tank volume may negatively affect the performance due to the short-circuiting at higher PIF. In fact, the higher PIF and MDF factors of 5.6 and 3.8 bolster the need for equalization volume for smooth operation.

### 1.3.7.1 Tertiary filters

Tertiary filters are provided to further reduce the phosphorus and TSS concentrations in the effluent. The filters are typically designed at peak hourly flow (MECP) with a filtration rate of < 3.3 L/m<sup>2</sup>/sec. The existing WWTP has 6 tertiary filters with a total surface area of 27.9 m<sup>2</sup> with a rated filtration capacity of 7,955 m<sup>3</sup>/day. However, the ECA lists the capacity of the filters at 6,500 m<sup>3</sup>/day. Without the equalization volume, the tertiary filters must be designed for PIF, which needs a huge surface area. The surface area requirement for filtration will increase significantly when PIF from the proposed expansions such as phase-II (12,902 m<sup>3</sup>/day), scenario-I (18,576 m<sup>3</sup>/day), and scenario-II (18,914 m<sup>3</sup>/day) is added. On the contrary, adding equalization volumes enables the filter’s design at MDF rather than PIF as given in Table 18.

Table 18: Tertiary filter surface area requirements at different flow conditions

Design Flow condition: MDF	Design criteria	Tertiary filters details
8,829 m <sup>3</sup> /day (Phase-II)	3.3 L/m <sup>2</sup> /sec	<ul style="list-style-type: none"> <li>Required surface area based on the MDF is 31.62 m<sup>2</sup></li> <li>6 existing filters having a surface area of 27.9 m<sup>2</sup></li> <li>Required additional surface area 3.72 m<sup>2</sup></li> </ul>
12,665 m <sup>3</sup> /day (Scenario-I)	3.3 L/m <sup>2</sup> /sec	<ul style="list-style-type: none"> <li>Required surface area based on the MDF is 46 m<sup>2</sup></li> <li>6 existing filters having a surface area of 27.9 m<sup>2</sup></li> <li>Required additional surface area 18 m<sup>2</sup></li> </ul>
12,899 m <sup>3</sup> /day (Scenario -II)	3.3 L/m <sup>2</sup> /sec	<ul style="list-style-type: none"> <li>Required surface area based on the MDF is 47 m<sup>2</sup></li> <li>6 existing filters having a surface area of 27.9 m<sup>2</sup></li> <li>Required additional surface area 19 m<sup>2</sup></li> </ul>

### 1.3.7.2 Sludge generation

The excess sludge produced will be wasted to maintain the required MLSS concentration in the aeration tank. The addition of the equalization volumes restricts the sludge generation at ADF loadings which significantly reduces the required sludge digestion and storage volumes. The sludge is collected in the existing hopper which has a capacity of 50 m<sup>3</sup>. The collected WAS will be pumped to the aerobic digesters at a solid consistency of approximately 2%. However, based on the historical data the average WAS flow rate (60.4 m<sup>3</sup>/day) is significantly high which might

be associated with the low solid consistently of  $\leq 1\%$  rather than 2% assumed in the class EA report. In fact, MECP guidelines also specified an average WAS of 0.8 to 1% in the extended aeration system. Assuming, WAS gets thickened to an average concentration of 1% in the sludge hopper, the same is adapted in our sludge volume calculations as given in Table 19 below.

Table 19: Sludge generation at different flow conditions

Design Flow condition: ADF	Design criteria	Sludge generation and flow rates
2,300 m <sup>3</sup> /day (Phase-II)	At ADF loading	<ul style="list-style-type: none"> <li>• 396 kg/day (ADF-based)</li> <li>• Percentage solids- 1%</li> <li>• Volume of the WAS is 39 m<sup>3</sup>/day</li> </ul>
3,333 m <sup>3</sup> /day (Scenario-I)	At ADF loading	<ul style="list-style-type: none"> <li>• 571 kg/day (ADF based)</li> <li>• Percentage solids- 1%</li> <li>• Volume of the WAS is 56.3 m<sup>3</sup>/day</li> </ul>
3,395 m <sup>3</sup> /day (Scenario -II)	At ADF loading	<ul style="list-style-type: none"> <li>• 581 kg/day (ADF based)</li> <li>• Percentage solids- 1%</li> <li>• Volume of the WAS is 57.4 m<sup>3</sup>/day</li> </ul>

### 1.3.7.3 Aerobic digestion

To reduce the sludge handling volumes and the VSS content of the WAS, the aerobic digestion process is used. The digested sludge will be transferred to dewatering units before storage whereas supernatant from the stage-2 digester will be pumped upstream of the aeration tank. The recommended aerobic digestion process is a two-stage process, wherein stage-1 constitutes 2/3 of the reactor volume whereas stage-2 holds 1/3 of the reactor volume. The minimum SRT for the aerobic digester should be 45 days (aerobic tank + digester SRT). The required digester volumes with respect to the flows are given in Table 20.

Table 20: Aerobic digestion requirements according to the developmental phases

Design Flow condition: ADF	Design criteria	Required aerobic digester volumes
2,300 m <sup>3</sup> /day (Phase-II)	SRT $\geq$ 45 days (Including aeration tank SRT)	<ul style="list-style-type: none"> <li>• Required total volume for aerobic digestion including 25% for supernatant separation is 1,195m<sup>3</sup></li> <li>• The Existing Aerobic Digester Volume is 468 m<sup>3</sup></li> <li>• Hence an additional volume of 1,105 m<sup>3</sup> is required.</li> <li>• Convert existing 4 sludge holding tanks each 150 m<sup>3</sup> into Aerobic digestors resulting in a total volume of 600 m<sup>3</sup>. Therefore, need an additional volume of 127 m<sup>3</sup></li> </ul>

Design Flow condition: ADF	Design criteria	Required aerobic digester volumes
		<ul style="list-style-type: none"> <li>Digested sludge flow rate @ 2% solids are 15.86 m<sup>3</sup>/day</li> </ul>
3,333 m <sup>3</sup> /day (Scenario-I)	SRT ≥ 45 days (Including aeration tank SRT)	<ul style="list-style-type: none"> <li>Required total volume for aerobic digestion including 25% for supernatant separation is 1,639 m<sup>3</sup></li> <li>The Existing Aerobic Digester Volume post phase-II expansion is 1,195 m<sup>3</sup>. Hence, need an additional volume of 444 m<sup>3</sup></li> <li>Digested sludge flow rate @ 2% solids are 23 m<sup>3</sup>/day</li> </ul>
3,395 m <sup>3</sup> /day (Scenario -II)	SRT ≥ 45 days (Including aeration tank SRT)	<ul style="list-style-type: none"> <li>Required total volume for aerobic digestion including 25% for supernatant separation is 1,747 m<sup>3</sup></li> <li>The Existing Aerobic Digester Volume post phase-II expansion is 1,195 m<sup>3</sup>. Hence, need an additional volume of 552 m<sup>3</sup></li> <li>Digested sludge flow rate @ 2% solids are 23.25 m<sup>3</sup>/day</li> </ul>

### 1.3.7.4 Sludge dewatering and storage Yard

The aerobically digested sludge will be further dewatered to a solid concentration of 18% using centrifuge units to reduce the solid handling volumes. This thick dewatered sludge cake will be collected and stored in dewatered sludge storage shed. Notably, the design of the sludge storage shed was calculated assuming the maximum stacking height of 1.5 meters as given in Table 21 below.

Table 21: Dewatering and sludge storage requirements for various developmental phases

Design Flow condition: ADF	Design criteria	Dewatered sludge details and sludge storage capacity requirement
2,300 m <sup>3</sup> /day (Phase-II)	Holding for 240 days	<ul style="list-style-type: none"> <li>Volume of dewatered Sludge @ 18% solids is 1.64 m<sup>3</sup>/day</li> <li>Required storage capacity for 240 days is 394 m<sup>3</sup></li> <li>The required surface area of the Storage shed will be 20m x 13.5m= 270 m<sup>2</sup> x 1.5 m stacking height</li> </ul>
3,333 m <sup>3</sup> /day (Scenario-I)	Holding for 240 days	<ul style="list-style-type: none"> <li>Volume of dewatered Sludge @ 18% solids is 2.36 m<sup>3</sup>/day</li> <li>Required storage capacity for 240 days is 567 m<sup>3</sup></li> </ul>

Design Flow condition: ADF	Design criteria	Dewatered sludge details and sludge storage capacity requirement
		<ul style="list-style-type: none"> <li>Existing surface area post-Phase-II expansion is 272 m<sup>2</sup></li> <li>The required additional surface area of the shed is 108 m<sup>2</sup> x 1.5 m stacking height</li> </ul>
3,395 m <sup>3</sup> /day (Scenario -II)	Holding for 240 days	<ul style="list-style-type: none"> <li>Volume of dewatered Sludge @ 18% solids is 2.4 m<sup>3</sup>/day</li> <li>Required storage capacity for 240 days is 577 m<sup>3</sup></li> <li>Existing surface area post-Phase-II expansion is 270 m<sup>2</sup></li> <li>The required additional surface area of the shed is 115 m<sup>2</sup> x 1.5 m stacking height</li> </ul>

### 1.3.7.5 Disinfection

Tertiary effluent is disinfected by an ultraviolet (UV) disinfection system consisting of two banks of UV lamps in series. Each bank of UV lamps contains 8 modules with 4 lamps per module. The ultraviolet disinfection system has an ECA-rated capacity of 6,500 m<sup>3</sup>/d. This existing UV capacity needs an expansion to PIF in case of the removal of equalization volume as proposed in the class EA. In fact, the UV system still needs an expansion considering MDF, for Phase-II (8,829 m<sup>3</sup>/day), scenario-I (12,665 m<sup>3</sup>/day), and scenario-II (12,889 m<sup>3</sup>/day). Hence, new UVs are needed for future expansions. The final effluent flow is measured by a Parshall flume and then discharged through the outfall to the Conestoga River.

### 1.3.7.6 Effluent conveyance and storage lagoons

Due to the low flow conditions in the Conestoga River between June to September, and the resultant impact on Total Ammonia Nitrogen and unionized ammonia concentrations in the river, no effluent is allowed to be discharged into the Conestoga River. For these months, the treated effluent will be going to be pumped to the storage lagoons having an existing capacity of 340,000 m<sup>3</sup>. Considering the proposed future developments, the existing conveyance piping and pumps must be upgraded. As per the Class EA, 2016 report, the secondary effluent will be pumped and stored in the existing lagoons during the non-discharge period. On the contrary, during the discharge period, this volume of secondary effluent will be conveyed back to an effluent pumping station and then filtered. This approach leads to transferring more TSS and nutrients to storage lagoons and may facilitate eutrophication, especially during the summertime. Apart from this, recycling the stored secondary effluent before the tertiary filterers results in an operation at higher filtration rates which may reduce the capacity, and performance, and require high maintenance. We are proposing to pump the filtered and disinfected effluent to the storage lagoons to overcome these issues. During the discharge period, the same will be directly channeled to the outfall. However, the existing effluent pumping system needs upgradation to meet the required future capacity.

For the phase-II expansion, at ADF of 2,300 m<sup>3</sup>/day, the required storage capacity will be approximately 304,000 m<sup>3</sup> which is less than the existing lagoon capacity. However, for future expansions, the storage requirements will increase to 441,672m<sup>3</sup>, and 449,592m<sup>3</sup> for Scenarios I and II, respectively. This will result in the requirement for additional storage capacity of 101,672 m<sup>3</sup> and 109,592 m<sup>3</sup> for scenarios I and II, respectively.

### 1.3.8 Phasing Plan

It is important to note that the proposed DLW recommendations for the liquid train are based on the inclusion of a dedicated equalization volume. This adjustment allows the downstream units to be designed for maximum day flows rather than peak wet weather flows. Additionally, the sludge management system design in the Class EA is based on a 2% solids consistency, which may result in an underestimation of the required aerobic digestion volume. In this context, DLW has assumed a 1% solids consistency for sludge volume calculations, aligning with the 0.8% to 1% solids content in the waste-activated sludge (WAS) as specified in the MECP guidelines.

Phase-II expansion components based on Class EA recommendations (ADF: 2,300 m <sup>3</sup> /day)	DLW recommended Phase-II modifications (ADF: 2,300 m <sup>3</sup> /day)	Scenario – I (ADF: 3,333 m <sup>3</sup> /d)	Scenario – II (ADF: 3,395 m <sup>3</sup> /d)
<b>Headworks Treatment</b>			
<ul style="list-style-type: none"> <li>• New Headworks building</li> <li>• One (1) Automatic cleaning Screen and One (1) manual cleaning screen each should be designed for the PIF of 480 m<sup>3</sup>/hr (with by-pass provision)</li> <li>• Two (2) vortex-type grit removal systems each designed for the PIF of 480 m<sup>3</sup>/hr</li> </ul>	<ul style="list-style-type: none"> <li>• Two additional (2) Automatic cleaning Screens, each should be designed for the PIF of 785 m<sup>3</sup>/hr (one must have a by-pass provision) (Proposed Phase-II expansion is designed for scenario-II PIF)</li> <li>• Two (2) additional vortex-type grit removal systems each designed for half of Scenario-2 PIF i.e., 393 m<sup>3</sup>/hr</li> </ul>	<ul style="list-style-type: none"> <li>• No expansion is required</li> </ul>	<ul style="list-style-type: none"> <li>• No expansion is required</li> </ul>
<b>Raw Sewage Equalization Tank</b>			
<ul style="list-style-type: none"> <li>• No provision of Equalization volume. In fact, the proposed equalization volume in the phase-I expansion will be converted into a secondary treatment system during the Phase-II expansion</li> </ul>	<ul style="list-style-type: none"> <li>• Dedicated equalization volume is proposed</li> <li>• Two (2) Raw Sewage Equalization Tanks (each with a volume of 662 m<sup>3</sup>)</li> <li>• Eight (8) mixers</li> <li>• Four (4) Aeration Tank Feed Pumps</li> </ul>	<ul style="list-style-type: none"> <li>• One (1) Raw Sewage Equalization Tank with a volume of 574 m<sup>3</sup></li> <li>• Four (4) mixers</li> </ul>	<ul style="list-style-type: none"> <li>• One (1) Raw Sewage Equalization Tank with a volume of 609 m<sup>3</sup></li> <li>• Four (4) mixers</li> </ul>
<b>Aeration Tanks</b>			

Phase-II expansion components based on Class EA recommendations (ADF: 2,300 m <sup>3</sup> /day)	DLW recommended Phase-II modifications (ADF: 2,300 m <sup>3</sup> /day)	Scenario – I (ADF: 3,333 m <sup>3</sup> /d)	Scenario – II (ADF: 3,395 m <sup>3</sup> /d)
<ul style="list-style-type: none"> <li>Part of the Equalization tank volume proposed in the phase-I expansion will be used as an aeration Tank (1,034 m<sup>3</sup>)</li> </ul>	<ul style="list-style-type: none"> <li>No changes are proposed (same as proposed in the class EA)</li> </ul>	<ul style="list-style-type: none"> <li>Two (2) additional aeration tanks (560 m<sup>3</sup>)</li> <li>Two (2) Aeration Grids</li> <li>Two (2) Aeration Blowers</li> </ul>	<ul style="list-style-type: none"> <li>Two (2) additional aeration tanks (560 m<sup>3</sup>)</li> <li>Two (2) Aeration Grids</li> <li>Two (2) Aeration Blowers</li> </ul>
<b>Secondary Clarifier</b>			
<ul style="list-style-type: none"> <li>Part of the Equalization volume will be used as secondary clarifiers (271 m<sup>2</sup>) resulting in a total clarifier surface area of 414 m<sup>2</sup> (As per the Class EA proposal for phase-II expansion). Capacity is sufficient for the PWWFs</li> </ul>	<ul style="list-style-type: none"> <li>No changes are proposed</li> </ul>	<ul style="list-style-type: none"> <li>No expansion is required because equalization volume will dampen the peak flows, resulting in a design of the downstream units at MDF instead of PWWF. Hence, capacity is sufficient for the MDF.</li> </ul>	<ul style="list-style-type: none"> <li>No expansion is required because equalization volume will dampen the peak flows, resulting in a design of the downstream units at MDF instead of PWWF. Hence, capacity is sufficient for the MDF.</li> </ul>
<b>Tertiary Filter building and Filters</b>			
<ul style="list-style-type: none"> <li>Tertiary filter building</li> <li>Add four (4) filters each having an area of 4.65 m<sup>2</sup></li> </ul>	<ul style="list-style-type: none"> <li>Only one (1) filter is required with an area of 4.65 m<sup>2</sup> (not the four filters proposed in the Class EA) because equalization volume requires filters to operate at maximum day flows</li> </ul>	<ul style="list-style-type: none"> <li>Add three (3) Filters having an area of 4.65 m<sup>2</sup></li> </ul>	<ul style="list-style-type: none"> <li>Add three (3) filters each having an area of 4.65 m<sup>2</sup></li> </ul>
<b>Ultraviolet Disinfection System</b>			
<ul style="list-style-type: none"> <li>No UV Expansion required</li> </ul>	<ul style="list-style-type: none"> <li>No changes are proposed</li> </ul>	<ul style="list-style-type: none"> <li>No expansion is required</li> </ul>	<ul style="list-style-type: none"> <li>No expansion is required</li> </ul>
<b>Aerobic digestion and sludge dewatering</b>			
<ul style="list-style-type: none"> <li>Required Total aerobic digestion volume is 598 m<sup>3</sup> based on the WAS % of 2%. (WAS at 2% is a very conservative number it may lead to under designing of the aerobic digestion system)</li> <li>Existing aerobic digestion volume is 468 m<sup>3</sup></li> <li>Conversion of 2 of the existing 4 sludge storage</li> </ul>	<ul style="list-style-type: none"> <li>Required Total aerobic digestion volume is 1,195 m<sup>3</sup> (as opposed to 598 m<sup>3</sup>) based on the WAS of 1% from the hopper.</li> <li>Conversion of all 4 of the existing sludge storage tanks (4 tanks x 150m<sup>3</sup> each) into aerobic digestors</li> </ul>	<ul style="list-style-type: none"> <li>Required Total aerobic digestion volume is 1,639 m<sup>3</sup></li> <li>Existing aerobic digestion volume post-Phase-II expansion is 1,195m<sup>3</sup></li> <li>Hence, need an additional volume of 444 m<sup>3</sup></li> <li>Add one (1) Aeration grid</li> </ul>	<ul style="list-style-type: none"> <li>Required total aerobic digestion volume is 1,747 m<sup>3</sup></li> <li>Existing aerobic digestion volume post-Phase-II expansion is 1,195m<sup>3</sup></li> <li>Hence, need an additional volume of 552 m<sup>3</sup></li> <li>Add one (1) Aeration grid</li> </ul>

<b>Phase-II expansion components based on Class EA recommendations (ADF: 2,300 m<sup>3</sup>/day)</b>	<b>DLW recommended Phase-II modifications (ADF: 2,300 m<sup>3</sup>/day)</b>	<b>Scenario – I (ADF: 3,333 m<sup>3</sup>/d)</b>	<b>Scenario – II (ADF: 3,395 m<sup>3</sup>/d)</b>
<p>tanks (2 tanks x 150m<sup>3</sup> each) into aerobic digestors. Thus, the total aerobic digestion volume will be 768 m<sup>3</sup>. Sufficient for phase-II expansion.</p> <ul style="list-style-type: none"> <li>• Add Two (2) Aeration grid</li> <li>• Add two (2) blowers</li> <li>• Dewatering geotubes 3 units (13.7 m circumference x 17.4m length)</li> </ul>	<ul style="list-style-type: none"> <li>• Additional aerobic digestion volume of 127 m<sup>3</sup> (1,195m<sup>3</sup>-468m<sup>3</sup>-600m<sup>3</sup>) is required</li> <li>• Add three (3) New Digester feed pumps</li> <li>• Add five (5) Aeration grids</li> <li>• Add Four (4) blowers</li> <li>• Add a Sludge dewatering building</li> <li>• Add two (2) centrifuge feeding pumps</li> <li>• Add two (2) centrifuge dewatering units</li> <li>• Add one (1) Filtrate Tank</li> <li>• Add two (2) Filtrate Transfer Pumps</li> </ul>	<ul style="list-style-type: none"> <li>• Add one (1) blower</li> </ul>	<ul style="list-style-type: none"> <li>• Add one (1) blower</li> </ul>
<b>Dewatered sludge storage</b>			
<ul style="list-style-type: none"> <li>• Provide a sludge storage volume of 420 m<sup>3</sup> based on 18% solids and 240 days of storage</li> </ul>	<ul style="list-style-type: none"> <li>• Provide a sludge storage volume of 405 m<sup>3</sup> or a yard having an area of 270 m<sup>2</sup> with 240 days of storage capacity for dewatered sludge assuming a stacking height of 1.5m</li> </ul>	<ul style="list-style-type: none"> <li>• Additional storage volume of 162 m<sup>3</sup> requiring an area of 108 m<sup>2</sup></li> </ul>	<ul style="list-style-type: none"> <li>• Additional storage volume of 173 m<sup>3</sup> requiring an area required will be 115 m<sup>2</sup></li> </ul>

## **PFDs: WWTP expansion**

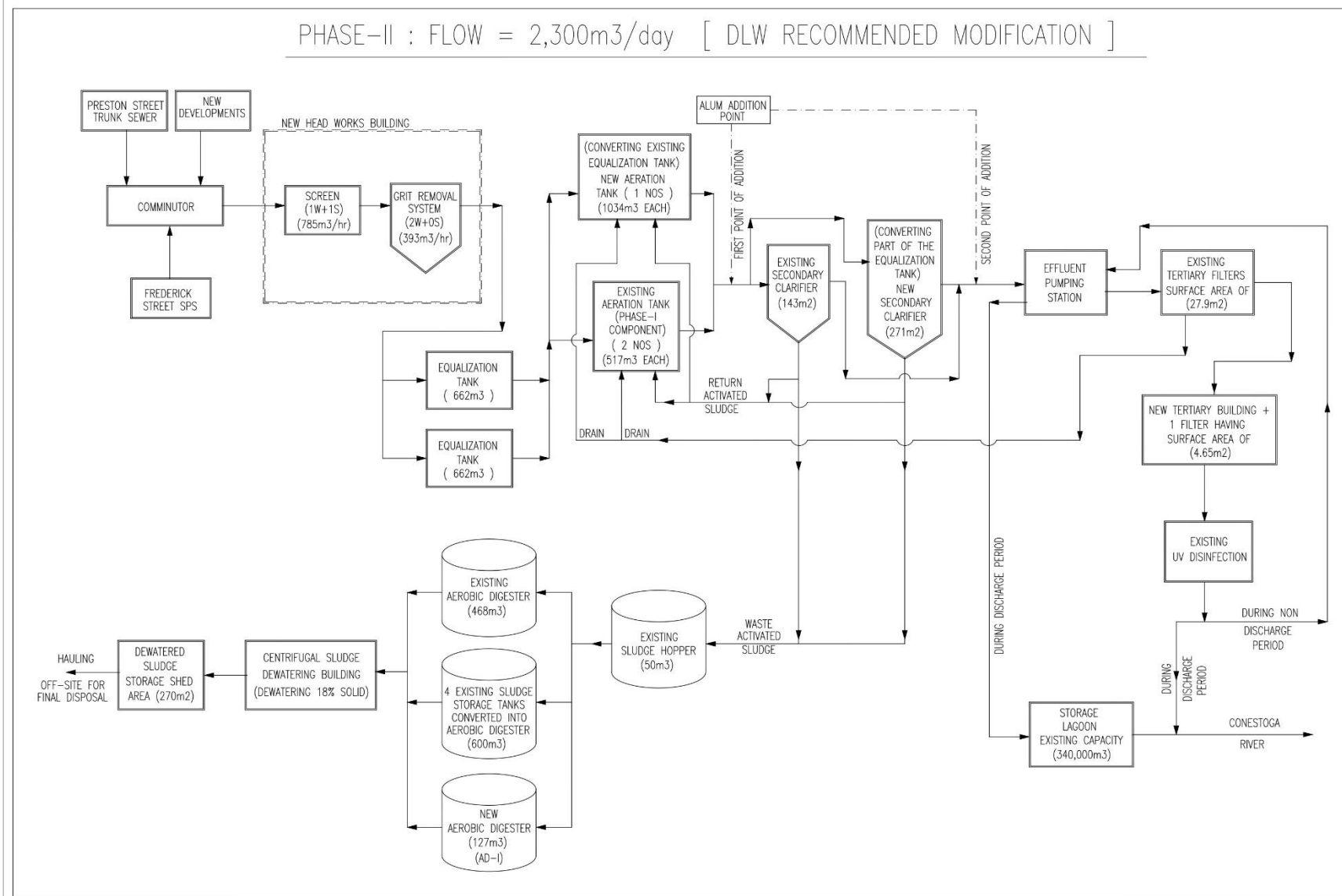


Figure 10: Process Flow Schematic for Phase-II Expansion Based on the Proposed DLW’s Recommendations



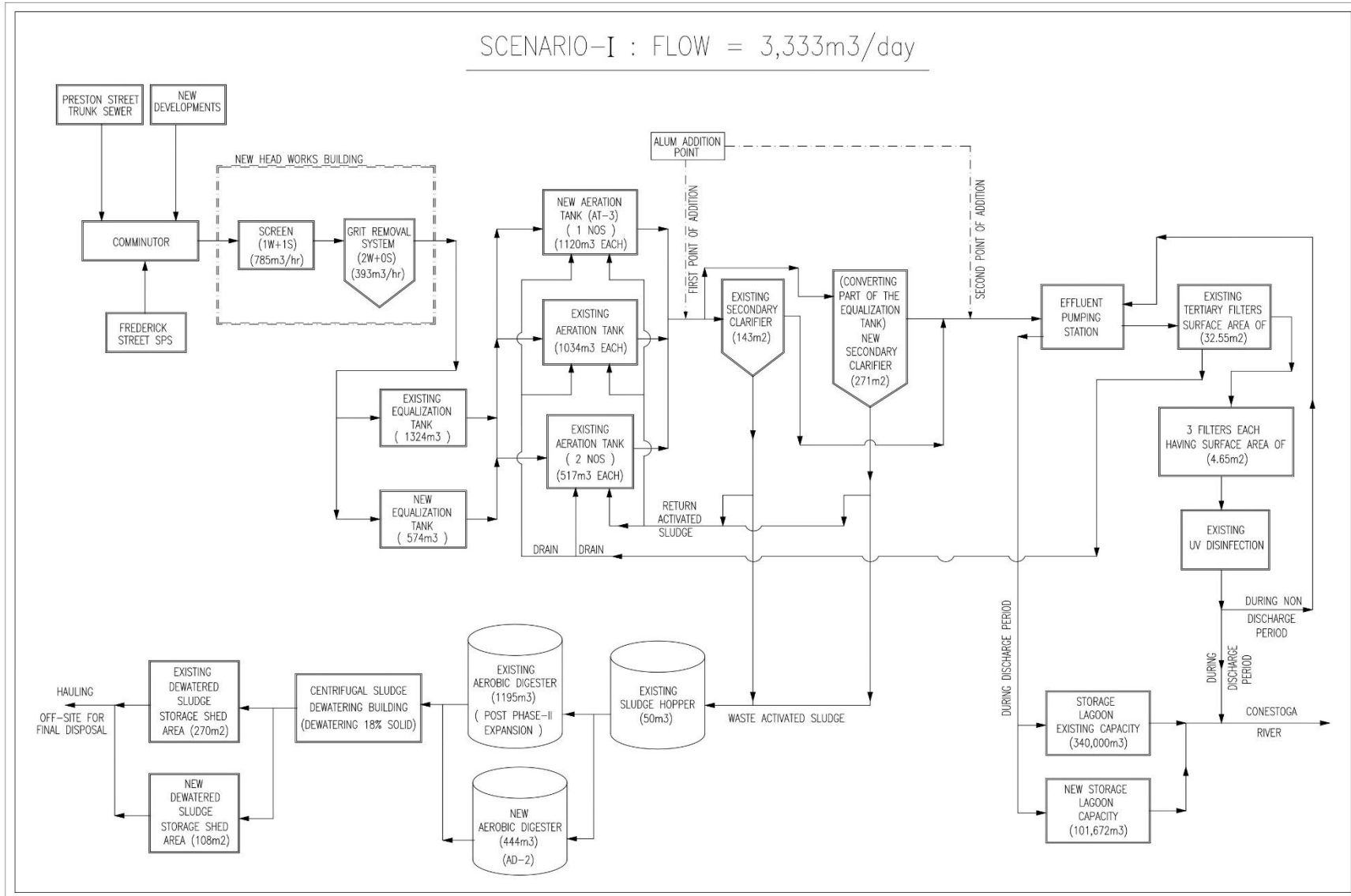


Figure 12: Process Flow Schematic for Scenario-I Expansion

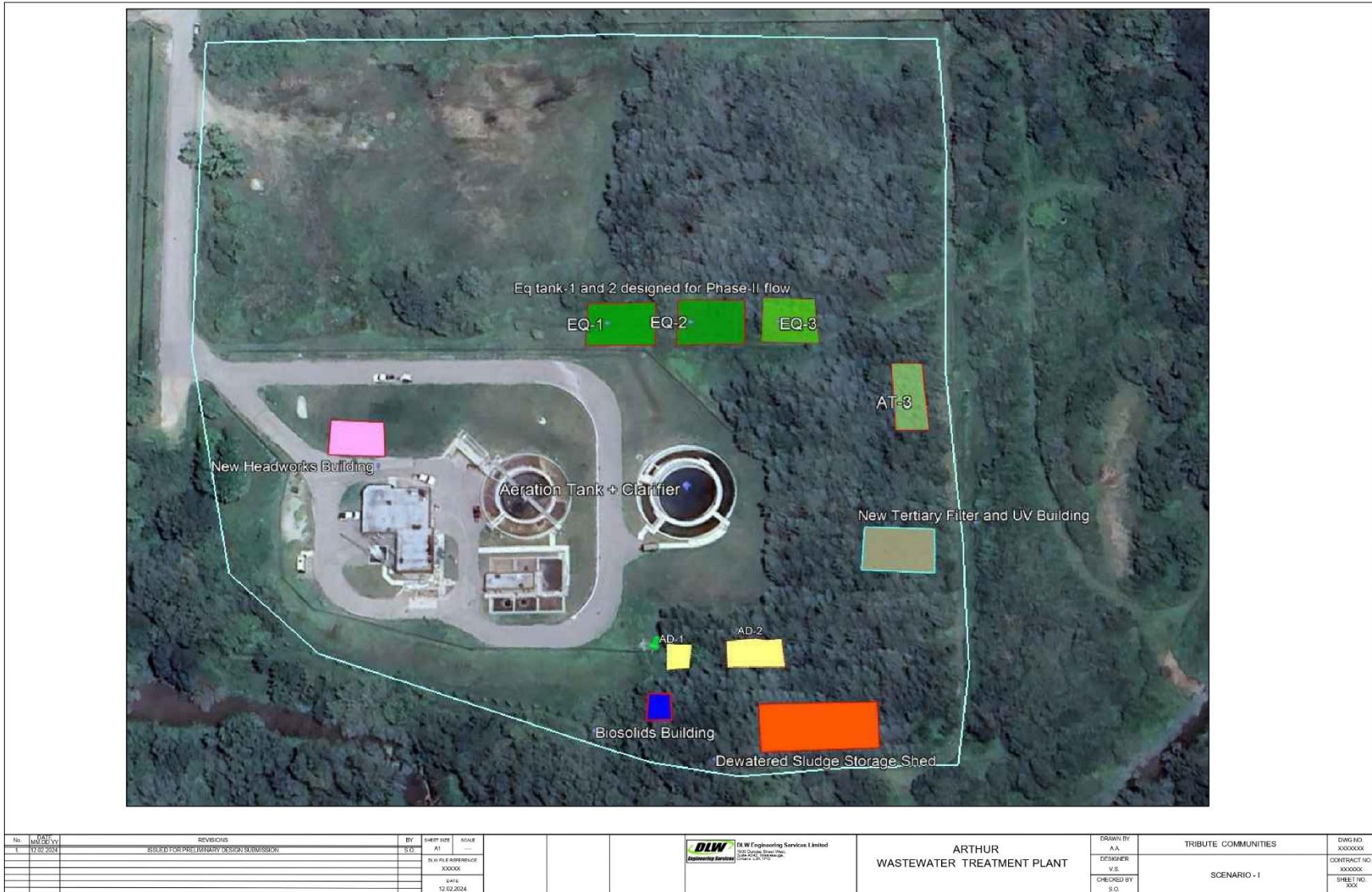


Figure 13: Google Image based on the Scenario-I Expansion

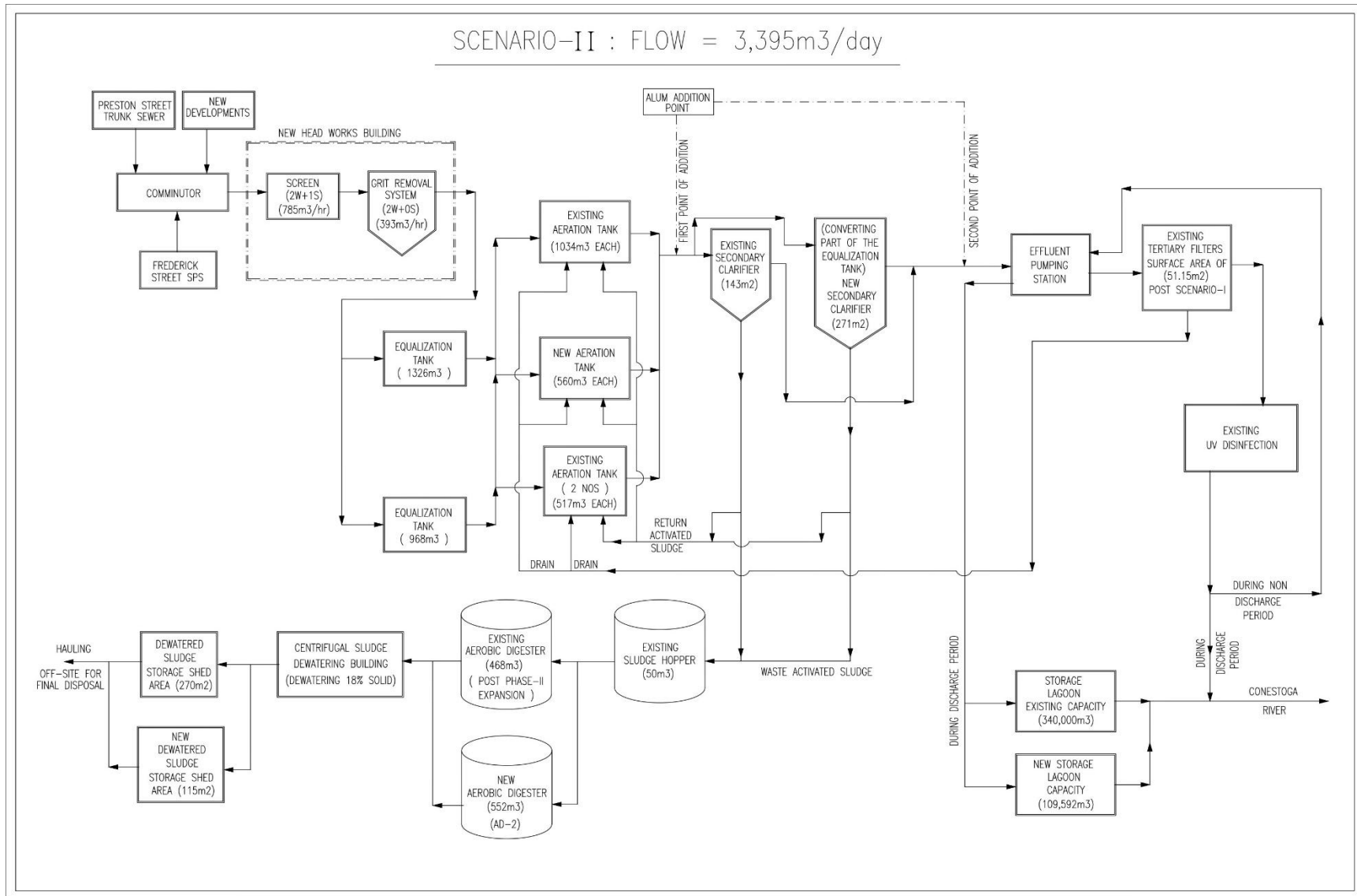


Figure 14: Process Flow Schematic for Scenario-II Expansion



## 1.4 CONCLUSIONS AND RECOMMENDATIONS

Based on the foregoing we are confident that there are viable water and wastewater servicing options for the Tribute Arthur development in the Township of Wellington North based on the expansion of the well water supply, treatment and storage system, and expansion of the existing Wastewater Treatment Plant.

### 1.4.1 Water Supply System

The existing Arthur water supply is based on 3 groundwater-based wells with a firm capacity of between 2,255 m<sup>3</sup>/day and 4,216 m<sup>3</sup>/day and an uncommitted reserve capacity of 480 - 489 residential units. The wells require treatment for iron and manganese and disinfection. The low end of supply capacity assumes that the smallest well (Well 7B) is out of service and only one of the 2 remaining wells on the Well 8A/8B site can operate. This is an extremely conservative approach to determining firm capacity. Based on the November 2024 Water System Supply Redundancy and Storage Public Information Centre Boards, with the addition of the proposed well there will be sufficient well and treatment capacity to service all of the proposed Tribute Arthur development. The Class EA also states that the proposed 900 m<sup>3</sup> elevated water storage tank is required to mitigate the small current storage deficit as well as to accommodate development to 2051. The Class EA identified the new well and storage facility to be located on the Tribute Arthur lands. However, this storage is insufficient to accommodate all of the Tribute Arthur development. The proposed elevated water storage tank should be increased to at least 1,800 m<sup>3</sup> or an additional reservoir will be required in the Tribute Arthur lands.

We have assumed the MECP formulation for the development of system storage and used the Triton April 2022 recommendation for maximum day flows based on 500 Lpcd. The storage for the Tribute development will be based on whether the storage is based on servicing strictly the Tribute Arthur development or on the impact to the full Arthur development will be as follows:

- For 823 development units, storage will be between 1,232 to 1,359 m<sup>3</sup>, and
- For 872 development units 1,271 to 1,406 m<sup>3</sup>.

The reservoir configuration at the Tribute Arthur development could be either of the following:

- Inground Concrete Reservoir with associated Booster Pumping Station, all located on the Tribute Arthur development site with jockey, duty, and fire pumps.
- Elevated Storage Tank located on the Tribute Arthur development site fed directly from the wells.

Based on the Class EA and the preliminary concept plan, the proposed well, pumphouse, and elevated water storage tank will be located on proposed Block 67 at the intersection of Wells Street and Macauley Street.

## 1.4.2 Wastewater System

The existing capacity of the Arthur wastewater treatment plant after the Phase-I expansion (as identified in the APR) is 1,860 m<sup>3</sup>/day with an uncommitted reserve capacity of 303 m<sup>3</sup>/day (Refer to Table 7). Under the phase-II expansion (also as identified in the Class EA), it is proposed to expand the plant to the capacity of 2,300 m<sup>3</sup>/day to cover the demand for the year 2031. The recently planned Tribute/Sorbara Arthur Holdings Inc. development will be based on 823 or 872 additional residential lots. This proposed development will require a new sewage pumping station located on the Tribute Arthur lands. The pumping station will be located on a 0.05 ha parcel of land on Block 70 at the intersection of Street R and Eliza Street. The pumping station will pump through a forcemain to the sanitary collection system at the intersection of Street A and Eliza Street. The proposed Tribute Arthur development will further need WWTP expansion to cater to future flows. This report discusses the possible solutions to integrate the proposed Phase-II WWTP expansion with the planned new developments that are being planned by Tribute/Sorbara Arthur Holdings Inc.. The key recommendations and conclusions are as follows:

1. The proposed headworks capacity of 12,887 m<sup>3</sup>/day for the phase-II PIF is not sufficient for the anticipated Tribute developments. Hence, either capacity increase or provision for additional units must be arranged for future Tribute developments (Scenario I and II).
2. For the Phase-II expansion, it is proposed to convert the equalization tank (constructed in Phase-I expansion) into a secondary treatment system (aeration tank + clarifier). This conversion will eliminate the WWTP's capacity to shave the future peak flow of 12,887 m<sup>3</sup>/day for phase-II expansion (Table 3). Consequently, it will result in the downstream process units being undersized and will work at rates that far exceed the MECF design guidelines for the aeration tankage, clarifiers, tertiary filters, and UV disinfection system. Therefore, it is imperative to provide sufficiently sized (>12 hours at ADF) equalization tankage in Phase-II to mitigate the potential for having to significantly upsize the secondary and tertiary treatment systems at substantial additional cost to the project. The equalization tankage would eliminate the carryover of the excess MLSS to the clarifiers at peak events, and the need for a large surface area for clarification, filtration, and ultraviolet disinfection units. The existing storage lagoons are sufficiently sized for the Phase-II expansion, but the effluent pumps will need to be upsized.
3. The design concentrations for the recently completed Phase-I (1,860 m<sup>3</sup>/day) and proposed Phase-II (2,300 m<sup>3</sup>/day) expansions were based on the typical organic loading specified in the MECF guidelines (Figure 5 and Table 11). However, the influent concentrations given in the annual performance reports indicate that the incoming BOD<sub>5</sub> concentrations are 35 to 40% higher. Hence, it is important to revise the proposed Phase-II designs accordingly and the same should be adopted for future expansions.
4. The total phosphorus (TP) and total ammonia nitrogen concentrations of 0.34 mg/l and 2.80 mg/l in the effluent (year 2019-2020) were higher than the approved concentration of 0.25 mg/l and 8.12 listed in the Class EA report (Table 6). Notably, the addition of the equalization tank, increased blower capacity, and high alum dosing in the Phase-I expansion resulted in an improved effluent quality (refer to 2021-22 APR) meeting the

ECA limits. However, converting the equalization tanks into a secondary treatment system may deteriorate the effluent quality.

5. Based on the available historical data, the WAS rate to the digester was 60.4 m<sup>3</sup>/day, which seems very high. This is possibly due to the low WAS consistency of less than 1% compared to the 2% assumed in the class EA and/or higher influent BOD<sub>5</sub> concentration compared to the design value. Even the maximum monthly sludge generation calculated in the class EA was only 27.4 m<sup>3</sup>/day which is still lower than the quantity observed in the historical data. Hence, it is important to evaluate the sludge treatment system and storage capacity considering the actual WAS consistency. The WAS range as per the MECP is 0.8 to 1.0% for extended aeration processes but class EA is considered 2%, assuming substantial thickening in the sludge hopper. This assumed WAS consistency of 2% may result in an under sizing of the sludge treatment system and storage units.
6. The requirement for holding the effluent in storage lagoons is based on the Total Ammonia Nitrogen concentrations in the effluent and the typically low flows in the Conestoga River from June to September. It may be possible to convince the MECP that with an enhanced level of treatment, the flow restriction to the Conestoga River may be eliminated. However, there is some risk associated with this approach and we have assumed that the storage and flow restriction to the river will continue for all future expansion phases. Thus, for all future flow conditions additional storage capacity is required for the non-discharge period (June to September).