Comox Valley
REGIONAL DISTRICT

Agenda

Notice of Meeting # 9 of the

LIQUID WASTE MANAGEMENT PLAN
JOINT TECHNICAL AND PUBLIC ADVISORY COMMITTEES (TACPAC)

Wednesday, March 4, 2020
Comox Valley Curling Centre, 4835 Headquarters Road, Courtenay
9:00 am - 3:00 pm

ITEM, TIME	DESCRIPTION	OWNER
9.1 9:00	Call to Order	Allison Habkirk
9.2 9:00-9:10	Review of Minutes of Meeting #8	Allison Habkirk
9.3 9:10-9:50	Update on Conveyance Shortlist • Discussion on timing of conveyance decisions and updates to shortlist	Kris La Rose, CVRD
	Break	
9.4 10:00-10:45	Short List Options - Treatment - Refresher • Refresher on level of treatment options and technical attributes	WSP
9.5 10:45-12:00	Evaluating Short List Options - Treatment • Review of evaluation system and previous evaluation • Discussion	Paul Nash & Allison Habkirk
	Lunch Break 12:00 - 12:30	
9.6 12:30-1:30	Evaluating Short List Options - Treatment - continued • Finalize preferred level of treatment decision ***Make a recommendation to the Comox Valley Sewage Commission on the preferred level of treatment for the Comox Valley Water Pollution Control Center***	Paul Nash & Allison Habkirk
1:30 -1:45	Reclaimed Water – Results of Ideas session at TACPAC 5, Feb 2019	Paul Nash
9.7 1:45-2:30	Resource Recovery Options • Review of resource recovery options	WSP
9.8 2:30-3:00	Evaluation of Resource Recovery Options • Finalize approach for incorporating resource recovery options and/or language in the LWMP • ***Make a recommendation to the Comox Valley Sewage Commission in regards to resource recovery for the Comox Valley Water Pollution Control Center***	Paul Nash & Allison Habkirk
9.9 3:00	Adjournment	Allison Habkirk

Attachments:

1. Minutes of TAC Meeting #8, December 5, 2019
2. Stage 2 Wastewater Treatment Assessment, WSP
3. Stage 2 Resource Recovery Options, WSP

Comox Valley
REGIONAL DISTRICT

Minutes

Minutes of the meeting of the Liquid Waste Management Plan (LWMP) Joint Technical and Public Advisory Committees (TACPAC) Meeting #8 held on Thursday, December 5, 2019 at the Comox Valley Regional District (CVRD) Boardroom, commencing at 9:00 am.

PRESENT:	A. Habkirk, Chair and Facilitator	
	P. Nash, LWMP Project Coordinator	
	M. Rutten, General Manager Engineering Services	CVRD
	K. La Rose, Senior Manager of Water/Wastewater	CVRD
	M. Imrie, Manager of Wastewater Services	CVRD
	J. Boguski, Branch Assistant – Engineering Services	CVRD
	Z. Berkey, Engineering Analyst	CVRD
	J. Morin	
	A. Bennett	WSP
	A. Gibb	WSP
	M. Swift, Town of Comox Councillor	PAC
	W. Cole-Hamilton, City of Courtenay Councillor	PAC
	A. Hamir, Lazo North – Electoral Area B Director	PAC
	C. McColl, K'ómoks First Nation	PAC/TAC
	A. Gower, Comox Valley Chamber of Commerce	PAC
	T. Ennis, CV Conservation Partnership Alternate	PAC
	S. Carey, Courtenay Resident Representative	PAC
	K. Niemi, Courtenay Resident Representative	PAC
	K. van Velzen, Comox Resident Representative	PAC
	D. Jacquest, Comox Resident Representative	PAC
	R. Craig, Comox Resident Representative	PAC
	A. Munro, BC Shellfish Growers Association	PAC
	J. Steel, Area B Resident Representative	PAC
	L. Aitken, Area B Representative Alternate (observer)	PAC
	M. Lang, Area B Resident Representative	PAC
	C. Davidson, City of Courtenay Engineering (alternate)	TAC
	S. Ashfield, Town of Comox Engineering	TAC

ITEM	DESCRIPTION	OWNER
8.1	**Call to Order** Meeting called to order at 9:00am	Allison Habkirk
8.2	**Review of Minutes of Meeting #7** MOTION: To adopt minutes of meeting #7 – R. Craig SECONDED – M. Swift CARRIED	
8.3	**K'ómoks First Nation Archaeology Presentation** Jesse Morin presented traditional territories of the Salish people, their history and the geographic regions of the different first nation languages.	Jesse Morin
8.4	**Break 10:00 – 10:20**	

ITEM	DESCRIPTION	OWNER
8.5	**LWMP Decision Making Process Presentation** Overview and clarification of the TAC/PAC's role in the LWMP process as referred to in the Terms of Reference. Will this group make only one recommendation? - The TAC/PAC will provide at least three recommendations, possibly more. One recommendation for each aspect, being conveyance, treatment and resource recovery. If the TAC/PAC only have one recommendation, can the Sewage Commission (Steering Committee) and CVRD Board say no? - The Sewage Commission makes the final decision, as referenced in the LWMP decision structure presentation and Terms of Reference. The Commission is provided with background information to help support recommendation decisions.	Allison Habkirk
8.6	**Treatment Level Assessments Presentation- Technical** WSP provided overview of levels of treatment assessment. The upgrades at the wastewater treatment plant will be triggered on flows not on year prescribed based on population projections and will likely be staged. The intent of the TACPAC is to determine the level of treatment at the plant, the actual scheduling of upgrades at the plant will be determined through the master planning process. The population estimates used for this analysis come from the 2016 ISL report. A review of population projections will be completed incorporating the following feedback from the TACPAC: • Table 1 in the report shows zero per cent growth for CFB Comox. That will need to be adjusted, Shelly Ashfield can provide those projection numbers. • A review of higher density projections from the Town of Comox that have been completed as part of additional study work, will be reviewed and considered within these population projections. • Universal water metering will likely effect sewage flows, resulting in changes to the staging of future upgrades. Is treatment of odour part of the LWMP? - Sewage plant odour was ranked high in the early evaluation process. - Odour treatment studies are happening in parallel to this process and will be brought forward to the Sewage Commission in early 2020. Odour control upgrades are not a differentiator between the levels of treatment options presented to the TACPAC. Union Bay growth projections and current applications for their effluent discharge into Hart Creek is very concerning to the community. - In parallel to the LWMP, the CVRD are looking at governance implications to convey and treat Electoral Area A wastewater. Why are the site plans presented by WSP quite different from the 2016 ISL report? - Upgrades to the plant can be configured a number of different ways. The site layout will be developed as part of the comprehensive	WSP

ITEM	DESCRIPTION	OWNER
8.6	master plan for the CVWPCC. The presentation is just a comparative cost estimate to determine treatment level. What happens to the biological material captured by the disc filter? How is the final effluent improved? - 95 per cent BOD/TSS is removed. That material goes to the solids removal process already in place and carries on to Skyrocket production. For Option 3, when would we bypass the advanced treatment? • Advanced treatment will only be bypassed during high flow events, likely one to two per cent of total flow through the plant would be bypassed annually. What is the lifespan of the disc filters? • A cloth media filter is replaced once in a while, it is a far lower cost option than membranes. What is the implication to the aquifer from reclaimed water use? • An environmental impact study would be required prior to implementation of reclaimed water use for irrigation. At this time, reclaimed water won't be used for irrigation, it would be used in the sewage treatment processing. Why is treatment focused on BOD and TSS? • BOD and TSS cause changes to the receiving environment and can cause oxygen deficiency in water and impact higher forms of life. Why the range in the removal of micro plastics for the different options? • Relatively new field, still understanding the impact of varying levels of treatment on micro plastics, it is largely based on the performance of the plant. The CVWPCC currently is far below its discharge limit for BOD and TSS and is treating wastewater to the same limits as presented within Option 3 and 4. If Option 2 is selected, it does not preclude the option to add tertiary treatment in the future, if regulations/needs change in future years. Consideration in the site layout as part of the master plan process must be done accordingly to allow such flexibility in the future.	WSP
8.9	**Lunch 12:00 – 12:35**	
8.10	**Treatment Level Assessments Discussion - Financial** Significant discussion occurred on the current effluent quality of the plant and the economic and social benefits of addition of disc filter at the plant if the plant currently outputs quality that would be achieved by a filter.	WSP/CVRD

ITEM	DESCRIPTION	OWNER
8.10	For the cost per connection impact analysis, were senior government grants taken into consideration on the assumptions? - No the analysis does not take into account grant funding, and presents the worst case scenario for users. Surprised that the cost per connection numbers are so low given the costs provided. - The Comox Valley Water Treatment Project went through a similar process, the LWMP process is consistent with that. The meeting discussions did not allow time for the TAC/PAC to make a recommendation. It is suggested to either extend today's meeting by 20 minutes or forward this discussion and decision to a new meeting. MOTION: To adjourn Meeting #8 and have a new meeting in late January to complete decision on levels of treatment. CARRIED The next LWMP meeting will combine further discussion and a recommendation for treatment with resource recovery discussion/recommendation.	WSP/CVRD
8.11	**Meeting Adjourned 3:05pm**	

MEMO

TO: CVRD LWMP TACPAC Committee

CC: Kris La Rose, P.Eng., CVRD, Zoe Berkey, EIT, CVRD, Paul Nash

FROM: Tyler Barber, MASc, P.Eng., Aline Bennett, MASc, P.Eng., Al Gibb, PhD, P.Eng.

SUBJECT: **CVRD LWMP – Stage 2 Wastewater Treatment Level Assessments**

DATE: **February 12, 2020**

INTRODUCTION

The Comox Valley Regional District (CVRD) has retained WSP Canada Group Ltd. to complete the Liquid Waste Management Plan (LWMP) for the District. As part of the work, WSP has completed the Stage 2 wastewater assessment for the Comox Valley Water Pollution Control Centre (CVWPCC). This work is a high-level review of the estimated capacity of the existing infrastructure at the CVWPCC, what is required for expansion to handle 2040 flows and loads into the CVWPCC and costing different level of wastewater treatment options for the CVWPCC.

This memo provides the following information:

- Updated CVWPCC population, flow and load projections;
- High-level review of the capacity of each unit process (attached in the Appendix);
- Cost estimates for upgrading the plant to meet 2040 capacity requirements and providing different levels of wastewater treatment including:
  - Option 1: was not advanced from the long-list
  - Option 2: Secondary treatment for entire flow with disinfection (base case)
  - Option 3: Addition of advanced treatment for 2xADWF
  - Option 4: Addition of advanced treatment for the entire flow
  - Option 5: Addition of reclaimed water for in plant use, which can be common to all options

Note that Option 1: was not advanced from the long-list, since this would represent a step back from current practice in terms of effluent quality.

The objective of this assessment is to enable decision making on the appropriate level of wastewater treatment to provide at the CVWPCC by comparing the costs and benefits of the different options. The *CVWPCC Capacity Assessment* completed by ISL Engineering and Land Services in 2016, was a significant input to this assessment.

REGULATORY REQUIREMENTS AND EFFLUENT QUALITY

The Wastewater Systems Effluent Regulations (WSER) is the only federal regulation that exists to control domestic wastewater discharges nationwide. The WSER is established under the Fisheries Act and includes mandatory minimum effluent quality standards that must be achieved through secondary wastewater treatment. The WSER applies to wastewater treatment systems that treat more than 100 m^3 of wastewater per day. The regulated compounds are total suspended solids (TSS), carbonaceous biochemical oxygen demand ($cBOD_5$), total residual chlorine, and un-ionized ammonia. In the case of the CVWPCC, the characteristics of the effluent must be equivalent to or better than an average monthly $cBOD_5$ and TSS concentrations of 25 mg/L.

The Provincial Municipal Wastewater Regulation (MWR) regulates wastewater discharges to waters in BC. Under the MWR, compounds such as pH, $cBOD_5$, TSS, and in some cases total phosphorus and ortho-phosphate are monitored, and their release to the receiving environment is controlled. The MWR requires that the CVWPCC effluent maximum day concentration of a $cBOD_5$ and TSS not exceed 45 mg/L.

The CVWPCC discharge is not currently registered under the MWR. Authorization of the discharge is grandfathered under Permit No. 5856. Under this Permit, the CVRD is required to meet the discharge criteria for a maximum daily discharge rate (18,500 m^3/d), maximum day BOD_5 (45 mg/L) and maximum day TSS (60 mg/L).

The CVWPCC effluent quality data was reviewed and analyzed for the period from 2014 to 2019. The effluent was sampled and analyzed for $cBOD_5$ and TSS at least once a month as required by the discharge permit ($cBOD_5$ and BOD_5 were both measured every 2 weeks). It should be noted that $cBOD_5$ analyses started in October 2014; prior to that, total BOD measurements were used.

The plant effluent concentration of TSS from 2014 to 2019 is shown in Figure 1 (monthly average concentration) and Figure 2 (daily concentration). The monthly average TSS concentration exceed the WSER criteria of 25 mg/L once in 2017 (Figure 1). As shown in Figure 2, the effluent daily TSS concentration was below the allowable maximum specified in both Permit No. 5856 (60 mg/L) and the MWR (45 mg/L). Study of Figure 2 shows that the monthly average effluent TSS concentration was typically in the range of 5 mg/L to 15 mg/L from 2014 to the present.

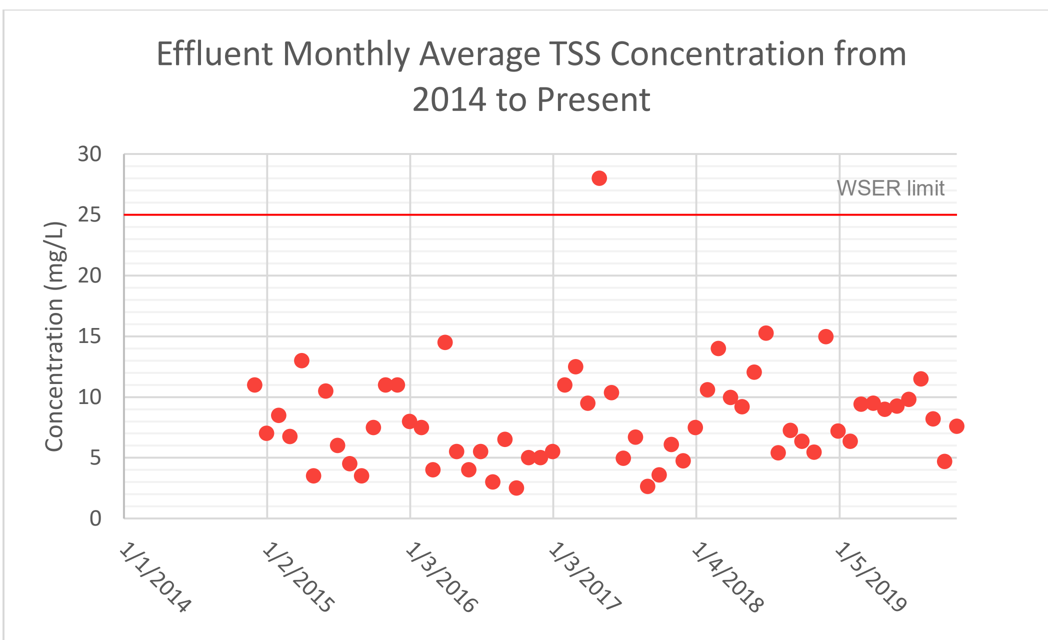

Figure 1 Effluent Monthly Average TSS Concentration (2014-2019)

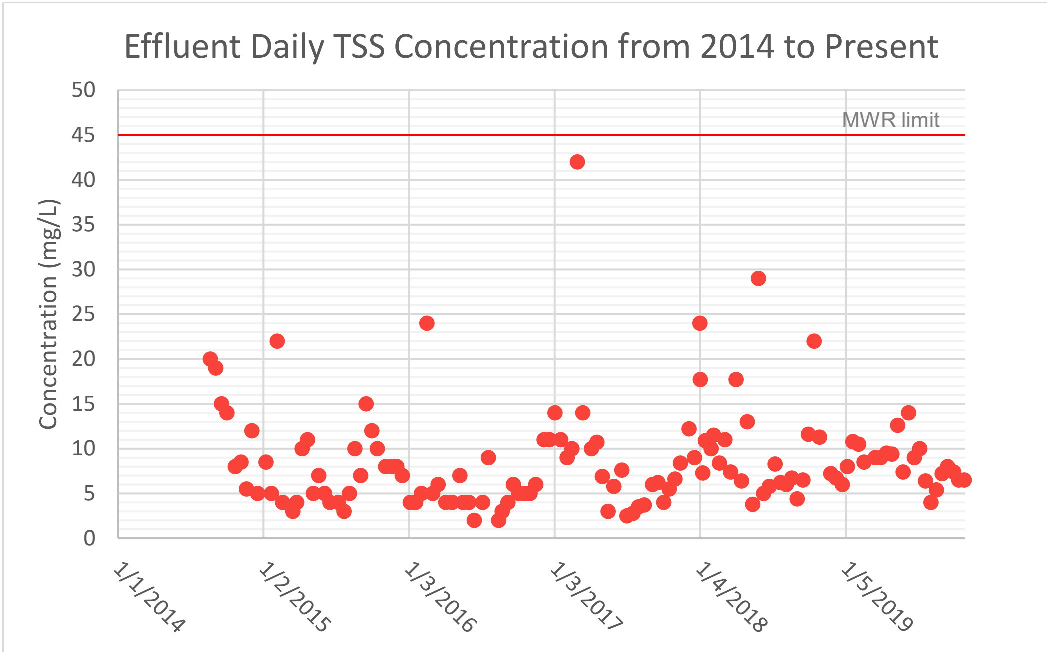

Figure 2 Effluent Daily TSS Concentration (2014-2019)

The plant effluent quality for $cBOD_5$ is shown in Figure 3 (monthly average) and Figure 4 (daily). All of the values are within the regulatory limits specified in the WSER, the MWR and Permit No. 5856. Similar to the data for TSS, the monthly average $cBOD_5$ concentration was typically in the range of 5 mg/L to 15 mg/L.

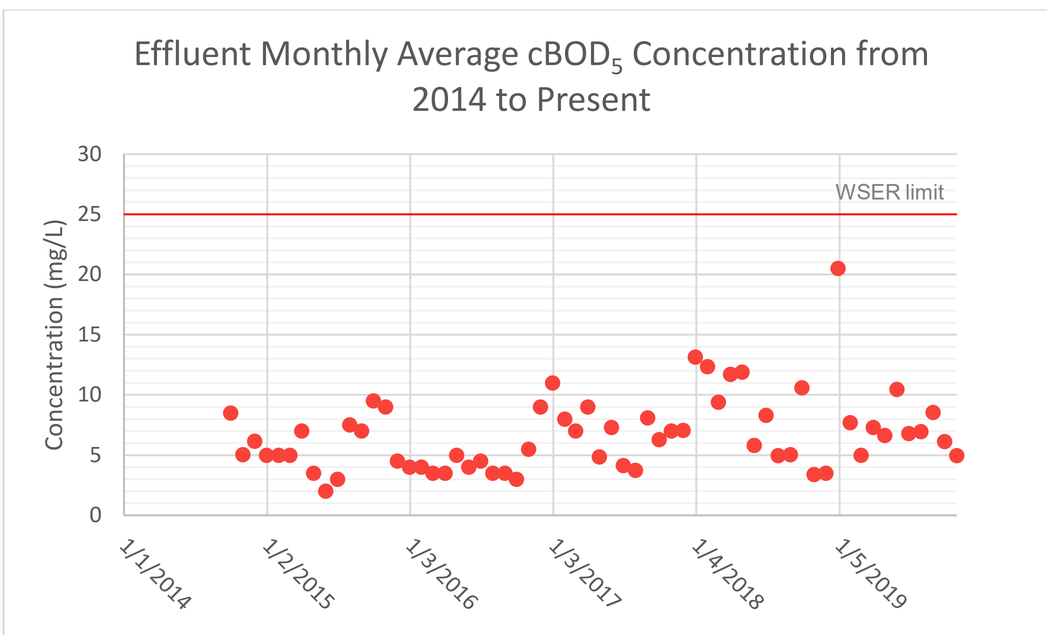

Figure 3 Effluent Monthly Average $cBOD_5$ Concentration (2014-2019)

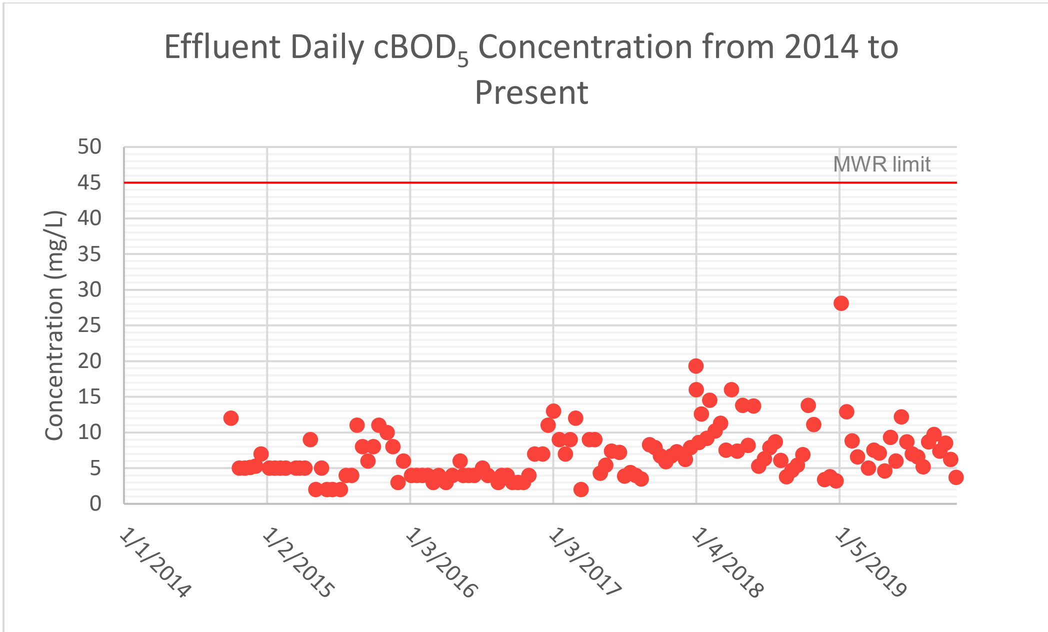

Figure 4 Effluent Daily cBOD5 Concentration (2014-2019)

A statistical analysis of effluent quality data is shown in Figure 5. The log-normal distribution of effluent $cBOD_5$ and TSS concentration data was used, where a particular sample value is a function of sample size and the rank of the particular sample. The sample values are ranked from smallest to largest and the corresponding plotting position is determined using the following formula:

$$Plotting\ Position, \% = \left(\frac{m}{n+1}\right) \times 100$$

where, m is the rank serial number and n is the number of observations. As shown in Figure 5, the effluent concentration of $cBOD_5$ was 14 mg/L or less 95% of the time, and the TSS concentration was 20 mg/L or less 95% of the time over a period of record. The steep rise in the curves beyond 95% show that a small number of data points (5%) significantly exceeded these values.

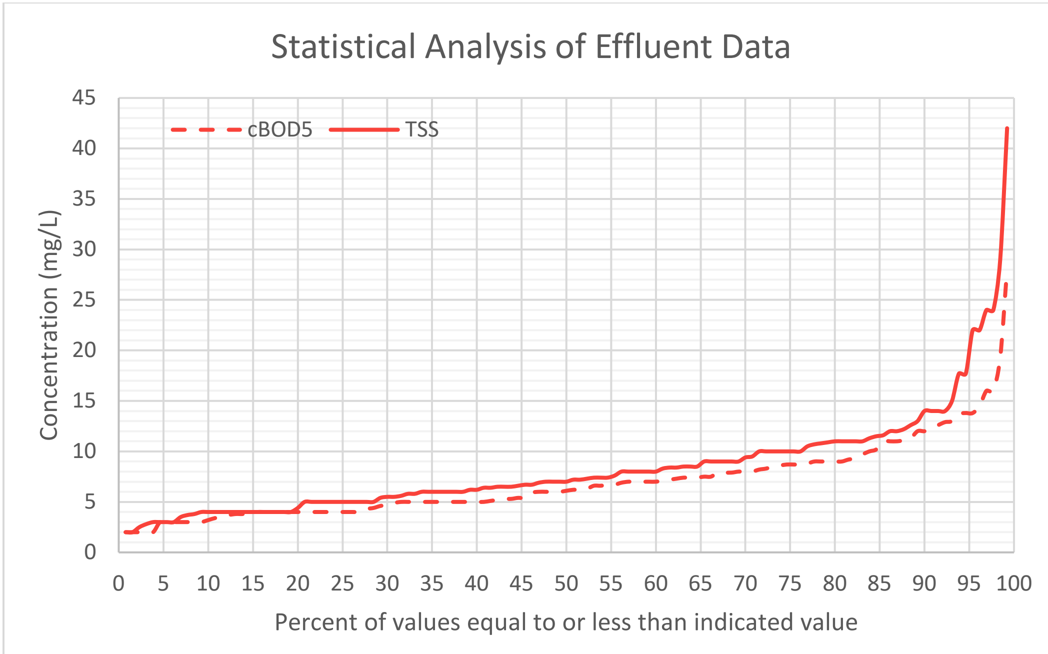

Figure 5 Statistical Analysis of Effluent Quality Data

TSS AND CBOD$_5$ REMOVAL RATES

The performance of the CVWPC treatment processes was assessed in term of removal of TSS and cBOD$_5$ from wastewater. The results are presented in Figure 6. The assessed period is from October 2014 to December 2017 due to a limited amount of influent data available for both parameters. There was no influent cBOD$_5$ data available, instead the influent BOD$_5$ data was used to estimate the cBOD$_5$ removal. The influent cBOD$_5$ and BOD$_5$ concentrations are expected to be similar as nitrifying bacteria are not commonly present in the influent wastewater. The average percentage removal of TSS and cBOD$_5$ during the assessed period (Oct 2014 to Dec 2017) was 97%. The removal rate for TSS was consistently high ranging from 95% to 99% most of the time with an average effluent concentration is less than 9 mg/L. The removal rate of cBOD$_5$ was above 93% and an average effluent concentration of less than 8 mg/L. Removal rates can be expected to decline as loading to the plant increases.

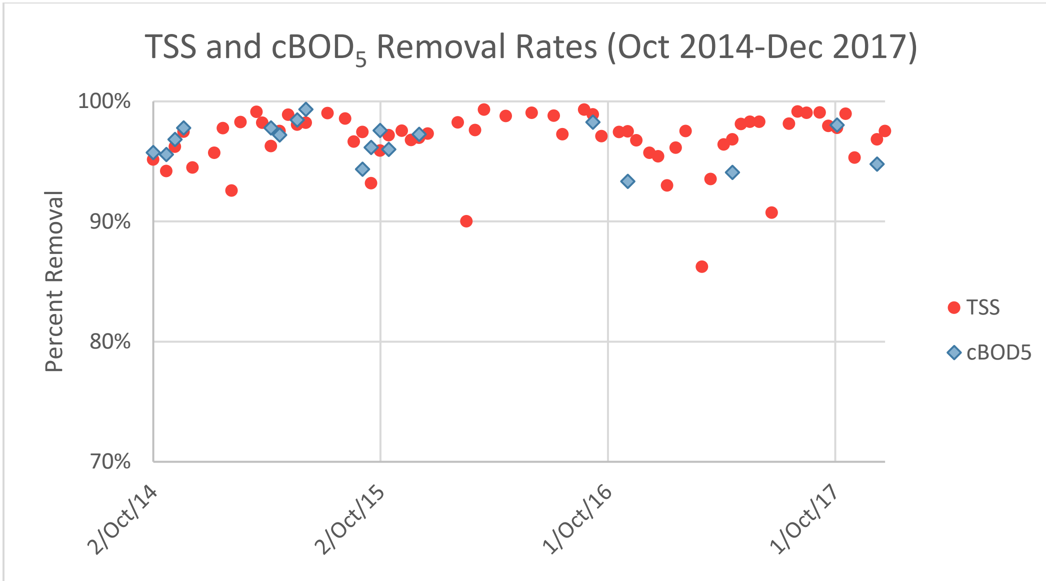

Figure 6 TSS and $cBOD_5$ Removal Rates

POPULATION, FLOW AND LOAD PROJECTIONS

CVWPCC POPULATION PROJECTIONS

Population for the CVWPCC service area is projected based on expected growth rates for the area. Current service areas to the CVWPCC include the City of Courtenay, the Town of Comox, CFB Comox and K'ómoks First Nation (KFN). Historical population for the City of Courtenay and the Town of Comox (includes KFN) was obtained from the BC Stats database. According to the 2016 ISL *CVWPCC Capacity Assessment* report, future connections to the CVWPCC service area include 400 single-family units referred to as the CVRD Annexation; this is also included in the population assessment shown in Table 1. Service area growth is projected using the annual growth rates used by ISL in their 2016 *CVWPCC Capacity Assessment*. Table 1 shows the historical and projected populations for the service area; as shown, the CVWPCC.

Table 1: Historical and Projected CVWPCC Service Population to Year 2060

YEAR	CITY OF COURTENAY [1]	TOWN OF COMOX [2]	CFB COMOX	CVRD ANNEXATION [3]	K'OMOKS FIRST NATION [4]	TOTAL
2013	24,815	13,933	966	-		39,714
2014	25,187	14,216	966	-		40,369
2015	25,782	14,518	966	-		41,266
2016	26,736	14,652	966	-		42,354
2017	27,146	14,850	966	-		42,962
2018	27,533	14,706	966	-	293	43,498
2019	28,117	14,994	966	-	293	44,370

YEAR	CITY OF COURTENAY [1]	TOWN OF COMOX [2]	CFB COMOX	CVRD ANNEXATION [3]	K'OMOKS FIRST NATION [4]	TOTAL
Projected Population						
2020	28,713	15,281	966		299	45,259
2030	33,053	17,558	966	1,098	343	53,018
2040	37,759	20,057	966	1,274	392	60,448
2050	43,135	22,913	966	1,478	448	68,940
2060	49,277	26,176	966	1,716	511	78,645

1 2020 – 2021 growth rate of 2.12% and 2022 – Future growth rate of 1.34% from ISL 2016
2 2020 – 2021 growth rate of 1.92% and 2022 – Future growth rate of 1.34% from ISL 2016
3 2020 – Future growth rate of 1.5% used from ISL 2016
4 Assuming 122 units, with 2.4 people per connection. Growth rate of 1.34%. .

FLOW PROJECTIONS

The 2013 to 2017 flow rates provided in Table 2 were used to generate average per capita flow rates into the CVWPCC. These were applied to future year population projections to determine future flow rates to year 2060. The flow rates were determined as follows:

- Average Dry Weather Flow (ADWF): Minimum 30-day rolling average flow for the year;
- Average Daily Flow (ADF): Average flow during the year;
- Average Wet Weather Flow (AWWF): Maximum 30-day rolling average flow for the year;
- Max day flow (MDF): Maximum single day flow in the year;
- Peak Hourly Flow (PHF): Peaking factor developed by ISL (2016) was used to determine projected PHF (3.0 x ADF); and
- Maximum Instantaneous Flow: Peaking factor developed by ISL (2016) was used to determined projected maximum instantaneous flow (3.2 x ADF).

Table 2: Historical Flows, 2013-2017

		HISTORICAL FLOWS [1], M^3/DAY				UNIT FLOWS, L/C/D			
Year	**Population**	**ADWF**	**ADF**	**AWWF**	**MDF**	**ADWF**	**ADF**	**AWWF**	**MDF**
2013	39,714	12,142	13,249	15,029	21,225	306	334	378	534
2014	40,369	11,906	14,221	20,000	38,462	295	352	495	953
2015	41,266	11,504	13,732	21,914	37,253	279	333	531	903
2016	42,354	11,518	15,462	23,533	39,998	272	365	556	944
2017	42,962	11,694	14,328	19,650	34,965	272	334	457	814
					Average	**285**	**343**	**484**	**830**

1 From Daily Influent Plant Data.

With the data available to WSP at the time of completing this memo, peak hourly flows (PHF) and maximum instantaneous flow were not able to be determined with the data, therefore the peaking factors from ISL (2016) were used. Table 3 shows these projected future flows from 2020 to 2060. These flow projections use the same per capita flows determined in Table 3.

Table 3: Flow Projections, 2020-2060

	2020	2030	2040	2050	2060
Population Projection	45,259	53,018	60,448	68,940	78,645
Flow Projections					
Average Dry Weather Flow (ADWF) (m^3/d)	12,885	15,094	17,210	19,627	22,390
Average Day Flow (ADF) (m^3/d)	15,542	18,206	20,758	23,674	27,007
Average Wet Weather Flow (AWWF) (m^3/d)	21,887	25,640	29,233	33,339	38,033
Max Day Flow (MDF) (m^3/d)	37,547	43,984	50,148	57,193	65,244
Peak Hour Flow [1] (PHF) (m^3/d)	46,626	54,619	62,274	71,022	81,020
Maximum Instantaneous [2] (m^3/d)	49,734	58,260	66,425	75,757	86,421
Maximum Instantaneous (L/s)	576	674	769	877	1,000

1 Peaking Factor of 3.0 was adapted from the ISL CVWPCC Capacity Assessment (2016).

2 Peaking Factor of 3.2 was adapted from the ISL CVWPCC Capacity Assessment (2016).

LOAD PROJECTIONS

Table 4 summarizes the historical (2013 to 2017) CVWPCC influent 5-day Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS) loadings used to develop average per capita unit loading rates. The $cBOD_5$ and TSS data are taken from weekly composite samples. Average BOD_5 and TSS influent loads to the CVWPCC are shown in Table 4.

Table 4: CVWPCC Historical Influent Loading, 2013-2017

		HISTORICAL INFLUENT LOADING [1] KG/D				INFLUENT UNIT LOADING `G/C/D			
Year	Population [2]	Average BOD_5	Max Month BOD_5	Average TSS	Max Month TSS	Average BOD_5	Max Month BOD_5	Average TSS	Max Month TSS
2013	39,714	3,327	4,085	3,425	4,383	84	103	86	110
2014	40,369	3,720	8,983	4,144	6,198	92	223	103	154
2015	41,266	3,675	5,641	3,977	5,351	89	137	96	130
2016	42,354	2,605	6,919	4,412	6,988	62	163	104	165
2017	42,962	2,946	4,306	4,116	5,189	69	100	96	121
					Average	**79**	**145**	**97**	**136**

1 Plant Data. We have assumed this data includes all return streams from the plant.

2 Population was obtained from BC Stats.

The average per capita loading for BOD_5 and TSS were rounded to 80 and 100 g/c/d. These values compare to the ISL (2016) per capita values used of 90 g/c/d and 100 g/c/d for BOD_5 and TSS, respectively. We have assumed that the loads from ISL (2016) and the data WSP analyzed includes the additional loading received from septage and return flows in the plant.

We note that the peaking factor between average and max month BOD_5 per capita loading (1.8) is more than what would be expected for typical domestic wastewater. Therefore, we have removed the 223 g/c/d data point for the year 2014 and are using an average max month per capita loading of 126 g/c/d for BOD_5. TSS max month loading was found to be 136 g/c/d. This compares with the max month loading from ISL (2016) of 117 g/c/d and 120 g/c/d for BOD_5 and TSS, respectively.

No data was available for Total Kjeldahl Nitrogen (TKN), therefore loading data is based on per capita unit rates from ISL (2016). The TKN loading determined in ISL (2016) was based on 13 g/c/d, which is considered typical for domestic wastewater without any industrial loading. They also determined a peaking factor of 1.1 between average and max month loading. These same values were carried forward for projecting TKN load to the CVWPCC. Table 5 shows the projected future loads to the CVWPCC for BOD_5, TSS, and TKN.

Table 5: Load Projections, 2020-2060

	2020	2030	2040	2050	2060
Population Projection	45,259	53,018	60,448	68,940	78,645
Load Projections					
BOD_5					
Average BOD_5 (kg/d)	3,621	4,241	4,836	5,515	6,292
Max month BOD_5 (kg/d)	5,693	6,669	7,603	8,672	9,892
TSS					
Average TSS (kg/d)	4,526	5,302	6,045	6,894	7,865
Max month TSS (kg/d)	6,155	7,210	8,221	9,376	10,696
TKN					
Average TKN (kg/d)	588	689	786	896	1,022
Max month TKN (kg/d)	647	758	864	986	1,125

CVWPCC UPGRADE OPTIONS

EXISTING WASTEWATER TREATMENT SYSTEM

The CVWPCC is a secondary treatment level activated sludge plant that was constructed in 1982 and receives flow from five (5) pump stations. The plant has the following treatment processes at the facility:

— Preliminary treatment with two coarse bar screens and three pre-aeration grit removal tanks;
— Three primary clarifiers;
— Three activated sludge aeration basins;
— Three secondary clarifiers;

- Effluent outfall and pump station for peak flows;
- Two gravity thickeners for the primary sludge (PS);
- Two dissolved air flotation (DAF) units for waste activated sludge (WAS) thickening;
- One combined (PS and WAS) thickened sludge storage tank;
- Two centrifuges for dewatering;
- Ancillary process such as odour control and grit classification.

A capacity assessment for each unit process is provided in the Appendix, which reviews the technical details for each of the unit processes and estimates the capacity for treatment. This assessment did not investigate the condition of the assets and assumes any infrastructure planned for reuse is in a serviceable condition.

OPTIONS FOR ASSESSMENT

The following options (Table 6) were developed from the Stage 1 LWMP long-list in January 2019. All the options, except for Option 1 were advanced to the Stage 2 shortlist for more detailed assessment. Option 1 was not carried forward since it represents a step-back form the existing treatment system.

Table 6: Options for Assessment

OPTION	DESCRIPTION	ADVANCE TO STAGE 2?
Option 1 – Secondary treatment for flows up to 2xADWF with disinfection	Secondary treatment for flows up to 2 x ADWF: — 5-Day BOD_5: Max day <45 mg/L; monthly average <25 mg/L — TSS: Max day <45 mg/L, monthly average <25 mg/L — pH 6 – 9 — Ammonia does not cause chronic toxicity at the edge of the IDZ — Total residual chlorine < 0.02 mg/L — Un-ionized ammonia < 1.25 mg/L at 15°C — Disinfection – fecal coliforms not to exceed 200 MPN/100 mL (end of pipe) Primary treatment for flows in excess of 2 x ADWF: — 5-day BOD_5: Max day <130 mg/L — TSS: Max day < 130 mg/L — In this scenario, primary treated flows >2xADWF are bypassed around secondary treatment and then blended with the secondary treated flow.	No
Option 2 - Secondary treatment for entire flow with disinfection (base case)	Secondary treatment for the entire plant flow: — 5-Day BOD_5: Max day <45 mg/L; monthly average <25 mg/L — TSS: Max day <45 mg/L, monthly average <25 mg/L — pH 6 – 9 — Ammonia does not cause chronic toxicity at the edge of the IDZ — Total residual chlorine < 0.02 mg/L — Un-ionized ammonia < 1.25 mg/L at 15°C — Disinfection – fecal coliforms not to exceed 200 MPN/100 mL (end of pipe)	Yes

OPTION	DESCRIPTION	ADVANCE TO STAGE 2?
Option 3 – Addition of advanced treatment for 2xADWF	Secondary treatment for the entire plant flow (as outlined in Option 2 – Base Case), and also include: — Advanced treatment (filtration) of the secondary treated effluent up to 2 x ADWF, with flows in excess of 2 x ADWF being bypassed around the effluent filters, and the two streams then blended prior to disinfection. — UV disinfection fecal coliform not to exceed < 200 MPN/100 mL for all flows (end of pipe).	Yes
Option 4 - Addition of advanced treatment for entire flow	Secondary treatment for the entire plant flow (as outlined in Option 2 – Base Case), and also include: — Advanced treatment (filtration) of the entire secondary treatment flow — UV disinfection on all filtered wastewater. Fecal coliform not to exceed 200 MPN/100 mL.	Yes
Option 5 – Reclaimed Water	Reclaimed water for in-plant use. Can be applied to any of Options 2, 3 or 4.	Yes

DESIGN CRITERIA

The options outlined below are based on the following design conditions and information available at the time of completing this assessment:

- Design horizon: 20-year design horizon to 2040
- Flows and loads as outlined for year 2040 in Table 3 and Table 5.
- We have assumed similar technologies that are currently in use will be used for expansion and have not compared other possible process options for treatment.
- Disinfection is to be included.
- Provincial and Federal effluent quality requirements are applicable, as outlined in Table 7.
- The purpose of this review is to provide sufficient information to decide on the treatment level to be implemented at the CVWPCC.

Table 7: Effluent Quality Criteria

EFFLUENT PARAMETER	PROVINCIAL REQUIREMENTS (MWR)	FEDERAL REQUIREMENTS (WSER)
5-day Biochemical Oxygen Demand	Max day < 45 mg/L	Monthly average < 25 mg/L
Total Suspended Solids (TSS)	Max day < 45 mg/L	Monthly average < 25 mg/L (carbonaceous BOD_5)
pH	6 – 9	N/A

EFFLUENT PARAMETER	PROVINCIAL REQUIREMENTS (MWR)	FEDERAL REQUIREMENTS (WSER)
Un-Ionized Ammonia	N/A	<1.25 mg/L
Total Residual Chlorine	N/A	<0.02 mg/L
Fecal Coliforms [1]	<14 MPN/100 mL at the edge of the initial dilution zone (IDZ)	N/A

1 Requirements for shellfish receiving waters

KEY CONSIDERATIONS

Several considerations should be given to how the plant might be laid out for future upgrades, and how new infrastructure components can fit into the existing plant and mesh with future plans for the facility. A new offline equalization tank is currently being constructed at the CVWPCC to handle peak flows to the treatment plant, and this should be incorporated into future planning of conceptual layouts if possible. A Master Plan should be undertaken to determine the optimum plant layout for future expansions and upgrades once the level of treatment has been identified.

Key considerations that have been identified for potential future upgrades and expansions of the CVWPCC are summarized in Table 9.

Table 8: CVWPCC Key Consideration Identification

KEY CONSIDERATION	RISK	POTENTIAL MITIGATION OPTIONS
1. How is new infrastructure integrated with the existing plant?	**A.** By 2040 the existing infrastructure will be 60 years old. Condition assessments may find that some of the assets may be at the end of their useful life and may not be in the plans for future expansions to 2060. **B.** The available head in the hydraulic profile is limited and may limit options to avoid pumping between unit processes. **C.** New equalization tank under construction reduces the available area for construction of other facilities in future.	**A.** Ensure new infrastructure can be used well into the future. **B.** Incorporate flow control options within plant layout, or allow for tie-in to future flow control options to maintain equal division of flows to multiple process units and allow addition of future processes and upgrades.

KEY CONSIDERATION	RISK	POTENTIAL MITIGATION OPTIONS
2. How much longer can the existing infrastructure be used?	**A.** The generally harsh conditions from H_2S exposure, can erode concrete and mechanical components in the headworks and primary clarifiers. Re-use of these systems beyond 2040 may be limited and new facilities will likely be required.	**A.** Condition assessments can be completed on these components that outline faulty or weak components and repairs can be designed to extend the life of the systems. This would be included for aeration basins and secondary clarifiers as well to ensure life beyond 2040.
3. What will the solids handling components look like in the future?	**A.** The wastewater treatment plant currently hauls dewatered waste solids to a composting facility. Changes in regulations, cost-benefit analysis, and other factors can drive decisions for future solids handling options such as anaerobic digestion where gas and energy can be recovered. **B.** Age of current solids handling equipment might require refurbishment to ensure the equipment will last until at least 2040.	**A.** Future space considerations for anaerobic digestion with resource recovery (biogas, fertilizer pellets) should be included when developing future plant layouts. Digestion could potentially be part of a future overall solids handling system upgrade. **B.** A condition assessment of structural and mechanical components on the thickeners (gravity and DAF units) can provide insight into repairs that may be needed to ensure the life of the equipment will last until at least 2040, as the components do have the capacity.
What are the geotechnical conditions of the site and post-disaster structural considerations?	**A.** Building codes and the status of wastewater facilities have become more stringent. New infrastructure at wastewater treatment plants now has to be "post-disaster", which means operable after a natural disaster, such as a major earthquake.	**A.** Complete geotechnical assessments to evaluate the ground conditions at the site in light of the new regulations. **B.** Complete a structural condition assessment to review the existing infrastructure, expected lifespan, and possible upgrades that may be required to make the infrastructure meet post-disaster requirements.

KEY CONSIDERATION	RISK	POTENTIAL MITIGATION OPTIONS
What are the odour concerns?	**A.** New infrastructure should not create increased odours in the area	**A.** Include allowances for odour connections and odour control measures in new infrastructure. It should be noted we have not reviewed the capacity of the existing odour control system.

OPTION 2 – SECONDARY TREATMENT WITH DISINFECTION FOR ENTIRE FLOW

Option 2 is the base case scenario that will provide secondary treatment for the entire wastewater flow, as is currently the case at the CVWPCC. The provincial and federal effluent criteria outlined in Table 9 are used in addition to the capacity in the existing system to determine upgrades required to provide capacity until 2040.

A UV disinfection system is included to disinfect the wastewater to not exceed 200 MPN/100 mL fecal coliform concentration at the end of the outfall pipe. Based on the dilution modelling in the Initial Dilution Zone (IDZ), there will be sufficient dilution at the edge of the IDZ to stay below the 14 MPN/100 mL requirement for protection of shellfish. The design criteria used to size the UV system are shown in Table 9.

Table 9: UV System Design Criteria

CRITERIA	
No. of Units	2
Design Flow: 2040 75% PHF per Unit (m^3/d)	46,706
Influent to Disinfection Process	30 mg/L TSS
UV Transmittance[1]	55%

1 A measure of the ability of UV light to penetrate wastewater and disinfect organisms. Typically determined from wastewater testing, which the CVRD has completed with a vendor previously.

The configuration of the UV system would be two UV disinfection channels with each UV bank be designed to treat 75% of the design flow with the largest unit out of service, in accordance with the provincial MWR Reliability Requirements. The UV system can be placed outside in concrete channels and does not need to be in a building. (However, in the Option 3 and Option 4 assessments, a building should be constructed for the disk filters, and we have assumed some additional floor area in the same building to house the UV system as well).

In this base case, the following items were identified as items requiring a capacity increase. A detailed condition assessment of some of the structures should be completed to fully assess the suitability of re-using some of the plant's existing infrastructure.

PRELIMINARY TREATMENT

- Upgraded grit removal is required and will benefit the plant in several ways, including improved sludge thickening in the primary clarifiers.

PRIMARY CLARIFIERS

- The existing primary clarifiers have adequate capacity to 2040.

AERATION BASINS

- There is not enough capacity in the existing three aeration basins to handle 2040 loads and a fourth aeration basin is required (refer to the Appendix for the capacity assessment).
- We have assumed that the existing blower room can be re-used and that there is sufficient blower capacity in the existing system (refer to capacity assessment). Although installing new, higher efficiency blowers, may be desired and would be evaluated in a pre-design.

SECONDARY CLARIFIERS

- A fourth secondary clarifier is required and would be installed to the north of the existing clarifiers.
- A new flow splitting box should be installed to ensure equal flow to the four clarifiers, if determined feasible during a pre-design.

UV SYSTEM

- A new UV disinfection system would be installed outdoors with the design criteria outlined in Table 9. The system is assumed to be a Trojan UV Signa system with 96 UV lamps.

The items identified are believed to make-up a possible upgrade scenario for the CVWPCC to meet treatment objectives until 2040 and provide flexibility for expansion beyond 2040. Investigating the feasibility of various upgrade options and scenarios, together with alternative treatment technologies, optimum plant layout, and a condition assessment of the existing infrastructure, should be completed during a Master Plan or Pre-Design to confirm the recommended approach.

OPTION 3 – ADDITION OF ADVANCED TREATMENT FOR 2XADWF

Option 3 includes the Option 2 components plus the inclusion of disk filters for advanced treatment to produce a higher quality effluent, i.e. less than 10 mg/L TSS and BOD_5, for flows up to 2xADWF. It is anticipated in this scenario a type of flow control weir would be installed to divert higher flows exiting secondary treatment around the disk filters and directly to UV disinfection. Disk filters, or cloth media filters, are disks covered in a cloth material that are placed in a channel where the wastewater meets the filters. The filters and the wastewater continue through the filters and to the outfall. A rendering from a disk filter proposal we received is shown in Figure 7.

Figure 7: Disk Filter Rendering (from Nexom Proposal in Appendix)

The disk filters were sized for the 35,000 m^3/d with an influent TSS concentration of 25 mg/L. Each unit is sized to treat 8,750 m^3/d. Four disk filter units are proposed to handle the 2040 flows, which provides 75% redundancy. The disk filters are recommended to be placed inside a building. Based on the size of the disk filters required to handle flows up to 2 x ADWF, we have used a building size of 20 by 15 meters. The building was sized to include the UV system, since the UV system has a generally small footprint and inclusion of the UV system indoors would have many benefits at a minimal cost.

OPTION 4 – ADDITION OF ADVANCED TREATMENT FOR ENTIRE FLOW

Option 4 is the same as Option 3, except the disk filters are sized for the entirety of the flow (62,000 m^3/d). This disk filter system is assumed to include eight disk filters, each sized for 8,750 m^3/d, providing a redundant unit. The larger disk filter footprint would require a building approximately 20 by 20 meters and would also include the UV system. The UV system in this scenario would be the smallest since the entire flow is treated to a higher standard and provide the highest quality. In this scenario, the UV system is assumed to be a Trojan Signa with 60% UVT and an influent TSS of 10 mg/L. This system would require a total of 56 UV lamps.

COST COMPARISON FOR OPTIONS 2, 3 AND 4

Preliminary planning capital cost estimates are based on the ISL report and other considerations developed by WSP during the capacity assessment to upgrade the treatment plant to handle the 2040 design flows and loads. These estimates provide a general outline for the work that may be required. A detailed Pre-design study with treatment process modelling will be needed to develop more detailed estimates and upgrade staging scenarios.

Included in the Option 3 cost estimate are the base case estimates described for Option 2 and inclusion of advanced treatment with disk filters. A benefit of the disk filter system is the higher quality water that is then sent for UV disinfection. With the higher quality wastewater, i.e. fewer solids, the UV system can be downsized. With disk filters there is a higher UV Transmittance (UVT) and thus fewer light bulbs are required in the UV system. In this scenario it is assumed that the Trojan Signa system would have 64 lamps, a UV transmittance of 60%, and TSS of 15 mg/L in the blended effluent.

The capital cost summary from the ISL (2016) report is summarized Table 10. The ISL (2016) report stages the work, therefore we have taken the Phase I (year 2017), Phase II (year 2024), and Phase III (year 2033/2034) cost estimates as they are the most comparable to the base case costs developed in this assessment. The cost numbers have been adjusted in the ISL (2016) report to be in 2019 Canadian dollars using the Engineer News-Record (ENR) indexing values. Note that the costs are taken from the recommended 'Option 3' in the ISL report. Additionally, we note that the ISL option includes disk filters for the full flow for initial removal of solids before a new secondary clarifier is installed, therefore we have removed the disk filter estimate component and provided it as a separate item, although in the ISL (2016) staging plan the disk filters would be required before the secondary clarifier is installed. Note that the ISL (2016) estimate does not include UV treatment which is included in all options developed in this assessment.

The estimates prepared in this assessment represent a total amount that would be required to meet 2040 treatment objectives. We have not phased the work as this would typically be completed during a Pre-design or Master Plan. We have compared the ISL (2016) estimates with the WSP estimates in Table 10, since they are both based on plans going forward to achieve the same treatment objectives for 2040. The ISL (2019 adjusted) estimate not including disk filters ($27.6M) would be comparable to Option 2 in this assessment and the ISL (2019 adjusted) estimate to include disk filters ($38.4M) would be comparable to Option 4 in this assessment, including disk filters to treat the entirety of the flow.

We note that the ISL (2016) estimate included more detail beyond 2040, therefore the total life cycle cost estimates for only the items selected to meet 2040 expansion could not be determined and compared to the WSP estimate. However, we would expect them to be similar.

Table 11 shows the ISL (2019 adjusted) cost estimate and the WSP Option 2 estimate which represent the estimated total cost for the CVRD to expand the plant to handle 2040 flows and loads assuming secondary treatment for all flows. The Option 3 and 4 estimates shown the incremental increase in cost associated with adding effluent filtration for 2xADWF and all flows, respectively. We note the estimates in this assessment include a 5% greater (45% vs 40%) engineering and contingencies amount than the ISL (2016) estimate. The actual path forward, and staging of the expansion, would be determined during a Pre-Design step.

Table 10: ISL (2016) Report - Option 3 Cost Estimate Comparison

STAGE	**ISL ESTIMATE 2016 CDN$**	**ISL ESTIMATE 2019 CDN$[1]**	**ISL ESTIMATE 2016 CDN$**	**ISL ESTIMATE 2019 CDN$[1]**
	ISL Option 3 Not Including Disk Filters		**ISL Option 3 Including Disk Filters**	
Phase I (2017)	$ 5,774,000[2]	$ 6,289,000	$ 11,063,000[2]	$ 12,050,000
Phase II (2024)	$4,721,000[3]	$5,142,000	$4,721,000[3]	$5,142,000
Phase III (2033/2034)	$7,651,000[4]	$8,333,000	$9,410,000[5]	$10,249,000
Engineering & Contingencies (40%)	$7,258,400	$7,906,000	$10,077,600	$10,976,000

STAGE	ISL ESTIMATE 2016 CDN$	ISL ESTIMATE 2019 CDN$[1]	ISL ESTIMATE 2016 CDN$	ISL ESTIMATE 2019 CDN$[1]
Total Capital Cost Estimate	$25,404,400	$27,670,000	$35,271,600	$38,417,000

1 ENR Index Values used for 2016: 10,339 and ENR Index Values used for 2019: 11,261
2 From ISL (2016) Table 12.1 for Option 3 – with and without disk filters line item.
3 From ISL (2016) Table 12.2 for Option 3
4 From ISL (2016) Table 12.3 for Option 3 Primary Clarifiers + Process Building – Year (2033) Line Item
5 From ISL (2016) Table 12.3 for Option 3 Primary Clarifiers + Process Building – Year (2033) Line Item and Upgrade Media Cloth Filter – Year (2034) Line Items.

Table 11: Capital Cost Comparison

	ISL (2019) ESTIMATE	OPTION 2	OPTION 3	OPTION 4	ISL (2019) ESTIMATE
	Not Including Disk Filters	Secondary Treatment for Entire flow w/ Disinfection	Addition of Advanced Treatment for 2XADWF	Addition of Advanced Treatment for Entire Flow	Including Disk Filters
Total Capital Cost Estimate	$27,670,000	$29,700,000	$38,000,000	$40,300,000	$38,417,000
20 Year Life Cycle Cost Estimate	-	$32,000,000	$40,500,000	$43,000,000	-

OPTION 5 – RECLAIMED WATER FOR IN-PLANT USE

Option 5 evaluated including reclaimed water around the CVWPCC for equipment wash water and other reuse items. Reclaimed water standards are set-out in the provincial regulation (MWR) and are classified by exposure potential to the public. Reclaimed water use within the treatment plant would need to meet the lowest exposure potential standards as the reclaimed water would be controlled in the plant setting.

The MWR requires, for low exposure potential, a maximum TSS and BOD_5 concentration of 45 mg/L, a disinfection to 200 MPN/100 mL, and maintaining a pH between 6.5 and 9. Additionally, the MWR requires the reclaimed water to be chlorinated to have a minimum of 0.5 mg/L chlorine residual in the reclaimed water at the point of use.

We have designed several reclaimed water systems for wastewater treatment plants in British Columbia. We have assumed a similar sized system would be installed at the CVWPCC. This system would include a pressure filter to remove TSS and a chlorination system to maintain a residual of 0.5 mg/L total chlorine. There would be a reclaimed water distribution pumping and piping network installed around the plant to service the various mechanical equipment, or onsite irrigation as maybe desirable.

The design criteria for the reclaimed water system is shown in Table 12.

Table 12: Reclaimed Water Design Criteria

CRITERIA	
Capacity	50 m^3/d
Pressure Filter Capacity (L/min)	100
Chlorine Dosing System Capacity (mL/min)	10 – 110
Distribution Pump Capacity (L/s)	5
Reclaimed Water Clearwell Tank (m3)	100 – 150
Chlorination Dosing	12% Sodium Hypochlorite @ 15 mg/L

A detailed investigation into the wash water requirements for the reclaimed water system was not included in the scope of work. However, our experience with using this sized system at other wastewater treatment plants indicate sufficient capacity to service most equipment around a wastewater treatment plant. We also have sodium hypochlorite cost estimates for approximately $1,000 per 1,100L tote, and have assumed approximately 1 tote every 3 months would be required. A cost estimate for Option 5 is shown in Table 13.

The cost estimate shown in Table 13 would be for a system treating Option 2 secondary effluent to reclaimed water standards. If disk filters are included and provide a higher quality effluent (Option 3 and 4), the reclaimed water system overall cost could potentially be reduced by approximately $100,000 - $150,000.

Table 13: Option 5 Cost Summary

OPTION 5 - RECLAIMED WATER	AMOUNT
Civil Works	$ 24,000
Process Mechanical	$ 130,000
Structural Components	$ 180,000
Plumbing & HVAC	$ 8,000
Electrical	$ 68,000
General Requirements	$ 109,000
Subtotal Option 5 Cost Estimate	**$ 519,000**
Engineering (15%)	$ 78,000
Contingency (30%)	$ 179,000
Total Option 5 Cost Estimate	**$ 776,000**
Estimated Annual O&M Addition	$ 6,900
PV Annual O&M (20 years, 5% Discount Rate)	$ 88,000
Total Net Present Value Option 5	**$ 864,000**

SUMMARY

A summary of the cost estimates for the different treatment level options is shown in Table 14. We note that due to the relatively similar amounts between ISL (2019) and the estimates developed by WSP in this assessment, the estimates developed in this assessment will be used for comparison purposes.

This estimate also includes the estimate from the ISL (2016) *Cape Lazo Outfall Capacity Assessment* for 'Option 3' of approximately $24.4M which is carried to indicate future capital upgrade requirements for the outfall. Also note that Option 5 is only for including reclaimed water and does not represent a standalone option for the District in terms of upgrading the secondary treatment capacity.

Upgrades to meet federal and provincial requirement by implementing secondary treatment upgrades are effective in protecting the receiving environment, removing microplastics and disinfecting the effluent prior to release in the receiving environment.

Currently, advanced treatment is not a regulatory requirement for an ocean discharge, and advanced treatment is not strictly required to meet the regulatory treatment objectives for the CVWPCC with appropriate expansion of the existing systems. To provide advanced treatment for the entire flow with disk filters, it is currently estimated as a 35% to 40% increase in capital costs (~$11M). To provide advanced treatment for 2xADWF with disk filters, it is estimated as an approximate 25% to 30% increase in capital costs (~$8M).

The added benefit of disk filters includes treating the effluent to a slightly higher standard, enhanced removal of microplastics, and additional removal of other contaminants associated with the solids in the effluent. As shown in Table 14, and in Figure 6, the CVWPCC currently achieves excellent removal of TSS and BOD_5, with average values for both parameters less than 10 mg/L; this would be expected to improve to less than 5 mg/L average with the addition of disk filters. If phosphorus removal becomes a regulatory requirement in the future, the disk filters would provide additional filtration to reduce phosphorus concentrations following chemical coagulation. Additionally, implementation of disk filters would meet the effluent standards for reclaimed water, enabling a wide range of uses. However, in the absence of a user for large scale reclaimed water, the estimated 35% increase in capital cost between Options 2 and 3 or 4 may not justify installation of disk filters for advanced treatment at this point in time.

A summary of the costs, risks and benefits of the different options is shown in Table 14.

Table 14: Summary of Wastewater Treatment Level Options

	OPTION 2	OPTION 3	OPTION 4
	Secondary Treatment w/ Disinfection Base Case	**Advanced Treatment for 2xADWF**	**Advanced Treatment for Entire Flow**
Sub-Total CVWPCC Upgrade Capital Costs	$ 29,700,000	$ 38,000,000	$ 40,300,000
Sub-Total Outfall Upgrades[1]		$24,400,000	
Total	**$ 54,100,000**	**$ 62,400,000**	**$ 64,700,000**
Subtotal Reclaimed Water (Option 5)		$800,000	
Total	**$ 54,900,000**	**$ 63,200,000**	**$ 65,500,000**
Benefits	— Upgrade path to meet capacity and regulatory requirements for the next 20 years — Secondary treatment removes 90% of organic material and solids on average (note that the CVWPCC currently achieves greater than 95% removal of TSS and greater than 93% removal of BOD_5) — Secondary treatment removes 80-95% of microplastics on average — Disinfection to meet shellfish standards — Reclaimed water can be incorporated. — Design can incorporate space for installation disk filters if required in the future. — Typical CVWPCC effluent quality for daily BOD_5 consistently less than 20 mg/L and TSS less than 25 mg/L, with average values less than 10 mg/L.	— Base case secondary treatment upgrades apply — Advanced treatment (filtration) for up to 2xADWF accounts for approximately 99% of the annual flow being treated to advanced standards. — Addition of advanced treatment filtration removes 96% of organic material and solids on average, a marginal increase of 6% over secondary treatment — Addition of disk filters removes 95-97% of microplastics on average, a marginal increase of 15-17% over secondary treatment — Large scale effluent reuse can be implemented — Disk filters can be implemented in the future once a user for reclaimed water is identified — Typical effluent quality for up to 2xADWF for daily BOD_5 and TSS consistently less than 10 mg/L, with average values less than 5 mg/L.	— Base case secondary treatment upgrades apply — Addition of disk filters removes 96% of organic material and solids on average, a marginal increase of 6% over secondary treatment — Addition of advanced treatment filtration removes 95-97% of microplastics on average, a marginal increase of 15-17% over secondary treatment — Large scale effluent reuse can be implemented — Disk filters can be implemented in the future once a user for reclaimed water is identified — Typical effluent quality for entire flow for BOD_5 and TSS consistently less than 10 mg/L, with average values less than 5 mg/L.
Risks	— Capital costs are dependent on condition assessment and outcome of a Pre-design study.	— Cost premium of approximately $8M for addition of disk filters to treat 2xADWF — Advanced treatment to the level provided by disk filters is not a regulatory requirement — Without a user for the reclaimed water, costs may not be justified at this point in time	— Cost premium of approximately $10.7M for addition of disk filters to treat the full flow — Advanced treatment to the level provided by disk filters is not a regulatory requirement — Without a user for the reclaimed water, costs may not be justified at this point in time

1 From ISL (2016) Cape Lazo Outfall Capacity Assessment, to be updated.

2 Cost estimates are in $2019 CAD. Estimates are appropriate for the purposes of comparing options.

A detailed Pre-design and Condition Assessment for the wastewater treatment plant is recommended. The purpose of these studies would be to:

- Detail the suitability of reusing existing infrastructure and identify any repairs that should be carried out before re-using;
- Create a process model for the treatment plant to identify if there are any modifications that can be done to the existing system to increase performance and capacity;
- Evaluate existing structures and geotechnical conditions that consider post-disaster seismic standards currently required by the B.C. Building Code (BCBC);
- Evaluate plant wide odour control systems and necessary upgrades;
- Complete a pre-design study that provides a detailed, staged expansion plan for the CVWPCC for the next 50 years and beyond;
- Undertake a complete hydraulic assessment of the plant systems;
- Review the plant electrical, controls, and SCADA systems;
- Complete detailed composite sampling to confirm loading in the influent and primary effluent.

A staged approach to upgrading the treatment plant would provide the greatest flexibility and assurance to the CVRD that the appropriate measures have been taken for the decisions that will be made about the future of the plant. The staging would involve completing a condition assessment first to assess the possibility of re-using certain assets and identifying their anticipated life expectancy. After this, a Pre-design Study can be completed knowing the specific condition of assets and creating a process model to identify and evaluate upgrade options so that the best upgrade path and site layout is selected. A preliminary cost estimate to complete these two studies is shown in Table 15.

Table 15: Pre-design and Condition Assessment Estimates

ITEM	ESTIMATE
CVWPCC Pre-Design Study	$150,000
Asset Condition Assessment	$150,000

We note that repairs to assets are not included in the estimate, nor is the engineering design for the repairs. The scope of work that would be required would be identified in the condition assessment report and an estimate of the repairs required would be provided then.

A possible timeline for completing plant upgrades for the 2040 horizon is shown in Figure 8. This estimated timeline would provide an upgraded facility for the CVRD by 2024 or 2025, and this timeline would be updated in a Pre-Design Study to confirm whether any upgrades need to be accelerated or can be delayed.

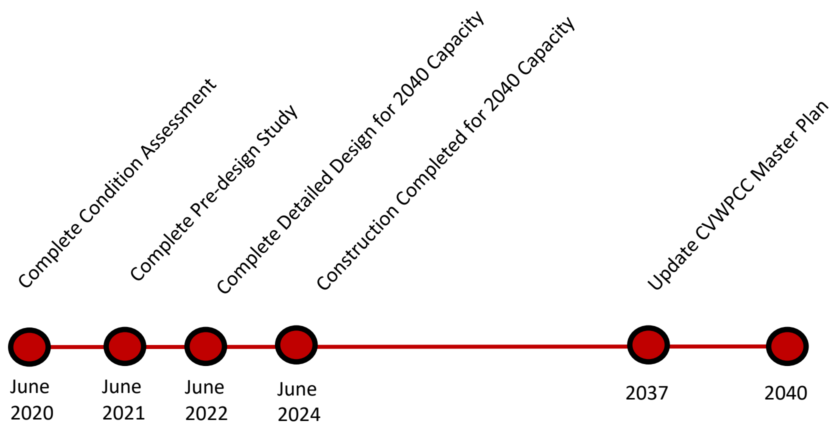

Figure 8: Project Timeline

APPENDIX

MEMO

TO: Kris La Rose, P.Eng., CVRD, Zoe Berkey, EIT, CVRD, Paul Nash

FROM: Tyler Barber, P.Eng., Aline Bennett, P.Eng., Al Gibb, P.Eng.

SUBJECT: **CVRD LWMP – Existing System Capacity Assessments (Appendix)**

DATE: **January 17, 2020**

INTRODUCTION

The Comox Valley Regional District (CVRD) has retained WSP Canada Group Ltd. to complete the Liquid Waste Management Plan (LWMP) for the District. As part of the work, WSP will complete the Stage 2 wastewater treatment assessment for the Comox Valley Water Pollution Control Centre (CVWPCC). To assist this work, this memo reviews, at a high-level, the estimated process capacity of the existing infrastructure at the CVWPCC and what is required for expansion to handle 2040 flows and loads into the CVWPCC, while considering implications for future expansion beyond 2040.

This memo describes the findings of this assessment and provides the following information:

- Updated flow and load information;
- High-level review of the capacity of each unit process;

The intent of this assessment is to provide a summary of the process units that require expansion at the CVWPCC. A base case can then be developed with a cost estimate for the minimum requirement of expansion to meet 2040 flows and loads. A large input to this memorandum is the *CVWPCC Capacity Assessment* completed by ISL Engineering and Land Services in 2016, which details operation of the plant and the existing system components.

POPULATION, FLOW AND LOAD PROJECTIONS

CVWPCC POPULATION PROJECTIONS

Population for the CVWPCC service area is projected based on expected growth rates for the area. Current service areas to the CVWPCC include the City of Courtenay, the Town of Comox, CFB Comox and K'ómoks First Nation (KFN). Historical population for the City of Courtenay and the Town of Comox (includes KFN) was obtained from the BC Stats database. According to the 2016 ISL *CVWPCC Capacity Assessment* report, future connections to the CVWPCC service area include 400 single-family units referred to as the CVRD Annexation, this is also included in the population assessment shown in Table 1. Service area growth is projected using the annual growth rates used by ISL in their 2016 *CVWPCC Capacity Assessment*. Table 1 shows the historical and projected populations for the service area.

Table 1: Historical and Projected Population to Year 2060

YEAR	CITY OF COURTENAY[(1)]	TOWN OF COMOX[(2)]	CFB COMOX	CVRD ANNEXATION[(3)]	K'OMO KS FN[4]	TOTAL
2013	24,815	13,933	966	-		39,714
2014	25,187	14,216	966	-		40,369
2015	25,782	14,518	966	-		41,266
2016	26,736	14,652	966	-		42,354
2017	27,146	14,850	966	-		42,962
2018	27,533	14,706	966	-	293	43,498
2019	28,117	14,994	966	-	293	44,370
	Future Population					
2020	28,713	15,281	966		299	45,259
2030	33,053	17,558	966	1,098	343	53,018
2040	37,759	20,057	966	1,274	392	60,448
2050	43,135	22,913	966	1,478	448	68,940
2060	49,277	26,176	966	1,716	511	78,645

[1]2020 – 2021 growth rate of 2.12% and 2022 – Future growth rate of 1.34% from ISL 2016
[2]2020 – 2021 growth rate of 1.92% and 2022 – Future growth rate of 1.34% from ISL 2016
[3]2020 – Future growth rate of 1.5% used from ISL 2016

FLOW PROJECTIONS

The 2013 to 2017 flow rates provided in Table 2 were used to generate average per capita flow rates into the CVWPCC. These were applied to future year population projections to determine future flow rates to year 2060. The flow rates were determined as follows:

- Average Dry Weather Flow (ADWF): Minimum 30-day rolling average flow for the year;
- Average Daily Flow (ADF): Average flow during the year;
- Average Wet Weather Flow (AWWF): Maximum 30-day rolling average flow for the year;
- Max day flow (MDF): Maximum single day flow in the year;
- Peak Hourly Flow (PHF): Peaking factor developed by ISL (2016) was used to determine projected PHF (3.0 x ADF); and
- Maximum Instantaneous Flow: Peaking factor developed by ISL (2016) was used to determined projected maximum instantaneous flow (3.2 x ADF).

Table 2: Historical Flows, 2013-2017

		HISTORICAL FLOWS[1], M^3/DAY				UNIT FLOWS, L/C/D			
Year	**Population**	**ADWF**	**ADF**	**AWWF**	**MDF**	**ADWF**	**ADF**	**AWWF**	**MDF**
2013	39,714	12,142	13,249	15,029	21,225	306	334	378	534
2014	40,369	11,906	14,221	20,000	38,462	295	352	495	953
2015	41,266	11,504	13,732	21,914	37,253	279	333	531	903
2016	42,354	11,518	15,462	23,533	39,998	272	365	556	944
2017	42,962	11,694	14,328	19,650	34,965	272	334	457	814
					Average	**285**	**343**	**484**	**830**

(1) From Daily Influent Plant Data.

With the data available to WSP at the time of completing this memo, PHF and maximum instantaneous flow were not able to be determined with the data, therefore the peaking factors from ISL (2016) were used. Table 3 shows these projected future flows from 2020 to 2060. These flow projections use the same per capita flows determined in Table 2.

Table 3: Flow Projections, 2020-2060

	2020	**2030**	**2040**	**2050**	**2060**
Population Projection	45,259	53,018	60,448	68,940	78,645
Flow Projections					
Average Dry Weather Flow (ADWF) (m3/d)	12,885	15,094	17,210	19,627	22,390
Average Day Flow (ADF) (m3/d)	15,542	18,206	20,758	23,674	27,007
Average Wet Weather Flow (AWWF) (m3/d)	21,887	25,640	29,233	33,339	38,033
Max Day Flow (MDF) (m3/d)	37,547	43,984	50,148	57,193	65,244
Peak Hour Flow[1] (PHF) (m3/d)	46,626	54,619	62,274	71,022	81,020
Maximum Instantaneous[2] (m3/d)	49,734	58,260	66,425	75,757	86,421
Maximum Instantaneous (L/s)	576	674	769	877	1,000

(1) Peaking Factor of 3.0 was adapted from the ISL CVWPCC Capacity Assessment (2016).
(2) Peaking Factor of 3.2 was adapted from the ISL CVWPCC Capacity Assessment (2016).

LOAD PROJECTIONS

Table 4 summarizes the historical (2013 to 2017) influent 5-day Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS) loadings used to develop average per capita unit loading rates. The BOD_5 data is taken from composite samples taken approximately once per week for the time periods indicated. Total Suspended Solids (TSS) samples are assumed to be grab samples that were taken approximately 3 – 4 times per week for the duration of the time periods indicated. Average BOD_5 and TSS loads are found in Table 4.

Table 4: Historical Influent Loading, 2013-2017

		HISTORICAL INFLUENT LOADING(1), KG/D				INFLUENT UNIT LOADING, G/C/D			
Year	**Population(2)**	**Average BOD_5**	**Max Month BOD_5**	**Average TSS**	**Max Month TSS**	**Average BOD_5**	**Max Month BOD_5**	**Average TSS**	**Max Month TSS**
2013	39,714	3,327	4,085	3,425	4,383	84	103	86	110
2014	40,369	3,720	8,983	4,144	6,198	92	223	103	154
2015	41,266	3,675	5,641	3,977	5,351	89	137	96	130
2016	42,354	2,605	6,919	4,412	6,988	62	163	104	165
2017	42,962	2,946	4,306	4,116	5,189	69	100	96	121
					Average	**79**	**145**	**97**	**136**

(1) Plant Data. We have assumed this data includes all return streams from the plant.
(2) Population was obtained from BC Stats.

The average per capita loading for BOD_5 and TSS were rounded to 80 and 100 g/c/d. These values compare to the ISL (2016) per capita values used of 90 g/c/d and 100 g/c/d for BOD_5 and TSS, respectively. We have assumed that the loads from ISL (2016) and the data WSP analyzed includes the additional loading received from septage and return flows in the plant.

We note that the peaking factor between average and max month BOD_5 per capita loading (1.8) is more than what would be expected for typical domestic wastewater. Therefore, we have removed the 223 g/c/d data point for the year 2014 and are using an average max month per capita loading of 126 g/c/d for BOD_5. TSS max month loading was found to be 136 g/c/d. This compares with the max month loading from ISL (2016) of 117 g/c/d and 120 g/c/d for BOD_5 and TSS, respectively.

No data was available for Total Kjeldahl Nitrogen (TKN), therefore loading data is based on per capita unit rates from ISL (2016). The TKN loading determined in ISL (2016) was based on 13 g/c/d, which is considered typical for domestic wastewater without any industrial loading. They also determined a peaking factor of 1.1 between average and max month loading. These same values were carried forward for projecting TKN load to the CVWPCC. Table 5 shows the projected future loads to the CVWPCC for BOD_5, TSS, and TKN.

Table 5: Load Projections, 2020-2060

	2020	2030	2040	2050	2060
Population Projection	45,259	53,018	60,448	68,940	78,645
Load Projections					
BOD_5					
Average BOD_5 (kg/d)	3,621	4,241	4,836	5,515	6,292
Max month BOD_5 (kg/d)	5,693	6,669	7,603	8,672	9,892
TSS					
Average TSS (kg/d)	4,526	5,302	6,045	6,894	7,865
Max month TSS (kg/d)	6,155	7,210	8,221	9,376	10,696
TKN					
Average TKN (kg/d)	588	689	786	896	1,022
Max month TKN (kg/d)	647	758	864	986	1,125

CVWPCC CAPACITY ASSESSMENT

EXISTING WASTEWATER TREATMENT SYSTEM

The CVWPCC is a secondary level activated sludge plant that was constructed in 1982 and receives flow from five (5) pump stations. The plant has the following treatment processes at the facility:

- Preliminary treatment with two coarse bar screens and three pre-aeration grit removal tanks;
- Three primary clarifiers;
- Three activated sludge aeration basins;
- Three secondary clarifiers;
- Effluent outfall and pump station for peak flows;
- Two gravity thickeners for the primary sludge (PS);
- Two dissolved air flotation (DAF) units for waste activated sludge (WAS) thickening;
- One combined (PS and WAS) thickened sludge storage tank;
- Two centrifuges for dewatering;
- Ancillary process such as odour control and grit classification.

The following sections provide a summary for each of the unit process in the liquid and solids treatment trains and a high-level capacity assessment of the equipment to handle the 2040 design flows and loads.

PRELIMINARY TREATMENT

The preliminary treatment at the CVWPCC consists of a headworks building with two mechanically raked bar screens located in two channels. The screens are 100% redundant and operate in a duty-standby configuration (ISL 2016).

Screen #1 was part of the original 1982 construction and has 12 mm bar openings in the screen. This spacing is large for newer wastewater treatment plants, and generally screens with 6 mm spacing are installed. Screen #2 (6 mm spacing) was installed in 2010 and replaced a manually raked bar screen in a bypass channel. According to the ISL (2016) report, the operator's have reported poor performance by the 12mm bar screen. There is an overflow weir in the channels to bypass the screens in the event of a peak flow event and significant screen blockage.

The capacity of the existing screening system is depicted in Table 6 (ISL 2016).

Table 6: Screenings System Capacity

DESCRIPTION	SCREEN #1	SCREEN #2
Number of Screens	1 (alternate duty/standby)	1 (alternate duty/standby)
Channel Width (m)	1.5	1.5
Channel Depth (m)	2.33	2.33
Bar Spacing (mm)	12	6
Rated Capacity (m^3/d)	75,000	75,000

The screening system is designed to handle the maximum instantaneous flow from the various pump stations that pump wastewater into the headworks. The current maximum instantaneous flow peaking factor as determined from ISL (2016) is 3.2 (times average daily flow). Using this peaking factor and the projected flow in Table 3 we can see that 75,000 m^3/d can service a population up until approximately 2050.

As outlined in the ISL 2016 report, the existing 12 mm mechanically raked bar screen should be upgraded to a 6 mm screen (like was done with Screen #2). The 12 mm screen from the original 1982 design is nearly 40 years old and is likely nearing the end of life as well.

ISL (2016) recommended replacing Screen #1 in the existing headworks channel. We believe this to be feasible; however, we note that the building and screenings channel are nearing 40 years old, and in our experience headworks buildings are exposed to a harsh environment due to high hydrogen sulfide (H_2S) concentrations. The H_2S can damage concrete and other structural components in the building. A detailed condition assessment can report on the suitability of the structure to be re-used. Other considerations, involve structural and geotechnical assessments for the existing infrastructure and the ability to meet current applicable building codes. Current codes require that wastewater treatment plants are "post-disaster" and are operable in a disastrous event, such as an earthquake.

Following screening, the wastewater is conveyed via an aerated channel to three pre-aeration grit tanks that are located in-line with the three primary clarifier tanks. Each pre-aeration grit tank is dedicated to the downstream primary clarifier unit it services. The aerated grit tanks provide some grit removal from the influent wastewater, the remaining grit is removed in the primary clarifier tanks. The pre-aeration grit tanks are covered to provide some odour control. The tanks are 3.7 meters long by 6.1 meters wide and have a depth of 3.6 meters (ISL 2016).

The grit slurry is removed from the pre-aeration grit tanks through two (one duty/one standby) dedicated grit pumps that pump the slurry to two grit classifiers. The grit from the classifiers is conveyed to two storage bins and then transported offsite for disposal. The primary clarifier sludge pump also pumps through a grit classifier to remove grit, before being conveyed to the gravity thickener (ISL 2016).

The pre-aeration grit tank system capacity assessment for grit removal from ISL (2016) is shown in Table 7. The assessment is based on the size of the tanks compared to textbook recommended sizes, as there is no grit data available to evaluate performance.

Table 7: Pre-Aeration Grit Tank Assessment

DESCRIPTION	PRE-AERATION GRIT TANKS	RECOMMENDED DESIGN VALUES
No. Units	3	-
Length (m)	3.7	-
Width (m)	6.1	-
Depth (m)	3.6	-
Volume (m^3)	81.3	-
L : W Ratio	0.61 : 1	3 – 5 : 1
2020 PHF Retention Time (Minutes)	1.8	2 – 5
W : D Ratio	1.7	0.8 – 1 : 1

The current pre-aeration grit tanks do not meet recommended design values for retention time, length to width ratios, and width to depth ratios. Based on the ISL (2016) assessment and the current configuration of the CVWPCC, it appears the grit removal system is atypical of a sewage treatment plant. The grit removal is accomplished partially through these pre-aeration grit tanks, and partially through the primary clarifiers. However, it is important to note that these two processes appear to achieve thorough grit removal, as the operators do not report any grit in processes downstream of the primary clarifiers.

Grit is harsh on pumps, pipes, diffuser membranes, and various other components within a treatment plant. Since it seems that most of the grit is removed in the first two unit-processes, the main concern would be the effects of grit on the primary sludge pumping. If anaerobic digestion of waste solids is added in the future, grit accumulation in the digesters from the primary sludge may be a concern.

The ISL (2016) report recommended including a vortex style grit removal system that would be designed to remove 95% of grit down to 140 mesh size (105 microns). This would allow the CVRD to decommission the grit classifier equipment that is used to classify grit in the primary sludge and to decommission or re-purpose the pre-aeration grit tanks ahead of the primary clarifiers. Additionally, as will be discussed in the next section, the addition of a proper grit removal system will allow the primary clarifiers to be operated normally and allow the sludge to thicken in the primary clarifiers and can then bypass the gravity thickeners (currently, the primary sludge is only reported to be 0.1% instead of 3% - 5%) (ISL 2016).

It is important to note that the ISL (2016) report included constructing a grit tank now and in the future (beyond 2040) when a new headworks building would be required with a new grit removal system. It is assumed this is required due to the plants hydraulic profile and the concept was to include two headworks systems (the current existing one and one constructed beyond 2040).

PRIMARY CLARIFIERS

The CVWPCC is equipped with three primary clarifier tanks from the original construction in 1982. Each tank is directly downstream from its own grit tank, as described in the previous section. Each primary clarifier is 32.65 meters long by 6.1 meters wide by 3.6 meters deep. The 1982 record drawings show that the average water depth in the primary clarifier is approximately 2.8 meters.

According to the 1982 record drawings, space was allocated to the west of the primary clarifiers, and provisions were set-in the original design for expansion of the plant with two additional primary clarifiers. This expansion has not yet occurred.

Sludge is removed from the primary clarifiers by the longitudinal chain and flight clarifier mechanism that scrapes sludge into a hopper at the inlet side of the clarifier tank. From here, the sludge is pumped by two primary sludge pumps (one duty/one standby). There is a common primary clarifier sludge collection header and the branch to each clarifier is fit with a control valve to alternate from which clarifier the sludge is drawn. The primary sludge is pumped by the two primary sludge pumps to a dedicated grit classifier (one duty/one standby) where grit is removed and washed. For the grit classifiers to work properly, the primary sludge needs be less than 0.5% total solids (ISL 2016). ISL (2016) reported that currently the primary sludge is withdrawn from the primary clarifiers at approximately 0.1%, much less than what would typically be seen in primary clarifiers (3% to 5%). De-gritted Primary sludge is then sent by gravity to the gravity sludge thickeners for thickening.

The primary clarifiers were evaluated based on surface overflow rate and detention time based on the updated flows from Table 3. The capacity assessment is shown in Table 8 for all units in service receiving 100% of the flow (column A), two out of three units receiving 100% of the flow (column B), two out of three units receiving 50% of the flow (column C). The MWR requirement is that the primary clarifiers need to treat 50% of the flow with the largest unit out of service. Note a red value indicates the value is outside of the recommended range (typical for all design tables).

Table 8: Primary Clarifier Capacity Assessment

DESCRIPTION	A. PRIMARY CLARIFIERS (ALL FLOW)	B. PRIMARY CLARIFIERS (ALL FLOW – ONE UNIT OUT OF SERVICE	C. PRIMARY CLARIFIERS (50% OF FLOW – ONE UNIT OUT OF SERVICE)	RECOMMENDED VALUES
No. of units	3	2	2	-
LxWxH (m)	32.65 x 6.10 x 2.80	-	-	-
Surface Area (m^2)	597	398	398	-
Volume (m^3)	1,673	1,115	1,115	-
2040 ADF (m^3/d)	20,758	20,758	10,379	-
2040 AWWF (m^3/d)	29,233	29,233	14,617	-
2040 2 x ADWF (m^3/d)	34,419	34,419	17,210	
2040 PHF (m^3/d)	62,274	62,274	31,137	-
2040 ADF Surface Overflow Rate ($m^3/m^2/d$)	35	52	26	30 – 50
2040 AWWF Surface Overflow Rate ($m^3/m^2/d$)	49	73	37	30 – 50
2040 2 x ADWF Surface Overflow Rate	58	86	43	30 – 50
2040 PHF Surface Overflow Rate ($m^3/m^2/d$)	104	156	78	80 – 120
2040 ADF Detention Time (hr)	1.9	1.3	2.6	1.5 – 2.5
2040 AWWF Detention Time (hr)	1.4	0.9	1.8	1.5 – 2.5

DESCRIPTION	A. PRIMARY CLARIFIERS (ALL FLOW)	B. PRIMARY CLARIFIERS (ALL FLOW – ONE UNIT OUT OF SERVICE	C. PRIMARY CLARIFIERS (50% OF FLOW – ONE UNIT OUT OF SERVICE)	RECOMMENDED VALUES
2040 2 x ADWF Detention Time (hr)	1.2	0.8	1.6	1.5 – 2.5
2040 PHF Detention Time (hr)	0.6	0.4	0.9	1.5 – 2.5

As can be seen in Table 8, there appears to be capacity in the existing primary clarifiers to reach the 2040 design horizon with all units in service (column A) and to meet the MWR requirement (column C). However, as flows increase, the removal efficiency of solids (and BOD_5) from primary treatment will also decrease, which will increase solids and organic loads to secondary treatment.

It is important to note that there are currently no mechanisms in place to assure even flow splitting between any of the unit processes (primary clarifiers, aeration basins, and secondary clarifiers). This can hinder performance if issues cause any one process to receive flow unequally. There does not appear to be enough hydraulic head available in the hydraulic profile to add flow splitting weir boxes for the primary clarifiers.

A condition assessment of the existing clarifiers structural condition should also be completed to assess their anticipated lifespan and any repairs that are required. Primary clarifiers can also have high exposure to H_2S leading to corrosion in metallic and concrete elements, shortening their lifespan.

As will be discussed later, there is an equalization tank currently being constructed directly adjacent to the primary clarifiers and the aeration basins. This equalization tank reduces the available space for secondary treatment expansion.

AERATION BASINS – ACTIVATED SLUDGE PROCESS

The wastewater flows out of the primary clarifiers and into a common channel for conveyance to the three aeration basins. The original construction installed two aeration basins, with a third added in 2008. The 2016 ISL report noted a plug flow conventional activated sludge (CAS) treatment system.

The first two aeration basins installed are 1,460 m^3 and the third aeration basin installed as part of a 2008 expansion is 1,539 m^3 (ISL 2016). The aeration basins are fit with fine bubble diffusers that are supplied from five (four duty/one standby) centrifugal blowers designed for 55 m^3/min at 48 kPa (ISL 2016). The blowers are located in the process building adjacent to the aeration tanks and primary clarifiers. Four blowers were installed in the 1982 construction with a fifth blower added during the 2008 expansion (ISL 2016).

The capacity of the aeration basins was reviewed based on the updated flow and load information to determine organic and hydraulic loading rates to the process, and these were compared with textbook recommended design values. The plug flow regime described in ISL 2016 is an important characteristic of the system as it effects the recommended organic loading rates that are used for design and operation of these processes. For the purposes of the activated sludge aeration basin capacity review we have assumed the maximum month BOD_5 loading to secondary treatment is applicable. We have also assumed that under maximum month conditions, there would be 35% BOD_5 removal in the primary clarifiers and 55% TSS removal. We note that further composite sampling testing would confirm this loading and the peaking factors used.

The capacity review for the conventional activated sludge system is summarized in Table 9. Table 9 shows the capacity review with all units receiving 100% of the load (column A), two out of three units receiving 100% of the load (column B), and two out of three units receiving 75% of the load (column C). The MWR redundancy requirement for aeration basins is that the treatment capacity shall be designed for 75% of the design flow (load) with the largest unit out of service.

Table 9: Aeration Basin Capacity Review All Tanks in Service Receiving 100% of Load

DESCRIPTION	**A. AERATION BASINS (ALL TANKS IN SERVICE 100% OF FLOW/LOAD)**	**B. AERATION BASINS (100% OF FLOW/LOAD ONE UNIT OUT OF SERVICE**	**C. AERATION BASINS (75% OF FLOW/LOAD ONE UNIT OUT OF SERVICE**	**RECOMMENDED VALUES**
No. Units	3	2	2	-
Total Volume (m^3)	5,998	4,459	4,459	
2040 Average BOD_5 Load to Aeration Basins (kg/d)[1]	2,902	2,902	2,177	-
2040 Max Month BOD_5 Load to Aeration Basins (kg/d)[2]	4,951	4,951	3,173	-
2040 ADF (m^3/d)	20,758	20,758	15,569	-
2040 AWWF (m^3/d)	29,233	29,233	21,925	-
2040 2 x ADWF	34,419	34,419	25,815	-
2040 Average Organic Loading Rate (kg/m^3*d)	0.65	0.99	0.75	0.3 – 0.7

DESCRIPTION	A. AERATION BASINS (ALL TANKS IN SERVICE 100% OF FLOW/LOAD)	B. AERATION BASINS (100% OF FLOW/LOAD ONE UNIT OUT OF SERVICE	C. AERATION BASINS (75% OF FLOW/LOAD ONE UNIT OUT OF SERVICE	RECOMMENDED VALUES
2040 Max Month Organic Loading Rate (kg/m^3*d)	0.92	1.70	1.27	0.3 – 0.7
2040 ADF Hydraulic Retention Time (hr)	5.2	3.4	4.5	4 – 8
2040 AWWF Hydraulic Retention Time (hr)	3.7	2.4	3.2	4 – 8
2040 2 x ADWF Hydraulic Retention Time (hr)	3.1	2.0	2.7	4 – 8

[1]Assumes 40% removal of BOD_5 during average loading in primary clarifiers
[2]Assumes 35% removal of BOD_5 during max month loading in primary clarifier.

As can be seen in Table 9, the organic loading capacity is pushing the limits of recommended design values under the maximum month and average load condition with all units in service (column A). The max month organic loading in 2040 with the largest unit out of service is 80% higher (1.27 vs. 0.7 kg BOD_5/m^3*d) than the maximum recommended value, indicating the current system will not meet the redundancy requirements in 2040 (column C). We have also included column B, which indicates what the loading to the aeration basins would be if an aeration basin ever needed to be taken offline during the max load and/or flow conditions. As can be seen in this scenario (column C), the aeration basins would be overloaded in 2040 and 2020 as well. If the basins are overloaded, as shown, this would indicate difficulty for the CVWPCC to meet effluent criteria.

The 2020 loading under max month condition with all units in service is at 0.95 kg BOD_5/m^3*d, also exceeding the recommended maximum design value. This indicates the need to confirm the requirement for increased activated sludge process volume; and potential construction of additional aeration basin volume as soon as possible to be prepared for current and future loads.

It is important to note that the recommended design values, taken from the Metcalf & Eddy (2014) textbook Table 8-19, are a general guideline or “rule-of-thumb” for plug flow conventional activated sludge (CAS) systems. We have worked with other municipalities where the operating organic loading rate is higher than recommended design ranges, and these facilities still meet target effluent criteria. However, these values do provide a good indication of where the normal operating range of the plant should be for optimum performance, without detailed modelling. We also found after a brief review of the effluent samples where the concentration of BOD_5 exceeded the permitted value 25 times in four years in bi-weekly grab (assumed) samples. Additionally, the

ISL (2016) report's review of plant effluent data indicated several occurrences where effluent BOD_5 exceeded the plant's permitted value of 45 mg/L between 2011 – 2015, indicating that the activated sludge process volume may be a concern at today's flows and loads.

As discussed previously, the textbook recommended organic loading rates selected are for a plug flow CAS process. The existing infrastructure could potentially be modified to a step-feed CAS system or a completely mixed activated sludge system (CMAS), where the organic loading rate is more forgiving and can be increased. For example, the range on a step-feed CAS system is 0.7 – 1.0 kg BOD_5/m^3*d and for a CMAS system it is 0.3 – 1.6 kg BOD_5/m^3*d (Metcalf & Eddy 2014). The feasibility of modifying the existing conventional activated sludge process to increase the allowable capacity with the existing tank volume would require a more detailed assessment and analysis. Regardless, we have assumed additional aeration basin volume is required for the plant. This is consistent with the ISL (2016) report which indicated additional process volume would be required by around 2024.We have assumed a volume of approximately 2,500 m^3 is required, this will provide the CVWPCC with enough volume to achieve 0.7 kg BOD_5/m^3*d in 2040 with all four tanks running. We have assumed the additional process volume could be achieved by making the new aeration tank the same footprint size, but with a deeper side water depth (6 meters vs. 3.6 meters).

With the construction of an equalization tank directly to the east of the aeration basins, the available space for secondary treatment expansion is limited.

The blower capacity was also reviewed for the aeration basins to confirm capacity of the existing blowers. According to the ISL (2016) report there are 5 blowers (4 duty/1 standby) that have a rated capacity of 55 m^3/min at 20 degrees Celsius and 48 kPa. The estimated 2040 blower capacity is outlined in Table 10. We have also included the estimated aeration demand for nitrification of ammonia to nitrate in the process tanks. While nitrification is not required to meet the effluent criteria, it will generally occur in aeration tanks during the summer months and provide an additional demand on the blowers.

Table 10: Estimated Blower Capacity

DESCRIPTION	BLOWER CAPACITY
2040 Max Month BOD_5 Load in Primary Effluent (kg/d)	4,951
2040 Estimated Total Airflow for BOD Removal (m^3/min)[1]	142
2040 Max Month TKN Load in Primary Effluent (kg/d)	562
2040 Estimate Total Airflow for TKN oxidation (m^3/min)[2]	75
2040 Estimated Total Airflow (m^3/min)	217

[1]Estimated airflow based on AOR 4,951 kg O_2/d, alpha factor of 0.6, beta of 0.95, summer temperature of 22 degrees Celsius, design dissolved oxygen concentration of 2 mg/L, and standard oxygen transfer efficiency of 20%.
[2]Estimated airflow based on AOR 2,585 kg O_2/d, alpha factor of 0.6, beta of 0.95, summer temperature of 22 degrees Celsius, design dissolved oxygen concentration of 2 mg/L, and standard oxygen transfer efficiency of 20%.
[3]Assumes 35% max month TKN removal in primary clarifiers.

It is estimated that the current airflow demand for 2040 maximum month conditions can be handled by the existing blowers. The condition of the existing blowers is unknown, however the blowers from the original design are nearly 40 years old and will be nearly 60 by 2040 and seemingly near the end of their serviceable life. However, we have assumed that the existing blowers can be used until 2040 to service all four aeration basins.

SECONDARY CLARIFIERS

Similar to the aeration basins, two secondary clarifiers were installed in the original 1982 construction with a third clarifier being added as part of the 2008 expansion. The original clarifiers are 23.17 meters in diameter and are 3.13 meters deep (ISL 2016). The third clarifier is 23.1 meters in diameter with a deeper depth of 5.0 meters (ISL 2016). It is noteworthy that an important aspect of secondary clarifier design is the tank side water depth (SWD). The SWD of the original clarifiers (3.13 meters) is considered shallow for this size of clarifier, hence likely why the third clarifier was constructed to a deeper depth.

The liquid that is separated from the solids by gravity settling in these clarifiers is then directed to the effluent outfall. The settled solids are directed through the return activated sludge (RAS) pumping system. There are three RAS pumps (three duty), one dedicated for each clarifier, located in the basement of the mechanical building (below the blowers).

There are two (one duty/one standby) waste activated sludge (WAS) pumps that remove sludge from the RAS piping to the solids processing facilities. According to the ISL report, the WAS pumps can remove sludge from the RAS line or directly from the aeration basins (ISL 2016).

The secondary clarifier capacity was reviewed with the updated flow information to determine surface overflow rates (SOR) and solids loading rates (SLR) to the clarifier. These loading factors are compared with textbook recommended design values to gauge the capacity of the existing system and when an upgrade may be required.

The capacity assessment is shown in Table 11 for the all the clarifiers receiving 100% of the flow, two of three clarifiers receiving 100% of the flow, and two of three clarifiers receiving 75% of the 2040 flow with one unit out of service (MWR requirement).

Table 11: Secondary Clarifier Capacity Assessment

DESCRIPTION	A. SECONDARY CLARIFIERS ALL UNITS IN SERVICE 100% OF FLOW	B. SECONDARY CLARIFIERS ONE UNIT OUT OF SERVICE 100% OF FLOW	C. SECONDARY CLARIFIERS ONE UNIT OUT OF SERVICE 75% OF FLOW	RECOMMENDED VALUES
No. Units	3	2	2	-
Total Surface Area (m^2)	1,262	841	841	
2040 ADF (m^3/d)	20,758	20,758	15,569	-
2040 AWWF (m^3/d)	29,233	29,233	21,925	-
2040 2 x ADWF (m^3/d)	34,419	34,419	25,815	-
2040 PHF (m^3/d)	62,274	62,274	46,706	-
2040 RAS Flow/2040 ADF	1.0	1.0	1.0	-
2040 MLSS Concentration (mg/L)	2,500	2,500	2,500	-
2040 Average Surface Overflow Rate (m^3/m^2*d)	16	25	19	16 – 28
2040 AWWF Surface Overflow Rate (m^3/m^2*d)	23	35	26	16 – 28
2040 2 x ADWF Surface Overflow rate (m^3/m^2*d)	27	41	31	16 – 28
2040 PHF Surface Overflow Rate (m^3/m^2*d)	49	74	56	40 – 64
2040 Average Solids Loading Rate (kg/m^2/hr)	3.4	5.1	3.9	4 – 6

DESCRIPTION	A. SECONDARY CLARIFIERS ALL UNITS IN SERVICE 100% OF FLOW	B. SECONDARY CLARIFIERS ONE UNIT OUT OF SERVICE 100% OF FLOW	C. SECONDARY CLARIFIERS ONE UNIT OUT OF SERVICE 75% OF FLOW	RECOMMENDED VALUES
2040 AWWF Solids Loading Rate (kg/m^2/hr)	4.1	6.2	4.6	4 – 6
2040 2 x ADWF Solids Loading Rate (kg/m^2/hr)	4.6	6.8	5.1	4 – 6
2040 PHF Solids Loading Rate (kg/m^2/hr)	6.9	10.3	7.7	8

The capacity assessment shown in seems to indicate there is sufficient secondary clarifier capacity to reach 2040 and still meet the MWR redundancy requirements (column C). However, at higher sustained flows with a clarifier out of service (column B), there is risk that the clarifier performance will be limited and cause effluent criteria to be exceeded for TSS and BOD5. In general, the solids loading rate criteria governs with clarifier design, and it can be seen in the above tables that the solids loading for the clarifiers are near the high-end values as recommended by Metcalf & Eddy (2014) with one unit out of service. This indicates that additional secondary clarifier capacity is required.

Additionally, the ISL (2016) report noted that the sludge settleability at the CVWPCC can be problematic. For example, the average sludge volume index (SVI) – a measure of sludge settleability – was 261 mL/g in 2015. For reference, 150 mL/g is considered average and 100 mL/g or less is considered good settling sludge. The CVWPCC poor settling sludge, at least in 2015, may suggest a need to modify the maximum allowable solids loading and surface overflow design range recommended values in the clarifiers. Additionally, in a review of the ISL (2016) report by Dr. Bill Oldham, he commented that the shallow depth of two of the secondary clarifiers may reduce their capacity by 20% to 25%, this reduction in capacity is not shown in Table 11.

Review of the effluent TSS concentration from the ISL (2016) report seem to indicate that the CVWPCC regularly meets the permissible effluent TSS concentration regulated by the MWR and the WSER. Based on data from 2011 to 2015 shown in ISL (2016), the monthly average effluent TSS concentration is below 20 mg/L, with occasional daily concentrations spiking as high as 35 mg/L, still below the permitted maximum daily effluent TSS concentration of 60 mg/L. However, a brief review of the 2013 – 2017 effluent data indicated 17 times where the concentration exceeded the permitted value of 60 mg/L based on 3 – 4 grab (assumed) samples taken per week.

Theoretically, the loading rates in 2040 are within normal operating points, although nearing the high-side during high flows. However, if poor settling sludge continues (e.g. SVI greater than 200 mL/g), a fourth clarifier would be required. Additionally, the 20 – 25% capacity reduction by Dr.

Oldham has not been factored into Table 11. Lack of a controlled flow splitting system may cause issues with the clarifiers as well. The assessment in Table 11 assumes an equal flow split.

Considering the possible less than adequate performance of the shallower clarifiers, we have included a fourth secondary clarifier of equal diameter (23.1 m) and depth (5.0 m) as clarifier #3, in the base case scenario for the 2040 design flows. This is consistent with the recommendations for the ISL 2016 report and the review completed by Dr. Oldham, where he recommended constructing a new secondary clarifier as soon as possible. We have assumed that the RAS pumps would be installed in the basement of the new headworks building.

GRAVITY THICKENERS

The circular gravity thickeners were originally constructed in 1982 to thicken combined primary sludge (PS) and WAS; however, when the dissolved air flotation (DAF) units were added this practice was abandoned. Currently, the gravity thickeners, which operate similarly to secondary clarifiers, only thicken PS that comes from the primary clarifier grit classifiers. There are two thickeners each with a diameter of 7.32 meters and depth of 3.05 meters.

The supernatant from the thickeners is returned to the liquid process and the thickened primary sludge (TPS) is directed to a thickened sludge storage tank where it is combined with the DAF thickened waste activated sludge (TWAS).

Like secondary clarifiers, thickeners are assessed on their surface overflow rate and solids loading rates. These rates, along with textbook recommended design values, are summarized in Table 12 to assess the capacity of the thickeners. There are no MWR redundancy requirements for gravity thickeners. The overflow rate is based on the primary sludge pumping capacity, which is reported as 33 L/s in ISL (2016).

Table 12: Gravity Thickeners Capacity Assessment

DESCRIPTION	GRAVITY THICKENERS (BOTH UNITS ONLINE)	GRAVITY THICKENERS (ONLY ONE UNIT ONLINE)	RECOMMENDED VALUES
No. Units	2	1	-
Total Surface Area (m^2)	84.2	42.1	-
2040 Average PS to Thickener (kg/d)[1]	3,929	3,929	-
2040 Max Month PS to Thickener (kg/d)[2]	4,522	4,522	-
PS Flow Rate (m^3/d (L/s))	2,850 (33)	2,850 (33)	-
Average Solids Loading Rate (kg/m^2*d)	47	93	100 – 150

DESCRIPTION	GRAVITY THICKENERS (BOTH UNITS ONLINE)	GRAVITY THICKENERS (ONLY ONE UNIT ONLINE)	RECOMMENDED VALUES
Max Month Solids Loading Rate (kg/m^2*d)	54	107	100 – 150
Surface Overflow Rate (m^3/m^2*d)	34	68	14.5 – 31

[1]Based on 65% TSS removal in Primary Clarifiers
[2]Based on 55% TSS removal in Primary Clarifiers

As can be in Table 12, the gravity thickeners are hydraulically overloaded due to a thin primary sludge. According to ISL (2016) the primary sludge concentration is less than 0.1%, which is the reason for the low solids loading but the high surface overflow rate. However, ISL (2016) reported that operations staff do not have any issues with the gravity thickeners and they perform well under the current loading conditions. The thin primary sludge is required for the grit classifier equipment to work ahead of the gravity thickeners. The very thin primary sludge is abnormal, typically primary sludge concentration should be 3% to 5% total solids. If proper grit removal equipment is installed that removes the entirety of the grit, then the primary clarifiers can be operated normally with a thicker primary sludge concentration pumped from the clarifiers. In this scenario, it would be expected that the gravity thickeners could be decommissioned, as the primary clarifiers would adequately thicken the sludge themselves; the primary sludge could be pumped directly to the thickened sludge storage tank where it is combined with the WAS.

DISSOLVED AIR FLOTATION

The dissolved air flotation (DAF) units were installed during the 2000's and the practice of co-thickening WAS and PS in the gravity thickeners was abandoned. The WAS from the secondary clarifiers is pumped to two DAF units through the WAS pumps. The DAF units are 9 meters long by 2.44 meters wide and 2.1 meters deep. Under normal operation, there is a single DAF unit operating and one-unit acts as a standby unit (ISL 2016). The average capacity of each DAF unit is 46 m^3/hr (ISL 2016). It is assumed that the WAS is dosed with a polymer system prior to entering the DAF to assist with flocculating and thickening the sludge, however information regarding the polymer system was not available at the time of this memorandum.

After the DAF, the TWAS is conveyed by gravity to a TWAS holding tank. From this tank it is pumped to the thickened primary sludge (TPS) holding tank where it is combined. From here, the combined TWAS and TPS is sent to the centrifuges for dewatering and then eventually trucked to composting. The liquid from the DAF unit is sent back to the liquid train of the treatment plant.

The DAF units' capacity that is reported in ISL (2016) is shown in Table 13.

Table 13: DAF Unit Process Description

DESCRIPTION	DAF CAPACITY (2 UNITS ONLINE)	DAF CAPACITY (1 UNIT ONLINE)
No. of Units	2	1
Rated Capacity (m^3/hr)[1]	130	65
Average Capacity (m^3/hr)[1]	92	46
2015 Average Daily WAS (m^3/d)[1]	695	695
2015 Maximum Month Daily WAS (m^3/d)[1]	884	884
Total Average Operating Time (hr/d)[1]	7.5	15
2040 Average Daily WAS (m^3/d)[2]	1,000	1,000
2040 Total Average Operating Time – (hr/d)	11	22

[1]From ISL (2016) Report
[2]Based on an assumed sludge yield of 1 g TS/g BOD_5 removed and secondary sludge concentration of 0.5%.

As shown in Table 13, the DAF units have ample capacity to handle anticipated sludge flows until 2040, even with just one unit in operation. However, by 2040 the DAF units will be 30+ years old and may be due for major refurbishment or replacement; this work can be part of a larger solids handling upgrade that may be required after an assessment has been completed. A condition assessment on the DAF's should be completed to review any structural and mechanical upgrades that may be required.

CENTRIFUGES

The plant has two centrifuges rated for a capacity of 36 m^3/hr that dewater the blended thickened sludge (ISL 2016). According to the ISL report, the blended sludge ranges in concentration from 3.0% to 3.5% solids, and from this the centrifuges produce a 25% cake product (ISL 2016). The thickened sludge is pumped to the centrifuges, however information regarding the pump capacity or age was not readily available. Information regarding the polymer system was also not available at the time of this memorandum. The dewatered sludge is transferred onto screw conveyors that load the sludge onto a truck for transport to the compost facility. The centrate produced from the centrifuges is sent to an onsite septage receiving tank where it is blended with septage before being pumped to the liquid treatment train.

The centrifuges capacity as reported in ISL (2016) is shown in Table 14.

Table 14: Centrifuge Rated Capacity

DESCRIPTION	CAPACITY (ONE CENTRIFUGE)	CAPACITY (TWO CENTRIFUGES)
No. of Units	1	2
Rated Capacity, each (m^3/hr)[1]	36	36
2015 Average TPS/TWAS (m^3/d)[1]	185	185
2015 Average Operating Hours per Week (hr)[1]	36	18
2040 Average TPS/TWAS (m^3/d)[2]	275 (~120 TPS / 155 TWAS)	275 (~120 TPS / 155 TWAS)
2040 Average Operating Hours per Week (hr)	54	27
2040 Max Month TPS/TWAS (m^3/d)[2]	405 (~140 TPS / 265 TWAS)	405 (~140 TPS / 265 TWAS)
2040 Max Month Operating Hours per Week (hr)	79	40

[1]From ISL (2016) Report

[2]Based on an total primary and secondary sludge going to centrifuges and assumed thickened sludge concentration of 3.25%.

The capacity shown in Table 14 indicates the ability of the centrifuges to handle increased sludge coming from the plant until 2040. This is a reasonable operating time for the centrifuges, especially if both run during the week. Even under maximum month loading conditions the total operating time would equal approximately 79 hours per week, or 40 hours per week for each centrifuge.

The centrifuges can generally handle the additional solids; however, a detailed condition assessment should be completed to see if any restorative work is required for the centrifuges to carry them to 2040. By 2040 the centrifuges will be 30+ years old and may require replacement or significant repairs. At this time, the performance and capacity should be reviewed, to advise on potential upgrade options.

Additionally, the thickened sludge is pumped from 2 x 330 m^3 sludge holding tanks. According to the ISL (2016) report the sludge is blended in one tank and then stored in the second tank for the centrifuges feed pumps to pump from. The single 330 m^3 tank provides almost two days of storage based on 2015 TPS/TWAS average flow numbers (185 m^3/d). By 2040 this storage will be reduced to almost one day of storage, meaning the centrifuges will need to run every day to empty the storage tank. Based on the current capacity of the centrifuges, it will take approximately 10 hours to empty the storage tank.

If blending is still desired in a separate tank, then a third storage tank may be required for 2040 to provide flexibility to the CVWPCC solids handling system. We have included an additional sludge storage tank as part of the base case.

MEMO

TO: CVRD LWMP TACPAC Committee

CC: Kris La Rose, P.Eng., CVRD
Zoe Berkey, EIT, CVRD
Paul Nash

FROM: Aline Bennett, P.Eng., Al Gibb, PhD, P.Eng. WSP

SUBJECT: **CVRD LWMP Stage 2 - Resource Recovery Options**

DATE: **February 20, 2020**

OVERVIEW

In recent years, there has been an increasing emphasis on recovery of resources that can be extracted from the wastewater stream or that can be produced during treatment. In British Columbia, the success of applications for grant funding assistance from senior government for design and construction of wastewater conveyance and treatment facilities often depend in part upon inclusion of resource recovery, which may include the following:

- use of reclaimed effluent for in-plant use, irrigation or other purposes;
- installation of heat exchangers in the wastewater stream for heating and cooling of buildings;
- production of biogas (methane) through treatment of waste solids, which can be used in combustion facilities designed for cogeneration of electrical power and heat or in boilers for hot water heating systems;
- use of digested waste solids as a natural solid conditioner/fertilizer, and/or use of waste solids as a feedstock to produce compost for household or commercial use;
- production of mineral pellets rich in nitrogen and phosphorus (struvite) for use as fertilizer; and
- use of hydroelectric turbines to generate electrical power from the outfall discharge.

The feasibility of the various resource recovery option must be carefully evaluated. The design and installation of resource recovery facilities can add substantially to the capital and operating costs of wastewater treatment facilities. If there are no potential customers for the recovered resources or if those customers are located far from the recovery location, investment in resource recovery may be inadvisable. Each situation must be evaluated on its own merits, beginning with identification of potential uses and users of the reclaimed resources. Brief discussions of each resource recovery option in the context of the CVRD LWMP are presented below.

RECLAIMED WATER

Some of the wastewater treatment options (namely Options 3 and 4) are designed to produce effluent quality that meets the requirements for use of reclaimed water. For Option 2, if one or more uses for reclaimed water are identified, the appropriate amount of secondary treated effluent can be diverted to a dedicated filtration and disinfection system to produce reclaimed water. As set

210 – 889 Harbourside Drive
North Vancouver, BC, Canada V7P 3S1
wsp.com

out in the Municipal Wastewater Regulation, it is required to maintain a chlorine residual in the reclaimed water at the point of use unless the addition of chlorine will detrimentally impact flora or fauna, or at the point of use fecal coliforms remain below levels set in municipal effluent quality requirements for reclaimed water, and users are adequately informed regarding appropriate use of the reclaimed water. Disinfection of reclaimed water is normally accomplished through the addition of sodium hypochlorite (bleach).

Production of reclaimed water adds to the cost of treatment, so it is important to identify the potential market for this resource. It is normally cost effective to use a portion of the treated effluent for non-potable applications within the treatment plant itself (e.g., for equipment sprays, washdown water, landscape irrigation, etc.). This typically represents a relatively small portion of the total wastewater flow, but it does offset use of potable water at the plant. A small amount of reclaimed effluent is currently used at the CVWPCC for washdown in enclosed areas. Opportunities for expanding use of reclaimed water within the plant should be considered during design of future upgrades.

Offsite applications may represent opportunities for use of larger amounts of reclaimed water (irrigation, industrial use, or stream and wetlands augmentation). The economics of offsite use depend heavily on the distance from the reclaimed water production facility to the user. Other factors include the seasonal pattern of demand for water, the cost of alternative water sources, and the water quality requirements of the potential user.

In cases where a significant potential user of reclaimed water has been identified but the distance between the main wastewater treatment plant and the user makes the project unfeasible for economic reasons, it may be possible to locate a relatively small water reclamation plant near the user and divert some of the untreated wastewater to that location for treatment and use. The feasibility of this will depend on the amount of reclaimed water to be produced and other local factors.

A summary of the results regarding potential uses for reclaimed water from the February 2019 TAC/PAC meeting is shown in Table 1. As shown, a large number of potential locations for effluent reuse were identified. Detailed studies would be required for each potential location, to assess water quality requirements, capital and operating costs for pumping and conveyance of reclaimed effluent from the CVWPCC to the site, revenue potential to offset costs, additional treatment requirements, environmental impacts, and other site-specific factors. The conveyance distance is a very important factor, since capital and operating costs rise significantly as distance increases. This will be the primary limitation for many of these options.

Future upgrades at the CVWPCC could be designed with the potential for reclaimed water production in mind, so that additional levels of treatment can be added if and when users are identified, without costly reconfigurations of the treatment facilities.

Table 1 - Summary of Potential Reclaimed Water Uses - from February 2019 TACPAC Meeting

Use (at each site)	**Water Quality Requirement**	**Volume (m3/day, summer)**	**On Sewer?**	**Nearby Localities (0-4km)**					**Farther Localities (4-8km)**						**Remote Localities (>8km)**						
Per site		Greater than X		CVWPCC	Lazo Beach Area	Queen's Ditch farm area	Airport	Comox (Town)	KFN	Estuary Farm area	Courtenay (East)	Crown Isle Resort	Anderton Rd (South of Ryan)	Little River	Courtenay (West)	Anderton Rd (North of Ryan)	Portuguese Creek Valley	Royston	Union Bay	Denman Island	Texada Island
Stream augmentation	GEP/IPR	10,000	N			Y											Y		Y		Y
Agriculture -spray irrigation, field crops	GEP	100	N			Y				Y			Y	Y		Y	Y	Y	Y	Y	
Concrete mixing	GEP	100	N												Y						
Airport (all outdoor uses)	GEP	100	N				Y														
Golf Course (each)	GEP	100	N				Y	Y				Y					Y				
Wetland augmentation	GEP/IPR	100	N		Y																
Agriculture - spray irrigation, forage	MEP	100	N			Y				Y			Y	Y		Y	Y	Y	Y	Y	
Mining	MEP	100	N																		Y
Irrigation playing field/school	GEP	10	N					Y			Y	Y	Y		Y			Y	Y		
Airport (all indoor uses)	GEP	10	Y				Y														
Gravel washing	GEP	10	N								Y				Y						
Dust Control	GEP	10	N								Y				Y						
Car Wash	GEP	10	Y					?			Y				Y						
Transit bus wash	GEP	10	Y												Y						
Comox marina (boat washing)	GEP	10	N					Y													
Irrigation - municipal park	GEP	10	N					Y	Y		Y	Y	Y		Y						
Irrigation - cemetery	GEP	10	N					?			?				?						
BC Ferries Little River	GEP	10	N								Y				Y						
Irrigation roadside	GEP	10	N					Y	Y		Y	Y			Y						
HMCS Quadra	GEP	10	Y					Y													
Tree Farm (Xmas, timber)	MEP	10	N														Y	Y		Y	Y
Commercial nursery, greenhouse	MEP/GEP	10	N								Y		Y		Y	Y	Y				
Agriculture- subsurface drip irrig.	MEP	10	N										Y	Y		Y	Y	Y	Y		
CVWPCC	MEP/GEP	10	Y	Y																	
Industrial process	MEP/GEP/IPR	10	Y								Y				Y						
Commercial laundry	MEP/IPR	10	Y								Y				Y						
Public washrooms	GEP	1	Y					Y	Y		Y	Y		Y	Y				Y		
Rural residential	IPR	1	N		Y				Y				Y	Y		Y	Y	Y	Y	Y	
Flood irrigation of cranberries	Not allowed	-																			
Approx Total Water (m3/day, summer)				**10**	**100**	**1000**	**100**	**100**	**10**	**1000**	**100**	**100**	**100**	**1000**	**1000**	**1000**	**10,000**	**1000**	**1000**	**100**	**100**

HEAT RECOVERY

Extraction of heat from the wastewater stream at pumping stations and treatment facilities for space heating of buildings is becoming more common (the same system can also be used for cooling in summer). As with reclaimed water, heat recovery for use onsite at wastewater treatment facilities is generally the most feasible from a cost standpoint. Use of this type of system can be considered for incorporation into future upgrades at the CVWPCC.

If a potential user or users of heat is located near the pumping station or wastewater treatment plant, it may be feasible to expand the system to export heat to a nearby specific user (an example of such a system is in place at the Saanich Peninsula wastewater treatment plant, where heat is extracted from the effluent for use at an adjacent municipal swimming pool). In some cases, if there is high density development near the treatment plant, it may be feasible to install a District Heating System that circulates recovered heat through a heating loop for use by multiple customers. Due to the cost involved in installing a District Heating System, it is preferred if there is a year-round demand for the recovered heat (e.g., swimming pool, commercial laundry).

A summary of the results regarding potential users of reclaimed heat from the February 2019 TACPAC meeting is shown in Table 2. A small number of existing potential users were identified, some within 2 km of the CVWPCC. As with reclaimed water, the distance between the facility where the heat is recovered (CVWPCC or pump station) and the user will have a significant impact on the economics of heat recovery.

Table 2 - Summary of Potential Reclaimed Heat Users - from February 2019 TACPAC Meeting

Use Category	Use	Existing	Within 2km CVWPCC?
Process Heat (year round)	CVWPCC Biogas processing	N	Y
	Lumber drying	N	N
	Fibre processing	N	N
	Biofuel processing	N	N
	Distilling	N	N
	Commercial laundry	N	N
	Other Industrial	N	N
	Airport (hot water)	Y	N
Space heat (winter)	CVWPCC space heating	Y	Y
	Rec. Centre	N	N
	School	N	N
	Commercial greenhouse	N	N
	Airport (space heat)	Y	N
	Houses (via district heat/reclaimed v	Y *	Y

Future upgrades at the CVWPCC (and at major pumping stations) could be designed with the potential for heat recovery in the future, so that the required facilities can be added if and when users are identified, without costly reconfiguration of the existing facilities. A detailed study to evaluate feasibility is recommended prior to implementing a heat recovery system.

PRODUCTION OF BIOGAS

At larger wastewater treatment plants (service population of at least 50,000 to 100,000 people), it may prove economical to install anaerobic digestion facilities for treatment of waste solids. Anaerobic digesters reduce the amount of solids (typically by approximately 50%) and produce methane gas that can be scrubbed and then used in cogeneration engines for production of combined heat and electrical power for use at the treatment plant, or the gas may be cleaned to the required standard for sale to the local natural gas utility. The residual solids remaining after anaerobic digestion, generally referred to as biosolids, are suitable for beneficial use provided that regulatory criteria are met (e.g. application to soil as a natural fertilizer/soil conditioner or feedstock for production of compost). Anaerobic digestion is not currently practiced at the CVWPCC, and economies of scale mean that it may not be economical at present; however, new technologies are always in development; the economics will also be affected by the potential to defer expansion of composting facilities, and by the capacity of the local market to accept the compost product. Digestion may be considered in future as a possible resource recovery strategy when the CVWPCC is next upgraded.

BENEFICIAL USE OF TREATED SOLIDS

Where digestion of waste solids is practiced at wastewater treatment plants, the solids product of digestion (biosolids) can be used as a soil conditioner and natural fertilizer, proved that it meets all of the required regulatory standards. Land application of treated biosolids to fertilize agricultural land, for reforestation, and for reclamation of disturbed sites is commonly practiced in British Columbia; however, this can be a costly undertaking, depending on the transportation distance to the biosolids use site and the topography of the site. In some cases, there has been public resistance to land application of biosolids, due mainly to concerns over odours and the presence of potentially harmful substances.

The CVWPCC dewaters waste solids and transports the dewatered cake to a nearby site for use as a composting feedstock. This does not require digestion prior to composting, and it produces a product called SkyRocket that is much more marketable than dewatered biosolids. Production of Class A compost (SkyRocket) as practiced by the CVRD allows sale of the compost product to households and commercial users. Proceeds from the sale of compost help to offset operating costs for solids handling. This is a sustainable strategy for beneficial use of treated wastewater solids as long as the local market can absorb the compost; at some point, digestion to reduce the solids stream to composting may be beneficial to reduce loading on the composting facilities and to reduce the amount of compost produced (see above).

EXTRACTION OF NITROGEN AND PHOSPHORUS FOR FERTILIZER PELLETS

Depending on the treatment processes used, some wastewater treatment plants produce relatively low-volume side streams of high-strength wastewater that would normally be routed back to join the plant influent wastewater for treatment (e.g., water produced as a result of dewatering digested waste solids or waste biological solids from biological nutrient removal processes). For these high-strength side streams it is in some cases economical to extract nitrogen and phosphorus in a small treatment reactor that causes precipitation of a mineral called magnesium ammonium phosphate, commonly referred to as struvite. The struvite pellets can be marketed as a commercial fertilizer, offsetting the production and use of chemical fertilizers. This would not be feasible at the CVWPCC at present, due to economies of scale and the treatment processes currently in use; however, it could be considered for use in future, depending on what processes are implemented at the treatment plant.

HYDROELECTRIC TURBINE FOR GENERATION OF ELECTRICAL POWER AT OUTFALL

In some cases where there is a large elevation difference between the treatment plant and the receiving water (i.e., the land section of the outfall has a steep downward slope), it is possible to install a small hydroelectric turbine to generate electricity. In our experience, this is not cost-effective at smaller plants, even if there is a large head loss available on the discharge to drive the turbine. In the case of the CVWPCC where there is minimal head loss under certain tidal conditions and effluent pumping is required, this type of energy recovery is unlikely to be a viable option.

SUMMARY

In general, resource recovery options must be carefully evaluated for feasibility before implementation. Through this LWMP process, a number of potential applications for reclaimed water and heat recovery were identified by the TAC/PAC committee though the primary limitation for feasibility of these potential resource recovery applications will be identifying users and the long-distance conveyance requirements.

In the future when upgrades to the CVWPCC are undertaken, studies should be completed prior to design to evaluate the addition of resource recovery processes and their feasibility. This may include reclamation of effluent, extraction of heat from the effluent for space heating and cooling, struvite crystallization for fertilizer production, or anaerobic digestion for generation of biogas where analysis shows that this is economically attractive.

Note that technologies for treatment of wastewater and waste solids are continually evolving, and research and development are ongoing. If resource recovery is not considered feasible at the time of CVWPCC upgrades, designs could incorporate flexibility so that facilities for resource recovery can be added to the plant without major disruptions or modifications to the existing facilities in the future.