

2023 Osoyoos Sanitary System Infrastructure Plan



ENGINEERING ■ PLANNING ■ URBAN DESIGN ■ LAND SURVEYING

September 2023

Project No. 302-2051

Distribution List

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Revision Log

Revision #	Revised by	Date	Issue / Revision Description
1	ED	2023/07/19	Issued for Client Review
2	ED	2023/09/14	Final

Report Submission

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Sept 14 2023
Permit to Practice
No. 1000129

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List of Acronyms

AAI	Average Annual Investment
AC	Asbestos Cement
AM	Asset Management
DI	Ductile Iron
ESL	Expected Service Life
FRP	Fiberglass Reinforced Plastic
GIS	Geographic Information System
HDPE	High Density Polyethylene
I&I	Inflow and Infiltration
MBBR	Moving Bed Biofilm Reactor
MSF	Monthly Service Fee
MWWPS	Main Wastewater Pumping Station
PE	Population Equivalent
PSAB	Public Sector Accounting Board
PVC	Polyvinyl Chloride
OCP	Official Community Plan
OIB	Osoyoos Indian Band
O&M	Operations and Maintenance
SCADA	Supervisory Control and Data Acquisition
TRUE	TRUE Consulting
WWTP	Wastewater Treatment Plant

Units of Measure

ft	feet
lgpm	Imperial gallons per minute
km	kilometre
L/d	Litres per day
L/m	Litres per minute
L/s	Litres per second
lpcd	Litres per capita per day
m	metre
mg/L	milligrams per Litre
mm	millimetre
NTU	Nephelometric Turbidity Units
psi	pounds per square inch
USgpm	US gallons per minute

Referenced Reports

2021 WWTP Long Term Plan	Analysis of Osoyoos WWTP, “2021 WWTP Long Term Plan”, April 2021
KWL Trunk Main Capacity Assessment	Analysis of trunk main sewer capacity, “Capacity Assessment Sanitary Sewer Infrastructure”, May 2014
2023 Starlite Relocation Assessment	Feasibility Assessment for Starlite LS Relocation, “Starlite Relocation, Upgrades & Dev. Feasibility Report”, June 2023
2014 Long Range Reclaimed Water Plan	Discussion paper relating to the long-term plan of the reclaimed wastewater storage and use, “Long Range Sanitary Sewer System Expansion Plan”, September 2014

1.0 Town of Osoyoos Sanitary Collection System

1.1 Introduction

TRUE Consulting (TRUE) has been retained by the Town of Osoyoos to perform an analysis of the existing sanitary collection system to identify future system upgrades based on Asset Management (AM) and Capital Planning. The purpose of this report is to evaluate the sanitary collection network performance under existing and future conditions in order to develop a guide for staged system improvements and expansions. A complete review of the Town's treatment system was completed by TRUE in 2021 as a part of the *WWTP Long Term Plan*. This report can be found in Appendix A for reference. The following report will only cover an in-depth analysis of the sanitary collection system.

The following general scope of work has been completed for this project:

- Review of historical population growth and forecast projections on a 20-year horizon.
- Review current inflow data at the Town's Main Wastewater Pumping Station (MWWPS).
- Project future MWWPS inflows based on population projection.
- Develop a sanitary collection model using PCSWMM including:
 - Review of record drawings and survey data
 - Review of Official Community Plan (OCP) for future land use
- Develop a system composite with GIS
- Develop a 20-year horizon workplan for capital, replacement, and O&M projects
- Develop a reserve balance model considering average annual investment and monthly service fees
- Preparation of a report to summarize the analysis and recommendations for replacement, rehabilitation, or improvements to infrastructure.

This report has been separated into four main sections which include:

1. Town of Osoyoos Sanitary Collection System,
2. Sanitary Collection System Asset Management Plan,
3. Sanitary Collection System Capital Plan, and
4. Sanitary Collection System Implementation Plan.

The first section describes the current system and serves as the basis for developing a composite system model. The composite system model then serves as the basis for developing asset inventories and the hydraulic model. Asset inventories are used for asset management which is the focus of Section 2.0. This section provides a risk-based approach for system assets, average annual investment figures, and a reserve balance model. Section 3.0 analyzes the hydraulic model as the basis for capital works planning. Based on the system performance and identified

deficiencies, proposed system upgrades are recommended and accompanied by cost estimates. Lastly, Section 4.0 discusses the implementation of the recommendations as presented in Section 2.0 and Section 3.0. This section provides a work forecast including replacement and capital project for a 20 year horizon, updated reserve balance model to calculate incremental MSF increase, capital financing model and a proposed incremental MSF increase forecast.

1.2 Background

The Town's sanitary sewerage system is split to in two distinct systems: the sanitary collection network, and the wastewater treatment system. The sanitary collection system consists of gravity pipe, forcemains, lift stations, and a MWWPS where primary screening occurs. The treatment system generally consists of additional screening capabilities, aeration and mixing basins, and disinfection. Further discussion into the processes can be found in the following sections as well as the report in Appendix A.

1.2.1 Sanitary Collection and Conveyance System

The Town's sanitary collection and conveyance system consists of approximately 51.1 km of gravity mains, 21 lift stations, and 18.1 kilometers of forcemains. Wastewater is pumped from each lift station to the MWWPS where it flows through a primary screening system before being pumped via forcemain up to the wastewater treatment facility.

A summary of the existing gravity mains sorted by installation date and material is shown below. It can be noted that Asbestos Cement (AC) pipe was the primary material used until the 1970's where there was a mixture between that and Polyvinyl Chloride (PVC). Since approximately 1980, PVC has been primarily utilized with the exception of some High-Density Polyethylene (HDPE).

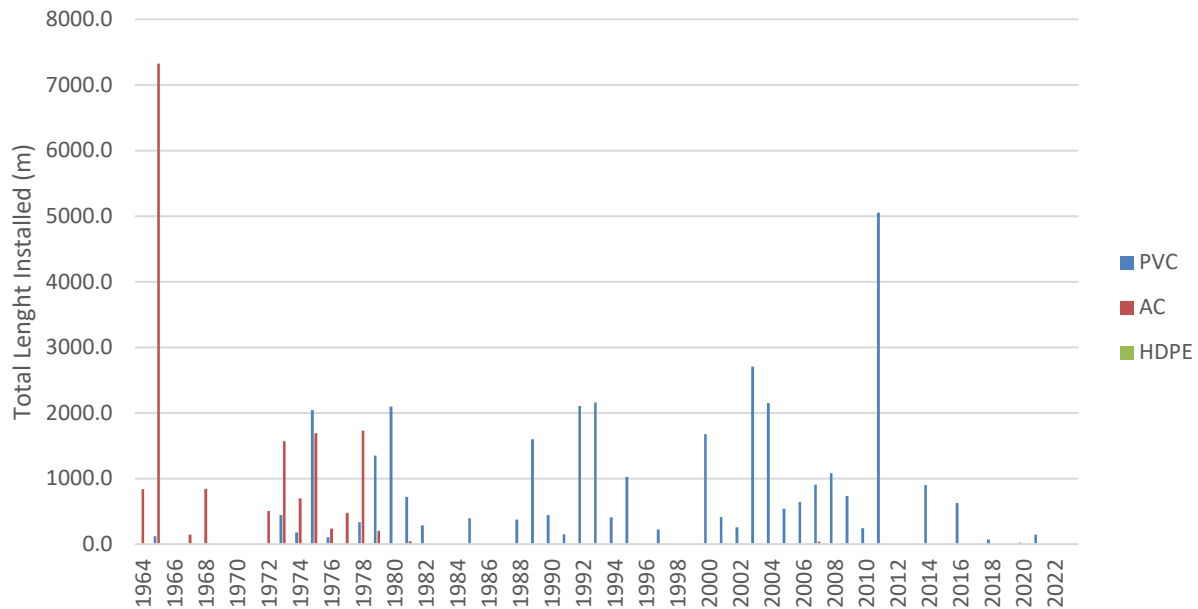


FIGURE 1-1: GRAVITY NETWORK MATERIAL AND INSTALLATION DATES

The existing sanitary collection system covers the current Town boundary as well as rural development areas to the north and south. Additionally, the Town has a servicing agreement with the Osoyoos Indian Band (OIB) where sanitary flows are pumped into the Town collection system with a maximum allowable flow rate. Throughout the serviced area there are 21 lift stations varying in size and capacity. A listing of the Town's lift stations is shown below.

- 95th Street
- 87th Street
- Inkaneep Point
- Roberts Point
- 81st Street
- Lacey Point
- Gala Lift Station
- Smith and Loveless
- Main Wastewater Pumping Station
- Meadowlark
- Blue Blinker
- Haynes Point
- Border Service
- Golf Course
- Cottonwood
- Starlite
- Harbour Key
- Maple Drive
- 44th Avenue
- Oasis
- Bayview

Installation dates for the Town's sanitary collection system begin in the 1960's with frequent updates since. The current network consists of both AC and PVC pipes and Fiberglass Reinforced Plastic (FRP) and Concrete lift stations

1.3 Wastewater Treatment Plant

The existing wastewater treatment plant process consists of a small complete mix aerated lagoon followed by two partial mix aerated lagoons.

Most of the wastewater inflow at the WWTP is received from the MWWPS where 1.5mm screens remove non-organic solids from the flow. Flow received from the golf course lift station is not screened, however the solids in this flow are shredded by chopper pumps. There are three cells at the WWTP, classified as complete mix and partial mix respectively. All cells provide means of separating the sludge from the wastewater before the reclaimed water is treated and stored in the large basins for reuse in the summer months.

A full description of the WWTP process and effluent quality can be found in the *2021 WWTP Long Term Plan* in Appendix A.

2.0 Sanitary System Asset Management Plan

The following sections summarize the development, methodology, and results of the hydraulic PCSWMM model from an **asset management (AM)** point of view. The PCSWMM model was initially developed to calculate and evaluate the capacity of the gravity sanitary system, however it has also been used as a tool to develop asset management strategies for the Town. To incorporate AM into this model, an in-depth review of the Town's manholes and linear assets was required. This work also resulted in an update of the Town's sanitary sewer composite drawing (Appendix B) and will be adopted into the Town's files.

The process of asset management begins by creating an extensive inventory of the Town's tangible capital assets (TCA). Once all assets are included in this inventory, each asset is assigned an expected life. Then, based on the install year and Expected Service Life (ESL), a replacement year is determined, along with an approximate replacement cost. These ideas serve as the basis in the Average Annual Investment (AAI) that needs to be set aside in a reserve to cover upcoming replacement costs for each asset.

2.1 Expected Service Life

The sanitary collection system components as described in Section 1.2.1 have been categorized for the purpose of assigning an ESL. The ESL was determined based on guidelines as outlined in the BC Government *Guide to the Amortization of Tangible Capital Assets (BCGATCA) 2008* as well as on a risk basis. In practice, the ESL estimates are based on best practice, and it is not uncommon that assets outlast their service life. For this reason, certain service lives have been adjusted due to TRUE's extensive experience with these assets in the Osoyoos area. For example, AC pipe in Osoyoos has been assigned a service life of 65 years opposed to 50 as recommended by BCGATCA. Additionally, the service lives have been adjusted for certain assets based on a Risk Assessment. This Risk Assessment is discussed in Section 2.1.1 below.

2.1.1 Risk based Expected Service Life

A risk model was developed to identify and prioritize replacement projects. This model was based on the following risk equation:

$$\text{Risk} = \text{Criticality} \times \text{Probability}$$

Probability was determined based on remaining useful life which is a function of material and installation year. Criticality is based on the size of pipe, indicating the capacity to convey high flows. Once the risk was determined, the assets were classified into high, medium, and low risk categories. Table 2-1 provides a break-down of the risk categories with the total length of pipe and the percentage of the total system per category.

TABLE 2-1: GRAVITY NETWORK PIPE TYPES.

Risk	Length (m)	%
High	374	0.7%
Medium	2,388	4.7%
Low	48,314	94.6%

While most of the gravity pipes in the network are classified as low risk, the medium and high risk pipes can be attributed to AC trunk mains that are nearing capacity as well as their anticipated service life.

The risk-based method for determining the ESL values is a practical tool to incorporate into system planning. This approach informs decision making for capital projects and assists with prioritization for replacement projects based on impacts to the system performance. Risk is also used to infer a linear asset's ESL value which can be reasonably modified to extend or decrease the ESL. For the purposes of this report, linear asset ESL values are calculated by multiplying the BCGATCA or experienced based ESL by a risk-based ESL factor. These factors are presented

in Table 2-2 which show a decrease in ESL for high-risk assets and an increase in ESL value for low-risk assets.

TABLE 2-2: RISK BASED ESL MULTIPLIERS

Risk	Multiplier
Low	1.2
Medium	1
High	0.8

This approach is more practical when assessing the system and planning for replacements as most of the system is at low risk of impacting the system. Additionally, extending the service lives reduces the Average Annual Investment. The risk model can also be a useful tool in determining additional asset replacement as it the focus can be placed on areas with assets at a higher risk.

2.2 Average Annual Investment and Asset Summary

The AAI was determined for each of the system components and is presented in the figure below. This value is calculated based on amortizing the replacement cost for each system component over the ESL of that specific component. By analyzing and determining the AAI in this format, continual replacement of each component is achieved. The AAI values are shown in both a visual and tabular form in Figure 2-1 and Table 2-3 below.

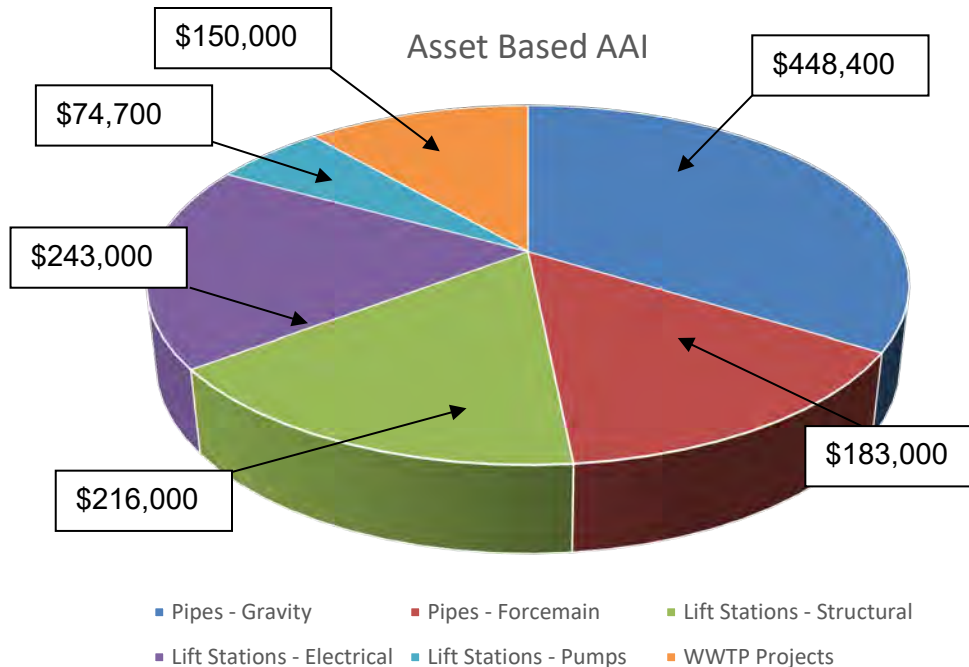


FIGURE 2-1: ASSET BASED AAI.

TABLE 2-3: ASSET BASED AAI.

PROJECTS	EXPECT SERVICE LIFE	Total REPLACEMENT COST (2023 CAD\$)	AAI (2023 CAD\$ PER YEAR)
GRAVITY PIPE NETWORK	65 – 96*	\$42,616,500	\$448,400
FORCEMAINS	80	\$15,521,900	\$183,000
LIFT STATION STRUCTURES	75	\$16,200,000	\$216,000
LIFT STATION ELECTRICAL COMPONENTS	15	\$4,650,000	\$243,000
LIFT STATION PUMPS	20	\$1,944,500	\$74,700
WWTP REPLACEMENT AND ENHANCEMENT	15	\$4,500,000	\$150,000
TOTAL		\$85,432,900	\$1,315,400

Services, manholes, and valves were accounted for in the per lineal meter price for the gravity network and forcemain analysis. Valve chambers and control valves were included in the Lift station breakdown, and general upgrades were considered in the WWTP replacement analysis. Road restoration as well as sidewalk, curb, and gutter were also included and blended throughout. The ESL of sidewalk, curb, and gutter are generally less than the ESL for underground assets therefore additional funds may be required for simple projects replacing sidewalk, curb, and gutter. However, this analysis includes the replacement of these assets in larger projects.

Important to note is that the AAI is calculated for assets that are a like for like replacement which will be funded by reserves. This distinction is critical when considering funding avenues for future projects, specifically capital funded projects. Capital funding can offset the AAI values (contributing to reserves) if a current asset is being modified under a capital funded project. In Section 3.0 of this report, capital projects are proposed that will modify and replace existing infrastructure. For the purposes of this report, it is assumed that any assets being modified, replaced or removed in the proposed capital projects will be funded by capital budgets. Thus, these assets were removed from the AAI calculation. A breakdown of these assets is provided in Section 3.0

This practice has a certain risk when considering the feasibility of completing the capital project. If the capital project is not completed, no reserves would be set aside for replacement of the infrastructure. If this is the case, the breakdown of these assets (in Section 2.0) also provides AAI values inform the Town of its AAI obligations if a proposed capital project is abandoned.

2.3 Annual Total Replacement Costs

The annual total replacement cost was determined based on the total replacement costs of all assets expected to be replaced in the corresponding year. Specifically, the replacement date was determined based on the asset's installation date and ESL. Annual total replacement costs are presented in Figure 2-2 and similar annual replacement cost figures for system component categories (as per Table 2-3) are presented in Appendix C. The peak in annual spending seen in year 2023 results from the assets that are currently beyond their expected ESL. Discussed further in Section 4.0, certain projects are deferred to maintain a more consistent workload in the coming years. Additional peaks in further years are likely due to the scheduled replacement of assets that were installed as a result of a large project (for example the northwest sector servicing) and are at the end of their ESL.

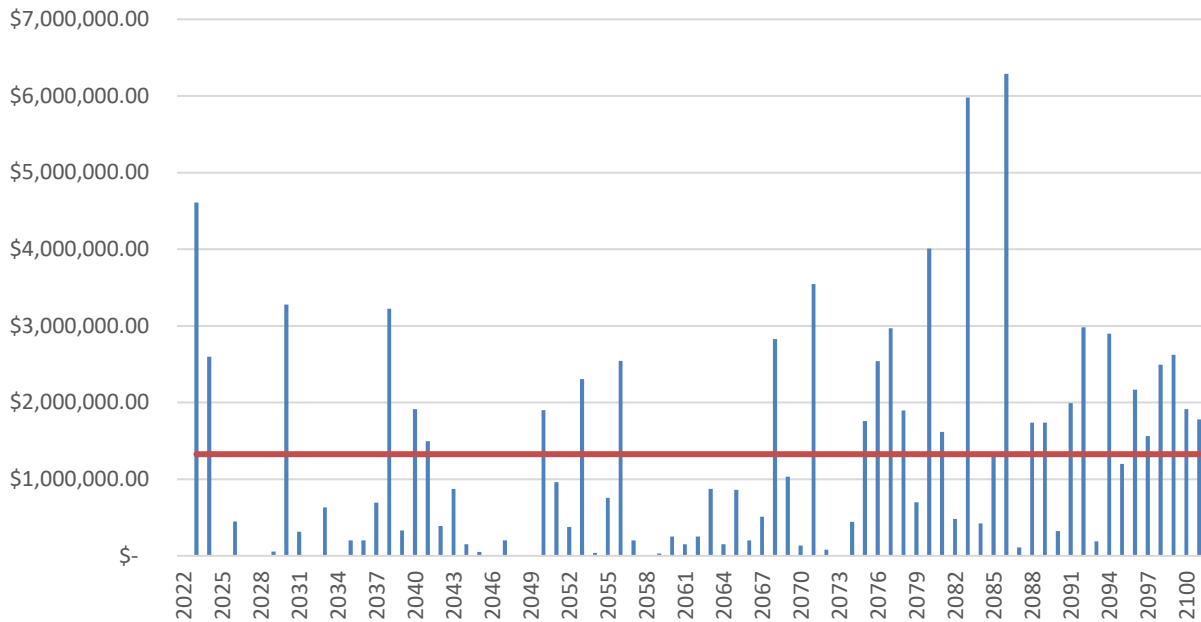


FIGURE 2-2: PROJECTED ANNUAL SPENDING.

2.4 Reserve Balance Model

A financial model was constructed showing a hypothetical reserve balance. This financial model can be viewed in Figure 2-3 and the corresponding data is provided in Appendix D. The reserve balance is calculated based on an initial reserve balance as provided by the Town of Osoyoos (\$2,527,500) and subsequent years being calculated as the previous year's balance plus the AAI and less the annual total spending. This analysis was conducted for a period which was based on all assets being replaced at least once. As such, this period corresponds to 96 years because this is the longest ESL in the asset inventory. Other assets with shorter ESL periods were repeated in the analysis to represent continual replacement costs. For example, the annual replacement cost for lift station electrical components cycles every 15 years as per the Electrical ESL. Repetition of system component categories can be observed in the annual replacement cost figures in Appendix C.

When analyzing Figure 2-3, the initial balance and the final balance are considered. The initial balance is a slight deficit which corresponds to the assumed replacement of all assets with outstanding expected replacement dates. This is slightly offset based on the starting reserve balance of \$2,527,500 as provided by the Town. It is understood that these assets will likely be replaced over several years which will minimize impacts to significant initial costs. The AAI quickly brings the reserve balance to a positive value and oscillates above zero. The final balance remains above zero with a final value of roughly \$15 million. This is expected because the AAI is continually contributing funds to continually replace infrastructure. Thus, the positive balance is

not a surplus. This financial model is intended to be used as a working model and will need to be updated based on actual system maintenance and upgrades.

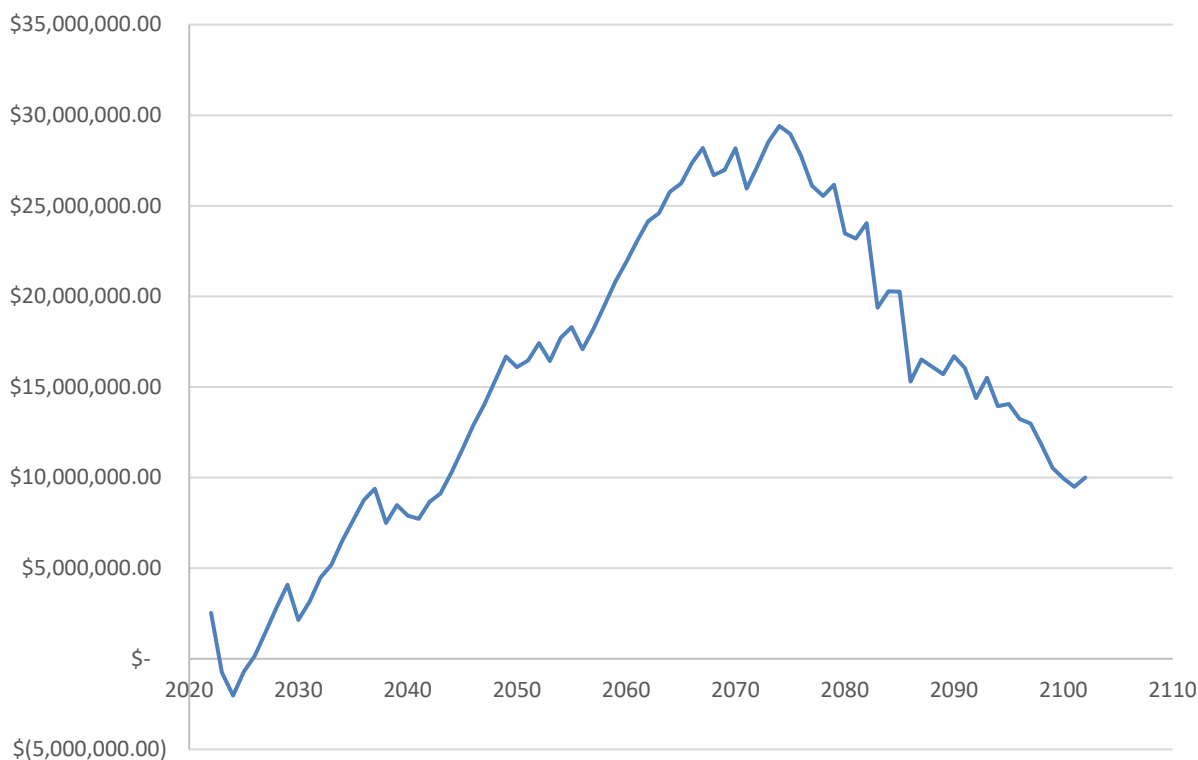


FIGURE 2-3: RESERVE BALANCE FOR EXISTING ASSETS.

2.5 Average Annual Investment and Asset Summary

The AAI was determined for each of the system components and is presented below. This value is calculated based on saving the replacement cost for each system component over the ESL of that system component. This method of calculating the AAI allows for continual replacement of system components. AAI values for each component category are presented in Table 2-3 and visually shown in Figure 2-1. In summary the total system AAI is calculated as approximately \$1,415,400. These values are intended for general use and updates to the anticipated AAI should be made based on actual system maintenance and upgrades.

2.5.1 Gravity Pipe Network

Currently, no gravity pipes installed are beyond their useful service life, not considering the risk-based approach. When considering risk, pipes in the Town's core are beyond their ESL. Specifically, the pipes that are original AC installed in the 1960's are at or near their ESL. For the purpose of calculating the AAI, all pipes are replaced based on their useful life. At the end of its

useful service life, the performance of each pipe is graded by reviewing a PCSWMM desktop model output field called *Max Flow/Cap*. This field analysis the total capacity of the pipe, and if it is greater than 0.9, the pipe is replaced with a pipe minimum one size larger. All replacement pipes are assumed to be PVC.

There may also be certain circumstances where the pipe is required to be updated strictly because of capacity. These scenarios are further discussed below. The replacement costs for gravity pipe are included in Appendix C.

2.5.2 [Forcemains](#)

The AAI for forcemains was determined strictly on the basis of replacement based on service life. It was assumed that most forcemains would be replaced with PVC pipe of the same diameter. An exception was made for the HDPE line that crossed underneath the lake.

2.5.3 [Lift Stations](#)

A similar analysis was performed for each lift station and their components. Replacement costs of key components were generalized based on lift station volume and pump capacity/horsepower, however the costs were adjusted as required for the larger more critical lift stations.

Currently, all lift stations are sized adequately. However, under future conditions there are two minor deficiencies and one major deficiency.

Under future conditions, the Starlite Lift Station has a 5 L/s pumping deficiency, however TRUE is currently working on the *2023 Starlite Relocation Assessment* which will confirm capacity concerns.

Additionally, under future conditions the Harbor Key and Maple Drive lift stations come within 95% and 80% of their capacities respectively. TRUE recommends replacing the existing pumps with larger capacity pumps as they come to the end of their service life.

3.0 Sanitary System Capital Plan

The Sanitary System Capital Plan will serve as a basis for improvements and upgrades to the Osoyoos Sanitary Collection system from 2024 to 2044. In this section, there will be recommendations for upgrades relating to the system performance and wastewater treatment process upgrades. Key considerations for these upgrades include anticipated future flows and wastewater treatment plant current shortcomings alongside anticipated population growth.

This Plan includes the assessment of an overall sanitary collection system model for the Town of Osoyoos' sanitary network. By reviewing the Town's system, it can be accurately assessed for potential constraints or areas of concern. This Capital Plan focuses on the collection, conveyance, and treatment of the wastewater system in the Osoyoos area.

3.1 Population and Flow Derivation

Osoyoos had a 2021 Census population of 5,556. Seasonal variations in population make it difficult to accurately estimate the current served population, as well as accurately project the flows into the future. For this reason, rather than basing the current and future flows on actual population, flows were based off individual parcel zoning as follows.

TABLE 3-1: TOWN OF OSOYOOS ZONING DESIGNATIONS

Zoning Designation	Zoning Description	Population Density per ha	Allowable Units per Parcel
AG	Agricultural		1
AG1	RDOS - Agricultural One Zone		1
C1	Downtown Commercial	50*	
C2	Shopping Centre Commercial	50*	
C3	Highway Commercial	50*	
C4	Tourist Commercial	62	
C5	Recreation Vehicle Park / Campground	62	
C8	Service Commercial	50*	
CD1	Comprehensive Development Zone 1 (Desert Mirage)	85	
CD2	Comprehensive Development Zone 2 (Village By The Lake)	85*	
CD7	Comprehensive Development Zone 7 (Indigo)	85*	
CR	Commercial Residential	50*	
CT1	RDOS - Tourist Commercial Zone 1	50*	
CT2	RDOS - Campground Commercial Zone	62	
M3	Special Industrial	50*	
PI	Public And Private Institution	50*	
PR	Parks And Recreation		1
R1	Single Family Residential		1*
R2	Single Family Residential Small Lot		1
R5	Manufactured Home Strata Development	30	
R6	Medium Density Residential	35	
R7	High Density Residential	75	
RA	RDOS - Resource Area		1*
RS1	RDOS - Residential Single Family One Zone		1
SH4	RDOS - Small Holdings Four Zone		1
OIB	Osoyoos 1, IR1 lands	n/a**	n/a**

**Assumed values*

***Derived in the table entitled OIB Development Summary (refer to Appendix A)*

3.2 Population Estimates

3.2.1 Current Population Estimate

Rather than using census populations, the serviced population was determined on a **Population Equivalent (PE)** basis. The PE was determined based on the zoning of the parcels in the current serviced area. The population density of each parcel (based on zoning) was multiplied by the area of each parcel, providing a buildout equivalent population for each specific parcel. Population density associated with each type of zoning can be seen in Table 3-1 as shown above.

Based on the Town's current zoning, the total Town serviced area has a population equivalent of 13,950.

3.2.2 Future Population Estimate

The future serviced population increases marginally due to the expected rezoning within the Town's boundaries. The anticipated amendments to the zoning of certain parcels were determined based on OCP guidelines and discussions with the Town. The majority of growth and the resulting impact on the Town's sewerage system is therefore associated with potential service extensions outside of the Town's municipal boundary (i.e., to service both RDOS and OIB development).

Based on the Town's future zoning, the total serviced area has a population equivalent of 17,210.

3.3 Model Parameters

The following parameters were used as input into the sanitary collection system model. The sources for these parameters vary as the best source was used for each specific parameter.

TABLE 3-2: MODEL INPUT PARAMETERS

<i>Parameter</i>	<i>Value</i>	<i>Units</i>	<i>Notes</i>
ADWF	230	L/cap/day	Sourced from KWL report
Diurnal Curve	Varies		Peak of 1.846
I&I	0.06	L/s/ha	Limited area of influence to 25% of total area
Mannings 'n'	0.01		

3.3.1 Average Dry Weather Flow (ADWF)

ADWF is used as the base loading for the entire system. It can be described as the base amount of flow that enters the system during typical dry weather conditions on a per capita basis.

The ADWF used in this analysis originated from the Kerr Wood Leidel Capacity Assessment (2014). KWL performed an in-depth analysis comparing flows received at the MWWPS to expected population and paired that with rainfall events.

Their analysis determined an ADWF of 230 L/capita/day. This value is consistent with other sources and TRUE was confident that it would provide an accurate basis.

It was assumed that the per capita daily flows would remain constant throughout the 20-year design horizon.

3.3.2 Diurnal Curve

A Diurnal Curve was applied to the ADWF to calibrate it into a Peak Wet Weather Flow (PWWF) and allow for a more accurate hourly loading of the system.

The diurnal curve allows for a peaking factor or 1.846 at 07:00. An illustration of the diurnal curve utilized for the PCSWMM model is as follows:

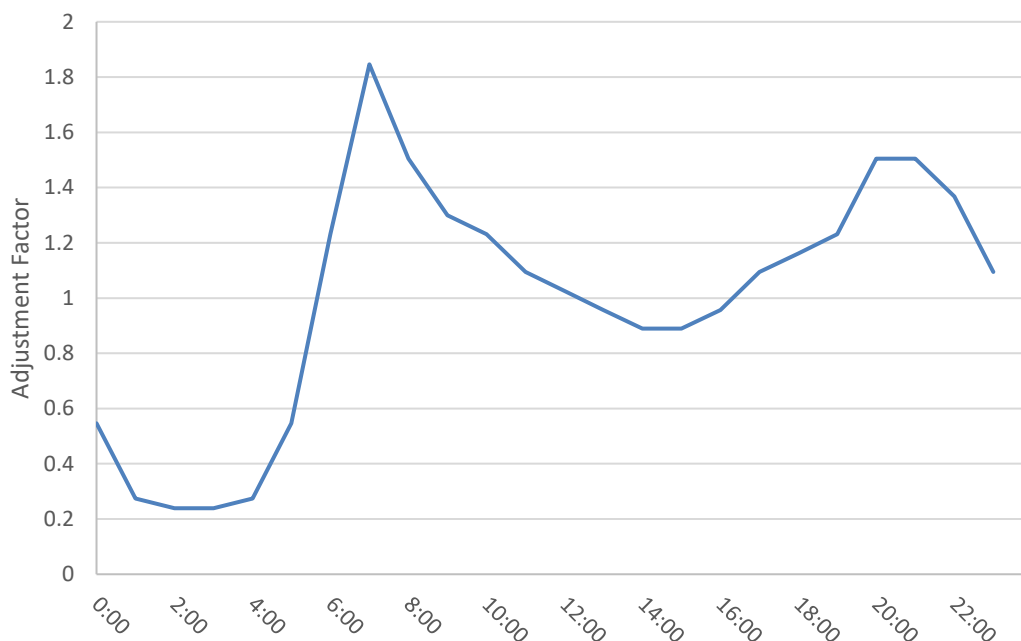


FIGURE 3-1: DIURNAL CURVE.

3.3.3 Inflow and Infiltration (I&I)

The Osoyoos Subdivision and Development Servicing Bylaw suggests an I&I allowance of 0.06 L/s/ha for all pipe installed out of the water table. When compared to the Osoyoos Sanitary SCADA Data, this estimate served to be overly conservative.

The I&I was reduced, under the assumption that only 25% of the connected area results in I&I that enters the sanitary collection system.

3.4 PWWF Calculation

3.4.1 Existing System PWWF Calculation

The ADWF and Inflow & Infiltration (I&I) flows were generated for each parcel based on the population density and per capita flows, as well as parcel area. Using the spatial join function in PCSWMM, the flows were assigned to the nearest manhole as per the updated GIS data. A diurnal flow pattern was applied to the base flow at each manhole. These flow assignments served as a basis for the calculated flows as per the PCSWMM model.

The PWWF at the MWWPS was then calculated using a diurnal peaking factor for the entire system. Initial computational results obtained from the PCSWMM model suggested that an ADWF of 230 L/cap/day and an I&I of 0.06 L/s/ha resulted in abnormally high flows at the MWWPS. The I&I was reduced, under the assumption that only 25% of the connected area produces I&I that enters the sanitary collection system.

The Town also provided Supervisory Control and Data Acquisition (SCADA) for the entire year of 2022, which allowed TRUE to reference calculated flows against recorded flows. The flows calculated as per the SCADA data suggest a PWWF of 141.8 L/s. The observed PWWF as compared to the calculated wastewater flows at the MWWPS were as shown.

TABLE 3-3: PWWF FLOWS AT THE MWWPS UNDER CURRENT CONDITIONS.

	<i>PWWF (L/s)</i>
Calculated	131.1
Extracted from SCADA	141.8

There is a slight variation in results when comparing the calculated PWWF to the observed PWWF as extracted from the Town's SCADA system. These numbers differ due to hydraulic peaking effects that arise when multiple lift stations operate simultaneously, thereby increasing peak flows observed at the Town's MWWPS.

As previously noted, the OIB and the Town currently have a servicing agreement in which a maximum of 16 L/s of wastewater can be pumped into the trunk main along Cottonwood drive. This servicing agreement is limited to 16 L/s until such time as the Cottonwood drive trunk sewer main is upsized.

3.4.2 Future Conditions PWWF Calculation

It was assumed that the per capita wastewater flow would remain constant at 230 L/cap/day for the future conditions scenario. The assumption of 0.06 L/s/ha for 25% of the connected area was also maintained.

The majority of the increase in flows comes from anticipated development on the OIB. As previously mentioned, the Town and the OIB currently have a servicing agreement in which the OIB is allowed to pump a maximum of 16 L/s into the Cottonwood lift station. Future buildout conditions for the OIB suggest flows of 22 L/s will be received in the Cottonwood trunk main. Additional development also suggests an additional 14 L/s will come from development near Nk'Mip Corner. Anticipated PWWF flows at the WWTP are shown below.

TABLE 3-4: PWWF FLOWS AT THE MWWPS UNDER FUTURE CONDITIONS.

	<i>PWWF (L/s)</i>
Calculated	186.1

3.5 Sanitary Gravity Model and Proposed Upgrades

The collection system gravity model was analyzed under both existing and future flow conditions. Results from the future flow conditions provided the basis for the analysis on the complete collection system, with deficiencies determined discussed below. These deficiencies are defined based on the calculated flows through each specific pipe as compared to the pipe's capacity. The PCSWMM model has an output parameter called *Max Flow/Cap* which describes what percentage of the pipes maximum capacity is being used under maximum flow conditions.

TABLE 3-5: MODEL RESULT PARAMETERS

<i>Max Flow/Capacity</i>	<i>Pipe Colour</i>	<i>Deficiency</i>
< 0.5	Green	Not Deficient
0.5 – 0.9	Yellow	Minor Deficiency
> 0.9	Red	Major Deficiency

The *Max Flow/Cap* parameter serves as the basis of the capacity assessment and highlights areas of concern. Note that in the figures below, orange pipes represent forcemains, and green squares represent lift stations. Full figures illustrating the max flow / capacity of each pipe are shown in Appendix E and Appendix F for both the existing and future scenarios.

3.5.1 Current System Deficiencies

As seen in yellow and red in the Figure below, there are two trunk mains on the east side of the Town that are currently undersized for future flows. Specifically, these are referred to as the Cottonwood and the Ponderosa/Maple Trunk Mains



FIGURE 3-2: CURRENT SYSTEM DEFICIENCIES ON THE EAST SIDE.

3.5.1.1 Cottonwood Trunk Main

Under the Town's servicing agreement with the OIB, at maximum flows the Cottonwood trunk main is undersized. To address this, additional pipe capacity is required to convey the PWWF. The additional flows that this trunk main will see are predominantly due to development on OIB lands.

3.5.1.2 Ponderosa/Maple Trunk Main

Similar to the Cottonwood trunk main, flows from the OIB development are received and pass through this area. Under future PWWF conditions the sanitary mains along Maple Drive and Ponderosa Drive are undersized and will require an increase in capacity to accommodate future flows.

3.5.1.3 Forcemain Upgrades at the WWTP

This deficiency is on the west side near the MWWPS, and may result from a modeling inaccuracy relating to the forcemains. The deficiency arises when all three lift stations (Starlite, Smith and Loveless, and 44th Avenue) are pumping **at the same** time into their common forcemain. There are two sections of 300mm pipe that exist between the outlet manhole and the MWWPS, which serves as a major bottleneck. This restriction in the flow causes ponding in the outlet manhole. When two or less pumps are running at the same time this is not an issue.

An attempt was made to determine the likelihood that all three lift stations would be pumping simultaneously, however a lack of available data in the 44th avenue lift station made that analysis not possible. While currently there may be a low possibility that all three lift stations pump together, as population and flows increase, the possibility of all three lift stations pumping together increases too. For this reason, TRUE suggests that the Town monitors the backflow/ponding in the outlet manhole under high flow conditions.

3.5.1.4 Starlite Lift Station

There are no current capacity concerns for the Starlite Lift Station however under future conditions the station becomes undersized. TRUE is currently working to develop an options analysis for relocating and or upsizing the Starlite Lift Station. For a complete assessment please refer to the report titled *2023 Starlite Relocation Assessment*.

3.5.1.5 SCADA System Upgrades

TRUE recommends that the SCADA system be analyzed to ensure that all possible relevant data is able to be collected and collect such data. Specifically, flow meters should be installed in all lift stations, and those flow meters should be connected to the SCADA system. Additionally, information on start/stop times for the lift station pumps would assist in system analysis. Performing this analysis will also give insight into the health of the electrical systems at each lift station and can either confirm or defer their proposed replacement date.

3.5.1.6 WWTP Cell 2 Desludging

During spring and early summer of 2023 an overturn in one of the WWTP aeration cells resulted in strong odour release. This event was likely related to the fact that Cell 2 has not been de-sludged in approximately 10 years. Once the cell is de-sludged, additional work can be completed of required.

3.5.1.7 Air Piping Upgrades at WWTP

During a recent site visit, it was noted that there are multiple air supply headers and main air supply pipes at the WWTP that are leaking. These leaks should be repaired to avoid failure and emergency repairs. These leaks will likely be addressed by installing new supply piping and headers for each cell, rather than one complete air piping network.

3.5.1.8 Manhole Smart Covers

In the PCSWMM model there were various manholes that showed the possibility of major or minor surcharging under high flow conditions. Installing smart covers in these areas would allow the Town to obtain and collect information on the frequency of these events and assist in determining if additional system upgrades are required. Additionally, installing a localized weather station will assist in quantifying the effect that I&I has on the system.

3.5.1.9 CCTV Program

Various CCTV programs that have occurred in recent years have identified that certain areas with PVC pipe installed between 1980 and 1995 seem to be in poor condition. A CCTV program targeting these areas would provide confirmation of the asset condition which could then be incorporated into the ongoing asset management plan.

3.5.1.10 Forcemain Odour Reduction

Currently, the Town of Osoyoos injects a BioMaxx Odour Reduction product in certain lift stations. Some of these injection systems were installed in a rudimentary fashion. It is recommended that the installation of these systems be updated, including uploading the dosing data to SCADA.

3.5.1.11 Cell 1 MBBR Conversion

Section 4.0 of the 2021 WWTP Long Term Plan discusses the options for augmenting the treatment process at the WWTP. Of the options discussed, the installation of an MBBR (Moving Bed Biofilm Reactor) greatly improves plant efficiency. The conversion to an MBBR would also require additional concrete tanks, aeration upgrades, and an odour containment building at the WWTP.

3.5.1.12 Golf Course Lift Station Screening

Additionally, to convert the existing treatment process to a MBBR, screening at the Golf Course Lift Station would be required. This could be accomplished by either installing a screening facility at the Golf Course lift station, or pumping the wastewater received at the Golf Course Lift Station to the MWWPS.

3.5.1.13 Reclaimed Wastewater Storage and Irrigation Extension Study

The 2021 WWTP Long Term Plan briefly discusses treated effluent storage and use, and the requirement for additional storage and irrigation areas. Additionally, a memo titled 2014 Long Range Reclaimed Water Plan discusses the future need for service expansions. An in-depth update has not been completed since 2014, however population trends suggest that there is adequate storage until 2053. It is recommended that the study is updated to confirm when the service expansions will be required.

3.5.1.14 Onsite Chlorine Generation

The Town has expressed interest in updating the chlorine gas system for the reclaimed water, potentially to an onsite chlorine generation system. This project could be partially funded through the replacement of the existing Chlorine system, with additional required funds coming through capital. This project would require a bulk solids storage facility, solids conveyance, a generation system, and associated buildings.

3.5.1.15 Chlorine Tank Recirculation

In addition to updating the chlorine generation and dosing system, recirculation may be required to help achieve proper contact time for the reclaimed water.

3.5.1.16 Wastewater Treatment Plant Auxiliary Power

Currently, there is no backup power at the WWTP, and power outages greatly disrupt the treatment process. This project involves the supply and install of an onsite backup generator, and SCADA integration.

3.5.1.17 Wastewater Collection and Treatment Master Plan Review

The Wastewater Collection and Treatment Master Plan Review requires updating,

3.5.1.18 Magnolia Place Reconstruction

The existing sanitary collection pipe installed on Magnolia Drive has major failures and is causing large backups in the collection system.

3.5.1.19 Wastewater Treatment Plant Design and Planning

The process selection, preliminary and detailed design, permitting, and construction of the WWTP upgrades will be a long-term, intensive project due to the regulatory requirements in place. This allocates funds to cover the costs incurred throughout those processes.

Project Sheets including Class D cost estimates are included in Appendix G.

3.5.2 Proposed System Upgrades Summary

This section provides a brief summary of the proposed projects discussed in the sections above. These projects aim to address current deficiencies and concerns that exist in the system. A high-level cost estimate as well as the anticipated year of the work is included in the table. It is important to note that the cost shown as well as the proposed project year are definitive and may be adjusted.

TABLE 3-6: PROPOSED CAPITAL PROJECTS.

PROJECT TAG	PROJECTS	COST PLUS ENGINEERING (15%) AND CONTINGENCY (30%)	INSTALLATION DATE
C1	<i>Cottonwood Upsizing</i>	\$1,740,170	2026
C2	<i>Maple Drive Upsizing</i>	\$1,497,230	2026
C3	<i>Starlite Lift Station Relocation</i>	\$3,625,000	2025
C4	<i>Forcemain Extension near WWTP</i>	\$290,000	2027
C5	<i>SCADA System Upgrades</i>	\$363,000	2024
C6	<i>WWTP Cell 2 Desludging</i>	\$2,900,000	2024
C7	<i>Air Piping Updates at WWTP</i>	\$1,450,000	2025
C8	<i>Manhole Smart Covers</i>	\$181,000	2024
C9	<i>CCTV Program</i>	\$139,000	2025
C10	<i>Forcemain Odour Reduction</i>	\$363,000	2028
C11	<i>Cell 1 MBBR Conversion</i>	\$7,613,000	2026
C12	<i>Golf Course Lift Station Screening</i>	\$725,000	2025
C13	<i>Reclaimed WW Storage and Extension Study</i>	\$109,000	2025
C14	<i>Onsite Chlorine Generation</i>	\$2,175,000	2026
C15	<i>Chlorine Contact Chamber Recirculation</i>	\$363,000	2029
C16	<i>WWTP Auxiliary Power</i>	\$218,000	2024
C17	<i>Wastewater Master Plan Review</i>	\$45,000	2028
C18	<i>Magnolia Place Reconstruction</i>	\$348,000	2024
C19	<i>WWTP Design and Planning</i>	\$1,015,000	2024

4.0 Sanitary System Implementation Plan

The Sanitary System Implementation Plan proposes practical steps to effectively achieve the recommendations of the Asset Management Plan and Capital Plan. This translates into a joint replacement and capital project work forecast. The main purpose for this approach was to eliminate mass replacement of assets in 2023 which is shown in the reserve balance model in Section 2.4 Based on the work forecast, the next steps to maintain and fund the proposed projects were determined. This entails updating the reserve balance model to consider the work plan with capital projects and replacement projects. The purpose for updating the reserve balance is to determine the proposed incremental monthly service fee (MSF). The MSF will be determined

based on multiple expenses including the existing AAI (as per Section 2.2), additional AAI for capital projects and financing capital projects.

Additional AAI corresponding to the capital projects will be required for ongoing maintenance. This entails updating the asset management plan (as per Section 2.0) and the reserve balance model with the recommended work forecast including both replacement and capital projects. These updates will show the Town what AAI obligations they will incur once the capital projects are completed and indicate how to budget for replacement of infrastructure throughout their lifespan. As previously mentioned in Section 2.2, proposed capital projects will modify, replace or abandon existing infrastructure. These assets were removed from the original asset management analysis because the replacement cost is considered to be funded by capital budgets. A complete table of modified assets can be viewed in Appendix H.

In certain cases where the replacement of an asset may be expedited by development (for example, the Cottonwood trunk main upsizing), it was assumed that the Town would only be responsible for a portion of the cost, with the remainder coming from the developer. The Town's portion was assumed to be financed through a loan. For the full cost sharing analysis refer to the appendices.

It was assumed that all capital projects except those listed below would be financed.

- *SCADA System Upgrades*
- *Manhole Smart Covers*
- *Reclaimed Wastewater Storage and Irrigation Extension Study*
- *Wastewater Collection and Treatment Master Plan Update*

A financial model was developed to determine the required MSF to support the proposed capital works. This model is presented in Section 4.4. In summary, the Sanitary System Implementation Plan determines an incremental MSF forecast to fund the proposed work forecast which was determined to be a practical and feasible approach for the Town of Osoyoos.

4.1 Work Forecast

The work forecast was developed with the intention that the Town would use it for planning purposes. It should be modified and updated as projects are completed and conditions change. This work forecast was developed on a 20-year horizon corresponding to the Capital Plan timeline, with a main focus on the initial 10 years. Due to a current backload of projects, following a standard timeline in replacing aging assets would result in an unreasonable amount of work in the next year and a half. For this reason, certain projects were deferred to lighten the workload. Projects were prioritized based on the factors listed below, listed in order of priority.

Apart from a few minor areas, the system is currently in reasonable condition from a capacity point of view. Most gravity collection pipes have adequate capacity to convey existing and future

flows, and all lift stations are currently sized adequately. As a result, most of the replacement projects suggest a relining of the pipe rather than a complete pipe replacement, provided the pipe is in acceptable condition to be lined.

- System Performance relating to day-to-day operations.
- System Capacity to ensure adequate system performance under future scenarios.
- Risk based on anticipated flows for that specific asset.
- Financial Planning to ensure the Town of Osoyoos/taxpayers aren't hit with a large financial burden.
- Grouping projects together to create efficient work plans.

Projects that were required to ensure or enhance system performance were considered the most important and were the least likely to be deferred. Examples for these projects include the SCADA System upgrades, resolving the odour issues at the WWTP, and developing a CCTV program to gain information on areas in town where the exact system condition is unknown.

Projects addressing capacity issues that were likely to be amplified in the near future were addressed next. These projects will likely be stemmed due to future development, specifically on the OIB lands to the east. The pipes are currently at capacity and any additional development will likely not be able to happen due to this restriction. Specifically, these projects are the Cottonwood and Ponderosa/Maple upsizing, and the forcemain upgrades at the WWTP.

As mentioned above, there is a current backlog of projects stemming from asset renewal. These are largely focused on electrical systems at various lift stations, and pipe relining projects, specifically in the Town's core where most of the sanitary pipe is AC pipe installed in the 1960's. The lift station electrical projects were deferred one to two years to allow for more information to be gained through the SCADA system review and upgrades as suggested in Section 3.5.1.5. Additionally, the outdated pipe lining projects were ranked based on risk (a function of diameter and flow) and deferred up to 8 years.

Other smaller projects or projects that relate to a low-risk asset were deferred up to 5 years to minimize costs in the first years of the capital plan. These projects were mainly pipe lining projects.

Finally, an attempt was made to group together projects where feasible to limit the amount of overhead and soft costs from each project.

Certain lift station pumps may need to be upsized once they reach the end of their ESL. It is recommended that when the Town replaces pumps or complete lift stations, a review of completed or proposed development in the catchment area be performed to ensure that the new infrastructure is sized adequately.

A bar graph outlining the Capital and Asset Management projects per year on a 12-year horizon is shown below.

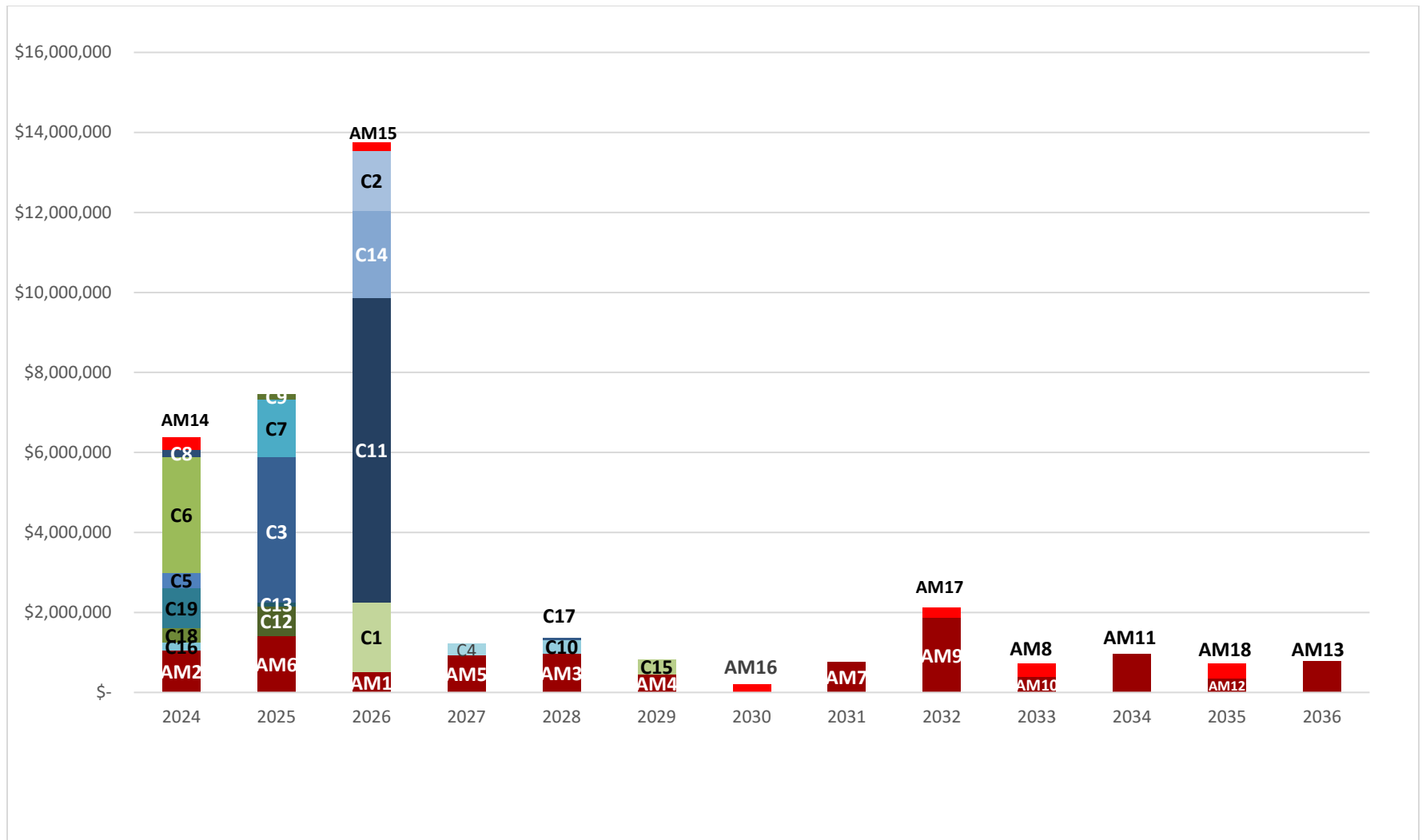


FIGURE 4-1: 12-YEAR HORIZON WORKLOAD.

TABLE 4-1: ASSET MANAGEMENT ADJUSTED PROJECT PLAN.

	PROJECTS	CONSTRUCTION COST	INSTALLATION DATE
AM1	Reclaimed Water Chlorination Upgrades	\$508,000	2026
AM2	Kingfisher Foreshore Relining	\$1,040,000	2024
AM3	89 th Street Relining	\$960,000	2028
AM4	62 nd Street Relining	\$460,000	2029
AM5	Oleander/89 th Foreshore Relining	\$930,000	2027
AM6	Harbour Key Upgrades	\$1,420,000	2025
AM7	South Town Center Relining	\$770,000	2031
AM8	Osprey Place Relining	\$330,000	2033
AM9	North Town Center Relining	\$1,870,000	2032
AM10	Hummingbird Lane Relining	\$400,000	2033
AM11	Alyssum Court Relining	\$960,000	2034
AM12	92 nd Ave Relining	\$350,000	2035
AM13	89 th Street North/Highway 3 Relining	\$780,000	2036
	<i>Total</i>	\$10,778,000	

4.1.1 Replacement Projects

Existing asset replacement projects represent works required by the risk-based asset management analysis presented in Section 2.0. These projects were created by grouping works as discussed in Section 4.1. This creates a logical work plan for the required projects, and allows projects to be deferred to limit the number of projects and spending in any particular year. The projects shown in Table 4-1 are further discussed below. The Class D cost estimate associated with each project includes 15% for Engineering and 30% for contingency. Additionally, the project price is inclusive of services and manholes.

Most of the projects below result from existing AC pipe reaching the end of its ESL. It has been assumed that all pipe will be in adequate condition and be able to be relined, rather than a complete open excavation replacement. Due to uncertainty in the actual condition of each pipe, there may be sections where complete replacement is required, which may increase or decrease the overall project cost.

- AM1: Reclaimed Water Chlorination Upgrades
 - It has been indicated that the Town would like to move away from the Chlorine gas injection system into reclaimed water at the WWTP and into a safer alternative. This project will likely be combined with Capital Projects 3.5.1.14 and 3.5.1.15.
- AM2: Kingfisher Foreshore Relining

- The existing AC trunk main along the lakeshore to the east of the MWWPS is nearing the end of its ESL.
- AM3: 89th Street Relining
 - Similar to the lakeshore relining the current pipe has adequate capacity, however, is at the end of its ESL. This project focuses on the pipe from 74th avenue to the MWWPS.
- AM4: 62nd Street Relining
 - This pipe capacity is acceptable however it is at the end of its ESL.
- AM5: Oleander/89th Street Relining
 - This pipe capacity is acceptable however it is at the end of its ESL.
- AM6: Harbour Key Upgrades
 - Various pipes near the Harbour Key lift station as well as the pipes along Harbort Key Drive require attention as they are at the end of their ESL.
- AM7: South Town Center Relining
 - This refers to the original AC pipe from the 1960's in the Town's core, south of the Highway. All pipes have adequate capacity.
- AM8: Osprey Place Relining
 - Pipes on and near Osprey Place require relining/replacement due to their ESL.
- AM9: North Town Center Relining
 - Similar to the South Town Center, this refers to the original AC pipe from the 1960's in the Town's core, north of the Highway. All pipes have adequate capacity.
- AM10: Hummingbird Lane Relining
 - Various pipes on and near Hummingbird Lane require relining/replacement due to their ESL.
- AM11: Alyssum Court Relining
 - Various pipes on and near Alyssum Court require relining/replacement due to their ESL.
- AM12: 92nd Ave Relining
 - Original pipes near and along 92nd avenue and Jubilee Drive are nearing their ESL.
- AM13: 89th Street North/Highway 3 Relining
 - This project includes the relining or replacement of original AC pipe located underneath Highway 3 and near the Buy-low Foods mall area.

In the 20-year horizon, the pumps in all of the pumps in the lift stations should be replaced as it has been assumed that pumps have a 20-year lifetime. The recommended order of replacement shown below, with the projects being prioritized based on capacity.

TABLE 4-2: LIFT STATION PUMP REPLACEMENT

	PUMP REPLACEMENT	CONSTRUCTION COST	INSTALLATION DATE
AM14	44 th Avenue Lift Station, Maple Drive Lift Station	\$300,000	2024
AM15	Border Lift Station, Cottonwood Lift Station	\$215,500	2026
AM16	Oasis Lift Station, Golf Course Lift Station	\$200,000	2030
AM17	Haynes Lift Station, Blue Blinker Lift Station, Lacey Point Lift Station	\$250,000	2032
AM18	All Lift Stations throughout the Northwest Sector	\$375,500	2035
AM19	MWWPS, Meadowlark Lift Station	\$300,000	2040
AM20	Smith and Loveless Lift Station, Gala Lift Station, Bayview Lift Station	\$194,000	2043
	<i>Total</i>	\$1,894,500	

It is definitive that there will be electrical upgrades required at multiple lift stations, however the exact replacement order and cost will depend highly on the outcome of the proposed SCADA System upgrades as discussed in Section 3.5.1.5. It is recommended that a work plan specific to the lift station electrical components be completed once the SCADA upgrades are done.

The forcemains out of the Blue Blinker lift station and the Maple Drive lift station near the end of the ESL in 2040. These projects may happen before the end of their ESL depending on the replacement of surrounding assets.

TABLE 4-3: FORCEMAIN REPLACEMENT

	PUMP REPLACEMENT	CONSTRUCTION COST	INSTALLATION DATE
AM21	Maple Drive Forcemain	\$54,000	2040
AM22	Blue Blinker Forcemain	\$54,500	2040
	<i>Total</i>	\$108,500	

No Lift Station Structures require replacement during the 20-year horizon.

The projected annual spending for Asset Replacement Projects over a 20-year horizon as per the modified work forecast is shown in Figure 4-2. Figure 1-1

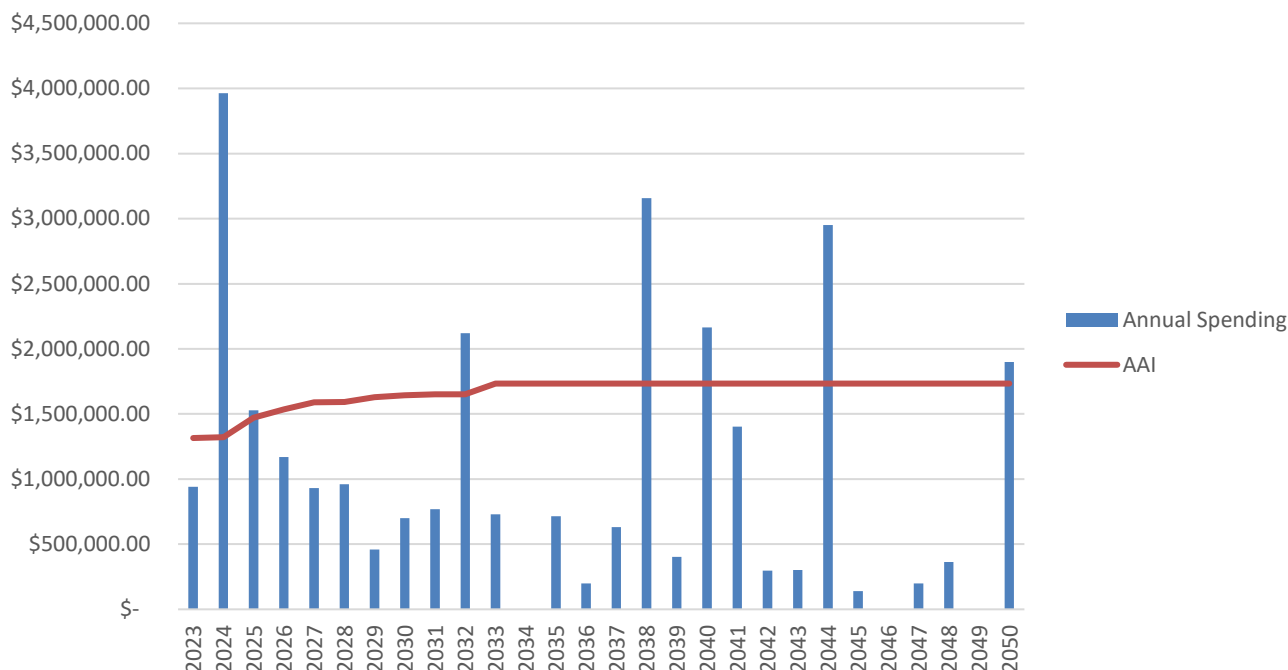


FIGURE 4-2: PROJECTED ANNUAL SPENDING RELATED TO ASSET RENEWAL WITH ADJUSTED WORKLOAD.

The large expenditure in 2024 is due to a slight backlog in projects that are required to be completed to enhance system performance and set up the system for further upgrades. The horizontal red line represents the unadjusted AAI value of \$1,326,271.

4.2 Capital Projects Average Annual Investment

The AAI for capital projects as described in Section 3.5.2 have been determined based on similar analyses discussed in Section 2.0 above.

- Replacement costs were determined based on similar projects with pricing including all manholes, services, and appurtenances.
- Once a certain project is complete, the Town begins collecting the AAI in the year following
- ESL values for linear assets were based on *BCGATCA* values.

The total cumulative AAI for the proposed capital projects is estimated at \$658,839, plus the AAI as outlined in Section 2.0. The total AAI values per Capital projects comprise multiple assets, however most have the same ESL. A detailed breakdown of costs is included in the appendices. It is recommended that the Town updates their AAI values gradually as the capital projects happen.

TABLE 4-4: AAI SUMMARY FOR PROPOSED UPGRADE PROJECTS.

PROJECT TAG	PROJECTS	TOTAL REPLACEMENT COST	AVERAGE ANNUAL INVESTMENT	INSTALLATION DATE	CUMULATIVE AAI
C5	SCADA System Upgrades	\$363,000.00	\$24,166.67	2024	\$ 203,967
C6	WWTP Cell 2 Desludging	\$2,900,000.00	\$145,000.00	2024	
C8	Manhole Smart Covers	\$181,000.00	\$9,062.50	2024	
C16	WWTP Auxiliary Power	\$218,000.00	\$8,700.00	2024	
C18	Magnolia Place Reconstruction	\$348,000.00	\$4,350.00	2024	
C19	WWTP Design and Planning	\$1,015,000.00	\$12,687.50	2024	
C3	Starlite Lift Station Relocation	\$3,625,000.00	\$45,312.50	2025	\$295,824
C7	Air Piping Updates at WWTP	\$1,450,000.00	\$18,125.00	2025	
C9	CCTV Program	\$139,000.00	\$13,920.00	2025	
C12	Golf Course Lift Station Screening	\$725,000.00	\$9,062.50	2025	
C13	Reclaimed WW Storage and Extension Study	\$109,000.00	\$5,437.50	2025	
C1	Cottonwood Upsizing	\$1,740,170.00	\$21,753.08	2026	\$613,598
C2	Maple Drive Upsizing	\$1,497,230.00	\$18,709.17	2026	
C11	Cell 1 MBBR Conversion	\$7,613,000.00	\$190,312.50	2026	
C14	Onsite Chlorine Generation	\$2,175,000.00	\$87,000.00	2026	
C4	Forcemain Extension near WWTP	\$290,000.00	\$3,625.00	2027	\$617,223
C10	Forcemain Odour Reduction	\$363,000.00	\$18,125.00	2028	\$644,339
C17	Wastewater Master Plan Review	\$45,000.00	\$8,990.00	2028	
C15	Chlorine Contact Chamber Recirculation	\$363,000.00	\$14,500.00	2029	\$658,839

Table 4-4 shows the capital project replacement costs and cumulative AAI forecast. For the initial 10 years there is a consistent increase in AAI due to the constant capital projects. Within 10 years, there is approximately \$25.2 million in capital projects which equates to an additional \$ 658.839 in AAI.

4.3 Reserve Balance Model

The reserve balance model was updated to show the range and increase of the monthly service fees with various scenarios. Below, a base case scenario is presented which represents the Public Sector Accounting board (PSAB) AAI values for the existing system and capital projects without considering an adjusted work forecast. This would be considered the maximum required AAI yielding the largest increase in MSF. In Section 4.3.2 the reserve balance model is updated to eliminate a deficit in 2023 and 2024. Additionally, there are three scenarios discussed with varying MSF and outcomes.

4.3.1 Capital Projects Reserve Balance Update

Three project categories have been discussed, with all funding stemming from different sources. The capital projects are assumed to be funded by capital funds (loans in most cases) with the asset management projects being funded from the Town's current reserve balance. A reserve balance model was developed considering the original system with capital projects. The initial 20 years of this reserve model can be seen in the figure below, with the full analysis graph in Appendix I. The capital reserve balance model was calculated similar to the asset reserve balance, with a few minor adjustments listed below.

- The AAI values were updated each year as capital projects were completed. The AAI for the completed project begins the year following project completion.
- The ESL for assets that are to be replaced multiple times throughout the analysis maintain the same ESL value throughout. However, there was no factor of risk taken into consideration. Considering risk could slightly increase or decrease the AAI value.
- The initial installation is considered to be funded by loans, which does not show the initial expenditure in the model. This results in the reserve balance increasing substantially, with a large expenditure at the end of the asset's ESL.

The figure below outlining the reserve balance shows a dip in reserve funds in the initial years. This is due to the large amount of replacement projects that are required up front. Eventually, the reserve climbs into the positives where it remains until major replacements are due.

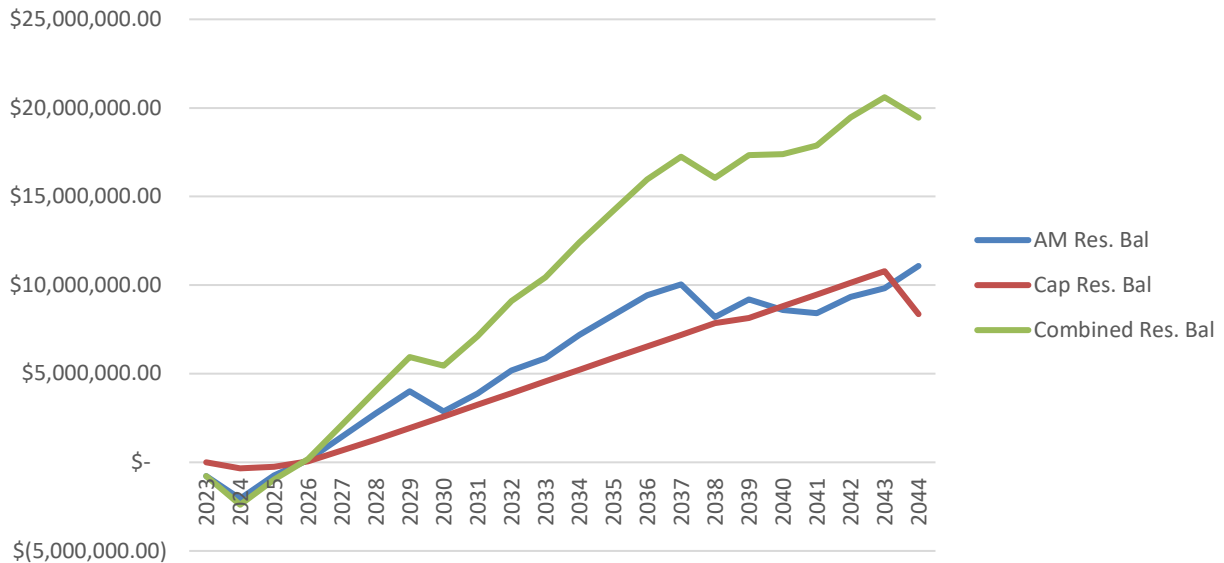


FIGURE 4-3: RESERVE BALANCE MODELS WITHOUT ADJUSTED WORKLOAD.

4.3.2 Work Forecast Reserve Balance Update

The work forecast reserve balance model updates the model to consider the proposed alternate work plan for the initial 20 years of the capital plan. This resulted in a reduced deficit in the initial years.

The 20-year forecast for the modified reserve balance is shown below with the full analysis in Appendix J. In this model, the workload has been adjusted and follows a plan as described in Section 4.1.1. By following the adjusted workload, the initial deficit is minimized, however there is still a heavy front-load of projects. The model considers three different scenarios:

- **Scenario 1: PSAB**
 - This case assumes the full AAI value is being collected for both replacement and capital projects. This is considered the upper bound of this analysis.
- **Scenario 2: Recommended Approach**
 - This scenario is recommended because it achieves the minimum sustainable MSF. This was achieved by holding the AAI for capital projects constant, and varying the AAI for replacement projects. The AAI for reserve projects was varied to achieve a minimum balance of \$0.00 at the trough, in approximately year 2032. The next time that the reserve balance oscillated below zero was beyond 2100 which was deemed acceptable. This scenario is recommended because it results in the minimum MSF while still protecting the Town's assets. This scenario does however result in a minor deficit from 2024 to 2028. This deficit could be avoided by a further increase in the MSF if the Town is interested.

- Scenario 3: Minimum Contribution
 - This scenario aims to achieve the minimum AAI value to minimize the MSF increase. Like Scenario 2, the AAI for capital projects was held constant and the AAI for current assets was varied. This Scenario assured that the combined reserve balance for all AM and Capital projects does not go below \$0.00 in 2032.

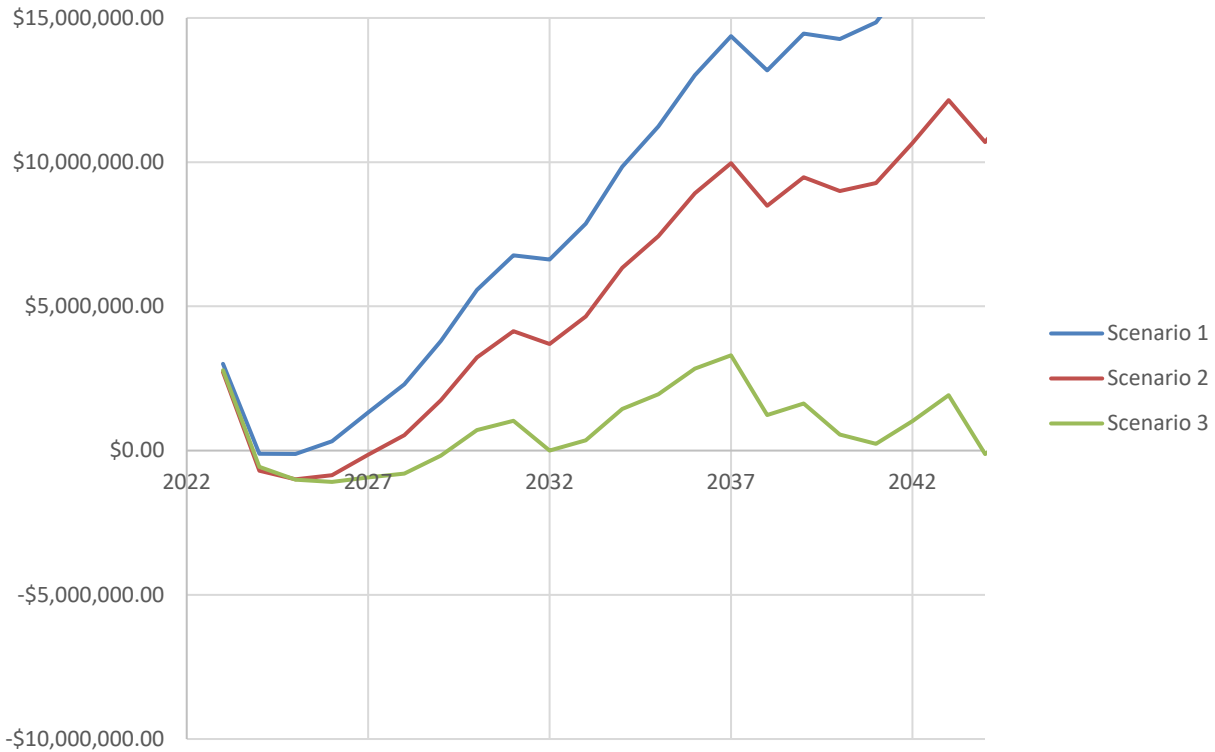


FIGURE 4-4: COMBINED RESERVE BALANCE WITH ADJUSTED WORK FORECAST.

Table 4-5 below presents the incremental MSF values for each scenario.

TABLE 4-5: CUMULATIVE MONTHLY SERVICE FEE INCREASE COMPARISON FOR CAPITAL PROJECTS.

YEAR	BASE CASE (PSAB)	SCENARIO 1	SCENARIO 2 (RECOMMENDED)	SCENARIO 3
2024	\$24.36	\$24.36	\$18.93	\$20.14
2025	\$28.14	\$28.14	\$22.71	\$20.14
2026	\$29.84	\$29.84	\$24.41	\$20.14
2027	\$35.72	\$35.72	\$30.30	\$20.14
2028	\$35.79	\$35.79	\$30.36	\$20.14
2029	\$36.29	\$36.29	\$30.87	\$20.14
2030	\$36.56	\$36.56	\$31.13	\$20.14
2031	\$36.56	\$36.56	\$31.13	\$20.14
2032	\$36.56	\$36.56	\$31.13	\$20.14
2033	\$36.56	\$36.56	\$31.13	\$20.14
2043	\$36.56	\$36.56	\$31.13	\$20.14

4.4 Capital Financing Model

A financing model was developed to represent borrowing funds for capital projects. The purpose of this model is to show the incremental increase in monthly service fees to support borrowing the funds. The financial model is shown below with the supporting calculations in Appendix K. The red represents the loan balance which decreases as payments are made, whereas the green trend represents the MSF required to maintain the loan payment. This was calculated by dividing the annual payment by 12 months, and the assumed 4500 service connections. The MSF is considered dynamic because initially as more projects are performed, the MSF increases incrementally, however once the loans are amortized, the MSF decreases incrementally as well.

The model was based on borrowing the total capital project values in the project installation year. Interest rates were based off the long-term lending rates reported by the Municipal Finance Authority of BC (MFABC). At the time of this analysis, an interest rate of 4.62% was reported by the MFABC for a 30-year loan period. This is the selected loan period because it will minimize the required increases in service fees. The MSF values are summarized in Section 4.5.

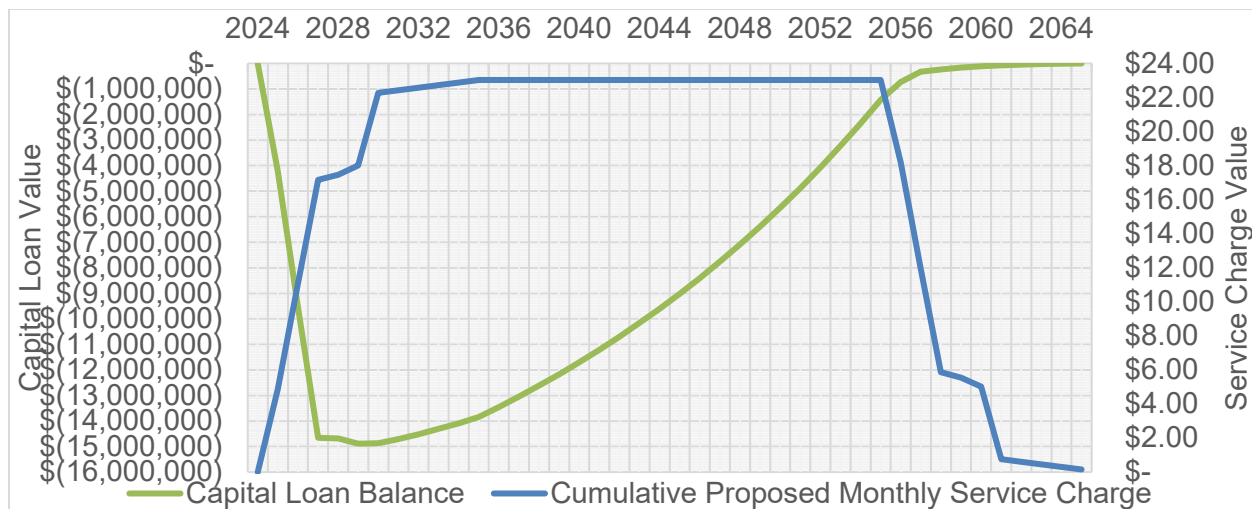


FIGURE 4-5: CAPITAL FINANCING MODEL.

4.5 Monthly Service Fee Summary

The recommended incremental MSF schedule is presented in Table 4-6. This considers the recommended Scenario 2 and the contribution for capital loan payments. The incremental fee decrease corresponding to capital loans coming to term are not considered because it is assumed fees will not decrease.

TABLE 4-6: RECOMMENDED INCREMENTAL MONTHLY SERVICE FEE INCREASE FOR 20 YEAR HORIZON

PG 13 14	SCENARIO 2	CAPITAL	TOTAL
2024	\$18.93	\$4.86	\$23.80
2025	\$22.71	\$11.07	\$33.78
2026	\$24.41	\$17.16	\$41.57
2027	\$30.30	\$17.47	\$47.77
2028	\$30.36	\$18.02	\$48.38
2029	\$30.87	\$22.28	\$53.14
2030	\$31.13	\$22.43	\$53.56
2031	\$31.13	\$22.58	\$53.71
2032	\$31.13	\$22.73	\$53.86
2033	\$31.13	\$22.88	\$54.01
2043	\$31.13	\$23.03	\$54.17

5.0 Conclusions and Recommendations

In summary, TRUE's recommendations are as follows:

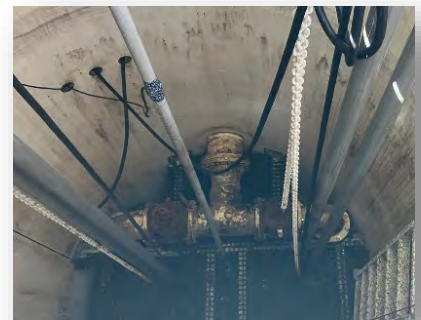
- Complete the recommended Capital and Renewal projects as outlined in Section 3.0 and 4.0 following a schedule that is close to as presented.
- Contribute the recommended AAI to the reserve balance to fund required projects and reserve balances as projects are completed.
- Research and apply for grants to assist in funding projects. This will minimize the amount of funding that will be required to come from the town, and in turn limit the annual spending.
- Update the reserve model annually as projects are completed, and as new projects arise.
- Implement the MSF schedule as presented in Table 4-6. This will ensure that the Town can fund the required capital projects to meet system performance and regulatory criteria, and fund ongoing replacement of infrastructure.

APPENDIX A

Referenced Reports

Starlite Lift Station

Relocation, Upgrade, and Development Feasibility



TRUE

ENGINEERING ■ PLANNING ■ URBAN DESIGN ■ LAND SURVEYING

June 2023

Project No. 302-2121

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Report Submission

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EGBC Permit to Practice
No. 1000129

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List of Acronyms

AAI	Average Annual Investment
AC	Asbestos Cement
AM	Asset Management
AWWA	American Water Works Association
DI	Ductile Iron
FRP	Fiberglass Reinforced Plastic
HDPE	High Density Polyethylene
HDD	Horizontal Directional Drilling
I&I	Inflow and Infiltration
LS	Lift Station
MWWPS	Main Wastewater Pumping Station
MMCD	Master Municipal Construction Documents
PE	Population Equivalent
PVC	Polyvinyl Chloride
PWWF	Peak Wet Weather Flow
OCP	Official Community Plan
OIB	Osoyoos Indian Band
SCADA	Supervisory Control and Data Acquisition
STEP	Septic Tank and Effluent Pump
TRUE	TRUE Consulting
VFD	Variable Frequency Drive
WWTP	Wastewater Treatment Plant

Units of Measure

ft	feet
lgpm	Imperial gallons per minute
km	kilometre
L/d	Litres per day
L/m	Litres per minute
L/s	Litres per second
lpcd	Litres per capita per day
m	metre
mg/L	milligrams per Litre
mm	millimetre
NTU	Nephelometric Turbidity Units
psi	pounds per square inch
USgpm	US gallons per minute

Referenced Reports

Capacity Assessment	Analysis of Trunk Main Sewer Capacity, “Capacity Assessment Sanitary Sewer Infrastructure” May 2014
Infrastructure Plan	Analysis of Sanitary Sewer System Capacity, “2023 Osoyoos Sanitary System Infrastructure Plan” DRAFT May 2023

Executive Summary

TRUE was engaged by the Town of Osoyoos to investigate the feasibility of upgrading, and/or relocating the existing Starlite lift station located at the north-west corner of 7710 Main Street. The original 1965 Starlite lift station has been replaced once in the same location and is approximately 29 years old. On the west side of 7710 Main Street is a narrow right-of-way that is constrained by the Holiday Inn on the west side, and the existing single-family home on the east side. During the last lift station replacement project, the existing 150mm forcemain was replaced with a newer PVC 250mm forcemain prior to the Holiday Inn being constructed.

Recently the property owner of 7710 Main Street has expressed interest in developing the property with higher density, triggering the Town's need to not only establish the impacts to the existing Starlite lift station, but to also assess the future impacts of the development. As the Holiday Inn restricts the workspace on the west side of the right-of-way, and a new structure would restrict the east side of the right-of-way, any future maintenance, repair, or replacement work within the right-of-way would be expected to be inaccessible.

To assess the future needs and the impacts of the proposed development of 7710 Main Street, TRUE has developed 5 scenarios as shown on Figure 1 in Appendix A with the following outcomes:

- Scenario 1: Maintain the existing lift station location for \$2.25-4.55M. (See Figure 2 in Appendix A)
 - Any equipment needed at the facility needs to be brought in along the beach
 - Trenchless pipe installation is proposed with a larger outer pipe casing and inner 250mm forcemain to allow the inner forcemain to be pulled out and re-installed as needed in the future at either end of the casing. Development would need to address pipe casing access needs
 - Sewage bypassing is required from the existing lift station to a proposed bypass manhole which forces sewage back into the existing forcemain downstream of the proposed works
 - Larger 67hp pumps on VFD's are proposed to meet the future PWWF
- Scenario 2: New lift station on the south-west corner of 7710 Main Street for \$4.79-\$7.09M. (see Figure 3 in Appendix A)
 - Access to the facility would be from Main Street.
 - Trenchless pipe installation is proposed with a larger outer pipe casing and inner 375mm deep sewer main to allow the inner pipe to be pulled out and re-installed as needed in the future at either end of the casing. Development would need to address pipe casing access needs.

- Sewage bypassing is required from the existing lift station to a proposed bypass manhole which forces sewage back into the existing forcemain downstream of the proposed works.
 - Additional public use land is required from 7710 Main Street.
 - A replacement lift station with 67hp pumps on VFD's is proposed to meet the future PWWF.
- Scenario 3: New lift station across Main Street from 7710 Main Street on Pioneer Walkway for \$3.43M. (see Figure 4 in Appendix A)
 - Access to the facility would be from Main Street and minimal impacts to Pioneer walkway once surface restoration is completed.
 - Trenchless pipe installation is proposed with a larger outer pipe casing and inner 375mm deep sewer main to allow the inner pipe to be pulled out and re-installed as needed in the future at either end of the casing. Development would only be potentially restricted near the existing lift station location to maintain access to the beach side of the pipe casing.
 - Sewage bypassing is required from the existing lift station to a proposed bypass manhole which forces sewage back into the existing forcemain downstream of the proposed works.
 - A replacement lift station with 44hp pumps on VFD's is proposed to meet the future PWWF.
 - Main Street is expected to be restricted to single lane alternating traffic during construction.
 - Scenario 4: New lift station west of 7305 Main Street for \$3.12-3.7M. (see Figure 5 in Appendix A)
 - Access to the facility would be from Main Street and minimal impacts to Pioneer walkway once surface restoration is completed.
 - Traditional construction methods utilized.
 - Sewage bypassing is limited to short periods of time for tie-ins of existing pipes
 - A replacement lift station with 54hp pumps on VFD's is proposed to meet the future PWWF.
 - Main Street is expected to be restricted to single lane alternating traffic during construction.
 - Scenario 5: New lift station on the North-west corner of 6901 Main Street for \$3.97-4.56M. (see Figure 6 in Appendix A)
 - Access to the facility would be from Harbour Key Drive
 - Traditional construction methods utilized.
 - Sewage bypassing is limited to short periods of time for tie-ins of existing pipes
 - A replacement lift station with 54hp pumps on VFD's is proposed to meet the future PWWF.
 - Main Street is expected to be restricted to single lane alternating traffic during construction but given the greater length of sewer main to be installed, the duration of traffic restriction would be greatest of all the scenarios.

Of the five scenarios, the Town would benefit longer term if Starlite is relocated beside Main Street to allow operators, maintenance crews and other parties easy access to the lift station in the event of planned or unplanned visits. Scenarios 4 and 5 would offer greater flexibility for future improvements by eliminating the narrow pipe right-of-way on 7710 Main Street while Scenario 1 and 2 could present property access challenges. To mitigate the risk of constrained property access for Scenario 1 and 2, 7710 Main Street could be purchased at a cost likely between the BC Assessment value of \$941,900 and the current MLS listing (June 2023) of \$2.3 million. The purchase of the land would address access issues and could be developed further to encourage greater public access to the north beach. As a compromise to scenario 1 and 2, Scenario 3's placement of Starlite on Pioneer walkway improves access off Main Street but requires the existing pipeline right-of-way to be maintained.

The final choice for the Starlite lift station requires a multitude of tools to assist the decision-making process. If the multivariable risk assessment tool is used, Scenario 4 offers the lowest risk and focuses primarily on traditional methods of construction. Following scenario 4 would be Scenario 1 and 5, followed by Scenario 3 and lastly Scenario 2. If capital costs are the key driving tool in the decision process, scenario 1 would offer the lowest capital cost but existing residual risks would remain. Other tools including community acceptance, local property owner biases, mayor and council opinions, Town staff opinions and other variables should also be considered in conjunction with the multivariable risk and capital cost tools outlined in this report.

The next step for the project would be selection of the chosen scenario followed by a formal geotechnical investigation to better understand the detailed geology of the area. If the soils are found to have a strong connection with Osoyoos Lake, dewatering would likely become challenging and timing the work during low lake levels would be essential.

1.0 Introduction

In response to the pre-development application for 7710 Main Street (Lot 6, KAP3531, DL 40 SDYD) for a proposed seven story residential apartment, TRUE has prepared this report. This report outlines our findings with respect to the existing Starlite lift station located near the north-west corner of the subject parcel for development, the impacted utilities located on a statutory right-of-way running along the west portion of the property and scenarios to replace the lift station in a new location. TRUE's opinion of the probable capital cost along with a decision risk-based assessment was completed to aid the Town with the decision making process.

2.0 Background

Several historical documents were reviewed during TRUE's assessment of the Starlite lift station including the following:

- The original Starlite lift station was constructed in 1965 along the north property line of 7710 Main Street and was constructed out of steel with a service life of 30 years before structural integrity was compromised.
- In 1994, the Starlite lift station was replaced to the south of the 1965 lift station with a premanufactured fiberglass structure.
- The Starlite sewage forcemain was replaced and upsized (150mm Asbestos Concrete to 250mm PVC) in 1997 from Osoyoos Lake to Main Street within the statutory right-of-way.
- Recent events captured in a TRUE Letter RE: 2007 Starlite Lift Station and Lot 6 Plan 3351 Owned by Mr. Keith Primeau:
 - In the summer of 1997, the owners of 7710 Main Street inquired about relocating the Starlite lift station to the south-west corner of the parcel, nearest to Main Street. As there was no benefit to the Town, the landowner did not pursue the relocation further.
 - From 1997 to 2007, significant growth occurred as a result of Osoyoos Indian Reserve #1 and in the east sector of town that relocation of the lift station was deemed impractical.
 - Two options were discussed including the south-west corner of 7710 Main Street or across Main Street on Pioneer Walkway with the walkway being preferred by the landowner.
 - The right-of-way along 7710 Main Street would make gravity sewer conveyance difficult due to the close proximity of building foundations, high groundwater level and reports of blasted rock in Main Street's (Highway 3) sub-grade.
- 2022 Flood Mitigation Plan prepared by TRUE in 2022: Starlite's access to the lift station should be placed at or greater than 279.5m for a flood return period of 50 years. To

address the risks of flooding, the report outlined that the facility risks could be mitigated by either relocation or flow control implementation. Flow control mitigation measures would be intended to manage inflows and infiltration into the submerged gravity conveyance system through the installation and closure of plug valves in key areas during the flood event. Relocation would place the assets at higher ground and away from the source of flood water.

Further to these previous report outcomes, the current lift station has been in service for 29 years and has considerable challenges relating to future upgrades to overcome any sanitary sewer flow increases. Currently the main access point would be through the right-of-way on 7710 Main Street which is only accessible for persons only, or along Osoyoos Lake's beach for equipment or persons. Depending on the nature and size of any new development on 7710 Main Street, repairs, rehabilitation, or replacement of pipelines or the lift station could be difficult to complete due to limited workspace, restrictive access, and shallow water levels.

2.1 Design Requirements

As part of this investigation, several requirements were identified as follows:

- Town of Osoyoos Subdivision and Development Servicing Bylaw 1100
 - Minimum grades from 0.14% to 0.40% were maintained from sizes 400mm to 200mm, respectively.
 - Minimum pipe size 200mm for gravity conveyance.
 - Manhole drop of 25mm and 50mm for horizontal alignment changes of less than 45 degrees and greater than 45 degrees but less than 90 degrees, respectively.
 - Mannings coefficient of 0.012 for PVC gravity pipe
 - Forcemain minimum velocity 0.75 m/s.
 - Hazen-Williams coefficient of 120 for worse case scenario and AWWA M32's coefficient (130-150) for best case scenario.
- MMCD Design Guidelines:
 - Gravity sewers to be designed up to 80% full (depth / pipe diameter)

Upon review of the 2023 Sanitary Sewer System Capacity and Capital Plan (Draft) prepared by TRUE, along with the previous 2014 Capacity Assessment Sanitary Sewer Infrastructure prepared by KWL, several challenges arise with lift station capacity vs PWWF as illustrated in Table 2-1 with the location of each facility shown on Figure 1 in Appendix A.

TABLE 2-1: EXISTING LIFT STATION CAPACITY AND PWWF

FACILITY	PUMP ON DUTY CYCLE	TRUE 2023 EXISTING PWWF (L/S)	TRUE 2023 FUTURE PWWF (L/S)	KWL 2014 EXISTING PWWF (L/S)	KWL 2014 FUTURE PWWF (L/S)	KWL 2014 LS STATED CAPACITY (L/S)
Starlite LS	1x Flygt 20hp NP3153 MT 433	59.2	81.7	42.3	50.3	65
44 th Ave LS	1x Flygt 25hp NP3171 HT 455	30.5	36.5	19.5	23.9	40
S&L LS	1x Flygt 20hp NP3153 MT 433	37	37.2	36.4	42.5	36.4

As shown in Table 2-1 the existing and particularly, the 2023 TRUE PWWF as modelled is expected to exceed the capacity of the lift stations as shown in red writing. The Smith & Loveless (S&L) lift station is identified as under capacity and the pumping capacity should be confirmed before any upgrades are planned. As the S&L, 44th and Starlite lift stations share a forcemain, the capacity of each lift station generally depends on the number of pumps operating along with the location of lift station along the forcemain. Capacity of each lift station would be discussed in the following section.

3.0 Conveyance Assessment

Following the historical path of the Starlite lift station and developing some new concepts to address development at 7710 Main Street, several scenarios were evaluated as follows:

- Scenario 1: Maintain the existing lift station location.
- Scenario 2: New lift station on the south-west corner of 7710 Main Street.
- Scenario 3: New lift station across Main Street from 7710 Main Street on Pioneer Walkway.
- Scenario 4: New lift station west of 7305 Main Street.
- Scenario 5: New lift station on the North-west corner of 6901 Main Street.

Each scenario was evaluated for conveyance requirements to direct flows to the lift station and pumping capacity with one to three lift stations operating (Starlite, 44th and S&L).

3.1 Scenario 1: Maintain the Existing Lift Station Location

For this scenario, the existing lift station would be maintained in its current location with all existing gravity mains remaining the same as shown on Figure 2 in Appendix A. To establish the capacity of the lift station, several drawdown-based pump tests were conducted. These tests were conducted on June 2, 2023 with a test average of 62.4 L/s with only Starlite operating and 54.6 L/s with three facilities (Starlite, S&L and 44th) operating together at once. When the field results are compared to the design work, the existing 20hp Flygt pump real life results were well within the design flow range for best and worst case as shown in Table 3-1.

Continuing with the same assumptions of best and worst case proven by real life testing, Table 3-1 shows the results with Starlite's previously stated capacity of 65 L/s. Under closer examination, Table 3-1 shows the 65 L/s stated capacity is generally within the best and worse case bounds of normal operation. Table 3-1 also illustrates that only Starlite will need upgrading as its current capacity does not meet future PWWF as highlighted in red.

TABLE 3-1: EXISTING LIFT STATION CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 20hp NP3153 MT 433	60.1-66.6	60.1-68.2	53.8-60	81.7
44 th Ave LS	1x Flygt 25hp NP3171 HT 455	47.2-53	42.3-50.9	38.3-44.1	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	53.1-60.5	49.5-55.7	37.2

To enable growth to occur, additional horsepower is needed to force higher sewage flows down the common forcemain. To achieve this, a single 67hp Flygt pump is proposed (bolded writing) with resultant flows shown in Table 3-2.

TABLE 3-2: EXISTING LIFT STATION WITH UPGRADED CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 67hp NP3202 HT 458	82*-91.3	92.7-106.9	86.9-96.8	81.7
44 th Ave LS	1x Flygt 25hp NP3171 HT 455	47.2-53	45.6-61.5	40.7-48	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	49.9-59.9	42-47.5	37.2

**Pump simulated to operate at reduced speed to illustrate lower flows and lower head requirements.*

As part of the design process, Figure 3-1 below illustrates the Starlite's Total Dynamic Head (TDH) from its current location to the Main Wastewater Pumping Station with one facility running and all three facilities (S&L, 44th Ave and Starlite) running at the same time.

The TDH chart is intended to show the pressure rise on the right of the chart, indicating the pump providing the head pressure. The colored line then slopes downward indicating internal pressure loss as sewage is pushed through the forcemain until the line crosscuts the pipe elevation where pressure in the pipe becomes atmospheric. At the point where pressure becomes atmospheric, the sewage would then flow by gravity (open channel flow) to its destination.

It should be noted that the slope of the TDH line often changes in its angle or slope due to the change in pipe diameter, or at points where flow rate changes due to 44th Avenue or S&L lift stations forcing additional flows into the common forcemain. Additionally, the pump's input head (where the line starts on the left hand side of the TDH chart) varies due to the hydraulically balanced pressure and flow balanced with the manufacturer's (Flygt) pump performance curve.

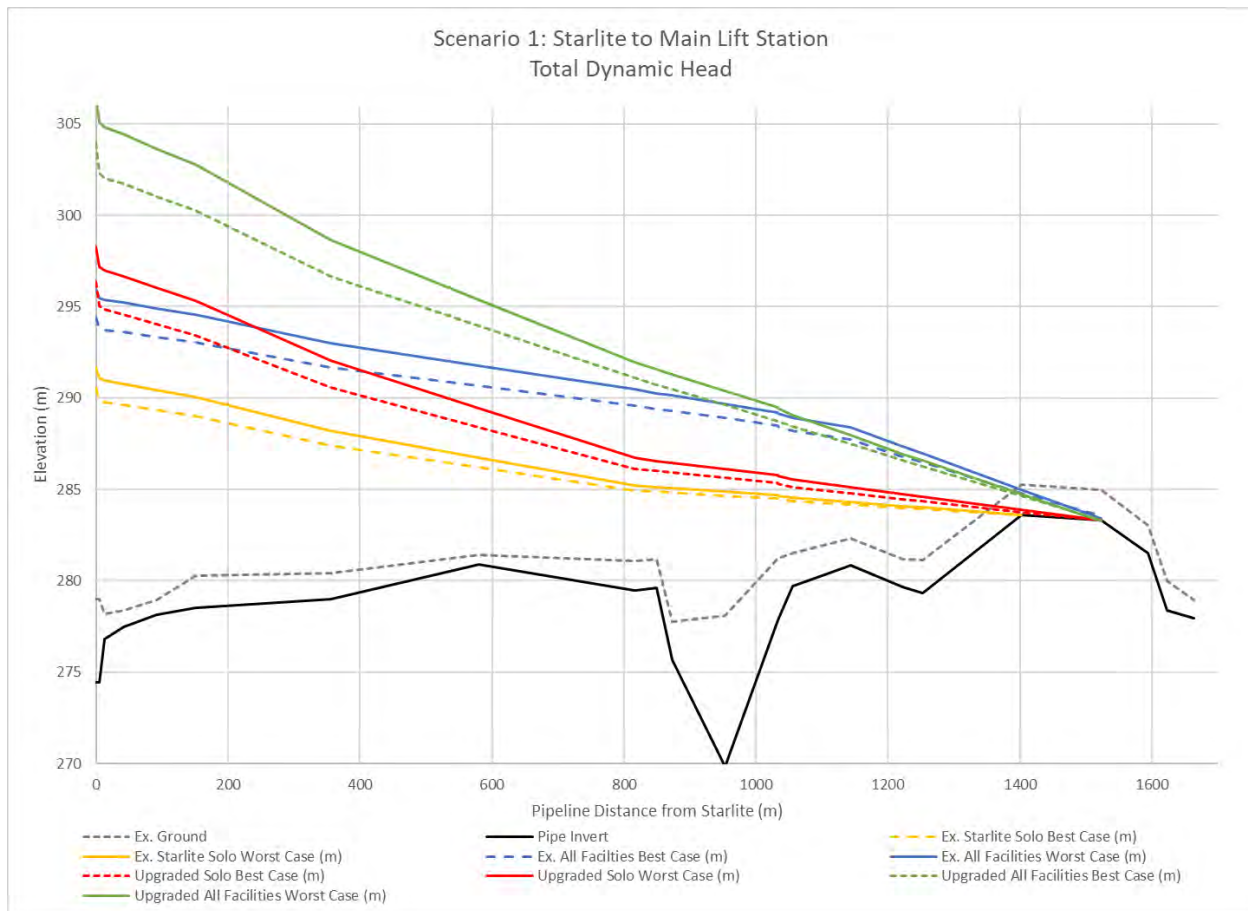


FIGURE 3-1: EXISTING STARLITE LIFT STATION LOCATION WITH EXISTING AND PROPOSED PUMPS

The discharge point of the effluent is expected to change as flows increase as shown on Figure 3-1 by noting the location where the TDH colored line connects with the pipeline elevation. Where the two lines intercept is the estimated location where pressurized flow changes to open channel (partially full pipe) flow. In this case, the switch from pressurized flow to open channel changes occurs at nearly 1.4 km from the existing Starlite lift station with one Starlite pump running, where as the rest of the scenarios increase the flow rate such that the open channel flow occurs at an estimated 1.5 km from the existing Starlite lift station.

Understanding that open channel flow is expected along Kingfisher Drive, the existing air valves should be confirmed prior to increasing any flow rate beyond the current arrangement as the increased sewage flow could cause air binding, surging, air efferece and related issues.

3.1.1 Scenario 1: Constructability

To address the existing 7710 Main Street right-of-way utility congestion, building foundations, and other utilities within the right-of-way, decluttering and removal would be beneficial to the Town if

repairs were to occur. Understanding the Town's budget is often constrained, removal costs were not included in the later cost estimates.

To facilitate the proposed larger development on 7710 Main Street, a larger pipe casing would be installed to house the new Starlite forcemain. The pipe casing would allow the Town to access the entirety of the internal forcemain by allowing removal of the internal pipe as needed, one pipe segment at a time from either side of the pipe casing without digging the length of the right-of-way. The pipe could be installed using traditional open cut trenching, but installation would be difficult with the existing home on 7710 Main Street on the east side of the pipeline and the Holiday Inn on the west side of the pipeline. To enable independent installation of the pipe on the right-of-way located within 7710 Main Street, trenchless technology has been assumed for installing the forcemain pipe casing. The first technology could be HDD, but would require the drill rig to be placed back from the start point of the pipe casing, and in this instance, would most likely put the drill rig into Osoyoos Lake. Although possible, a barge or fixed platform would need to be brought in to allow the HDD drill rig to sit back as required. Once the HDD is underway to do the drill and to install the pipe casing, additional risks could impact the aquatic environment due to the risk of leaking hydraulic fluids used by the drill rig and migrating drill mud used as part of the drilling process. As a result, guided pipe auguring or ramming is anticipated as the auger or air hammer could be dropped into an excavation near to the pipe entry and isolate the trenchless technology equipment from Osoyoos Lake. In this instance, several drillers expect that the drill would take 1-2 weeks to complete from buildup of the site to teardown.

Inside the larger pipe casing would be the 250mm diameter restrained forcemain pipe on low friction casing spacers. As this restrained pipe is segmented into pieces, the area needed to replace or repair the pipe would be limited to the beach area near Osoyoos Lake or near Main Street that would need to be incorporated into the approval process of any development on 7710 Main Street.

To manage risks of damaging existing pipes and allow for proper testing and tie-in of the new force main, a hot-tap bypass manhole structure along with isolation valve would be placed on the existing AWWA C900 PVC forcemain on the south-west corner of 7710 Main Street near Main Street. Conceptually, a pair of 75hp Godwin NC150S Dri-prime diesel pumps (one duty and one standby) would be placed near the existing lift station, air plugs placed into the pipe between the last manhole and Starlite wet well, and Godwin pump suction pipes dropped into the manhole. Once the manhole is surcharged, the duty Godwin pump would suck out the raw sewage and pump it around the trench work in an aboveground HDPE pipeline to the bypass manhole connecting to the existing downstream forcemain, allowing sewage to be returned on its path to the Main Street lift station. Once the bypass is underway, the wet well piping, valving and appurtenances could be repaired, rehabilitated, or replaced if budgets allow. At a minimum, the downtime would allow the proposed pumps to be installed along with new VFDs.

3.2 Scenario 2: Location A Replacement Starlite Lift Station

For this scenario, the existing lift station would be replaced and installed at the South-west corner of 7710 Main Street as shown on Figure 3 in Appendix A.

Like Scenario 1, a single 67hp Flygt pump is proposed (bolded writing) to achieve the target future wet weather flow as shown in Table 3-3.

TABLE 3-3: LOCATION A REPLACEMENT LIFT STATION WITH UPGRADED CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP(S) ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 67hp NP3202 HT 458	81.7*-121.5	81.7*-116.8	81.7*-94.7	81.7
44 th Ave LS	2x Flygt 25hp NP3171 HT 455	55.7-64.1	44-61.6	41.8-49.2	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	48.9-60	46.2-52	37.2

**Note: Starlite's replacement lift station is assumed to function under both wet well level and flow-based control logic*

As indicated in Table 3-3, 44th Avenue will require upgrading to include the 3rd pump as the shorter pipeline for Starlite increases the flow sufficiently to force 44th Avenue's pump to dip below the target future wet weather flow. It will be essential that the Starlite replacement lift station run under VFD control to not only save energy but slow the pump cycle along with improving the pump reliability.

Under this scenario, the 44th Avenue lift station with both pumps running does bring concerns of increased risks of clogging, increased energy consumption and reduced service life due to the new operating point with Starlite and S&L operating. If the Town would like to minimize these risks, a single Flygt 3202 may be a reasonable alternative but should be confirmed. Alternatively, evaluating the annual pump run hours throughout the year may show that this risk is limited to short periods of time in the summer combined with rainfall events. A simple upgrade to the PLC may alleviate unwanted operator call-outs due to clogging through implementing, if not already in place, advanced Flygt self-cleaning clog removal programming.

Figure 3-2 below shows the Starlite Total Dynamic Head (TDH) to the Main Street Lift Station with one facility running and the worst case of all three facilities (S&L, 44th Ave and Starlite) running at the same time.

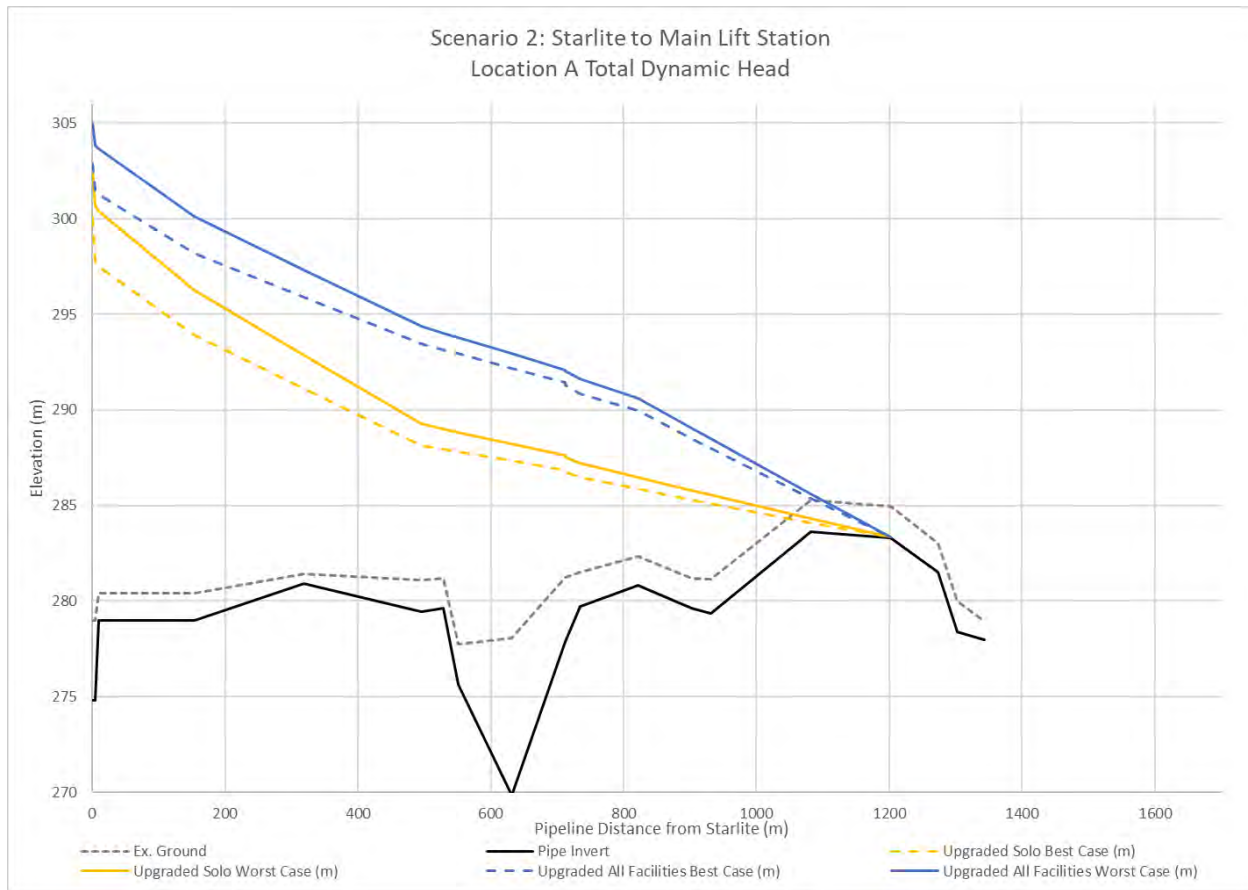


FIGURE 3-2: LOCATION A REPLACEMENT STARLITE LIFT STATION LOCATION WITH PROPOSED PUMP(S)

For this scenario, as shown in Figure 3-2 the point of switching from pressurized flow to open channel occurs at nearly 1.2 km from the proposed replacement Starlite lift station. As noted in scenario 1, at 1.2 km and onward, air management through the existing air valves along Kingfisher Drive should be confirmed for sizing when upgrading occurs.

3.2.1 Scenario 2: Constructability

For this scenario, constructability comments of Scenario 1 remain the same, but instead of a shallow forcemain, a 375mm (15") SDR35 PVC pipe would need to be installed upwards of 5.7m deep at a 2% grade to create a reasonable conservative slope for the trenchless pipe installers install the pipe casing for the host gravity pipe to be slide into the pipe casing. Unlike the forcemain, a stiff pipe casing would be required to ensure the internal gravity pipe is held at a constant grade from end to end of the casing. Additionally, the 2% slope of the inlet pipe to the lift station wet well would assist with conforming to guidance provided in the first edition AWWA E110-22 Solids Handling Pump standard along with other Hydraulic Institute standards.

Like Scenario 1, decluttering of the 7710 Main Street right-of-way of un-used assets could be beneficial to the Town by increasing accessibility of necessary utilities along with avoiding

confusion in the future for any repairs, rehabilitation or replacement projects. Understanding that Scenario 2 will take considerably longer to construct than Scenario 1, the Town could entertain the idea of connecting the bypass diesel pump manifold to the existing Startlite lift station. As HDPE is fully restrained, an aboveground HDPE pipeline would be more than capable of allowing the Startlite lift station to convey sewage in the temporary HDPE pipe during construction and only use the diesel pumps during tie-ins and periods of power loss. This approach would also reduce the carbon footprint of the project by only operating temporary diesel pumps when electrically driven pumps are not available.

The wet well is expected to have a finished floor elevation of 273.2m, or nearly 7.1m deep (assuming a ground elevation of 280.3m) plus additional depth for the concrete slab and any leveling granular material used. Understanding that blast rock has been found in the past in the highway (Main Street) road subgrade, a geotechnical investigation should be conducted. Open cut with shoring or sheet piles is probable as illustrated on Figure 3 in Appendix A or alternatively a method of a sinking caisson with hydrovac soil removal could be done. In either method, detailed survey, slope analysis, dewatering risks, and geotechnical will be essential as the depth of the wet well could prove to be impractical from either a constructability or cost prospective. Well point or deep well dewatering may be required if dewatering cannot be done conventionally. Additionally, with such deep depths and proximity to the lake, floatation of the wet well will need to be considered, and typically concrete will need to cast around the FRP wet well to resist such forces.

In any case, due to the depth of all the proposed assets, additional right-of-way lands on 7710 Main Street is needed to facilitate the installation of the lift station.

3.3 Scenario 3: Location B Replacement Startlite Lift Station

Scenario 3 builds on Scenario 2 but relocates the lift station across Main Street and onto Pioneer Walkway. Pioneer Walkway would be expected to only be impacted during construction and the original alignment can be restored when the new lift station is completed as shown on Figure 4 in Appendix A.

As noted previously in the report, a larger pump is required to meet future PWWF but given the shorter pipeline distance to the common forcemain, a single 44hp Flygt pump is proposed (bolded writing) to achieve the target future PWWF as shown in Table 3-4.

TABLE 3-4: LOCATION B REPLACEMENT LIFT STATION WITH UPGRADED CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP(S) ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 44hp NP3202 HT 462	81.7*-123.5	81.7*-108	81.7*-98	81.7
44 th Ave LS	2x Flygt 25hp NP3171 HT 455	55-7-64.1	46.2-64	41.6-49.2	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	49.5-60	46.3-52.1	37.2

**Note: Starlite's replacement lift station is assumed to function under both wet well level and flow-based control logic*

Following similar results from Scenario 2, higher flow rates coming from Starlite along with S&L lift stations causes the 44th Avenue lift station to operate in a zone where there are concerns of increased risks of clogging, increased energy consumption and reduced service life. As noted in Scenario 2, different pumps or advanced VFD control logic would aid in reducing the number of callouts to the Town's operators to remove blockages.

Figure 3-3 below shows the Starlite Total Dynamic Head (TDH) to the Main Wastewater Pumping Station with one facility running and the worst case of all three facilities (S&L, 44th Ave and Starlite) running at the same time. As noted in Scenario 1 & 2, the TDH curve is hydraulically balanced with the manufacturer's (Flygt) pump performance curve and the total system pressure losses.

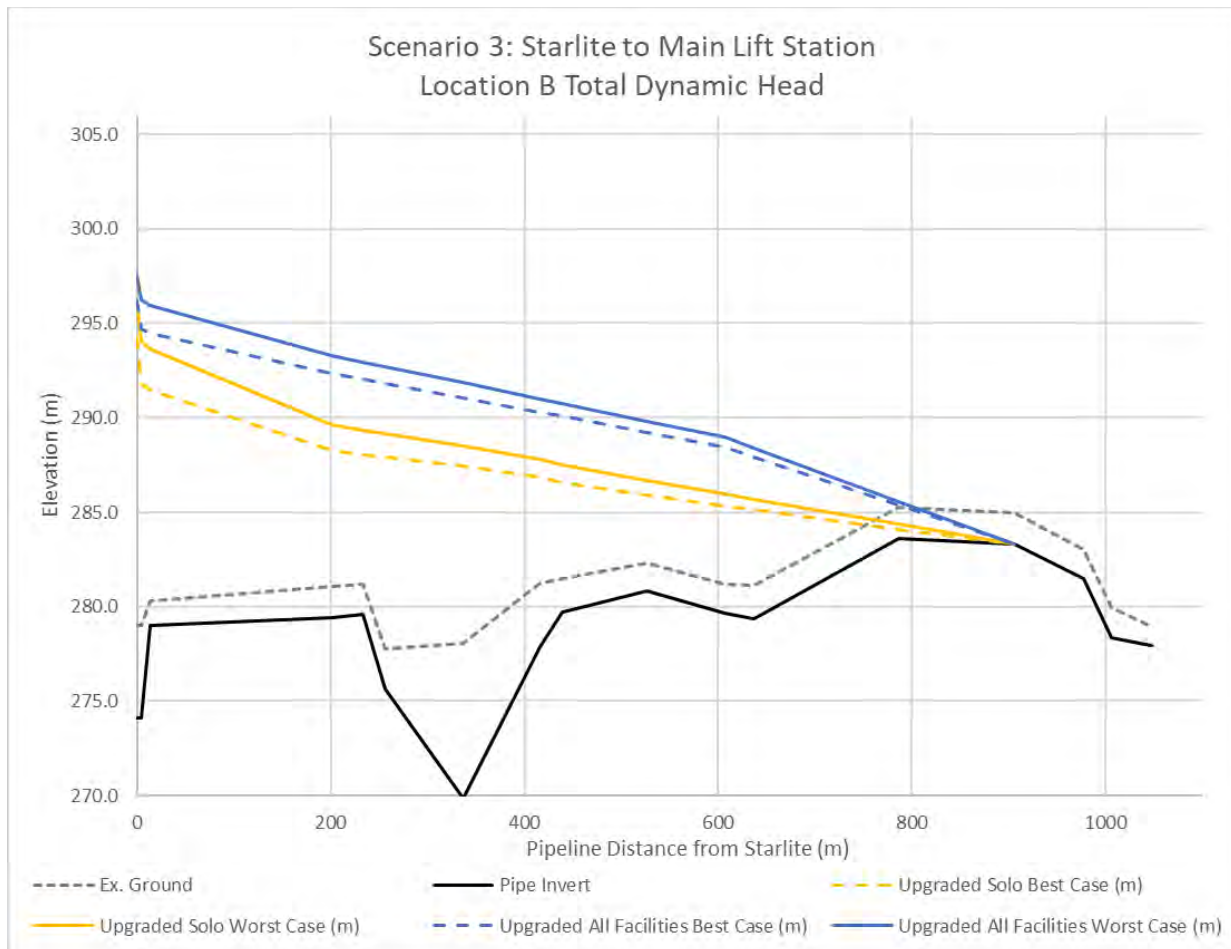


FIGURE 3-3: LOCATION B REPLACEMENT STARLITE LIFT STATION LOCATION WITH PROPOSED PUMP(S)

Figure 3-3 illustrates that the discharge location is approximately 900m from the proposed replacement Starlite lift station and does not change as a result of Starlite or all lift stations operating making future air management assessment straight forward. Like Scenario 1 and 2, the existing air valves along Kingfisher Drive should be confirmed for sizing as the increased flow rate could cause air binding, surging, air efference and related issues, slowing the flow rate for all lift stations.

3.3.1 Scenario 3: Constructability

For this scenario, constructability risks of Scenario 2 remain the same, but the length of the gravity pipeline grows to extend to pioneer walking trail on the opposite side of Main Street, bringing the depth of burial to an estimated depth of 6.4m. As noted in Scenario 2, the 2% grade for the trenchless pipe installation remains the same for the same reasons as Scenario 2, but may require sheet piling or shoring to maintain traffic on Main Street after the risks are assessed in the future geotechnical investigation.

Like Scenario 1, decluttering of the 7710 Main Street right-of-way would be the ideal outcome pending budgets, and Scenario 2 type temporary sewage bypass pumping is needed.

The wet well is expected to have a finished floor elevation of 272.5m, or estimated at nearly 8m deep plus additional depth for the concrete slab and any leveling granular material used. Like Scenario 2, blast rock, dewatering risks, flotation, and safe working slopes would need to be addressed during detailed design and post geotechnical investigation.

3.4 Scenario 4: Location C Replacement Starlite Lift Station

For this scenario, the existing lift station would be replaced and installed on Pioneer Walkway to the west of 7305 Main Street in close proximity to the existing public use gazebo as shown on Figure 5 in Appendix A.

For this scenario, a single 54hp Flygt pump is proposed (bold writing) to achieve the target future wet weather flow as shown in Table 3-5.

TABLE 3-5: LOCATION C REPLACEMENT LIFT STATION WITH UPGRADED CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP(S) ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 54hp NP3202 HT 460	81.7*-117.1	81.7*-112.8	81.7*-99.5	81.7
44 th Ave LS	2x Flygt 25hp NP3171 HT 455	55.7-64.1	44.2-64	39.8-49.3	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	48.5-60	45.1-50.8	37.2

**Note: Starlite's replacement lift station is assumed to function under both wet well level and flow-based control logic*

Following similar results from Scenario 3, the higher combined sewage flows with Starlite and 44th Avenue lift stations operating, the 44th Avenue lift station with both pumps running does bring concerns of increased risk of clogging, increased energy consumption and reduced service life. As noted in Scenario 3, different pumps or advanced VFD control logic would aid in reducing the number of callouts to the Town's operators to remove blockages.

Figure 3-4 below shows the Starlite Total Dynamic Head (TDH) to the Main Wastewater Pumping Station with one facility running and the worst case of all three facilities (S&L, 44th Ave and Starlite) running at the same time.

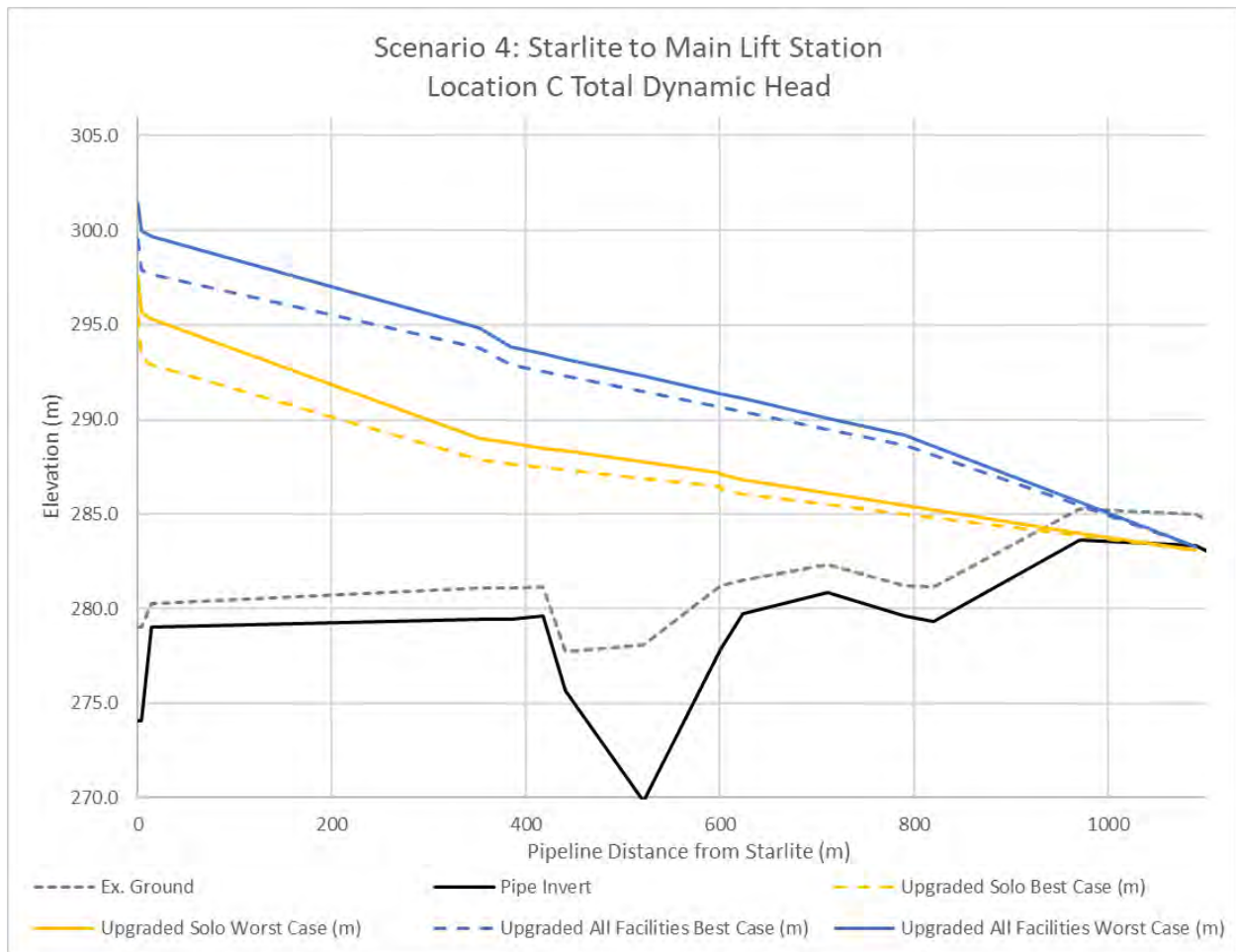


FIGURE 3-4: LOCATION C REPLACEMENT STARLITE LIFT STATION LOCATION WITH PROPOSED PUMP(S)

Following the same trend as Scenario 3, Figure 3-4 shows that one or all pumps in operation does not change for detailed design air management. Like other scenarios, as expected, air valves along Kingfisher Drive should be confirmed for sizing during detailed design.

3.4.1 Scenario 4: Constructability

For this scenario, the pipe alignment routing would differ from Scenario 1 through 3. To facilitate flow from 7906 Main Street (Holiday Inn & Suites) to 7310 Main Street (Desert Surfside) for reverse flow to the new lift station location, several options present itself.

- Option 1: Conveyance of wastewater by conventional 200mm PVC gravity main, starting at a depth of 1m burial. The pipe main and manholes would be placed along the beach to the north or south of the existing main. Challenges would include groundwater, working near water, environmental concerns and reconnecting existing sewer services which may require regarding of pipes back to the building or even pumping.
- Option 2: Gravity conveyance like Option 1, but with the exception that the pipe would be placed in the shoulder or back of curb of Main Street. This option would remove the environmental risks of working beside the lake but may still have dewatering challenged along with traffic control as Main Street would be impacted. Rerouting of building services may be challenging as services would need to be routed from the rear of the building to the front of the property.
- Option 3: Low pressure sewer could be used. This method would typically utilize septic tanks to collect solids, and clear effluent would pass onto a downstream pump tank with an effluent pump. The effluent pump would send wastewater down a smaller service pipe to the common forcemain in the street or beach. Routing of the service line would be simple as design grades are less critical than gravity lines any may be a good alternate if one or more properties cannot be connected by gravity if gravity flow is selected as the primary connection method. Challenges to low pressure sewer include locating the spot for the solids collecting septic tanks, ensuring property owners pump out their tanks on a normal frequency, and the property owners are open to installing, maintaining, and servicing their own septic tanks and effluent pump (often 2hp or less).

In any of the optional pipe routes to service the properties from 7906 Main Street to 7310 Main Street, sewage generated from the north-east side of Osoyoos including Osoyoos Indian Band IR #1, would need to be redirected from the existing Starlite lift station towards the new proposed replacement Starlite lift station location to the west of 7305 Main Street. To reverse flow, the existing pipe main running along the beach would be redirected into a new 375mm (15") pipe running between 7308 Main Street and 7310 Main Street, within the existing public corridor between the two privately owned parcels. Open cut trenching along with trench boxes is anticipated as space on the south side of Main Street is limited at best. Trenchless pipe installation was not suggested due to the cost but could be added if geotechnical or detailed design require this technology.

Once the new reverse flow main is across main street (as shown on Figure 5 in Appendix A) and in front of 7305 Main Street, the pipe would turn 90 degrees to run along Main Street's shoulder to stay away from the congested zone near the sidewalk of utilities including gas, sewage forcemain, water pipes, and other utilities. To the west of the existing public washroom parking lot and the east of the public gazebo would be the new replacement Starlite lift station.

The wet well is expected to have a finished floor elevation of 273.5m (shallowest of all options) or estimated at nearly 7m deep plus additional depth for the concrete slab and any leveling granular material used. Like Scenario 2 and 3, blast rock, dewatering risks, flotation, and safe working slopes would need to be addressed during detailed design.

The advantage of this scenario is that sewage bypassing would be limited to hours, not days or weeks at a time, reducing risks of vandalism along with around the clock monitoring. It would be anticipated that most of the assets could be installed, tested, and commissioned with fresh water prior to switching to the new system. Bypassing would likely be limited to tying in the existing and proposed sewage pipe mains.

3.5 Scenario 5: Location D Replacement Starlite Lift Station

For this scenario, the existing lift station would be replaced and installed on the northwest corner of 6901 Main Street as shown on Figure 6 in Appendix A. To allow future PWWF's to be pushed to the Main Wastewater Pumping Station, a 54hp Flygt pump is proposed (bolded writing) as shown in Table 3-6.

TABLE 3-6: LOCATION D REPLACEMENT LIFT STATION WITH UPGRADED CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP(S) ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 54hp NP3202 HT 460	81.7*-103.3	81.7*-110.8	81.7*-91.8	81.7
44 th Ave LS	2x Flygt 25hp NP3171 HT 455	55.7-64.1	30.9-64	30.1-36.2	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	49.5-60	47.6-53	37.2

**Note: Starlite's replacement lift station is assumed to function under both wet well level and flow-based control logic*

Following similar results from Scenario 4 and increased working pressures in the common forcemain, the 44th Avenue lift station with both pumps running does bring concerns of inadequate future PWWF capacity, increased risks of clogging, increased energy consumption and reduced service life. To address 44th Avenue's deficiencies as shown in Table 3-6 with red writing, a single VFD controlled 54hp pump, identical to the Starlite replacement lift station would meet the local target future PWWF.

Figure 3-5 below shows the Starlite Total Dynamic Head (TDH) to the Main Wastewater Pumping Station with one facility running and the worst case of all three facilities (S&L, 44th Ave and Starlite) running at the same time.

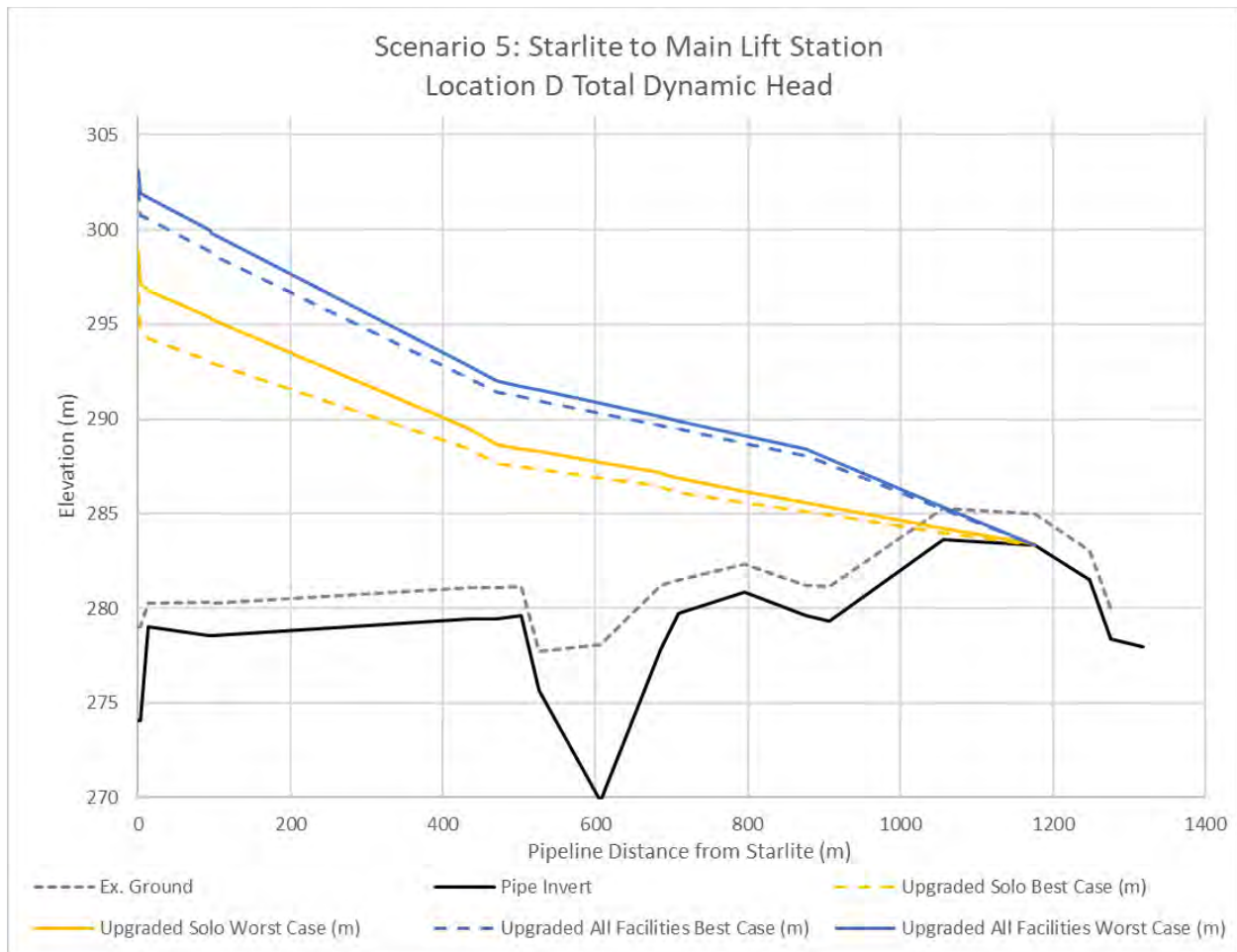


FIGURE 3-5: LOCATION D REPLACEMENT STARLITE LIFT STATION LOCATION WITH PROPOSED PUMP(S)

Also shown on Figure 3-5, the point of switching from pressurized flow to open channel changes is consistent like Scenario 2 through 4, making future air management design straight forward. The existing air valves along Kingfisher Drive should be confirmed for sizing as the increased flow rate could cause air binding, surging, air efferece and related issues, slowing the flow rate for all lift stations.

3.5.1 Scenario 5: Constructability

For this scenario, the pipe alignment routing would be identical to Scenario 4 for the beach properties from 7906 Main Street (Holiday Inn & Suites) to 7310 Main Street (Desert Surfside) to reverse flow to the new lift station location. The choice of gravity or low-pressure sewer would remain the same as Scenario 4 along with the redirected new 375mm (15") pipe running between 7308 Main Street and 7310 Main Street, within the existing public corridor between the two privately owned parcels. Open cut trenching with trench boxes is anticipated like Scenario 4.

Once the new reverse gravity flow main is across Main Street and in front of 7305 Main Street, the pipe would turn 90 degrees to run along Main Street's shoulder to stay away from the congested zone near the sidewalk of utilities including gas, sewage forcemain, water pipes, and other utilities. The proposed gravity sewer main would move east towards 6901 Main Street to a shared manhole with a new trunk main crossing Main Street south down Ponderosa Drive.

The sewage flows south of Cottonwood Drive running along the lake would be collected along Maple Drive as shown on Figure 6 in Appendix A. In collaboration with the recent sewage master plan, the gravity sewage main would be upsized to 300 (12") from

- Poplar Court to Cottonwood Drive (heading north-west)
- Along Cottonwood Drive between Maple Drive and Jackpine Lane
- New main constructed to cross Cottonwood Drive along Ponderosa Drive, with the new pipe placed to the south of the existing sewage pipes and manholes.
- New main would follow Ponderosa Drive south and terminate at the common manhole on the south side of Main Street in front of 6901 Main Street
- A new main heading south from the junction of the two new gravity pipe mains into 6901 Main Street where the new replacement Starlite lift station would be constructed

The relocation of the Starlite lift station to the east of the existing Starlite lift station presents challenges due to added lengths deepening the depths of the main, reaching nearly 6m deep. At this depth, the most practical option is to either use trench boxes, or shoring. For the Ponderosa and Main Street pipe crossing, a local detour along Harbour Key Drive would provide flexibility in timing for the Contractor.

The wet well is expected to have a finished floor elevation of 272.9m or estimated at nearly 5-7.5m deep (survey needed to confirm as ground drops off from Main Street's elevation) plus additional depth for the concrete slab and any leveling granular material used. Like Scenario 2 to 4, blast rock, dewatering risks, flotation, and safe working slopes would need to be addressed during detailed design.

Like Scenario 4, sewage bypassing would be limited to short durations rather than long periods of time as explained in Scenarios 1 to 3.

It should be noted that the work along Main Street from 7305 Main Street to 6901 Main Street may need to be delayed until the replacement Starlite lift station is operational with limited inflows as a full bypass may be needed for low sewage generation draining down Harbor Key Drive. This delay is likely due to the new pipe main installation likely undermining the existing manholes.

4.0 Transient Analysis

To meet the future PWWF, sewage flows would need to increase, and this section is intended to outline the risks and mitigation measures to offset the increased sewage flows. The existing common forcemain for the Starlite, 44th Avenue and S&L LS primarily consists of:

- 250mm, 3000mm AWWA C900 PVC Pressure PC100 (DR41)
- 350mm AWWA C906 HDPE, DR17
- 200mm Coated AWWA C200 Steel Pipe (assumed Schedule 40)
- The piping likely includes FRP and Asbestos concrete pipe, where the records do not provide sufficient information to verify their performance.

4.1.1 Transient Surge

Transient surges are typically a result of either a valve closing too quickly with sewage being pumped or a lift station losing power before backup generator can restart the facility. The type of valve, pump start/stop method and process design can impact the severity of the transient surge.

In review of all the scenarios, the transient surges are expected to be in the order of 120-130psi on PVC/ HDPE pipes and approaching 760psi on steel pipes. Although the pipes technically can withstand the transient surge pressures, new measures would be suggested particularly as the strength of the asbestos concrete and FRP pipe is not well understood.

4.1.2 Mitigation Measures

The common forcemain has few existing buried slow closing valves which should be maintained to ensure inadvertent closure of a valve by operators.

Inside the wet wells and valve chambers, the current quarter turn valves should have either labels placed near the valves warning the operators to not close them while pumps are operating or add gearboxes to mechanically force the valve closure to be slow.

The existing check valves appear to be the typical HDL/Flygt ball check valves which when combined with the current across-the-line starting (on-off pump cycling), can lead to considerable transient surges. Pressure monitoring would be suggested at the existing lift stations to see if check valves should be changed to a lower surge type or VFD's installed. These pressure gauges use an isolated diaphragm to avoid the typical solids plugging of the conventional pressure gauge. The added benefit of installing these gauges would be the ability of operators, maintenance or consultants to witness pressure characteristics during operation to assist with troubleshooting problems or monitoring pumps for wear to schedule pump repairs.

For the replacement Starlite lift station, the mechanical design is expected to include AWWA C520 slow closing knife gate valves, AWWA C517 buried service plug valves, anti-surge type AWWA

C512 solid float combination air valve with air accumulator chamber, AWWA C504 check valves and variable frequency drives on each AWWA E110 submersible pumps.

5.0 Probable Capital Costs

TRUE has prepared our opinion of probable capital costs based on projects of similar nature and of similar scope. The values suggested in these estimates were prepared with site information available at the time and based on probable conditions affecting the project. The estimates are intended to indicate an approximate magnitude of cost used for planning, assists with establishing a more specific project definition and used for preliminary project discussion for long term capital plans.

As part of the probable capital cost estimate preparation, TRUE included the construction risk, construction contingency and an allowance for engineering fees was incorporated.

5.1 Construction Risk

Construction risk is defined as the uncertainty identified in preparing the opinion of probable capital cost estimate which covers quantity/quality of pre-engineering information, tender and construction schedules, the construction market, and non-quantifiable items.

- Quality/Quantity of pre-engineering information (15%): work involving removal or extension of existing facilities or pipelines when existing conditions are not completely known. Geotechnical soil excavation revealing presence of rock, or other unsuitable soils, groundwater, and other soil conditions.
- Tender and Construction Schedule (5%): projects with a tight schedule that could result in delays beyond a critical date (such as winter or bird nesting windows or in-stream fish windows) that lead to additional labour or construction method costs. Additional costs relating to delays in permits, project award or permissions with regulators. Risks of construction bidding process it is proposed to occur in one season only or several construction seasons.
- Construction Market (15%): Availability of contractors, materials and conditions within the supply chain. Risks of changing local contractor availability or desire by the local contractors to bid along with inflation and evolving labour costs.
- Non-quantifiable Items (5%): Risks related to new materials or construction methods, availability of special materials or local materials or equipment that cannot be determined in advance and premium placed on specialized or local contractor work.

5.2 Construction Contingency

Construction Contingency (10%) is an allowance for unforeseeable elements of costs which covers small design changes, supply problems, small quantity changes, reasonable inflation rate, and other undefined risks which could not be identified prior to construction.

5.3 Engineering Costs

The engineering costs are an allowance taken from federally available guidelines based on historical statistical analysis of probable engineering effort that are a percentage of the project capital cost. The engineering fee allowance includes:

- Project definition: planning, conceptualization, and defining scope, standard and requirements of project.
- Investigation and Design: Field survey, geotechnical investigation, preliminary and detailed design including up to preparing for tendering.
- Construction Inspections: field activities required to oversee the construction phase of the project.
- Project Management: engineering activities required to oversee the delivery of the project including engineering budgets, progress payments to the contractor, schedule monitoring and cost control.

For this project, an engineering fee allowance of 14.5% to 16.1% has been calculated. Although this fee could be seen as a fixed amount, the value is a statistical amount based on historical federal statistics intended solely for budget planning and should be more or less than the actual effort. Factors that impact the engineering fee include, but are not limited to: Experience of the contractor, supply chain health, contractor labor or equipment availability, contractor/sub-contractor/vender financial health to deliver the project, number of changes by the contractor or owner during stages of the project, changing ground conditions or unforeseen additional effort required to complete the project.

5.4 Estimates

TRUE has summarized the cost estimates below and a more detailed opinion of the probable capital cost estimate is available in Appendix B. The percentages as described above for construction risk, contingency and engineering fee allowance were incorporated into the cost estimates.

The reader should understand the current market conditions are still under considerable stress and our estimate is our opinion of costs based on rapidly changing historical costs. Stresses appear to remain in the market and can include high inflationary conditions, changing

demographics as baby boomers leave the work force, supply chain problems and global market conditions impacting the entire supply chain including key considerations such as foundry availability, raw material costs, and manufacturing capacity.

5.4.1 Scenario 1: Maintain Existing Lift Station Location

For this scenario, the opinion of probable capital cost is shown in Table 5-1.

TABLE 5-1: SCENARIO 1 OPINION OF PROBABLE CAPITAL COST

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$2,000
2	Temporary Sewage Bypass	\$320,000
3	Sewage Forcemain Installed by Trenchless Pipe Installation and Tie-ins	\$501,000
4	Lift Station Upgrades	\$234,000
5	Electrical Upgrades	\$235,000
Subtotal		\$1,292,000
Construction Risk & Contingency		\$646,000
Engineering Fee Allowance		\$208,000
Taxes (GST)		\$107,000
Project Amount (SAY)		\$2,250,000
Optional Purchase of 7710 Main Street at MLS asking price		\$2,299,000
Optional Purchase of Land Project Amount (SAY)		\$4,549,000

5.4.2 Scenario 2: Location A Replacement Lift Station

For this scenario, the opinion of probable capital cost is shown in Table 5-2.

TABLE 5-2: SCENARIO 2 OPINION OF PROBABLE CAPITAL COST

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$2,000
2	Temporary Sewage Bypass	\$480,000
3	Sewage Gravity Main Installed by Trenchless Pipe Installation, Manholes and Tie-ins	\$594,000
4	Reconnect Sanitary Sewer Services	\$8,000
5	Lift Station Replacement	\$744,000
6	Sheet Piles and Dewatering Allowance	\$600,000
7	Electrical Upgrades	\$345,000
Subtotal		\$2,773,000
Construction Risk & Contingency		\$1,386,000
Engineering Fee Allowance		\$402,000
Taxes (GST)		\$228,000
Project Amount (SAY)		\$4,790,000
Optional Purchase of 7710 Main Street at MLS asking price		\$2,299,000
Optional Purchase of Land Project Amount (SAY)		\$7,089,000

5.4.3 Scenario 3: Location B Replacement Lift Station

For this scenario, the opinion of probable capital cost is shown in Table 5-3.

TABLE 5-3: SCENARIO 3 OPINION OF PROBABLE CAPITAL COST

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$10,000
2	Temporary Sewage Bypass	\$224,000
3	Sewage Gravity Main Installed by Trenchless Pipe Installation, Manholes and Tie-ins	\$604,000
4	Reconnect Sanitary Sewer Services	\$8,000
5	Lift Station Replacement	\$676,000
6	Dewatering Allowance	\$100,000
7	Electrical Upgrades	\$345,000
Subtotal		\$1,967,000
Construction Risk & Contingency		\$983,000
Engineering Fee Allowance		\$317,000
Taxes (GST)		\$163,000
Project Amount (SAY)		\$3,430,000

5.4.4 Scenario 4: Location C Replacement Lift Station

Unlike Scenario 1 to 3, this scenario takes the higher costs of trenchless pipe installation through the tight corridor and invests in linear pipe works along less constrained routes. The probable capital cost is shown in Table 5-3 and as noted before, is based on conveying sewage to the replacement lift station using gravity pipes with manholes.

TABLE 5-4: SCENARIO 4 OPINION OF PROBABLE CAPITAL COST WITH GRAVITY PIPE CONVEYANCE

<i>ITEM</i>	<i>DESCRIPTION</i>	<i>ESTIMATED AMOUNT</i>
1	Traffic Control	\$20,000
2	Temporary Sewage Bypass	\$64,000
3	Sewage Gravity Mains with Manholes	\$466,000
4	Reconnect Sanitary Sewer Services	\$91,000
5	Lift Station Replacement	\$706,000
6	Dewatering Allowance	\$100,000
7	Electrical Upgrades	\$345,000
Subtotal		\$1,792,000
Construction Risk & Contingency		\$896,000
Engineering Fee Allowance		\$288,000
Taxes (GST)		\$149,000
Project Amount (SAY)		\$3,120,000

As an alternate to traditional gravity mains with manholes, septic tanks with low pressure effluent pumps were investigated to see if cost efficiencies could be gained as shown in Table 5-5.

TABLE 5-5: SCENARIO 4 OPINION OF PROBABLE CAPITAL COST WITH STEP SYSTEMS

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$20,000
2	Temporary Sewage Bypass	\$160,000
3	Sewage Gravity Mains with Manholes	\$381,000
4	Reconnect Sanitary Sewer Services using STEP System	\$427,000
5	Lift Station Replacement	\$706,000
6	Dewatering Allowance	\$100,000
7	Electrical Upgrades	\$345,000
Subtotal		\$2,139,000
Construction Risk & Contingency		\$1,070,000
Engineering Fee Allowance		\$310,000
Taxes (GST)		\$176,000
Project Amount (SAY)		\$3,700,000

5.4.5 Scenario 5: Location D Replacement Lift Station

For this scenario, the method of using traditional means of construction is utilized like Scenario 4, but the sewage conveyance network is separated on Ponderosa Drive to address long term capacity constraints (completed by TRUE on another report) and relocate the lift station away from nearby beaches or public nature walkways. The probable capital cost is shown in Table 5-6 and as noted before, is based on conveying sewage to the replacement lift station using gravity pipes with manholes.

TABLE 5-6: SCENARIO 5 OPINION OF PROBABLE CAPITAL COST WITH GRAVITY PIPE CONVEYANCE

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$75,000
2	Temporary Sewage Bypass	\$112,000
3	Sewage Gravity Mains with Manholes	\$808,000
4	Reconnect Sanitary Sewer Services	\$91,000
5	Lift Station Replacement	\$716,000
6	Dewatering Allowance	\$150,000
7	Electrical Upgrades	\$345,000
Subtotal		\$2,297,000
Construction Risk & Contingency		\$1,148,000
Engineering Fee Allowance		\$333,000
Taxes (GST)		\$189,000
Project Amount (SAY)		\$3,970,000

As an alternate to traditional gravity mains with manholes, septic tanks with low pressure effluent pumps were investigated to see if cost efficiencies could be gained as shown in Table 5-7.

TABLE 5-7: SCENARIO 3 OPINION OF PROBABLE CAPITAL COST WITH STEP SYSTEMS

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$75,000
2	Temporary Sewage Bypass	\$192,000
3	Sewage Gravity Mains with Manholes	\$736,000
4	Reconnect Sanitary Sewer Services using STEP System	\$427,000
5	Lift Station Replacement	\$716,000
6	Dewatering Allowance	\$150,000
7	Electrical Upgrades	\$345,000
Subtotal		\$2,642,000
Construction Risk & Contingency		\$1,321,000
Engineering Fee Allowance		\$383,000
Taxes (GST)		\$217,000
Project Amount (SAY)		\$4,560,000

6.0 Risk Assessment

For any project, it becomes necessary to look at several variables for each project and quantify the risks and weighted importance of each variable to assist with design choices. For this assessment to occur, several variables were defined and ranked into consequence categories as follows:

		Consequence				
		Negligible 1	Minor 2	Moderate 3	Major 4	Catastrophic 5
Likelihood	5 Almost certain	Moderate 5	High 10	Extreme 15	Extreme 20	Extreme 25
	4 Likely	Moderate 4	High 8	High 12	Extreme 16	Extreme 20
	3 Possible	Low 3	Moderate 6	High 9	High 12	Extreme 15
	2 Unlikely	Low 2	Moderate 4	Moderate 6	High 8	High 10
	1 Rare	Low 1	Low 2	Low 3	Moderate 4	Moderate 5

FIGURE 6-1: RISK MATRIX USED FOR ASSESSMENT

- Negligible:
 - Minor issue of little concern to community
 - Cost \$1,000
- Minor:
 - Isolated case, some small disruptions
 - Cost \$1,000 – \$50,000
- Moderate:
 - Requires attention, inconvenience.
 - \$50,000 – \$200,000
- Major:
 - Requires urgent attention, impact delays
 - \$200,000 – \$1M
- Catastrophic:
 - Wide damage, lasting weeks, impact high
 - \$>1M

6.1 Assessment Results

The subjective risk of occurrence was developed based on the author's opinion which was developed by leveraging historical experience in the industry. Figure 6-1 shows the risk matrix used to populate Table 6-1 with assumptions for each parameter described in later sections.

TABLE 6-1: WEIGHTED OPTION RESULTS

VARIABLE	SCENARIO					% WEIGHT
	1	2	3	4	5	
Health and Safety	15	12	8	3	3	5
Public Disruption	5	10	15	15	15	5
Town of Osoyoos Coordination	10	15	15	5	5	5
Temporary Bypass	15	15	15	2	2	10
Traffic Disruption	1	8	8	25	25	10
Construction Schedule	6	8	6	2	2	5
Constructability	12	16	12	6	8	10
Long Term Serviceability	9	6	4	2	2	10
Capital Costs	11	25	17	16	21	20
Operation and Maintenance	12	8	8	4	4	20
Total	10.2	13.4	11.3	8.8	9.9	100

When reviewing Table 6-1, lower risk scores along with a lower weighted average total score is the key driver for this assessment. Although only one tool in the decision process, Table 6-1 provides some aid to the Town for selecting a scenario to proceed with.

6.1.1 Health and Safety

Safety is top of mind for all workers including the Town of Osoyoos, contractors, consultants, and the public. Scenario 1 places high risk onto the Town due to the lack of dry valve chamber, limited access for emergency crews to respond and minimal space available for pipeline installation.

Scenarios 2 and 3 increase the risk to workers working near existing building foundations to install the assets but this scenario reduces the access risks and moves key maintenance valves outside the wet well confined space. Scenario 4 and 5 move construction to traditional risks and reduce risks identified in 1, 2, and 3.

6.1.2 Public Disruption

The public would generally be impacted by loss of service on walkways, beaches, parallel parking and traffic flow in all scenarios to a differing degree with Scenarios 3 to 5 having the largest impact, while Scenario 2 reducing the risk and Scenario 1 having the lowest probable impact.

6.1.3 Town of Osoyoos Coordination

Shutting down lift stations and assisting with sewage bypass systems can be time consuming for the Town and place additional efforts on the Town to assist during emergencies. Scenario 1 through 3 have similar efforts with Scenario 1 having a reduced construction time due to shallower trenching of the forcemain. Scenario 4 and 5 has minimal impacts to the Town except for sewage tie-ins for short periods of time.

6.1.4 Temporary Bypass

Bypassing sewage can be challenging as failure of one temporary pump can lead to sewage entering Osoyoos Lake. To offset this, the existing Starlite lift station could be modified to provide service for longer periods of time using aboveground temporary piping. While the modifications of the lift station are underway, temporary diesel pumps would be used. The temporary pipe would likely be HDPE to resist damage from vandalism, mechanical damage, and quick deployment onto the surface around the active construction site. A bypass manhole would be required to re-inject sewage bypassed from Starlite LS to the existing sewage main, and this work would be specialized and would need to be coordinated for timing. Scenarios 1 to 3 would require bypassing for extended periods of time, whereas scenarios 4 and 5 would only require short duration bypasses for tie-ins.

6.1.5 Traffic Disruption

Several of the scenarios including 4, and 5 will require traffic to be diverted to single lane while Scenario 3 assumes trenchless pipe installation can be done. Scenario 5 can use Harbour Key Drive as a detour for the Ponderosa Drive to Main Street pipe crossing to reduce the traffic flow impact. Scenario 2 would have limited impacts to traffic relating to the congested space available to install the lift station to deep depths. No impact for Scenario 1.

6.1.6 Construction Schedule

Timing of the work should be done during low lake levels to mitigate dewatering efforts, and water removed from the trench would need to be discharged to the storm system, sanitary sewer system or as directed by the environmental professional. Depending on the contractor's crew size, method of installation and as-found ground conditions, Scenario 1 and 2 would hold the higher

risk due to the lack of available work area, while Scenario 1 and 3 would could leverage trenchless pipe installation to offset some of the trenching risks related to the close building foundations. Scenario 4 and 5 have traditional risks with higher risks relating to the option of installing new pipework in the beach.

6.1.7 Constructability

Scenario 1 through 3 have the higher challenges to construct due to the narrow nature of the right-of-way and trenchless pipe installation could be used to help with building foundation challenges if the ground conditions allow. Whereas Scenario 4 and 5 are traditional in nature, and the main challenge is managing traffic flow on Main Street and managing dewatering due to the deep depth of the proposed improvements.

6.1.8 Long Term Serviceability

Osoyoos is likely to grow in population and change in parcel usage over time, and Scenario 1 brings the greatest challenges unless 7710 Main Street is purchased by the Town. Beach only access along with proximity to the lake makes access difficult for serving the lift station, fueling / maintaining the generator and day-to-day operational monitoring. Scenario 2 through 5 offer better access for the Town but Scenario 2 could have challenges with the existing parcel owner to obtain additional right-of-way for the lift station. Scenarios 3 to 5 would provide the most access to the Town and would allow the community to change as the parcel owners see fit with Scenario 4 and 5 having the highest flexibility as there would be no right-of-way on 7710 Main Street.

6.1.9 Capital Costs

Budgets for smaller municipalities are constrained, and as such, a proportional cost weighting is used for this aspect rather than the risk matrix. The opinion of probable capital cost was utilized to establish the ranking that was pro-rated to a value of 25 for the highest cost.

6.1.10 Operation and Maintenance

Scenario 2 through 5 have a new facility proposed, reducing the initial O&M costs along with reasonable access to the facilities. Scenario 1 would have the least access and would increase risks of maintenance costs if entirely accessed by the beach. The pipeline for Scenario 1 through 3 would present additional costs down the road if the pipeline were to leak, need replacement or repair. A bypass manhole would be needed for all three scenarios to ensure service is maintained when pipeline repairs are done within the 7710 Main Street right-of-way.

7.0 Conclusions and Recommendations

The existing Starlite lift station has considerable challenges to address future PWWF's along with primarily beach access and limited walking space access through a narrow sewer main and sewer

forcemain right-of-way along the west side of 7710 Main Street. In all the scenarios, deep depths of excavation come out as a considerable risk due to the proximity of Osoyoos Lake, proximity of nearby structures, and the probable presence of blast rock under Main Street (Highway 3).

Of the five scenarios, the Town would benefit longer term if Starlite is relocated beside Main Street to allow operators, maintenance crews and other parties easy access to the lift station in the event of planned or unplanned visits. Scenarios 4 and 5 would offer greater flexibility for future improvements by eliminating the narrow pipe right-of-way on 7710 Main Street while scenarios 1 and 2 could present property access challenges. To mitigate the risk of property access for Scenario 1 and 2, 7710 Main Street could be purchased by the Town at a cost likely between the BC Assessment value of \$941,900 and the current MLS listing (June 2023) of \$2.3 million. The purchase of the land would address lift station and pipeline access issues and could be developed to encourage greater public access to the north beach. As a compromise to Scenario 1 and 2, Scenario 3's placement of Starlite on Pioneer walkway improves access off Main Street but requires the existing pipeline right-of-way to be maintained.

7.1 Recommendations and Next Steps

The final choice for the Starlite lift station requires a multitude of tools to assist the decision-making process. If the multivariable risk assessment tool is used, Scenario 4 offers the lowest risk and focuses primarily on traditional methods of construction. Following Scenario 4 would be Scenario 1 and 5, followed by Scenario 3 and lastly Scenario 2. If capital costs are the key driving tool in the decision process, Scenario 1 would offer the lowest capital cost but existing residual risks would remain. Other tools including community acceptance, local property owner biases, mayor and council opinions, Town staff opinions and other variables should also be considered in conjunction with the multivariable risk and capital cost tools outlined in this report.

For Scenarios 1 to 3, any large scale multifamily or commercial development could proceed if the proposed pipe casing is installed within the existing right-of-way to minimize risks of needing to open a trench along the west edge of the property. To minimize risks for the Town's future repairs, rehabilitation or replacement of the sewer main within the pipe casing, any future development on 7710 Main Street should be done in consultation with the Town and the Town's engineering team.

The next step for the project would be selection of the scenario followed by a formal geotechnical investigation to better understand the detailed geology of the area. If the soils are found to have a strong connection with Osoyoos Lake, dewatering would likely become challenging and timing the work during low lake levels would be essential. In review of recent Zosel Dam reports, construction could start as soon as November 1 and be complete no later than March 31 as lake levels is permitted to drop to 909 ft from a peak permitted level of 912 ft to 912.5 ft during the period of June 1st to September 15th.

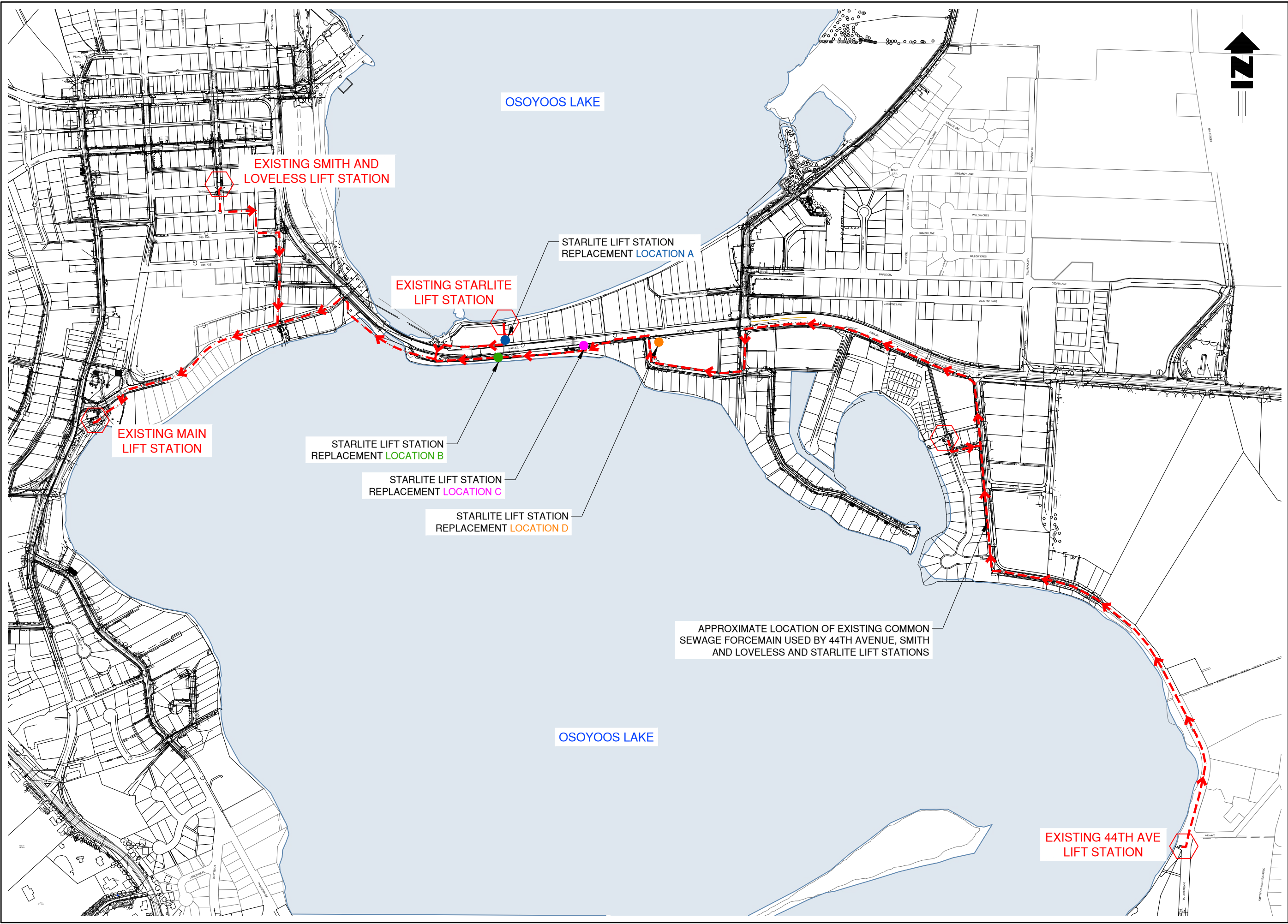
8.0 References

Nk'mip (Osoyoos Lake) The Heard of the Watershed Final Report, 2022 Osoyoos Lake Water Science Forum

2021 Annual Report to the International Joint Commission, International Osoyoos Lake Board of Control.

APPENDIX A

Figures



LEGEND		
EXISTING	PROPOSED	DESCRIPTION
		SANITARY MAIN
		SANITARY FORCEMAIN
		SANITARY MANHOLE
		CLEANOUT
		LIFT STATION
		SEPTIC TANK
		STORM MAIN
		STORM DITCH
		CULVERT
		STORM MANHOLE
		CATCHBASIN
		STORM DRYWELL
		WATER MAIN
		WATER SERVICE
		CURB STOP
		VALVE
		STANDPIPE
		HYDRANT
		AIR RELEASE VALVE
		REDUCER
		SPRINKLER
		IRRIGATION BOX
		END CAP
		POWER
		TEL
		CABLE
		STREET LIGHT
		GAS
		POWER POLE
		LAMP STANDARD
		LIGHT POST
		POLE ANCHOR
		JUNCTION BOX
		TEST PIT
		IRON PIN
		SURVEY CONTROL

CONSULTANT SEAL



**STARLITE LIFT
STATION UPGRADES
& DEVELOPMENT
CONSIDERATIONS**

**OVERALL LIFT
STATION
LOCATION
PLAN**

SCALE 1:7500 (11x17)

DESIGN BY DK

DRAWN BY MC

DATE JUNE 2023

PROJECT REFERENCE No. 302-2121

DRAWING No.

302-2121

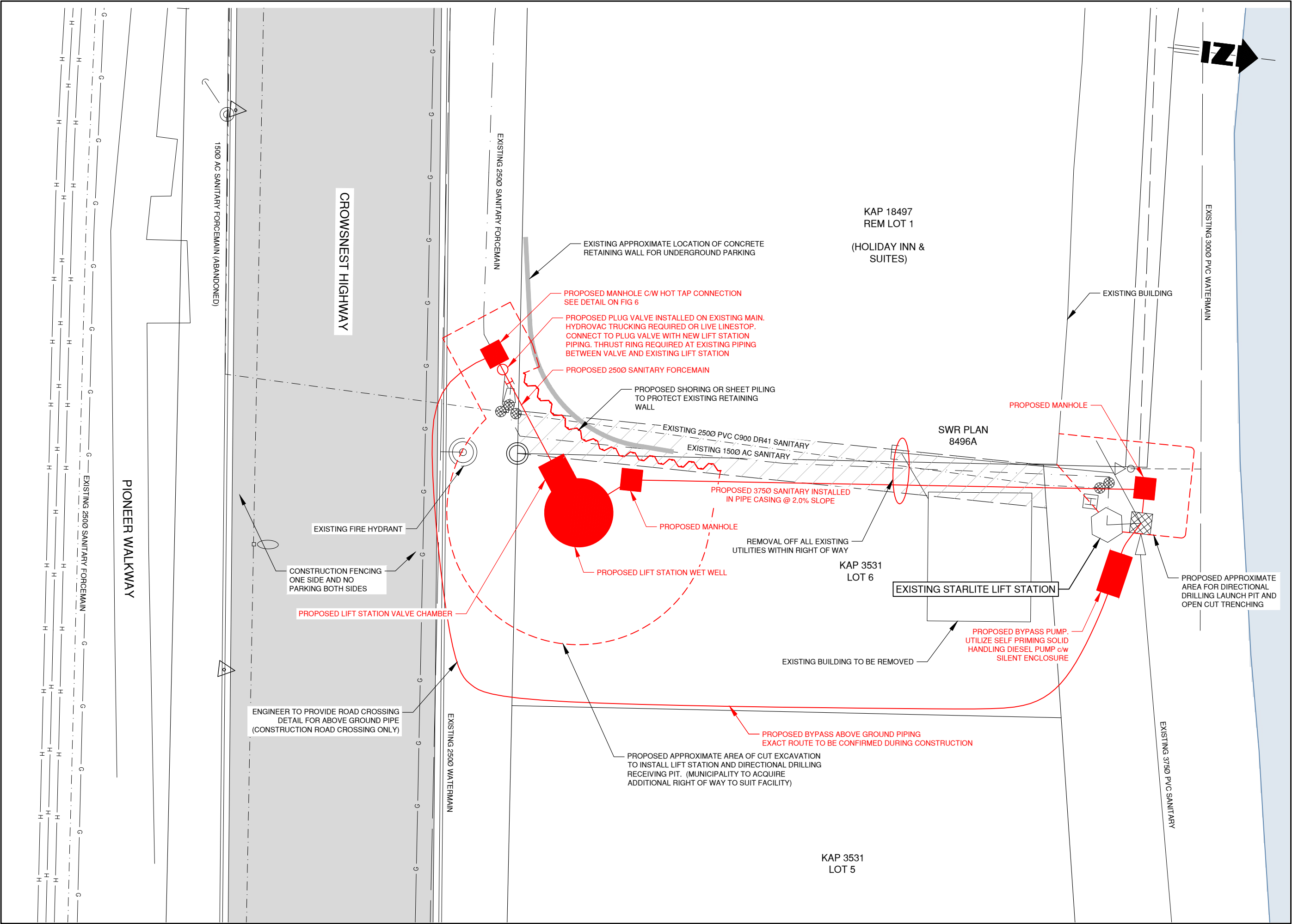
FIG.1

SHEET

OF

ISSUE/REV.

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LEGEND		
EXISTING	PROPOSED	DESCRIPTION
		SANITARY MAIN
		SANITARY FORCEMAIN
		SANITARY MANHOLE
		CLEANOUT
		LIFT STATION
		SEPTIC TANK
		STORM MAIN
		STORM DITCH
		CULVERT
		STORM MANHOLE
		CATCHBASIN
		STORM DRYWELL
		WATER MAIN
		WATER SERVICE
		CURB STOP
		VALVE
		STANDPIPE
		HYDRANT
		AIR RELEASE VALVE
		REDUCER
		SPRINKLER
		IRRIGATION BOX
		END CAP
		POWER TEL
		CABLE
		STREET LIGHT
		GAS
		POWER POLE
		LAMP STANDARD
		LIGHT POST
		POLE ANCHOR
		JUNCTION BOX
		TEST PIT
		IRON PIN
		SURVEY CONTROL

CONSULTANT SEAL



STARLITE LIFT STATION UPGRADES & DEVELOPMENT CONSIDERATIONS

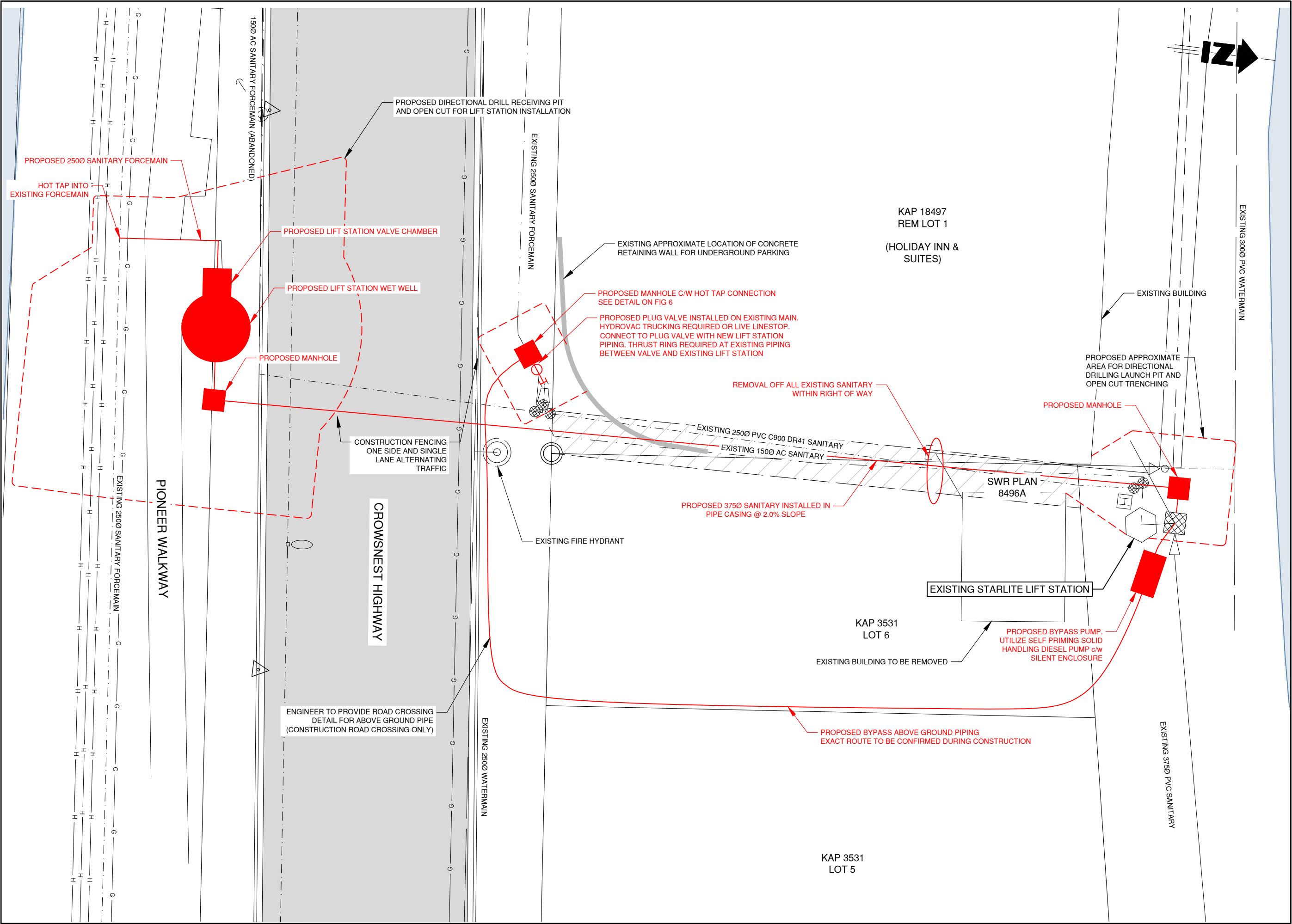
STARLITE LIFT STATION REPLACEMENT

SCENARIO 2 LOCATION A

SCALE	1:250 (11x17)
DESIGN BY	DK
DRAWN BY	MC
DATE	JUNE 2023
PROJECT REFERENCE No.	302-2121

DRAWING No.	302-2121	SHEET	OF
	FIG.3	ISSUE/REV.	

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LEGEND		
EXISTING	PROPOSED	DESCRIPTION
		SANITARY MAIN
		SANITARY FORCEMAIN
		SANITARY MANHOLE
		CLEANOUT
		LIFT STATION
		SEPTIC TANK
		STORM MAIN
		STORM DITCH
		CULVERT
		STORM MANHOLE
		CATCHBASIN
		STORM DRYWELL
		WATER MAIN
		WATER SERVICE
		CURB STOP
		VALVE
		STANDPIPE
		HYDRANT
		AIR RELEASE VALVE
		REDUCER
		SPRINKLER
		IRRIGATION BOX
		END CAP
		POWER
		TEL
		CABLE
		STREET LIGHT
		GAS
		POWER POLE
		LAMP STANDARD
		LIGHT POST
		POLE ANCHOR
		JUNCTION BOX
		TEST PIT
		IRON PIN
		SURVEY CONTROL

CONSULTANT SEAL



**STARLITE LIFT
STATION UPGRADES
& DEVELOPMENT
CONSIDERATIONS**

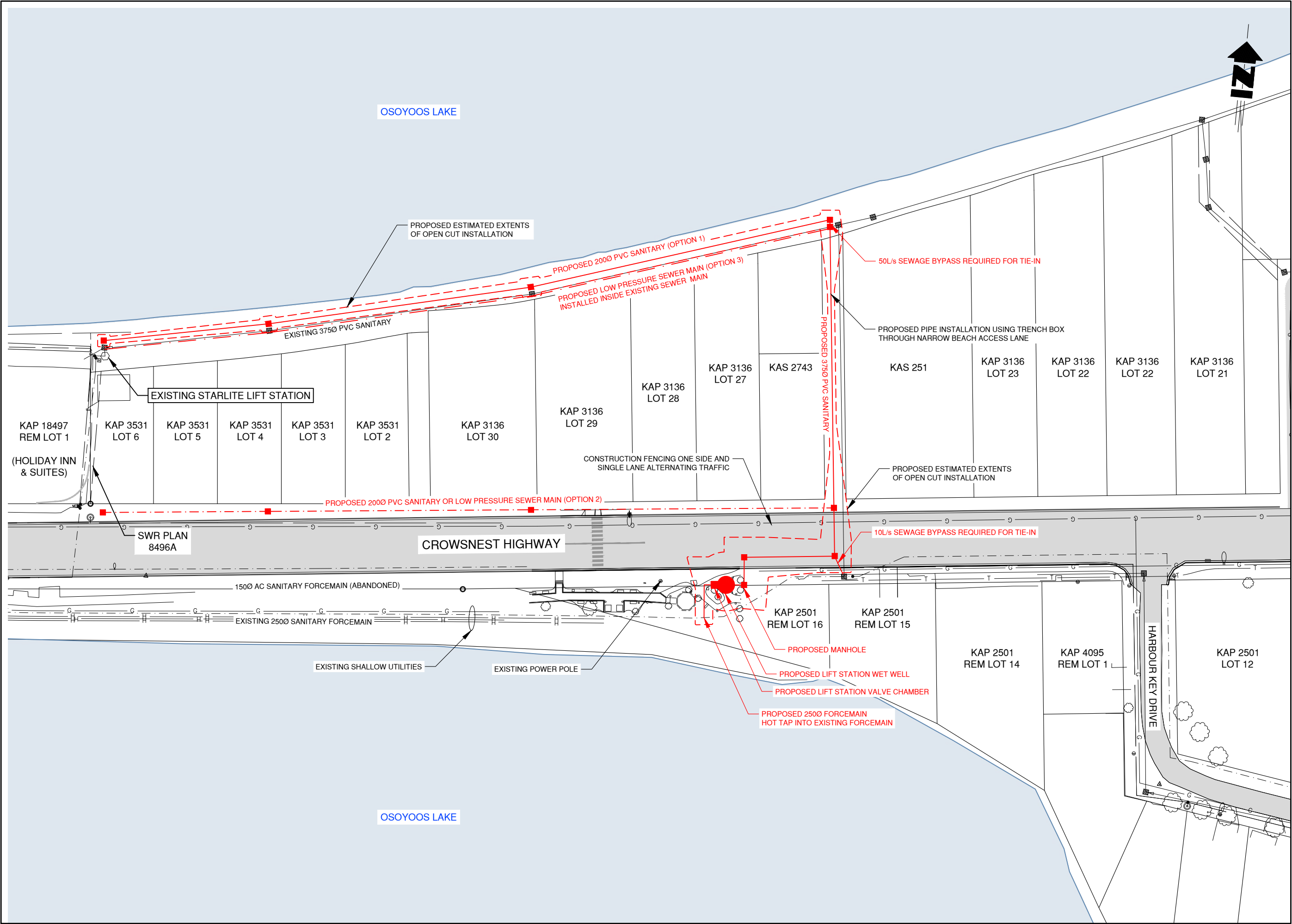
**STARLITE LIFT
STATION
REPLACEMENT**

**SCENARIO 3
LOCATION B**

SCALE	1:250 (11x17)
DESIGN BY	DK
DRAWN BY	MC
DATE	JUNE 2023
PROJECT REFERENCE No.	302-2121

DRAWING No.	302-2121	SHEET	OF
	FIG.4	ISSUE/REV.	

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LEGEND		
EXISTING	PROPOSED	DESCRIPTION
		SANITARY MAIN
		SANITARY FORCEMAIN
		SANITARY MANHOLE
		CLEANOUT
		LIFT STATION
		SEPTIC TANK
		STORM MAIN
		STORM DITCH
		CULVERT
		STORM MANHOLE
		CATCHBASIN
		STORM DRYWELL
		WATER MAIN
		WATER SERVICE
		CURB STOP
		VALVE
		STANDPIPE
		HYDRANT
		AIR RELEASE VALVE
		REDUCER
		SPRINKLER
		IRRIGATION BOX
		END CAP
		POWER
		TEL
		CABLE
		STREET LIGHT
		GAS
		POWER POLE
		LAMP STANDARD
		LIGHT POST
		POLE ANCHOR
		JUNCTION BOX
		TEST PIT
		IRON PIN
		SURVEY CONTROL

CONSULTANT SEAL



STARLITE LIFT STATION UPGRADES & DEVELOPMENT CONSIDERATIONS

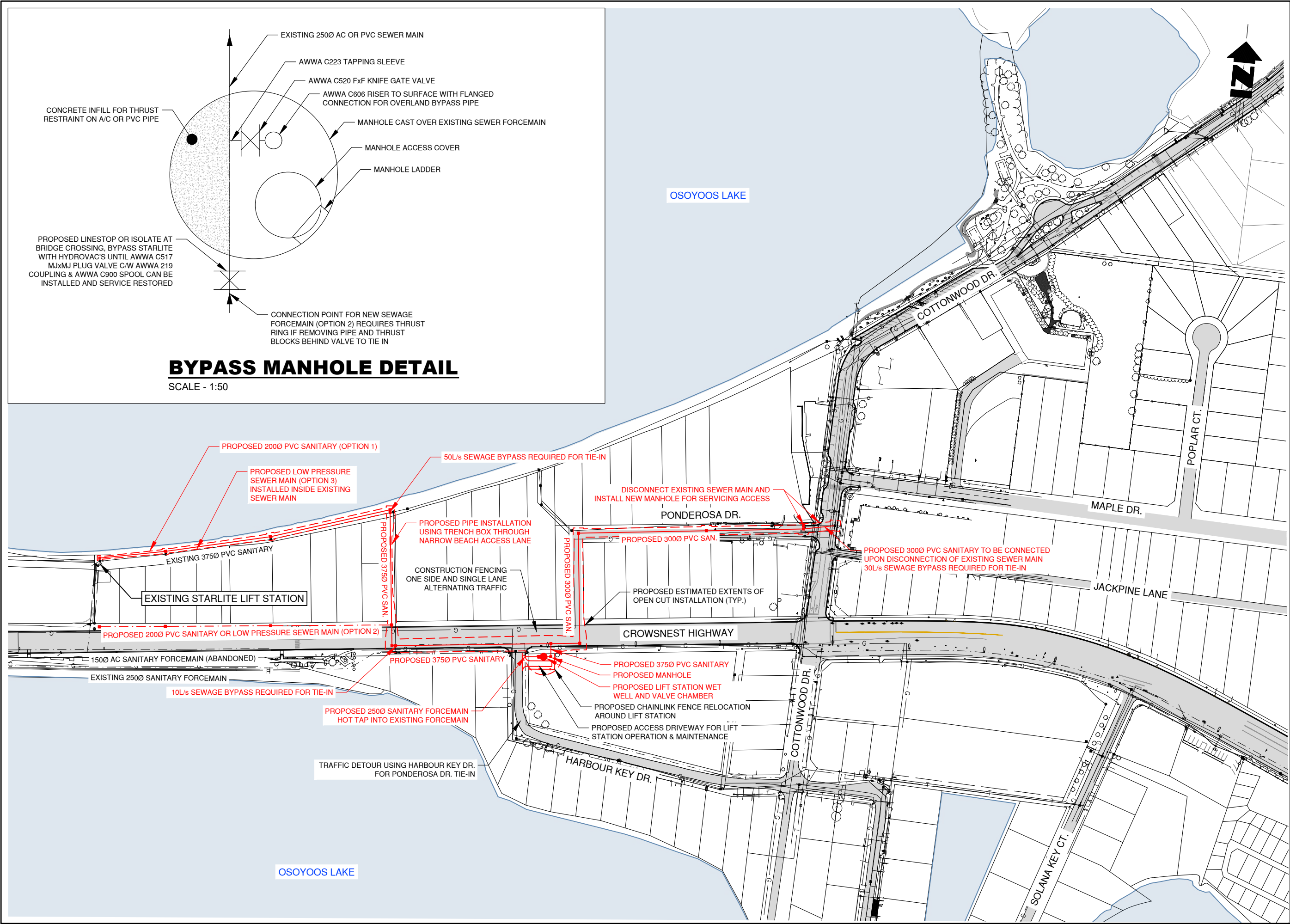
STARLITE LIFT STATION REPLACEMENT

SCENARIO 4 LOCATION C

SCALE	1:1000 (11x17)
DESIGN BY	DK
DRAWN BY	MC
DATE	JUNE 2023
PROJECT REFERENCE No.	302-2121
DRAWING No.	302-2121
SHEET	OF
ISSUE/REV.	

FIG.5

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BYPASS MANHOLE DETAIL

SCALE - 1:50

LEGEND		DESCRIPTION
EXISTING	PROPOSED	SANITARY MAIN
		SANITARY FORCEMAIN
		SANITARY MANHOLE
		CLEANOUT
		LIFT STATION
		SEPTIC TANK
		STORM MAIN
		STORM DITCH
		CULVERT
		STORM MANHOLE
		CATCHBASIN
		STORM DRYWELL
		WATER MAIN
		WATER SERVICE
		CURB STOP
		VALVE
		STANDPIPE
		HYDRANT
		AIR RELEASE VALVE
		REDUCER
		SPRINKLER
		IRRIGATION BOX
		END CAP
		POWER
		TEL
		CABLE
		STREET LIGHT
		GAS
		POWER POLE
		LAMP STANDARD
		LIGHT POST
		POLE ANCHOR
		JUNCTION BOX
		TEST PIT
		IRON PIN
		SURVEY CONTROL

CONSULTANT SEAL



STARLITE LIFT STATION UPGRADES & DEVELOPMENT CONSIDERATIONS

STARLITE LIFT STATION REPLACEMENT

SCENARIO 5 LOCATION D

SCALE	1:2500 (11x17)
DESIGN BY	DK
DRAWN BY	MC
DATE	JUNE 2023
PROJECT REFERENCE No.	302-2121
DRAWING No.	302-2121
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ISSUE/REV.	

FIG.6

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APPENDIX B

Cost Estimates



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 1: Maintain Existing Lift Station Location

PAYMENT				UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA	DESCRIPTION				
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$2,000	\$2,000
Subtotal Part 1.0							\$2,000
PART 2.0 - UTILITIES							
2.01	33 30 01	1.6.8	Temporary Sewer Bypass	Day	20	\$16,000	\$320,000
2.02	33 30 01	1.6.9	Trenchless Pipe Installation of Pipe Casing c/w Casing Spacers and End Seals	LS	1	\$380,000	\$380,000
2.03	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	I.m.	5	\$500	\$2,500
2.04	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Restrained and Installed in Casing Pipe	I.m.	45	\$250	\$11,250
2.05	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ-MJ	Each	2	\$8,000	\$16,000
2.06	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	2	\$4,000	\$8,000
2.07	33 34 01	1.8.9	Reinforced Concrete Thrust Ring	Each	1	\$8,000	\$8,000
2.08	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	2	\$20,000	\$40,000
2.09	33 34 01	1.8.11	Sewage Bypass Manhole c/w AWWA C223 Tapping Sleeve, AWWA C520 Knife Gate Valve and AWWA C606 Fittings/Flanges	LS	1	\$35,000	\$35,000
2.10	33 44 01	1.8.12	67hp AWWA E110 Flygt Submersible Pump	Each	2	\$117,000	\$234,000
Subtotal Part 2.0							\$1,054,750
PART 3.0 - ELECTRICAL							
3.01	26 XX XX SS		New VFD Kiosk	LS	1	\$75,000	\$75,000
3.02	26 XX XX SS		New Generator	LS	1	\$80,000	\$80,000
3.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
3.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$70,000	\$70,000
Subtotal Part 3.0							\$235,000

PAYMENT				EST.	UNIT	TOTAL	
ITEM	SECTION	PARA	DESCRIPTION	UNIT	QUANT.	PRICE	PAYMENT
SUMMARY							
			Part 1.0 - General Requirements				\$2,000
			Part 2.0 - Utilities				\$1,054,750
			Part 3.0 - Electrical				\$235,000
			Subtotal Parts 1.0 to 3.0				\$1,291,750
			Construction Risk & Contingency Allowance (50%)				\$646,000
			Engineering Fee Allowance (16.1%)				\$208,000
			GST (5% of Subtotal and Contingency Allowance)				\$107,000
			Total Contract Sum (SAY)				\$2,250,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 2: Location A Replacement Lift Station

PAYMENT				UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA	DESCRIPTION				
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$2,000	\$2,000
Subtotal Part 1.0							\$2,000
PART 2.0 - UTILITIES							
2.01	33 01 30.1		CCTV Pipeline Inspection	l.m.	40	\$50	\$2,000
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
2.02	33 30 01		Remove and dispose of existing sewer system Mainline Piping	l.m.	80	\$150	\$12,000
2.03	33 30 01		Remove and dispose of existing sewer system Service Piping	l.m.	20	\$80	\$1,600
2.04	33 30 01	1.6.3	Sanitary Service Connections, Disconnect from existing and Connect to New Main	Each	1	\$1,000	\$1,000
2.05	33 30 01	1.6.1, 1.6.2	Service Sewer Pipe PVC DR28, Re-grade to Existing Building, Allowance	l.m.	20	\$250	\$5,000
2.06	33 30 01	1.6.7	Sanitary Tie-In, existing manhole to new manhole	Each	1	\$8,000	\$8,000
2.07	33 30 01	1.6.8	Temporary Sewer Bypass	Day	30	\$16,000	\$480,000
2.08	33 30 01	1.6.9	Trenchless Pipe Installation of Pipe Casing c/w Casing Spacers and End Seals	LS	1	\$380,000	\$380,000
2.09	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	l.m.	10	\$500	\$5,000
2.10	33 34 01	1.8.2, 1.8.3	Sewer Pipe AWWA C900 PVC DR18 375 mm diameter, Restrained and Installed in Casing Pipe	l.m.	40	\$350	\$14,000
2.11	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ-MJ	Each	1	\$8,000	\$8,000
2.12	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	2	\$4,000	\$8,000
2.13	33 34 01	1.8.9	Reinforced Concrete Thrust Ring	Each	1	\$8,000	\$8,000
2.14	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	1	\$20,000	\$20,000
2.15	33 34 01	1.8.11	Sewage Bypass Manhole c/w AWWA C223 Tapping Sleeve, AWWA C520 Knife Gate Valve and AWWA C606 Fittings/Flanges	LS	1	\$35,000	\$35,000
2.16	33 44 01	1.8.12	67hp AWWA E110 Flygt Submersible Pump	Each	2	\$117,000	\$234,000

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 2.0 - UTILITIES (Continued)							
2.17	33 44 01	1.8.13	Xylem FRP Wet Well and Valve Chamber c/w Flygt guide rails, AWWA C220 piping, AWWA C226/C228/C606 fittings, AWWA C520 knife gate valves, AWWA C504 check valve, AWWA C512 combo air valve, AWWA C751 Flowmeter, Carbon Odor Control, Dexon Blower and Pressure Indication.	LS	1	\$510,000	\$510,000
2.19	33 44 01	1.5.1.1	Manhole: base, lid, slab,cover and frame 1200mm diameter	Each	2	\$9,000	\$18,000
2.20	33 44 01	1.5.1.4	Re-bench Existing Manhole	Each	1	\$2,000	\$2,000
2.21	33 44 01	1.5.1.2	Manhole Riser and Tee Riser 1200mm diameter	v.m.	12	\$2,000	\$24,000
2.22	33 30 01		Sheet Pile Wall and Dewatering Allowance	LS	1	\$600,000	\$600,000
Subtotal Part 2.0							\$2,425,600

PART 3.0 - ELECTRICAL

3.01	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
3.02	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
3.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
3.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
Subtotal Part 3.0							\$345,000

SUMMARY

Part 1.0 - General Requirements	\$2,000
Part 2.0 - Utilities	\$2,425,600
Part 3.0 - Electrical	\$345,000
Subtotal Parts 1.0 to 3.0	\$2,772,600
Construction Risk & Contingency Allowance (50%)	\$1,386,000
Engineering Fee Allowance (14.5%)	\$402,000
GST (5% of Subtotal and Contingency Allowance)	\$228,000
Total Contract Sum (SAY)	\$4,790,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 3: Location B Replacement Lift Station

PAYMENT				UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA	DESCRIPTION				
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$10,000	\$10,000
Subtotal Part 1.0							\$10,000
PART 2.0 - UTILITIES							
2.01	33 01 30.1		CCTV Pipeline Inspection	l.m.	75	\$50	\$3,750
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
2.02	33 30 01		Remove and dispose of existing sewer system Mainline Piping	l.m.	80	\$150	\$12,000
2.03	33 30 01		Remove and dispose of existing sewer system Service Piping	l.m.	20	\$80	\$1,600
2.04	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 375mm diameter, Native Backfill	l.m.	5	\$1,100	\$5,500
2.05	33 30 01	1.6.3	Sanitary Service Connections, Disconnect from existing and Connect to New Main	Each	1	\$1,000	\$1,000
2.06	33 30 01	1.6.1, 1.6.2	Service Sewer Pipe PVC DR28, Re-grade to Existing Building, Allowance	l.m.	20	\$250	\$5,000
2.07	33 30 01	1.6.7	Sanitary Tie-In, existing manhole to new manhole	Each	1	\$8,000	\$8,000
2.08	33 30 01	1.6.8	Temporary Sewer Bypass	Day	14	\$16,000	\$224,000
2.09	33 30 01	1.6.9	Trenchless Pipe Installation of Pipe Casing c/w Casing Spacers and End Seals	LS	1	\$380,000	\$380,000
2.10	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	l.m.	10	\$500	\$5,000
2.11	33 34 01	1.8.2, 1.8.3	Sewer Pipe AWWA C900 PVC DR18 375 mm diameter, Restrained and Installed in Casing Pipe	l.m.	70	\$350	\$24,500
2.12	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ-MJ	Each	1	\$8,000	\$8,000
2.13	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	1	\$4,000	\$4,000
2.14	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	1	\$20,000	\$20,000
2.15	33 34 01	1.8.11	Sewage Bypass Manhole c/w AWWA C223 Tapping Sleeve, AWWA C520 Knife Gate Valve and AWWA C606 Fittings/Flanges	LS	1	\$35,000	\$35,000
2.16	33 44 01	1.8.12	44hp AWWA E110 Flygt Submersible Pump	Each	2	\$83,000	\$166,000

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 2.0 - UTILITIES (Continued)							
2.17	33 44 01	1.8.13	Xylem FRP Wet Well and Valve Chamber c/w Flygt guide rails, AWWA C220 piping, AWWA C226/C228/C606 fittings, AWWA C520 knife gate valves, AWWA C504 check valve, AWWA C512 combo air valve, AWWA C751 Flowmeter, Carbon Odor Control, Dexon Blower and Pressure Indication.	LS	1	\$510,000	\$510,000
2.18	33 44 01	1.5.1.1	Manhole: base, lid, slab,cover and frame 1200mm diameter	Each	2	\$9,000	\$18,000
2.19	33 44 01	1.5.1.4	Re-bench Existing Manhole	Each	1	\$2,000	\$2,000
2.20	33 44 01	1.5.1.2	Manhole Riser and Tee Riser 1200mm diameter	v.m.	14	\$2,000	\$28,000
5.32	33 30 01		Dewatering Allowance	LS	1	\$100,000	\$100,000
Subtotal Part 2.0							\$1,611,350
PART 3.0 - ELECTRICAL							
3.01	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
3.02	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
3.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
3.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
Subtotal Part 3.0							\$345,000
SUMMARY							
Part 1.0 - General Requirements							\$10,000
Part 2.0 - Utilities							\$1,611,350
Part 3.0 - Electrical							\$345,000
Subtotal Parts 1.0 to 3.0							\$1,966,350
Construction Risk & Contingency Allowance (50%)							\$983,000
Engineering Fee Allowance (16.1%)							\$317,000
GST (5% of Subtotal and Contingency Allowance)							\$163,000
Total Contract Sum (SAY)							\$3,430,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 4: Location C Replacement Lift Station (Option 1 & 2 Gravity Conveyance)

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$20,000	\$20,000
Subtotal Part 1.0							\$20,000
PART 2.0 - UTILITIES							
2.01	33 01 30.1		CCTV Pipeline Inspection	I.m.	340	\$15	\$5,100
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
2.02	33 30 01		Remove and dispose of existing sewer system Service Piping	I.m.	240	\$80	\$19,200
2.03	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 200mm diameter, Native Backfill	I.m.	210	\$350	\$73,500
2.04	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 375mm diameter, Native Backfill	I.m.	130	\$1,100	\$143,000
2.05	33 30 01	1.6.3	Sanitary Service Connections, Disconnect from existing and Connect to New Main	Each	12	\$1,000	\$12,000
2.06	33 30 01	1.6.1, 1.6.2	Service Sewer Pipe PVC DR28, Re-grade to Existing Building, Allowance	I.m.	240	\$250	\$60,000
2.07	33 30 01	1.6.7	Sanitary Tie-In, existing manhole to new manhole	Each	2	\$8,000	\$16,000
2.08	33 30 01	1.6.8	Temporary Sewer Bypass	Day	4	\$16,000	\$64,000
2.09	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	I.m.	12	\$500	\$6,000
2.10	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ-MJ	Each	1	\$8,000	\$8,000
2.11	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	1	\$4,000	\$4,000
2.12	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	1	\$20,000	\$20,000
2.13	33 44 01	1.8.12	54hp AWWA E110 Flygt Submersible Pump	Each	2	\$98,000	\$196,000
2.14	33 44 01	1.8.13	Xylem FRP Wet Well and Valve Chamber c/w Flygt guide rails, AWWA C220 piping, AWWA C226/C228/C606 fittings, AWWA C520 knife gate valves, AWWA C504 check valve, AWWA C512 combo air valve, AWWA C751 Flowmeter, Carbon Odor Control, Dexon Blower and Pressure Indication.	LS	1	\$510,000	\$510,000
2.15	33 44 01	1.5.1.1	Manhole: base, lid, slab,cover and frame 1200mm diameter	Each	9	\$9,000	\$81,000

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 2.0 - UTILITIES (Continued)							
2.16	33 44 01	1.5.1.4	Re-bench Existing Manhole	Each	1	\$2,000	\$2,000
2.17	33 44 01	1.5.1.6	Add Manhole to Existing System 1200mm diameter	Each	1	\$15,000	\$15,000
2.18	33 44 01	1.5.1.2	Manhole Riser and Tee Riser 1200mm diameter	v.m.	21	\$2,000	\$42,000
5.32	33 30 01		Dewatering Allowance	LS	1	\$100,000	\$100,000
Subtotal Part 2.0							\$1,426,800

PART 3.0 - ELECTRICAL							
3.01	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
3.02	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
3.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
3.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
Subtotal Part 3.0							\$345,000

SUMMARY							
			Part 1.0 - General Requirements				\$20,000
			Part 2.0 - Utilities				\$1,426,800
			Part 3.0 - Electrical				\$345,000
			Subtotal Parts 1.0 to 3.0				\$1,791,800
			Construction Risk & Contingency Allowance (50%)				\$896,000
			Engineering Fee Allowance (16.1%)				\$288,000
			GST (5% of Subtotal and Contingency Allowance)				\$149,000
			Total Contract Sum (SAY)				\$3,120,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 4: Location C Replacement Lift Station (Option 3 STEP Systems)

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$20,000	\$20,000
Subtotal Part 1.0							\$20,000
PART 2.0 - UTILITIES							
2.01	33 01 30.1		CCTV Pipeline Inspection	I.m.	131	\$30	\$3,930
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
2.02	33 30 01		Remove and dispose of existing sewer system Service Piping	I.m.	240	\$80	\$19,200
2.03	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 375mm diameter, Native Backfill	I.m.	131	\$1,100	\$144,100
2.04	33 30 01	1.6.3	Sanitary Service Connections, Disconnect from existing and Connect to New Main	Each	12	\$1,000	\$12,000
2.05	33 30 01	1.6.7	Sanitary Tie-In, existing manhole to new manhole	Each	2	\$8,000	\$16,000
2.06	33 30 01	1.6.8	Temporary Sewer Bypass	Day	10	\$16,000	\$160,000
2.07	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	I.m.	12	\$500	\$6,000
2.08	33 34 01	1.8.2, 1.8.3	Sewer Pipe AWWA C906 HDPE DR11, Common Main for STEP Systems	I.m.	210	\$200	\$42,000
2.09	33 34 01	1.8.2, 1.8.3	Sewer Pipe AWWA C901 HDPE DR9 50mm diameter, Service pipe for STEP Systems	I.m.	240	\$250	\$60,000
2.10	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ-MJ	Each	1	\$8,000	\$8,000
2.11	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	1	\$4,000	\$4,000
2.12	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	1	\$20,000	\$20,000
2.13	33 44 01	1.8.12	54hp AWWA E110 Flygt Submersible Pump	Each	2	\$98,000	\$196,000
2.14	33 44 01	1.8.13	Xylem FRP Wet Well and Valve Chamber c/w Flygt guide rails, AWWA C220 piping, AWWA C226/C228/C606 fittings, AWWA C520 knife gate valves, AWWA C504 check valve, AWWA C512 combo air valve, AWWA C751 Flowmeter, Carbon Odor Control, Dexon Blower and Pressure Indication.	LS	1	\$510,000	\$510,000

PAYMENT				EST.	UNIT	TOTAL	
ITEM	SECTION	PARA	DESCRIPTION	UNIT	QUANT.	PRICE	PAYMENT
PART 2.0 - UTILITIES (Continued)							
2.15	33 44 01	1.5.1.1	Manhole: base, lid, slab,cover and frame 1200mm diameter	Each	4	\$9,000	\$36,000
2.16	33 44 01	1.5.1.4	Re-bench Existing Manhole	Each	1	\$2,000	\$2,000
2.17	33 44 01	1.5.1.6	Add Manhole to Existing System 1200mm diameter	Each	1	\$15,000	\$15,000
2.18	33 44 01	1.5.1.2	Manhole Riser and Tee Riser 1200mm diameter	v.m.	17	\$2,000	\$34,000
2.19	33 34 01		Low Pressure STEP System (one per Lot)	Each	12	\$28,000	\$336,000
5.32	33 30 01		Dewatering Allowance	LS	1	\$100,000	\$100,000
Subtotal Part 2.0							\$1,774,230

PART 3.0 - ELECTRICAL

3.01	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
3.02	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
3.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
3.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
Subtotal Part 3.0							\$345,000

SUMMARY

Part 1.0 - General Requirements	\$20,000
Part 2.0 - Utilities	\$1,774,230
Part 3.0 - Electrical	\$345,000
Subtotal Parts 1.0 to 3.0	\$2,139,230
Construction Risk & Contingency Allowance (50%)	\$1,070,000
Engineering Fee Allowance (14.5%)	\$310,000
GST (5% of Subtotal and Contingency Allowance)	\$176,000
Total Contract Sum (SAY)	\$3,700,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 5: Location D Replacement Lift Station (Option 1 & 2 Gravity Conveyance)

PAYMENT				EST.	UNIT	TOTAL	
ITEM	SECTION	PARA	DESCRIPTION	UNIT	QUANT.	PRICE	PAYMENT
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$75,000	\$75,000
Subtotal Part 1.0							\$75,000
PART 2.0 - EARTHWORKS							
2.01	31 11 07	1.4.1, 1.4.2	Clearing and Grubbing	m ²	250	\$4	\$1,000
2.02	31 22 01	1.4.1	Topsoil Stripping and Reuse	m ²	250	\$6	\$1,500
Subtotal Part 2.0							\$2,500
PART 3.0 - ROADS AND SITE IMPROVEMENTS							
3.01	32 11 16.1	1.4.2	Granular Sub-Base 300mm Thickness for Roads or Sidewalks (Access Driveway)	m ²	100	\$30	\$3,000
3.02	32 11 23	1.4.2	Granular Base 100mm Thickness for Roads or Sidewalks (Access Driveway)	m ²	100	\$20	\$2,000
3.03	32 12 16	1.5.7	Saw Cut Asphalt or Concrete Pavements (Access Driveway)	lm	15	\$20	\$300
3.04	34 31 13	1.5.3	Remove & Reinstall Chain Link Fence (Lift Station Outside Existing Fence)	l.m.	30	\$80	\$2,400
Subtotal Part 3.0							\$7,700
PART 4.0 - UTILITIES							
4.01	33 01 30.1		CCTV Pipeline Inspection	l.m.	705	\$15	\$10,575
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
4.02	33 30 01		Remove and dispose of existing sewer system Service Piping	l.m.	240	\$80	\$19,200
4.03	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 200mm diameter, Native Backfill	l.m.	210	\$350	\$73,500
4.04	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 300mm diameter, Native Backfill	l.m.	280	\$450	\$126,000
4.05	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 375mm diameter, Native Backfill	l.m.	215	\$1,100	\$236,500
4.06	33 30 01	1.6.3	Sanitary Service Connections, Disconnect from existing and Connect to New Main	Each	12	\$1,000	\$12,000
4.07	33 30 01	1.6.1, 1.6.2	Service Sewer Pipe PVC DR28, Re-grade to Existing Building, Allowance	l.m.	240	\$250	\$60,000

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 4.0 - UTILITIES (Continued)							
4.08	33 30 01	1.6.7	Sanitary Tie-In, existing manhole to new manhole	Each	3	\$8,000	\$24,000
4.09	33 30 01	1.6.8	Temporary Sewer Bypass	Day	7	\$16,000	\$112,000
4.10	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	l.m.	10	\$500	\$5,000
4.11	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ- MJ	Each	1	\$8,000	\$8,000
4.12	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	2	\$4,000	\$8,000
4.13	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	1	\$20,000	\$20,000
4.14	33 44 01	1.8.12	54hp AWWA E110 Flygt Submersible Pump	Each	2	\$98,000	\$196,000
4.15	33 44 01	1.8.13	Xylem FRP Wet Well and Valve Chamber c/w Flygt guide rails, AWWA C220 piping, AWWA C226/C228/C606 fittings, AWWA C520 knife gate valves, AWWA C504 check valve, AWWA C512 combo air valve, AWWA C751 Flowmeter, Carbon Odor Control, Dexon Blower and Pressure Indication.	LS	1	\$510,000	\$510,000
4.16	33 44 01	1.5.1.1	Manhole: base, lid, slab,cover and frame 1200mm diameter	Each	12	\$9,000	\$108,000
4.17	33 44 01	1.5.1.4	Re-bench Existing Manhole	Each	2	\$2,000	\$4,000
4.18	33 44 01	1.5.1.6	Add Manhole to Existing System 1200mm diameter	Each	2	\$15,000	\$30,000
4.19	33 44 01	1.5.1.2	Manhole Riser and Tee Riser 1200mm diameter	v.m.	52	\$2,000	\$104,000
4.20	33 30 01		Dewatering Allowance	LS	1	\$150,000	\$150,000
						Subtotal Part 4.0	\$1,866,775
PART 5.0 - ELECTRICAL							
5.01	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
5.02	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
5.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
5.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
						Subtotal Part 5.0	\$345,000

SUMMARY

Part 1.0 - General Requirements	\$75,000
Part 2.0 - Earthworks	\$2,500
Part 3.0 - Roads and Site Improvements	\$7,700
Part 4.0 - Utilities	\$1,866,775
Part 5.0 - Electrical	\$345,000
Subtotal Parts 1.0 to 5.0	\$2,296,975
Construction Risk & Contingency Allowance (50%)	\$1,148,000
Engineering Fee Allowance (14.5%)	\$333,000
GST (5% of Subtotal and Contingency Allowance)	\$189,000
Total Contract Sum (SAY)	\$3,970,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 5: Location D Replacement Lift Station (Option 3 STEP Systems)

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$75,000	\$75,000
Subtotal Part 1.0							\$75,000
PART 2.0 - EARTHWORKS							
2.02	31 11 07	1.4.1, 1.4.2	Clearing and Grubbing	m²	250	\$6	\$1,500
3.03	31 22 01	1.4.1	Topsoil Stripping and Reuse	m²	250	\$4	\$1,000
Subtotal Part 2.0							\$2,500
PART 3.0 - ROADS AND SITE IMPROVEMENTS							
3.01	32 11 16.1	1.4.2	Granular Sub-Base 300mm Thickness for Roads or Sidewalks (Incidental)	m²	100	\$30	\$3,000
3.02	32 11 23	1.4.2	Granular Base 100mm Thickness for Roads or Sidewalks (Incidental)	m²	100	\$20	\$2,000
3.03	32 12 13.1	1.5.1	Asphalt Tack Coat (Incidental)	Note	-	-	-
3.04	32 12 13.2	1.5.1	Asphalt Prime (Incidental)	Note	-	-	-
3.05	32 12 16	1.5.7	Saw Cut Asphalt or Concrete Pavements (Incidental)	lm	15	\$20	\$300
3.06	32 12 16	1.5.8	Permanent Pavement Restoration (Incidental)	Note	-	-	-
3.07	32 17 23	1.5.2	Permanent Painted Pavement Markings (Incidental)	Note	-	-	-
3.08	34 31 13	1.5.3	Remove & Reinstall Chain Link Fence (Lift Station Outside Existing Fence)	l.m.	30	\$80	\$2,400
3.09	32 92 23	1.8.2	Nursery Sod	Note	-	-	-
3.10	32 93 01	1.9.1	Remove, and Reinstall Existing Trees, Shrubs and Ground Cover	Note	-	-	-
Subtotal Part 3.0							\$7,700
PART 4.0 - UTILITIES							
4.01	33 01 30.1		CCTV Pipeline Inspection	l.m.	495	\$30	\$14,850
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
4.02	33 30 01		Remove and dispose of existing sewer system Service Piping	l.m.	240	\$80	\$19,200
4.03	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 300mm diameter, Native Backfill	l.m.	280	\$450	\$126,000

Subtotal Part 4.0	\$2,211,550

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 5.0 - ELECTRICAL							
5.04	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
5.04	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
5.04	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
5.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
Subtotal Part 5.0							\$345,000

SUMMARY							
			Part 1.0 - General Requirements				\$75,000
			Part 2.0 - Earthworks				\$2,500
			Part 3.0 - Roads and Site Improvements				\$7,700
			Part 4.0 - Utilities				\$2,211,550
			Part 5.0 - Electrical				\$345,000
			Subtotal Parts 1.0 to 5.0				\$2,641,750
			Construction Risk & Contingency Allowance (50%)				\$1,321,000
			Engineering Fee Allowance (14.5%)				\$383,000
			GST (5% of Subtotal and Contingency Allowance)				\$217,000
			Total Contract Sum (SAY)				\$4,560,000

Prepared by: D. Kronebusch, P. Eng.

Wastewater Treatment Long Term Plan

Town of Osoyoos



ENGINEERING ■ PLANNING ■ URBAN DESIGN ■ LAND SURVEYING

April 2021

Project No. 302-1354

Distribution List

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Revision Log

Revision #	Revised by	Date	Issue / Revision Description

Report Submission

Report Prepared By:

Report Reviewed By:



2021-04-26

Rob Wall, P Eng
Project Engineer



May 12 2021

Steve Underwood, P. Eng.
Project Engineer

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1.0 Existing Treatment

1.1 Wastewater Treatment Design

The existing wastewater treatment process consists of a small complete mix aerated lagoon followed by two partial mix aerated lagoons. A process flow diagram is provided (Appendix A).

95% of the sewage inflow received at the plant comes from the Main Lift Station. 1.5mm screens at this lift station remove solids such as wipes and other debris from the flow. Flow is also received from the Golf Course lift station. The solids in this flow are shredded by chopper pumps, but are not removed.

Cell 1 is classified as a 'complete mix' aerated lagoon (whereby solids are not intended to settle in the lagoon). Aeration is very intense in order to achieve this effect. Cells 2 and 3 are 'partial mix' aerated lagoons. Enough air is added to maintain appropriate dissolved oxygen levels in these reactors, but not so much as to agitate the contents. Sludge settles to the bottom of the partial mix reactors for further degradation and storage. The volume of the complete mix lagoon (Cell 1) is approximately 3,300m³ (calculated from drawings). Cells 2 and 3 are significantly larger, at 27,000m³ and 58,000m³.

The treated wastewater is stored in large basins for reuse in the summer months. While a total of 565,000m³ can be stored, which is equivalent to 10 months of 'winter' flow, 60 days of capacity must be held in reserve. This is equivalent to around 100,000m³. The storage of the reclaimed water also further improves its quality. When water is needed for irrigation it is pumped to a chlorine contact basin for disinfection and distribution to various customers;

- Osoyoos Golf and Country Club (65.6 ha)
- Osoyoos Ball Diamonds (2.4 ha)
- Desert Park Infield Area (5.0 ha)
- Secondary School Playing Fields (2.8 ha)
- Tree Farm Standpipe and Infiltration

There is also a subsurface disposal field for disposal of excess reclaimed water, but this is not routinely used.

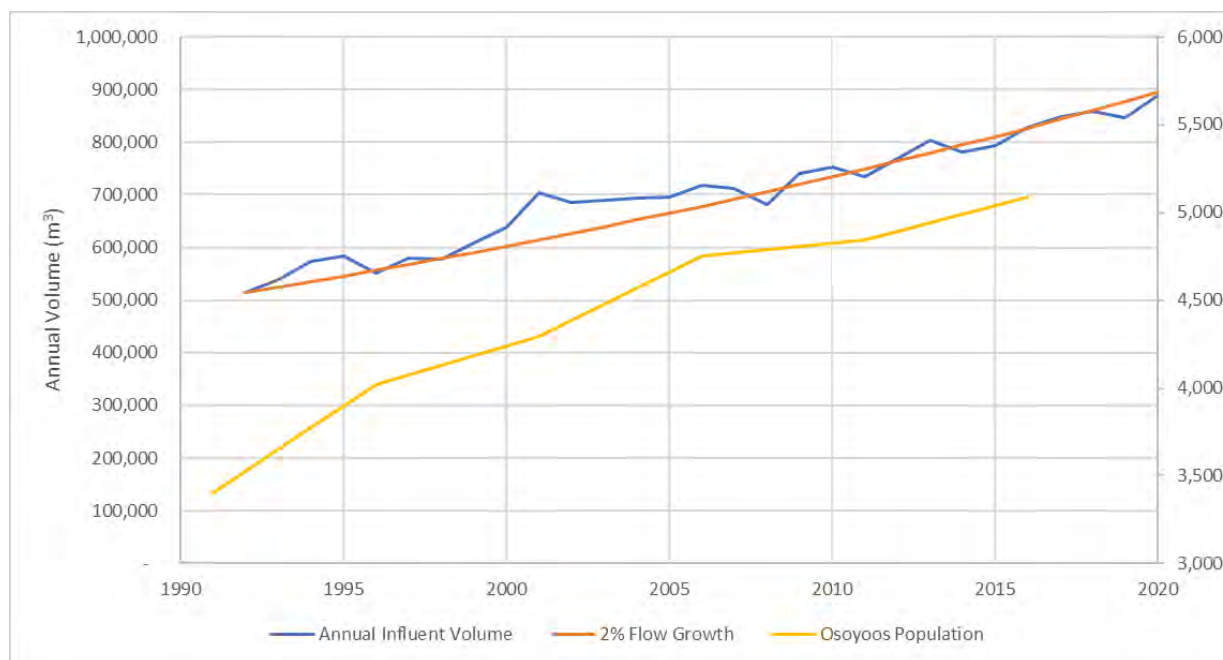
FIGURE 1-1: OSOYOOS AERATED LAGOONS AND RECLAIMED WATER STORAGE



1.2 Flows and Loads

The flows at the Main Lift Station and the Golf Course Lift Station are combined to give the total plant inflow. There has been a consistent growth in the plant flows since 1992. Annual wastewater volumes have increased at an average rate of 2% in the period since 1992.

FIGURE 1-2: ANNUAL WASTEWATER VOLUME VS POPULATION WITHIN TOWN BOUNDARY



The data has been further broken down into monthly flows in order to show how seasonal activity is affecting flow (See Figure 1-3).

The flow that occurs in the winter period reflects base flow from the resident population. The growth rate in winter flows from the resident population is estimated at 1.8% per annum. The Census population growth from 1996 to 2016 averaged 1.3%. The growth in flows was also affected by bringing in new sewer areas, such as the North West Sector. The annual maximum monthly flow includes flows related to tourist activity. The total flow appears to be increasing at the same rate as the minimum flow. The difference between winter and peak summer appears to be relatively stable at around 1,500m³/d. This has been the case since 2001 and seems to indicate stable tourist activity.

The monthly average flow does not capture peaks related to holiday weekends and rainfall. The daily flows for 2019 are shown on Figure 1-5. It can be seen that the annual peak flow coincides with the Canada Day and BC Day long weekends. Winter flows occur from November to April.

Flow projections were made by Kerr Wood Leidal in 2014 in their report 'Capacity Assessment Sanitary Sewer Infrastructure'. They projected the population and flow both within and outside the town boundary. Their expectation was that the resident population within the town boundary would grow by 17% over 22 years from 2011 to 2033, which is an average growth rate of 0.8%. Industrial / commercial / institutional and seasonal growth would be around 0.4% over the same period. KWL predicted significant population growth outside the town boundary. How much of the additional flow is permitted with the sewer system would be at the discretion of the Town.

FIGURE 1-3: PLANT INFLOW (AVERAGE BY MONTH), 1995 – 2019

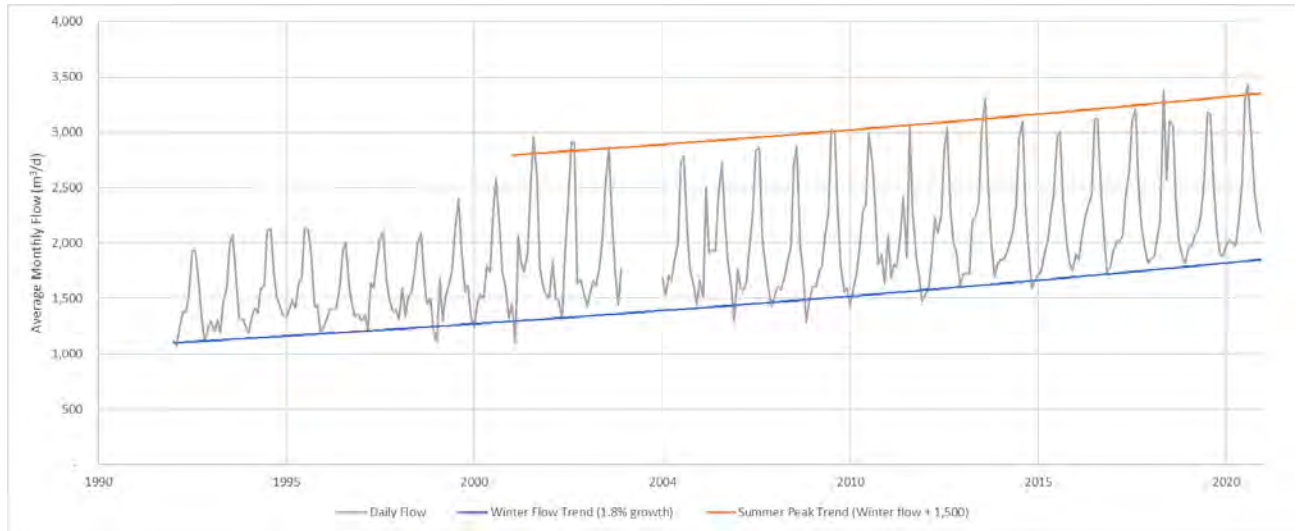


FIGURE 1-4: PLANT INFLOW (AVERAGE BY MONTH), 2012 - 2019

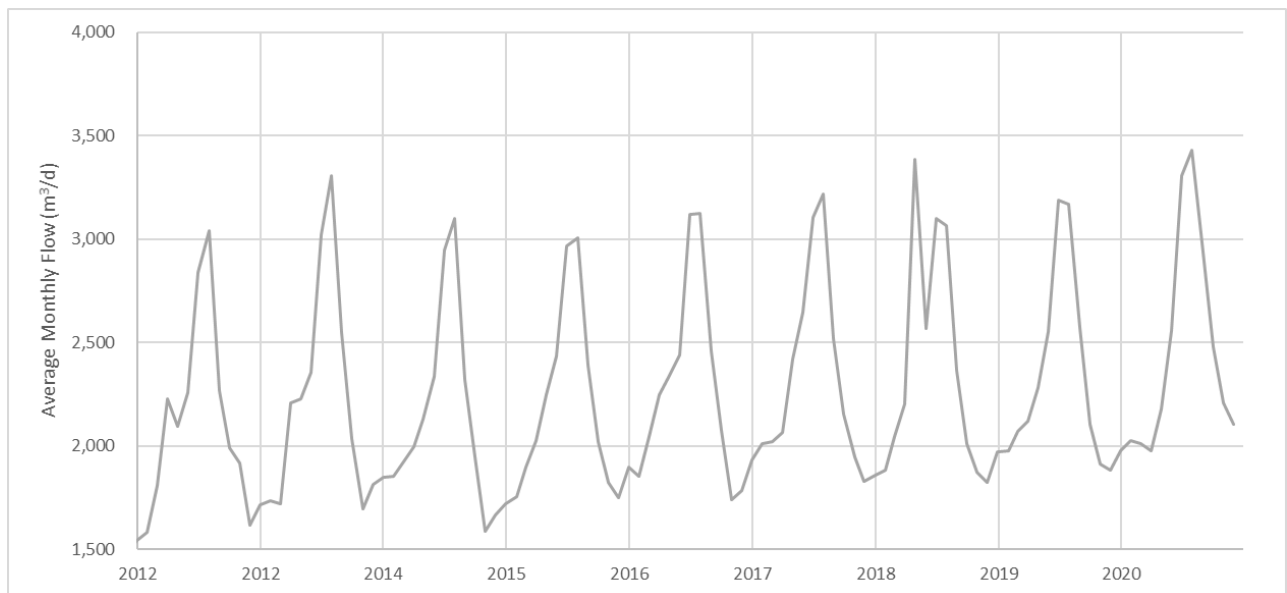


FIGURE 1-5: DAILY FLOW (2019)

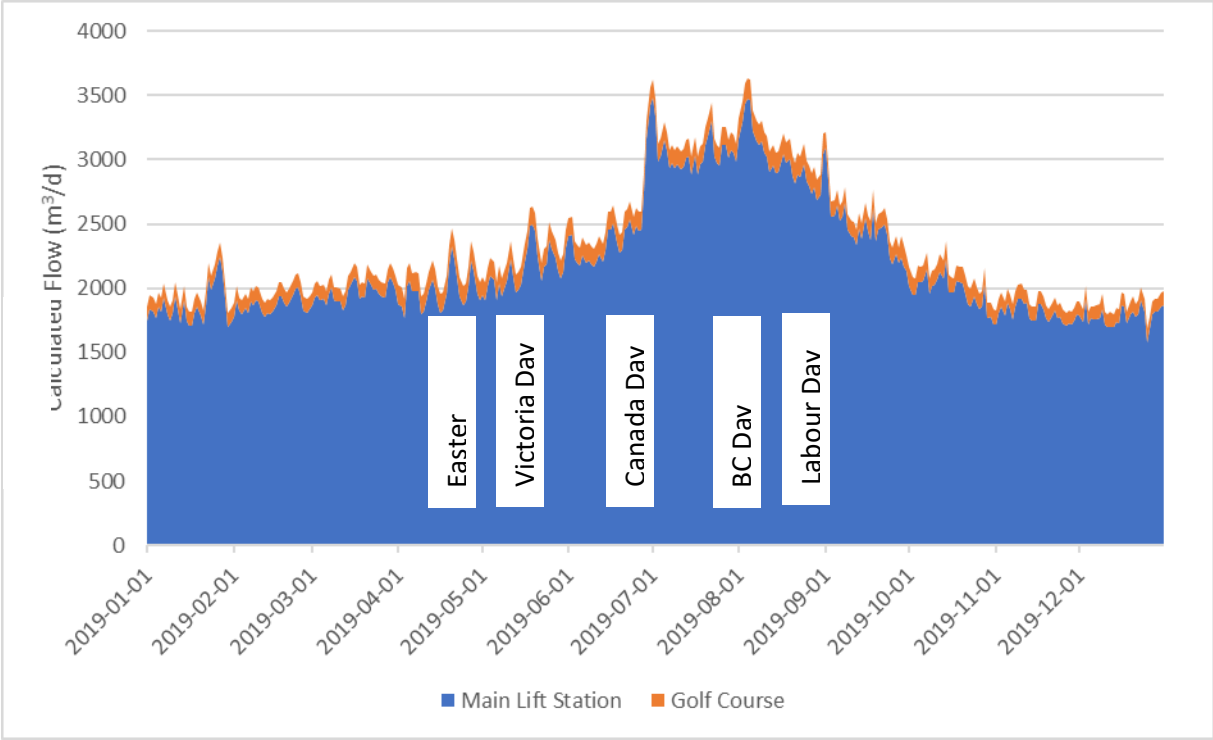


FIGURE 1-6: IRRIGATION FLOW (MONTHLY AVERAGE)

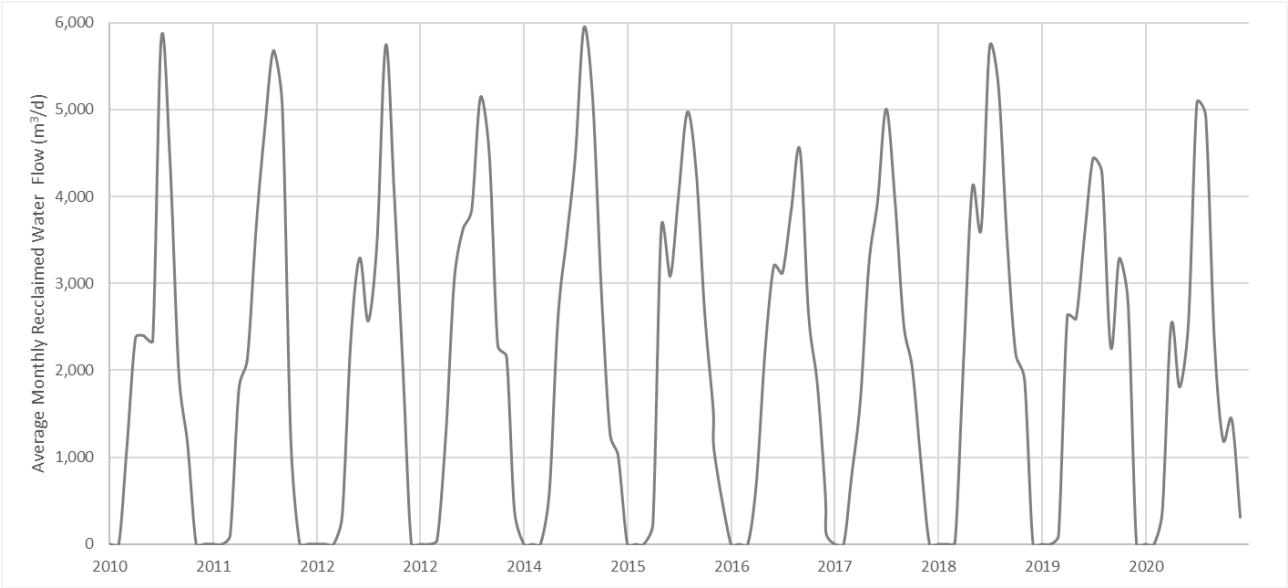
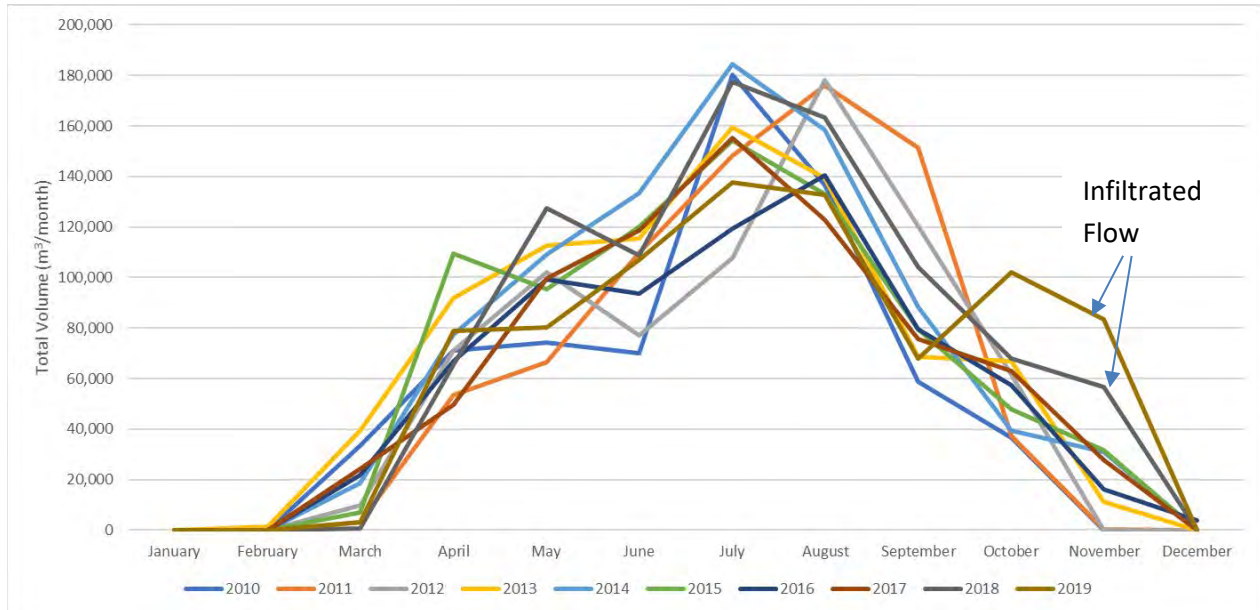
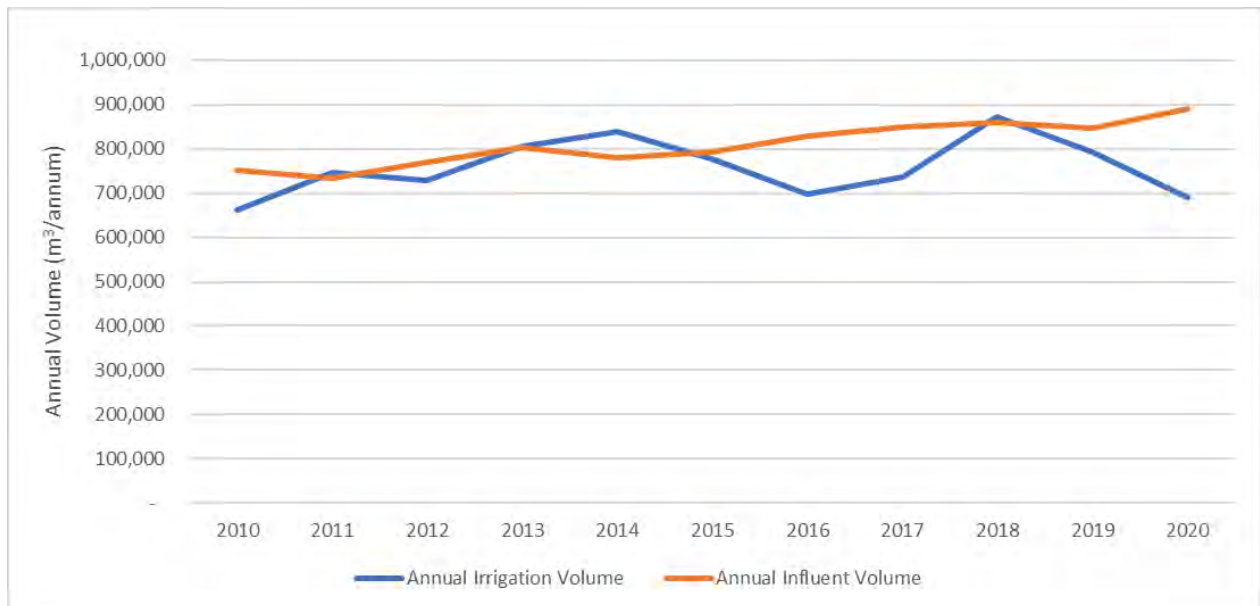


FIGURE 1-7: IRRIGATED / INFILTRATED VOLUME BY MONTH



It is worth noting that while inflow continues to increase, the volume irrigated has not always kept pace with this flow. New users for irrigated water will have to be identified as inflow increases, or the excess water will need to be disposed of by infiltration.

FIGURE 1-8: ANNUAL INFLOW VS IRRIGATION / INFILTRATION VOLUME

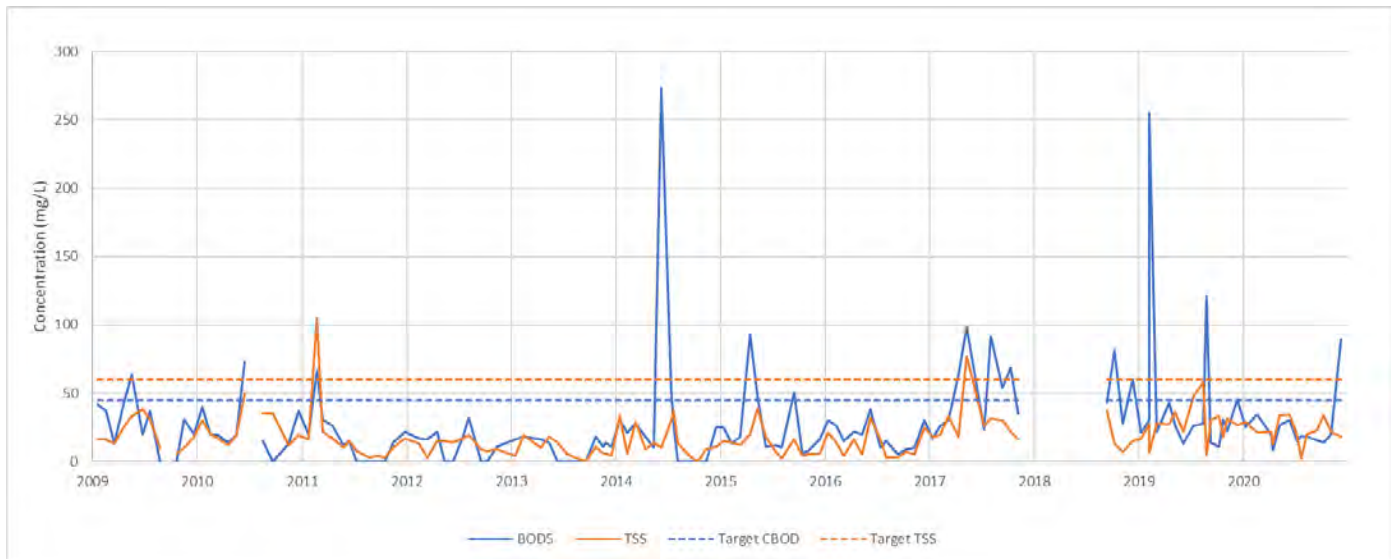


1.3 Existing Treatment Performance

1.3.1 Effluent Quality

The basic treatment performance of the plant is based on five-day biochemical oxygen demand (BOD₅) and Total Suspended Solids (TSS). These results are shown as Figure 1-9.

FIGURE 1-9: LAGOON EFFLUENT QUALITY



Effluent samples were not taken from January to August 2018 while the lagoon liner and aeration system were being replaced in Cell 3.

It can be seen that effluent BOD and TSS have been greatly in excess of target (permit) values in recent times. At face value they indicate a major issue with the treatment system. **However, there is reason to doubt the results, as there are inconsistencies that cast doubt on their accuracy.** A specialist sampling contractor has been engaged to ensure consistency in sample collection.

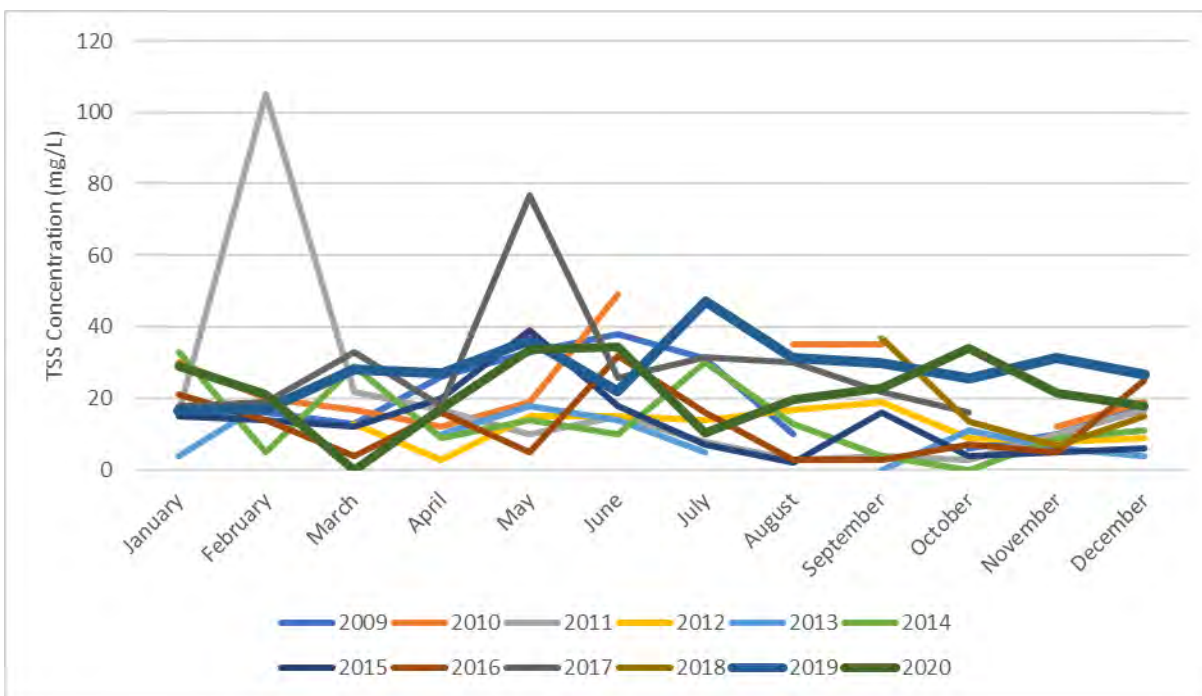
In addition, the BOD results do not directly compare with the BOD test method required by the permit. The Municipal Wastewater Regulation (2012) defines BOD₅ as being the five day carbonaceous biochemical oxygen demand (noted in this report as CBOD). The plant's operational certificate (ME-12214) allows a maximum CBOD of 45mg/L. Water quality testing to date has generally been the standard biochemical oxygen demand (BOD₅), which is the default used by BC laboratories unless they are told to do otherwise.

While the results of the BOD₅ and CBOD test are often similar, they can differ significantly in aerated lagoon effluent. The conventional five-day biochemical oxygen demand tests carbonaceous biochemical oxygen demand (CBOD) plus nitrogenous biochemical oxygen demand (NBOD). The nitrogenous biochemical oxygen demand consists of the oxygen required

to convert ammonia to nitrate. Because aerated lagoons operate at the ambient temperature, little nitrification occurs in the cooler months. At times when the BOD sample is inoculated with nitrifying bacteria but incomplete nitrification has occurred, the NBOD component can be significant. As a consequence, the system may fully comply with the maximum 45mg/L CBOD requirement, but appear to fail based on BOD₅. A period of analysis of both parameters (BOD and CBOD) is recommended to demonstrate the relationship.

The plant has also had exceedances on total suspended solids. The suspended solids don't show a clear seasonal pattern (Figure 1-10). Summer values are commonly high as a result of algae growth.

FIGURE 1-10: ANNUAL TSS TREND



Nitrification to remove ammonia adds to oxygen requirements, but significantly reduces chlorine demand. Figure 1-11 and Figure 1-12 indicate that nitrification has been occurring in the summer months.

Nitrification should also be occurring in the storage lagoons during the warmer months. However, because the reservoirs receive a large part of their flow during periods when little nitrification is happening, reclaimed water ammonia levels don't reach the low levels which occur during summer in the Cell 3 effluent.

FIGURE 1-11: CELL 3 EFFLUENT NITROGEN – JULY SAMPLE

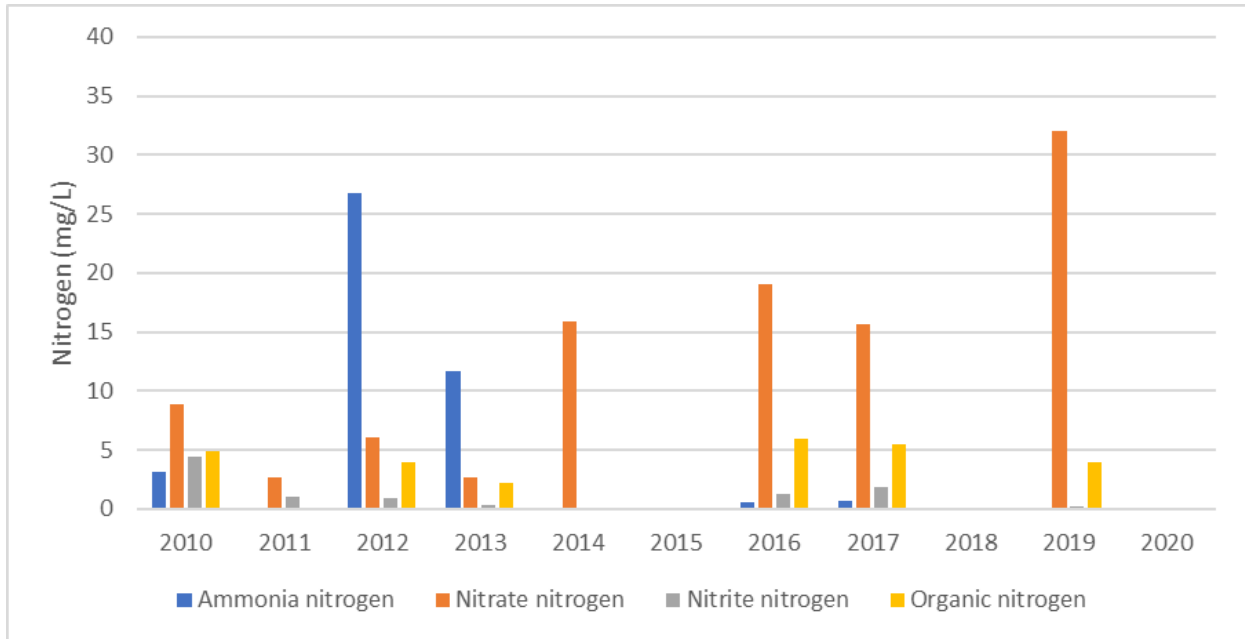


FIGURE 1-12: CELL 3 EFFLUENT NITROGEN – JANUARY SAMPLE

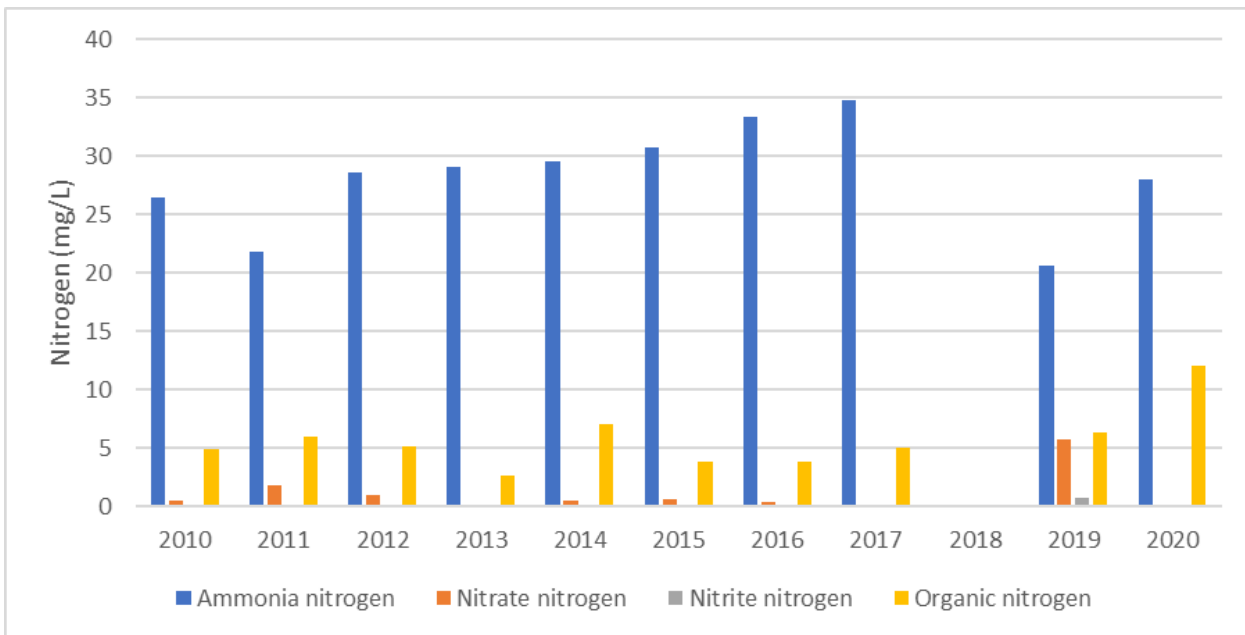


FIGURE 1-13: RECLAIMED WATER NITROGEN – JULY SAMPLE

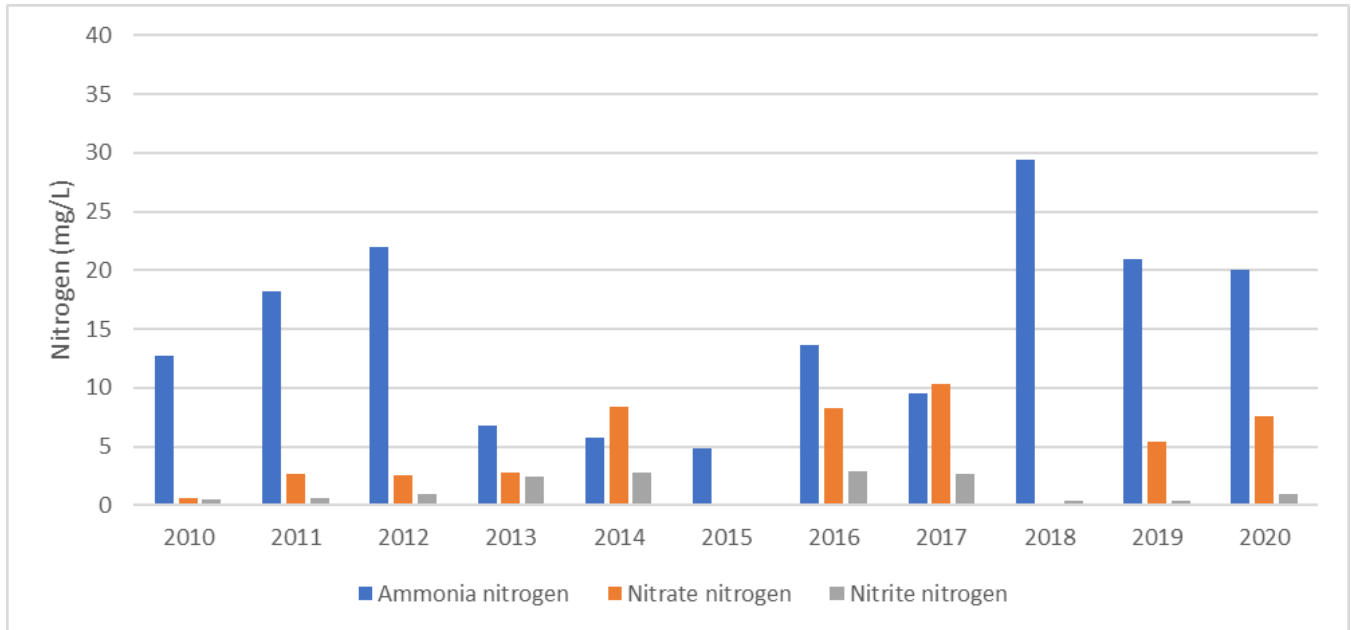
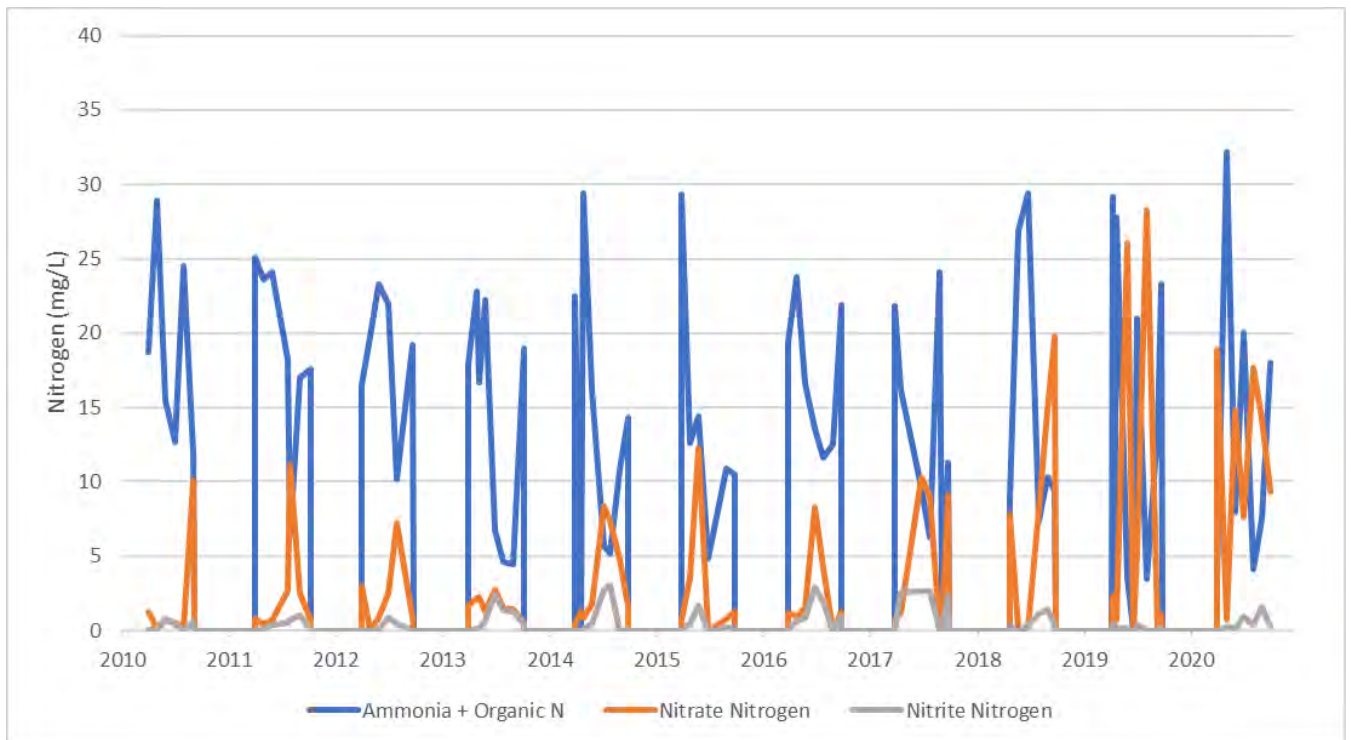


FIGURE 1-14: RECLAIMED WATER NITROGEN – ALL SAMPLES



1.3.2 Odour Control

Odour has been a major issue at the WWTP, and in the collection system. There have been numerous complaints and the issue has featured in news articles. The issue appears to relate to hydrogen sulphide generation in the sewer force mains. This is released later when the gas can escape, such as at the WWTP. Hydrogen sulphide has a characteristic rotten egg odour and is detectable at concentrations as low as 1 part per billion, so the odour can carry long distances.

Due to the layout of the sewer system and the topography of the Town, Osoyoos has many long force mains with correspondingly long residence times. All sewage flow is directed to the Main Lift Station, except flow from the Golf Course lift station.

Aside from the odour, hydrogen sulphide is toxic at elevated concentrations. It also combines with water in damp conditions to form sulphuric acid, which corrodes metals and concrete. It is common to find significant decay of infrastructure in gravity sewers downstream of forcemains connections. Therefore, the sulphide is more than an odour issue and it needs to be addressed to avoid the cost of widespread sewer replacement.

The Main Lift Station includes a fan to draw air from the building for treatment in an activated carbon filter. Nevertheless, hydrogen sulphide levels have been measured inside the manhole upstream of the Main Lift station (PSAB A189 / ID SMH0289) in excess of 100mg/L. Elevated levels around 6mg/L are commonly recorded inside the Main Lift Station building. This gas has already caused blackening of copper water pipe inside the new building. In British Columbia, the Occupational Exposure Limit for H₂S is 10 ppm. It appears that this limit is occasionally exceeded (see Figure 1-15). No forcemain swabbing has been undertaken, as the forcemains are not configured for this function. Forcemain swabbing can have some impact on odour by removing the slime layer on the pipe walls. An alternative to swabbing is shock dosing of sodium hydroxide, where the forcemain pH is elevated above 12.5 for at least 30 minutes to inactivate the sulphate reducing bacteria.

The most effective approach to addressing the pipeline odour is ongoing chemical addition to inhibit H₂S formation. Various chemical dosing options are available;

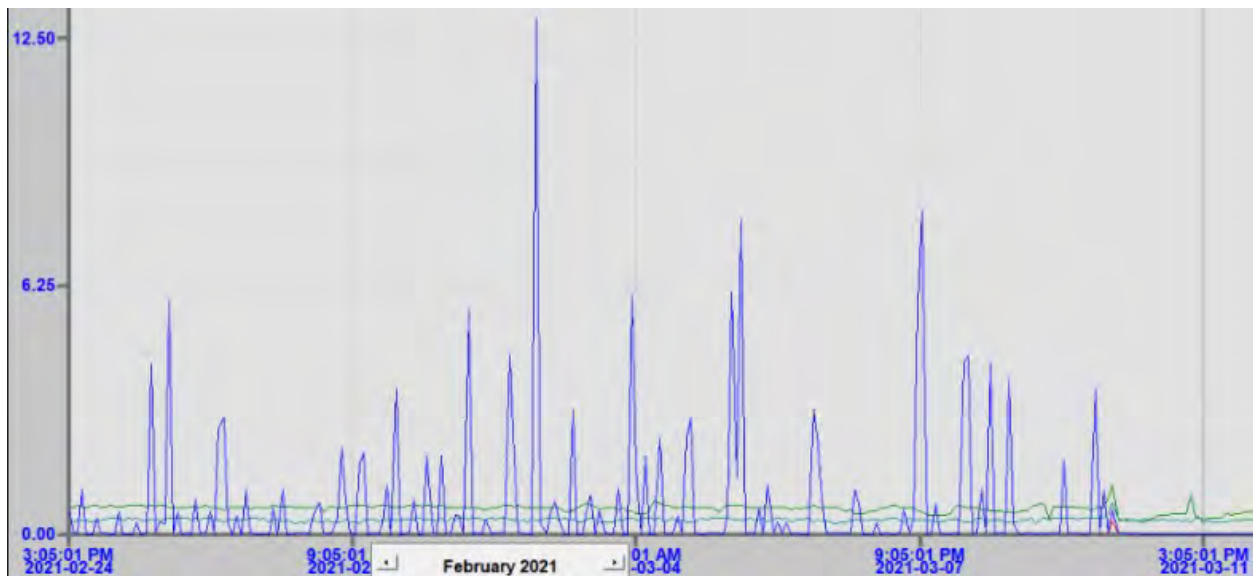
- Air, oxygen
- Nitrate
- Chemical oxidants (hydrogen peroxide, hypochlorite, ozone, potassium permanganate)
- Iron salts (ferrous sulfate, ferrous chloride, ferric sulfate, ferric chloride)
- pH adjustment chemicals to increase pH over 8 (magnesium hydroxide, calcium hydroxide, sodium hydroxide, lime)
- Proprietary formulations (e.g. Biomaxx)

The choice of chemical is dependent on many factors including equipment costs, chemical costs, severity of the issue and chemical handling constraints.

A proprietary formulation (Biomaxx®) was dosed at some time after April 2008 at Lakeshore at 44th LS and Oasis LS to address odour issues at the Main Lift Station. It is proposed that this be reinstated on a trial basis, as the dosing equipment is in place and chemical handling safety systems such as safety showers are not needed.

The Biomaxx system could be extended to other force mains based on need, and other chemicals could be considered. For example, if dosing were to be implemented at the Main Lift Station, then options such as magnesium hydroxide or ferric salts may be more cost effective due to the high flowrate and the availability of amenities to support more complex dosing systems. This is the subject of a separate study and is not discussed further in this report.

FIGURE 1-15: H₂S LEVELS INSIDE MAIN LIFT STATION BUILDING (FEBRUARY 2021)



2.0 Component Status

2.1.1 Complete Mix Lagoon (Cell 1)

Cell 1 does not have an ideal configuration for use as a complete mix reactor. It was originally designed in 1978 to operate as a sludge settling lagoon. It had surface aerators and a paved ramp for drive in access.

The width and sloping sides mean that there is effectively no flat floor area. The bottom elevation is also 1.2m higher than the other lagoons. As a result, the suspended aerators are estimated to be set around 1.8 – 2.5m higher than the other lagoons, and only one third of the area of the lagoon can be aerated.

Having the aerators set higher than the other lagoons means that air will preferentially flow to these diffusers. This can be controlled by throttling the valves on the laterals but, as the necessary air pressure is being generated anyway, it makes sense to have the diffusers set at the same elevation as the other lagoons. This is because the deeper a diffuser is set, the more efficient the transfer of air becomes.

With such a small part of the lagoon area being aerated and mixed, only part of the lagoon is actually operating as a complete mix reactor. The active volume would account for perhaps half of the total volume. As a result, the effectiveness of treatment is significantly reduced, and large parts of the lagoon are only aerated through circulation of the contents. This may contribute to the generation of odour.

Because the diffuser membranes on the Cell 1 aeration system had not been replaced since 1994, they were replaced in 2020. This should greatly improve aeration efficiency in this cell.

FIGURE 2-1: CELL 1 GENERAL ARRANGEMENT

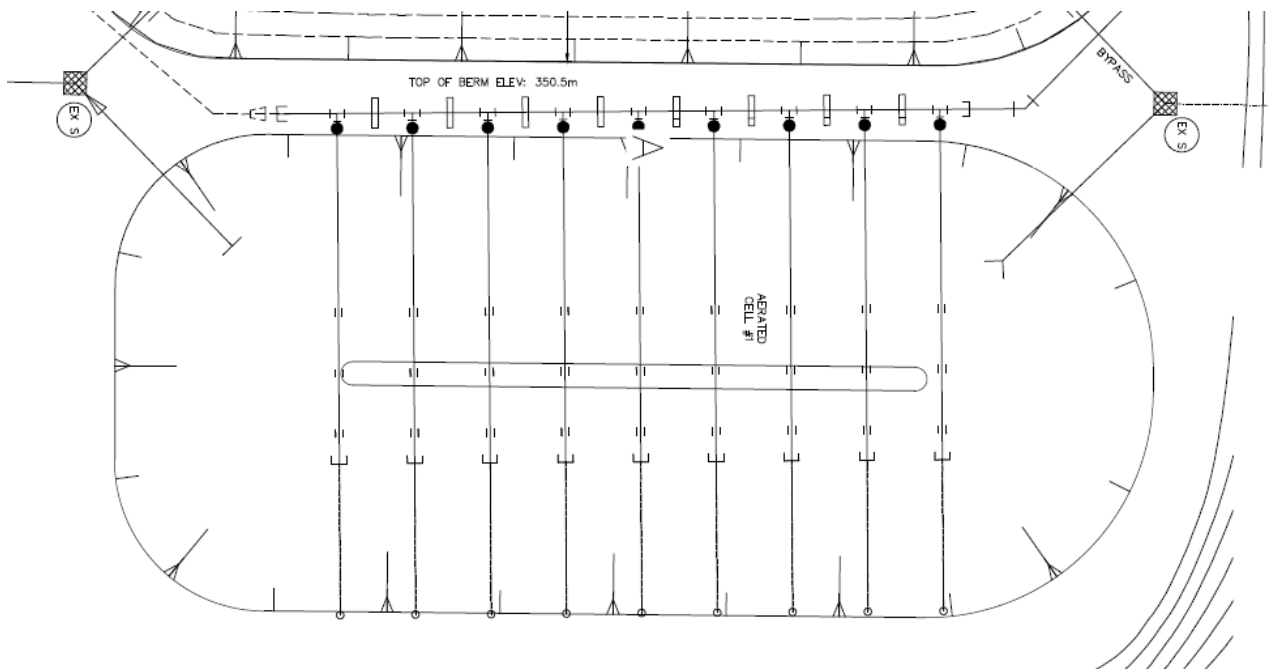


FIGURE 2-2: GOOGLE EARTH IMAGE OF CELL 1 (2013)



2.1.2 Partial Mix Lagoons (Cell 2 and Cell 3)

The existing partial mix lagoons are in good condition and appear to be operating within normal parameters. While they have theoretical capacity for the proposed future flows, the final word is their measured performance. As described previously, if the reported effluent data is correct, this indicates unexpected issues with performance which would require further investigation.

2.1.3 Air System

The aeration systems were installed in the lagoons in 1998 (Cell 1), 2010 (Cell 2) and 2018 (Cell 3). The key component is an EPDM membrane with fine perforations that allows very small bubbles to pass into the lagoon. The membrane is supported on a weighted plastic structure that can be raised to the surface for maintenance. The whole unit is referred to as a diffuser. The expected life of the EPDM membranes is 5 – 10 years. After this time the membranes lose elasticity and effectiveness. The membranes in Cell 1 and Cell 2 were replaced in 2020.

It should be noted that the diffusers in Cell 1 differ significantly from what is listed on the original drawings (28 x Magnum 88S). Aeration capacity is less than originally designed. Further, the diffusers marked as 70S in the table are a non-standard size with four 700mm long membrane tubes. A 1000mm membrane must be trimmed to fit them. The 70S label was created for this project and the manufacturer does not sell a unit for lagoons with these dimensions.

TABLE 2-1: GENERAL AERATION CHARACTERISTICS

Cell	Treatment Volume (m ³)	Residence Time (Summer Peak Flow ¹) Days	Number of EDI Aeration Diffusers	Design Aeration Capacity ² (Nm ³ /min / scfm)	Design Aeration Intensity (Nm ³ /min/m ³)
Cell 1	3,300	0.9	6 x Magnum 88S 5 x Magnum '70S' 15 x Magnum 44S	17 / 600	0.005
Cell 2	27,000	8	15 x Magnum 88F 29 x Magnum 44F	29 / 1000	0.001
Cell 3	58,000	17	11 x Magnum 84F 21 x Magnum 88F	26 / 900	0.001

Air requirements and treatment performance are closely related to ambient temperature. The seasonal variation is shown on Figure 2-3.

When the aeration membranes in Cells 1 and 2 were replaced in 2020 it was noted that the plant is subject to extreme levels of diffuser 'ragging'. Rag materials such as non-degradable 'flushable' wipes wind into rope that collects on equipment in the lagoons. The accumulation is accelerated by flow patterns in the lagoon that take continuous water flow past the aeration diffusers. Rag

¹ A flow rate of 3,500m³/d has been selected to indicate a realistic sustained high flowrate.

² Based on operation at half of maximum capacity for long membrane life.

collects on both the aeration components and the float that allows staff to locate the unit. The rag wrapped around the aeration diffuser can impede air flow and makes it more challenging to recover the unit. Rag on the float can cause the float to sink or even break the rope, making the diffuser difficult to find.

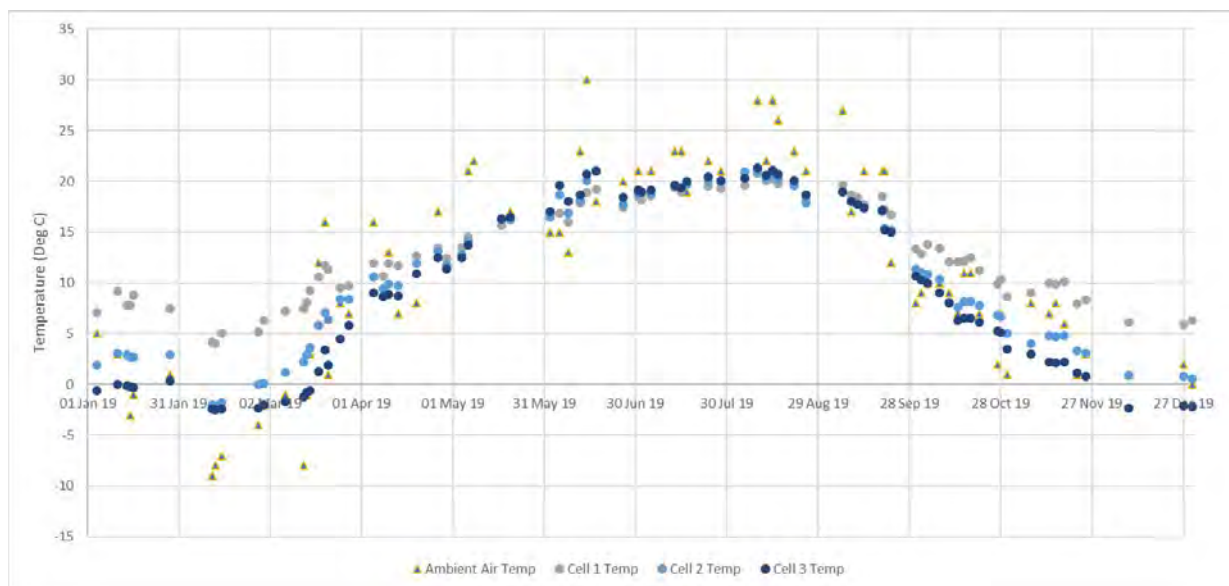
With the commissioning of screening at the Main Lift Station, rag arriving at the plant is now limited to material passing through the Golf Course Lift Station chopper pumps. The Town wishes to remove the rag that remains in the lagoons from the previous years of use in order to preserve aeration performance and avoid harm to the system.

The plant has three Kaeser FB440C blowers operating at up to 2690 rpm, each with a capacity of 978 cfm free air delivery based on 10.2 psig discharge pressure. The blowers run from VFDs but are generally operating at 100% speed. Their inlet capacity is 1650 scfm³.

The blower operating hours are indicative of the air requirements in summer and winter. All three blowers are being used during the peak summer period. Three blowers produce 5,000 scfm. The combined design flow through the diffusers is 2,600 scfm and their maximum (short term) flow capacity is around 5,000 scfm. This indicates that the aeration diffusers are currently being operated at their maximum capacity, which shortens the expected life of the diffuser membranes.

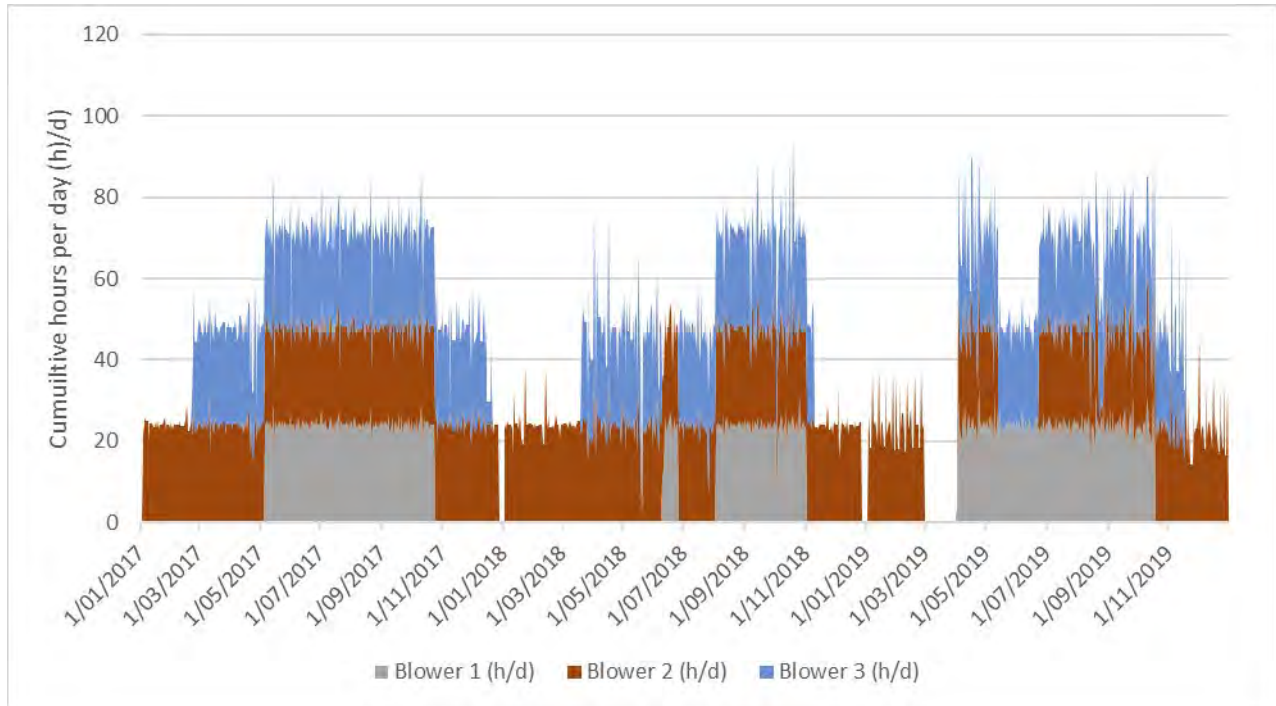
If the existing 60hp motors are replaced with 75 hp motors, the blowers can be upgraded to run at 3,600 rpm for a discharge capacity of 1,225 cfm. 1225 cfm free air delivery is equivalent to 60 Nm³/min (2,100 scfm). Therefore, 180 Nm³/min (6,000 scfm) is feasible if needed for future air requirements.

FIGURE 2-3: AMBIENT AND LAGOON TEMPERATURE TREND (2019)



³ Due to the compressibility of air, blower capacity is commonly based on inlet (atmospheric) air flow.

FIGURE 2-4: BLOWER OPERATING HOURS (2017-2019)



2.1.4 Sludge Accumulation

Sedimentation (settling) of suspended solids is a key function of the partial mix aerated lagoons. Eventually the solids need to be removed from the system once the useful volume and treatment effectiveness are affected. The rate of sludge accumulation in the system depends on several variables including:

- The amount of inert suspended solids entering the system in the raw sewage. The quantity of inert suspended solids will be relatively low because the influent is screened.
- The amount of non-volatile biological solids entering the system with the raw sewage. As sludge accumulates in the bottom of the aerated cells, anaerobic digestion processes occur resulting in volatile solids removal and corresponding reductions in the accumulated solids volume and overall mass.
- The degree of consolidation of the settled solids. As sludge accumulates on the bottom of the lagoons, compression of the actual sludge mass will occur resulting in higher solids concentrations expressed as % solids. Typically, sludge accumulations in aerated cells have solids concentrations in the 5 to 12% range.

Sludge will tend to accumulate near the Cell 2 inlet and away from diffusers. At present it is expected that sludge depths will be less than 1m based on the time since the lagoons were last desludged.

2.1.5 Reclaimed Water Storage

Osoyoos has a total available reclaimed water storage volume of 565,000m³ (Reservoir 1: 270,000 m³, Reservoir 2: 295,000 m³). The system is required to store a minimum of 60 days of flow when storage levels are at their minimum. This is equivalent to approximately 140,000m³. Therefore, the useable volume is 425,000m³ if the reservoirs are 100% full at the start of the irrigation season. Figure 2-5 shows that the reservoirs are typically less than full at the start of the irrigation season. Figure 2-6 indicates the capacity that remains unused by year, taking account of storage volume that has not been filled by the start of the irrigation season.

At present the reservoirs are drained on a more or less alternating basis (see Figure 2-7). The impact this has on chlorination is discussed later in this report.

TABLE 2-2: STORAGE VOLUMES BY YEAR (M³)

Year	Max Volume	Min Volume	Volume Used	60 day storage	Spare Capacity
2010	408,371	238,503	169,868	140,000	255,132
2011	511,515	265,137	246,378	140,000	178,622
2012	527,294	283,799	243,495	140,000	181,505
2013	527,234	265,800	261,434	140,000	163,566
2014	502,308	232,050	270,257	140,000	154,743
2015	477,064	175,531	301,533	140,000	123,467
2016	458,005	237,091	220,914	140,000	204,086
2017	512,561	217,088	295,473	140,000	129,527
2018	499,834	255,615	244,219	140,000	180,781
2019	482,098	212,696	269,401	140,000	155,599
2020	436,449	257,872	178,577	140,000	246,423

FIGURE 2-5: TOTAL STORAGE VOLUME BY YEAR

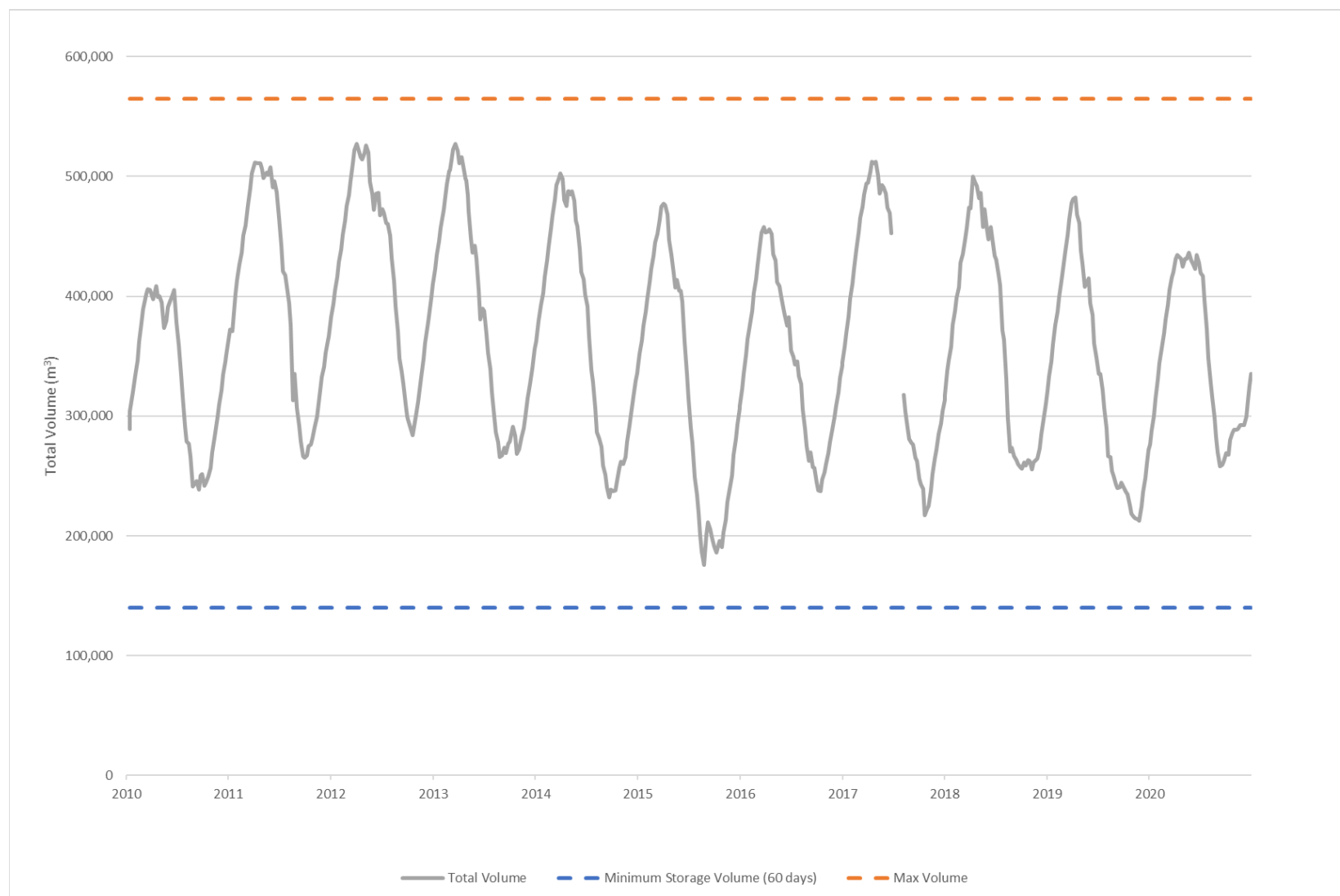


FIGURE 2-6: RESERVOIR SPARE CAPACITY BY YEAR

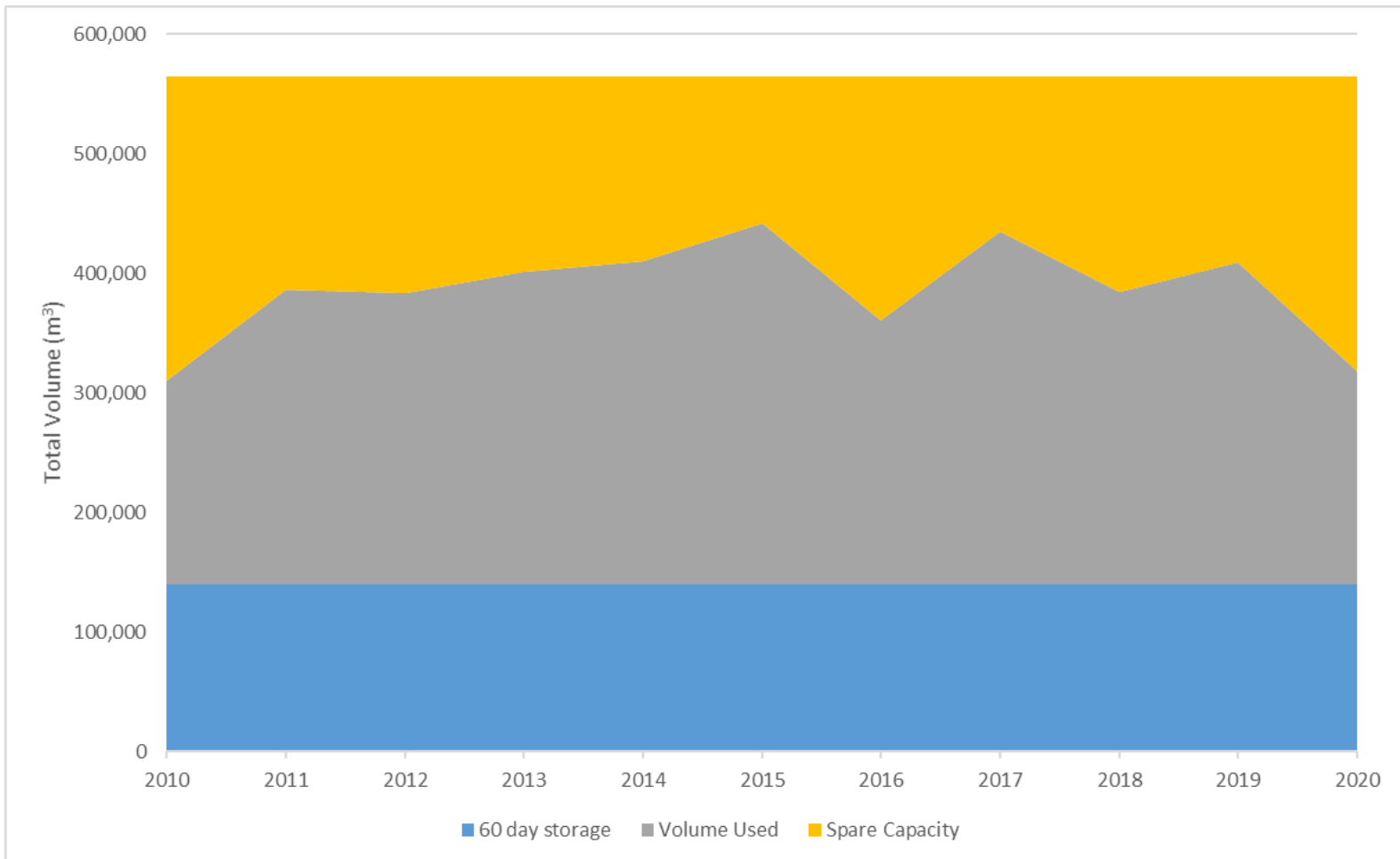
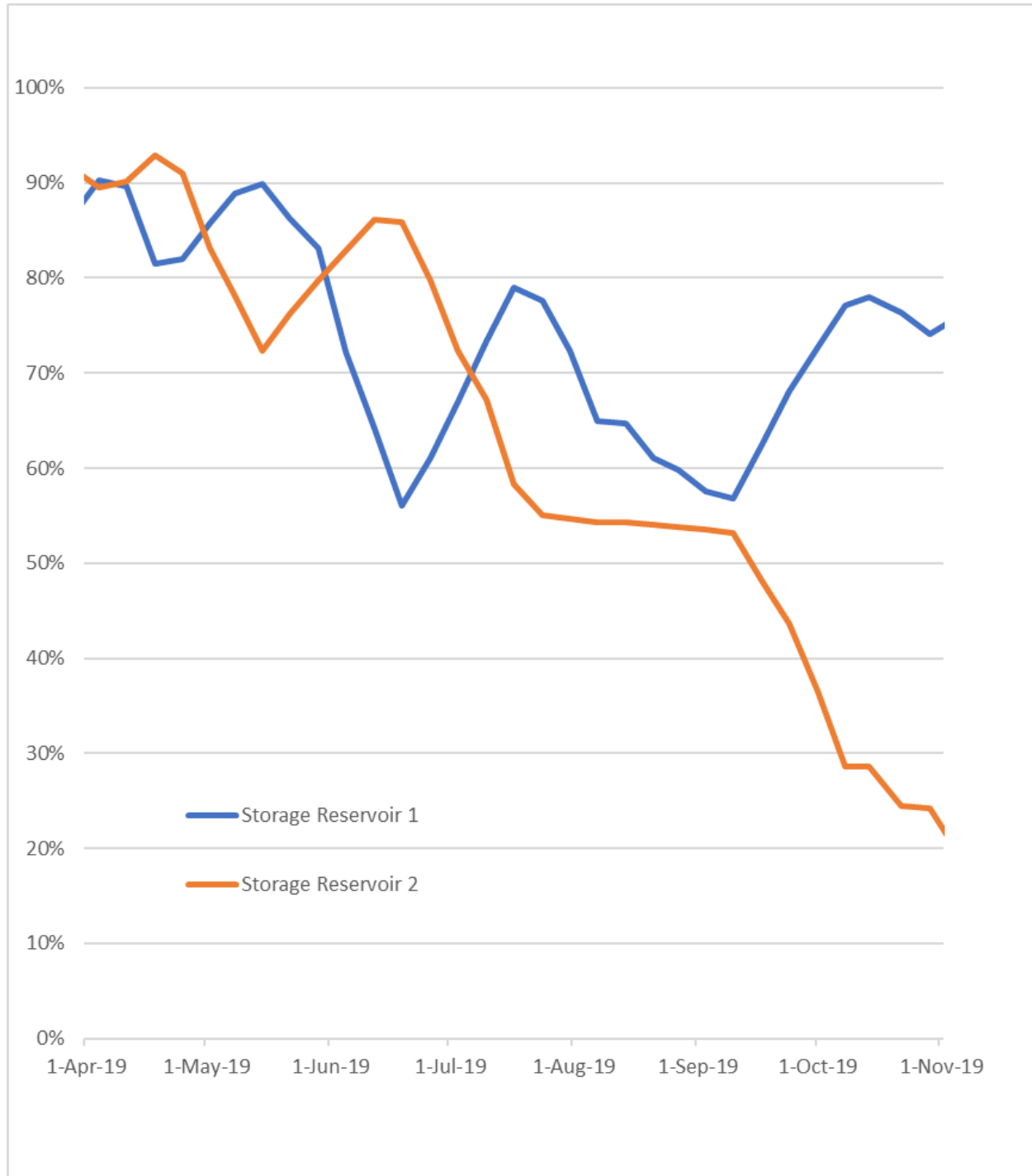


FIGURE 2-7: RESERVOIR % FULL (2019)



3.0 Design Conditions

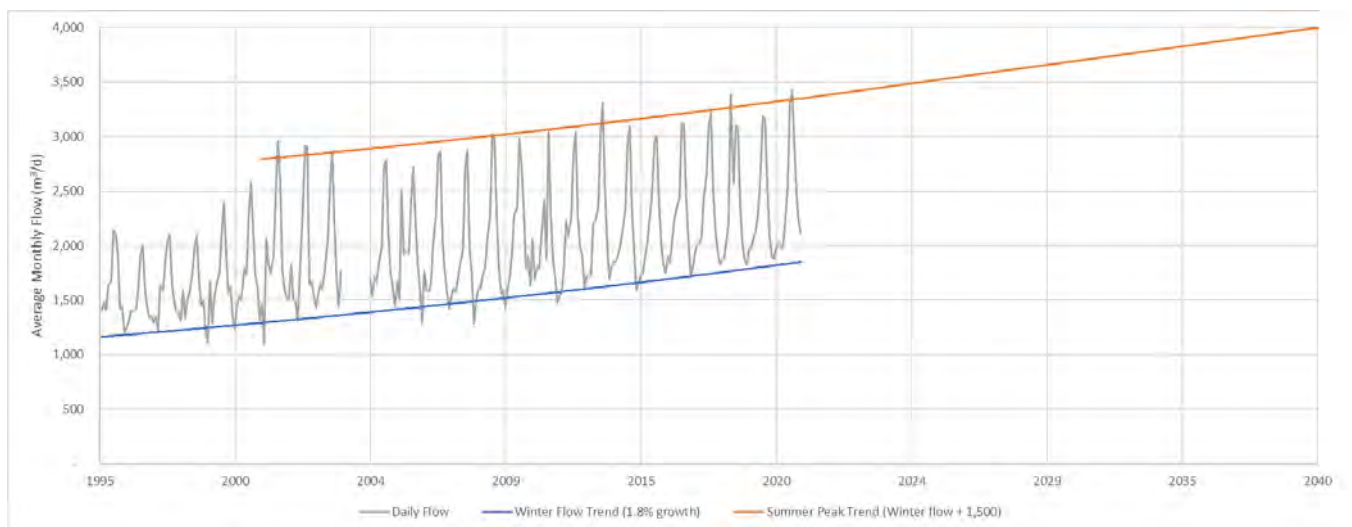
3.1 Design Flow

The required plant treatment capacity is driven by peak flowrate. The **proposed design flow for the design of treatment systems is 4,000m³/d**, which represents the expected summer flow in 20 years at the current rate of growth of the permanent population plus an allowance for the summer population.

It is proposed that this allowance for future growth is conservative for the following reasons;

- The Kerr Wood Leidal report states that there are limited opportunities for population growth inside the town boundary.
- Part of the increase in flow from 2013 to 2016 was due to service area expansion outside the town boundary, not just population growth.
- Future service area expansion has not been decided. Developers outside the town boundary may choose to build independent sewage collection and treatment systems. The Town may also choose not to accept additional waste.

FIGURE 3-1: PROJECTED 2040 FLOW



3.2 Target Effluent and Reclaimed Water Quality

The plant treatment targets are set by Operational Certificate ME-12214 (CBOD 45 mg/L and TSS 60 mg/L). These targets are consistent with the requirements of the Municipal Wastewater Regulation for lagoon effluent.

Treatment targets that are relevant to the Osoyoos WWTP discharge are as follows;

TABLE 3-1: RELEVANT EFFLUENT TARGETS FROM OPERATIONAL CERTIFICATE ME-12214

Quality Parameters	Cell 3 Effluent	Reclaimed Water
Carbonaceous BOD ₅	Max 45 mg/L	NA
Total Suspended Solids	Max 60 mg/L	NA
Turbidity	NA	NA
pH	NA	NA
Fecal Coliforms	NA	<2.2. MPN/100ml
Nitrogen (mg/L as N)	Monitoring required	Monitoring required
Phosphorus	Monitoring required	Monitoring required

The plant Operational Certificate does not align closely with the reclaimed water (greater exposure potential) limits specified by the Municipal Wastewater Regulation (Table 3-2). As a result, reclaimed water monitoring does not generally include CBOD, TSS and turbidity and pH.

TABLE 3-2: MUNICIPAL EFFLUENT QUALITY REQUIREMENTS FOR RECLAIMED WATER (MWR)

Parameters		Indirect potable reuse	Greater exposure potential	Moderate exposure potential	Lower exposure potential
pH		site specific	6.5 to 9	6.5 to 9	6.5 to 9
BOD ₅	Max	5 mg/L	10 mg/L	25 mg/L	45 mg/L
TSS	Max	< 5 mg/L	10 mg/L	25 mg/L	45 mg/L
Turbidity (NTU)	Avg		2	n/a	n/a
	Max	1	5	n/a	n/a
Fecal coliform (/100 mL)	Median	< 1 CFU / < 2.2 MPN	< 1 CFU / < 2.2 MPN	100 CFU	200 CFU
	Max		14 CFU	400 CFU	1000 CFU

The BC Municipal Wastewater Regulation requires a minimum chlorine residual of 0.5 mg/L.

4.0 Lagoon Improvement Options

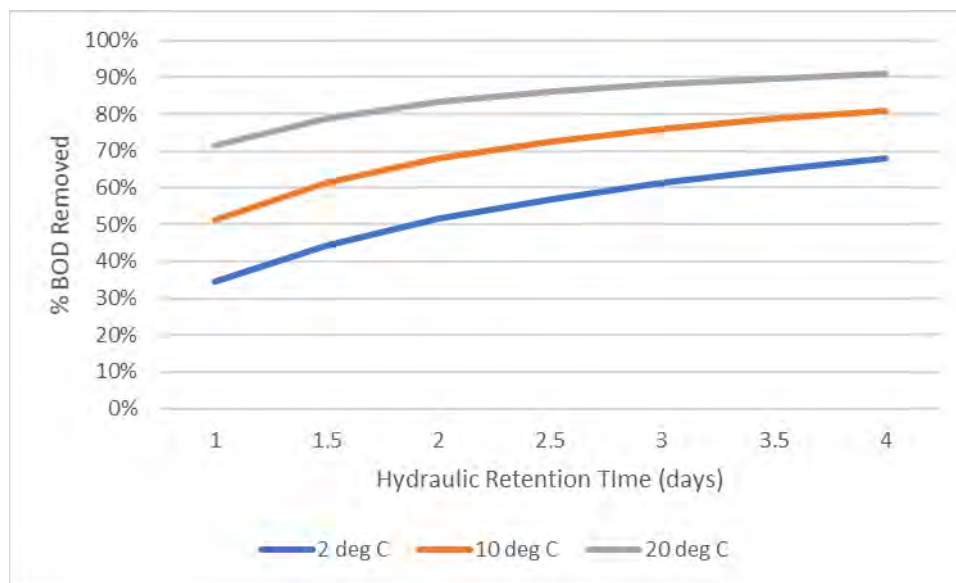
Taken at face value, lagoon effluent quality monitoring (Figure 1-9) indicates that the plant is beginning to exceed effluent quality criteria required under the Operating Certificate (Appendix A). An aeration upgrade was completed in 2020 and the sampling methodology has been revised on the basis of inconsistencies in the results. Monitoring in 2021 will be analysed to determine whether treatment performance is acceptable.

Nevertheless, plant inflow continues to increase, and the day will come when an upgrade is needed. A number of upgrade options have been identified for when the capacity and treatment performance of the plant needs to be improved. These are described in the following sections.

4.1 Upgrade Complete Mix Lagoon System

Complete mix lagoons have a high theoretical treatment capacity relative to partial mix lagoons. Therefore, this process should contribute significantly to treatment capacity. The theoretical BOD removal efficiency for a complete mix reactor is shown by Figure 4-1 based on a temperature dependant rate constant for BOD removal of 2.5 d^{-1} .

FIGURE 4-1: THEORETICAL BOD REMOVAL VS RETENTION TIME AT VARIOUS TEMPERATURES



Because flows have increased steadily over the years and the summer HRT in Cell 1 has fallen below 1 day (based on the full cell volume, not the active area). The USEPA recommends a retention time of 1.5 – 3 days in a complete mix reactor. In order to bring the performance of the complete mix stage closer to recommended conditions, it would be necessary to increase the capacity of the reactor to 2 – 3 days. A $10,000\text{m}^3$ reactor with a residence time of 2.5 days has been assumed. The expected BOD removal can be inferred from Figure 4-1.

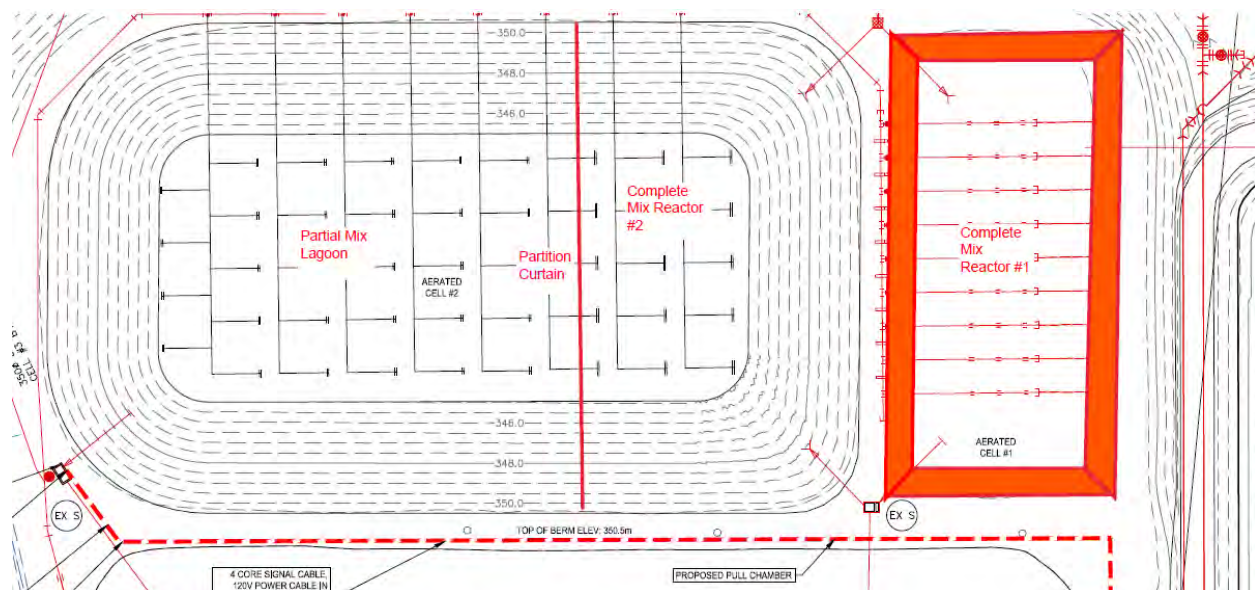
If only one complete mix reactor was required, then partitioning Cell 2 into a complete mix zone and a partial mix zone would be feasible. Cell 1 could be abandoned. However, with this configuration, it would not be feasible to take the combined treatment cell off-line for maintenance while maintaining effluent quality. As a result, this option is considered to be impractical.

The Municipal Wastewater Regulation (s35) requires the plant to have at least two aerated lagoons, with a capacity of 75% of the maximum design flow with one lagoon out of service. In order to achieve these criteria, two complete mix reactors would be needed.

The second complete mix lagoon could therefore be rebuilt within Cell 1 with concrete lined sloping (1:1) or vertical walls to reduce the 'dead' area and increase overall treatment volume. Cell #3 could also be converted to a complete mix / partial mix lagoon although the lagoon shape would make this more challenging for the creation of rational flow paths.

The proposed complete mix system will be larger than the existing system in Cell 1 and the overall WWTP air requirements would increase if operated in the same way. However, because most of the air is used for mixing rather than aeration, it is possible to cycle the air through the laterals in the complete mix system using automated valves, which means the total air requirement can remain within the capacity of the existing infrastructure.

FIGURE 4-2: COMPLETE MIX REACTOR (OPTION 1)



The cost to enhance the existing complete mix reactor for future flows is estimated as follows;

Complete Mix Lagoon Treatment ▪ Excavate and rebuild Cell #1 concrete walls and 5m water depth. ▪ Line with concrete walls and concrete floor ▪ Sludge removal and disposal from Cell #2 ▪ Partition curtain in Cell #2 ▪ New aeration systems c/w automated valve control	\$2,500,000 ± 50%

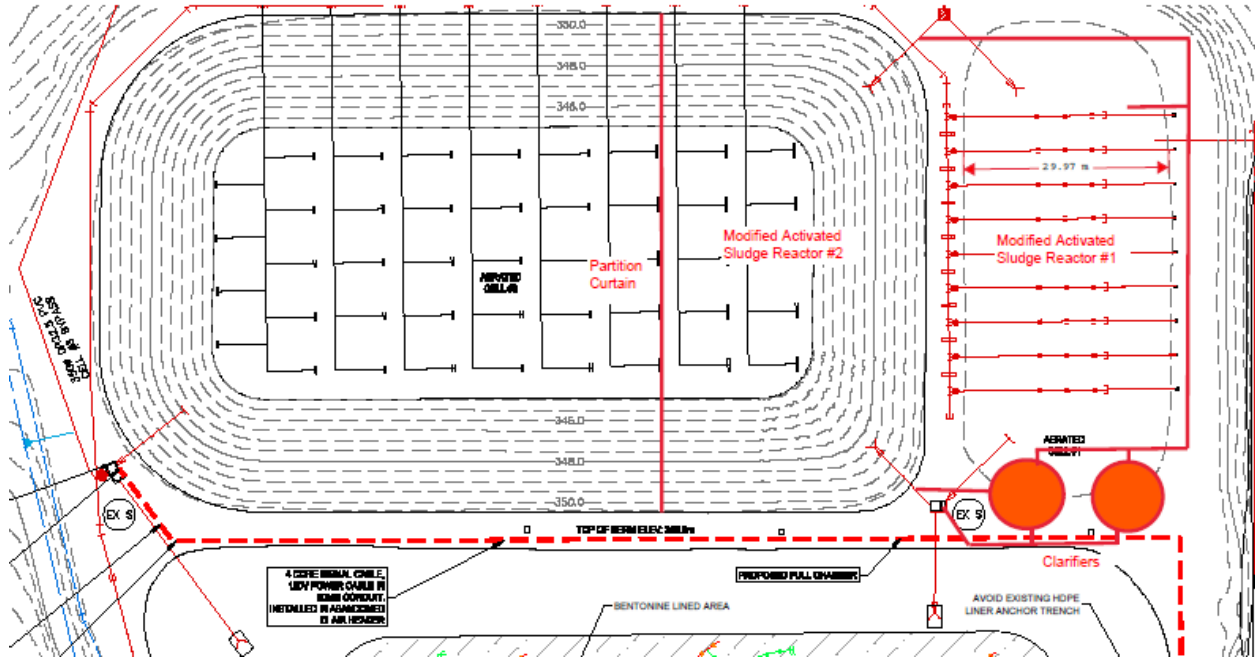
4.2 Modified Activated Sludge Process

Complete mix aerated lagoons are designed to operate as flow-through reactors. Most systems are operated without solids recycle. However, with the addition of systems to recycle solids, the process becomes a basic form of the activated sludge process.

Under this scenario, a pair of ~10m diameter clarifiers would be constructed to allow a proportion of biological solids to be returned to Cell 1. This would prevent the washout of biomass and lead to a higher level of treatment. The design has been assumed to be based on a conventional clarifier. Rather than replacing the existing process with activated sludge plant, the process could operate at a very high rate, with the objective of simply pre-treating the influent. Waste sludge would either be disposed of to the partial mix lagoon system, or dewatered and taken to landfill.

This process would make plant operation more complex due to a need to manage a sludge disposal process in addition to the aerated lagoons. Air demand will increase due to the retention of biological solids in the reactor. The Municipal Wastewater Regulation would also require the construction of two activated sludge reactor tanks in addition to two clarifiers. This would be reasonably challenging to achieve.

FIGURE 4-3: MODIFIED ACTIVATED SLUDGE PROCESS CONVERSION



The cost to complete a modified activated sludge upgrade using the wedge-shaped settling tank is estimated as follows;

Modified Activated Sludge Treatment	\$3,500,000
- Sludge removal and disposal Cell #2 - Partition curtain in Cell #2 - New aeration systems - Additional blower capacity? 300mmØ aeration header extension 300mmØ wastewater pipework 10mØ clarifiers - Sludge recirculation pumping	± 50%

This option might be made more cost effective if a modified version of the clarifier used in the Ecofluid USBF reactor and some oxidation ditches could be adapted for use in a lagoon. This would consist of an open bottom wedge-shaped settling tank mounted off the lagoon floor with an effluent outlet pipe and a waste sludge pump taking effluent and sludge to the next cell. The mixing of retained biological solids back to the lagoon inlet may require a second collection and transfer pumping system. This system would be a novel treatment configuration and would require significant experimentation to achieve the required performance. It is not proposed at this stage as we would essentially be inventing a new treatment process.

4.3 Sequential Batch Reactor

Aerated lagoons can be converted to operate as sequential batch reactors, which fill and draw the waste in order to settle out and retain solids.

The most feasible approach to this upgrade would be to convert an existing cell from an aerated lagoon process to an intermittently decanted extended aerated lagoon / sequential batch reactor system. This process would treat the wastewater in batches, periodically decanting the effluent and maintaining high concentrations of microorganisms without washout. This process could achieve both improved BOD reduction and nitrification, reducing ammonia levels for a greater part of the year. The process requires increased aeration and a decanting system. Construction costs would primarily relate to upgrades to the aeration system, partitions and any pumping requirements.

There are multiple configurations available for this process. The water level can be allowed to rise and fall around 0.5m, or moving baffles can be used to allow the water level to remain constant. The process is marketed in North America by Environmental Dynamics International (IDEAL process) and Bioworks North America, among others.

The Municipal Wastewater Regulation (s35) requires aeration basins to have at least two cells, each with a capacity of 75% of the maximum design flow. A sequential batch reactor (SBR) basin would be created in Cell #1, as well as in part of Cell #2. A new partition curtain would be required in Cell 2 to create two treatment zones.

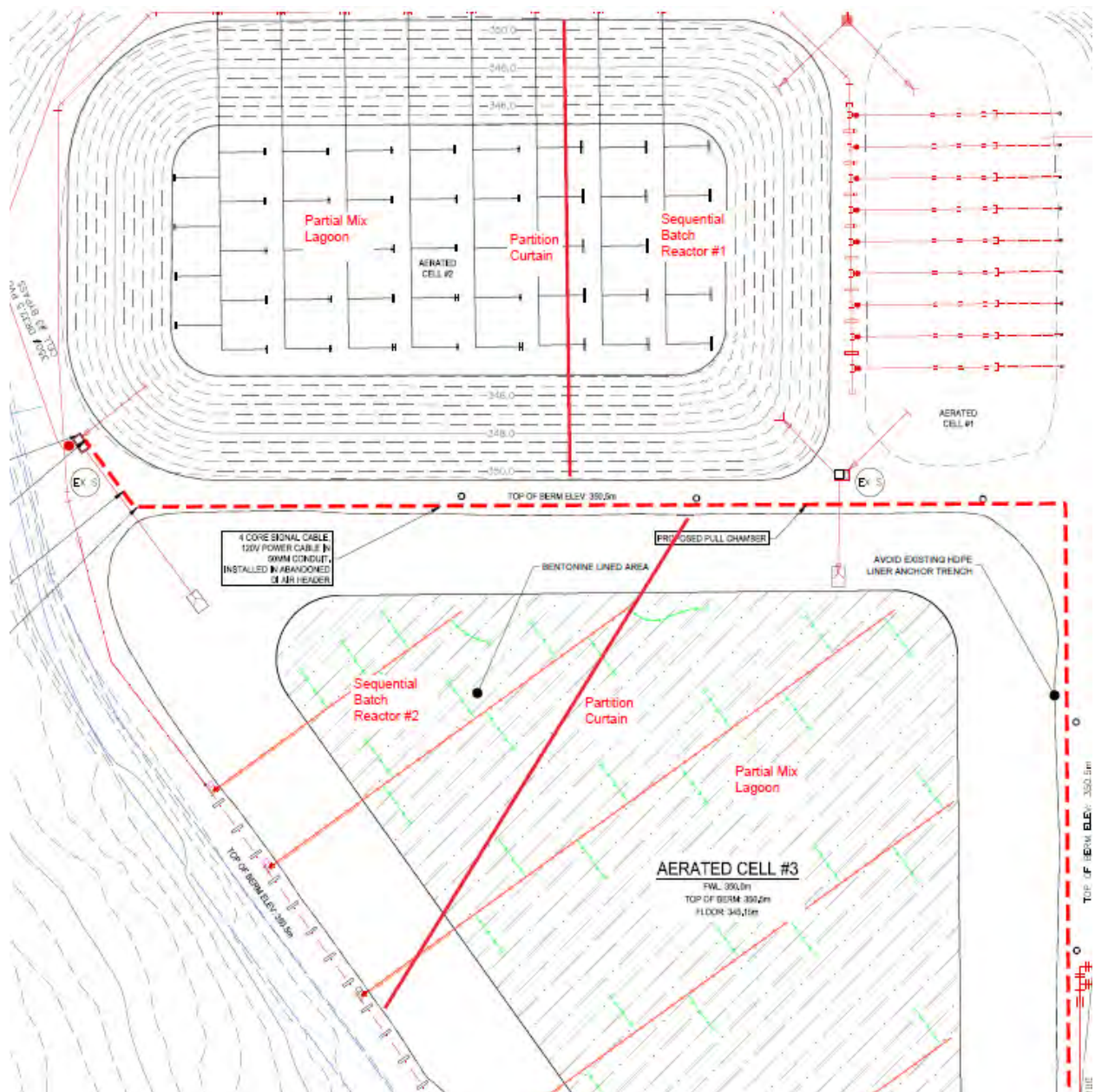
Sludge generated by the process must be collected and disposed of. It has been assumed that the sludge stream would be mixed with the effluent and continue to the partial mix reactors for settling and storage. This is equivalent to what happens at present with the complete mix reactor.

An alternative configuration would see the waste sludge collected, digested, thickened and dewatered for disposal. In this configuration the waste sludge would be discharged to aerobic digesters. Sludge would be thickened by decanting. The thickened sludge would then need to be dewatered by centrifuge or a similar process prior to disposal. This option would be cost prohibitive and difficult to accommodate on the existing site footprint.

The process would be automated with alternating aeration in the two treatment cells. Increased operator expertise would be required to manage this process.

The achievable effluent quality has the potential to meet 25/25 mg/L CBOD/TSS or better if the sludge is removed from the SBR effluent stream (rather than going to the partial mix lagoons). This would require the construction of the separate sludge management processes.

FIGURE 4-4: SEQUENTIAL BATCH REACTOR LAYOUT



The cost for a sequential batch reactor upgrade is estimated as follows;

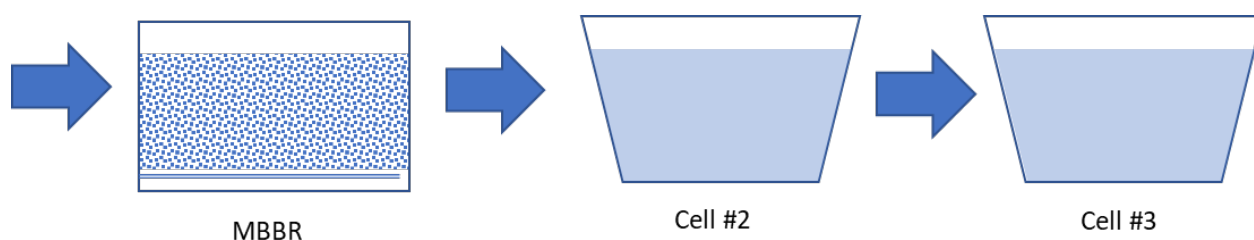
Reconfigure Cells 1 and 2 to SBR Process ▪ Sludge removal ▪ New aeration systems ▪ Excavation, new walls and lower floor in Cell 1 c/w concrete lining and asphalt floor. ▪ Partition curtains in Cell 2 and Cell 3 ▪ Sludge pumping and pipework to Cells 2 and 3 ▪ Effluent decanter systems	\$2,000,000 ± 50%
Stand Alone Sludge Management – Treatment and Dewatering ▪ Aerobic digesters ▪ Centrifuge in new building	\$0.8 – 1.5M ⁴ ± 50%

4.4 Moving Bed Biofilm Reactor

A Moving Bed Biofilm Reactor (MBBR) consists of an aerated concrete tank containing neutrally buoyant media which retains high concentrations of microorganisms without washout. The method of operation is very similar to an aerated lagoon, with sludge management continuing as it is now and little operator training required.

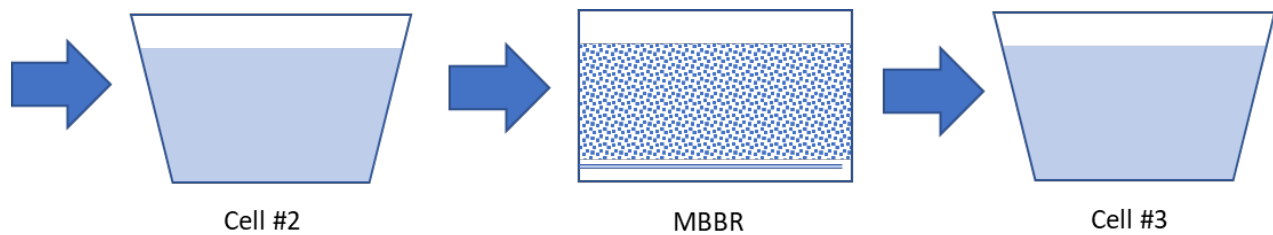
A Moving Bed Biofilm Reactor (MBBR) can be installed upstream of the existing system as pre-treatment, part way through the aerated lagoon process, or downstream for effluent polishing and ammonia removal.

FIGURE 4-5: MBBR PRE-TREATMENT CONFIGURATION



⁴ Cost is heavily dependant on details of the design

FIGURE 4-6: MBBR WITH INTERMEDIATE LOCATION



There is an example of MBBR pre-treatment in Radium Hot Springs, BC. While it is a pre-treatment system, this reactor has a clarifier and sludge digester downstream, rather than discharging directly from the MBBR to their partial mix aerated lagoons (see Figure 4-7). This was intended to reduce the de-sludging frequency of the lagoons. It does mean that operators now need to operate a clarifier and manage sludge daily. They are also dealing with issues related to clarifier performance, such as poorly settling filamentous sludge.

It is recommended that any implementation of the MBBR technology at Osoyoos be kept relatively simple, without separate clarifiers or sludge management. An MBBR pre-treatment configuration as shown in Figure 4-5 will greatly reduce the load on the partial mix lagoons downstream and improve overall BOD removal.

The MBBR process is particularly dependant on efficient influent screening, making the screening of the Golf Course Lift Station flows an essential part of this design. This would require the Golf Course lift station flow to be redirected to the Main Lift Station, or a screening system would need to be installed at the Golf Course lift station. One product that is soon to come onto the market can be fitted inside the lift station and brings screenings to ground level. This system has been included in the budget cost. Screening would also be beneficial for the other treatment options listed in this report.

The intermediate location shown in Figure 4-6 would pass the flow through Cell 2 which would mean that modifications to the Golf Course lift station could be avoided. A simple screen would still be needed on the inlet to the MBBR. Although improved ammonia removal is not strictly necessary at this plant, the intermediate location would achieve this outcome and increase overall BOD reduction, without a need for sedimentation tanks for the MBBR process. The rate of aeration in Cell 2 would need to increase to avoid overloading. This may be as simple as replacing the EDI 44F diffusers with 88F diffusers, which could be done without draining the lagoons.

The MBBR reactor could be placed in the location of Cell 1, using the Cell 1 air supply. The process would be partitioned into two tanks, each with a capacity for 75% of the maximum design flow in accordance with Municipal Wastewater Regulation requirements. As a general concept, the process would be constructed as two 200m³ vertical wall concrete tanks, each containing 80m³ of media, with a media retention screen at the outlet.

The total MBBR air demand would be expected to be around 700 - 1300 scfm. Air flow must be maintained in each operating tank for appropriate turbulence and mixing. Air demand to the

remaining partial mix lagoons would be significantly reduced meaning that the total air flow would be within the capability of the existing blowers. An MBBR aeration system is the coarse bubble type, meaning that regular replacement of diffuser membranes is not needed.

Apart from the competitive cost, some key advantages of the MBBR option are;

- The process is in use or planned at a number of Canadian locations
- The footprint of the existing plant is reduced, making space available for future sludge geotube drying, or other uses.
- Treatment capacity can be increased gradually by adding media and new tanks can be added in the future
- System operation is simple and similar to the existing plant.

FIGURE 4-7: MBBR TANK AT RADIUM HOT SPRINGS WWTP (HIGHLIGHTED)

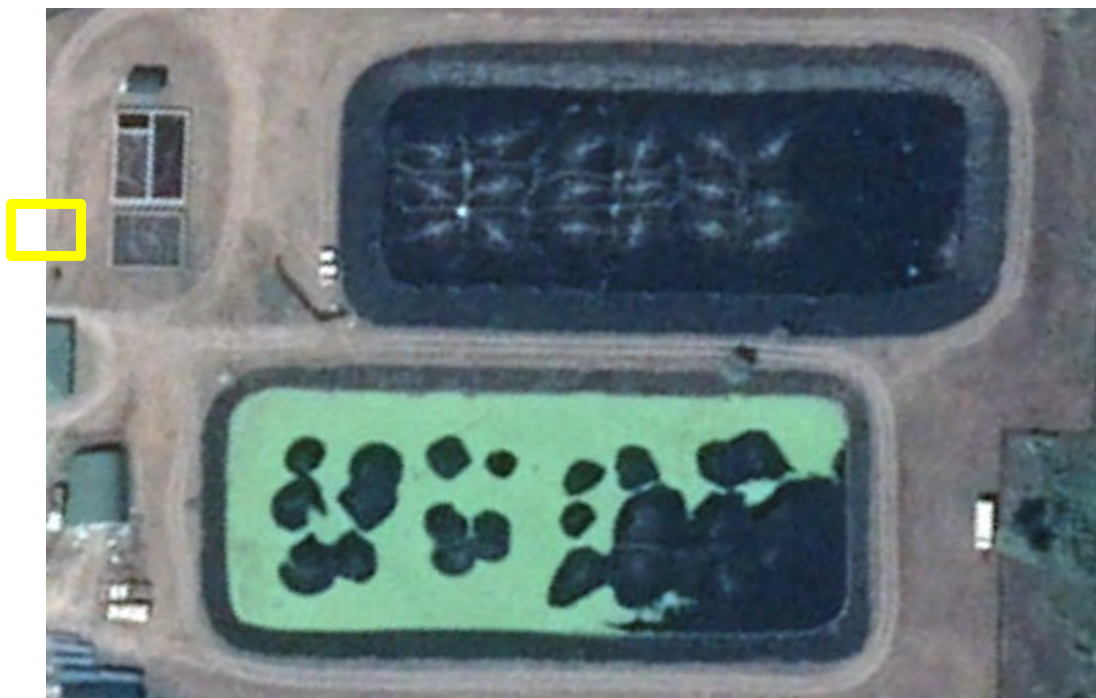
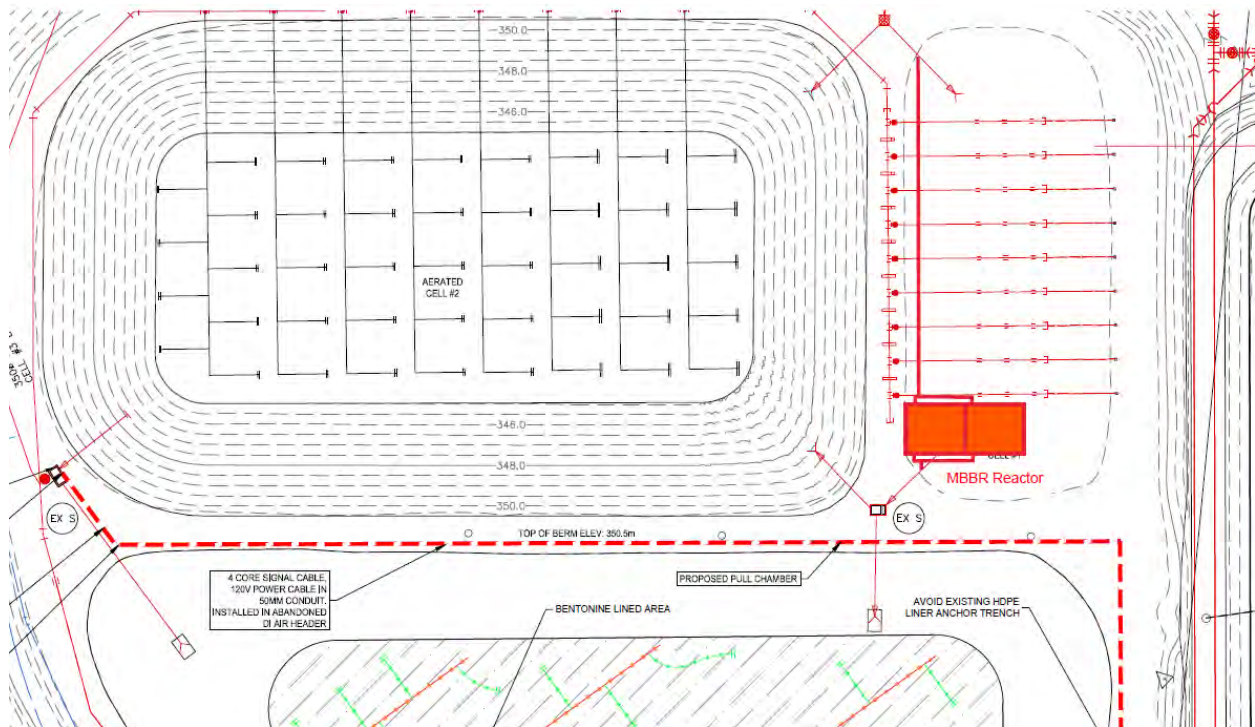


FIGURE 4-8: CONCEPTUAL LAYOUT FOR MBBR PRE-TREATMENT OR INTERMEDIATE TREATMENT



The cost for a Moving Bed Biofilm Reactor upgrade is estimated as follows;

Construct MBBR Process	\$1,500,000 ± 50%
▪ Golf course screen unit. ▪ 2 x 500 m³ concrete tank (2 x 5m deep x 10m square) ▪ Coarse bubble aeration system ▪ 50% full of media (can be up to 70% full) ▪ Media strainer	

It should be noted that not all of the MBBR media would be needed at first. As such, media could be added as the population expands.

The cost of decommissioning Cell 1 has not been directly considered.

5.0 Reclaimed Water Improvement Options

5.1 Reclaimed Water Quality

Additional criteria may be enforced for reclaimed water quality in the future. In particular, the following additional criteria are specified under the Municipal Wastewater Regulation for reclaimed water with 'greater exposure potential'.

Carbonaceous BOD₅ Max 10 mg/L

Total Suspended Solids Max 10 mg/L

Turbidity Average < 2 NTU, Max < 5 NTU

At present, there are no limits set for these parameters for reclaimed water at the Osoyoos WWTP. The lagoon effluent target is significantly less stringent at CBOD 45 mg/L and TSS 60 mg/L. Of course, it is worth noting that significant remediation occurs in the storage reservoirs. Samples for BOD₅ (not CBOD) and TSS were taken in 2009 and 2020. They indicated relatively good water quality. While this is a small dataset, the results are fairly consistent with the equivalent samples from the Oliver storage reservoir.

TABLE 5-1: RECLAIMED WATER SAMPLING

Date	Biochemical Oxygen Demand (mg/L)	Total Suspended Solids (mg/L)
2009-05-19	<10	7
2009-06-23	37	2
2009-07-21	<10	5
2009-08-25	12	12
2009-10-20	<10	1
2020-04-04	9	13
2020-07-14	19	<2

Assuming that the treatment lagoons are working effectively, it is expected that the reclaimed water compliance will depend greatly on the concentration of algae suspended in the stored water. Clearly algal growth is challenging to control when the water is enriched with nutrients. It is also true that the algae contribute dissolved oxygen and take up nutrients which contributes to the continued treatment of the stored water. In order to keep the reclaimed water suspended solids concentration low, it is assumed that the Town would aim for low algal counts. Many methods are available to control algae with a view to maintaining consistent reclaimed water quality. At this point algae has not been raised as a notable concern in the Osoyoos reclaimed water storage.

Filtration would be required in the event that enhanced TSS and turbidity removal is needed. Options include granular media filters, disk filters and drum filters. An indicative cost for Dynasand® filtration for a capacity of 8,000m³/d operating 24 hours per day is \$5,500,000. The concept would include a fabric partition in one of the reservoirs to provide filtered water storage. Pilot testing of the proposed filtration technology is recommended as a clarifier may be needed as pretreatment.

5.2 Future Reclaimed Water Storage and Use

5.2.1 Future Reclaimed Water Storage Requirement

Total inflow to the plant currently averages 850,000 m³/annum. Present trends (Figure 1-3) indicate that inflow may continue to increase. The Long Term Sewer System Expansion Plan Discussion Paper (2014) used population projections to estimate a 2053 annual wastewater volume of 1,200,000m³.

The Long Term Sewer System Expansion Plan Discussion Paper (2014) found that by 2053 around 145,000m³ of storage will be needed. This plan identified the Landfill and gravel pit as feasible locations for a third 150,000m³ storage reservoir. It would have an area of 3.0 – 3.5 ha depending on water depth. Two alternative sites are indicated on the drawing provided as Appendix A.

A less favored option would be to convert the wastewater treatment process to more intensive treatment and convert existing treatment Cell #3 to storage. This would only add 58,000m³ if the cell is not made deeper. This is not recommended at this stage.

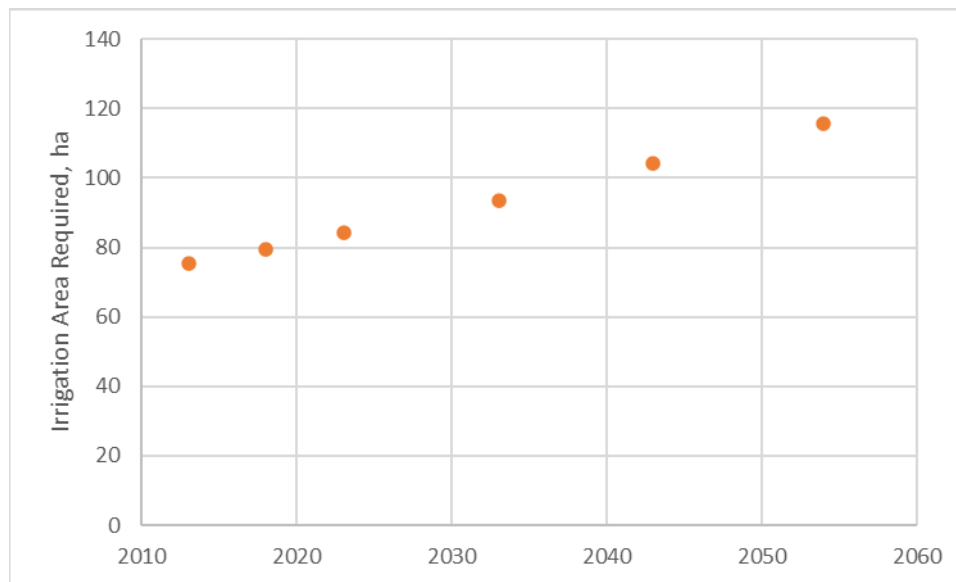
Meanwhile, the maximum use of existing storage can be achieved by;

1. Ensuring the storage reservoirs are at 100% of capacity at the start of the irrigation season.
2. Diverting flow to the rapid infiltration basins if storage is projected to be full by the start of the irrigation season.
3. Make arrangements with existing reclaimed water irrigators to increase or reduce water use based upon availability.

5.2.2 Future Irrigation Areas

The Long Term Sewer System Expansion Plan Discussion Paper (2014) estimated the total required irrigation area that will be required in 2053 as 116ha. The available area is currently 75ha, meaning an additional 41ha would be required by 2053.

FIGURE 5-1: CALCULATED IRRIGATION AREA REQUIREMENT



A number of potential future irrigation sites were identified, as follows, totaling 31ha.

Location	Area (ha)
1. Parcel South of Secondary School (Rem 1003 Plan 31183)	3.5
2. South End of Desert Park	1.5
3. East Side of Desert Park Site	4.8
4. Crown Land west of Desert Park Site	3.0
5. Airport Site	1.5
6. LAR 75 East of Landfill	16.6
Expansion Opportunities Total	30.9

A drawing showing existing and future irrigation locations is provided in Appendix A. If irrigation of the area east of the landfill is unacceptable to the OIB then 14.4ha remains available on land within the town boundary, which would be sufficient up to about 2040.

Other potential opportunities for future use reclaimed water are:

- Industrial uses, i.e. concrete manufacture, cooling, etc.
- Expanded irrigation on the Desert 18 of the golf course.
- Irrigation of vineyards north of the airport site.
- Irrigation of vineyards or other grassland type cover on completed areas of the landfill site.
- Irrigation to orchard crops.

Reclaimed water contains higher levels of metals than many fresh water supplies. Irrigation to food crops may require further treatment in order to comply with relevant criteria, such as the British Columbia Approved Water Quality Guidelines: Aquatic Life, Wildlife & Agriculture (2017). This additional treatment could be limited to the water that is used by the irrigators. The irrigators may also need to make changes to their irrigation systems in order to accept the water. The BC Ministry of Environment Reclaimed Water Guideline (2013) gives the following requirements:

- Drip and trickle irrigation systems must be used for sweet corn, berry and fruit crops.
- Vegetable crops must be irrigated with a subsurface drip irrigation system.
- Root crops may not be irrigated with reclaimed water if they are likely to be eaten raw.
- Commercially processed vegetables should be thoroughly washed and cooked following industry accepted procedures for reclaimed water applications

If the water was offered at a minimum charge, it may still be appealing to an orchard irrigator. Vineyards have a relatively low water requirement and reclaimed water use may not be cost effective.

An aquifer recharge and recovery system may be more appealing to agricultural users. Reclaimed water would be discharged to the infiltration system and then wells would pump the infiltrated water to the agricultural users. With appropriate design, filtration through the soil could eliminate any need for treatment. Managed aquifer recharge and recovery is relatively common in Europe for the production of drinking water.

6.0 Chlorination Improvement Options

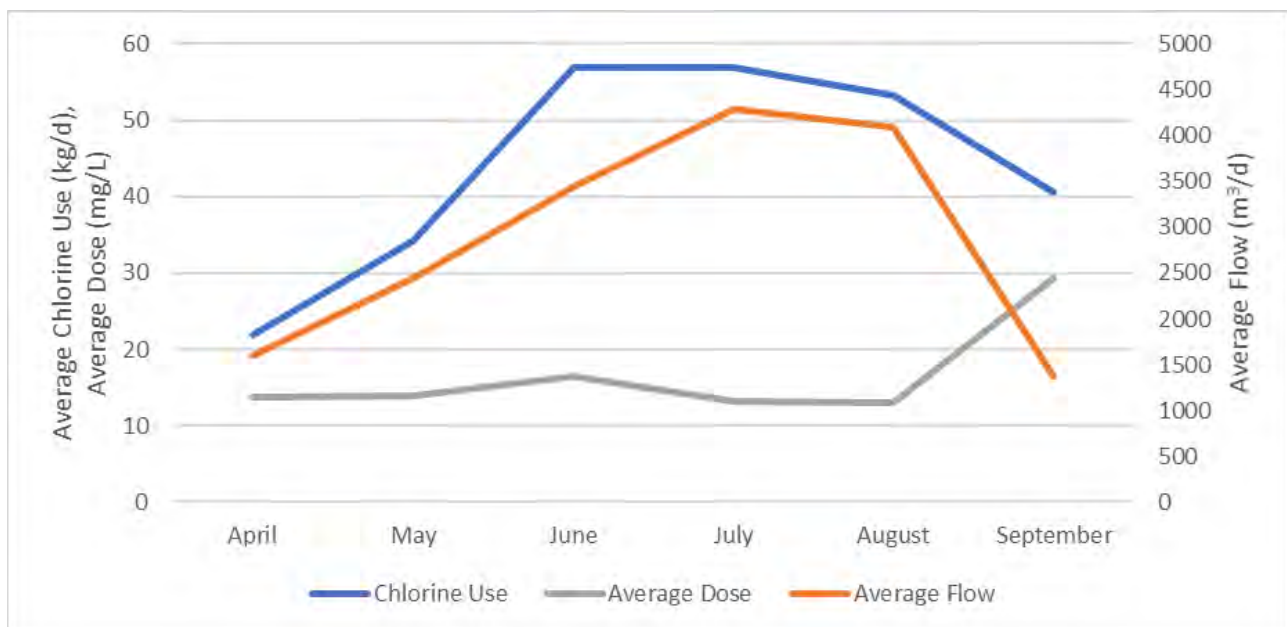
Chlorine gas is currently used for disinfection. This gas is highly toxic. As a result, the storage and handling of chlorine gas is considered to be hazardous and is subject to very stringent regulatory requirements compared to the alternatives. While relatively inexpensive, gaseous chlorine has gone out of favour in North America. The Town is interested in switching to an alternative disinfectant capable of replacing the chlorine gas.

When converting systems from chlorine gas, one of two chemicals are typically selected. These are sodium hypochlorite and calcium hypochlorite. Other chemicals, such as ozone, are effective at destroying pathogens but do not maintain a residual. As a result, MOE requires a form of chlorine to be used.

The alternatives are described in the following sections.

6.1 Current Chlorine Consumption Rates

The Operational Certificate requires a chlorine residual of 1.0 mg/L immediately prior to irrigating⁵. The chlorine consumption averaged nearly 60 kg/d in 2019 and the chlorine dose rate averaged about 15mg/L.



The impact the chlorine dose has on free chlorine residual is strongly affected by the chlorine demand of the reclaimed water. This is discussed further in Section 6.6. For the purpose of assessing alternatives, a dose of 30 mg/L has been used for estimating approximate storage

⁵ The BC Reclaimed Water Guideline specifies a disinfection residual of 0.5mg/L at the point of use.

requirements. This is based on an assumption that there may be periods with elevated demand, but they will generally be managed to keep consumption at no more than double the average value.

6.2 Sodium Hypochlorite (12%)

Sodium hypochlorite (NaOCl) is the active ingredient in household bleach. The chemical is less hazardous than chlorine gas and there are fewer training requirements and regulations for storage and handling. Sodium hypochlorite (12%) is corrosive, and particularly hazardous if splashed into the eyes. A safety shower and eyewash are required.

Sodium hypochlorite can be delivered as 1230L totes or by bulk tanker. It is normally provided in as 12% strength. There is a small cost saving for bulk tanker delivery, but there is additional cost in setting up tanks and containment.

6.2.1 System Design – Tote Delivery

A tote based system is typically operated along with a day tank fitted with level monitoring. A 5,000 litre day tank has been assumed for budgeting purposes. It is also possible to run the system directly from totes. However, there is a greater risk of leakage, and the monitoring of the connected totes is more problematic.

The operator would routinely transfer the contents of the totes to the day tank. With this arrangement, the operator has the reliable remote level monitoring and can top up levels to ensure that the storage will be sufficient until the next operator visit.

The day tank would have a containment system to protect against the escape of leakage. Totes can also be located on spill containment units. The cost estimate does not allow for individual containment pallets. It is assumed that a forklift would be used for moving the totes. The totes can also be moved around the flat floor using a pallet jack.

It is assumed that a small number of totes could be located inside the chlorine drum room with others under a porch-style structure.

Operational hours have been assumed to be an average of two visits per week with some additional hours for duties such as chemical ordering.

6.2.2 System Design – Bulk Delivery

A bulk delivery and storage system would typically comprise two or more tanks sized to take a full tanker delivery. There needs to be sufficient additional storage to allow for a delay between ordering and delivery. A total stored volume of 15m³ has been selected for this study based on a reasonable truck load. Under what we believe to be peak use conditions (future irrigated flow of 7,000 m³/d and an elevated chlorine dose of 30 mg/L) the 15m³ of storage would last for 8 days. However, under average conditions recorded over the 2019 irrigation season (3,500 m³/d and 20 mg/L) the tank contents would last for 25 days. It has been assumed that the Town could call for

restraint from irrigators in the event that a delivery was delayed and water demand was exceptionally high. In an emergency, one of the bulk tanks could also be partly filled with a mix of calcium hypochlorite pucks and water.

A delivery quantity of 10m³ has been assumed. Bulk delivery camlock connections could be located inside or outside the building. The existing building would need to be extended if it is to house the chlorine tanks. Alternatively, because the system can be drained in winter, the tanks could be installed under a porch structure with the dosing systems inside the existing building. An outdoor tank would increase chemical degradation if it was uninsulated (see section 6.2.3).

Operational hours have been assumed to be an average of one visit per week with some additional hours for duties such as chemical ordering.

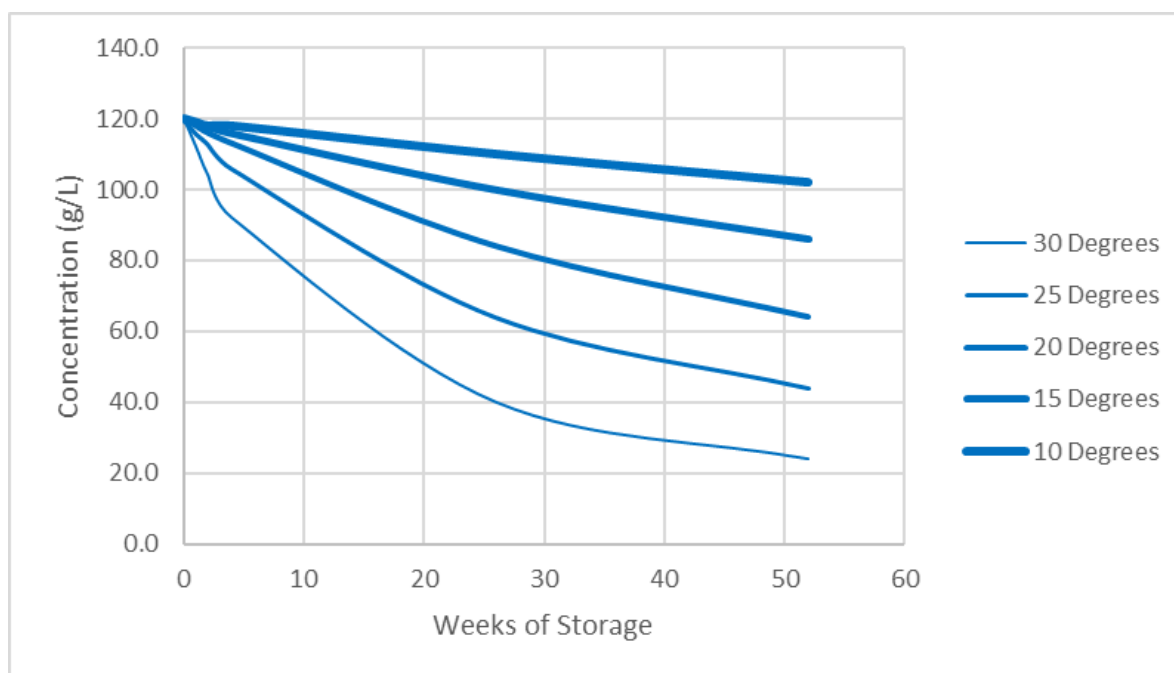
The City of Kamloops operates a similar sized reclaimed water disinfection system. They replaced their drum chlorine system with a bulk sodium hypochlorite system in 2017.

6.2.3 Breakdown of Sodium Hypochlorite

The strength of sodium hypochlorite degrades relatively quickly during storage in warm temperatures. As a result, during the peak summer months the quantity stored should be kept to a minimum and the oldest product should be used first. Air conditioning of the storage space may be cost effective.

Other important contributors to degradation rate include UV light exposure and whether the chemical was filtered at the factory. These are not expected to be issues for this application.

FIGURE 6-1: STABILITY OF 12% SODIUM HYPOCHLORITE SOLUTION VS TEMPERATURE



6.3 Sodium Hypochlorite (0.8%)

Sodium hypochlorite can be generated on-site. The raw materials for the generation of 0.8% sodium hypochlorite are salt, electricity and water. These raw materials are relatively inexpensive. The main cost with on-site systems is the installation cost. Installation costs have been presented as an annual cost so that the alternatives can be fairly compared.

The 0.8% strength solution is corrosive to a similar degree as household bleach. It is not classed as a hazardous chemical for transportation purposes. Eyewash facilities are necessary. The main hazard with bulk sodium hypochlorite systems is the release of explosive hydrogen gas. This must be vented for safety.

Degradation of the chemical, as discussed for bulk sodium hypochlorite, is not a significant issue for sodium hypochlorite generated onsite because the chemical is stored for a short period and the stability of the solution increases markedly at lower concentrations. In addition, air locks are much less of an issue due to the lower concentration and typically high fluid velocities in dosing lines.

The raw material (salt) can be stored indefinitely if kept dry. This minimizes the time required in accepting deliveries.

If the ton cylinder room were to be left unheated over winter, the system would be more challenging to winterize than the alternative options. The other systems could be winterized by pumping a glycol solution through the system. This would be damaging to the electrolyser cells. No winterization is required for brine.

The main disadvantages of sodium hypochlorite generation systems include high capital cost and the complexity of the facility, although they tend to operate reliably.

6.3.1 [System Design](#)

The chlorine ton cylinder room can be converted to house a sodium hypochlorite generation system. Based on the initial system being sized for current use with the ability to catch up after maintenance, the proposed system would be the Chlortec CT450, or an equivalent unit from another manufacturer. The CT-450 has a capacity equivalent to 204 kg Cl_2/d . We have assumed a single duty unit. Sodium hypochlorite from totes could be dosed in the event of an electrolyser failure. Dilution may be needed depending on dosing pump turndown capability.

Salt handling is a significant cost if the brine system is to be designed for bulk deliveries. A cost for a Bryneer large enough to accept blown in truck deliveries would be roughly \$150,000. It is anticipated that the Bryneer would be located outside (as is done at the Mission Hill WTP).

Because the system would only be used in the warmer months, an outdoor chlorine storage tank could also be used. The tank would need to be emptied in November.

6.4 Calcium Hypochlorite

Calcium hypochlorite is commonly used as a household and municipal pool chemical. It is the safest option in terms of material handling, particularly in the tablet / puck or granule format. Basic protective equipment is required for handling. Once it is dissolved, the hazards are equivalent to bulk sodium hypochlorite.

This chemical can be dosed automatically using a system that dissolves pucks or granules into a tank. The liquid chemical is dosed in the conventional way with a dosing pump. As the strength of the solution can vary, the system depends on feedback from a chlorine analyser for dosing accuracy.

FIGURE 6-2: CALCIUM HYPOCHLORITE MAKE-UP AND DOSING SYSTEM



FIGURE 6-3: PAILS OF CALCIUM HYPOCHLORITE TABLETS AS SUPPLIED BY BI PURE WATER.



Because calcium hypochlorite is very stable in the solid form, it can easily be stored over the winter period.

Calcium hypochlorite is 65% chlorine, so the material storage and handling quantities will be much lower than for sodium hypochlorite. Nevertheless, there will be a need to manage up to 350 kg/d of calcium hypochlorite pucks or granules by 2040.

Calcium hypochlorite is widely used at swimming pools due to its relative safety. There is a risk of fire or explosion if calcium hypochlorite is mixed with oil, although the pucks are less reactive than powder. Chlorine gas is released when hypochlorites are mixed with acids. There is no reason to expect either situation to affect this installation.

The capital cost for a calcium hypochlorite system is relatively low. The main disadvantage of calcium hypochlorite is the high cost of the chemical. There is also less accuracy in the dose achieved, but this is an issue that could be resolved. Calcium hypochlorite systems are used successfully at most swimming pools and many small water supplies.

6.4.1 System Design

An Accu-tab chlorination system has been assumed. To meet the peak flows and to include some redundancy, three Accu-tab 361000 hoppers would be used with a 500 gallon solution tank. The supplier (BI Pure Water) has assumed that the chlorinated solution would be fed to the chlorine contact basin with four pumps and the dosing flow controlled with metering valves. The proposed system has a very small footprint at 2.5m (W) x 3.0m (L) x 1.9m (H). Given that the calcium hypochlorite is 65% chlorine, there is space in the existing building for dosing and limited storage.

6.5 Comparison of Chlorine Gas Alternatives

The key advantages and disadvantages of the alternative chemicals considered are described in Table 4.1.

TABLE 6-1: ALTERNATIVE CHEMICALS – ADVANTAGES AND DISADVANTAGES

Disinfectant	Advantages	Disadvantages
Chlorine gas	- Existing system - Reliable operation - Low chemical cost - Indefinite shelf life - Town operating three sites with ton cylinders.	- Hazardous material. - Comprehensive operator training needed - Additional regulatory requirements (WSB)
Sodium hypochlorite (12%)	- Same efficacy and residual protection as chlorine gas. - Fewer training requirements than chlorine gas. - Fewer regulations than chlorine gas.	- Limited shelf-life - Chemical requires careful handling - Crystallization can clog injection ports. - Air locks from oxygen release can clog pipes, pumps, and valves (auto purge systems available). - Chlorine gas release hazard if mixed with acid (no reason to expect this to happen).
Sodium hypochlorite (0.8%)	- Same efficacy and residual protection as chlorine gas. - Fewer training requirements than chlorine gas. - Not classed as a 'hazardous' chemical. - Long shelf life for raw materials and product - Less by-product formation than bulk hypochlorite. - Less corrosive than bulk hypochlorite.	- High capital cost - Relatively complex equipment
Calcium hypochlorite	- Low capital cost - Much more stable than sodium hypochlorite allowing long-term storage. - Fewer safety regulations. - Relatively safe to handle. - Relatively little operator expertise required.	- High operating cost. - Impractical lifting and handling requirements. - Fire, explosion and chlorine gas release hazard if mixed with incompatible materials (no reason to expect this to happen). - Calcium carbonate buildup in the makeup system needs regular cleaning.

TABLE 6-2: COMPARATIVE LIFE CYCLE COST OF OPTIONS

Disinfectant	Capital Cost	Chemical / Maintenance Cost	20 Year Annualized Cost
Chlorine gas	\$0	\$17,000	\$17,000
Sodium hypochlorite (12%) - totes	\$220,000	\$41,000	\$58,000
Sodium hypochlorite (12%) - bulk	\$380,000	\$33,000	\$63,000
Sodium hypochlorite generation (0.8%)	\$1,120,000	\$39,000	\$129,000
Calcium hypochlorite	\$260,000	\$66,000	\$87,000

Comparative costs for the options have been compared on a 20 year annualized basis with a 5% rate of return. Maintenance cost has been typically set at 1.5% of capital cost. A value of 5% has been assumed for trucking 0.8% sodium hypochlorite. The chemical cost listed for sodium hypochlorite generation includes both salt and electricity.

It is suggested that the existing system is the most cost effective. There are safety systems available to quench chlorine gas if it were to escape. These could be considered as an alternative to changing the chemical dosed.

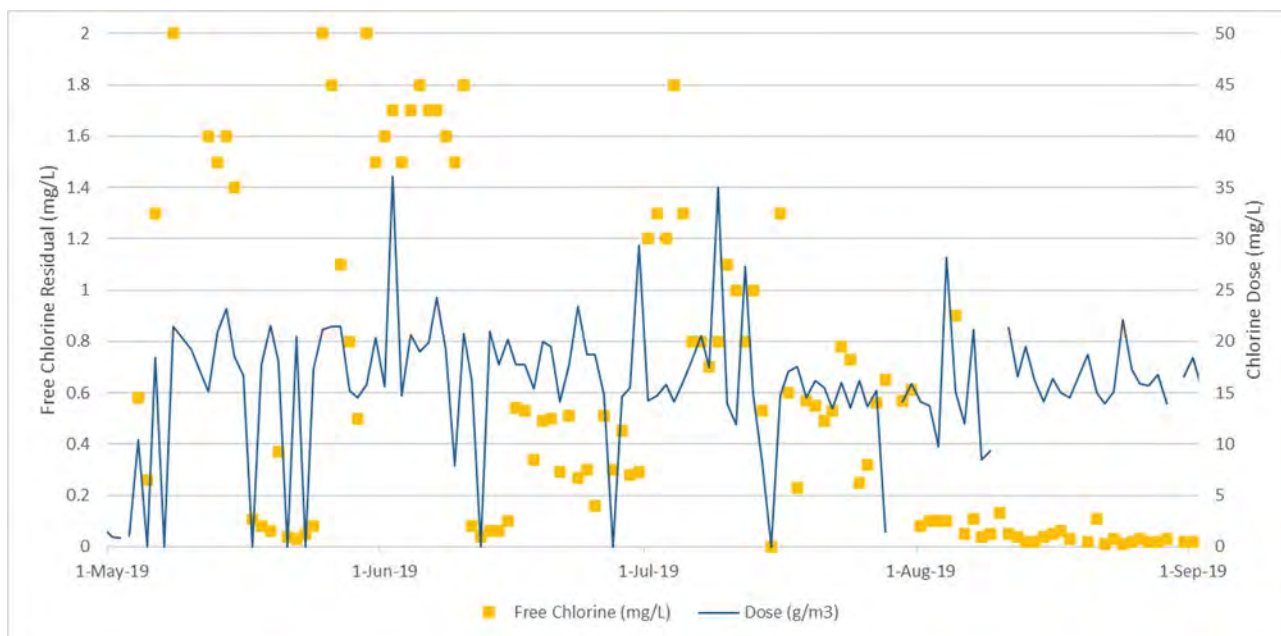
If the chemical is to be changed then sodium hypochlorite bulk deliveries are recommended based on cost and practicality.

6.6 Minimization of Chlorine Consumption

Chlorine compounds are highly reactive, which makes them effective in disinfection. Chlorine reacts with many natural compounds in water which together make up the chlorine demand of the water. These compounds include organics, iron, manganese, sulphide and ammonia. Ammonia is especially potent with the ratio of ammonia nitrogen to chlorine being around 10:1. Given that the effluent ammonia concentration varies from 0 to 35mg/L, the degradation of ammonia within the treatment system (nitrification) has a dramatic impact on chlorine use.

Figure 6-4 shows that the reclaimed water free chlorine levels do not closely relate to the dose rate and are likely to relate more to the concentration of ammonia in the water. This will vary seasonally and Reservoir 1 is likely to be different to Reservoir 2.

FIGURE 6-4: FREE CHLORINE RESIDUAL VS DOSE RATE



It is recommended that the reservoir filling and emptying schedules be planned to minimise the use of high ammonia source water. Treatment lagoon effluent will be relatively low in ammonia from mid-late summer to fall. Similarly, the ammonia concentration of the stored water should fall over that period.

Year-round lagoon-based ammonia removal is expensive to achieve because the metabolism and growth of nitrifiers reduces dramatically at low temperatures. However, relatively inexpensive lagoon retrofits are marketed for summer nitrification, consisting of high surface area nitrifier support media suspended in oxygen rich locations in the final treatment cell (e.g. EDI BioReef BioCurtain). These may serve to improve ammonia removal and extend the period over which ammonia is removed, without significant expense. This upgrade may pay for itself in reduced chlorine demand.

It may also be feasible to install nitrification modules (e.g. Bishop Water BioCord™ Reactors) in the reservoirs to reduce ammonia levels over the period the water is in storage. It would be important to ensure that having this form of treatment occurring in the storage reservoirs is consistent with the Permit conditions.

FIGURE 6-5: BISHOP WATER BIOCORD UNIT



Any system of this type will rely on the efficient removal of rag by the later stages of treatment to avoid having the rag tangled up with the media.

6.7 UV Disinfection

At some point in the future, the regulations for use of reclaimed water may become more stringent to take better account of the inactivation of protozoa. Protozoa are inactivated by chlorine so long as there is sufficient dose and contact time. However, contact times required are fairly long and it may prove more economical to use a technique just as ultraviolet disinfection to avoid large scale chlorinated water storage.

UV disinfection could also be considered for primary disinfection with the dosing of chlorine as a secondary disinfectant. UV disinfection could not replace chlorination, as a chlorine residual is required by the regulations and the permit.

A UV disinfection system for Osoyoos is most likely to be an open channel, low pressure lamp system located adjacent to the chlorine contact basin. Construction cost would be in the order of \$500,000 - \$800,000.

7.0 Recommendations

7.1 Treatment Improvements

The recommended design for the future upgrade of the Osoyoos WWTP is the MBBR process in the pre-treatment or intermediate treatment configuration. This process addition has the following technical advantages;

- Small footprint which leaves space for future upgrades.
- Ease of expansion
- Similar operation to existing plant with minimal operator intervention required
- Sludge can continue to be collected and digested in the partial mix lagoons
- Construction can be completed with little disruption to existing treatment.
- Easy to meet reliability requirements required by Municipal Wastewater Regulation.
- Easy to add more capacity by adding treatment cells.

It is proposed that the process would be split into multiple tanks to allow one tank to be taken offline during winter to reduce air and energy consumption. Multiple tanks also give flexibility to allow maintenance to be undertaken while continuing to treat incoming wastewater.

7.2 Reclaimed Water Improvements

Options for upgrades to reclaimed water quality will need to be evaluated based on the specific requirements that arise but would ideally be limited to chlorination.

Future Storage has not been proposed at this stage as

- Storage should be adequate to 2040, depending on usage patterns by irrigators.
- It would depend on the identification and procurement of suitable additional land
- It is more likely that the rapid infiltration basins would be used to dispose of excess water.

Future irrigation areas have been identified. A separate study is recommended to identify new areas and estimate costs.

7.3 Disinfection Improvements

All of the options analysed increase the cost of disinfection. However, continuing the use of chlorine increases the health and safety risk to staff and the public.

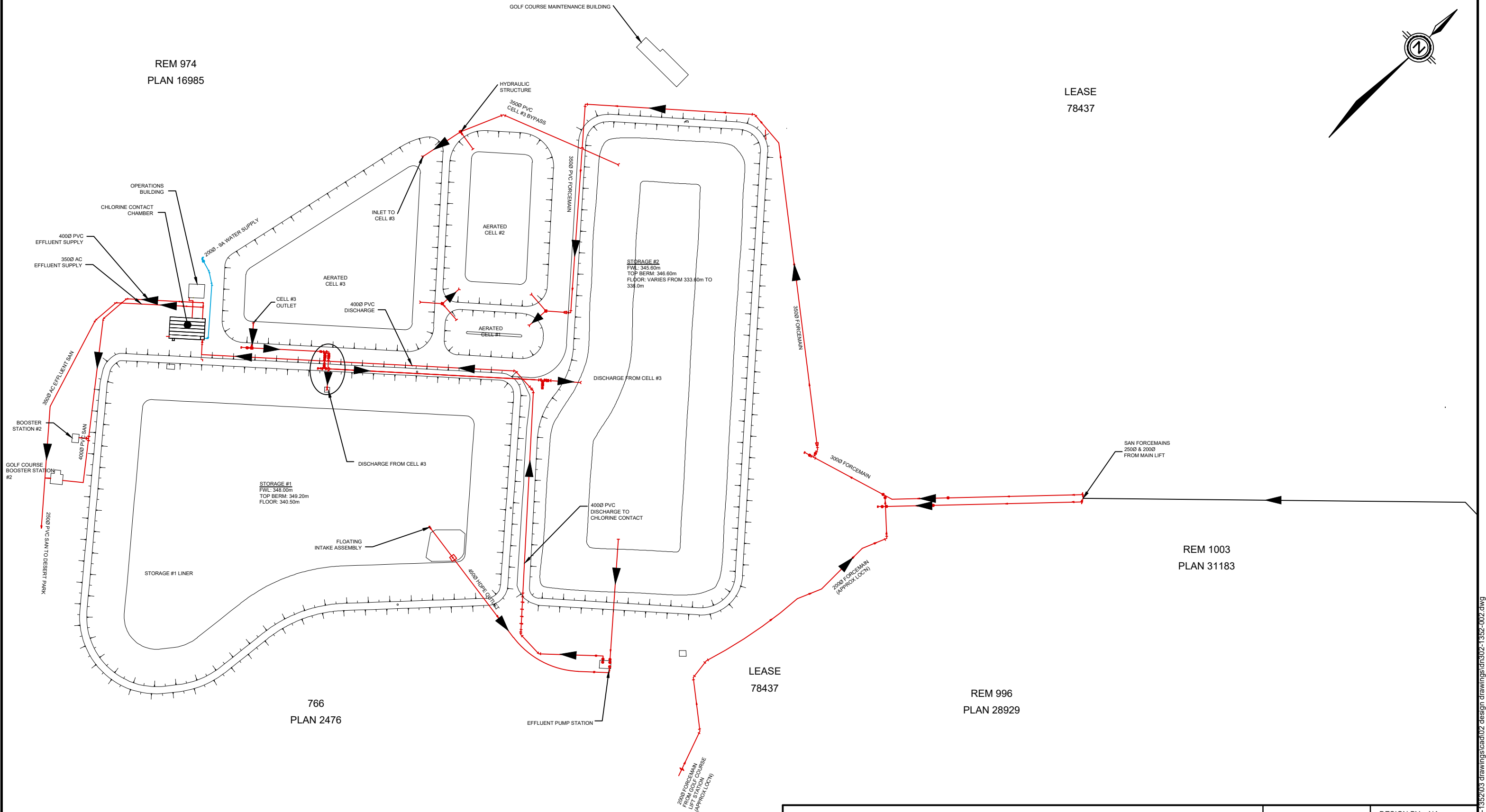
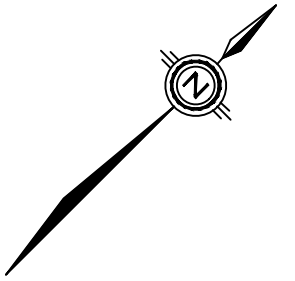
The most cost-effective replacement option would be to buy 12% sodium hypochlorite in totes or as a bulk tanker delivery. The proposed system is relatively easy to install and uses commonly available technology.

APPENDIX A

Drawings

REM 974
PLAN 16985

LEASE
78437



766
PLAN 2476

LEASE
78437

REM 996
PLAN 28929

REM 1003
PLAN 31183



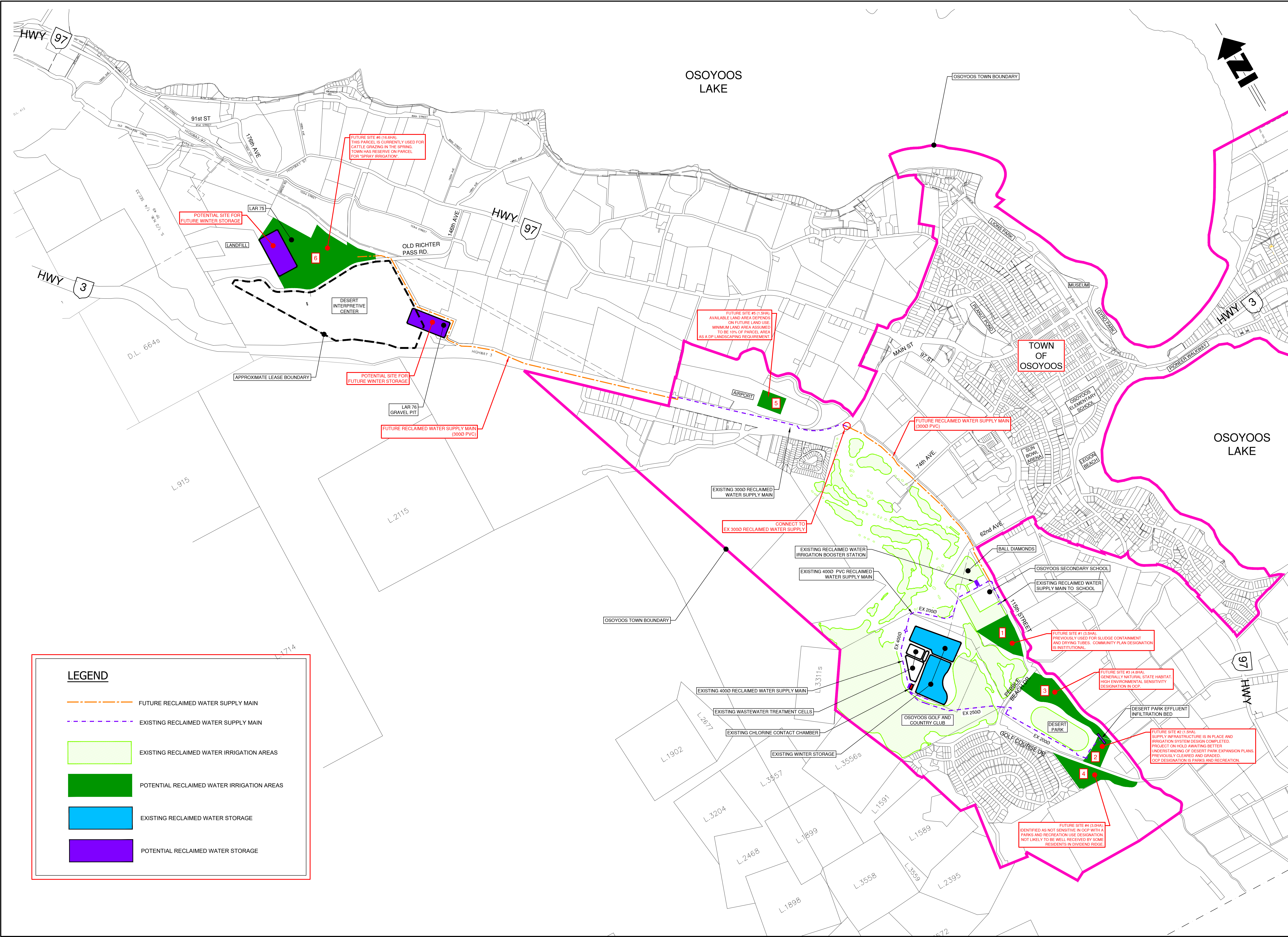
TREATMENT AND STORAGE SITE PLAN



DRAWN BY: DL
DATE: March 2018

DESIGN BY: NA	
SCALE: 1:2500 (11x17)	
DWG NO.: FIGURE	REV:

FILE: r:\clients\300-399\302\302-1352\03 drawings\cad\02 design drawings\dm302-1352-002.dwg



LEGEND

FUTURE RECLAIMED WATER SUPPLY MAIN

EXISTING RECLAIMED WATER SUPPLY MAIN

EXISTING RECLAIMED WATER IRRIGATION AREAS

POTENTIAL RECLAIMED WATER IRRIGATION AREAS

EXISTING RECLAIMED WATER STORAGE

POTENTIAL RECLAIMED WATER STORAGE

No.	DATE	DESCRIPTION	BY	APPD
ISSUES / REVISIONS				
CONSULTANT SEAL				

TRUE CONSULTING

201 - 2079 Falcon Road ■ Kamloops BC ■ V2C 4J2
tel 250.828.0881 ■ fax 250.828.0717
info@TRUE.bc.ca

TOWN OF OSOYOOS

RECLAIMED WATER SYSTEM

STORAGE AND IRRIGATION EXPANSION PLAN

SCALE	Scale 1:10,000
DESIGN BY	RW
DRAWN BY	DL
DATE	MAR 2021
PROJECT REFERENCE No.	302-1474
DRAWING No.	302-1474
SHEET OF	05
REVISION	

FILE: c:\pwworking\302-1354\05.dwg 3/24/2021 1:04:05 PM

APPENDIX B

Effluent Monitoring Data

Town of Osoyoos
WWTP Compliance Testing - Comparison with Operational Certificate Criteria

Location	Headworks					Outlet of Cell #3												Chlorine Contact Chamber												Sludge	Ground and Surface	
EMS site number	E230037					E105001												E207444												E230278		
Parameter	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Biochemical Oxygen Demand	Total suspended solids	Carbonaceous biochemical oxygen demand	Carbonaceous biochemical oxygen demand	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Total dissolved phosphorus as P, mg/L	Total nitrogen, mg/L	Ammonia nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Organic nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Faecal coliforms, MPN/100 ml	Total coliforms, MPN/100 ml	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus, mg/L	Total nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Chlorine residual, mg/L	Chloride, mg/L	Sodium, mg/L	Sulphate, mg/L	See 5.6.2 of Certificate	See 5.7 of Certificate
Frequency (Operational Certificate)	semi-annual	semi-annual	Not required	Not required	semi-annual	monthly	Not required	monthly	semi-annual	semi-annual	semi-annual	semi-annual	semi-annual	semi-annual	semi-annual	semi-annual	Use to calculate organic N	monthly	monthly	Not required	Not required	monthly	monthly	Not required	Not required	Not required	weekly	monthly	monthly	monthly	Annual	
Sample type	Grab	Grab			Grab	Grab		Grab	Grab	Grab	Grab	Grab	Grab	Grab	Grab	Grab		Grab	Grab			Grab	Grab				On-site	grab	grab	grab	Grab	Grab
Limit						<45		<60										<2.2 MPN														
Date																																
2009-01-20							42	16																								
2009-02-17							37	16																								
2009-03-17							13	13																								
2009-04-21							44	26	5.09																							
2009-05-19							64	33												<10	7											
2009-06-23							20	38												37	2											
2009-07-21							37	31												<10	5											
2009-08-25							<10	10												12	12											
2009-09-29	6.42								7													4.65										
2009-10-20							<10	6												<10	1											
2009-11-17							31	10																								
2009-12-22							20	18																								
2010-01-19							40	30	4.55	4.16	4.2	32	26.5	0.52	<0.01	4.9	31.4															
2010-02-16	6.22	4.37	180				19	20																								
2010-03-16							19	17																								
2010-04-20							14	12										< 3.0	< 3.0			5.34	20	1.31	<0.01	18.7						
2010-05-18							19	19										< 3.0	< 3.0			4.8	29.1	0.13	0.09	28.9						
2010-06-15							73	49										< 3.0	460			6.03	17	0.71	0.75	15.5						
2010-07-13									4.93	2.47	4.43	20.9	3.17	8.92	4.47	4.32	7.49															
2010-07-20																		< 3.0	7.3			4.69	13.8	0.55	0.48	12.7						
2010-08-17	5.68	3.27	190				15	35										2400	15000			5.53	24.7	0.21	<0.01	24.5						
2010-09-21							<10	35										< 3.0	91			7.14	22.6	10.1	0.59	11.9						
2010-11-16							13	12																								
2010-12-21							37	19																								
2011-01-18							20	16	3.5	3.26	3.09	29.8	21.8	1.85	0.13	6	27.8															
2011-02-15	5.81	3.82	220				66	105																								
2011-03-15							31	22																								
2011-04-19							26	17										< 3.0	< 3.0			4.86	26	0.82	0.07	25.1						
2011-05-17							12	10										< 3.0	< 3.0			2.77	24.1	0.47	0.12	23.6						
2011-05-25																																
2011-06-14							15	15						2.66	1.08			21	460			3.05	25.1	0.66	0.39	24.1						
2011-07-19							< 10	8										< 3.0	< 3.0													
2011-08-09																						5.7	21.4	2.63	0.57	18.2						
2011-08-16							< 10	3										43	>1100			3.27	18.1	11.1	0.71	6.37						
2011-08-24																																
2011-09-20							< 10	4										15	230			7.24	20.5	2.53	1.02	17						
2011-10-18							< 10	3																								
2011-10-25																		< 3.0	3.6			6.72	18.5	0.692	0.16	17.6						
2011-11-15							14	10																								
2011-12-20							22	17																								
2012-01-17							18	16	4.22	3.99	3.91	34.8	28.6	1.02	<0.010	5.1	33.7															
2012-02-14	3.85	3.74	140				17	13																								
2012-03-13							16	3																								
2012-04-17							22	15										< 3.0	< 3.0			5.7	19.5	2.95	<0.01	16.5						
2012-05-15							< 10	15										< 3.0	36			2.08	19.8	<0.01	0.034	19.8						

[illegible]

[illegible]

[illegible]

Location	Headworks					Outlet of Cell #3												Chlorine Contact Chamber												Sludge	Ground and Surface	
EMS site number	E230037					E105001												E207444												E230278		
Parameter	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Biochemical Oxygen Demand	Total suspended solids	Carbonaceous biochemical oxygen demand	Carbonaceous biochemical oxygen demand	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Total dissolved phosphorus as P, mg/L	Total nitrogen, mg/L	Ammonia nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Organic nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Faecal coliforms, MPN/100 ml	Total coliforms, MPN/100 ml	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus, mg/L	Total nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Chlorine residual, mg/L	Chloride, mg/L	Sodium, mg/L	Sulphate, mg/L	See 5.6.2 of Certificate	See 5.7 of Certificate
2017-09-12							54.2	30										75	>1100			9.63	24.1	<0.010	<0.010	24.1						
2017-09-19																		<3.0	<3.0							1.91						
2017-09-26																		<3.0	<3.0													
2017-10-03																		9.1	75													
2017-10-10							68.6	22										36	2400			6.49	22.7	9.06	2.31	11.3						
2017-10-17																		93	460													
2017-11-07							35.3	16																								
2018-02-06	5.29	3.23	205																													
2018-04-03																		<3.0	<3.0								0.1					
2018-04-10																		<3.0	<3.0													
2018-04-17																		<3.0	<3.0													
2018-04-24																		<3.0	<3.0													
2018-05-01																		<3.0	<3.0									0.3				
2018-05-08																		<3.0	<3.0			3.95	16.9	7.79	0.324	8.82						
2018-05-15																		750	24000													
2018-05-24																		23	460													
2018-05-29																		<3.0	<3.6													
2018-06-06																		<3.0	15			4.7	26.9	0.02	<0.010	26.9	0.3					
2018-06-12																		<3.0	4600													
2018-06-19																		<3.0	<3.0													
2018-06-26																		<3.0	<3.0													
2018-07-10																		<3.0	1100			4.33	30	0.183	0.39	29.4						
2018-07-17																		2400	11000													
2018-07-25																		430	24000													
2018-07-31																		750	24000													
2018-08-07																		<3.0	<3.0													
2018-08-14	9.14	3.85	301															<3.0	23			4.93	16.7	8.18	1.08	7.42						
2018-08-21																		<3.0	<3.0													
2018-08-28																		<3.0	<3.0													
2018-09-04																		<3.0	<3.0													
2018-09-11							42.8	37										<3.0	<3.0			7.1	26.5	14.8	1.42	10.3						
2018-09-18																		<3.0	<3.0													
2018-09-25																		<3.0	<3.0													
2018-10-02																		<3.0	<3.0													
2018-10-09							81.5	13.5										<3.0	<3.0			6.13	29.6	19.8	0.348	9.39						
2018-11-05							28.1	6.9																								
2018-12-11							60.3	15																								
2019-01-08							21.1	16.7	4.18	3.73	3.93	33.5	20.6	5.77	0.785	6.3	26.9															
2019-02-05	6.77	2.24	255				30	27.5																								
2019-03-05							21.6	28																								
2019-04-01																												0.14				
2019-04-08																												0.33				
2019-04-16							42.9	27																								
2019-04-19																													0.37			
2019-04-20																												0.34				
2019-04-23																		<3.0	<3.0			5.14	32	2.32	0.47	29.2						
2019-04-27																												0.54				
2019-04-30																		<3.0	<3.0			Full	Suite	2.24	0.489							
2019-05-04																												0.58				
2019-05-07							26.6	36.5										<3.0	<3.0			4.86	28.6	0.71	0.09	27.8						
2019-05-11																												2.1				
2019-05-14																		<3.0	<3.0													
2019-05-18																												0.08				
2019-05-21																		91	46000													
2019-05-25																												2				
2019-05-28																		3.6	2400											</		

Location	Headworks					Outlet of Cell #3												Chlorine Contact Chamber												Sludge	Ground and Surface
EMS site number	E230037					E105001												E207444												E230278	See 5.7 of Certificate
Parameter	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Biochemical Oxygen Demand	Total suspended solids	Carbonaceous biochemical oxygen demand	Carbonaceous biochemical oxygen demand	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Total dissolved phosphorus as P, mg/L	Total nitrogen, mg/L	Ammonia nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Organic nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Faecal coliforms, MPN/100 ml	Total coliforms, MPN/100 ml	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus, mg/L	Total nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Chlorine residual, mg/L	Chloride, mg/L	Sodium, mg/L	Sulphate, mg/L	See 5.6.2 of Certificate
2019-06-04							13.1	22										3.6	23												
2019-06-08																										1.6					
2019-06-11																		<3.0				4.02	29.8	26.1	0.156	3.54					
2019-06-15																										0.1					
2019-06-17																		<3.0	91												
2019-06-22																										0.51					
2019-06-25																		<3.0	91												
2019-06-29																										0.28					
2019-07-02									6.65	3.95	6.03	40.7	0.142	32	0.172	8.338	8.48	<3.0	<3.0			6.65	40.7								
2019-07-06																										0.8					
2019-07-09							26.2	47																							
2019-07-13																										1					
2019-07-16																		<3.0	<3.0			6.03	26.7	5.37	0.407	21					
2019-07-20																										0.55					
2019-07-23																		<3.0	<3.0												
2019-07-27																										0.56					
2019-07-30																		<3.0	<3.0												
2019-08-03																										0.1					
2019-08-06																		<3.0	93												
2019-08-10																										0.13					
2019-08-13							28	58										9.1	1100												
2019-08-17																										0.06					
2019-08-20																		<3.0	93			2.95	31.8	28.3	<0.010	3.46					
2019-08-24																										0.01					
2019-08-26	4.55	2.05	121																												
2019-08-27																		<3.0	23												
2019-08-31																										0.02					
2019-09-03																		<3.0	7.3												
2019-09-07																										0.1					
2019-09-10							13.8	30										23	150												
2019-09-14																										0.05					
2019-09-21																										0.08					
2019-09-24																		<3.0	<3.0							13.6					
2019-09-28																										0.06					
2019-09-30																		<3.0	<3.0												
2019-10-05																										0.09					
2019-10-08							10.6	33										<3.0	<3.0			6.21	24.6	1.14	0.241	23.3					
2019-10-16																		6.3	112												
2019-10-20																															
2019-11-05							22.5	31.3																							
2019-12-10							45.7	26.8																							
2020-01-07						missing	25.5	29																							
2020-02-18						missing	69.1																								
2020-02-19						missing	34.2	21.3																							
2020-03-03	6.28	3.59	288																												
2020-03-10						missing	73.3																								
2020-03-17									4.94	3.79	4.29	40.2	28	<0.010	0.014	12.1	40.2														
2020-03-24																		<1	<1												
2020-03-31																		<1	<1												
2020-04-07						missing	19.5	22										<1	1												
2020-04-14						missing	8.6	12.6										<1	<1	8.6	12.6	6.46	22.3	18.9	<0.010	3.49					
2020-04-21																		<1	<1												
2020-04-28			226	164														<1	<1												
2020-05-05						missing	26.8	33.7										<1	<1												
2020-05-11																															
2020-05-12																		<1	<1												
2020-05-19																		<1	<1			5.65	33.3	0.806	0.3	32.2					

Wastewater Treatment Long Term Plan

Town of Osoyoos



ENGINEERING ■ PLANNING ■ URBAN DESIGN ■ LAND SURVEYING

April 2021

Project No. 302-1354

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Report Submission

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2021-04-26

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May 12 2021

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Project Engineer

R:\Clients\300-399\302\302-1354\05 Reports\302-1354-Osoyoos -WWTP Long Term Plan-April 2021.docx

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1.0 Existing Treatment

1.1 Wastewater Treatment Design

The existing wastewater treatment process consists of a small complete mix aerated lagoon followed by two partial mix aerated lagoons. A process flow diagram is provided (Appendix A).

95% of the sewage inflow received at the plant comes from the Main Lift Station. 1.5mm screens at this lift station remove solids such as wipes and other debris from the flow. Flow is also received from the Golf Course lift station. The solids in this flow are shredded by chopper pumps, but are not removed.

Cell 1 is classified as a 'complete mix' aerated lagoon (whereby solids are not intended to settle in the lagoon). Aeration is very intense in order to achieve this effect. Cells 2 and 3 are 'partial mix' aerated lagoons. Enough air is added to maintain appropriate dissolved oxygen levels in these reactors, but not so much as to agitate the contents. Sludge settles to the bottom of the partial mix reactors for further degradation and storage. The volume of the complete mix lagoon (Cell 1) is approximately 3,300m³ (calculated from drawings). Cells 2 and 3 are significantly larger, at 27,000m³ and 58,000m³.

The treated wastewater is stored in large basins for reuse in the summer months. While a total of 565,000m³ can be stored, which is equivalent to 10 months of 'winter' flow, 60 days of capacity must be held in reserve. This is equivalent to around 100,000m³. The storage of the reclaimed water also further improves its quality. When water is needed for irrigation it is pumped to a chlorine contact basin for disinfection and distribution to various customers;

- Osoyoos Golf and Country Club (65.6 ha)
- Osoyoos Ball Diamonds (2.4 ha)
- Desert Park Infield Area (5.0 ha)
- Secondary School Playing Fields (2.8 ha)
- Tree Farm Standpipe and Infiltration

There is also a subsurface disposal field for disposal of excess reclaimed water, but this is not routinely used.

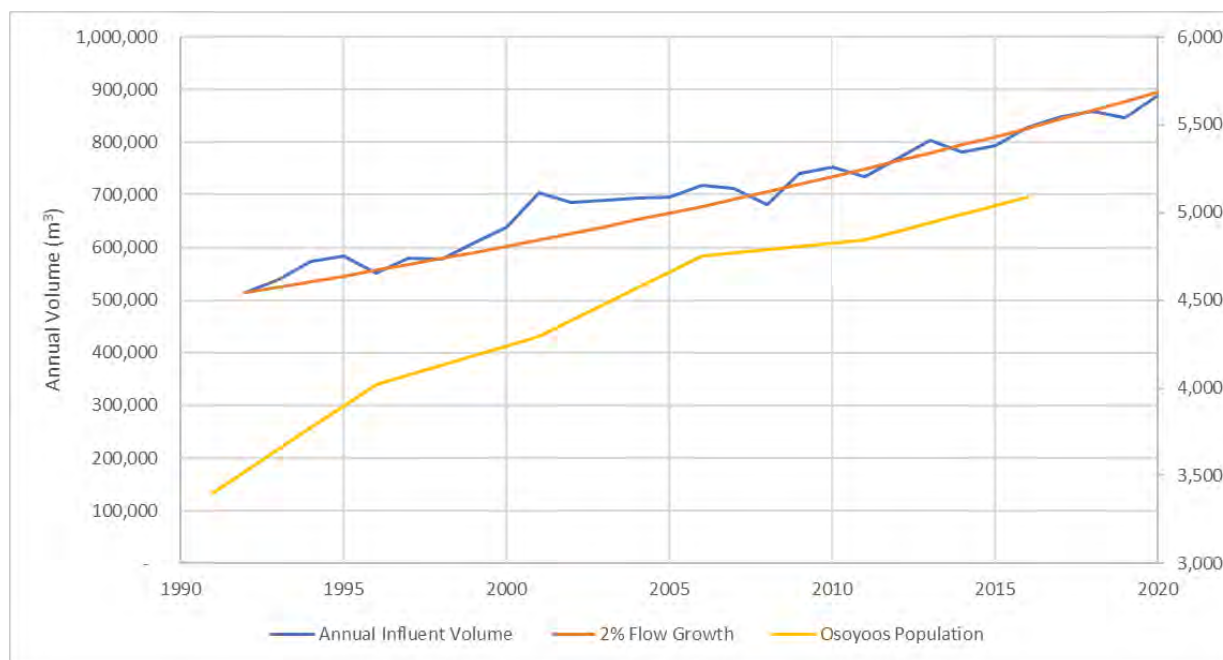
FIGURE 1-1: OSOYOOS AERATED LAGOONS AND RECLAIMED WATER STORAGE



1.2 Flows and Loads

The flows at the Main Lift Station and the Golf Course Lift Station are combined to give the total plant inflow. There has been a consistent growth in the plant flows since 1992. Annual wastewater volumes have increased at an average rate of 2% in the period since 1992.

FIGURE 1-2: ANNUAL WASTEWATER VOLUME VS POPULATION WITHIN TOWN BOUNDARY



The data has been further broken down into monthly flows in order to show how seasonal activity is affecting flow (See Figure 1-3).

The flow that occurs in the winter period reflects base flow from the resident population. The growth rate in winter flows from the resident population is estimated at 1.8% per annum. The Census population growth from 1996 to 2016 averaged 1.3%. The growth in flows was also affected by bringing in new sewer areas, such as the North West Sector. The annual maximum monthly flow includes flows related to tourist activity. The total flow appears to be increasing at the same rate as the minimum flow. The difference between winter and peak summer appears to be relatively stable at around 1,500m³/d. This has been the case since 2001 and seems to indicate stable tourist activity.

The monthly average flow does not capture peaks related to holiday weekends and rainfall. The daily flows for 2019 are shown on Figure 1-5. It can be seen that the annual peak flow coincides with the Canada Day and BC Day long weekends. Winter flows occur from November to April.

Flow projections were made by Kerr Wood Leidal in 2014 in their report 'Capacity Assessment Sanitary Sewer Infrastructure'. They projected the population and flow both within and outside the town boundary. Their expectation was that the resident population within the town boundary would grow by 17% over 22 years from 2011 to 2033, which is an average growth rate of 0.8%. Industrial / commercial / institutional and seasonal growth would be around 0.4% over the same period. KWL predicted significant population growth outside the town boundary. How much of the additional flow is permitted with the sewer system would be at the discretion of the Town.

FIGURE 1-3: PLANT INFLOW (AVERAGE BY MONTH), 1995 – 2019

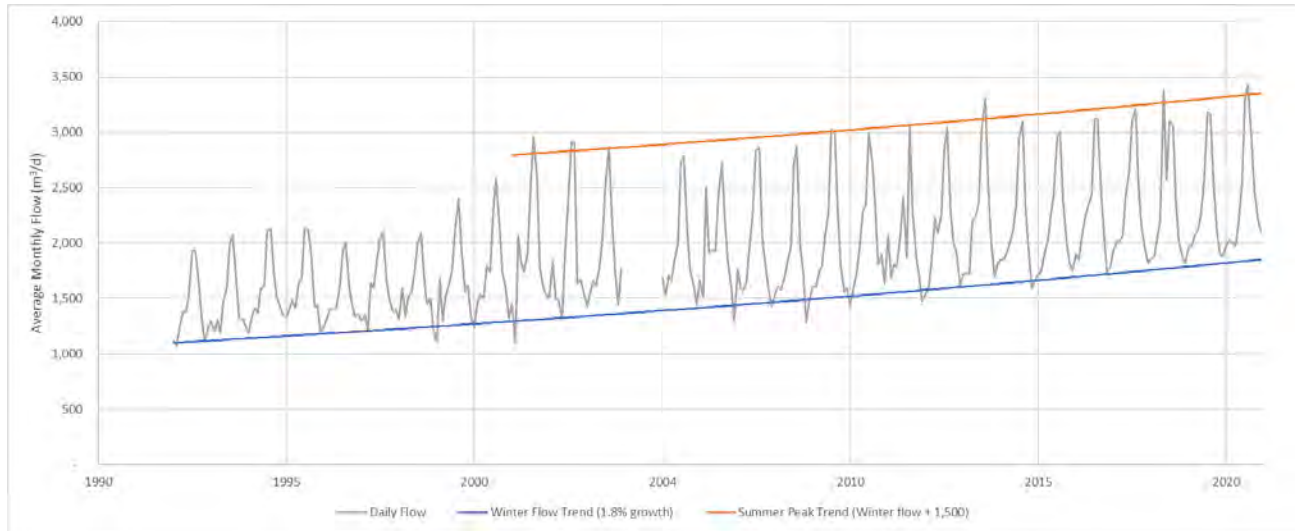


FIGURE 1-4: PLANT INFLOW (AVERAGE BY MONTH), 2012 - 2019

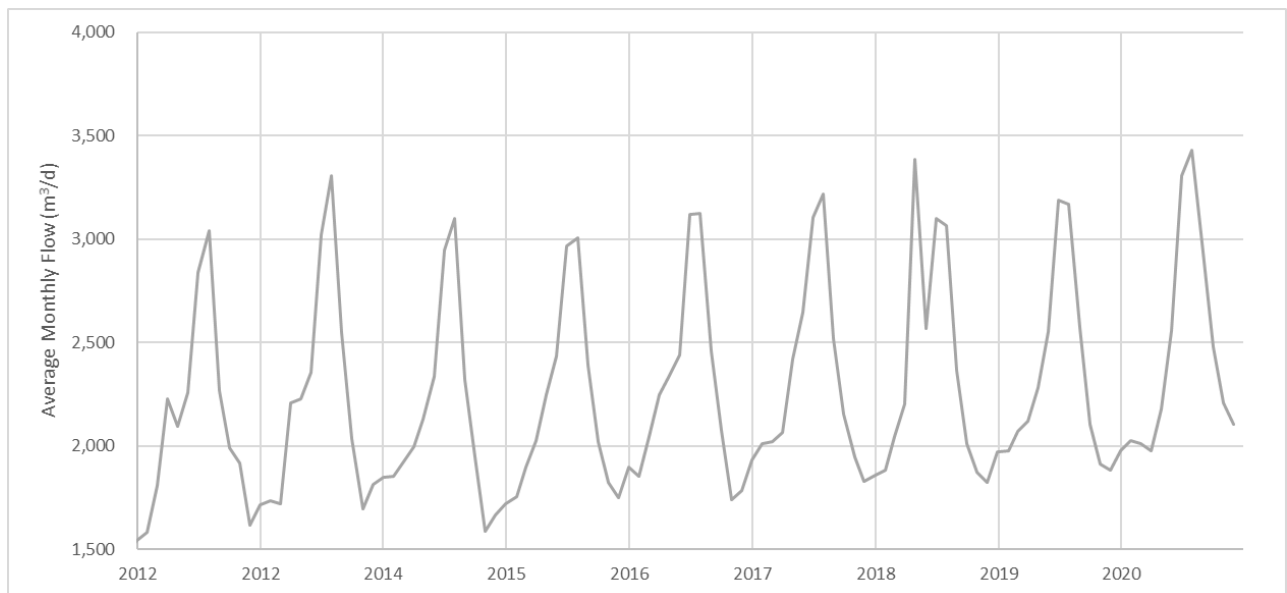


FIGURE 1-5: DAILY FLOW (2019)

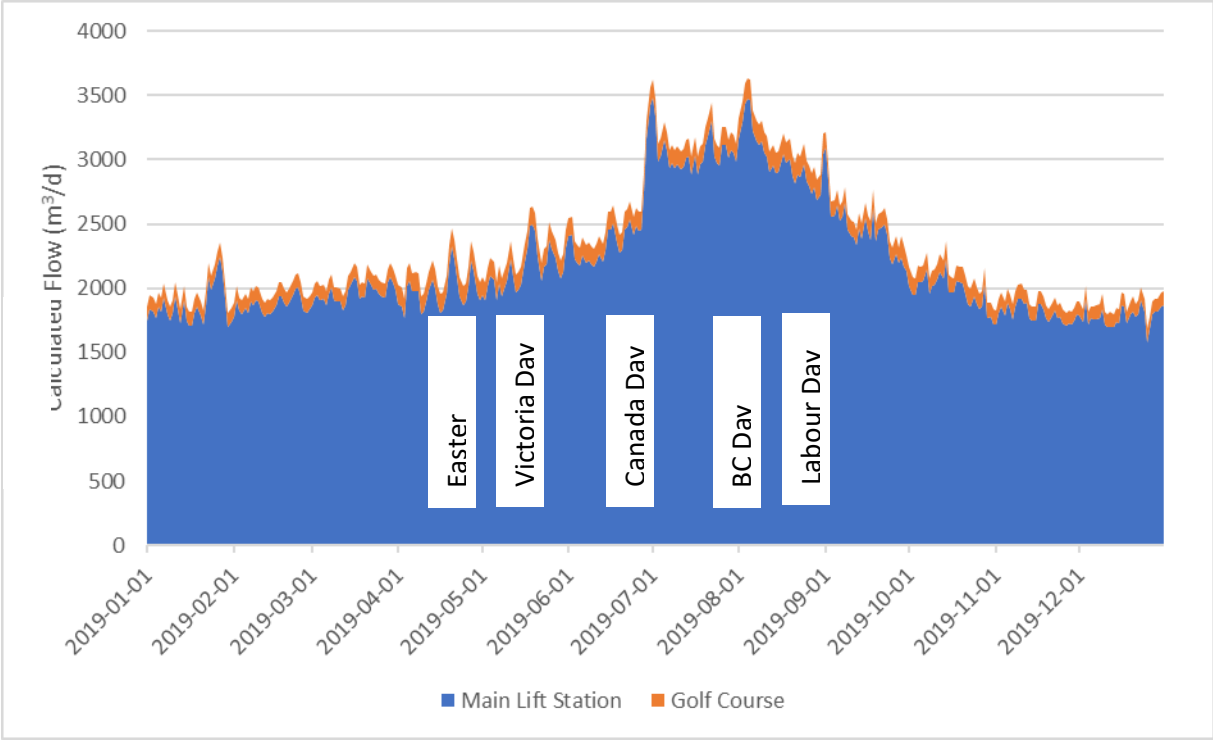


FIGURE 1-6: IRRIGATION FLOW (MONTHLY AVERAGE)

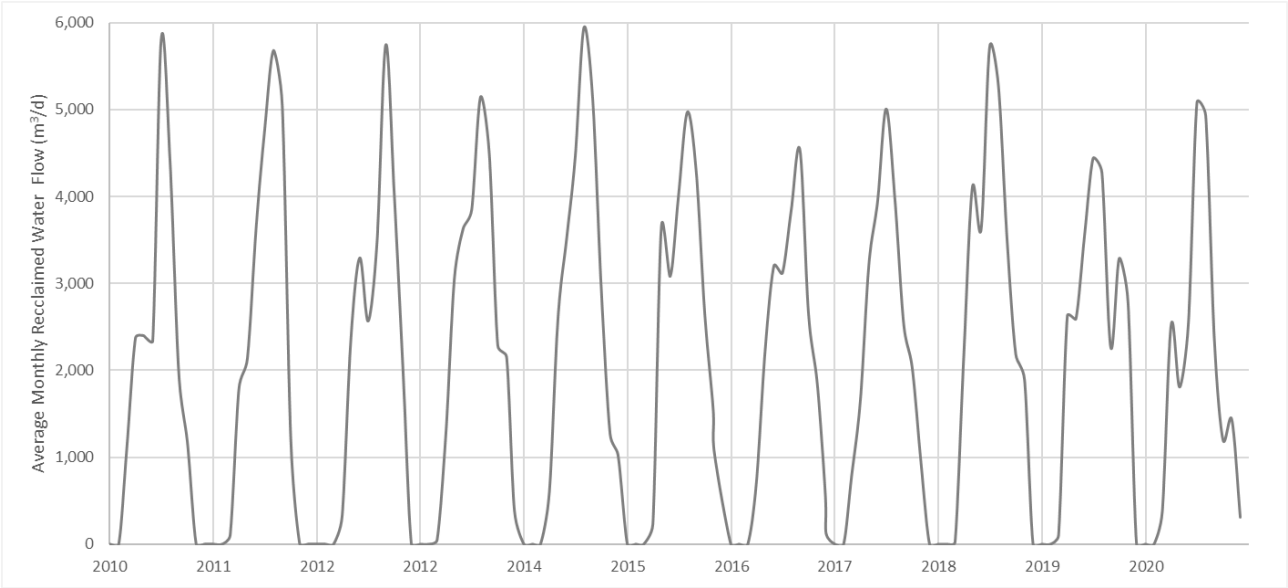
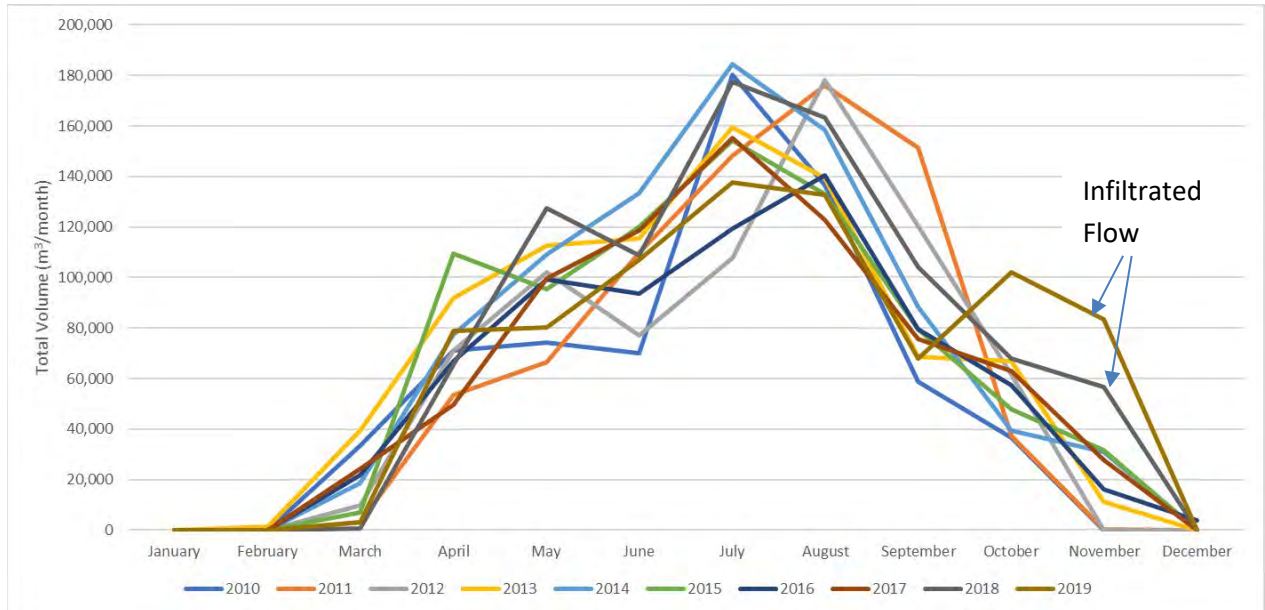
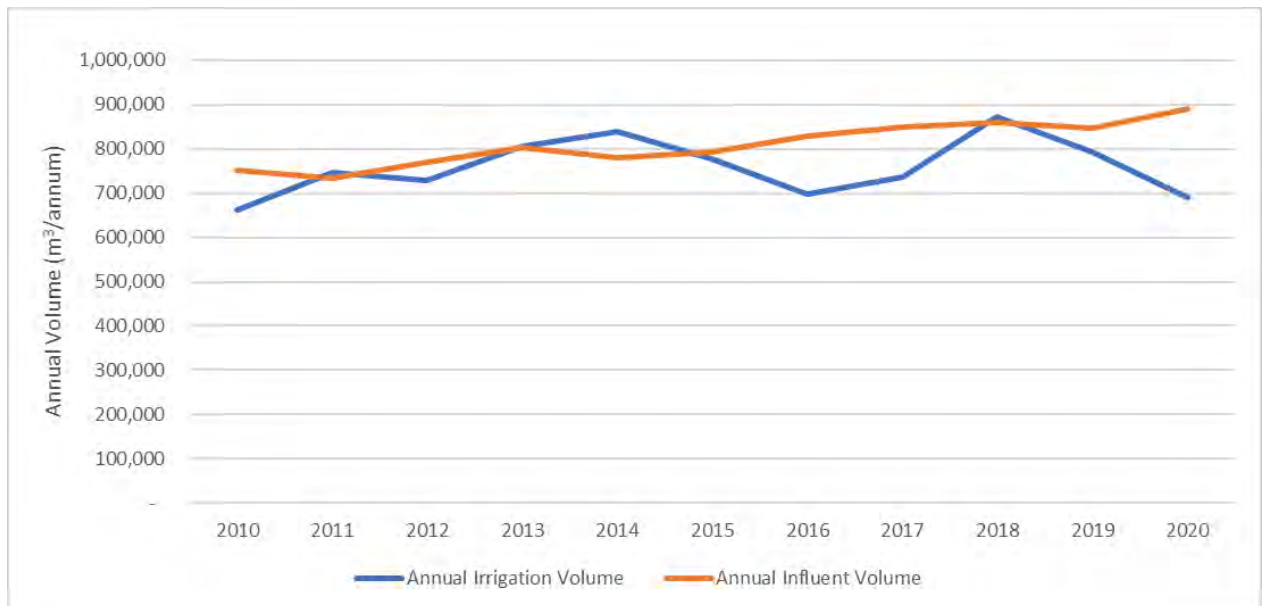


FIGURE 1-7: IRRIGATED / INFILTRATED VOLUME BY MONTH



It is worth noting that while inflow continues to increase, the volume irrigated has not always kept pace with this flow. New users for irrigated water will have to be identified as inflow increases, or the excess water will need to be disposed of by infiltration.

FIGURE 1-8: ANNUAL INFLOW VS IRRIGATION / INFILTRATION VOLUME

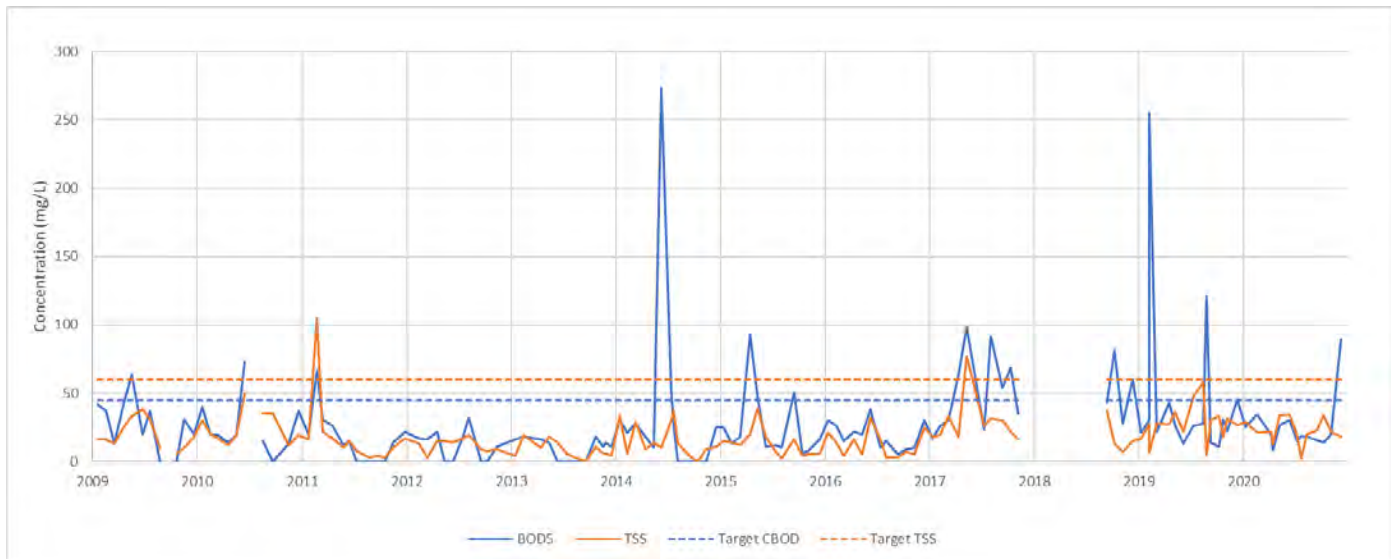


1.3 Existing Treatment Performance

1.3.1 Effluent Quality

The basic treatment performance of the plant is based on five-day biochemical oxygen demand (BOD₅) and Total Suspended Solids (TSS). These results are shown as Figure 1-9.

FIGURE 1-9: LAGOON EFFLUENT QUALITY



Effluent samples were not taken from January to August 2018 while the lagoon liner and aeration system were being replaced in Cell 3.

It can be seen that effluent BOD and TSS have been greatly in excess of target (permit) values in recent times. At face value they indicate a major issue with the treatment system. **However, there is reason to doubt the results, as there are inconsistencies that cast doubt on their accuracy.** A specialist sampling contractor has been engaged to ensure consistency in sample collection.

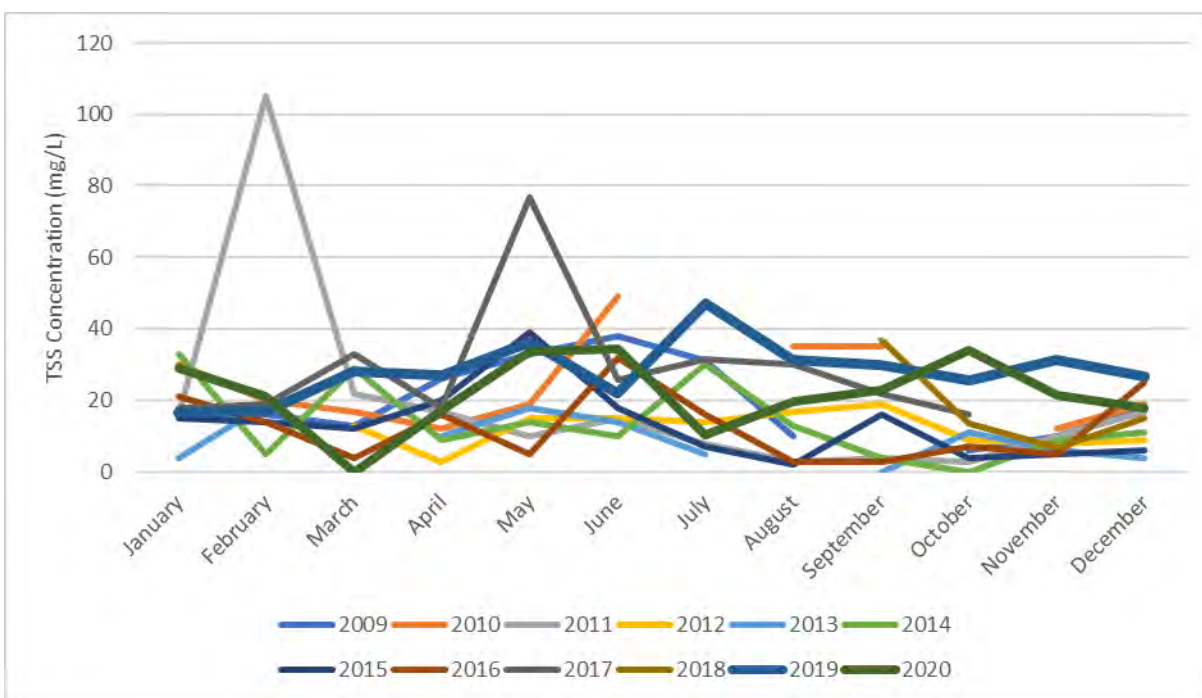
In addition, the BOD results do not directly compare with the BOD test method required by the permit. The Municipal Wastewater Regulation (2012) defines BOD₅ as being the five day carbonaceous biochemical oxygen demand (noted in this report as CBOD). The plant's operational certificate (ME-12214) allows a maximum CBOD of 45mg/L. Water quality testing to date has generally been the standard biochemical oxygen demand (BOD₅), which is the default used by BC laboratories unless they are told to do otherwise.

While the results of the BOD₅ and CBOD test are often similar, they can differ significantly in aerated lagoon effluent. The conventional five-day biochemical oxygen demand tests carbonaceous biochemical oxygen demand (CBOD) plus nitrogenous biochemical oxygen demand (NBOD). The nitrogenous biochemical oxygen demand consists of the oxygen required

to convert ammonia to nitrate. Because aerated lagoons operate at the ambient temperature, little nitrification occurs in the cooler months. At times when the BOD sample is inoculated with nitrifying bacteria but incomplete nitrification has occurred, the NBOD component can be significant. As a consequence, the system may fully comply with the maximum 45mg/L CBOD requirement, but appear to fail based on BOD₅. A period of analysis of both parameters (BOD and CBOD) is recommended to demonstrate the relationship.

The plant has also had exceedances on total suspended solids. The suspended solids don't show a clear seasonal pattern (Figure 1-10). Summer values are commonly high as a result of algae growth.

FIGURE 1-10: ANNUAL TSS TREND



Nitrification to remove ammonia adds to oxygen requirements, but significantly reduces chlorine demand. Figure 1-11 and Figure 1-12 indicate that nitrification has been occurring in the summer months.

Nitrification should also be occurring in the storage lagoons during the warmer months. However, because the reservoirs receive a large part of their flow during periods when little nitrification is happening, reclaimed water ammonia levels don't reach the low levels which occur during summer in the Cell 3 effluent.

FIGURE 1-11: CELL 3 EFFLUENT NITROGEN – JULY SAMPLE

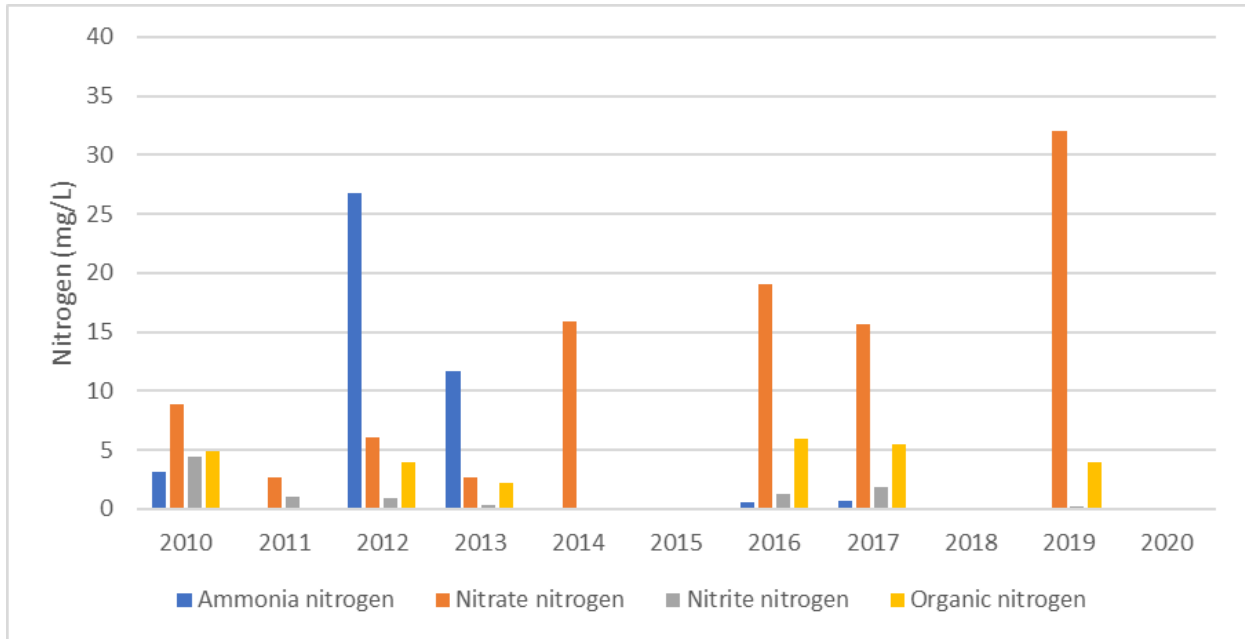


FIGURE 1-12: CELL 3 EFFLUENT NITROGEN – JANUARY SAMPLE

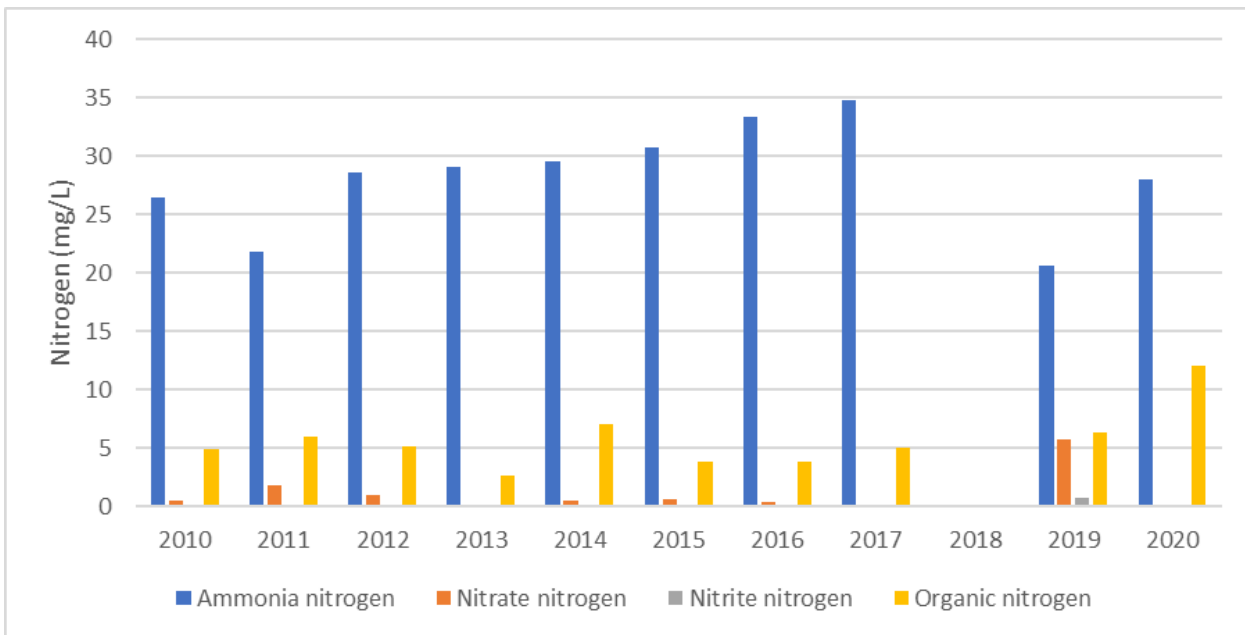


FIGURE 1-13: RECLAIMED WATER NITROGEN – JULY SAMPLE

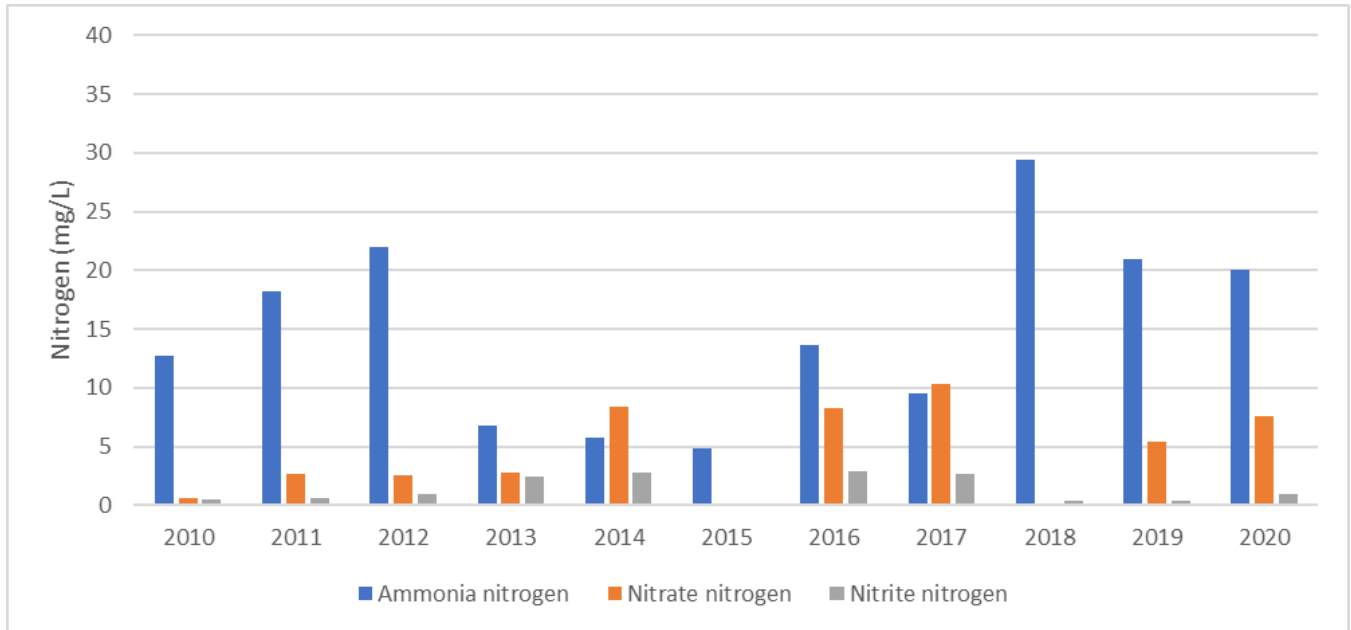
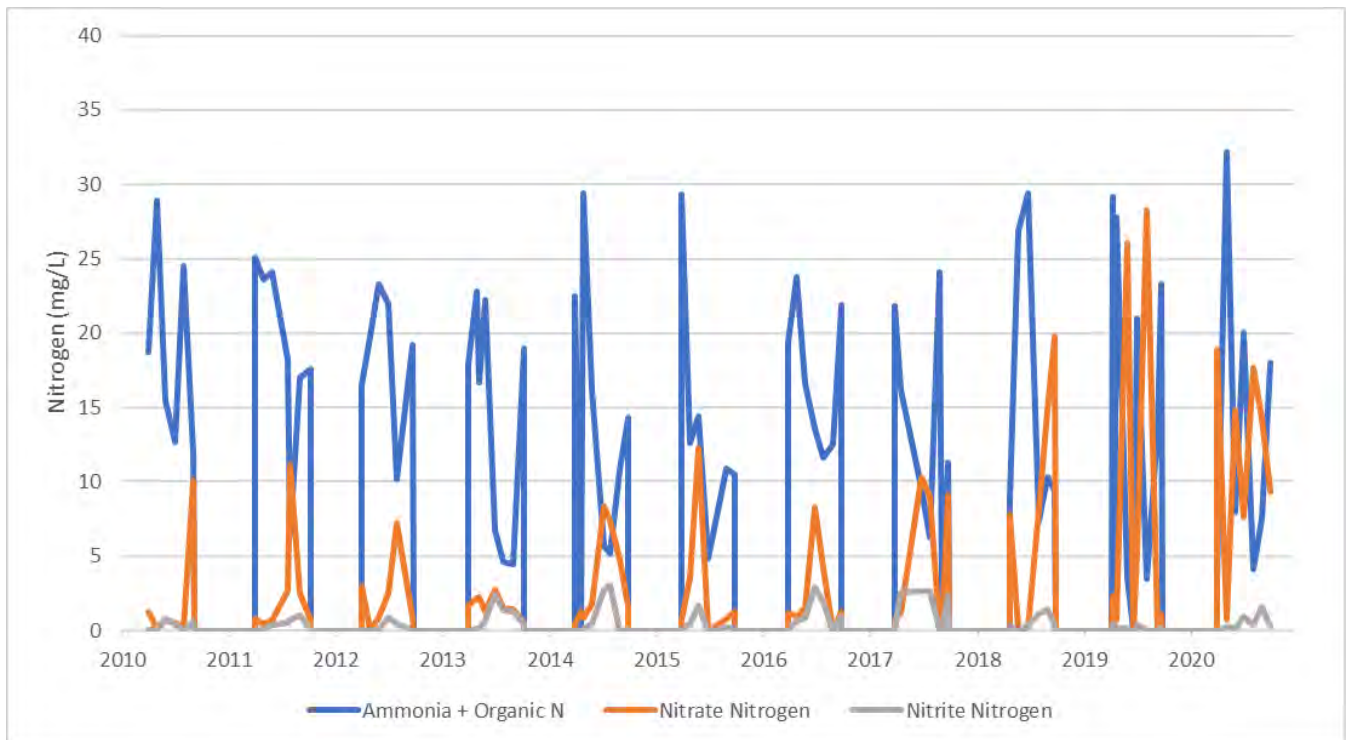


FIGURE 1-14: RECLAIMED WATER NITROGEN – ALL SAMPLES



1.3.2 Odour Control

Odour has been a major issue at the WWTP, and in the collection system. There have been numerous complaints and the issue has featured in news articles. The issue appears to relate to hydrogen sulphide generation in the sewer force mains. This is released later when the gas can escape, such as at the WWTP. Hydrogen sulphide has a characteristic rotten egg odour and is detectable at concentrations as low as 1 part per billion, so the odour can carry long distances.

Due to the layout of the sewer system and the topography of the Town, Osoyoos has many long force mains with correspondingly long residence times. All sewage flow is directed to the Main Lift Station, except flow from the Golf Course lift station.

Aside from the odour, hydrogen sulphide is toxic at elevated concentrations. It also combines with water in damp conditions to form sulphuric acid, which corrodes metals and concrete. It is common to find significant decay of infrastructure in gravity sewers downstream of forcemains connections. Therefore, the sulphide is more than an odour issue and it needs to be addressed to avoid the cost of widespread sewer replacement.

The Main Lift Station includes a fan to draw air from the building for treatment in an activated carbon filter. Nevertheless, hydrogen sulphide levels have been measured inside the manhole upstream of the Main Lift station (PSAB A189 / ID SMH0289) in excess of 100mg/L. Elevated levels around 6mg/L are commonly recorded inside the Main Lift Station building. This gas has already caused blackening of copper water pipe inside the new building. In British Columbia, the Occupational Exposure Limit for H₂S is 10 ppm. It appears that this limit is occasionally exceeded (see Figure 1-15). No forcemain swabbing has been undertaken, as the forcemains are not configured for this function. Forcemain swabbing can have some impact on odour by removing the slime layer on the pipe walls. An alternative to swabbing is shock dosing of sodium hydroxide, where the forcemain pH is elevated above 12.5 for at least 30 minutes to inactivate the sulphate reducing bacteria.

The most effective approach to addressing the pipeline odour is ongoing chemical addition to inhibit H₂S formation. Various chemical dosing options are available;

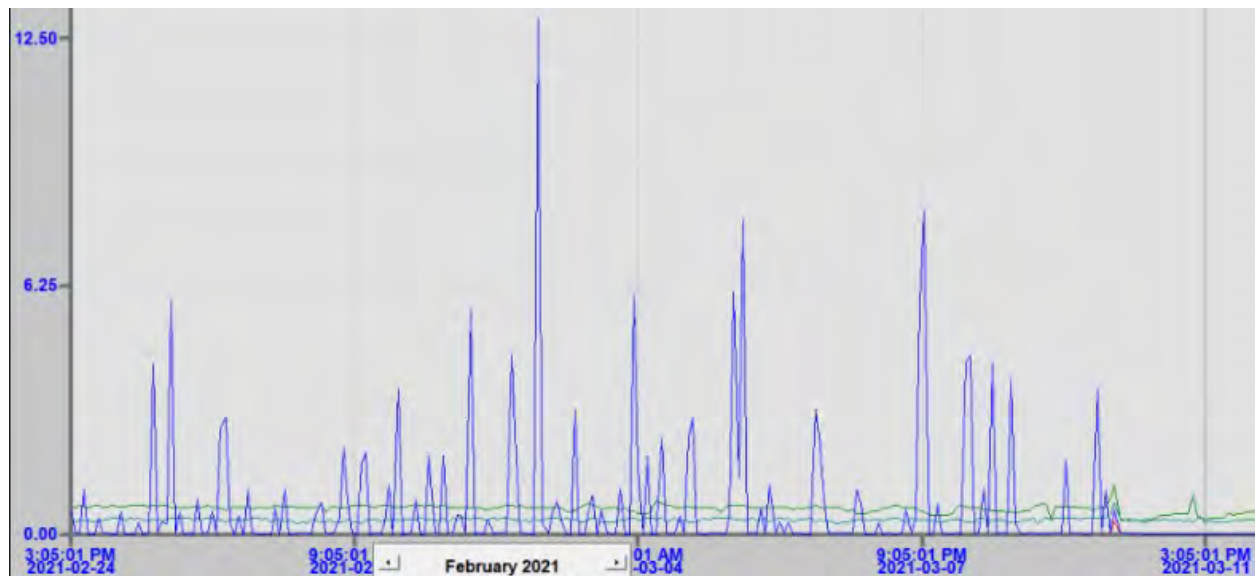
- Air, oxygen
- Nitrate
- Chemical oxidants (hydrogen peroxide, hypochlorite, ozone, potassium permanganate)
- Iron salts (ferrous sulfate, ferrous chloride, ferric sulfate, ferric chloride)
- pH adjustment chemicals to increase pH over 8 (magnesium hydroxide, calcium hydroxide, sodium hydroxide, lime)
- Proprietary formulations (e.g. Biomaxx)

The choice of chemical is dependent on many factors including equipment costs, chemical costs, severity of the issue and chemical handling constraints.

A proprietary formulation (Biomaxx®) was dosed at some time after April 2008 at Lakeshore at 44th LS and Oasis LS to address odour issues at the Main Lift Station. It is proposed that this be reinstated on a trial basis, as the dosing equipment is in place and chemical handling safety systems such as safety showers are not needed.

The Biomaxx system could be extended to other force mains based on need, and other chemicals could be considered. For example, if dosing were to be implemented at the Main Lift Station, then options such as magnesium hydroxide or ferric salts may be more cost effective due to the high flowrate and the availability of amenities to support more complex dosing systems. This is the subject of a separate study and is not discussed further in this report.

FIGURE 1-15: H₂S LEVELS INSIDE MAIN LIFT STATION BUILDING (FEBRUARY 2021)



2.0 Component Status

2.1.1 Complete Mix Lagoon (Cell 1)

Cell 1 does not have an ideal configuration for use as a complete mix reactor. It was originally designed in 1978 to operate as a sludge settling lagoon. It had surface aerators and a paved ramp for drive in access.

The width and sloping sides mean that there is effectively no flat floor area. The bottom elevation is also 1.2m higher than the other lagoons. As a result, the suspended aerators are estimated to be set around 1.8 – 2.5m higher than the other lagoons, and only one third of the area of the lagoon can be aerated.

Having the aerators set higher than the other lagoons means that air will preferentially flow to these diffusers. This can be controlled by throttling the valves on the laterals but, as the necessary air pressure is being generated anyway, it makes sense to have the diffusers set at the same elevation as the other lagoons. This is because the deeper a diffuser is set, the more efficient the transfer of air becomes.

With such a small part of the lagoon area being aerated and mixed, only part of the lagoon is actually operating as a complete mix reactor. The active volume would account for perhaps half of the total volume. As a result, the effectiveness of treatment is significantly reduced, and large parts of the lagoon are only aerated through circulation of the contents. This may contribute to the generation of odour.

Because the diffuser membranes on the Cell 1 aeration system had not been replaced since 1994, they were replaced in 2020. This should greatly improve aeration efficiency in this cell.

FIGURE 2-1: CELL 1 GENERAL ARRANGEMENT

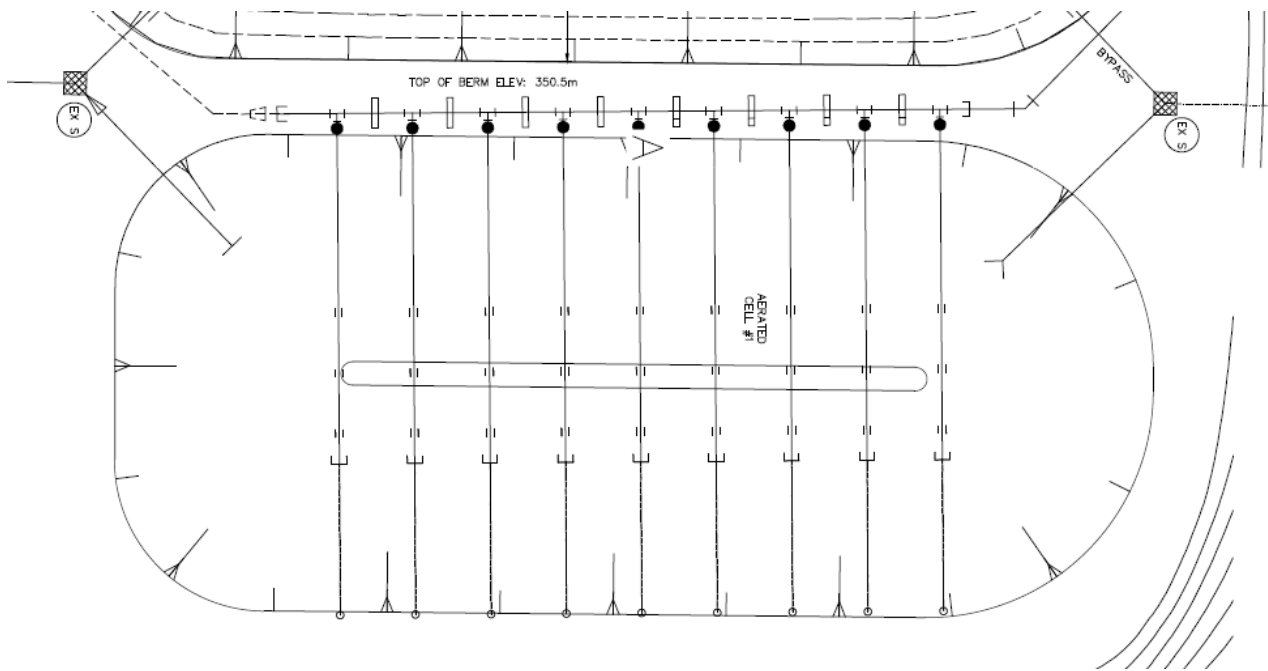


FIGURE 2-2: GOOGLE EARTH IMAGE OF CELL 1 (2013)



2.1.2 Partial Mix Lagoons (Cell 2 and Cell 3)

The existing partial mix lagoons are in good condition and appear to be operating within normal parameters. While they have theoretical capacity for the proposed future flows, the final word is their measured performance. As described previously, if the reported effluent data is correct, this indicates unexpected issues with performance which would require further investigation.

2.1.3 Air System

The aeration systems were installed in the lagoons in 1998 (Cell 1), 2010 (Cell 2) and 2018 (Cell 3). The key component is an EPDM membrane with fine perforations that allows very small bubbles to pass into the lagoon. The membrane is supported on a weighted plastic structure that can be raised to the surface for maintenance. The whole unit is referred to as a diffuser. The expected life of the EPDM membranes is 5 – 10 years. After this time the membranes lose elasticity and effectiveness. The membranes in Cell 1 and Cell 2 were replaced in 2020.

It should be noted that the diffusers in Cell 1 differ significantly from what is listed on the original drawings (28 x Magnum 88S). Aeration capacity is less than originally designed. Further, the diffusers marked as 70S in the table are a non-standard size with four 700mm long membrane tubes. A 1000mm membrane must be trimmed to fit them. The 70S label was created for this project and the manufacturer does not sell a unit for lagoons with these dimensions.

TABLE 2-1: GENERAL AERATION CHARACTERISTICS

Cell	Treatment Volume (m ³)	Residence Time (Summer Peak Flow ¹) Days	Number of EDI Aeration Diffusers	Design Aeration Capacity ² (Nm ³ /min / scfm)	Design Aeration Intensity (Nm ³ /min/m ³)
Cell 1	3,300	0.9	6 x Magnum 88S 5 x Magnum '70S' 15 x Magnum 44S	17 / 600	0.005
Cell 2	27,000	8	15 x Magnum 88F 29 x Magnum 44F	29 / 1000	0.001
Cell 3	58,000	17	11 x Magnum 84F 21 x Magnum 88F	26 / 900	0.001

Air requirements and treatment performance are closely related to ambient temperature. The seasonal variation is shown on Figure 2-3.

When the aeration membranes in Cells 1 and 2 were replaced in 2020 it was noted that the plant is subject to extreme levels of diffuser 'ragging'. Rag materials such as non-degradable 'flushable' wipes wind into rope that collects on equipment in the lagoons. The accumulation is accelerated by flow patterns in the lagoon that take continuous water flow past the aeration diffusers. Rag

¹ A flow rate of 3,500m³/d has been selected to indicate a realistic sustained high flowrate.

² Based on operation at half of maximum capacity for long membrane life.

collects on both the aeration components and the float that allows staff to locate the unit. The rag wrapped around the aeration diffuser can impede air flow and makes it more challenging to recover the unit. Rag on the float can cause the float to sink or even break the rope, making the diffuser difficult to find.

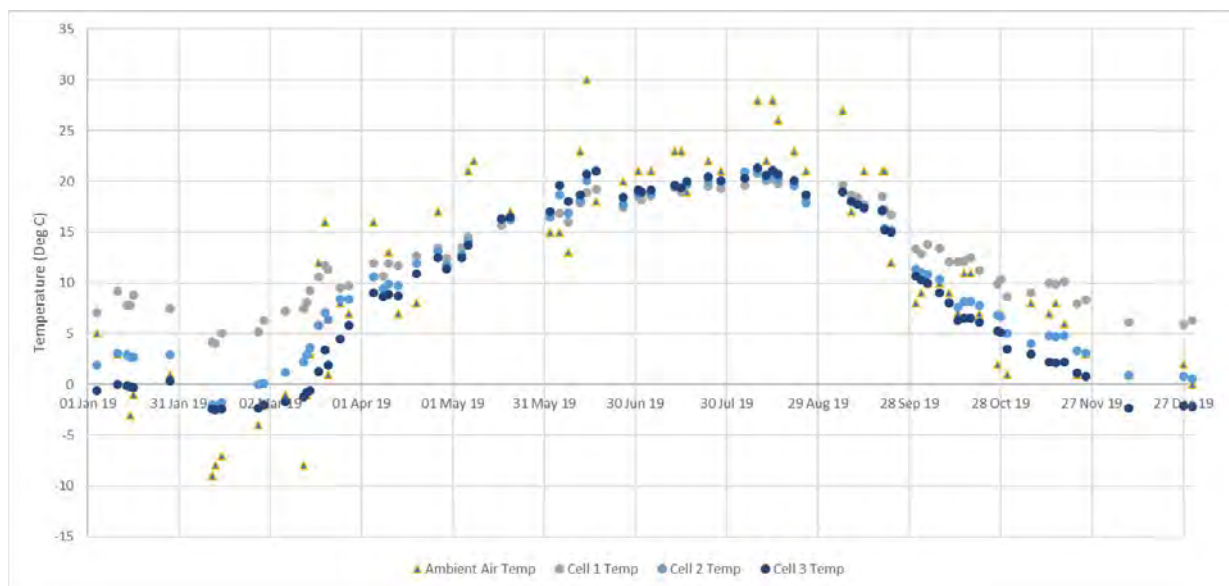
With the commissioning of screening at the Main Lift Station, rag arriving at the plant is now limited to material passing through the Golf Course Lift Station chopper pumps. The Town wishes to remove the rag that remains in the lagoons from the previous years of use in order to preserve aeration performance and avoid harm to the system.

The plant has three Kaeser FB440C blowers operating at up to 2690 rpm, each with a capacity of 978 cfm free air delivery based on 10.2 psig discharge pressure. The blowers run from VFDs but are generally operating at 100% speed. Their inlet capacity is 1650 scfm³.

The blower operating hours are indicative of the air requirements in summer and winter. All three blowers are being used during the peak summer period. Three blowers produce 5,000 scfm. The combined design flow through the diffusers is 2,600 scfm and their maximum (short term) flow capacity is around 5,000 scfm. This indicates that the aeration diffusers are currently being operated at their maximum capacity, which shortens the expected life of the diffuser membranes.

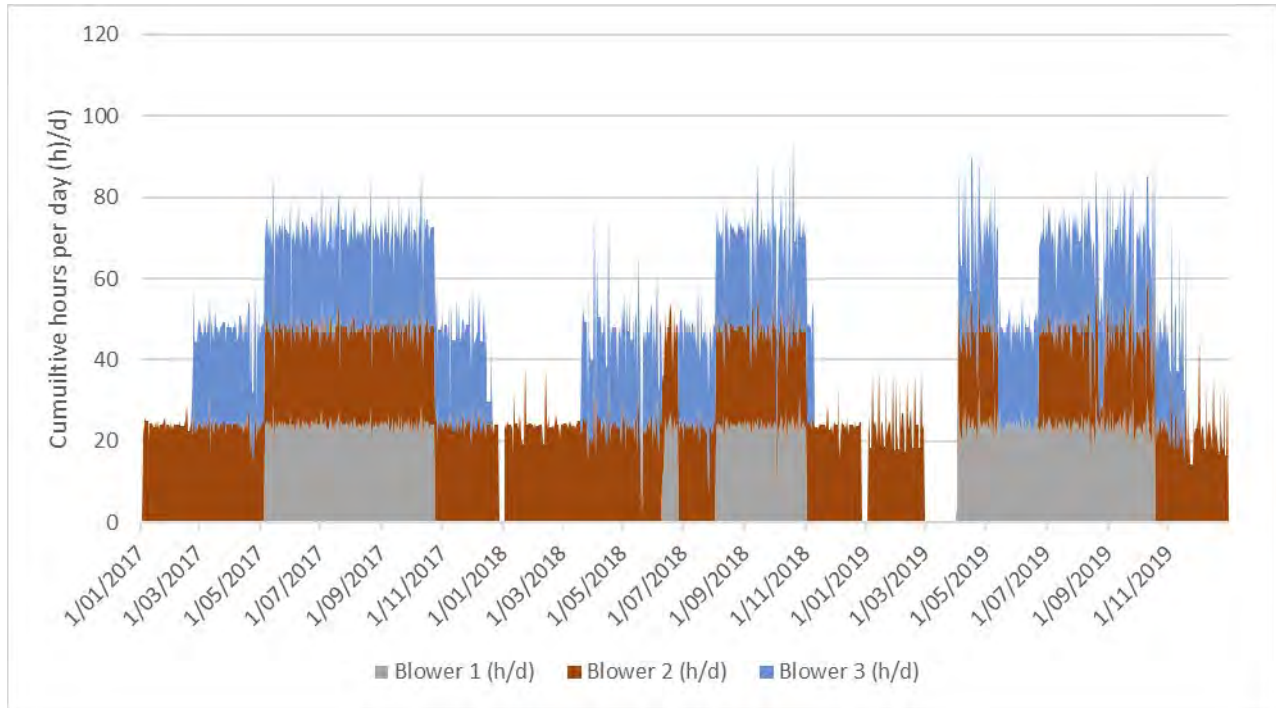
If the existing 60hp motors are replaced with 75 hp motors, the blowers can be upgraded to run at 3,600 rpm for a discharge capacity of 1,225 cfm. 1225 cfm free air delivery is equivalent to 60 Nm³/min (2,100 scfm). Therefore, 180 Nm³/min (6,000 scfm) is feasible if needed for future air requirements.

FIGURE 2-3: AMBIENT AND LAGOON TEMPERATURE TREND (2019)



³ Due to the compressibility of air, blower capacity is commonly based on inlet (atmospheric) air flow.

FIGURE 2-4: BLOWER OPERATING HOURS (2017-2019)



2.1.4 Sludge Accumulation

Sedimentation (settling) of suspended solids is a key function of the partial mix aerated lagoons. Eventually the solids need to be removed from the system once the useful volume and treatment effectiveness are affected. The rate of sludge accumulation in the system depends on several variables including:

- The amount of inert suspended solids entering the system in the raw sewage. The quantity of inert suspended solids will be relatively low because the influent is screened.
- The amount of non-volatile biological solids entering the system with the raw sewage. As sludge accumulates in the bottom of the aerated cells, anaerobic digestion processes occur resulting in volatile solids removal and corresponding reductions in the accumulated solids volume and overall mass.
- The degree of consolidation of the settled solids. As sludge accumulates on the bottom of the lagoons, compression of the actual sludge mass will occur resulting in higher solids concentrations expressed as % solids. Typically, sludge accumulations in aerated cells have solids concentrations in the 5 to 12% range.

Sludge will tend to accumulate near the Cell 2 inlet and away from diffusers. At present it is expected that sludge depths will be less than 1m based on the time since the lagoons were last desludged.

2.1.5 Reclaimed Water Storage

Osoyoos has a total available reclaimed water storage volume of 565,000m³ (Reservoir 1: 270,000 m³, Reservoir 2: 295,000 m³). The system is required to store a minimum of 60 days of flow when storage levels are at their minimum. This is equivalent to approximately 140,000m³. Therefore, the useable volume is 425,000m³ if the reservoirs are 100% full at the start of the irrigation season. Figure 2-5 shows that the reservoirs are typically less than full at the start of the irrigation season. Figure 2-6 indicates the capacity that remains unused by year, taking account of storage volume that has not been filled by the start of the irrigation season.

At present the reservoirs are drained on a more or less alternating basis (see Figure 2-7). The impact this has on chlorination is discussed later in this report.

TABLE 2-2: STORAGE VOLUMES BY YEAR (M³)

Year	Max Volume	Min Volume	Volume Used	60 day storage	Spare Capacity
2010	408,371	238,503	169,868	140,000	255,132
2011	511,515	265,137	246,378	140,000	178,622
2012	527,294	283,799	243,495	140,000	181,505
2013	527,234	265,800	261,434	140,000	163,566
2014	502,308	232,050	270,257	140,000	154,743
2015	477,064	175,531	301,533	140,000	123,467
2016	458,005	237,091	220,914	140,000	204,086
2017	512,561	217,088	295,473	140,000	129,527
2018	499,834	255,615	244,219	140,000	180,781
2019	482,098	212,696	269,401	140,000	155,599
2020	436,449	257,872	178,577	140,000	246,423

FIGURE 2-5: TOTAL STORAGE VOLUME BY YEAR

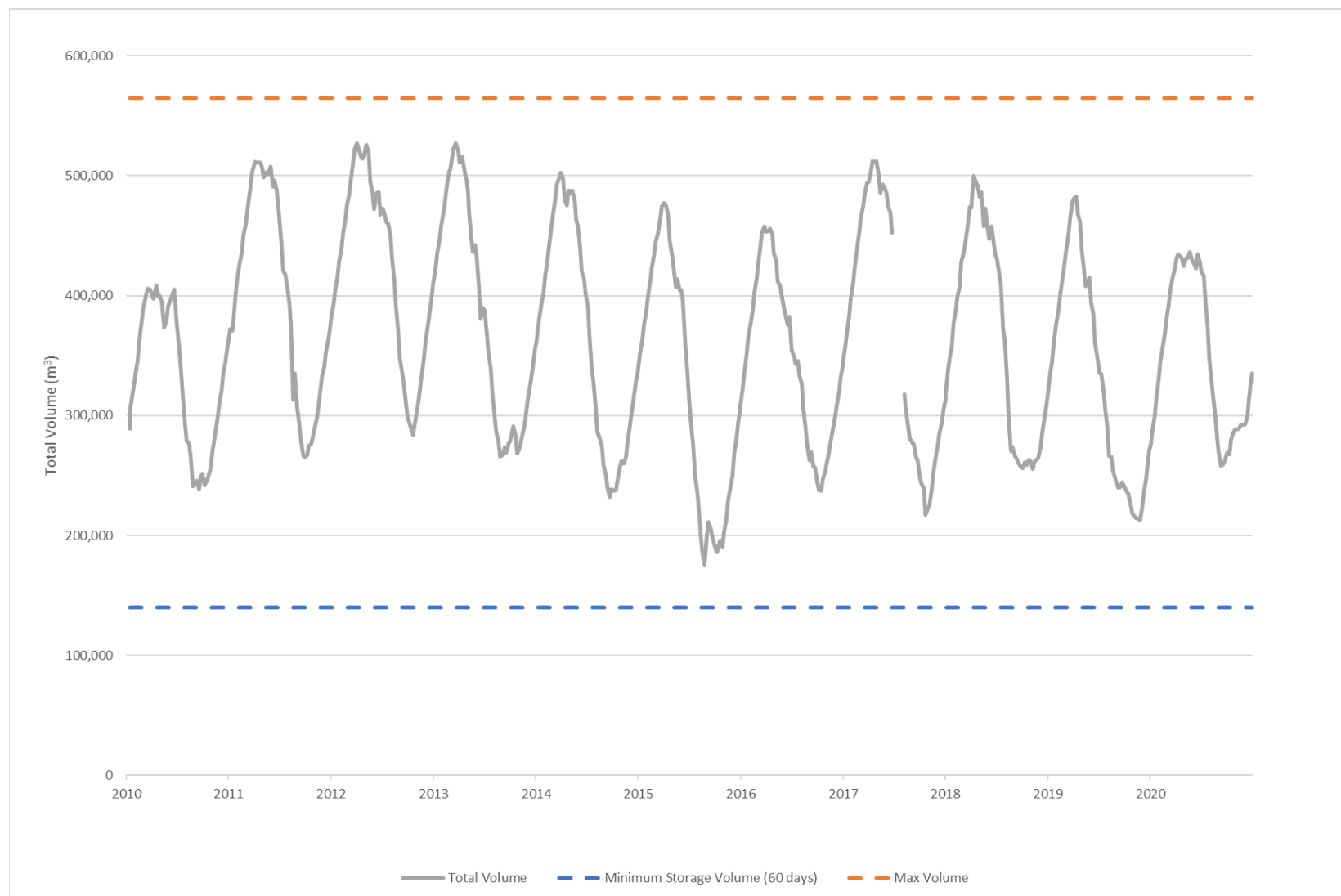


FIGURE 2-6: RESERVOIR SPARE CAPACITY BY YEAR

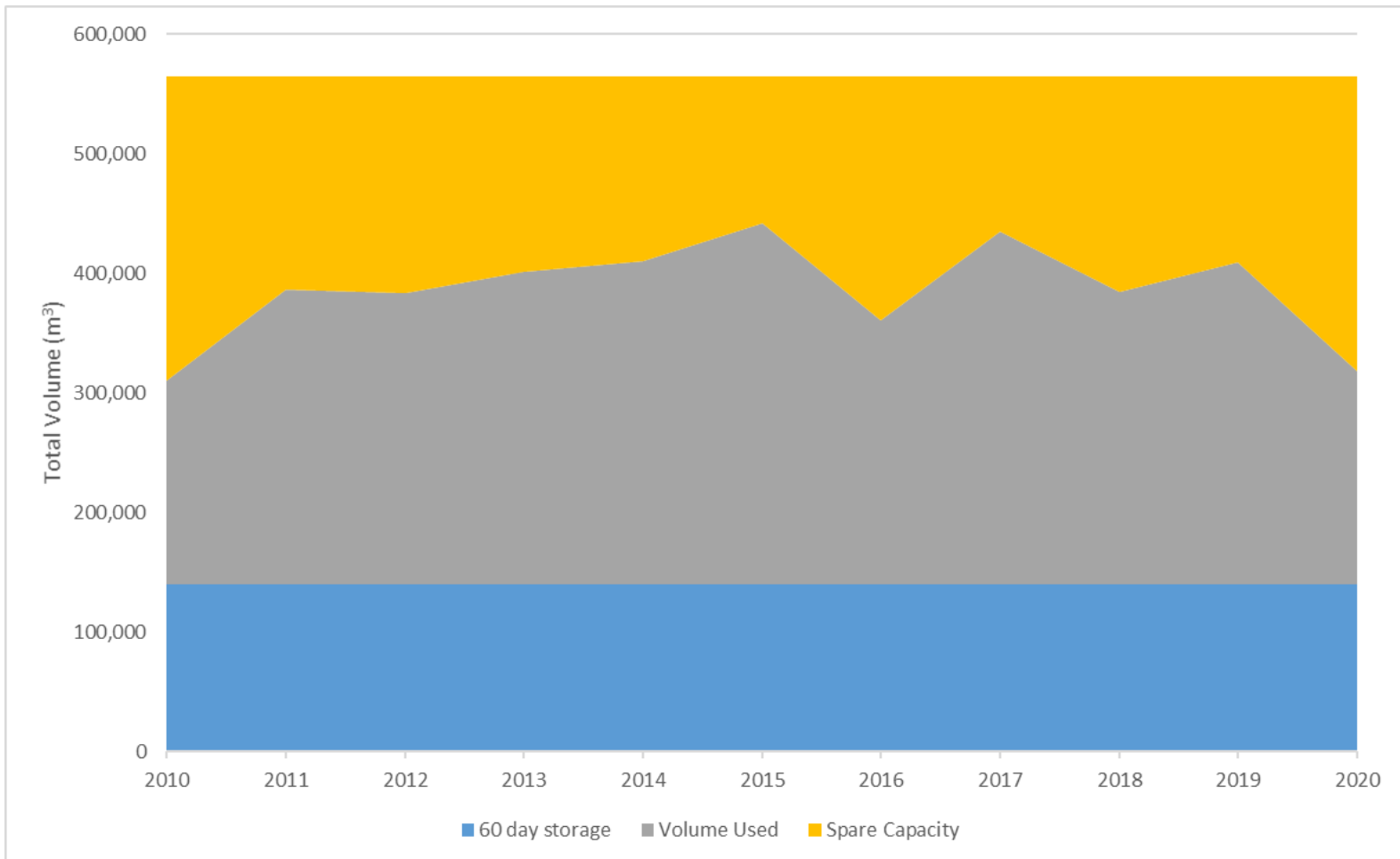
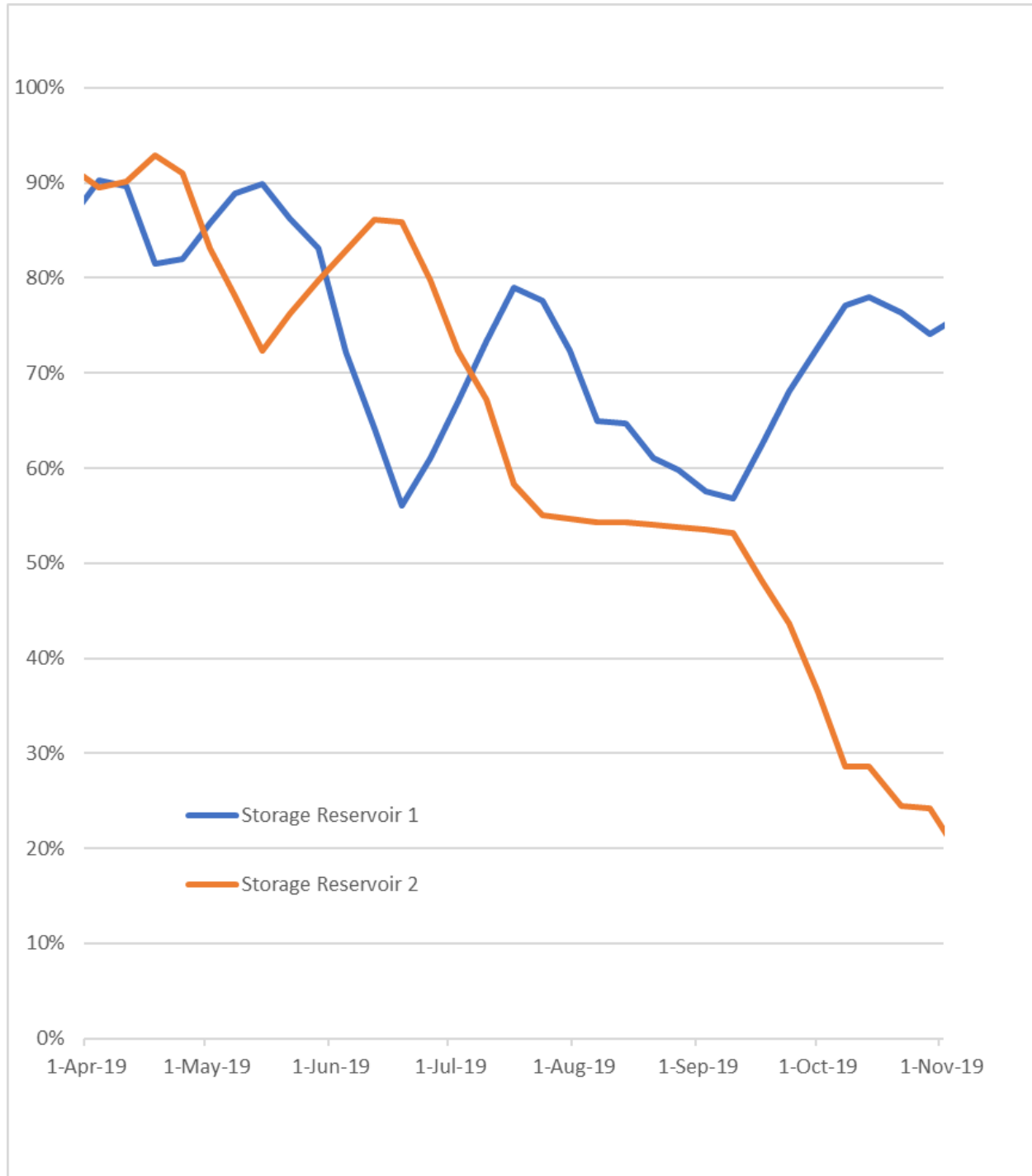


FIGURE 2-7: RESERVOIR % FULL (2019)



3.0 Design Conditions

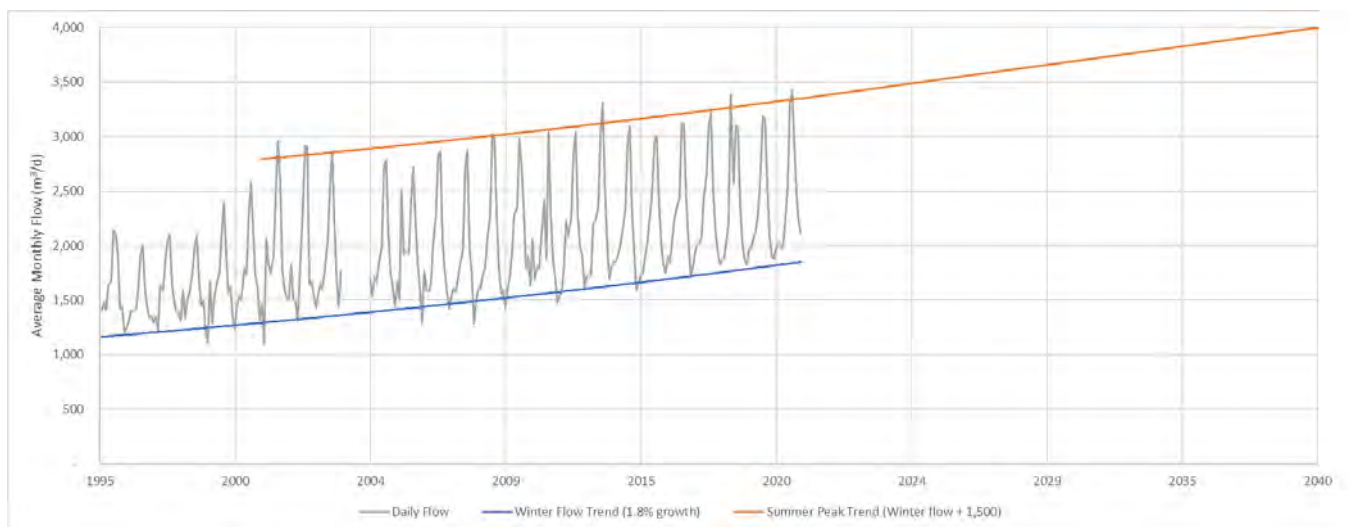
3.1 Design Flow

The required plant treatment capacity is driven by peak flowrate. The **proposed design flow for the design of treatment systems is 4,000m³/d**, which represents the expected summer flow in 20 years at the current rate of growth of the permanent population plus an allowance for the summer population.

It is proposed that this allowance for future growth is conservative for the following reasons;

- The Kerr Wood Leidal report states that there are limited opportunities for population growth inside the town boundary.
- Part of the increase in flow from 2013 to 2016 was due to service area expansion outside the town boundary, not just population growth.
- Future service area expansion has not been decided. Developers outside the town boundary may choose to build independent sewage collection and treatment systems. The Town may also choose not to accept additional waste.

FIGURE 3-1: PROJECTED 2040 FLOW



3.2 Target Effluent and Reclaimed Water Quality

The plant treatment targets are set by Operational Certificate ME-12214 (CBOD 45 mg/L and TSS 60 mg/L). These targets are consistent with the requirements of the Municipal Wastewater Regulation for lagoon effluent.

Treatment targets that are relevant to the Osoyoos WWTP discharge are as follows;

TABLE 3-1: RELEVANT EFFLUENT TARGETS FROM OPERATIONAL CERTIFICATE ME-12214

Quality Parameters	Cell 3 Effluent	Reclaimed Water
Carbonaceous BOD ₅	Max 45 mg/L	NA
Total Suspended Solids	Max 60 mg/L	NA
Turbidity	NA	NA
pH	NA	NA
Fecal Coliforms	NA	<2.2. MPN/100ml
Nitrogen (mg/L as N)	Monitoring required	Monitoring required
Phosphorus	Monitoring required	Monitoring required

The plant Operational Certificate does not align closely with the reclaimed water (greater exposure potential) limits specified by the Municipal Wastewater Regulation (Table 3-2). As a result, reclaimed water monitoring does not generally include CBOD, TSS and turbidity and pH.

TABLE 3-2: MUNICIPAL EFFLUENT QUALITY REQUIREMENTS FOR RECLAIMED WATER (MWR)

Parameters		Indirect potable reuse	Greater exposure potential	Moderate exposure potential	Lower exposure potential
pH		site specific	6.5 to 9	6.5 to 9	6.5 to 9
BOD ₅	Max	5 mg/L	10 mg/L	25 mg/L	45 mg/L
TSS	Max	< 5 mg/L	10 mg/L	25 mg/L	45 mg/L
Turbidity (NTU)	Avg		2	n/a	n/a
	Max	1	5	n/a	n/a
Fecal coliform (/100 mL)	Median	< 1 CFU / < 2.2 MPN	< 1 CFU / < 2.2 MPN	100 CFU	200 CFU
	Max		14 CFU	400 CFU	1000 CFU

The BC Municipal Wastewater Regulation requires a minimum chlorine residual of 0.5 mg/L.

4.0 Lagoon Improvement Options

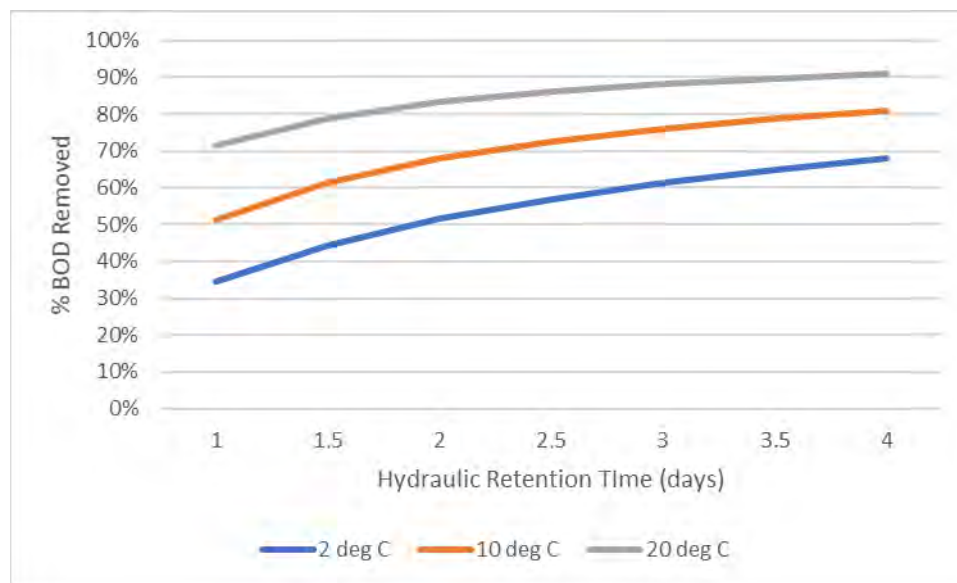
Taken at face value, lagoon effluent quality monitoring (Figure 1-9) indicates that the plant is beginning to exceed effluent quality criteria required under the Operating Certificate (Appendix A). An aeration upgrade was completed in 2020 and the sampling methodology has been revised on the basis of inconsistencies in the results. Monitoring in 2021 will be analysed to determine whether treatment performance is acceptable.

Nevertheless, plant inflow continues to increase, and the day will come when an upgrade is needed. A number of upgrade options have been identified for when the capacity and treatment performance of the plant needs to be improved. These are described in the following sections.

4.1 Upgrade Complete Mix Lagoon System

Complete mix lagoons have a high theoretical treatment capacity relative to partial mix lagoons. Therefore, this process should contribute significantly to treatment capacity. The theoretical BOD removal efficiency for a complete mix reactor is shown by Figure 4-1 based on a temperature dependant rate constant for BOD removal of 2.5 d^{-1} .

FIGURE 4-1: THEORETICAL BOD REMOVAL VS RETENTION TIME AT VARIOUS TEMPERATURES



Because flows have increased steadily over the years and the summer HRT in Cell 1 has fallen below 1 day (based on the full cell volume, not the active area). The USEPA recommends a retention time of 1.5 – 3 days in a complete mix reactor. In order to bring the performance of the complete mix stage closer to recommended conditions, it would be necessary to increase the capacity of the reactor to 2 – 3 days. A $10,000\text{m}^3$ reactor with a residence time of 2.5 days has been assumed. The expected BOD removal can be inferred from Figure 4-1.

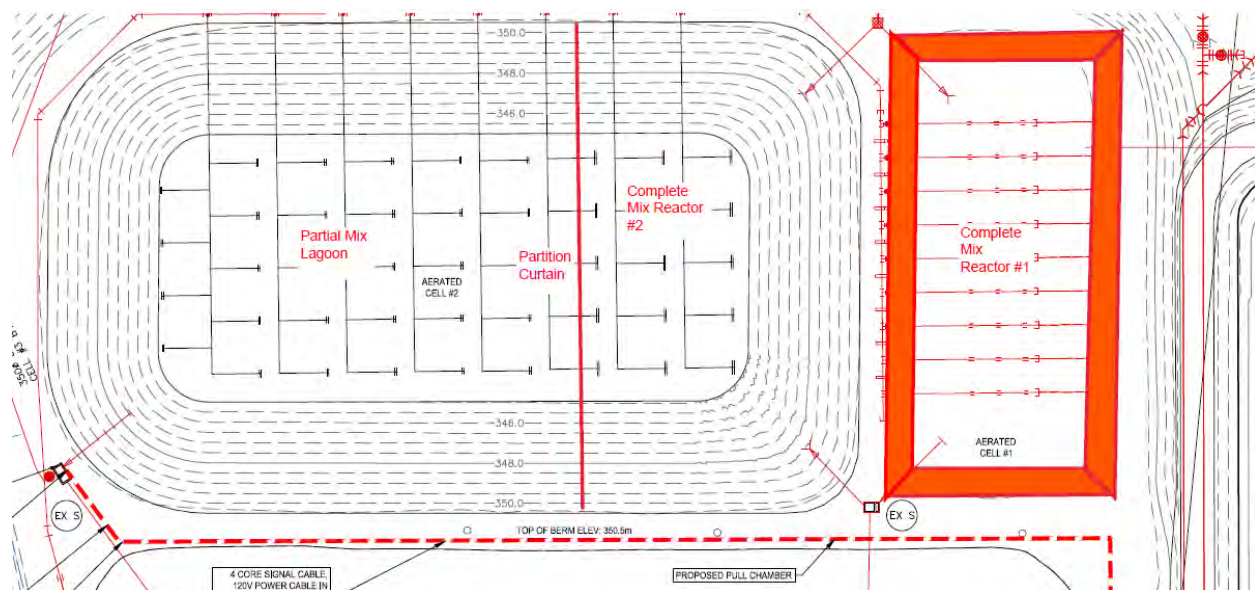
If only one complete mix reactor was required, then partitioning Cell 2 into a complete mix zone and a partial mix zone would be feasible. Cell 1 could be abandoned. However, with this configuration, it would not be feasible to take the combined treatment cell off-line for maintenance while maintaining effluent quality. As a result, this option is considered to be impractical.

The Municipal Wastewater Regulation (s35) requires the plant to have at least two aerated lagoons, with a capacity of 75% of the maximum design flow with one lagoon out of service. In order to achieve these criteria, two complete mix reactors would be needed.

The second complete mix lagoon could therefore be rebuilt within Cell 1 with concrete lined sloping (1:1) or vertical walls to reduce the 'dead' area and increase overall treatment volume. Cell #3 could also be converted to a complete mix / partial mix lagoon although the lagoon shape would make this more challenging for the creation of rational flow paths.

The proposed complete mix system will be larger than the existing system in Cell 1 and the overall WWTP air requirements would increase if operated in the same way. However, because most of the air is used for mixing rather than aeration, it is possible to cycle the air through the laterals in the complete mix system using automated valves, which means the total air requirement can remain within the capacity of the existing infrastructure.

FIGURE 4-2: COMPLETE MIX REACTOR (OPTION 1)



The cost to enhance the existing complete mix reactor for future flows is estimated as follows;

Complete Mix Lagoon Treatment	\$2,500,000
▪ Excavate and rebuild Cell #1 concrete walls and 5m water depth.	± 50%
▪ Line with concrete walls and concrete floor	
▪ Sludge removal and disposal from Cell #2	
▪ Partition curtain in Cell #2	
▪ New aeration systems c/w automated valve control	

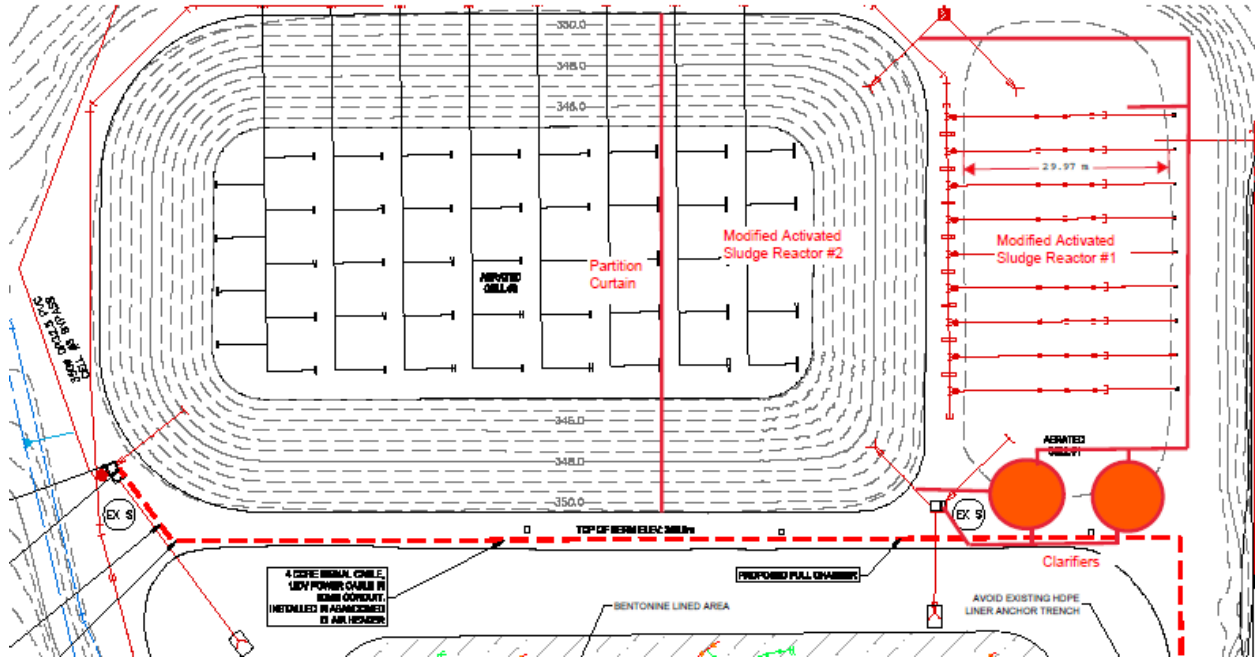
4.2 Modified Activated Sludge Process

Complete mix aerated lagoons are designed to operate as flow-through reactors. Most systems are operated without solids recycle. However, with the addition of systems to recycle solids, the process becomes a basic form of the activated sludge process.

Under this scenario, a pair of ~10m diameter clarifiers would be constructed to allow a proportion of biological solids to be returned to Cell 1. This would prevent the washout of biomass and lead to a higher level of treatment. The design has been assumed to be based on a conventional clarifier. Rather than replacing the existing process with activated sludge plant, the process could operate at a very high rate, with the objective of simply pre-treating the influent. Waste sludge would either be disposed of to the partial mix lagoon system, or dewatered and taken to landfill.

This process would make plant operation more complex due to a need to manage a sludge disposal process in addition to the aerated lagoons. Air demand will increase due to the retention of biological solids in the reactor. The Municipal Wastewater Regulation would also require the construction of two activated sludge reactor tanks in addition to two clarifiers. This would be reasonably challenging to achieve.

FIGURE 4-3: MODIFIED ACTIVATED SLUDGE PROCESS CONVERSION



The cost to complete a modified activated sludge upgrade using the wedge-shaped settling tank is estimated as follows;

Modified Activated Sludge Treatment	\$3,500,000
- Sludge removal and disposal Cell #2 - Partition curtain in Cell #2 - New aeration systems - Additional blower capacity? 300mmØ aeration header extension 300mmØ wastewater pipework 10mØ clarifiers - Sludge recirculation pumping	± 50%

This option might be made more cost effective if a modified version of the clarifier used in the Ecofluid USBF reactor and some oxidation ditches could be adapted for use in a lagoon. This would consist of an open bottom wedge-shaped settling tank mounted off the lagoon floor with an effluent outlet pipe and a waste sludge pump taking effluent and sludge to the next cell. The mixing of retained biological solids back to the lagoon inlet may require a second collection and transfer pumping system. This system would be a novel treatment configuration and would require significant experimentation to achieve the required performance. It is not proposed at this stage as we would essentially be inventing a new treatment process.

4.3 Sequential Batch Reactor

Aerated lagoons can be converted to operate as sequential batch reactors, which fill and draw the waste in order to settle out and retain solids.

The most feasible approach to this upgrade would be to convert an existing cell from an aerated lagoon process to an intermittently decanted extended aerated lagoon / sequential batch reactor system. This process would treat the wastewater in batches, periodically decanting the effluent and maintaining high concentrations of microorganisms without washout. This process could achieve both improved BOD reduction and nitrification, reducing ammonia levels for a greater part of the year. The process requires increased aeration and a decanting system. Construction costs would primarily relate to upgrades to the aeration system, partitions and any pumping requirements.

There are multiple configurations available for this process. The water level can be allowed to rise and fall around 0.5m, or moving baffles can be used to allow the water level to remain constant. The process is marketed in North America by Environmental Dynamics International (IDEAL process) and Bioworks North America, among others.

The Municipal Wastewater Regulation (s35) requires aeration basins to have at least two cells, each with a capacity of 75% of the maximum design flow. A sequential batch reactor (SBR) basin would be created in Cell #1, as well as in part of Cell #2. A new partition curtain would be required in Cell 2 to create two treatment zones.

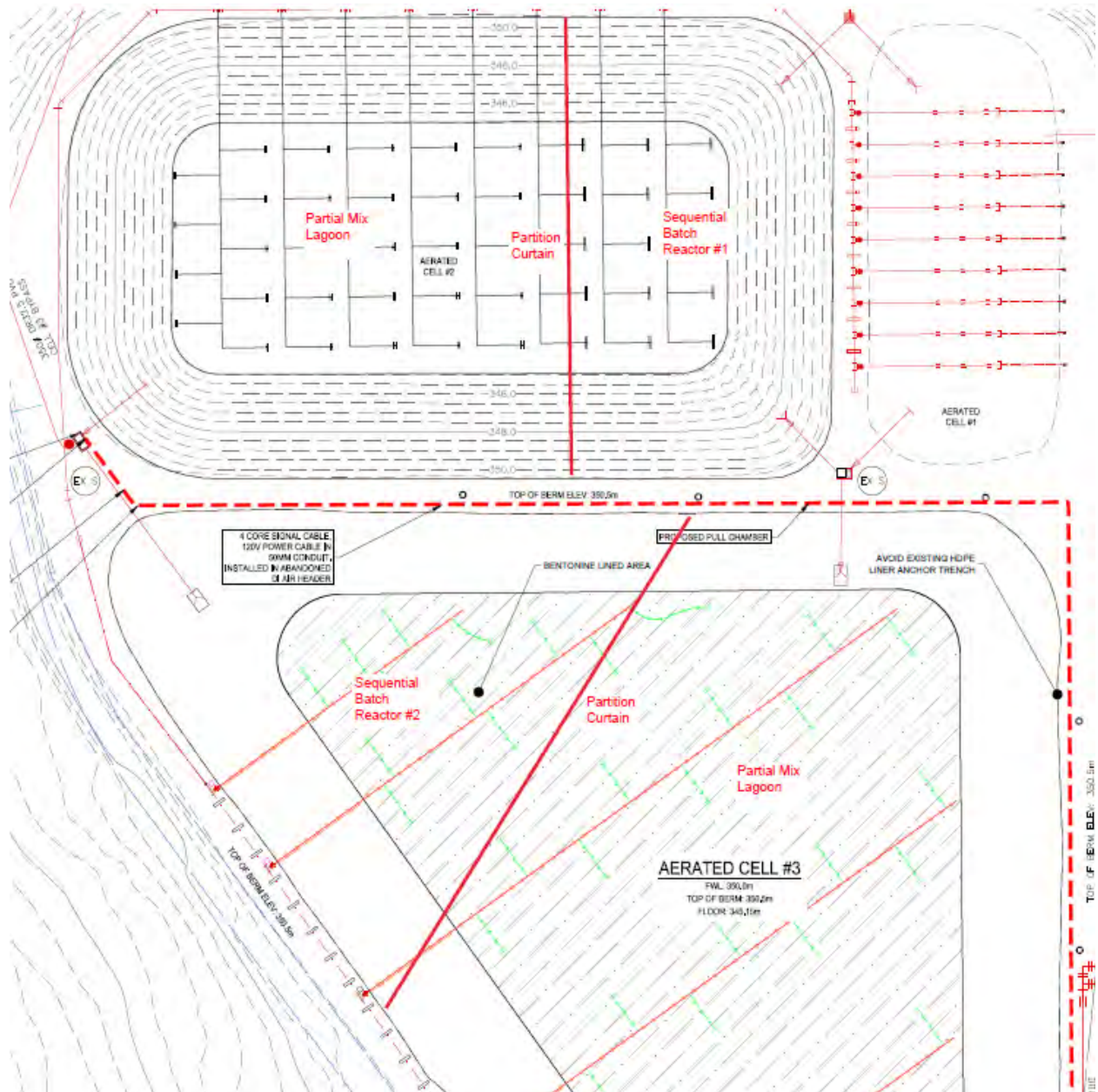
Sludge generated by the process must be collected and disposed of. It has been assumed that the sludge stream would be mixed with the effluent and continue to the partial mix reactors for settling and storage. This is equivalent to what happens at present with the complete mix reactor.

An alternative configuration would see the waste sludge collected, digested, thickened and dewatered for disposal. In this configuration the waste sludge would be discharged to aerobic digesters. Sludge would be thickened by decanting. The thickened sludge would then need to be dewatered by centrifuge or a similar process prior to disposal. This option would be cost prohibitive and difficult to accommodate on the existing site footprint.

The process would be automated with alternating aeration in the two treatment cells. Increased operator expertise would be required to manage this process.

The achievable effluent quality has the potential to meet 25/25 mg/L CBOD/TSS or better if the sludge is removed from the SBR effluent stream (rather than going to the partial mix lagoons). This would require the construction of the separate sludge management processes.

FIGURE 4-4: SEQUENTIAL BATCH REACTOR LAYOUT



The cost for a sequential batch reactor upgrade is estimated as follows;

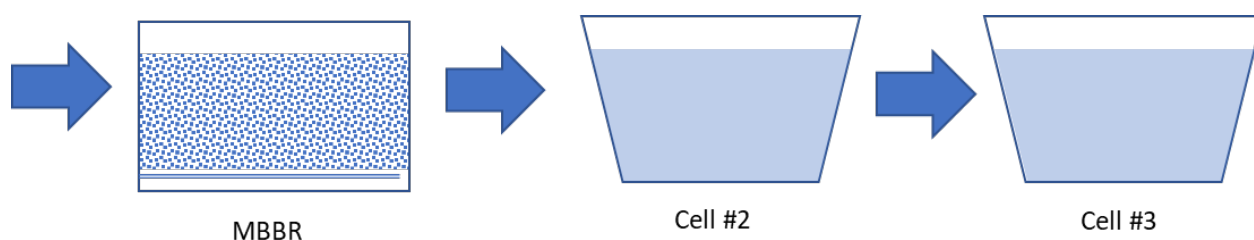
Reconfigure Cells 1 and 2 to SBR Process ▪ Sludge removal ▪ New aeration systems ▪ Excavation, new walls and lower floor in Cell 1 c/w concrete lining and asphalt floor. ▪ Partition curtains in Cell 2 and Cell 3 ▪ Sludge pumping and pipework to Cells 2 and 3 ▪ Effluent decanter systems	\$2,000,000 ± 50%
Stand Alone Sludge Management – Treatment and Dewatering ▪ Aerobic digesters ▪ Centrifuge in new building	\$0.8 – 1.5M ⁴ ± 50%

4.4 Moving Bed Biofilm Reactor

A Moving Bed Biofilm Reactor (MBBR) consists of an aerated concrete tank containing neutrally buoyant media which retains high concentrations of microorganisms without washout. The method of operation is very similar to an aerated lagoon, with sludge management continuing as it is now and little operator training required.

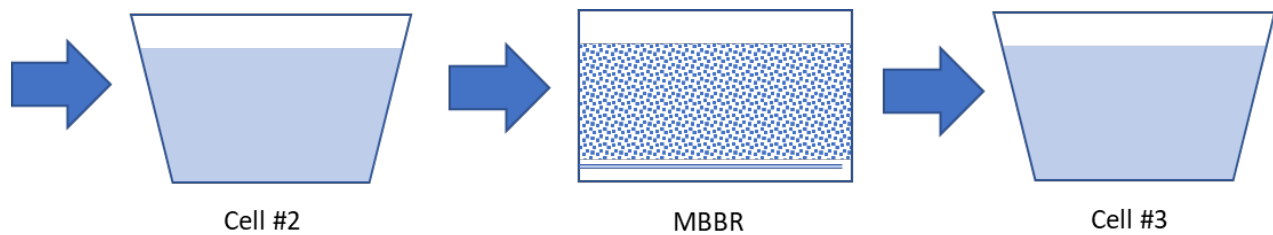
A Moving Bed Biofilm Reactor (MBBR) can be installed upstream of the existing system as pre-treatment, part way through the aerated lagoon process, or downstream for effluent polishing and ammonia removal.

FIGURE 4-5: MBBR PRE-TREATMENT CONFIGURATION



⁴ Cost is heavily dependant on details of the design

FIGURE 4-6: MBBR WITH INTERMEDIATE LOCATION



There is an example of MBBR pre-treatment in Radium Hot Springs, BC. While it is a pre-treatment system, this reactor has a clarifier and sludge digester downstream, rather than discharging directly from the MBBR to their partial mix aerated lagoons (see Figure 4-7). This was intended to reduce the de-sludging frequency of the lagoons. It does mean that operators now need to operate a clarifier and manage sludge daily. They are also dealing with issues related to clarifier performance, such as poorly settling filamentous sludge.

It is recommended that any implementation of the MBBR technology at Osoyoos be kept relatively simple, without separate clarifiers or sludge management. An MBBR pre-treatment configuration as shown in Figure 4-5 will greatly reduce the load on the partial mix lagoons downstream and improve overall BOD removal.

The MBBR process is particularly dependant on efficient influent screening, making the screening of the Golf Course Lift Station flows an essential part of this design. This would require the Golf Course lift station flow to be redirected to the Main Lift Station, or a screening system would need to be installed at the Golf Course lift station. One product that is soon to come onto the market can be fitted inside the lift station and brings screenings to ground level. This system has been included in the budget cost. Screening would also be beneficial for the other treatment options listed in this report.

The intermediate location shown in Figure 4-6 would pass the flow through Cell 2 which would mean that modifications to the Golf Course lift station could be avoided. A simple screen would still be needed on the inlet to the MBBR. Although improved ammonia removal is not strictly necessary at this plant, the intermediate location would achieve this outcome and increase overall BOD reduction, without a need for sedimentation tanks for the MBBR process. The rate of aeration in Cell 2 would need to increase to avoid overloading. This may be as simple as replacing the EDI 44F diffusers with 88F diffusers, which could be done without draining the lagoons.

The MBBR reactor could be placed in the location of Cell 1, using the Cell 1 air supply. The process would be partitioned into two tanks, each with a capacity for 75% of the maximum design flow in accordance with Municipal Wastewater Regulation requirements. As a general concept, the process would be constructed as two 200m³ vertical wall concrete tanks, each containing 80m³ of media, with a media retention screen at the outlet.

The total MBBR air demand would be expected to be around 700 - 1300 scfm. Air flow must be maintained in each operating tank for appropriate turbulence and mixing. Air demand to the

remaining partial mix lagoons would be significantly reduced meaning that the total air flow would be within the capability of the existing blowers. An MBBR aeration system is the coarse bubble type, meaning that regular replacement of diffuser membranes is not needed.

Apart from the competitive cost, some key advantages of the MBBR option are;

- The process is in use or planned at a number of Canadian locations
- The footprint of the existing plant is reduced, making space available for future sludge geotube drying, or other uses.
- Treatment capacity can be increased gradually by adding media and new tanks can be added in the future
- System operation is simple and similar to the existing plant.

FIGURE 4-7: MBBR TANK AT RADIUM HOT SPRINGS WWTP (HIGHLIGHTED)

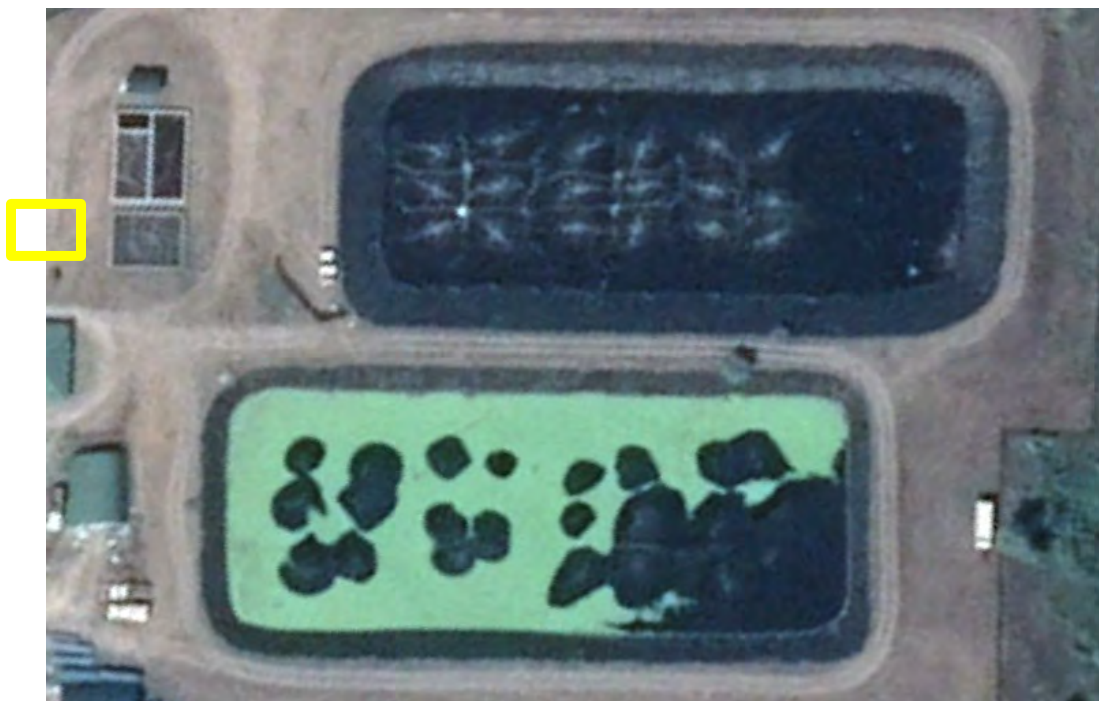
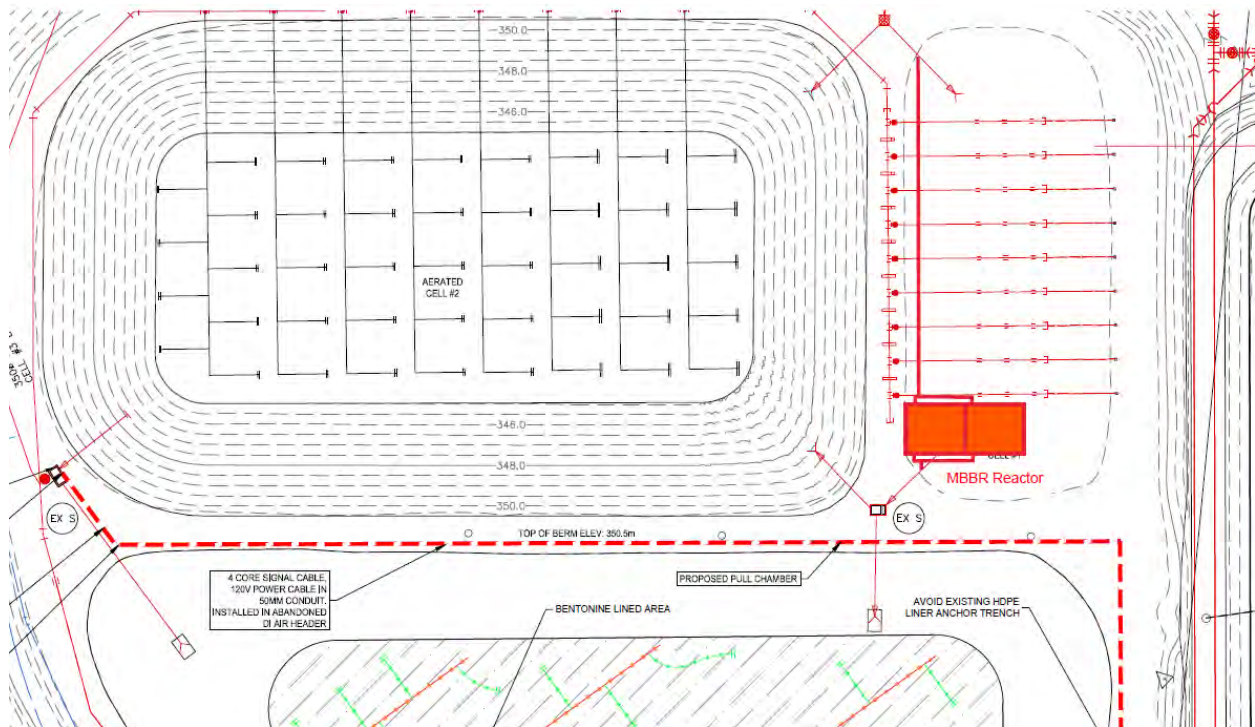


FIGURE 4-8: CONCEPTUAL LAYOUT FOR MBBR PRE-TREATMENT OR INTERMEDIATE TREATMENT



The cost for a Moving Bed Biofilm Reactor upgrade is estimated as follows;

Construct MBBR Process	\$1,500,000
- Golf course screen unit. 2 x 500 m³ concrete tank (2 x 5m deep x 10m square) - Coarse bubble aeration system 50% full of media (can be up to 70% full) - Media strainer	± 50%

It should be noted that not all of the MBBR media would be needed at first. As such, media could be added as the population expands.

The cost of decommissioning Cell 1 has not been directly considered.

5.0 Reclaimed Water Improvement Options

5.1 Reclaimed Water Quality

Additional criteria may be enforced for reclaimed water quality in the future. In particular, the following additional criteria are specified under the Municipal Wastewater Regulation for reclaimed water with 'greater exposure potential'.

Carbonaceous BOD₅ Max 10 mg/L

Total Suspended Solids Max 10 mg/L

Turbidity Average < 2 NTU, Max < 5 NTU

At present, there are no limits set for these parameters for reclaimed water at the Osoyoos WWTP. The lagoon effluent target is significantly less stringent at CBOD 45 mg/L and TSS 60 mg/L. Of course, it is worth noting that significant remediation occurs in the storage reservoirs. Samples for BOD₅ (not CBOD) and TSS were taken in 2009 and 2020. They indicated relatively good water quality. While this is a small dataset, the results are fairly consistent with the equivalent samples from the Oliver storage reservoir.

TABLE 5-1: RECLAIMED WATER SAMPLING

Date	Biochemical Oxygen Demand (mg/L)	Total Suspended Solids (mg/L)
2009-05-19	<10	7
2009-06-23	37	2
2009-07-21	<10	5
2009-08-25	12	12
2009-10-20	<10	1
2020-04-04	9	13
2020-07-14	19	<2

Assuming that the treatment lagoons are working effectively, it is expected that the reclaimed water compliance will depend greatly on the concentration of algae suspended in the stored water. Clearly algal growth is challenging to control when the water is enriched with nutrients. It is also true that the algae contribute dissolved oxygen and take up nutrients which contributes to the continued treatment of the stored water. In order to keep the reclaimed water suspended solids concentration low, it is assumed that the Town would aim for low algal counts. Many methods are available to control algae with a view to maintaining consistent reclaimed water quality. At this point algae has not been raised as a notable concern in the Osoyoos reclaimed water storage.

Filtration would be required in the event that enhanced TSS and turbidity removal is needed. Options include granular media filters, disk filters and drum filters. An indicative cost for Dynasand® filtration for a capacity of 8,000m³/d operating 24 hours per day is \$5,500,000. The concept would include a fabric partition in one of the reservoirs to provide filtered water storage. Pilot testing of the proposed filtration technology is recommended as a clarifier may be needed as pretreatment.

5.2 Future Reclaimed Water Storage and Use

5.2.1 Future Reclaimed Water Storage Requirement

Total inflow to the plant currently averages 850,000 m³/annum. Present trends (Figure 1-3) indicate that inflow may continue to increase. The Long Term Sewer System Expansion Plan Discussion Paper (2014) used population projections to estimate a 2053 annual wastewater volume of 1,200,000m³.

The Long Term Sewer System Expansion Plan Discussion Paper (2014) found that by 2053 around 145,000m³ of storage will be needed. This plan identified the Landfill and gravel pit as feasible locations for a third 150,000m³ storage reservoir. It would have an area of 3.0 – 3.5 ha depending on water depth. Two alternative sites are indicated on the drawing provided as Appendix A.

A less favored option would be to convert the wastewater treatment process to more intensive treatment and convert existing treatment Cell #3 to storage. This would only add 58,000m³ if the cell is not made deeper. This is not recommended at this stage.

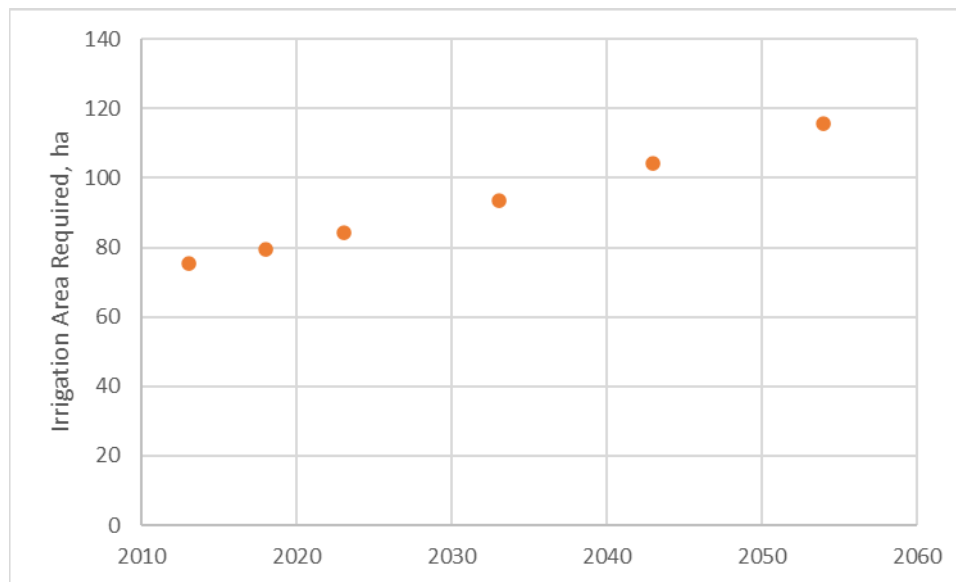
Meanwhile, the maximum use of existing storage can be achieved by;

1. Ensuring the storage reservoirs are at 100% of capacity at the start of the irrigation season.
2. Diverting flow to the rapid infiltration basins if storage is projected to be full by the start of the irrigation season.
3. Make arrangements with existing reclaimed water irrigators to increase or reduce water use based upon availability.

5.2.2 Future Irrigation Areas

The Long Term Sewer System Expansion Plan Discussion Paper (2014) estimated the total required irrigation area that will be required in 2053 as 116ha. The available area is currently 75ha, meaning an additional 41ha would be required by 2053.

FIGURE 5-1: CALCULATED IRRIGATION AREA REQUIREMENT



A number of potential future irrigation sites were identified, as follows, totaling 31ha.

Location	Area (ha)
1. Parcel South of Secondary School (Rem 1003 Plan 31183)	3.5
2. South End of Desert Park	1.5
3. East Side of Desert Park Site	4.8
4. Crown Land west of Desert Park Site	3.0
5. Airport Site	1.5
6. LAR 75 East of Landfill	16.6
Expansion Opportunities Total	30.9

A drawing showing existing and future irrigation locations is provided in Appendix A. If irrigation of the area east of the landfill is unacceptable to the OIB then 14.4ha remains available on land within the town boundary, which would be sufficient up to about 2040.

Other potential opportunities for future use reclaimed water are:

- Industrial uses, i.e. concrete manufacture, cooling, etc.
- Expanded irrigation on the Desert 18 of the golf course.
- Irrigation of vineyards north of the airport site.
- Irrigation of vineyards or other grassland type cover on completed areas of the landfill site.
- Irrigation to orchard crops.

Reclaimed water contains higher levels of metals than many fresh water supplies. Irrigation to food crops may require further treatment in order to comply with relevant criteria, such as the British Columbia Approved Water Quality Guidelines: Aquatic Life, Wildlife & Agriculture (2017). This additional treatment could be limited to the water that is used by the irrigators. The irrigators may also need to make changes to their irrigation systems in order to accept the water. The BC Ministry of Environment Reclaimed Water Guideline (2013) gives the following requirements:

- Drip and trickle irrigation systems must be used for sweet corn, berry and fruit crops.
- Vegetable crops must be irrigated with a subsurface drip irrigation system.
- Root crops may not be irrigated with reclaimed water if they are likely to be eaten raw.
- Commercially processed vegetables should be thoroughly washed and cooked following industry accepted procedures for reclaimed water applications

If the water was offered at a minimum charge, it may still be appealing to an orchard irrigator. Vineyards have a relatively low water requirement and reclaimed water use may not be cost effective.

An aquifer recharge and recovery system may be more appealing to agricultural users. Reclaimed water would be discharged to the infiltration system and then wells would pump the infiltrated water to the agricultural users. With appropriate design, filtration through the soil could eliminate any need for treatment. Managed aquifer recharge and recovery is relatively common in Europe for the production of drinking water.

6.0 Chlorination Improvement Options

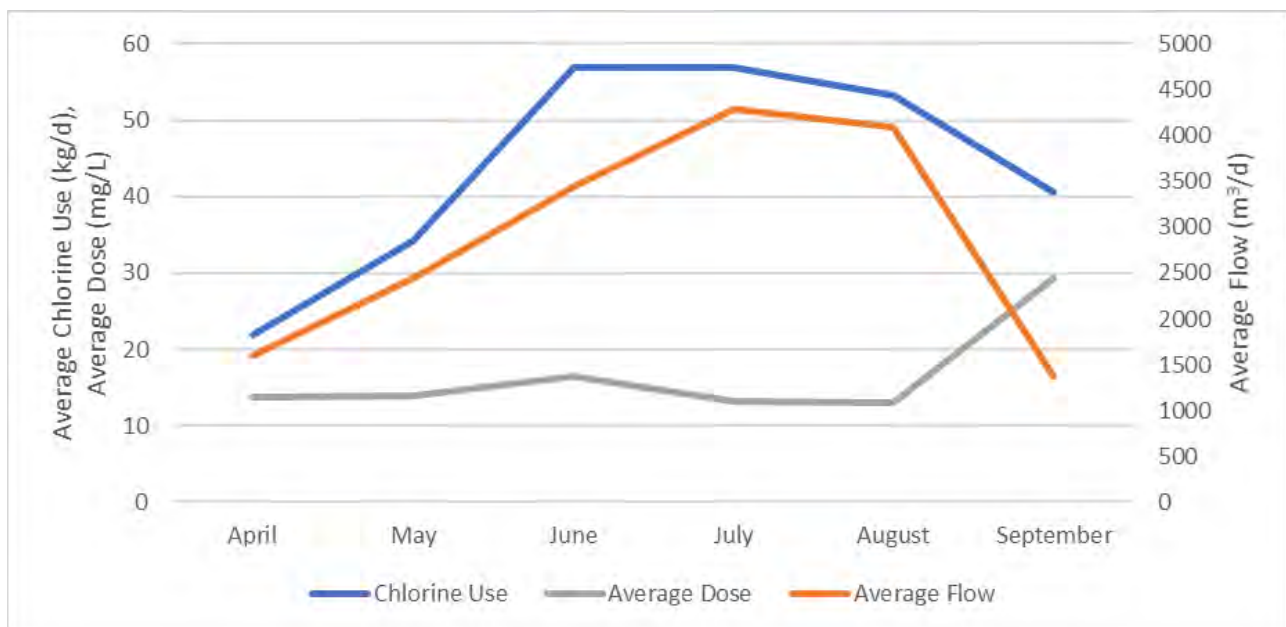
Chlorine gas is currently used for disinfection. This gas is highly toxic. As a result, the storage and handling of chlorine gas is considered to be hazardous and is subject to very stringent regulatory requirements compared to the alternatives. While relatively inexpensive, gaseous chlorine has gone out of favour in North America. The Town is interested in switching to an alternative disinfectant capable of replacing the chlorine gas.

When converting systems from chlorine gas, one of two chemicals are typically selected. These are sodium hypochlorite and calcium hypochlorite. Other chemicals, such as ozone, are effective at destroying pathogens but do not maintain a residual. As a result, MOE requires a form of chlorine to be used.

The alternatives are described in the following sections.

6.1 Current Chlorine Consumption Rates

The Operational Certificate requires a chlorine residual of 1.0 mg/L immediately prior to irrigating⁵. The chlorine consumption averaged nearly 60 kg/d in 2019 and the chlorine dose rate averaged about 15mg/L.



The impact the chlorine dose has on free chlorine residual is strongly affected by the chlorine demand of the reclaimed water. This is discussed further in Section 6.6. For the purpose of assessing alternatives, a dose of 30 mg/L has been used for estimating approximate storage

⁵ The BC Reclaimed Water Guideline specifies a disinfection residual of 0.5mg/L at the point of use.

requirements. This is based on an assumption that there may be periods with elevated demand, but they will generally be managed to keep consumption at no more than double the average value.

6.2 Sodium Hypochlorite (12%)

Sodium hypochlorite (NaOCl) is the active ingredient in household bleach. The chemical is less hazardous than chlorine gas and there are fewer training requirements and regulations for storage and handling. Sodium hypochlorite (12%) is corrosive, and particularly hazardous if splashed into the eyes. A safety shower and eyewash are required.

Sodium hypochlorite can be delivered as 1230L totes or by bulk tanker. It is normally provided in as 12% strength. There is a small cost saving for bulk tanker delivery, but there is additional cost in setting up tanks and containment.

6.2.1 System Design – Tote Delivery

A tote based system is typically operated along with a day tank fitted with level monitoring. A 5,000 litre day tank has been assumed for budgeting purposes. It is also possible to run the system directly from totes. However, there is a greater risk of leakage, and the monitoring of the connected totes is more problematic.

The operator would routinely transfer the contents of the totes to the day tank. With this arrangement, the operator has the reliable remote level monitoring and can top up levels to ensure that the storage will be sufficient until the next operator visit.

The day tank would have a containment system to protect against the escape of leakage. Totes can also be located on spill containment units. The cost estimate does not allow for individual containment pallets. It is assumed that a forklift would be used for moving the totes. The totes can also be moved around the flat floor using a pallet jack.

It is assumed that a small number of totes could be located inside the chlorine drum room with others under a porch-style structure.

Operational hours have been assumed to be an average of two visits per week with some additional hours for duties such as chemical ordering.

6.2.2 System Design – Bulk Delivery

A bulk delivery and storage system would typically comprise two or more tanks sized to take a full tanker delivery. There needs to be sufficient additional storage to allow for a delay between ordering and delivery. A total stored volume of 15m³ has been selected for this study based on a reasonable truck load. Under what we believe to be peak use conditions (future irrigated flow of 7,000 m³/d and an elevated chlorine dose of 30 mg/L) the 15m³ of storage would last for 8 days. However, under average conditions recorded over the 2019 irrigation season (3,500 m³/d and 20 mg/L) the tank contents would last for 25 days. It has been assumed that the Town could call for

restraint from irrigators in the event that a delivery was delayed and water demand was exceptionally high. In an emergency, one of the bulk tanks could also be partly filled with a mix of calcium hypochlorite pucks and water.

A delivery quantity of 10m³ has been assumed. Bulk delivery camlock connections could be located inside or outside the building. The existing building would need to be extended if it is to house the chlorine tanks. Alternatively, because the system can be drained in winter, the tanks could be installed under a porch structure with the dosing systems inside the existing building. An outdoor tank would increase chemical degradation if it was uninsulated (see section 6.2.3).

Operational hours have been assumed to be an average of one visit per week with some additional hours for duties such as chemical ordering.

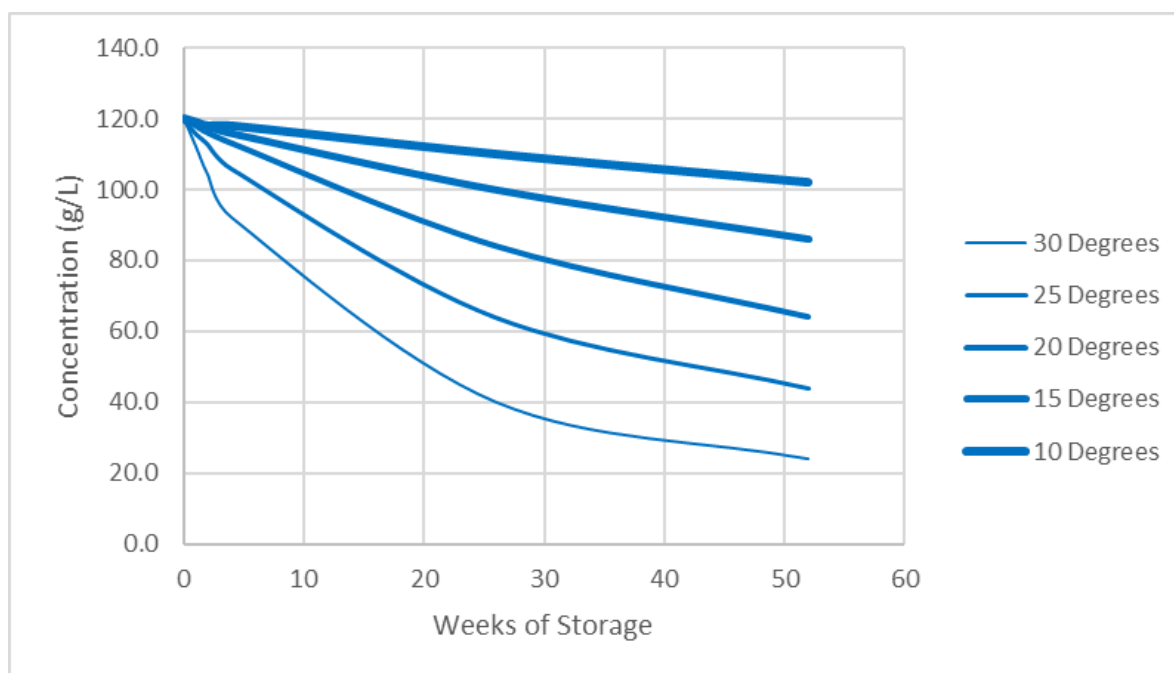
The City of Kamloops operates a similar sized reclaimed water disinfection system. They replaced their drum chlorine system with a bulk sodium hypochlorite system in 2017.

6.2.3 [Breakdown of Sodium Hypochlorite](#)

The strength of sodium hypochlorite degrades relatively quickly during storage in warm temperatures. As a result, during the peak summer months the quantity stored should be kept to a minimum and the oldest product should be used first. Air conditioning of the storage space may be cost effective.

Other important contributors to degradation rate include UV light exposure and whether the chemical was filtered at the factory. These are not expected to be issues for this application.

FIGURE 6-1: STABILITY OF 12% SODIUM HYPOCHLORITE SOLUTION VS TEMPERATURE



6.3 Sodium Hypochlorite (0.8%)

Sodium hypochlorite can be generated on-site. The raw materials for the generation of 0.8% sodium hypochlorite are salt, electricity and water. These raw materials are relatively inexpensive. The main cost with on-site systems is the installation cost. Installation costs have been presented as an annual cost so that the alternatives can be fairly compared.

The 0.8% strength solution is corrosive to a similar degree as household bleach. It is not classed as a hazardous chemical for transportation purposes. Eyewash facilities are necessary. The main hazard with bulk sodium hypochlorite systems is the release of explosive hydrogen gas. This must be vented for safety.

Degradation of the chemical, as discussed for bulk sodium hypochlorite, is not a significant issue for sodium hypochlorite generated onsite because the chemical is stored for a short period and the stability of the solution increases markedly at lower concentrations. In addition, air locks are much less of an issue due to the lower concentration and typically high fluid velocities in dosing lines.

The raw material (salt) can be stored indefinitely if kept dry. This minimizes the time required in accepting deliveries.

If the ton cylinder room were to be left unheated over winter, the system would be more challenging to winterize than the alternative options. The other systems could be winterized by pumping a glycol solution through the system. This would be damaging to the electrolyser cells. No winterization is required for brine.

The main disadvantages of sodium hypochlorite generation systems include high capital cost and the complexity of the facility, although they tend to operate reliably.

6.3.1 [System Design](#)

The chlorine ton cylinder room can be converted to house a sodium hypochlorite generation system. Based on the initial system being sized for current use with the ability to catch up after maintenance, the proposed system would be the Chlortec CT450, or an equivalent unit from another manufacturer. The CT-450 has a capacity equivalent to 204 kg Cl_2/d . We have assumed a single duty unit. Sodium hypochlorite from totes could be dosed in the event of an electrolyser failure. Dilution may be needed depending on dosing pump turndown capability.

Salt handling is a significant cost if the brine system is to be designed for bulk deliveries. A cost for a Bryneer large enough to accept blown in truck deliveries would be roughly \$150,000. It is anticipated that the Bryneer would be located outside (as is done at the Mission Hill WTP).

Because the system would only be used in the warmer months, an outdoor chlorine storage tank could also be used. The tank would need to be emptied in November.

6.4 Calcium Hypochlorite

Calcium hypochlorite is commonly used as a household and municipal pool chemical. It is the safest option in terms of material handling, particularly in the tablet / puck or granule format. Basic protective equipment is required for handling. Once it is dissolved, the hazards are equivalent to bulk sodium hypochlorite.

This chemical can be dosed automatically using a system that dissolves pucks or granules into a tank. The liquid chemical is dosed in the conventional way with a dosing pump. As the strength of the solution can vary, the system depends on feedback from a chlorine analyser for dosing accuracy.

FIGURE 6-2: CALCIUM HYPOCHLORITE MAKE-UP AND DOSING SYSTEM



FIGURE 6-3: PAILS OF CALCIUM HYPOCHLORITE TABLETS AS SUPPLIED BY BI PURE WATER.



Because calcium hypochlorite is very stable in the solid form, it can easily be stored over the winter period.

Calcium hypochlorite is 65% chlorine, so the material storage and handling quantities will be much lower than for sodium hypochlorite. Nevertheless, there will be a need to manage up to 350 kg/d of calcium hypochlorite pucks or granules by 2040.

Calcium hypochlorite is widely used at swimming pools due to its relative safety. There is a risk of fire or explosion if calcium hypochlorite is mixed with oil, although the pucks are less reactive than powder. Chlorine gas is released when hypochlorites are mixed with acids. There is no reason to expect either situation to affect this installation.

The capital cost for a calcium hypochlorite system is relatively low. The main disadvantage of calcium hypochlorite is the high cost of the chemical. There is also less accuracy in the dose achieved, but this is an issue that could be resolved. Calcium hypochlorite systems are used successfully at most swimming pools and many small water supplies.

6.4.1 System Design

An Accu-tab chlorination system has been assumed. To meet the peak flows and to include some redundancy, three Accu-tab 361000 hoppers would be used with a 500 gallon solution tank. The supplier (BI Pure Water) has assumed that the chlorinated solution would be fed to the chlorine contact basin with four pumps and the dosing flow controlled with metering valves. The proposed system has a very small footprint at 2.5m (W) x 3.0m (L) x 1.9m (H). Given that the calcium hypochlorite is 65% chlorine, there is space in the existing building for dosing and limited storage.

6.5 Comparison of Chlorine Gas Alternatives

The key advantages and disadvantages of the alternative chemicals considered are described in Table 4.1.

TABLE 6-1: ALTERNATIVE CHEMICALS – ADVANTAGES AND DISADVANTAGES

Disinfectant	Advantages	Disadvantages
Chlorine gas	- Existing system - Reliable operation - Low chemical cost - Indefinite shelf life - Town operating three sites with ton cylinders.	- Hazardous material. - Comprehensive operator training needed - Additional regulatory requirements (WSB)
Sodium hypochlorite (12%)	- Same efficacy and residual protection as chlorine gas. - Fewer training requirements than chlorine gas. - Fewer regulations than chlorine gas.	- Limited shelf-life - Chemical requires careful handling - Crystallization can clog injection ports. - Air locks from oxygen release can clog pipes, pumps, and valves (auto purge systems available). - Chlorine gas release hazard if mixed with acid (no reason to expect this to happen).
Sodium hypochlorite (0.8%)	- Same efficacy and residual protection as chlorine gas. - Fewer training requirements than chlorine gas. - Not classed as a 'hazardous' chemical. - Long shelf life for raw materials and product - Less by-product formation than bulk hypochlorite. - Less corrosive than bulk hypochlorite.	- High capital cost - Relatively complex equipment
Calcium hypochlorite	- Low capital cost - Much more stable than sodium hypochlorite allowing long-term storage. - Fewer safety regulations. - Relatively safe to handle. - Relatively little operator expertise required.	- High operating cost. - Impractical lifting and handling requirements. - Fire, explosion and chlorine gas release hazard if mixed with incompatible materials (no reason to expect this to happen). - Calcium carbonate buildup in the makeup system needs regular cleaning.

TABLE 6-2: COMPARATIVE LIFE CYCLE COST OF OPTIONS

Disinfectant	Capital Cost	Chemical / Maintenance Cost	20 Year Annualized Cost
Chlorine gas	\$0	\$17,000	\$17,000
Sodium hypochlorite (12%) - totes	\$220,000	\$41,000	\$58,000
Sodium hypochlorite (12%) - bulk	\$380,000	\$33,000	\$63,000
Sodium hypochlorite generation (0.8%)	\$1,120,000	\$39,000	\$129,000
Calcium hypochlorite	\$260,000	\$66,000	\$87,000

Comparative costs for the options have been compared on a 20 year annualized basis with a 5% rate of return. Maintenance cost has been typically set at 1.5% of capital cost. A value of 5% has been assumed for trucking 0.8% sodium hypochlorite. The chemical cost listed for sodium hypochlorite generation includes both salt and electricity.

It is suggested that the existing system is the most cost effective. There are safety systems available to quench chlorine gas if it were to escape. These could be considered as an alternative to changing the chemical dosed.

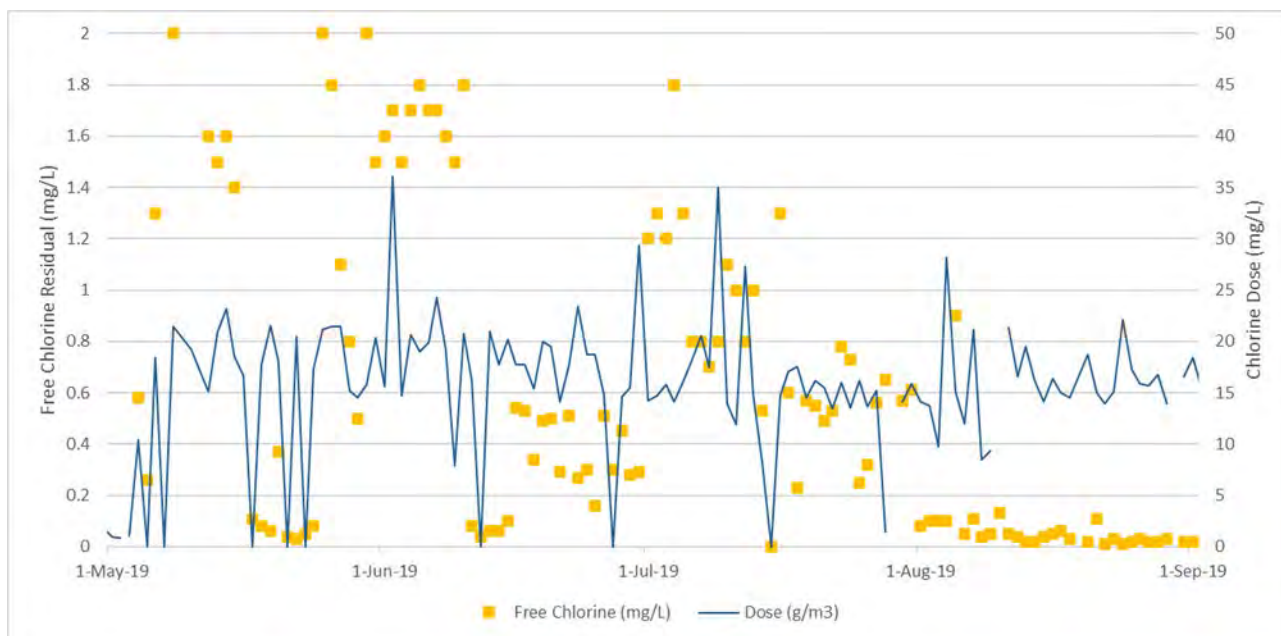
If the chemical is to be changed then sodium hypochlorite bulk deliveries are recommended based on cost and practicality.

6.6 Minimization of Chlorine Consumption

Chlorine compounds are highly reactive, which makes them effective in disinfection. Chlorine reacts with many natural compounds in water which together make up the chlorine demand of the water. These compounds include organics, iron, manganese, sulphide and ammonia. Ammonia is especially potent with the ratio of ammonia nitrogen to chlorine being around 10:1. Given that the effluent ammonia concentration varies from 0 to 35mg/L, the degradation of ammonia within the treatment system (nitrification) has a dramatic impact on chlorine use.

Figure 6-4 shows that the reclaimed water free chlorine levels do not closely relate to the dose rate and are likely to relate more to the concentration of ammonia in the water. This will vary seasonally and Reservoir 1 is likely to be different to Reservoir 2.

FIGURE 6-4: FREE CHLORINE RESIDUAL VS DOSE RATE



It is recommended that the reservoir filling and emptying schedules be planned to minimise the use of high ammonia source water. Treatment lagoon effluent will be relatively low in ammonia from mid-late summer to fall. Similarly, the ammonia concentration of the stored water should fall over that period.

Year-round lagoon-based ammonia removal is expensive to achieve because the metabolism and growth of nitrifiers reduces dramatically at low temperatures. However, relatively inexpensive lagoon retrofits are marketed for summer nitrification, consisting of high surface area nitrifier support media suspended in oxygen rich locations in the final treatment cell (e.g. EDI BioReef BioCurtain). These may serve to improve ammonia removal and extend the period over which ammonia is removed, without significant expense. This upgrade may pay for itself in reduced chlorine demand.

It may also be feasible to install nitrification modules (e.g. Bishop Water BioCord™ Reactors) in the reservoirs to reduce ammonia levels over the period the water is in storage. It would be important to ensure that having this form of treatment occurring in the storage reservoirs is consistent with the Permit conditions.

FIGURE 6-5: BISHOP WATER BIOCORD UNIT



Any system of this type will rely on the efficient removal of rag by the later stages of treatment to avoid having the rag tangled up with the media.

6.7 UV Disinfection

At some point in the future, the regulations for use of reclaimed water may become more stringent to take better account of the inactivation of protozoa. Protozoa are inactivated by chlorine so long as there is sufficient dose and contact time. However, contact times required are fairly long and it may prove more economical to use a technique just as ultraviolet disinfection to avoid large scale chlorinated water storage.

UV disinfection could also be considered for primary disinfection with the dosing of chlorine as a secondary disinfectant. UV disinfection could not replace chlorination, as a chlorine residual is required by the regulations and the permit.

A UV disinfection system for Osoyoos is most likely to be an open channel, low pressure lamp system located adjacent to the chlorine contact basin. Construction cost would be in the order of \$500,000 - \$800,000.

7.0 Recommendations

7.1 Treatment Improvements

The recommended design for the future upgrade of the Osoyoos WWTP is the MBBR process in the pre-treatment or intermediate treatment configuration. This process addition has the following technical advantages;

- Small footprint which leaves space for future upgrades.
- Ease of expansion
- Similar operation to existing plant with minimal operator intervention required
- Sludge can continue to be collected and digested in the partial mix lagoons
- Construction can be completed with little disruption to existing treatment.
- Easy to meet reliability requirements required by Municipal Wastewater Regulation.
- Easy to add more capacity by adding treatment cells.

It is proposed that the process would be split into multiple tanks to allow one tank to be taken offline during winter to reduce air and energy consumption. Multiple tanks also give flexibility to allow maintenance to be undertaken while continuing to treat incoming wastewater.

7.2 Reclaimed Water Improvements

Options for upgrades to reclaimed water quality will need to be evaluated based on the specific requirements that arise but would ideally be limited to chlorination.

Future Storage has not been proposed at this stage as

- Storage should be adequate to 2040, depending on usage patterns by irrigators.
- It would depend on the identification and procurement of suitable additional land
- It is more likely that the rapid infiltration basins would be used to dispose of excess water.

Future irrigation areas have been identified. A separate study is recommended to identify new areas and estimate costs.

7.3 Disinfection Improvements

All of the options analysed increase the cost of disinfection. However, continuing the use of chlorine increases the health and safety risk to staff and the public.

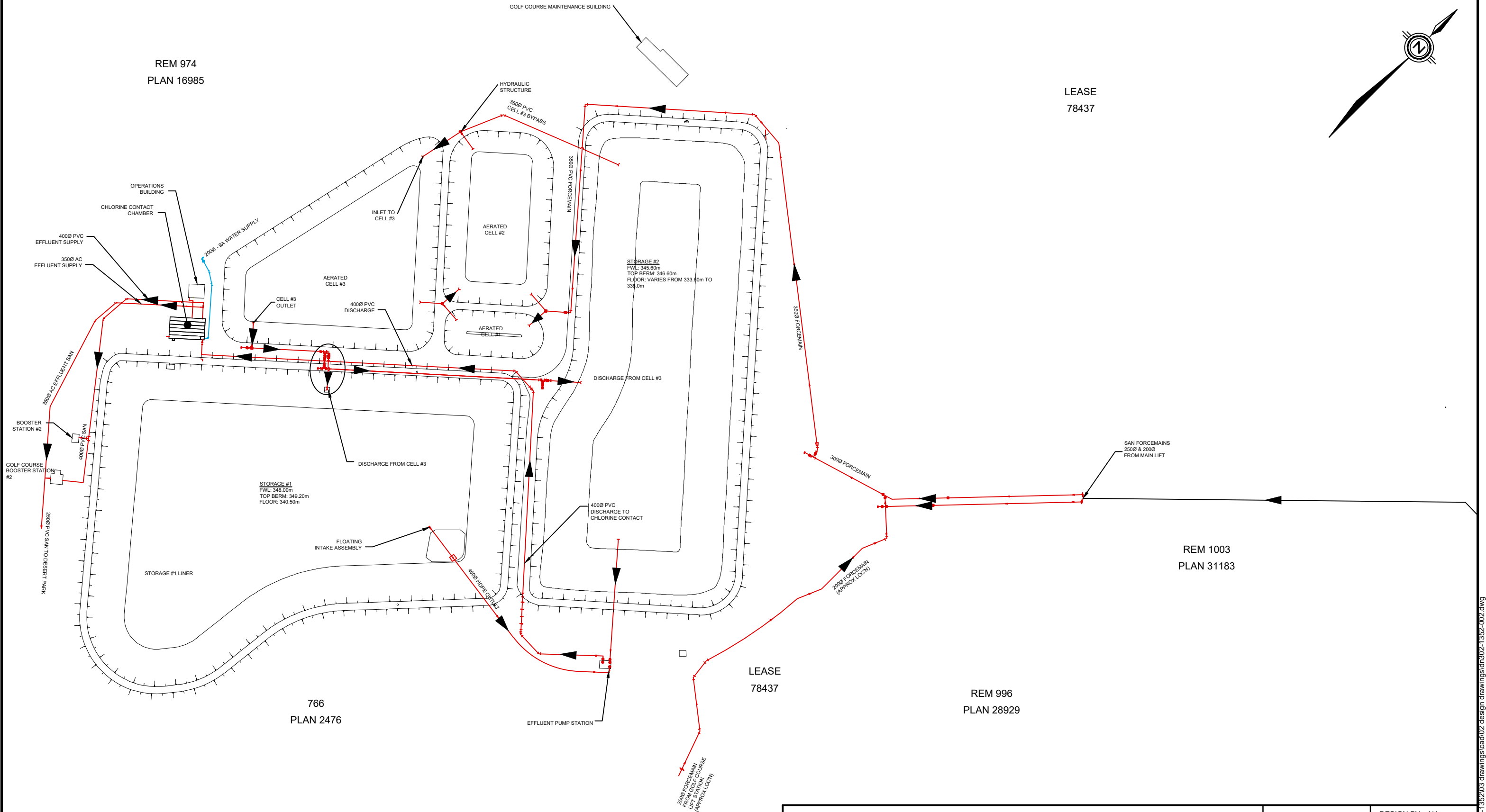
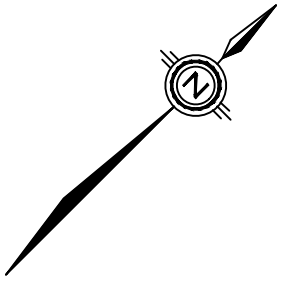
The most cost-effective replacement option would be to buy 12% sodium hypochlorite in totes or as a bulk tanker delivery. The proposed system is relatively easy to install and uses commonly available technology.

APPENDIX A

Drawings

REM 974
PLAN 16985

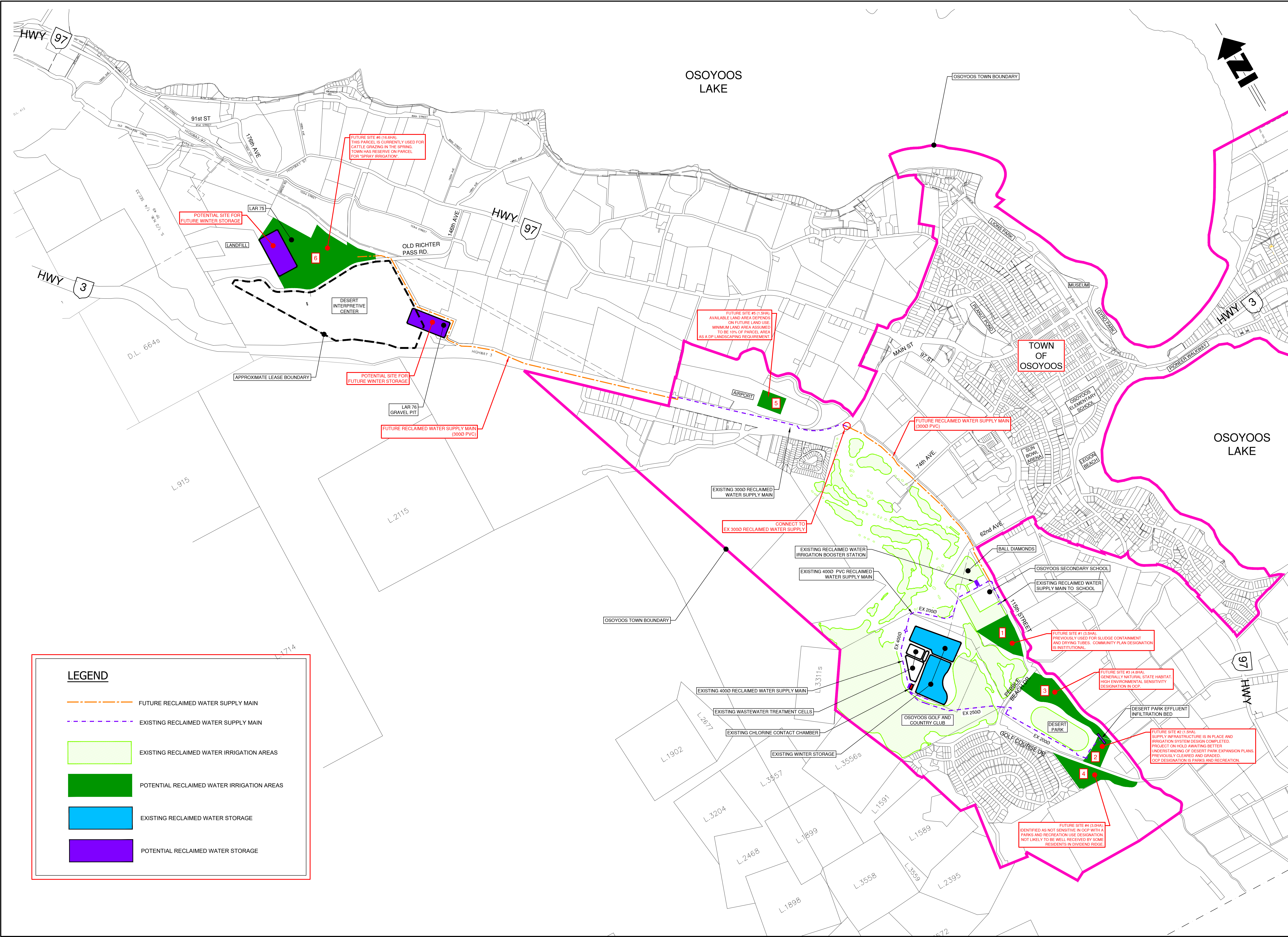
LEASE
78437



TREATMENT AND STORAGE SITE PLAN

DESIGN BY: NA	
SCALE: 1:2500 (11x17)	
DWG NO.: FIGURE	REV:
DRAWN BY: DL	
DATE: March 2018	

FILE: I:\clients\300-399\302\302-1352\03 drawings\cad\02 design drawings\dm302-1352-002.dwg



No.	DATE	DESCRIPTION	BY	APPD
ISSUES / REVISIONS				
CONSULTANT SEAL				

201 - 2079 Falcon Road ■ Kamloops BC ■ V2C 4J2
tel 250.828.0881 ■ fax 250.828.0717
info@TRUE.bc.ca

TOWN OF OSOYOOS

RECLAIMED WATER SYSTEM

STORAGE AND IRRIGATION EXPANSION PLAN

SCALE Scale 1:10,000	
DESIGN BY RW	SHEET OF REVISION
DRAWN BY DL	
DATE MAR 2021	
PROJECT REFERENCE No. 302-1474	
DRAWING No.	
302-1354-05	

FILE: c:\pwworking\302-1354\05.dwg 3/2/2021 1:04:05 PM

APPENDIX B

Effluent Monitoring Data

Town of Osoyoos
WWTP Compliance Testing - Comparison with Operational Certificate Criteria

Location	Headworks					Outlet of Cell #3												Chlorine Contact Chamber												Sludge	Ground and Surface	
EMS site number	E230037					E105001												E207444												E230278		
Parameter	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Biochemical Oxygen Demand	Total suspended solids	Carbonaceous biochemical oxygen demand	Carbonaceous biochemical oxygen demand	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Total dissolved phosphorus as P, mg/L	Total nitrogen, mg/L	Ammonia nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Organic nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Faecal coliforms, MPN/100 ml	Total coliforms, MPN/100 ml	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus, mg/L	Total nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Chlorine residual, mg/L	Chloride, mg/L	Sodium, mg/L	Sulphate, mg/L	See 5.6.2 of Certificate	See 5.7 of Certificate
Frequency (Operational Certificate)	semi-annual	semi-annual	Not required	Not required	semi-annual	monthly	Not required	monthly	semi-annual	semi-annual	semi-annual	semi-annual	semi-annual	semi-annual	semi-annual	semi-annual	Use to calculate organic N	monthly	monthly	Not required	Not required	monthly	monthly	Not required	Not required	Not required	weekly	monthly	monthly	monthly	Annual	
Sample type	Grab	Grab			Grab	Grab		Grab	Grab	Grab	Grab	Grab	Grab	Grab	Grab	Grab		Grab	Grab			Grab	Grab				On-site	grab	grab	grab	Grab	Grab
Limit						<45		<60										<2.2 MPN														
Date																																
2009-01-20							42	16																								
2009-02-17							37	16																								
2009-03-17							13	13																								
2009-04-21							44	26	5.09																							
2009-05-19							64	33												<10	7											
2009-06-23							20	38												37	2											
2009-07-21							37	31												<10	5											
2009-08-25							<10	10												12	12											
2009-09-29	6.42								7													4.65										
2009-10-20							<10	6												<10	1											
2009-11-17							31	10																								
2009-12-22							20	18																								
2010-01-19							40	30	4.55	4.16	4.2	32	26.5	0.52	<0.01	4.9	31.4															
2010-02-16	6.22	4.37	180				19	20																								
2010-03-16							19	17																								
2010-04-20							14	12										< 3.0	< 3.0			5.34	20	1.31	<0.01	18.7						
2010-05-18							19	19										< 3.0	< 3.0			4.8	29.1	0.13	0.09	28.9						
2010-06-15							73	49										< 3.0	460			6.03	17	0.71	0.75	15.5						
2010-07-13									4.93	2.47	4.43	20.9	3.17	8.92	4.47	4.32	7.49															
2010-07-20																		< 3.0	7.3			4.69	13.8	0.55	0.48	12.7						
2010-08-17	5.68	3.27	190				15	35										2400	15000			5.53	24.7	0.21	<0.01	24.5						
2010-09-21							<10	35										< 3.0	91			7.14	22.6	10.1	0.59	11.9						
2010-11-16							13	12																								
2010-12-21							37	19																								
2011-01-18							20	16	3.5	3.26	3.09	29.8	21.8	1.85	0.13	6	27.8															
2011-02-15	5.81	3.82	220				66	105																								
2011-03-15							31	22																								
2011-04-19							26	17										< 3.0	< 3.0			4.86	26	0.82	0.07	25.1						
2011-05-17							12	10										< 3.0	< 3.0			2.77	24.1	0.47	0.12	23.6						
2011-05-25																																
2011-06-14							15	15						2.66	1.08			21	460			3.05	25.1	0.66	0.39	24.1						
2011-07-19							< 10	8										< 3.0	< 3.0													
2011-08-09																						5.7	21.4	2.63	0.57	18.2						
2011-08-16							< 10	3										43	>1100			3.27	18.1	11.1	0.71	6.37						
2011-08-24																																
2011-09-20							< 10	4										15	230			7.24	20.5	2.53	1.02	17						
2011-10-18							< 10	3																								
2011-10-25																		< 3.0	3.6			6.72	18.5	0.692	0.16	17.6						
2011-11-15							14	10																								
2011-12-20							22	17																								
2012-01-17							18	16	4.22	3.99	3.91	34.8	28.6	1.02	<0.010	5.1	33.7															
2012-02-14	3.85	3.74	140				17	13																								
2012-03-13							16	3																								
2012-04-17							22	15										< 3.0	< 3.0			5.7	19.5	2.95	<0.01	16.5						
2012-05-15							< 10	15										< 3.0	36			2.08	19.8	<0.01	0.034	19.8						

[illegible]

[illegible]

[illegible]

Location	Headworks					Outlet of Cell #3												Chlorine Contact Chamber												Sludge	Ground and Surface	
EMS site number	E230037					E105001												E207444												E230278		
Parameter	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Biochemical Oxygen Demand	Total suspended solids	Carbonaceous biochemical oxygen demand	Carbonaceous biochemical oxygen demand	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Total dissolved phosphorus as P, mg/L	Total nitrogen, mg/L	Ammonia nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Organic nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Faecal coliforms, MPN/100 ml	Total coliforms, MPN/100 ml	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus, mg/L	Total nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Chlorine residual, mg/L	Chloride, mg/L	Sodium, mg/L	Sulphate, mg/L	See 5.6.2 of Certificate	See 5.7 of Certificate
2017-09-12							54.2	30										75	>1100			9.63	24.1	<0.010	<0.010	24.1						
2017-09-19																		<3.0	<3.0							1.91						
2017-09-26																		<3.0	<3.0													
2017-10-03																		9.1	75													
2017-10-10							68.6	22										36	2400			6.49	22.7	9.06	2.31	11.3						
2017-10-17																		93	460													
2017-11-07							35.3	16																								
2018-02-06	5.29	3.23	205																													
2018-04-03																		<3.0	<3.0								0.1					
2018-04-10																		<3.0	<3.0													
2018-04-17																		<3.0	<3.0													
2018-04-24																		<3.0	<3.0													
2018-05-01																		<3.0	<3.0									0.3				
2018-05-08																		<3.0	<3.0			3.95	16.9	7.79	0.324	8.82						
2018-05-15																		750	24000													
2018-05-24																		23	460													
2018-05-29																		<3.0	<3.6													
2018-06-06																		<3.0	15			4.7	26.9	0.02	<0.010	26.9	0.3					
2018-06-12																		<3.0	4600													
2018-06-19																		<3.0	<3.0													
2018-06-26																		<3.0	<3.0													
2018-07-10																		<3.0	1100			4.33	30	0.183	0.39	29.4						
2018-07-17																		2400	11000													
2018-07-25																		430	24000													
2018-07-31																		750	24000													
2018-08-07																		<3.0	<3.0													
2018-08-14	9.14	3.85	301															<3.0	23			4.93	16.7	8.18	1.08	7.42						
2018-08-21																		<3.0	<3.0													
2018-08-28																		<3.0	<3.0													
2018-09-04																		<3.0	<3.0													
2018-09-11							42.8	37										<3.0	<3.0			7.1	26.5	14.8	1.42	10.3						
2018-09-18																		<3.0	<3.0													
2018-09-25																		<3.0	<3.0													
2018-10-02																		<3.0	<3.0													
2018-10-09							81.5	13.5										<3.0	<3.0			6.13	29.6	19.8	0.348	9.39						
2018-11-05							28.1	6.9																								
2018-12-11							60.3	15																								
2019-01-08							21.1	16.7	4.18	3.73	3.93	33.5	20.6	5.77	0.785	6.3	26.9															
2019-02-05	6.77	2.24	255				30	27.5																								
2019-03-05							21.6	28																								
2019-04-01																												0.14				
2019-04-08																												0.33				
2019-04-16							42.9	27																								
2019-04-19																													0.37			
2019-04-20																												0.34				
2019-04-23																		<3.0	<3.0			5.14	32	2.32	0.47	29.2						
2019-04-27																												0.54				
2019-04-30																		<3.0	<3.0			Full	Suite	2.24	0.489							
2019-05-04																												0.58				
2019-05-07							26.6	36.5										<3.0	<3.0			4.86	28.6	0.71	0.09	27.8						
2019-05-11																												2.1				
2019-05-14																		<3.0	<3.0													
2019-05-18																												0.08				
2019-05-21																		91	46000													
2019-05-25																												2				
2019-05-28																		3.6	2400											</		

Location	Headworks					Outlet of Cell #3												Chlorine Contact Chamber												Sludge	Ground and Surface	
EMS site number	E230037					E105001												E207444												E230278		
Parameter	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Biochemical Oxygen Demand	Total suspended solids	Carbonaceous biochemical oxygen demand	Carbonaceous biochemical oxygen demand	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus as P, mg/L	Ortho phosphorus as P, mg/L	Total dissolved phosphorus as P, mg/L	Total nitrogen, mg/L	Ammonia nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Organic nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Faecal coliforms, MPN/100 ml	Total coliforms, MPN/100 ml	Biochemical Oxygen Demand	Total suspended solids	Total phosphorus, mg/L	Total nitrogen, mg/L	Nitrate nitrogen, mg/L	Nitrite nitrogen, mg/L	Total Kjeldahl Nitrogen, mg/L	Chlorine residual, mg/L	Chloride, mg/L	Sodium, mg/L	Sulphate, mg/L	See 5.6.2 of Certificate	See 5.7 of Certificate
2019-06-04							13.1	22										3.6	23													
2019-06-08																											1.6					
2019-06-11																		<3.0				4.02	29.8	26.1	0.156	3.54						
2019-06-15																											0.1					
2019-06-17																		<3.0	91													
2019-06-22																											0.51					
2019-06-25																		<3.0	91													
2019-06-29																											0.28					
2019-07-02									6.65	3.95	6.03	40.7	0.142	32	0.172	8.338	8.48	<3.0	<3.0			6.65	40.7									
2019-07-06																											0.8					
2019-07-09							26.2	47																								
2019-07-13																											1					
2019-07-16																		<3.0	<3.0			6.03	26.7	5.37	0.407	21						
2019-07-20																											0.55					
2019-07-23																		<3.0	<3.0													
2019-07-27																											0.56					
2019-07-30																		<3.0	<3.0													
2019-08-03																											0.1					
2019-08-06																		<3.0	93													
2019-08-10																											0.13					
2019-08-13							28	58										9.1	1100													
2019-08-17																											0.06					
2019-08-20																		<3.0	93			2.95	31.8	28.3	<0.010	3.46						
2019-08-24																											0.01					
2019-08-26	4.55	2.05	121																													
2019-08-27																		<3.0	23													
2019-08-31																											0.02					
2019-09-03																		<3.0	7.3													
2019-09-07																											0.1					
2019-09-10							13.8	30										23	150													
2019-09-14																											0.05					
2019-09-21																											0.08					
2019-09-24																		<3.0	<3.0							13.6						
2019-09-28																											0.06					
2019-09-30																		<3.0	<3.0													
2019-10-05																											0.09					
2019-10-08							10.6	33										<3.0	<3.0			6.21	24.6	1.14	0.241	23.3						
2019-10-16																		6.3	112													
2019-10-20																																
2019-11-05							22.5	31.3																								
2019-12-10							45.7	26.8																								
2020-01-07						missing	25.5	29																								
2020-02-18						missing	69.1																									
2020-02-19						missing	34.2	21.3																								
2020-03-03	6.28	3.59	288																													
2020-03-10						missing	73.3																									
2020-03-17									4.94	3.79	4.29	40.2	28	<0.010	0.014	12.1	40.2															
2020-03-24																		<1	<1													
2020-03-31																		<1	<1													
2020-04-07						missing	19.5	22										<1	1													
2020-04-14						missing	8.6	12.6										<1	<1	8.6	12.6	6.46	22.3	18.9	<0.010	3.49						
2020-04-21																		<1	<1													
2020-04-28			226	164														<1	<1													
2020-05-05						missing	26.8	33.7										<1	<1													
2020-05-11																																
2020-05-12																		<1	<1													
2020-05-19																		<1	<1			5.65	33.3	0.806	0.3	32.2						



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Capacity Assessment Sanitary Sewer Infrastructure

May 2014
Final Report
KWL Project No. 3233.001

Prepared for:
Town of Osoyoos





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Executive Summary



Executive Summary

Kerr Wood Leidal Associates Ltd. (KWL) was retained to perform a comprehensive capacity assessment analysis to the Town of Osoyoos' Sanitary Sewer System. Some important questions (and KWL's respective responses) regarding the current and future elements of the system has been brought forward from residents, council, and staff that are to be answered in this assessment. These include:

1. Can the existing transmission and treatment system handle the current seasonal loads as well as take on the development currently being constructed?

KWL found that the existing transmission system does have some deficiencies. These were found to be in one sewer main in the Industrial Park / Airport trunk sewer, and in the Main Lift Station. The future also includes an upgrade to the 250 mm dia. sewers along Cottonwood Dr.

KWL understands that the existing effluent disposal system needs an expansion to the reclaimed water system now and in the future. Since demand fluctuates from year to year, consideration for another means, such as dispersal to ground to relieve excess reclaimed water, can help buffer fluctuations with respect to winter effluent storage. Further investigations should be completed to assess the biological treatment capacity of the system to determine if additional related improvements are required.

2. What is the capacity of the system to service future growth of the Town?

The system does have capacity to accommodate anticipated future growth over the next 20 years. There are current upgrades necessary as well as ones that will be triggered in the future depending on the rate of future growth.

3. What is the anticipated growth for Osoyoos as well as the surrounding areas within the Osoyoos OIB and the Regional District of Okanagan-Similkameen (RDOS)?

The anticipated growth for Osoyoos, OIB, and RDOS are included with this report. The population projections section addresses this. Correspondence with OIB and RDOS could only yield short term growth projections.

4. What improvements in the future are triggered by new developments and growth, inside and outside the town boundary, to connect to the sanitary sewer system?

As detailed in the following report, improvements to the 250mm diameter Cottonwood Dr. sewers and the chlorine contact chamber will be necessary in the future due to growth inside and outside the Town of Osoyoos. Upgrades may also be required to the treatment plant depending on the biological treatment capacity of the system.

KWL suggests that the reclaimed water discharge system will be expanded to prevent surplus storage and an appropriate alternative discharge to handle/buffer seasonal flows be established. In order to expand the reclaimed water discharge system and/or the lagoon system, KWL suggests additional land acquisition by the Town is completed as necessary.



5. What are the costs of these necessary improvements to the system and how does Osoyoos recover those costs from the developers?

The estimates of probable costs are detailed in Tables 3-4 and 4-6 in the report. The total amount for all improvements between now and the next 20 years is estimated at \$11,613,000.

Note: This estimate does not include expansion to the reclaimed water system which also needs to be completed. These costs are too variable to estimate at this stage. KWL notes that a minimum of 15ha will be required over the next 10 years. Therefore these costs cannot be included with this report.

6. What is “appropriate per equivalent dwelling unit” charge for service to lands outside the municipal boundary?

Based on the assumptions made in Table 5-1, a cost-sharing rate of \$17,300 per equivalent du can be applied to all new developments connecting to the Town’s sanitary sewer system in the next 20 years. This rate does not include a contribution/investment from the residents for these assets and should therefore not be used directly as a sewer DCC rate.

The analysis was split into two categories:

1. Trunk sewer capacity assessment; and
2. Wastewater treatment assessment.

Before the two assessments could effectively be accomplished, a population and growth projection of the Town and surrounding areas was also completed. These projections were necessary in order to establish a timeline for requisite improvements to the systems as identified in the report.

Population Projections

The population projections looked at both the in-town and out-of-town factors of all residential, commercial, institutional and seasonal residents of Osoyoos. Looking at trends and the Town’s development infill appraisal, KWL was able to establish the effective population for the 0, 5, 10, 15, and 20 year horizons. Table 2-4 and Table 2-5 contained in Section 2 summarize our population projections.

With these populations, KWL was able to apply their relative demand on the sanitary sewer systems to determine what improvements are necessary now and triggered in the future.

Trunk Sewer Capacity Assessment

With the population information and record drawings obtained from Osoyoos and TRUE, KWL developed a trunk sewer model for both existing and future development scenarios (Figure 3-1 and 3-2).

A computer desktop spreadsheet model was developed for each trunk sewer system, which calculates peak flows within each of the subject sewers and compares the peak flows with the calculated pipe capacities.

The spreadsheet models for existing and long range future development peak wastewater flows (PWWF) were established with the above information and were compared to the existing infrastructure capacities. A number of capital improvements were identified as a result of these as identified in Table 3-3. A summary of the related trunk sewer capital improvement costs and their respective timing is illustrated in Table 3-4.



Wastewater Treatment Capacity Assessment

Utilizing the population projections and information provided by TRUE and Osoyoos as well as establishing assumptions, an assessment of the capacity of the Osoyoos wastewater treatment plant was completed. Sufficient information regarding the design and operation of the aeration system was not available at the time of writing of this report for a complete assessment of the biological treatment capacity of the three aerated lagoon cells. KWL was able to assess the hydraulic capacity of the existing treatment plant by calculating the existing Hydraulic and Solid Retention Times (HRT and SRT) as identified in Table 4-1. It was noted that other than the depths of the storage/aeration cells, the operating characteristics of the lagoon system are special. The projections of wastewater flow and loading for the town based on in-town growth is illustrated in Table 4-2.

All operational, processes, and capacities of the treatment facilities were assessed including the discharge methods. As noted in Table 4-3, there are a number of park and golf club areas that are used to discharge the reclaimed water.

Surplus Wastewater

Based on current conditions and future development, the current operating processes will yield a problematic surplus of treated wastewater if there is no additional irrigation to eliminate the surplus during the summer irrigation season. Cooler summer weather will result in an increased demand for storage with a lower reclaimed water irrigation rate. Table 4-5 shows how this surplus will happen in the next 20 years in hot/dry, average, and cold/wet seasons. Regardless of the season, an annual balance of the flows will not be achieved under the current conditions without accumulation in the effluent storage basins.

Dealing with Surplus Wastewater

Based on the capacity assessment presented in the preceding sections, over the next 20 years there will be an increasing quantity of effluent that must be treated and disposed.

Assuming that the current strategy of winter effluent storage and summer reclamation of effluent for irrigation is continued, there will be a need to build additional storage cells and/or find additional land area for irrigation. It should be noted that KWL has not included an estimate of probable cost for expansion(s) to the reclaimed water distribution system. These costs are considered to be too variable to estimate for this assignment. Osoyoos will need to consider this aspect in addition to the cost estimates presented in this report.

Effluent disposal to water may be an alternative to irrigation of reclaimed water. However, this would require a significant upgrade of the treatment plant from lagoons to a tertiary treatment plant, as well as construction of a new outfall from the treatment plant to Osoyoos Lake. Considering the location of the existing lagoon system in relation to the lake and the size of the Town, this method is not feasible.

If additional land area for irrigation cannot be obtained, KWL suggests Osoyoos to expand the effluent storage lagoon as detailed and estimated in Table 4-6. This work for the capital improvement project includes the construction of a new lined effluent storage cell for an additional volume of 300,000 m³, piping and appurtenances to connect the new cell to the effluent pumping system, and expansion of the chlorine contact tank. For planning purposes this upgrade has been assumed to occur in Year 10 and is based on having some 15 ha of additional land area for irrigation before Year 10.

Cost Sharing Opinion

To fund the proposed upgrades, a per-du (dwelling unit) cost sharing approach can be used. Based on the assumptions made in Table 5-1, a cost-sharing rate of \$17,300 per equivalent du can be applied to all new developments connecting to the Town's sanitary sewer system in the next 20 years, regardless of their locations



being inside or outside of Town boundaries. For the new developments in the Osoyoos Indian Band (OIB), an additional cost-sharing rate of \$1,360 per equivalent du should be applied.

It should be noted that the \$17,300 per equivalent du assumes that all costs will be obtained directly from development. It does not include a suitable contribution/investment from the residents for these assets.



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Section 1

Introduction



1. Introduction

1.1 General

Kerr Wood Leidal Associates Ltd. (KWL) was retained by the Town of Osoyoos (Osoyoos) to provide a sanitary sewer system capacity study to assist the town with a planning objective to manage its sewer system assets over the next 20 years. In general, the following tasks have been completed:

1. Projecting future growth;
2. Assessment of the current condition of the sanitary sewer distribution and treatment systems;
3. Develop improvement requirements to both systems for the next 20 years;
4. High level estimates of probable costs for these improvements; and
5. Policy recommendations on future development applications.

1.2 Background Information

Existing System

Growth is an inevitable and common process for municipalities within BC and the Town of Osoyoos. Given its seasonal popularity, as well as growth in the community in general, Osoyoos has been observing consistent growth for many decades. Osoyoos is in a position to have a sewer system assessment completed to address current and future capacities as growth to the sanitary sewer service area, over the past two decades, is being requested outside its boundary area.

The 2012 Annual Report on the Wastewater Treatment and Reclaimed Water Irrigation Systems shows the buffering capacity on the storage cells at the treatment plant is reducing every year to the point where concern on its ability to service the system is being compromised.

Osoyoos recognizes that a significant planning exercise, such as this one, is necessary to alleviate this issue as well as foresee additional ones in the immediate and long-term future.

Need for a Capacity Assessment

The 2012 Wastewater Annual Report leads to believe that there is unease in accepting additional development applications within and outside the town's boundary. Some questions that KWL will assist to address include:

- Can the existing transmission and treatment system handle the current seasonal loads as well as take on the development currently being planned/constructed?
- What are the 'bottle-necks' or 'pinch-points' of the existing and future systems and how critical are each?
- How will the system handle future growth?
- What is the anticipated growth for Osoyoos as well as the surrounding areas within the OIB and the RDOS?
- What improvements in the future are triggered by new developments, inside and outside the Town boundary, to connect to the sanitary sewer system?



- What are the costs of these necessary improvements to the system and how does Osoyoos recover those costs from the developers?

KWL understands that both Osoyoos and TRUE have considerable knowledge of the system and insight to the above issues. TRUE has been the Town's Consulting Engineer for many years and KWL has been able to obtain enough information to allow us to complete this assessment.



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Section 2

Osoyoos Population Projections



2. Osoyoos Population Projections

2.1 Residential Population (2011)

Statistics Canada's 2011 Census data shows that the Town of Osoyoos had 4,845 residents in 2011, with a total of 2,966 private dwellings of which 2,261 dwellings were occupied by usual residents.

Using 2011 Census and GIS data, KWL used a spatial algorithm to identify and allocate the Town's Census population to residential lots based on BC Assessment Authority (BCAA) 'actual use codes', which describes the land use at each particular lot. Census populations were distributed to single and two-family lots based on the number of dwelling units, and to multi-family lots based on lot area. It is noted that residential dwelling units located on agricultural land also received census population even though these lots typically are not serviced by the Town's sewer system.

The census population distribution reveals that the existing single-family household occupancy ratio is averaged to be 2.3 people/dwelling unit (pop/du), while the multi-family household occupancy ratio is calculated to be 1.7 pop/du, with an overall Town-wide occupancy ratio of 2.1 pop/unit. Of the total 4,845 residents in the Town, 4,776 residents are serviced by the Town's sanitary sewer system.

Outside of the Town's boundaries, estimated residential populations for the developments were also assessed based on aerial photography provided by the Town. Serviced residential properties outside the Town's municipal boundaries mainly include Northwest Sector Area of Electoral Area A (137 lots) of RDOS, and Southeast Sector Area of Electoral Area A (5 lots) of RDOS. For each residential lot identified, 2.1 pop/lot was allocated.

2.2 Industrial, Commercial, Institutional (ICI) Equivalent Population (2011)

Industrial-Commercial-Institutional (ICI) lots were identified based on BCAA use codes and populated using a population equivalent (PE) per hectare approach. Table 2-1 shows ICI population densities for the three categories. These equivalents are based on our experience with flow monitoring data in other studies, and represent typical values for the respective ICI category.

Table 2-1: ICI Population Densities

Land Use	Density (PE/ha)
Industrial	25
Commercial	60
Institutional	50

The same density criteria were also applied for the U.S.A-Canada border facilities, which is of commercial use in nature.



2.3 Seasonal Population

The tourism in Osoyoos is primarily limited to summer months, which increases temporary populations in the Town significantly during July and August of each year. According to an interview conducted by Osoyoos Times¹ to business owners of hotels, motels and campgrounds in Osoyoos, the average occupancy of these tourism-related accommodations is at about 80 – 90% full capacity in summer months.

To estimate these seasonal people, an average density of 62 units/ha (equivalent to C4/C5 zoning density for campsites) with 2.1 pop/unit (the average household size) was assumed for these tourism-related accommodation facilities unless details are provided in the Town's zoning bylaw (e.g., CD5). Using 80% occupancy, the maximum people in these facilities were then estimated.

Considering the nature of commercial use for motels and hotels, an average occupancy of 15% was assumed during their winter operations. These winter visitors are treated as ICI equivalents rather than seasonal people.

Seasonal people were also calculated for the outside-of-boundary users, which include:

- OIB located north of Cottonwood Drive (220 +/- campsites, Spirit Ridge Vineyard Resort & Spa, golf course and affiliated amenities);
- Haynes Point Provincial Park (41 campsites); and
- Cabana Beach Campground & RV Park; and Tamri Motel & Campground at Southeast Sector Area of Electoral Area A of RDOS.

2.4 Existing Total Population

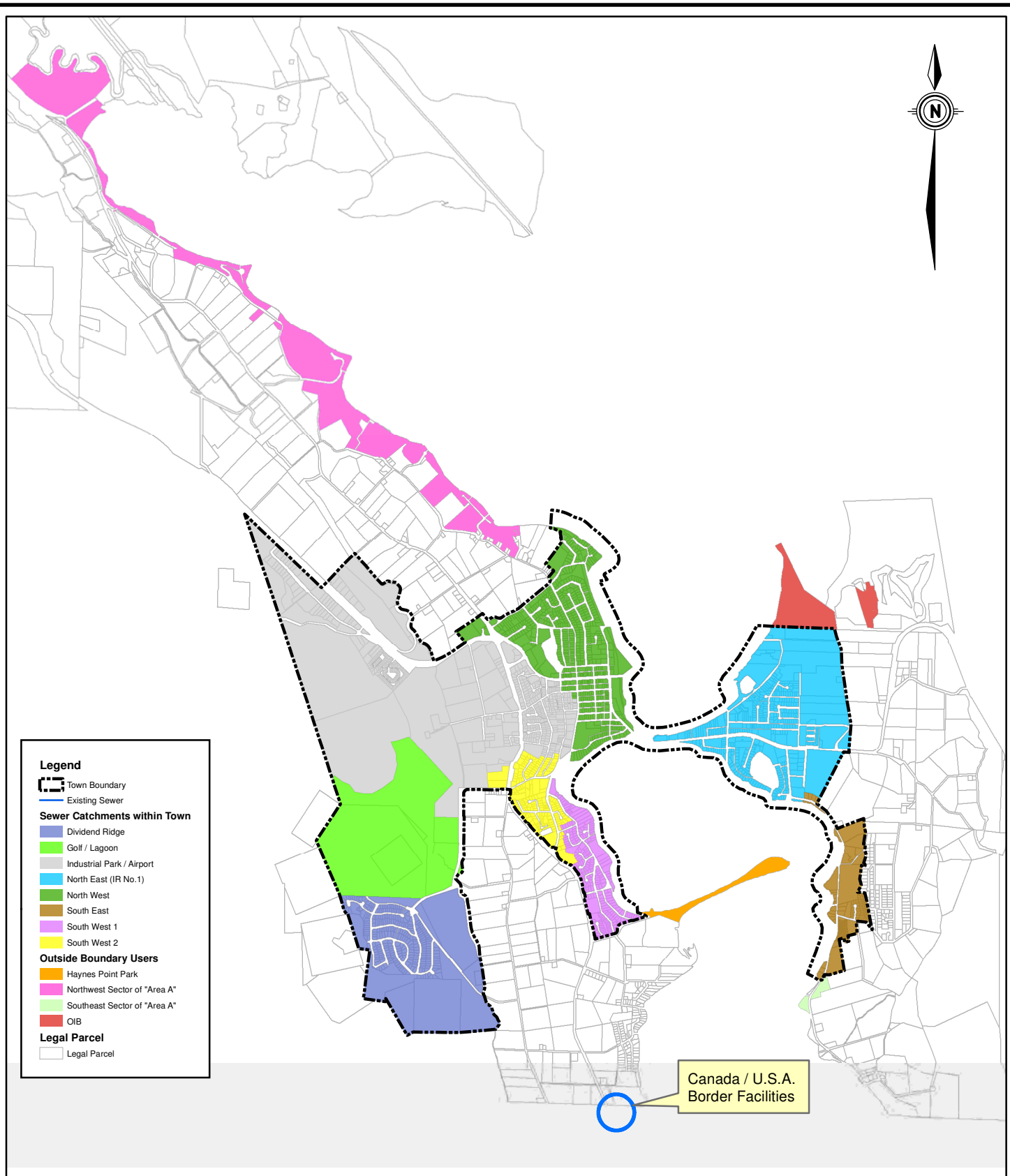
Based on 2011 Census population distribution and estimation of ICI PE and seasonal people, the Town's existing serviced populations are summarized in Table 2-2 below. Only these serviced populations were used to calculate per-capita sewer loading rate.

Table 2-2: Existing Serviced Population Summary

Type	In-Town Population	Outside-Town Population	Total Population
Residential	4,776	298	5,074
Industrial-Commercial-Institutional	2,282	140	2,422
Seasonal	3,468	870	4,338
Total	10,526	1,308	11,834

Figure 2-1 displays the catchment areas by the Town's sanitary sewer system. Table 2-3 shows the breakdowns for the existing serviced populations in each catchment area.

¹ Tourist Numbers Slipping in Osoyoos, by admin, July 29, 2008. Osoyoos Times.



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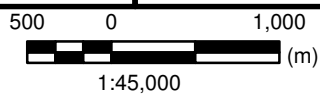
Town of Osoyoos Sewer System Capacity Assessment

Project No.

3233.001

Date

January 2014



Sanitary Sewer Catchment Areas

Figure 2-1



Table 2-3: Existing Populations by Sewer Catchments

Sewer Catchment	Within Town Boundary		Outside Town Boundary		Total		Note
	Total PE	Serviced Area (ha)	Total PE	Serviced Area (ha)	Total PE	Serviced Area (ha)	
Dividend Ridge	507	18	0	0	507	18	
Golf / Lagoon	105	3	0	0	105	3	
North West	2,539	63	288	21	2,827	83	North West Sewer Extension for 137 lots
Industrial Park / Airport	2,038	52	0	0	2,038	52	
South East	1,467	16	138	2	1,605	18	Electoral Area 'A' at South East
South West 1	682	23	127	4	809	27	Haynes Point Provincial Park; Canada / USA border facilities
South West 2	750	19	0	0	750	19	
North East (IR No.1)	2,439	48	755	13	3,194	61	OIB north of Cottonwood
Grand Total	10,526	241	1,308	40	11,834	281	

2.5 Future Population Forecast

Growth within the Town Boundary

The Town has experienced fast growth since the 1980s until 2006, with over 10% of population increase in every 5 years (i.e., Census interval). However, the latest 2011 Census data showed that the Town only had minor growth since 2006 (2% over 5 years). The trend of growth appears to have been tempered.

The Town's Official Community Plan² (OCP) projected a total population to be 7,000 (low growth scenario) to 8,140 people (high growth scenario) by 2025 in the Town. As the OCP was approved in 2007 when the fast growing period just ended, annual growth rates (1.5-2.0% for low, 2- 3% for high) were optimistically assumed in the plan based on the historical growth patterns. The projected growth amount in the planned timeframe hence appears to be too conservative considering the minor growth in the past 5 years.

According to the South Okanagan Regional Growth Strategy³, the RDOS is expected to see a moderate population increase of more than 1.5% per year by 2031, and there is also a trend of a greater population increase in the Similkameen than in the South Okanagan.

² Town of Osoyoos Official Community Plan 2007, Bylaw No.1230, 2007

³ South Okanagan Regional Growth Strategy, Bylaw 2421, 2007, re-read Oct 22, 2009 – Schedule 'A', RDOS.



The regional growth strategy was supported by the South Okanagan QUEST (SOQ) program to define a preferred future growth scenario. The base assumption made in the development of a preferred growth scenario was the expectation of moderate growth in the region for the next 40 years. This assumption was based on 33% increase in the population over that time period.

In the context of a moderate growth⁴ in the South Okanagan region, the Town is anticipated to have a slow to moderate growth pattern in the next a few decades. Considering the currently tempered growth in the Town and the long range population forecasts in the region, an annual growth rate ranging between 0.8 – 1.2% would be a reasonable assumption to serve the purposes of this study. Table 2-4 provided the projected population growth within Town Boundaries in the next 20 years. As Osoyoos has become a major Okanagan retirement attraction since 2001, the population growth in the next 20 years is anticipated largely from retirees coming into Osoyoos from other areas of BC or elsewhere, while the labour force may expand at a lower rate than its population growth. It is assumed that ICI and seasonal PE will grow at only 50% of the population growth rate.

Table 2-4: Projected 20-Year Population Growth in the Town

	0-5 Year (By 2018)	6-10 Year (By 2023)	11-20 Year (By 2033)	20 Year Growth	Note
Annual Growth Rate	0.5%	0.8%	1.2%		
Residential	4,896	5,092	5,702	926	
ICI PE	2,311	2,357	2,499	216	Assume 50% of annual growth rate
Seasonal PE	3,511	3,581	3,796	328	Assume 50% of annual growth rate
Total PE	10,718	11,030	11,997	1,471	

Over the long term, the Town will face considerable growth constraints because of Osoyoos Lake, adjoining OIB, surrounding agricultural lands, and environmentally-sensitive areas. To accommodate the population growth, the Town has established a Town Growth Strategy, and drawn up a Town Growth Program in the OCP, which target to promote full use of existing urban areas to minimize demands for additional lands. The Town Growth Program encompasses eight existing urban areas, four new growth areas and one area proposed to be added through a municipal boundary extension. The Town has also developed the Southeast Meadowlark Area Plan, the largest growth area in the Town.

Long Term Population Projection

Quite a number of development applications were received by the Town in the recent years. The Town's planning department provided a 'Development Infill Appraisal (2013-09-17)' to help facilitate future population allocation in this study. The Development Infill Appraisal included information such as the proposed development locations, unit counts, estimated phasing / timing and likelihood of development to occur in a five-year increment in the next 20 years. The proposed infill developments are basically consistent with the Town's growth program. In addition to the infill development applications, potential ICI growth was assumed to include eight lots in the airport industrial growth area to allow industrial expansion according to the Town policies for its growth program. The total population and equivalent people from the above future developments (regardless of the likelihood to occur in the

⁵ The total populations from Oliver, Penticton and Summerland increased by 3.9% from 2006 to 2011, according to Statistics Canada 2011 Census data.



20-year timeframe) is estimated to be 3,275 PE. This growth amount is essentially equivalent to a 40 - 50-year planning horizon given the anticipated slow-to-moderate future growth pattern in the Town, and can be used as a reasonable yet conservative design population for sizing new infrastructure which normally have a design lifespan greater than 50 years for pipes, or 30 years for pump stations.

Growth from the Outside Users

Consultations with the surrounding OIB and RDOS were conducted to understand their future plans for the areas that would like to be serviced by the Town's sanitary sewer system, in addition to the existing committed service.

- RDOS: Future growth area would mainly be in the Northwest Sector Area of Electoral Area 'A':
 - The current plan has proposed 80 residential units at the Willow Beach area with potential for additional units when the sewer servicing reaches the area. A contingency factor of two was assumed. In addition, allowance of additional 15 units on the lake was assumed based on information obtained from the Town's consultant, TRUE. The total additional units will be 175; and
 - There are potentially up to 400 people in the East Bench area that would be serviced by the Town's sanitary sewer system (likely in 10 years)
- OIB: The OIB is currently in the middle of updating its land management plan for the south end of OIB. Only a high level estimate of the land usage is available. Information indicated that potential developments would include:
 - RV Park: 20 to 30 cabins;
 - Spirit Ridge: 250 to 300 more units;
 - Affordable housing: 120 to 200 units;
 - Other residential: 100 units; and
 - Commercial retail space: 10,000 to 20,000 square feet.

The estimated population and equivalent people for the planned new developments from the outside Town boundaries, which would like to get serviced by the Town's sanitary servicing, are summarized in Table 2-5.

Table 2-5: Planned Future Population Growth from Outside Town Boundaries

	RDOS Electoral Area 'A' Growth	OIB Growth	Total Growth
Residential	768	630	1,398
ICI	-	137	137
Seasonal	-	428	428
Total Growth (PE)	768	1,196	1,964

As the outside Town boundary future developments under discussion will primarily consist of land expansions for residential, ICI and seasonal uses, the growth amount may not present a similar growth pattern viewed from a larger geographic area like Osoyoos. It's at the Town's discretion to decide whether to extend its sanitary servicing to those new developments, while considering the following:



- Upon meeting the service need for the existing and 20-year population growth within the Town, the Town's sanitary sewer system (with necessary upgrades to meet its design level) will still have capacities to accommodate additional developments from its outside boundary users; and
- An appropriate funding mechanism is in place to fund additional upgrades (beyond the in-town growth needs) resulting from the outside boundary new developments.

Given the extent of the planned new developments, it is anticipated that the total growth number in Table 2-5 exceeds the 20-year planning horizon. The total growth amount is deemed as a long-term growth projection (30-40 years) that can be used as design population to identify deficiencies in the Town's sanitary system in addition to those being identified without them. Table 2-6 provides an estimated population growth in a 20-year time frame from the developments outside Town boundaries.

Table 2-6: Projected 20-Year Population Growth Outside Town Boundaries

	0-5 Year (By 2018)	6-10 Year (By 2023)	11-20 Year (By 2033)	20 Year Growth
Residential	623	948	1,197	899
ICI PE	157	174	209	69
Seasonal PE	924	977	1,084	214
Total PE	1,703	2,099	2,490	1,182



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Section 3

Trunk Sewer Capacity Assessment



3. Trunk Sewer Capacity Assessment

3.1 Town's Sanitary Sewer Conveyance System

The Town's sanitary sewer conveyance system is comprised of the following trunk sewer systems.

- **South East Sector System:** The system provides sanitary servicing to the properties abutting Lakeshore Drive and to the east. The southern portion of the sewer catchment is serviced by the Oasis Lift Station which is equipped with two Flygt NP 3102-MT, 5.0 HP pumps. The Oasis Lift Station also receives sewage from some properties to the south of the Town's municipal boundary, then pumps to the gravity system flowing north to the 44th Ave Lift Station. The 44th Ave Lift Station, currently equipped with two Flygt NP 3171-HT, 30 HP pumps (with room for a third pump), collects all the wastewater from the catchment and pumps to the Main Lift Station at Kingfisher Dr. & 89th St. through a 2,600 m of 250 mm dia. PVC forcemain and 850 m of 300 mm dia. PVC forcemain. The 300 mm dia. forcemain section is also shared with the Starlite Lift Station that services the Osoyoos IR No.1 Sector and with the Smith & Loveless Lift Station that services the North West Sector.
- **Osoyoos IR No. 1 Sector System:** The system provides sewer servicing to the northeastern quadrant of the Town plus developments in OIB north of the Municipal boundary. The 250/375 mm dia. trunk sewer, running along Cottonwood Dr., carries pumped sewage flows from a Town pump station (North Cottonwood Lift Station) and an OIB private pump station together with tributary gravity flows within the Town to the Starlite Lift Station, which is located on Main St. east of the Lake crossing. The Starlite Lift Station has two Flygt NP 3153, 20 HP pumps, and a 220 m of 250 mm dia. forcemain that joins the 300 mm dia. forcemain being shared with the 44th Ave Lift Station.
- **North West Sector System:** The sewer system in this sector collects and carries sewage flows towards the recently upgraded Smith & Loveless Lift Station from the eastern portion of the northwestern quadrant of the Town, plus a cluster of residential properties abutting the west lakeshore line in 'Electoral Area A' of the RODS. The trunk sewer runs from 87th St. at 92nd Ave. to the Smith & Loveless Lift Station, with 200 mm dia. (for the upstream sections) and 375 mm dia. in size. The recently upgraded Smith & Loveless LS is equipped with two Flygt NP 3153-MT, 20 HP pumps, with design capacity of 45 L/s with one pump operating. The 200 mm dia. Smith & Loveless LS forcemain joins the 300 mm dia. forcemain being shared with the 44th Ave and Starlite Lift Stations.
- **Industrial Park / Airport Sector System:** The sewer system in this sector provides sanitary services to the western portion of the northwestern quadrant of the Town, including all the industrial developments and the Airport along Hwy 3. The trunk sewer is a gravity system, starting from the Airport area on Hwy 3 and running through Main St, 89th St. till to the Main Lift Station, with pipe sizes ranging from 200 to 300 mm dia.
- **South West Sector System:** This sector includes all the properties south of 62 Ave, east of Hwy 97 and west of the Lake in the Town. Sanitary services are also provided for the Haynes Point Provincial Park to the east and the Canada-USA Customs to the south through local pumping systems. Sewage flows in the sector are conveyed to the Main Lift Station through a 250/300 mm dia. trunk sewer along the west lakeshore line, and a smaller 200 mm dia. sewer along 89th St. The wastewater in the trunk sewer gets lifted at Magnolia Pl. by the Blue Blinker Lift Station at half way to the Main Lift Station. The Blue Blinker LS is equipped with two Flygt CP 3082, 2.5 HP pumps,



and has a design capacity of 12.6 L/s with one pump running. The forcemain of the Blue Blinker LS is 150 mm dia., about 80 m in length.

- **Dividend Ridge Sector System:** The sewer system in this sector provides services to the Dividend Ridge area. All the wastewater from this sector is collected and pumped by the Golf Course LS to the Wastewater Treatment Lagoon through a 650 m of 200 mm dia. dedicated forcemain and a 500 m of 350 mm dia. shared forcemain with the Main Lift Station. .

Except for the Dividend Ridge Sector sewer flows, the Main Lift Station collects all the rest of the sewage flows and pumps to the Wastewater Treatment Lagoon. As reported by TRUE, the forcemain from the Main LS to the area of the treatment system has recently been upgraded. The upgrading comprises about 1,700 m of 400 mm dia. PVC forcemain which will replace the existing 200/250 mm dia. twin forcemains. This will increase the pumping capacity from 69 L/s to 95 L/s with one pump operating, or from 86 L/s to 135 L/s with two pumps operating. The next phase of improvements will involve replacement of the Main Lift Station.

The South East, Osoyoos IR No.1, North West, Industrial Park / Airport, and South West trunk sewer systems were analyzed in this study.

3.2 Sanitary Loading

Base Sanitary Flow Loading Rate

The per capita base sanitary flow (BSF) loading rate is the underlying daily volume applied to both residential and non-residential populations. According to the flow data reported in the Town's 2012 annual report (the 2012 Annual Report) for wastewater treatment and reclaimed water irrigation systems⁵, the maximum month average flow in 2012 was 3,041 m³/day. This corresponds to the total existing serviced population of 11,883 PE in the summer season.

As the total flow volume also included some amount of Inflow & Infiltration (I&I) due to both groundwater infiltration (GWI) and Rainfall-Dependant Inflow & Infiltration (RDII), the I&I volume needs to be excluded for the calculation of BSF. The Town's historical precipitation records indicated that the average rainfall amount is about 317 mm/year, and 21 – 36 mm/month in the summer months (Jun – Aug). Therefore, the total RDII volume is anticipated to be less significant in the total flow volume (while the peak RDII flow may not). For GWI, as it is not directly influenced by a particular rainfall event, its continuous contribution to the total flow volume is also expected to be low due to the arid desert geomorphy in Osoyoos and the observed constant groundwater levels over a year⁵. KWL's past flow monitoring experience from over 100 flow monitoring sites suggested that GWI may be as low as 1,000 L/ha/day (based on gross area by definition).

With a GWI rate of 1,000 L/ha/day and a total serviced lot area⁶ of 281 ha, a per capita loading rate of 231 L/PE/day was derived.

In order to verify the loading rate calculated by using the peak month average flow and population, a separate calculation was conducted to determine the loading rate by using the average winter month flow and population. The 2012 Annual Report showed that the average winter flow (from Oct to Apr)

⁵ Wastewater Treatment and Reclaimed Water Irrigation Systems 2012 Annual Report, April, 2013, TRUE Consulting. Project No. 302-1352.

⁶ A Net Area Factor (NAF) of 1.2 was applied to account for the use of net lot area rather than gross catchment area for I&I flow calculation. i.e., GWI rate will be 1,200 L/ha/day when net lot area is used. The same factor also applies to RDII.



was 1,812 m³/day, which is deemed to reflect the average usage from the local permanent population (i.e., the flows from seasonal visitors were excluded). The population in the winter months was about 85% of the population in May (i.e. the month when the Census data was taken) according to the monthly flow records in the 2012 Annual Report. The calculated population equivalents in the winter months were 6,372 (i.e., $0.85 \times (5,074 + 2,422)$), leading to a per capita loading rate of 228 L/PE/day, which is nearly consistent with the rate derived using the peak month average flow and population. In this study, a rounded number, 230 L/PE/day, was used for the trunk sewer hydraulic assessment.

I&I Estimate

The storm pattern in the area is generally characterized as high-intensity, short-duration thunderstorm, which normally leads to a considerable amount of direct inflow to the sanitary sewer system if defects exist in the manhole lid / cover in the low-lying areas or there are direct storm-sewer cross connections (e.g., from roof leader, etc). The lack of flow/rainfall monitoring data for a continuous period posed a challenge to estimating the I&I amount for assessing system hydraulic performance as well as for designing sewer infrastructure upgrades.

The Town of Osoyoos Subdivision and Development Servicing Bylaw No.1100, Schedule “A” – Subdivision and Development Standards and Specifications, requires that infiltration allowances shall not be less than 0.6 L/s/ha for designing new sewer infrastructure. However, this number is considered too conservative for assessing the existing system hydraulic performance, as the I&I alone would have overloaded many critical infrastructures, including the Main Lift Station which has an upgraded capacity of 135 L/s (with two pumps running), compared to the calculated I&I flow of 188 L/s with the 0.6 L/s/ha I&I rate.

The Town Operations advised that there were times both pumps were turned on in the summer (prior to the upgrades), and heavy rains made the situation more critical – however, overflows were never observed at the pump station. As the storms (and the associated RDII) may have occurred at any time during a day with the domestic flow varying diurnally (i.e., they could have occurred in the afternoon when the domestic flow was off peak, or in the morning / evening when the domestic flow was at peak), an average RDII without considering the timing of the storm events would reasonably be assumed to be the difference between the pumping capacity and the average dry weather flow, which is 53 L/s from a total contributing lot area of 261 ha. Therefore, the total I&I is estimated to be 15,600 L/ha/day, or 0.18 L/s/ha, compared to the design criteria I&I rate of 0.6 L/s/ha. The return period and duration of storm that resulted in this I&I is unknown. (Given the average daily flow being nearly steady in the past 10 years and no reported overflows at the Main LS, the return period is likely greater than five years). It is noted that the Environmental Management Act Municipal Sewage Regulation states that a sanitary sewer overflow shall not be allowed to occur during storm or snowmelt events with less than a five-year return period.

It is also noted that efforts in I&I reduction have been made in the past couple of years by reducing the risk of overflows at the Main LS. To better understand the I&I status within the Town, an I&I flow monitoring program is recommended to help establish a reasonable yet conservative I&I design value.

3.3 Trunk Sewer Capacity Assessment

A computer desktop spreadsheet model was developed for each trunk sewer system, which calculates peak flows within each of the subject sewers and compares the peak flows with the calculated pipe capacities. The following parameters were used to develop the spreadsheet model:



Loading

- Base Sanitary Flow (BSF): 230 L/PE/day (derived in this study)
- Peaking Factor: Harmon formula (per Bylaw No.1100)
- I&I: 15,600 L/ha/day or 0.18 L/s/ha (estimated in this study)

Hydraulic Calculation

- Head Loss Equation: Manning's (gravity), Hazen-Williams (forcemain)
- Roughness Coefficient: 0.012 for PVC gravity main, 0.013 for AC / concrete gravity main, 120 for forcemain.

Capacity Analysis

- Loading Condition: Peak Domestic Flow plus I&I
- Maximum d/D ratio: 0.82
- Maximum Q/Qfull ratio: 1.0

The following development loading scenarios were analysed, as shown in Table 3-1.

Table 3-1: Model Development Loading Scenarios

Model Scenario	Description	In-Town PE	Outside-Town PE	Total PE
Existing Development	Existing development condition	10,526	1,308	11,834
Long Range Future Development	Existing plus all the potential developments from the Town's Development Infill Appraisal. Maintain currently committed service to the users outside Town's boundaries.	13,801	1,308	15,109
Future Development plus Outside Growth	Existing plus all the potential developments from the Town's Development Infill Appraisal, and plus planned new developments outside Town's boundaries.	13,801	3,272	17,073

Existing Development Model

The existing development model reflects the existing peak flow conditions in the Town's sanitary sewer system. It assumes that peak dry weather flow in summer time when the total population is at peak is associated with I&I from a greater than 5-year return period storm.

The model results indicate that only one gravity sewer is inadequate to convey existing Peak Waste Water Flow (PWWF) and causes slightly surcharging above the pipe crown. This sewer is in the Industrial Park / Airport trunk sewer system, from Manhole A86 to A85. Figure 3-1 illustrates the hydraulic analysis results for the trunk sewers, including PWWF, full pipe capacity, and remaining capacity expressed as total equivalent population. The per capita PWWF was calculated to be 0.03 L/PE. In order to calculate a per capita peak flow, the following assumptions were made:



- Per household size: 2.1 PE/unit;
- Lot size: 0.05 ha;
- I&I rate: 0.18 L/s/ha; and
- Peaking factor: 3.5.

Except for the Main LS, all other lift stations in the trunk sewer systems under discussion are capable of conveying existing peak flows. The Main LS was identified as being at risk of overflow under the existing PWWF. Replacement of the pump station has been planned by the Town as the next phase of improvements.

Table 3-2 provides a summary of the predicted existing PWWF and estimated pumping capacity for the pump stations under discussion.

Table 3-2: Existing PWWF vs. Pump Station Capacity

Lift Station	Existing PWWF (L/s)	Existing Capacity (L/s)	Source of Pumping Capacity
Oasis LS	7.1	16.4 L/s @ 9.44 m TDH	Design drawing
44th Ave LS	19.5	40 L/s @ 25 m TDH	Design drawing
Starlite LS	42.3	65 L/s	Estimated based on Flygt NP3153-433 Imp, 20 HP
Smith & Loveless LS	36.4	52	Field test, reported by TRUE
Blue Blinker LS	5.5	12.6 L/s @ 3 m TDH	Design drawing
Main LS	147.2	95 L/s (1) / 135 L/s (2)	Provided by TRUE

The hydraulic calculations and trunk sewer hydraulic profiles are provided in Appendix A.

Long Range Future Development Model

The long range future development model assumes all the potential developments (from the Town's Development Infill Appraisal) being in place, with the intention to identify all the potential upgrades required to provide sanitary service to that population level. The long range future population level provides the upper limit for assessing system hydraulic constraints and for infrastructure sizing. This scenario does not include future population growth from new developments outside Town's boundaries, although the currently committed service to the outside Town boundary users is maintained.

The long range future development model results show that, one more gravity sewer (from Manhole A83 to A82) in the Industrial Park/Airport trunk sewer system will be marginally at capacity to convey the future PWWF, but surcharging is unlikely due to the improved hydraulic condition in the downstream sewers. Upgrading of this sewer is not recommended from a hydraulic capacity perspective unless new developments, other than the comprehensive development at 9120 Main St (KAP1949) and industrial expansion at the Airport (eight lots), from its upstream would be proposed. Figure 3-2 displays the hydraulic analysis results for the trunk sewers.

All the pump stations are expected to be capable to convey future PWWF (assuming the Main LS is replaced and adequately sized). The predicted long range future PWWF and estimated pumping capacity for each pump station are summarized in Table 3-3.



Table 3-3: Long Range Future PWWF vs. Pump Station Capacity

Lift Station	Future PWWF (L/s)	Existing Capacity (L/s)
Oasis LS	7.6	16.4 L/s @ 9.44 m Total Dynamic Head (TDH)
44th Ave LS	23.9	40 L/s @ 25 m TDH
Starlite LS	50.3	65 L/s
Smith & Loveless LS	42.5	52 L/s
Blue Blinker LS	5.8	12.6 L/s @ 3 m TDH
Main LS	175.8	Upgraded capacity is assumed to be 180 L/s

Future Development plus Outside Growth Model

This model scenario assumes all the potential developments within and outside Town boundaries are in place to identify hydraulic deficiencies in the sanitary sewer system in addition to those being identified from the Town's long range future development scenario.

With additional 768 PE from Electoral Area 'A' and 1,196 PE from OIB, additional hydraulic deficiencies were identified in the sewer system as discussed below:

OIB Trunk Sewer

- All the 250 mm dia. trunk sewers on Cottonwood Dr. from MH C18 to C11 (512 m +/-) will exceed pipe capacities, while the sections from C18 to C13 will be backwatered and surcharged above pipe crown. Upgrading of the entire 250 mm dia. sections of sewer is required to accommodate the increased flow from long-term growth in OIB; and
- PWWF at the Starlite LS is predicted to be 65 L/s, reaching the pumping capacity estimated to be 65 L/s with one pump operating. A pumping test is recommended to confirm the existing pumping capacity so as for requirements for upgrading to handle the design flows with the OIB planned growth.

North West Trunk Sewer

- Only one sewer will be marginally at capacity (B52 to B51), surcharging is however unlikely. Upgrading of this pipe is not recommended unless the pipe condition requires it.

3.4 Capital Improvement Projects

Based on the hydraulic analyses of the trunk sewer systems in the Town, the deficiencies were found to be in one sewer (MH A86 to A85) in the Industrial Park / Airport trunk sewer, and in the Main Lift Station – if the Town does not extend its sewer servicing for new developments outside the Town boundaries. As the analyses indicated that deficiencies were present, upgrading these two mains should be budgeted for construction as a short-term task, i.e. 1-3 year.

With the planned new development outside Town boundary being serviced, additional deficiencies were found to be in all of the 250 mm dia. sewers along Cottonwood Dr. This is due to the increased flows from new developments in the OIB. The timing for this upgrade to occur will depend on the actual rate of development in its upstream tributary areas. Information provided in Figure 3-1 and Figure 3-2 can be used as a guide to determine the timing of upgrade.



Table 3-4 shows the proposed capital improvements.

Table 3-4: Proposed Capital Improvements

Upgrade	Location	Existing Size	Length (m)	Upgrading Size
Pipe Upgrade	A86 to A85	200 mm dia.	62	300 mm dia.
	C18 to C11	250 mm dia.	512	375 mm dia.
Pump Station Upgrade	Main LS	95 L/s Capacity (for 1 pump)		180 L/s Capacity (for 1 pump)

3.5 Capital Cost Estimate

The cost estimates provided in this study are of Class ‘D’ accuracy. This means that the general requirements for upgrading including size and approximate depth of excavation, as well as some general site conditions are known. The projects identified have not considered the following factors affecting construction:

- Relocation of adjacent services (water, hydro, etc.);
- Special permitting requirements (fisheries windows, contaminated site, etc.);
- Land acquisition; and
- Critical market shortages of materials.

As the above factors have not been allowed for in estimating construction unit rates or project design, the following factors are applied to all projects:

- PST – 7%;
- Contractor Mark-up/Overhead – 6%;
- Mobilization/Demobilization – 10%;
- Bonding/Insurance – 2%;
- Engineering – 20%; and
- Contingency – 40%.

GST is not included in the estimated project costs.

The unit prices reflect KWL’s recent experience with similar work, and therefore represent the best prediction of actual (2013) costs as of the date prepared. Actual tendered costs would depend on such things as market conditions generally, remoteness factor the time of year, contractors’ workloads, any perceived risk exposure associated with the work, and unknown conditions.

Table 3-5 provides the estimated costs for the identified capital projects at a “Class D” level.



Table 3-5: Class “D” Cost Estimates

Item	Description	Unit	Estimated Quantity	Unit Cost	Cost	Note
PS1 - Main Pump Station Upgrade	Civil Construction	L.S.	1	\$500,000	\$500,000	- Excavation, backfill, concrete work, bypass pumping, traffic control, and site restoration, etc.
	Mechanical Work	L.S.	1	\$350,000	\$350,000	- Pump supply / installation, mechanical piping, valve chamber incl. valving & piping, and commissioning, etc.
	Electrical Work	L.S.	1	\$500,000	\$500,000	- Power supply, transformer, electric ducting and installation, control kiosk supply and installation, genset full enclosure, and commissioning, etc.
	Subtotal For Task				\$1,350,000	
	Mob./Demob., Bonding, Insurance, Overhead	18%			\$243,000	
	Engineering & Construction Management	20%			\$270,000	
	Contingencies	40%			\$540,000	
	Total Cost				\$2,403,000	
GM1 - Gravity Main Upgrade	A86 to A85, 300 mm dia.	Lin.m	62	\$548	\$33,976	Average cover depth of 1.5 m, asphalt pavement, imported backfill, and extra traffic control
	Subtotal For Task				\$33,976	
	Mob./Demob., Bonding, Insurance, Overhead	18%			\$6,116	
	Engineering & Construction Management	20%			\$6,795	
	Contingencies	40%			\$13,590	
	Total Cost				\$60,000	
GM2 - Gravity Main Upgrade	C18 to C11, 375 mm dia.	Lin.m	512	\$604	\$309,248	Average cover depth of 1.9 m, asphalt pavement, imported backfill, and regular traffic control
	Subtotal For Task				\$309,248	
	Mob./Demob., Bonding, Insurance, Overhead	18%			\$55,665	
	Engineering & Construction Management	20%			\$61,850	
	Contingencies	40%			\$123,699	
	Total Cost				\$550,000	



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Section 4

Wastewater Treatment System



4. Wastewater Treatment System

This section summarizes the key findings from the preliminary capacity assessment of the Osoyoos WWTP to handle existing and longer term wastewater flows.

4.1 Introduction

The Osoyoos wastewater treatment plant (WWTP) is an aerated sewage treatment lagoon system located to the south-west of the town. The major WWTP components include two influent lift stations, three aerated cells, two winter effluent storage basins, isolation piping for the treatment plant, effluent pump station, and one chlorine contact basin. Effluent used for irrigation is conveyed via booster stations located downstream of the chlorine contact basin. The WWTP effluent is stored during the winter in the storage basins and is used as reclaimed water for irrigation in the summer. All WWTP dimensions, volumes, wastewater flow rates, and wastewater characteristics were obtained from the information provided by Osoyoos and TRUE.

Existing Discharge Permit

The Ministry of Environment (MOE) discharge permit ME-12214 for the Osoyoos WWTP allows for a maximum authorized discharge rate of 2,720 m³/d and an average monthly authorized discharge rate of 4,340 m³/d. The characteristics of the discharge after Aeration Cell #3, prior to storage, for BOD₅ and TSS are not to exceed 45 mg/L and 60 mg/L, respectively.

Existing WWTP Facilities

System Description

The existing Osoyoos WWTP is an aerated lagoon system consisting of two influent lift stations (Main lift station and Golf Course lift station) and bypass facilities, three aeration cells connected in series, followed by two effluent storage basins, an effluent pump station, and a chlorine contact chamber. Treated effluent is discharged from the chlorine contact chamber into booster stations which distribute the effluent to three separate irrigation fields. The influent flow is measured by separate flow meters installed at the Main lift station and the Golf Course lift station. The effluent flow is measured by two flow meters, identified as the Desert Nine and Meadows meters, located downstream of the chlorine contact basin.

The Osoyoos WWTP can generally be classified as an aerobic flow-through lagoon with typical operating characteristics of a well-designed and operated lagoon (Reference: Metcalf and Eddy, 4th Edition) outlined in Table 4-1 below. It is noted that information on the design and configuration of the aeration system was not provided to KWL at the time of writing of this report. However, the aeration system was recently upgraded in 2010 and it is assumed that sufficient air and mixing is provided to the aeration cells.



Table 4-1: Typical Operating Characteristics of an Aerobic Flow-through Lagoon

Parameter	Typical Value	Osoyoos WWTP
Hydraulic Retention Time (HRT)	3-6 days, typically 5 days	43 days ⁽¹⁾
Solids Retention Time (SRT)	3-6 days, typically 5 days	> 43 days ⁽²⁾
Depth	2-5 m	3-5 m
Minimum mixing power ⁽³⁾	5.0-8.0 W/m ³	Information not available
Effluent BOD ₅	20-45	16 ⁽⁴⁾
Effluent TSS	30-60	13 ⁽⁴⁾
Notes - HRT and SRT were estimated based on an average daily wastewater flow rate of 2,055 m³/d from 2009-2012. - SRT will vary according to the degree of mixing power input into the aeration cells and is assumed to be insufficient to provide complete mixing in the cells. If the mixing power is sufficient to provide complete mixing, then SRT will be equal to the HRT. Otherwise, the SRT will be greater than HRT. - Measured as volume of the aerated lagoons. - Average BOD₅ and TSS concentrations are based on the sampling results reported in the 2012 Annual Report.		

Influent/Headworks

Wastewater is discharged into the treatment system from two lift stations, the Main lift station and the Golf Course lift station.

The Main lift station, located to the north-east of the WWTP, has a capacity of 7,200 m³/day with one pump in operation, and 8,969 m³/day with two pumps in operation. Wastewater from the Main lift Station is conveyed to the treatment plant through a forcemain comprised of twin pipelines, a 200 mm diameter AC and a 250 mm PVC main.

The Golf Course lift station, located at the north-east corner of the wastewater treatment plant, conveys wastewater into the treatment system through a 200 mm diameter forcemain. The lift station has a stated capacity of approximately 8,200 m³/d with one pump in operation, and approximately 11,600 m³/d with two pumps in operation.

The wastewater from the two lift stations is directed into Aeration Cell #1 of the WWTP through a 350mm diameter forcemain. There is also a bypass connection to Aeration Cell #2.

The forcemain from the Main lift Station to the treatment system is currently being upgraded. The existing 200 mm and 250 mm twin forcemains are being replaced by a 1,700 m long, 400 mm diameter PVC forcemain. The upgrade allows for a capacity increase of 9,820 m³/day to 17,020 m³/day with one pump in operation, and 14,009 m³/day to 24,978 m³/day with two pumps in operation.

The existing treatment plant does not have any primary treatment facilities such as coarse screening, fine screening, grit removal or primary sedimentation.

Aeration Cells

Table 4-2 provides a summary of the physical characteristics of the three aeration cells at the Osoyoos WWTP.



Table 4-2: Physical characteristics of the Osoyoos WWTP aeration cells

	Depth (m)	Volume (m ³)	HRT at Average Daily Flow (days) ⁽¹⁾	HRT at Maximum Monthly Flow (days) ⁽²⁾
Aeration Cell #1	3.5	5,000	2.4	1.7
Aeration Cell #2	4.7	24,500	11.9	8.1
Aeration Cell #3	4.7	60,000	29.2	19.8
Notes: 1. Estimated HRT is based on an average daily wastewater flow rate of 2,055 m ³ /d from 2009-2012 flow records. 2. Estimated HRT is based on a maximum monthly wastewater flow rate of 3,028 m ³ /d from 2009-2012 flow records.				

The Osoyoos WWTP consists of three aeration cells connected in series. Wastewater discharged from the lift stations is first collected into Aeration cell #1. Aeration Cell #1 discharges wastewater into Aeration Cell #2, and has a bypass connection to Aeration Cell #3. A bypass connection also exists from Aeration Cell #2 into Effluent Storage Basin #2. The total combined aeration cell volume is approximately 89,500 m³, and since the three aeration cells are connected in series, the total hydraulic retention time for aeration is approximately 43 days. As there does not appear to be any external sedimentation, it is assumed that sedimentation is occurring in Cell #3. Treated effluent from Aeration Cell #3 is discharged into Effluent Storage Basin #1 and Effluent Storage Basin #2. The discharge from Cell #3 to the two storage basins is controlled via two gate valves for manual flow splitting.

Below is a list of major upgrades completed for the aeration cells at the Osoyoos WWTP:

- Aeration Cell #1:
 - o A liner replacement was completed in 1994.
 - o An aeration system upgrade was completed in 1998.
- Aeration Cell #2:
 - o The existing liner was replaced in 2010.
 - o The static tube aeration system was removed and replaced with a more energy efficient fine bubble diffuser system in 2010.
 - o The blowers were replaced to provide more efficient aeration in 2010.
 - o A solar panel array was installed with the objective of supplying some portion of the blower electrical energy requirements in 2010.
- Aeration Cell #3:
 - o The liner was previously replaced in 1978.
 - o An aeration system upgrade was completed in 2002.

Effluent Storage Basins

There are two effluent storage basins at the Osoyoos WWTP. Effluent storage basin #1 has a length of approximately 283 m and a variable width of 100 m to 175 m. Effluent storage basin #2 has a length of



approximately 320 m and a variable width of 100 m to 200 m. The depth of effluent storage basin #1 is 7.5 m. The depth of effluent storage basin #2 is between 7.6-12 m. All basin side slopes are noted to be 3:1. The total liquid volume in storage basin #1 and storage basin #2 is approximately 270,000 m³ and 295,000 m³, respectively. Effluent storage basin #1 was expanded in 2008 and the liner was also replaced at that time. An infiltration bed near the effluent storage basins provides an alternate means of effluent disposal. This infiltration bed is understood to be a permanent facility.

Effluent Pump Station

An effluent pump station is located at the downstream end of storage basins #1 and #2 at the Osoyoos WWTP. Effluent is received into the pump station from storage basins #1 and #2 through a 450 mm and a 400 mm diameter HDPE inlet, respectively. The pump station has three single stage vertical turbine pumps connected in parallel. Each pump has a capacity of 10,474 m³/day (1,600 GPM) at 23 m of total dynamic head (TDH).

The pump station discharges effluent to the chlorine contact chamber through a 400 mm diameter PVC outlet pipe.

Based on the 2012 Annual Report, the effluent supply pump station was upgraded in 2008.

Chlorine Contact Chamber (Disinfection)

A reinforced cast in place concrete chlorine contact basin was constructed at the Osoyoos WWTP in 2008. The chlorine contact chamber has a length of 24.38 m and a width of 14.63 m. The depth of chamber, based on the TRUE drawing (#302-1141-41) from January 2007, is 3.6 m. Based on these drawings, the total liquid volume of the chlorine contact chamber is estimated to be 1,284 m³. The disinfected effluent is released from the chamber through a 600 mm diameter PVC outlet towards the booster stations.

Effluent Flow Monitoring

The effluent flow, discharged from the chlorine contact chamber, is measured by two flow meters (labelled as Desert Nine and Meadows) located in the supply mains immediately downstream of the chlorine contact basin.

Reclaimed Water

The areas of the West Bench that are irrigated with reclaimed water from the WWTP are provided in Table 4-3 below:

Table 4-3: Irrigated Areas in Osoyoos using Wastewater Effluent

Location	Irrigation Area (ha)
The Osoyoos Golf and Country Club	65.6
Baseball diamonds to the west of 115 th Street	2.4
Desert Park Infield Area	5.0
Total	73.0



4.2 WWTP Flows and Loads

The 2012 Annual Report from TRUE included summarized influent sewer flow data for a 21 year period from 1992-2012. From 1992 to 2001, the total annual sewage flow volume into the WWTP plant increased by 40%, from 523,255 m³ to 731,198 m³, respectively. However, from 2001 to 2012, the total annual sewage flow volume into the treatment plant was relatively constant between 731,198 m³ to 733,885 m³, respectively.

Based on the population equivalent projections presented in Section 2 of this report, the flows to the WWTP are anticipated to increase as shown in Table 4-4 below.

Table 4-4: WWTP Flow and Loading Projections based on Osoyoos In-Town Growth Only

Parameter	2013	0-5 Year (By 2018)	6-10 Year (By 2023)	11-20 Year (By 2033)
Average Daily Flow (ADF, m ³ /d)	2,143	2,197	2,286	2,576
BOD ₅ mass loading at ADF (kg/d) ⁽¹⁾	531	545	567	639
TSS mass loading at ADF (kg/d) ⁽²⁾	536	549	572	644
Maximum Monthly Influent Flow Volume (m ³)	82,400	84,500	87,900	99,000
Annual Influent Flow Volume (m ³)	870,000	890,000	922,000	1,028,000
Estimated Annual Losses From WWTP Lagoons (m ³) ⁽¹⁾	124,000	124,000	124,000	124,000
Net Annual Flow Volume (m ³) ⁽⁴⁾	746,000	766,000	798,000	904,000
Notes: 1. Based on an assumed influent BOD ₅ concentration of 248 mg/L, as reported in the 2012 Annual Report, Appendix C. 2. Based on an assumed influent TSS value of 250 mg/L. Influent TSS data was unavailable at the time of writing of this report. 3. Assuming no additional lagoon cells are constructed and net losses due to evapotranspiration remains constant. 4. Based on a hydraulic balance of influent flows minus losses.				

The WWTP flow projections are based on a hydraulic balance using data from the 2012 Annual Report. The net annual flow volume estimate takes into account the volumetric gains and losses to and from the WWTP, and includes an estimate of evapotranspiration at the lagoons. The influent flow volume is thus a reasonable estimate considering the planning nature of this assignment. As a check on the flow projections, the influent flow volume estimated using the Town population and per capita base sanitary flow rates plus average groundwater infiltration agrees with the flow projections presented above.

The mass loading of wastewater constituents are expected to increase over time due to population growth. However, it is assumed that the influent concentration of wastewater constituents such as BOD₅ and TSS will not significantly change from those currently measured at the WWTP since the population growth is expected to be predominantly due to an increase in residential population.



4.3 WWTP Capacity

WWTP Aeration Cells

Sufficient information regarding the design and operation of the aeration system was not available at the time of writing of this report for a complete assessment of the biological treatment capacity of the three aerated lagoon cells.

However, from a hydraulic perspective, the aerated lagoon cells (with an estimated total combined volume of 89,000 m³) have sufficient hydraulic capacity to provide a hydraulic residence time of 3-6 days under both average dry weather flow conditions as well as the peak wet weather flow (as estimated in Section 3).

Provided that the system is designed and operated as an aerobic flow-through lagoon, the aeration cells are expected to have sufficient capacity to meet the 20-year projected flows if sufficient aeration and mixing is provided in the cells for biological treatment, and a sufficient volume of the aeration cell is reserved for sedimentation.

Effluent Storage Basins

Based on the current flows, the total annual volume discharged to the effluent storage basins (approximately 746,000 m³) is offset by the volume of reclaimed water used for irrigation and some loss due to evapotranspiration. However, this balance may fluctuate based on the weather and the amount of irrigation water that is required during the summer months. During periods of cool and wet weather, there may be a lower demand for reclaimed water and lower volume losses due to evapotranspiration. As illustrated in the 2012 Annual Report, it was apparent that a net accumulation of effluent in the storage basins was observed due to a lower irrigation demand compared to previous years. It is noted that the Town is currently planning to expand the reclaimed water system.

Currently, a total land area of 73 ha is irrigated with reclaimed water. Thus, an annual irrigation demand of over 1,000 mm/ha is required to achieve a balance between total inflow and total outflow. Based on the current flows, annual irrigation demands of less than 1,000 mm/ha would result in a net accumulation in the storage basins. Table 4-5 below illustrates the sensitivity of the irrigation rate on the net accumulation in the storage basins.

Table 4-5: Sensitivity of annual irrigation rate on net accumulation in the effluent storage basins

		Annual Irrigation Demand Scenario	2013	0-5 Year (By 2018)	6-10 Year (By 2023)	11-20 Year (By 2033)
Net Storage Basins Inflow (m ³) ⁽¹⁾		-	746,000	766,000	798,000	904,000
Annual Reclaimed Water Consumption (m ³) ⁽²⁾	a) hot/dry	1,000 mm/ha	(730,000)			
	b) average	900 mm/ha	(657,000)			
	c) cold/dry	800 mm/ha	(584,000) ⁽³⁾			
Net Annual	a) hot/dry	1,000 mm/ha	16,000	36,000	68,000	174,000



		Annual Irrigation Demand Scenario	2013	0-5 Year (By 2018)	6-10 Year (By 2023)	11-20 Year (By 2033)
Accumulation in Storage Basin (m ³) ⁽⁴⁾	b) average	900 mm/ha	89,000	109,000	141,000	247,000
	c) cold/dry	800 mm/ha	162,000	182,000	214,000	320,000
Notes: 1. Refer to Table 4-4 above 2. Based on a constant irrigation area of 73 ha 3. KWL is aware that Osoyoos is extending their reclaimed water system; this assessment does not include these proposed works. 4. Total Storage Basin Volume: 565,000 m ³						

The net accumulation in the storage basins is sensitive to two factors affecting the irrigation volume - the area being irrigated and the weather which affects the irrigation demand. From Table 4-5, the net balance between the inflows and effluent flows plus evapotranspiration is close to balanced, with only an excess of 16,000 m³/year or 3% of the total storage volume, as long as irrigation demands are in the range of 1,000 mm/ha, indicating hot and dry weather conditions. If the weather is cold and wet, the accumulation could be as high as 162,000 m³, which is approximately 28% of the total storage volume.

In the future, a net accumulation in the storage basins is expected due solely to population growth and resulting higher flows, even during hot, dry years. The accumulation could be even higher during periods of cool and wet weather during the irrigation season, when the demand for irrigation water is reduced.

Please refer to the following figures:

- Figure 4-1 presents the net annual accumulation of flow in the storage basins for the 1000 mm/ha, 900 mm/ha and 800 mm/ha irrigation scenarios.
- Figures 4-2, 4-3, and 4-4 present the net cumulative accumulation of flow in the storage basin after one year plus the total flow from October to May, when effluent is not being discharged; the three irrigation scenarios are presented. These graphs are meant to highlight that at some point in time, the accumulated flow remaining at the start of fall plus the winter flow will exceed the existing storage capacity. In the summer, the irrigation demand is typically higher than the flow into the storage basins and the basins are drawn down slowly.

Based on the foregoing, it is clear that the capacity of the effluent storage basins will be reached in the short-term if additional land area for reclaimed water irrigation is not found. Additional land area that can accept reclaimed water could help reduce the accumulation in the storage basins and potentially eliminate the need for additional storage. If additional land area is not available, then additional storage will be required. It is noted that the Town is currently planning on expanding the reclaimed water system and is actively seeking additional land area for irrigation. Alternative methods of effluent disposal such as the disposal of effluent to Osoyoos Lake could also reduce the quantity placed in storage in the long-term and alleviate the need for effluent storage.

Figure 4-1. Annual Excess Effluent Produced at the Osoyoos WWTP

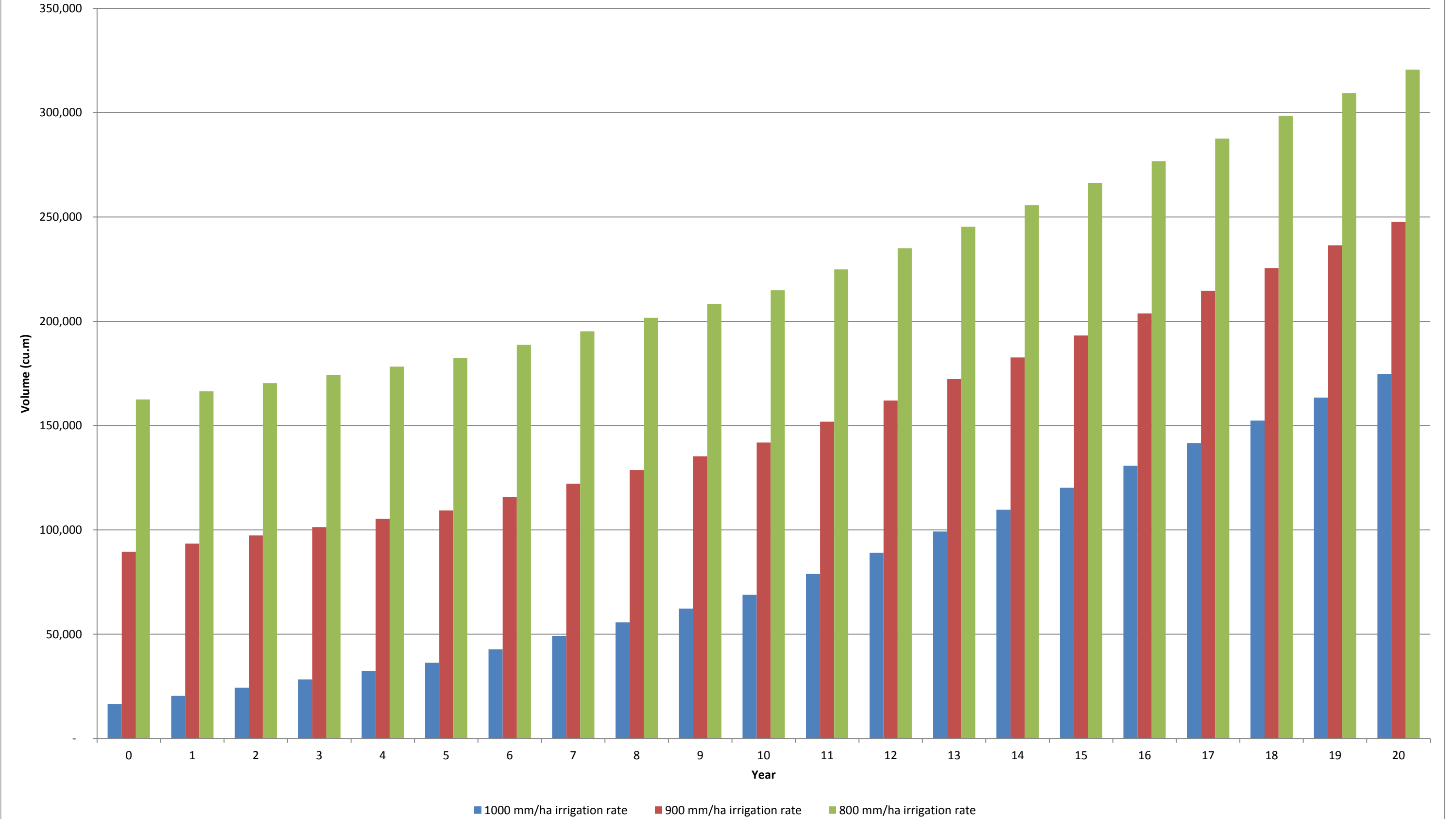


Figure 4-2. Annual Effluent Storage Requirements at the Osoyoos WWTP
1000 mm/ha irrigation rate

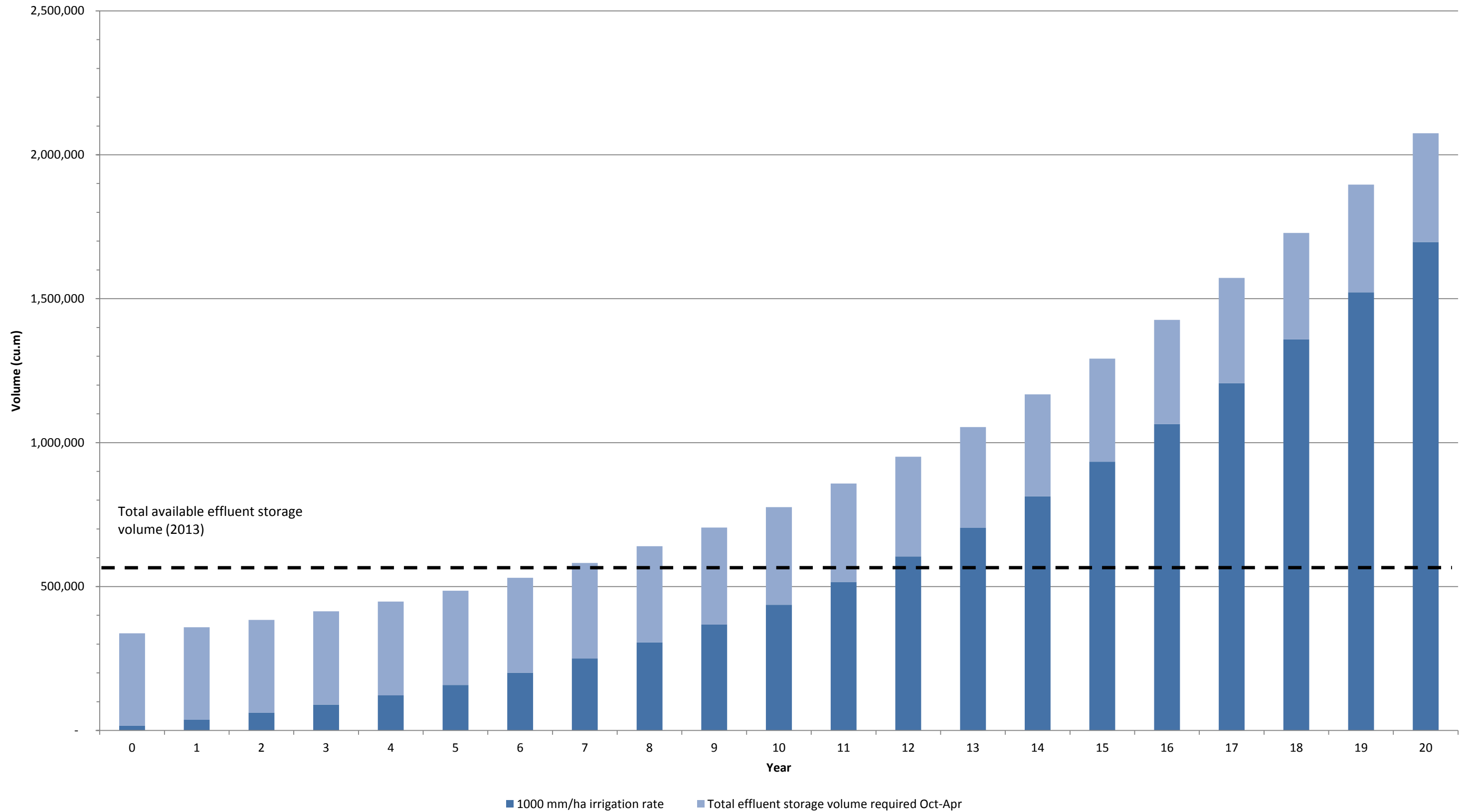


Figure 4-3. Annual Effluent Storage Requirements at the Osoyoos WWTP
900 mm/ha irrigation rate

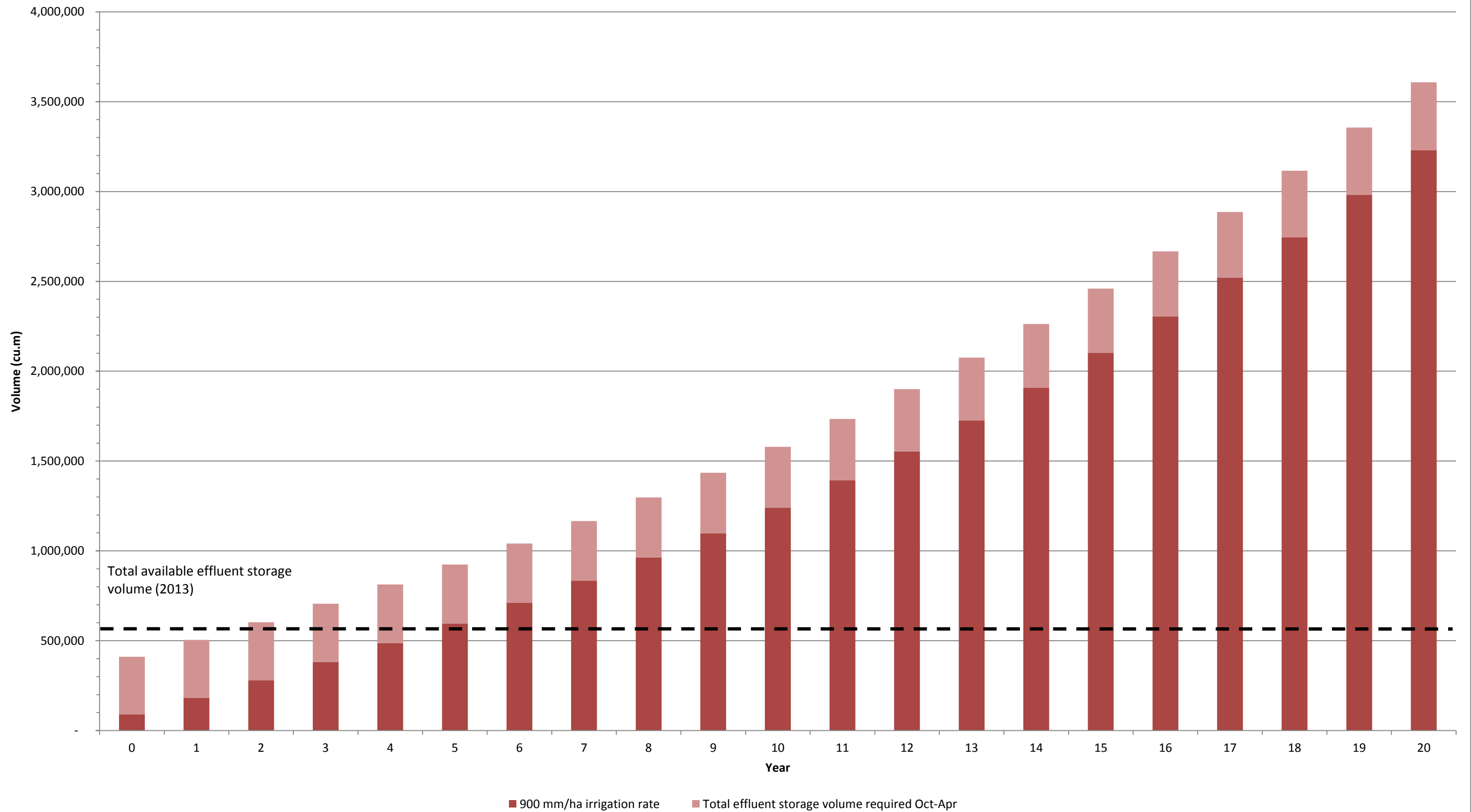
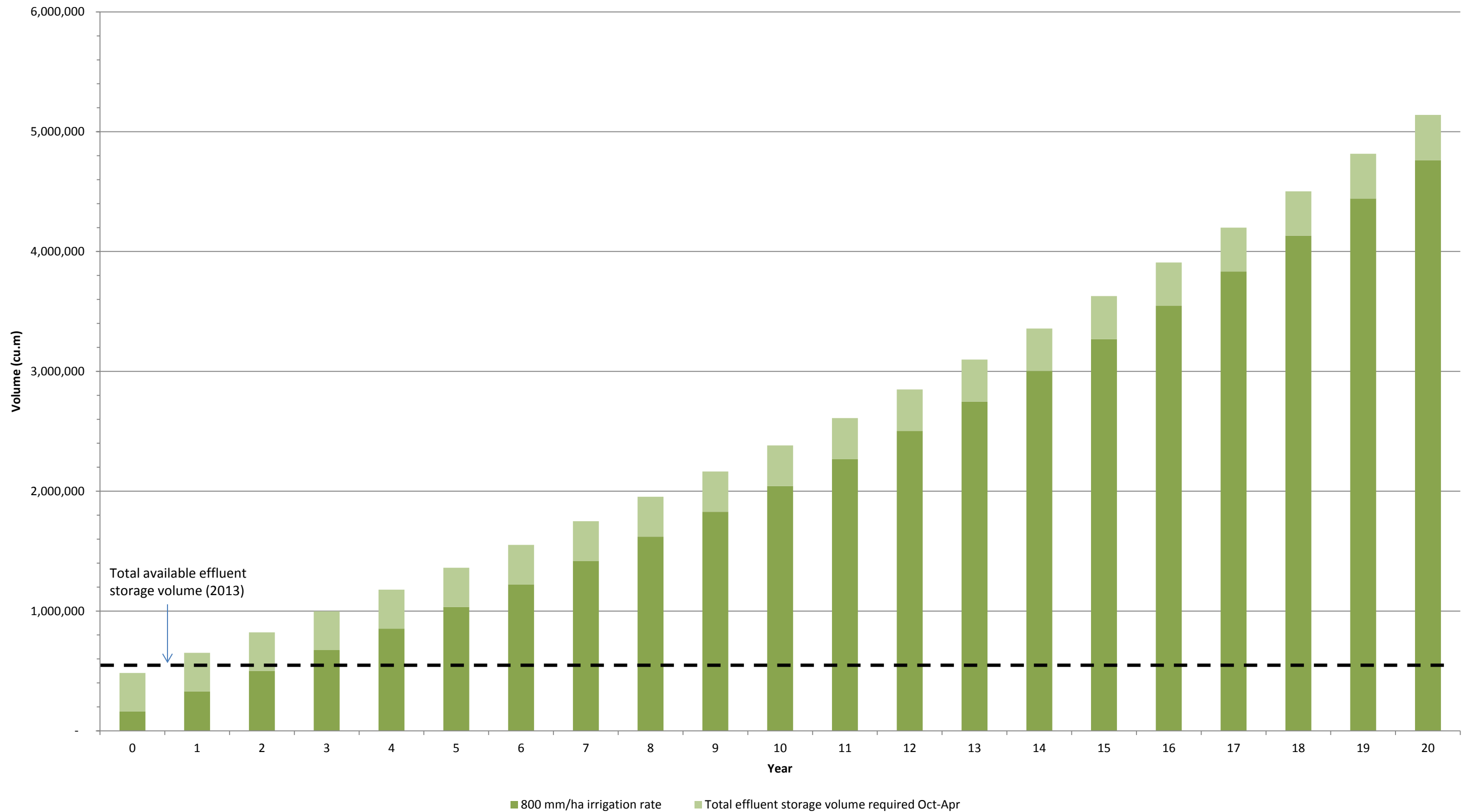


Figure 4-4. Annual Effluent Storage Requirements at the Osoyoos WWTP
800 mm/ha irrigation rate





4.4 Sludge Disposal

It is noted that sludge is periodically removed from the lagoon cells. In 2010, Aeration Cell #2 was taken out of service and the accumulated sludge from 32 years (1978 to 2010) was removed. Sludge removal was completed in Aeration Cell #1 in 2013.

As the mass loading of wastewater constituents such as BOD₅ and TSS increase over time, the quantity of sludge generated in the WWTP will also increase and will result in the need for more frequent lagoon desludging. Since the scope of this report is specifically related to the hydraulic capacity of the WWTP, further commentary on solids disposal has not been included.

4.5 Capital Improvement Projects

Capital improvement projects at the WWTP will be required as the projected flows from the Town increase over the next 20 years to meet the effluent quality and quantity requirements. Based on the capacity assessment presented in the preceding sections, over the next 20 years there will be an increasing quantity of effluent that must be treated and disposed.

Assuming that the current strategy of winter effluent storage and summer reclamation of effluent for irrigation is continued, there will be a need to build additional storage cells and/or find additional land area for irrigation. The existing storage cells may be nearing its operational capacity if the existing irrigation area remains constant. The available capacity may be significantly reduced in average or cold/dry conditions where there is less irrigation and more seasonal accumulation. In the short term, additional storage can help alleviate the problem of sufficient land area for irrigation by storing any excess effluent that is not used during the summer. In the long term, additional land area would be required. At an assumed average seasonal irrigation rate of 900 mm/ha, an additional 30 ha of land area would be required over the next 20 years for irrigation to achieve a net zero annual accumulation in the cells.

Assuming that sufficient land area for irrigation is available, the required effluent discharge rate over the next 20 years for seasonal irrigation is expected to increase to 177 L/s assuming an average irrigation rate of 900 mm/ha. At a firm effluent pumping capacity of approximately 3,500 USGPM (221 L/s) in the existing system, it is assumed that the existing effluent pumping system, piping and booster pumps can handle future flows. The major work for the capital improvement project is thus assumed to include the construction of a new lined effluent storage cell providing 300,000 m³ of storage volume, piping and appurtenances to connect the new cell to the effluent pumping system, and expansion of the chlorine contact tank. For the purposes of capital planning, the additional storage cell is assumed to be constructed in Year 10 combined with an additional land area for irrigation of 15 ha. It is noted that if sufficient additional land area can be used in Year 0 for irrigation, then there would be a net zero accumulation beginning in Year 0 and potentially eliminate the need for additional storage.

Effluent disposal to water may be an alternative to irrigation of reclaimed water. However, this would require a significant upgrade of the treatment plant from lagoons to a tertiary treatment plant, as well as construction of a new outfall from the treatment plant to Osoyoos Lake (assuming all regulatory requirements could be met and approval from MOE via a registration under the Municipal Wastewater Regulation is obtained). For treatment systems in the Okanagan Basin, phosphorus removal to less than 0.25 mg/L on average, and total nitrogen removal not to exceed 6 mg/L is required according to the Municipal Wastewater Regulation. This would represent a significant additional cost to upgrade the treatment plant to a tertiary treatment system to remove BOD₅, TSS, and nutrients to meet the regulatory requirements. This option has not been included as a capital improvement project but may need to be considered in greater detail if the Town is unable to find additional land area for irrigation.



4.6 Capital Cost Estimate

The cost estimates provided in this study are of Class 'D' accuracy. This means that the general requirements for upgrading including size and approximate depth of excavation, as well as some general site conditions are known. The projects identified have not considered the following factors affecting construction:

- Relocation of existing services;
- Land acquisition for the effluent storage cells and additional land area for irrigation; and
- Critical market shortages of materials.

As the above factors have not been allowed for in estimating construction unit rates or project design, the following factors are applied to all projects:

- PST – 7%;
- Contractor Mark-up/Overhead – 6%;
- Mobilization/Demobilization – 10%;
- Bonding/Insurance – 2%;
- Engineering – 20%; and
- Contingency – 40%.

GST is not included in the estimated project costs.

The unit prices reflect KWL's recent experience with similar work, and therefore represent the best prediction of actual (2013) costs as of the date prepared. Actual tendered costs would depend on such things as market conditions generally, remoteness factor the time of year, contractors' workloads, any perceived risk exposure associated with the work, and unknown conditions.

Table 4-6: Class 'D' Capital Cost Estimate for Wastewater Treatment Plant Capital Projects

Item	Description	Unit	Estimated Quantity	Unit Cost	Cost	Note
Expansion of the effluent storage lagoon	Effluent storage	L.S.	1	\$4,000,000	\$4,000,000	- Civil works for construction of an additional cell with 300,000 m ³ of additional effluent storage, including cell liner.
	Mechanical Work	L.S.	1	\$200,000	\$200,000	- Floating decanter. - Site piping from new effluent storage cell to effluent pumps. - Existing piping and effluent pumping system has sufficient hydraulic capacity to discharge at the reclaimed water usage rate (i.e. the existing irrigation rate).
	Disinfection	L.S.	1	\$300,000	\$300,000	- Assume expansion of the existing chlorine contact process to maintain the existing HRT of 2 hours with future flows.
	Subtotal For Tasks				\$4,500,000	



Item	Description	Unit	Estimated Quantity	Unit Cost	Cost	Note
	Mob./Demob., Bonding, Insurance, Overhead	18%			\$800,000	
	Engineering & Construction Management	20%			\$1,100,000	
	Contingencies	40%			\$2,200,000	
	Total Cost				\$8,600,000	



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Section 5

Cost Sharing Opinion



5. Cost Sharing Opinion

To fund the proposed upgrades, a per-du (dwelling unit) cost sharing approach can be used. As additional upgrades would be triggered by the outside Town boundary users, the cost sharing was structured to distinguish between within Town and outside Town boundary developers.

5.1 Basic Assumptions

In order to develop a total equivalent dwelling units, the following assumptions were made:

- Low Density Residential (density < 40 du/ha): occupancy rate 2.3 pop/du, or 1 equivalent du;
- Medium Density Residential (density $\geq$ 40, < 90 du/ha): occupancy rate 2.1 pop/du, or 0.91 equivalent du;
- High Density Residential (density $\geq$ 90 du/ha): occupancy rate 1.7 pop/du, or 0.74 equivalent du;
- The residential growth in the Town for the next 20 years is assumed to have 50% for low density, 30% for medium density, and 20% for high density;
- Commercial (density = 60 PE/ha): 26.1 equivalent du per ha;
- Industrial (density = 25 PE/ha): 10.9 equivalent du per ha; and
- Institutional (density = 50 PE/ha): 21.7 equivalent du per ha.

Based on the above assumptions and Table 2-4, a total equivalent dwelling units was calculated to be 639 du, as shown in Table 5-1.

Table 5-1: Total Equivalent Dwelling Units

Land Use	Base Units	20-Year Growth	Equivalency Per du	Equivalent du
Low Density Residential (< 40 du/ha)	du	273	1	273
Medium Density Residential ($\geq$ 40, < 90 du/ha)	du	179	0.91	163
High Density Residential ($\geq$ 90 du/ha)	du	148	0.74	109
Commercial	ha	1	26.1	26
Industrial	ha	5	10.9	58
Institutional	ha	1	21.7	11
Total Equivalent Dwelling Units				639

5.2 Cost Sharing for All New Developments

The required capital upgrades (PS1, GM1, and SL1) is assumed to be funded by all new developments connecting to the Town's sanitary sewer system in the next 20 years, regardless of their locations being inside or outside of Town boundaries. This is because the infrastructure upgrades were sized to handle flows to 40 years' growth projection which has a total in-Town growth of 3,275 PE. This is roughly equivalent to a combination of the Town's 20-year growth projection (1,417 PE) and the outside Town growth projection (1,964 PE). A per-du cost sharing rate was derived to be \$17,300, as displayed below:



Capital Item	Capital Costs (\$)
PS1	\$2,400,000
GM1	\$60,000
SL1	\$8,600,000
Sum	\$11,060,000
Total du	639
Per-du Cost	\$17,300

To apply this rate, a proposed new development should be converted to equivalent dwelling units first. For example, a proposed 50-unit townhouse development (assuming a density of 60 du/ha) is calculated to be 45.5 equivalent dwelling units. A total contribution of \$787,000 is then obtained.

5.3 Additional Cost-Sharing for Outside Town Users

Additional upgrades will be on Cottonwood Drive, which are triggered by new developments in OIB. The per-du cost for the new developments in OIB was calculated to be the total upgrading costs (GM2, \$550,000) divided by the least remaining capacity in the sewer that triggers the entire 250 mm dia. sewer upgrade, which is 931 PE or 405 equivalent du, based on the Town's long range future development model (refer to Figure 3-2). In such a way, adequate fund can be collected from the new contributors of wastewater in OIB prior to the point when upgrades are required. The derived cost-sharing rate will be applied to the new developments in OIB in addition to the cost-sharing rate for all developments. The cost-sharing rate for new developments in OBI is calculated to be \$1,360 per equivalent du.

5.4 Limited usage of Cost Sharing Rate

As indicated above, a per-du cost of \$17,300 has been calculated based on the findings of suggested improvements in this study. It is recommended that this rate not be used directly as a DCC (Development Cost Charge) to charge land owners for development as this rate does not include a contribution/investment consideration from the residents for these assets.

Also, KWL understands that currently the Town charges \$8,000/unit for development applications when connecting a new sewer service. KWL also understands that the Town currently allocated approximately \$500,000/year from the tax base for capital improvements expenditures. The improvements suggested in this report are considered to fall into this category. The capital improvements expenditure budget has not been accounted for in establishing the cost per-du rate above.

KWL suggests the Town perform, a more comprehensive analysis of their sewer DCC rate while, at a minimum, consider the current DCC's, the capital improvement expenditures budget, the assets of future improvements, future costs for land acquisition and reclaimed water system expansion, and the cost per-du rate as established in this report.



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Section 6

Conclusions



6. Conclusions

KWL has performed a comprehensive capacity assessment analysis to the Town of Osoyoos' Sanitary Sewer System. The analysis was split into two categories: The trunk sewer capacity assessment and the waste water treatment assessment. Before the two assessments could effectively be accomplished, a population and growth projection of the Town and surrounding areas was also completed. These projections were necessary in order to establish a timeline for requisite improvements to the systems as identified in the report.

6.1 Population Projections

The population projections looked at both the in-town and out-of-town factors of all residential, commercial, institutional, and seasonal residents of Osoyoos. Looking at trends and the Town's development infill appraisal, KWL was able to establish the effective population for the 0, 5, 10, 15, and 20 year horizons. Table 2-4 and Table 2-5 of section 2 summarize our population projections.

With these populations, KWL was able to apply their relative demand on the sanitary sewer systems to determine what improvements are necessary now and triggered in the future.

6.2 Trunk Sewer Capacity Assessment

With the population information and record drawings obtained from Osoyoos and TRUE, KWL developed a trunk sewer model for both existing and future development scenarios (Figure 3-1 and 3-2). A base sanitary Flow loading rate was established at 230 L/PE/day and used to create the trunk sewer hydraulic assessment.

Infiltration and Inflow (I&I) rates were applied to the model as well. Although KWL estimated that the overall I&I rate is 0.18 L/s/ha, the design criteria I&I rate of 0.6 L/s/ha was used as the storm period and duration to establish this I&I based on pump station operations was unknown. Using the 0.6 L/s/ha is considered overly conservative for the purpose of this study.

A computer desktop spreadsheet model was developed for each trunk sewer system, which calculates peak flows within each of the subject sewers and compares the peak flows with the calculated pipe capacities. The following parameters were used to develop the spreadsheet model:

Loading

- Base Sanitary Flow (BSF): 230 L/PE/day (derived in this study)
- Peaking Factor: Harmon formula (per Bylaw No.1100)
- I&I: 15,600 L/ha/day or 0.18 L/s/ha (estimated in this study)

Hydraulic Calculation

- Head Loss Equation: Manning's (gravity), Hazen-Williams (forcemain)
- Roughness Coefficient: 0.012 for PVC gravity main, 0.013 for AC / concrete gravity main, 120 for forcemain

Capacity Analysis

- Loading Condition: Peak Domestic Flow plus I&I
- Maximum d/D ratio: 0.82
- Maximum Q/Qfull ratio: 1.0



The spreadsheet models for existing and long range future development peak waste water flows (PWWF) were established with the above information and were compared to the existing infrastructure capacities. A number of capital improvements were identified as a result of these as identified in Table 3-3. A summary of the related trunk sewer capital improvement costs and their respective timing is illustrated in Table 3-4.

6.3 Wastewater Treatment Capacity Assessment

Utilizing the population projections and information provided by TRUE and Osoyoos as well as establishing assumptions, an assessment of the capacity of the Osoyoos wastewater treatment plant was completed. Sufficient information regarding the design and operation of the aeration system was not available at the time of writing of this report for a complete assessment of the biological treatment capacity of the three aerated lagoon cells. KWL was able to assess the hydraulic capacity of the existing treatment plant by calculating the existing Hydraulic and Solid Retention Times (HRT and SRT) as identified in Table 4-1. It was noted that other than the depths of the storage/aeration cells, the operating characteristics of the lagoon system are not typical. The projections of wastewater flow and loading for the town based on in-town growth is illustrated in Table 4-2.

All operational, processes, and capacities of the treatment facilities were assessed including the discharge methods. As noted in Table 4-3, there are a number of park and golf club areas that are used to discharge the reclaimed water.

Surplus Wastewater

Based on current conditions and future development, the current operating processes will yield a problematic surplus of treated wastewater if there is no additional irrigation to eliminate the surplus during the summer irrigation season. Table 4-5 shows how this surplus will happen in the next 20 years in hot/dry, average, and cold/wet seasons. Regardless of the season, an annual balance of the flows will not be achieved under the current conditions.

Additional land area that can accept reclaimed water could help reduce the accumulation in the storage basins. Alternative methods of effluent disposal such as the disposal of effluent to Osoyoos Lake could reduce the quantity placed in storage in the long-term and alleviate the need for effluent storage.

Dealing with Surplus Wastewater

Based on the capacity assessment presented in the preceding sections, over the next 20 years there will be an increasing quantity of effluent that must be treated and disposed.

Assuming that the current strategy of winter effluent storage and summer reclamation of effluent for irrigation is continued, there will be a need to build additional storage cells and/or find additional land area for irrigation. It should be noted that KWL has not included an estimate of probable cost for expansion(s) to the reclaimed water distribution system. These costs are considered to be too variable to estimate for this assignment. Osoyoos will need to consider this aspect in addition to the cost estimates presented in this report.

Effluent disposal to water may be an alternative to irrigation of reclaimed water. However, this would require a significant upgrade of the treatment plant from lagoons to a tertiary treatment plant, as well as construction of a new outfall from the treatment plant to Osoyoos Lake. Considering the location of the existing lagoon system in relation to the lake and the size of the Town, this method is not feasible.

KWL suggests Osoyoos to expand the effluent storage lagoon as detailed and estimated in Table 4-6. This work for the capital improvement project includes the construction of a new lined effluent storage cell for an additional volume of 300,000 m³, piping and appurtenances to connect the new cell to the



effluent pumping system, and expansion of the chlorine contact tank. For planning purposes this upgrade has been assumed to occur in Year 10 and is based on having some 15 ha of additional land area for irrigation before Year 10.

6.4 Cost Sharing Opinion

To fund the proposed upgrades, a per-du (dwelling unit)) cost sharing approach can be used. Based on the assumptions made in Table 5-1, a cost-sharing rate of \$17,300 per equivalent du can be applied to all new developments connecting to the Town's sanitary sewer system in the next 20 years, regardless of their locations being inside or outside of Town boundaries. For the new developments in OIB, an additional cost-sharing rate of \$1,360 per equivalent du should be applied.



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consulting engineers

Section 7

Report Submission



7. Report Submission

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Statement of Limitations

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This document represents KWL's best professional judgement based on the information available at the time of its completion and as appropriate for the project scope of work. Services performed in developing the content of this document have been conducted in a manner consistent with that level and skill ordinarily exercised by members of the engineering profession currently practising under similar conditions. No warranty, express or implied, is made.

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Revision History

Revision #	Date	Status	Revision	Author
0	March 17, 2014	DRAFT		WW/ME
1	March 31, 2014	DRAFT	Added Executive Summary and included some minor changes	WW/ME
2	April 23, 2014	DRAFT	Revised report based on discussions with council and staff	WW/ME
3	May 1, 2014	DRAFT	Revised document wording and formatting	WW/ME
4	May 8, 2014	FINAL	Minor formatting, Final issued to Client	RR

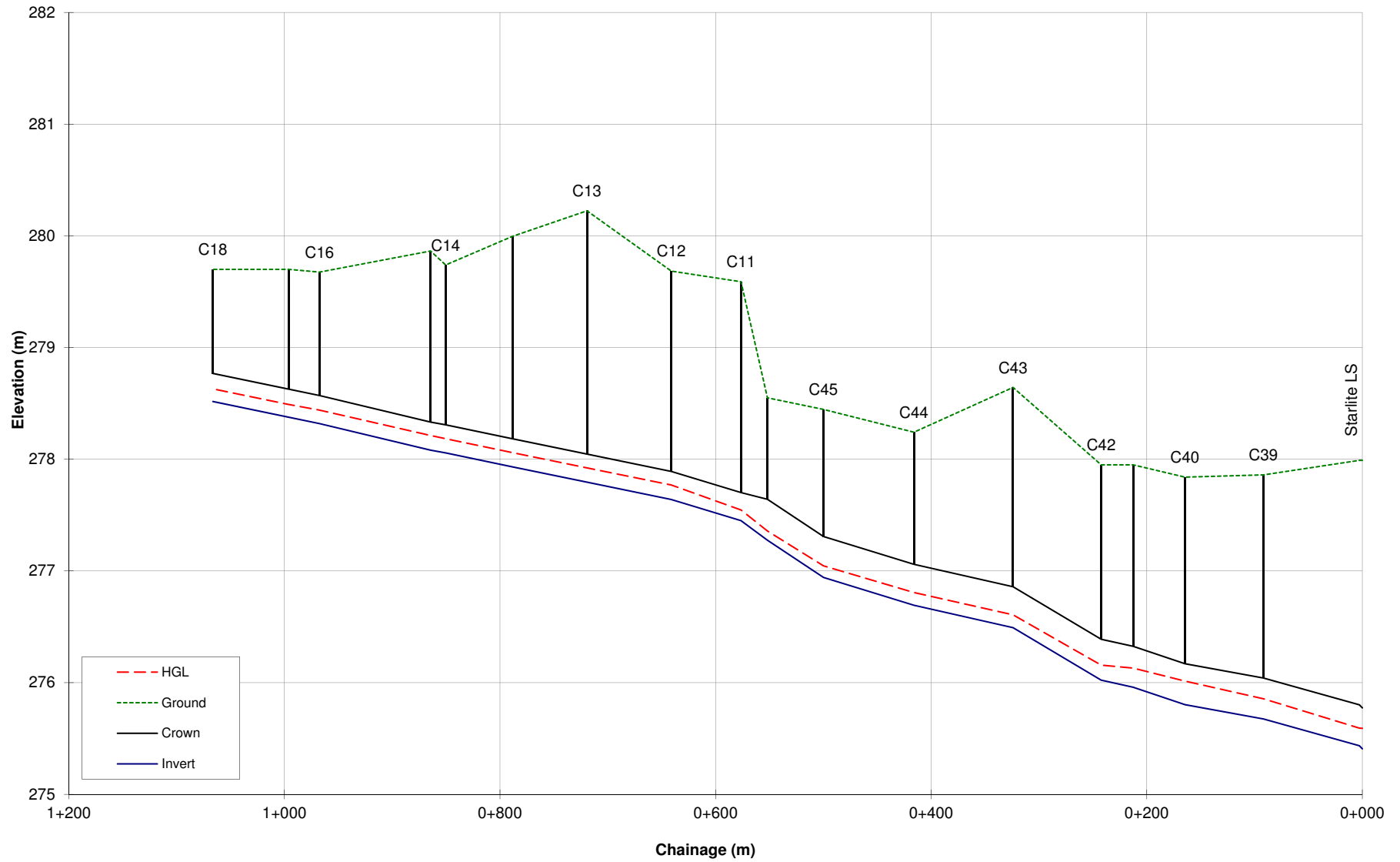


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Appendix A

Trunk Sewer Hydraulic Calculations and Profile

**OIB Trunk Sewer System Profile
Existing PWWF**



Sanitary Sewer Capacity Assessment

Collection Pipe Spreadsheet Model

KWL No. 3233.001

filespec:

ft to m

in to mm

0.3048

25.4

n =

n =

0.012 PVC

0.013 AC

RDII

GW1

17,520 L/halday

1,200 L/halday

Per unit lot size

Per household siz

Peak Factor

Per unit PWWF

0.05 ha

2.1 pop/unit

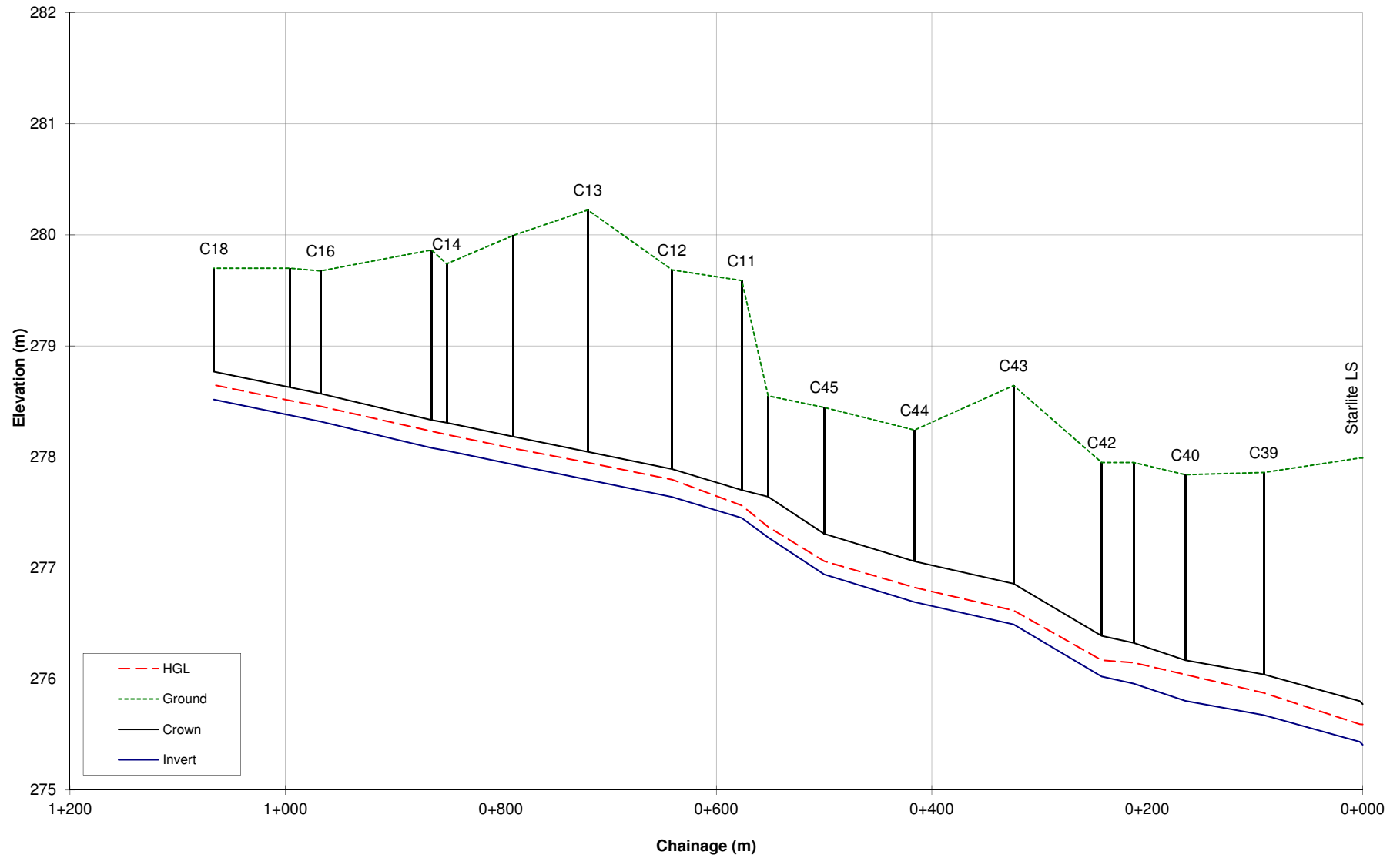
3.5

0.0304 L/s/unit

OIB Trunk Sewer - Long Range Future Scenario

OIB Trunk Sewer - Long Range Future Scenario																														per/capita flow										230 L/cap/d									
Pipe #	u/s MH	d/s MH	Dia mm	Material	Year Const	Mannings	Length (m)	u/s Invert (m)	d/s Invert (m)	u/s Rim (m)	d/s Rim (m)	Ref As Built #	Slope (%)	Velocity (m/s)	Capacity (Q) L/s	Trib PE Pop	u/s Pop	TOTAL Pop	Pump Flow (L/s)	ADWF (L/s)	Harmon (P.F.)	PDWF (L/s)	Trib Area	RDII Rate (L/h/d)	GW1 Rate (L/h/d)	Trib I&I (L/s)	u/s I&I (L/s)	Total I&I (L/s)	PWWF (L/s)	ADWF/QI (I) (%)	PDWF/QI (II) (%)	PWWF/QI (III) (%)	Comments (I)	Comments (II)	Comments (III)	Remaining Capacity (L/s)	Remaining Capacity (units)												
<descr>	<descr>	<descr>	<input>	<descr>	<desc>	'n'	<input>	<input>	<input>			<desc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<input>	<calc>	<calc>	<calc>	<calc>	<ref>	<ref>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>				<calc>	<calc>											
Add Branch Flow	C18	C18	C17	251 ?		0.012	70.30	278.52	278.38	279.70		GIS	0.2%	0.589	29.3	1160		1160	0	3.1	3.8	11.6	19.3	17,520	1,200	4.2		4.2	15.8	10.6%	39.6%	53.9%	OK	OK	OK	13.5	443												
	C17	C17	C16	251 ?		0.012	28.61	278.38	278.32	279.70		GIS	0.2%	0.589	29.3	15	1160	1174	0	3.1	3.8	11.7	0.3	17,520	1,200	0.1	4.2	4.2	16.0	10.7%	40.1%	54.6%	OK	OK	OK	13.3	437												
	C16	C16	C15	251 PVC SDR35		0.012	103.02	278.32	278.08	279.68		279.86 Strong Lamb and Nelson Ltd, Drawing No K 10 4205	0.23%	0.633	31.4	138	1174	1312	0	3.5	3.7	13.0	4.6	17,520	1,200	1.0	4.2	5.2	18.2	11.1%	41.4%	58.0%	OK	OK	OK	13.2	434												
	C15	C15	C14	251 PVC SDR35		0.012	14.02	278.08	278.06	279.86		279.74 Strong Lamb and Nelson Ltd, Drawing No K 10 4205	0.17%	0.549	27.3		1312	1312	0	3.5	3.7	13.0		17,520	1,200	0.0	5.2	5.2	18.2	12.8%	47.6%	66.8%	OK	OK	OK	9.1	298												
	C14	C14	C13a	251 PVC SDR35		0.012	61.87	278.06	277.93	279.74		280.00 Strong Lamb and Nelson Ltd, Drawing No K 10 4205	0.20%	0.592	29.4	20	1312	1332	0	3.5	3.7	13.2	0.4	17,520	1,200	0.1	5.2	5.3	18.5	12.1%	44.8%	62.9%	OK	OK	OK	10.9	359												
	C13a	C13a	C13	251 PVC SDR35		0.012	69.19	277.93	277.79	280.00		280.22 Strong Lamb and Nelson Ltd, Drawing No K 10 4205	0.20%	0.587	29.1		1332	1332	0	3.5	3.7	13.2		17,520	1,200	0.0	5.3	5.3	18.5	12.2%	45.2%	63.5%	OK	OK	OK	10.6	350												
	C13	C13	C12	251 PVC SDR35		0.012	78.25	277.79	277.64	280.22		279.68 Strong Lamb and Nelson Ltd, Drawing No K 10 4205	0.20%	0.587	29.2	171	1332	1503	0	4.0	3.7	14.7	1.2	17,520	1,200	0.3	5.3	5.6	20.3	13.7%	50.5%	69.6%	OK	OK	OK	8.9	291												
	C12	C12	C11	251 PVC SDR35		0.012	64.65	277.64	277.51	279.68		279.59 Strong Lamb and Nelson Ltd, Drawing No K 10 4205	0.20%	0.586	29.1	30	1503	1533	0	4.1	3.7	15.0	0.4	17,520	1,200	0.1	5.6	5.7	20.7	14.0%	51.5%	71.0%	OK	OK	OK	8.4	278												
	C11	C11	C46	251 PVC DR35	2006	0.012	24.50	277.45	277.31	279.59		278.55 True Consulting Drawing No 302-1072-01/02	0.59%	1.015	50.4		1533	1533	0	4.1	3.7	15.0		17,520	1,200	0.0	5.7	5.7	20.7	8.1%	29.7%	41.0%	OK	OK	OK	29.7	978												
	C46	C46	C45	366 PVC DR35		0.012	52.00	277.28	276.96	278.55		278.45 True Consulting Drawing No 302-1072-01/02	0.60%	1.314	138.5		1533	1533	0	4.1	3.7	15.0		17,520	1,200	0.0	5.7	5.7	20.7	2.9%	10.8%	14.9%	OK	OK	OK	117.9	3,877												
	C45	C45	C44	366 PVC DR35		0.012	83.90	276.94	276.71	278.45		278.24 True Consulting Drawing No 302-1072-01/02	0.27%	0.885	93.3	73	1533	1606	0	4.3	3.7	15.6	0.8	17,520	1,200	0.2	5.7	5.9	21.5	4.6%	16.8%	23.0%	OK	OK	OK	71.8	2,362												
	C44	C44	C43	366 PVC DR35		0.012	91.95	276.69	276.51	278.24		278.64 True Consulting Drawing No 302-1072-01/02	0.20%	0.751	79.2	29	1606	1635	0	4.4	3.7	15.9	0.4	17,520	1,200	0.1	5.9	5.9	21.8	5.5%	20.1%	27.6%	OK	OK	OK	57.4	1,888												
	C43	C43	C42	366 PVC DR35	2009	0.012	81.80	276.49	276.05	278.64		277.95 True Consulting Drawing No 302-1072-01/02	0.54%	1.242	131.0	901	1635	2536	0	6.8	3.5	23.7	16.7	17,520	1,200	3.6	5.9	9.6	33.2	5.2%	18.1%	25.4%	OK	OK	OK	97.7	3,215												
	C42	C42	C41	366 PVC DR35	2009	0.012	29.60	276.02	275.93	277.95		277.95 True Consulting Drawing No 302-1072-01/02	0.31%	0.949	100.1	73	2536	2609	0	6.9	3.5	24.3	0.6	17,520	1,200	0.1	9.6	9.7	34.0	6.9%	24.2%	33.9%	OK	OK	OK	66.1	2,175												
	C41	C41	C40	366 PVC DR35	2009	0.012	48.10	275.96	275.83	277.95		277.84 True Consulting Drawing No 302-1072-01/02	0.26%	0.867	91.4	1169	2609	3778	0	10.1	3.4	33.7	20.5	17,520	1,200	4.4	9.7	14.1	47.9	11.0%	36.9%	52.4%	OK	OK	OK	43.5	1,431												
	C40	C40	C39	366 PVC DR35	2009	0.012	72.80	275.80	275.71	277.84		277.86 True Consulting Drawing No 302-1072-01/02	0.13%	0.621	65.5	166	3778	3944	0	10.5	3.3	35.1	0.5	17,520	1,200	0.1	14.1	14.2	49.3	16.0%	53.5%	75.2%	OK	OK	EXCEEDS 75%	16.2	534												
	C39	C39	C1	366 PVC DR35	2009	0.012	89.00	275.68	275.47	277.86		277.99 True Consulting Drawing No 302-1072-01/02	0.24%	0.823	86.7	97	3944	4041	0	10.8	3.3	35.8	0.9	17,520	1,200	0.2	14.2	14.4	50.3	12.4%	41.3%	57.9%	OK	OK	OK	36.5	1,200												
	C1	C1	Starlite LS	366 PVC DR35		0.012	2.75	275.44	275.41	277.99		277.99 True Consulting Drawing No 302-1072-01/02	1.0%	1.678	176.9		4041	4041	0	10.8	3.3	35.8		17,520	1,200	0.0	14.4	14.4	50.3	6.1%	20.2%	28.4%	OK	OK	OK	126.7	4,167												

OIB Trunk Sewer System Profile Long Range Future PWWF



Sanitary Sewer Capacity Assessment
Collection Pipe Spreadsheet Model
KWL No. 3233.001
filespec:

ft to m	0.3048
in to mm	25.4
n =	0.012 PVC
n =	0.013 AC

0.012 PVC
0.013 AC

RDII	17,520 L/ha/day
GWI	1,200 L/ha/day

17,520 L/ha/day
1,200 L/ha/day

Per unit lot size	0.05 ha
Per household size	2.1 pop/unit
Peak Factor	3.5
Per unit PWWF	0.0304 L/s/unit

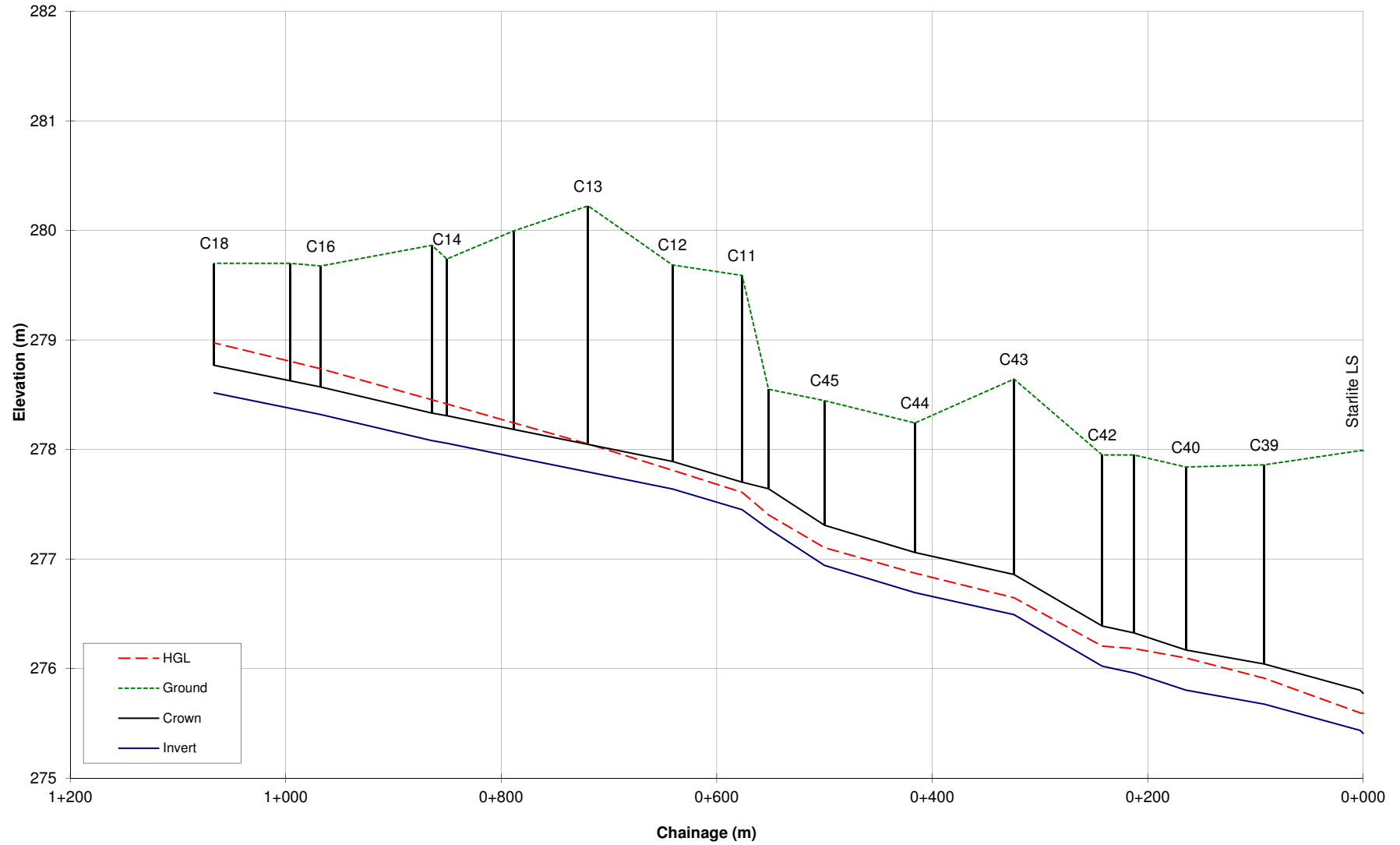
0.05 ha
2.1 pop/unit
3.5
0.0304 L/s/unit

per/capita flow **230** L/cap/d

OIB Trunk Sewer - Long Range Future + OIB Growth Scenario

	Pipe #	u/s MH	d/s MH	Dia mm	Material	Year Const	Mannings	Length (m)	u/s Invert (m)	d/s Invert (m)	u/s Rim (m)	d/s Rim (m)	Ref As Built #	Slope (%)	Velocity (m/s)	Capacity (Q) L/s	Trib PE Pop	u/s Pop	TOTAL Pop	Pump Flow (L/s)	ADWF (L/s)	Harmon (P.F.)	PDWF (L/s)	Trib Area	RDI Rate (L/h/d)	GWl Rate (L/h/d)	Trib I&I (L/s)	u/s I&I (L/s)	Total I&I (L/s)	PWWF (L/s)	ADWF/Q ^I (%)	PDW/Q ^I (%)	PWWF/Q ^I (%)	Comments (I)	Comments (II)	Comments (III)	Remaining Capacity (L/s)	Remaining Capacity (units)
	<descr>	<descr>	<descr>	<input>	<descr>	<desc>	'n'	<input>	<input>	<input>	<input>	<input>	<desc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<input>	<calc>	<calc>	<calc>	<calc>	<ref>	<ref>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>
Main Branch Flow	C18	C18	C17	251 ?			0.012	70.30	278.52	278.38	279.70	GIS		0.2%	0.589	29.3	2355		2355	0	6.3	3.5	22.1	45.3	17,520	1,200	9.8		9.8	31.9	21.4%	75.6%	109.2%	OK	EXCEEDS 75%	EXCEEDS CAP.	-2.7 -	88
	C17	C17	C16	251 ?			0.012	28.61	278.38	278.32	279.70	GIS		0.2%	0.589	29.3	15	2355	2370	0	6.3	3.5	22.3	0.3	17,520	1,200	0.1	9.8	9.9	32.1	21.6%	76.0%	109.8%	OK	EXCEEDS 75%	EXCEEDS CAP.	-2.9 -	94
	C16	C16	C15	251 PVC SDR35			0.012	103.02	278.32	278.08	279.68	279.86 Strong Lamb and Nelson Ltd, Drawing No K 10 4205		0.23%	0.633	31.4	138	2370	2508	0	6.7	3.5	23.4	4.6	17,520	1,200	1.0	9.8	10.9	34.3	21.2%	74.5%	109.1%	OK	OK	EXCEEDS CAP.	-2.8 -	94
	C15	C15	C14	251 PVC SDR35			0.012	14.02	278.08	278.06	279.86	279.74 Strong Lamb and Nelson Ltd, Drawing No K 10 4205		0.17%	0.549	27.3		2508	2508	0	6.7	3.5	23.4		17,520	1,200	0.0	10.9	10.9	34.3	24.5%	85.8%	125.6%	OK	EXCEEDS 75%	EXCEEDS CAP.	-7.0 -	230
	C14	C14	C13a	251 PVC SDR35			0.012	61.87	278.06	277.93	279.74	280.00 Strong Lamb and Nelson Ltd, Drawing No K 10 4205		0.20%	0.592	29.4	20	2508	2528	0	6.7	3.5	23.6	0.4	17,520	1,200	0.1	10.9	11.0	34.5	22.9%	80.2%	117.5%	OK	EXCEEDS 75%	EXCEEDS CAP.	-5.1 -	160
	C13a	C13a	C13	251 PVC SDR35			0.012	69.19	277.93	277.79	280.00	280.22 Strong Lamb and Nelson Ltd, Drawing No K 10 4205		0.20%	0.587	29.1		2528	2528	0	6.7	3.5	23.6		17,520	1,200	0.0	11.0	11.0	34.5	23.1%	80.9%	118.6%	OK	EXCEEDS 75%	EXCEEDS CAP.	-7.4 -	160
	C13	C13	C12	251 PVC SDR35			0.012	78.25	277.79	277.69	280.22	279.68 Strong Lamb and Nelson Ltd, Drawing No K 10 4205		0.20%	0.587	29.2	171	2528	2698	0	7.2	3.5	25.0	1.2	17,520	1,200	0.3	11.0	11.2	36.2	24.6%	85.7%	124.2%	OK	EXCEEDS 75%	EXCEEDS CAP.	-5.1 -	232
	C12	C11	251 PVC SDR35				0.012	64.65	277.69	277.51	279.68	279.59 Strong Lamb and Nelson Ltd, Drawing No K 10 4205		0.20%	0.586	29.1	30	2698	2729	0	7.3	3.5	25.3	0.4	17,520	1,200	0.1	11.2	11.3	36.6	24.8%	86.7%	125.6%	OK	EXCEEDS 75%	EXCEEDS CAP.	-5.5 -	235
	C11	C11	251 PVC DR35			2006	0.012	24.50	277.69	277.59	279.59	278.55 True Consulting Drawing No 302-1072-01-02		0.10%	0.513	25.9		2729	2729	0	5.0	3.3	25.3		17,520	1,200	0.0	11.3	11.3	37.2	14.4%	72.0%	121.3%	OK	OK	OK	10.0 -	454
	C46	C46	366 PVC DR35				0.012	52.00	277.28	276.96	278.55	278.45 True Consulting Drawing No 302-1072-01-02		0.60%	1.314	138.5		2729	2729	0	7.3	3.5	25.3		17,520	1,200	0.2	11.3	11.3	36.6	5.2%	18.2%	26.4%	OK	OK	OK	102.0	345
	C45	C45	C44	366 PVC DR35			0.012	83.90	276.94	276.71	278.45	278.24 True Consulting Drawing No 302-1072-01-02		0.27%	0.885	93.3	73	2729	2801	0	7.5	3.5	25.9	0.8	17,520	1,200	0.2	11.3	11.5	37.3	8.0%	27.7%	40.0%	OK	OK	OK	55.9	1,840
	C44	C44	C43	366 PVC DR35			0.012	91.95	276.69	276.51	278.24	278.04 True Consulting Drawing No 302-1072-01-02		0.20%	0.751	79.2	29	2801	2830	0	7.5	3.5	26.1	0.4	17,520	1,200	0.1	11.5	11.6	37.7	9.5%	32.9%	47.6%	OK	OK	OK	41.5	1,367
	C43	C43	C42	366 PVC DR35		2009	0.012	81.80	276.49	276.05	278.64	277.95 True Consulting Drawing No 302-1072-01-02		0.54%	1.242	131.0	901	2830	3732	0	9.9	3.4	33.4	16.7	17,520	1,200	3.6	11.6	15.2	48.6	7.6%	25.5%	37.1%	OK	OK	OK	82.4	2,710
	C42	C42	C41	366 PVC DR35		2009	0.012	29.60	276.02	275.93	277.95	277.95 True Consulting Drawing No 302-1072-01-02		0.31%	0.949	100.1	73	3732	3804	0	10.1	3.4	34.0	0.6	17,520	1,200	0.1	15.2	15.3	49.3	10.1%	33.9%	49.2%	OK	OK	OK	50.8	1,671
	C41	C41	C40	366 PVC DR35		2009	0.012	48.10	275.96	275.83	277.95	277.84 True Consulting Drawing No 302-1072-01-02		0.26%	0.867	91.4	1169	3804	4973	0	13.2	3.2	43.0	20.5	17,520	1,200	4.4	15.3	19.8	62.8	14.5%	47.0%	68.7%	OK	OK	OK	28.6	942
	C40	C40	C39	366 PVC DR35		2009	0.012	72.80	275.80	275.71	277.84	277.86 True Consulting Drawing No 302-1072-01-02		0.13%	0.621	65.5	166	4973	5139	0	13.7	3.2	44.2	0.5	17,520	1,200	0.1	19.8	19.9	64.1	20.9%	67.5%	97.9%	OK	OK	EXCEEDS 75%	1.4	46
	C39	C39	C1	366 PVC DR35		2009	0.012	89.00	275.68	275.47	277.86	277.99 True Consulting Drawing No 302-1072-01-02		0.24%	0.823	86.7	97	5139	5236	0	13.9	3.2	45.0	0.9	17,520	1,200	0.2	19.9	20.1	65.0	16.1%	51.8%	75.0%	OK	OK	OK	21.7	714
	C38	C31	Starlite LS	366 PVC DR35		2009	0.012	2.75	275.44	275.41	277.99	277.99 True Consulting Drawing No 302-1072-01-02		1.0%	1.678	176.9		5236	5236	0	13.9	3.2	45.0		17,520	1,200	0.0	20.1	20.1	65.0	7.9%	25.4%	36.8%	OK	OK	OK	111.9	3,681

OIB Trunk Sewer System Profile Future plus OIB Growth PWWF



Sanitary Sewer Capacity Assessment
Collection Pipe Spreadsheet Model
KWL No. 3233.001
filespec:

ft to m
in to mm

n =
n =

0.3048
25.4

0.012 PVC
0.013 AC

RDI
GWI

17,520 L/ha/day
1,200 L/ha/day

Per unit lot size
Per household size
Peak Factor
Per unit PWWF

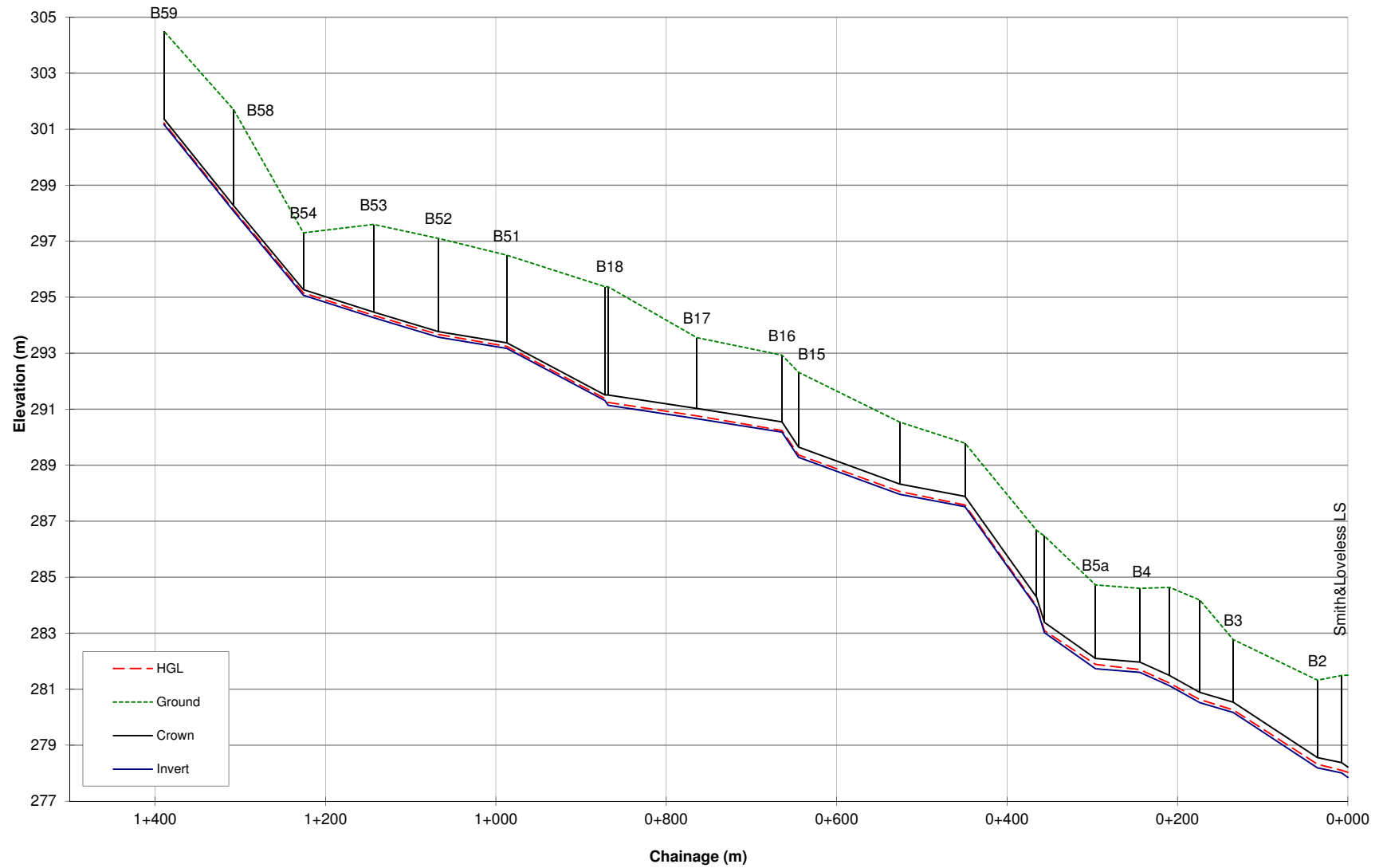
0.05 ha
2.1 pop/unit
3.5
0.0304 L/s/unit

North West Trunk Sewer - Existing Scenario

per/capita flow 230 L/cap/d

Pipe #	u/s MH	d/s MH	Dia mm	Material	Year Const	Mannings	Length (m)	u/s Invert (m)	d/s Invert (m)	u/s Rim (m)	d/s Rim (m)	Ref As Built #	Slope (%)	Velocity (m/s)	Capacity (Qf) L/s	Trib PE Pop	u/s Pop	TOTAL Pop	Pump Flow (L/s)	ADWF (L/s)	Harmon (P.F.)	PDWF (L/s)	Trib Area	RDI Rate (L/ha/d)	GWl Rate (L/ha/d)	Trib I&I (L/s)	u/s I&I (L/s)	Total I&I (L/s)	PWWF (L/s)	ADWF/Qf (I) (%)	PDWF/Qf (II) (%)	PWWF/Qf (III) (%)	Comments (I)	Comments (II)	Comments (III)	Remaining Capacity (L/s)	Remaining Capacity (units)
<descr>	<descr>	<descr>	<input>	<descr>	<desc>	<ref>	<input>	<input>	<input>			<desc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<input>	<calc>	<calc>	<calc>	<calc>	<ref>	<ref>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>
B59	B59	B58	200	AC		0.013	81.80	301.17	298.10	304.50	301.70	302-1132-01	3.75%	2.02	63.5	423		423	0	1.1	4.0	4.5	27.3	17,520	1,200	5.9	5.9	5.9	10.4	1.8%	7.1%	16.4%	OK	OK	OK	53.1	1,747
B58	B58	B54	200	AC		0.013	81.90	298.07	295.10	301.70	297.30	302-1132-01	3.63%	1.99	62.5		423	423	0	1.1	4.0	4.5		17,520	1,200	0.0	5.9	5.9	10.4	1.8%	7.2%	16.7%	OK	OK	OK	52.0	1,712
B54	B54	B53	200	AC		0.013	82.30	295.07	294.30	297.30	297.60	302-1132-01	0.94%	1.01	31.7	68	423	491	0	1.3	4.0	5.2	2.1	17,520	1,200	0.5	5.9	6.4	11.6	4.1%	16.4%	36.5%	OK	OK	OK	20.2	663
B53	B53	B52	200	AC		0.013	76.20	294.27	293.60	297.60	297.10	302-1132-01 & 02	0.88%	0.98	30.8	7	491	498	0	1.3	4.0	5.3	0.2	17,520	1,200	0.0	6.4	6.4	11.7	4.3%	17.1%	38.0%	OK	OK	OK	19.1	627
B52	B52	B51	200	AC		0.013	80.30	293.57	293.20	297.10	296.50	302-1132-02	0.46%	0.71	22.3	7	498	505	0	1.3	4.0	5.3	0.2	17,520	1,200	0.0	6.4	6.5	11.8	6.0%	24.0%	53.0%	OK	OK	OK	10.5	344
B51	B51	B18	200	AC		0.013	107.60	293.17	291.34	296.50	295.38	GIS (length)	1.70%	1.36	42.8	12	505	517	0	1.4	4.0	5.5	0.4	17,520	1,200	0.1	6.5	6.5	12.0	3.2%	12.8%	28.1%	OK	OK	OK	30.8	1,012
B18	B18	B18a	366	PVC	2011	0.012	3.77	291.31	291.17	295.38	295.37	302-964	3.71%	3.26	344.1	490	517	1007	0	2.7	3.8	10.2	14.6	17,520	1,200	3.2	6.5	9.7	19.9	0.8%	3.0%	5.8%	OK	OK	OK	324.2	10,666
B18a	B18a	B17	366	PVC	2011	0.012	104.18	291.14	290.69	295.37	293.56	302-964	0.43%	1.11	117.4		1007	1007	0	2.7	3.8	10.2		17,520	1,200	0.0	9.7	9.7	19.9	2.3%	8.7%	17.0%	OK	OK	OK	97.5	3,206
B17	B17	B16	366	PVC	2011	0.012	99.91	290.66	290.21	293.56	292.93	302-964	0.45%	1.14	119.8		1007	1007	0	2.7	3.8	10.2		17,520	1,200	0.0	9.7	9.7	19.9	2.2%	8.5%	16.6%	OK	OK	OK	100.0	3,288
B16	B16	B15	366	PVC	2011	0.012	19.51	290.18	289.31	292.93	292.32	302-964	4.46%	3.58	377.1	29	1007	1037	0	2.8	3.8	10.5	0.9	17,520	1,200	0.2	9.7	9.9	20.4	0.7%	2.8%	5.4%	OK	OK	OK	356.7	11,735
B15	B15	B13a	366	PVC	2011	0.012	118.69	289.28	287.99	292.32	290.54	302-964	1.09%	1.77	186.2		1037	1109	0	3.0	3.8	11.1	2.6	17,520	1,200	0.6	9.9	10.5	21.6	1.6%	6.0%	11.6%	OK	OK	OK	164.6	5,413
B13a	B13a	B13b	366	PVC	2011	0.012	76.50	287.96	287.55	290.54	289.79	302-964	0.54%	1.24	130.7		1109	1109	0	3.0	3.8	11.1		17,520	1,200	0.0	10.5	10.5	21.6	2.3%	8.5%	16.5%	OK	OK	OK	109.1	3,590
B13b	B13b	B13c	366	PVC	2011	0.012	84.14	287.52	283.95	289.79	286.67	302-964	4.24%	3.49	367.8		1109	1109	0	3.0	3.8	11.1		17,520	1,200	0.0	10.5	10.5	21.6	0.8%	3.0%	5.9%	OK	OK	OK	346.2	11,389
B13c	B13c	B13d	366	PVC	2011	0.012	9.16	283.92	283.05	286.67	286.47	302-964	9.50%	5.22	550.3		1109	1109	0	3.0	3.8	11.1		17,520	1,200	0.0	10.5	10.5	21.6	0.5%	2.0%	3.9%	OK	OK	OK	528.7	17,393
B13d	B13d	B5a	366	PVC	2011	0.012	59.98	283.02	281.76	286.47	284.72	302-964	2.10%	2.45	258.8		1109	1109	0	3.0	3.8	11.1		17,520	1,200	0.0	10.5	10.5	21.6	1.1%	4.3%	8.3%	OK	OK	OK	237.2	7,803
B5a	B5a	B4	366	PVC	2011	0.012	52.10	281.73	281.63	284.72	284.60	302-964	0.19%	0.74	78.2	516	1109	1624	0	4.3	3.7	15.8	15.0	17,520	1,200	3.3	10.5	13.7	29.5	5.5%	20.2%	37.8%	OK	OK	OK	48.7	1,602
B4	B4	B4a	366	PVC	2011	0.012	34.95	281.60	281.15	284.60	284.64	302-964	1.29%	1.92	202.6	195	1624	1820	0	4.8	3.6	17.5	3.8	17,520	1,200	0.8	13.7	14.6	32.1	2.4%	8.6%	15.8%	OK	OK	OK	170.5	5,610
B4a	B4a	B4b	366	PVC	2011	0.012	35.29	281.12	280.55	284.64	284.19	302-964	1.6%	2.15	226.9		1820	1820	0	4.8	3.6	17.5		17,520	1,200	0.0	14.6	14.6	32.1	2.1%	7.7%	14.1%	OK	OK	OK	194.9	6,410
B4b	B4b	B3	366	PVC	2011	0.012	39.24	280.52	280.20	284.19	282.77	302-964	0.8%	1.53	161.3		1820	1820	0	4.8	3.6	17.5		17,520	1,200	0.0	14.6	14.6	32.1	3.0%	10.9%	19.9%	OK	OK	OK	129.2	4,249
B3	B3	B2	366	PVC	2011	0.012	98.96	280.17	278.22	282.77	281.32	302-964	2.0%	2.38	250.7	115	1820	1935	0	5.2	3.6	18.5	2.2	17,520	1,200	0.5	14.6	15.0	33.6	2.1%	7.4%	13.4%	OK	OK	OK	217.1	7,142
B2	B2	B2a	366	PVC	2011	0.012	28.25	278.19	278.04	281.32	281.49	302-964	0.5%	1.23	130.1	64	1935	1999	0	5.3	3.6	19.1	1.5	17,520	1,200	0.3	15.0	15.4	34.5	4.1%	14.7%	26.5%	OK	OK	OK	95.7	3,147
B2a	B2a	Smith&Loveless LS	366	PVC	2011	0.012	8.70	278.01	277.85	281.49	281.50	302-964	1.8%	2.30	242.2	162	1999	2161	0	5.8	3.6	20.5	2.6	17,520	1,200	0.6	15.4	15.9	36.4	2.4%	8.5%	15.0%	OK	OK	OK	205.8	6,768

North West Trunk Sewer System Profile
Existing PWWF



Sanitary Sewer Capacity Assessment
Collection Pipe Spreadsheet Model
KWL No. 3233.001
filespec:

ft to m
in to mm

n =
n =

0.3048
25.4

0.012 PVC
0.013 AC

RDII
GWI

17,520 L/ha/day
1,200 L/ha/day

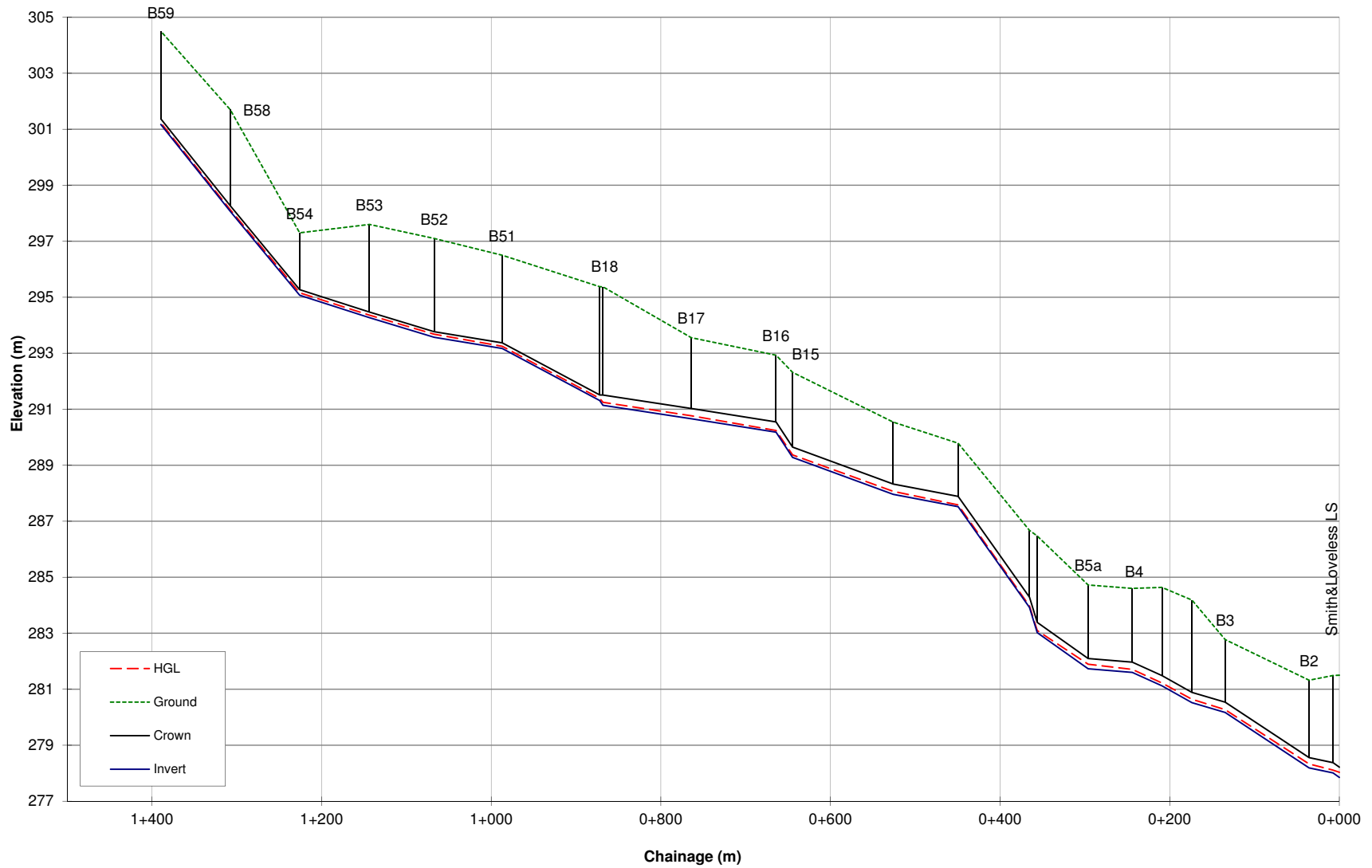
Per unit lot size
Per household size
Peak Factor
Per unit PWWF

0.05 ha
2.1 pop/unit
3.5
0.0304 L/s/unit

North West Sector - Long Range Future Scenario

North West Sector - Long Range Future Scenario																																			per/capita flow		230 Lcap/d																				Remaining Capacity	Remaining Capacity
Pipe #	u/s MH	d/s MH	Dia	Material	Year Const	Mannings 'n'	Length (m)	u/s Invert (m)	d/s Invert (m)	u/s Rim (m)	d/s Rim (m)	Ref As Built #	Slope (%)	Velocity (m/s)	Capacity (Qf) L/s	Trib PE Pop	u/s Pop	TOTAL Pop	Pump Flow (L/s)	ADWF (L/s)	Harmon (P.F.)	PDWF (L/s)	Trib Area	RDII Rate (L/ha/d)	GWl Rate (L/ha/d)	Trib I&I (L/s)	u/s I&I (L/s)	Total I&I (L/s)	PWWF (L/s)	ADWF/Qf (I) (%)	PDWF/Qf (II) (%)	PWWF/Qf (III) (%)	Comments (I)	Comments (II)	Comments (III)	Remaining Capacity (L/s)	Remaining Capacity (units)																					
<descr>	<descr>	<descr>	<input>	<descr>	<desc>	<ref>	<input>	<input>	<input>			<desc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<input>	<calc>	<calc>	<calc>	<calc>	<ref>	<ref>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>																			
B59	B59	B58	200	AC		0.013	81.80	301.17	298.10	304.50	301.70	302-1132-01	3.75%	2.02	63.5	589		589	0	1.6	3.9	6.2	29.5	17,520	1,200	6.4	6.4	12.6	2.5%	9.7%	19.8%	OK	OK	OK	51.0	1,677																						
B58	B58	B54	200	AC		0.013	81.90	298.07	295.10	301.70	297.30	302-1132-01	3.63%	1.99	62.5		589	589	0	1.6	3.9	6.2		17,520	1,200	0.0	6.4	12.6	2.5%	9.9%	20.1%	OK	OK	OK	49.9	1,641																						
B54	B54	B53	200	AC		0.013	82.30	295.07	294.30	297.30	297.60	302-1132-01	0.94%	1.01	31.7	68	589	657	0	1.7	3.9	6.8	2.1	17,520	1,200	0.5	6.4	6.9	13.7	5.5%	21.5%	43.2%	OK	OK	OK	18.0	593																					
B53	B53	B52	200	AC		0.013	76.20	294.27	293.60	297.60	297.10	302-1132-01 & 02	0.88%	0.98	30.8	7	657	664	0	1.8	3.9	6.9	0.2	17,520	1,200	0.0	6.9	6.9	13.8	5.7%	22.5%	44.9%	OK	OK	OK	16.9	557																					
B52	B52	B51	200	AC		0.013	80.30	293.57	293.20	297.10	296.50	302-1132-02	0.46%	0.71	22.3	7	664	671	0	1.8	3.9	7.0	0.2	17,520	1,200	0.0	6.9	6.9	13.9	8.0%	31.3%	62.6%	OK	OK	OK	8.3	274																					
B51	B51	B18	200	AC		0.013	107.60	293.17	291.34	296.50	295.38	GIS (length)	1.70%	1.36	42.8	12	671	683	0	1.8	3.9	7.1	0.4	17,520	1,200	0.1	6.9	7.0	14.1	4.3%	16.6%	33.0%	OK	OK	OK	28.7	943																					
B18	B18	B18a	366	PVC	2011	0.012	3.77	291.31	291.17	295.38	295.37	302-964	3.71%	3.26	344.1	490	683	1173	0	3.1	3.8	11.7	14.6	17,520	1,200	3.2	7.0	10.2	21.9	0.9%	3.4%	6.4%	OK	OK	OK	322.2	10,599																					
B18a	B18a	B17	366	PVC	2011	0.012	104.18	291.14	290.69	295.37	293.56	302-964	0.43%	1.11	117.4		1173	1173	0	3.1	3.8	11.7		17,520	1,200	0.0	10.2	10.2	21.9	2.7%	10.0%	18.7%	OK	OK	OK	95.4	3,140																					
B17	B17	B16	366	PVC	2011	0.012	99.91	290.66	290.21	293.56	292.93	302-964	0.45%	1.14	119.8		1173	1173	0	3.1	3.8	11.7		17,520	1,200	0.0	10.2	10.2	21.9	2.6%	9.8%	18.3%	OK	OK	OK	97.9	3,221																					
B16	B16	B15	366	PVC	2011	0.012	19.51	290.18	289.31	292.93	292.32	302-964	4.46%	3.58	377.1	29	1173	1203	0	3.2	3.7	12.0	0.9	17,520	1,200	0.2	10.2	10.4	22.4	0.8%	3.2%	5.9%	OK	OK	OK	354.7	11,668																					
B15	B15	B13a	366	PVC	2011	0.012	118.69	289.28	287.99	292.32	290.54	302-964	1.09%	1.77	186.2	72	1203	1275	0	3.4	3.7	12.7	2.6	17,520	1,200	0.6	10.4	11.0	23.6	1.8%	6.8%	12.7%	OK	OK	OK	162.5	5,347																					
B13a	B13a	B13b	366	PVC	2011	0.012	76.50	287.96	287.55	290.54	289.79	302-964	0.54%	1.24	130.7		1275	1275	0	3.4	3.7	12.7		17,520	1,200	0.0	11.0	11.0	23.6	2.6%	9.7%	18.1%	OK	OK	OK	107.1	3,523																					
B13b	B13b	B13c	366	PVC	2011	0.012	84.14	287.52	283.95	289.79	286.67	302-964	4.24%	3.49	367.8		1275	1275	0	3.4	3.7	12.7		17,520	1,200	0.0	11.0	11.0	23.6	0.9%	3.4%	6.4%	OK	OK	OK	344.2	11,323																					
B13c	B13c	B13d	366	PVC	2011	0.012	9.16	283.92	283.05	286.67	286.47	302-964	9.50%	5.22	550.3		1275	1275	0	3.4	3.7	12.7		17,520	1,200	0.0	11.0	11.0	23.6	0.6%	2.3%	4.3%	OK	OK	OK	526.7	17,326																					
B13d	B13d	B5a	366	PVC	2011	0.012	59.98	283.02	281.76	286.47	284.72	302-964	2.10%	2.45	258.8		1275	1275	0	3.4	3.7	12.7		17,520	1,200	0.0	11.0	11.0	23.6	1.3%	4.9%	9.1%	OK	OK	OK	235.2	7,737																					
B5a	B5a	B4	366	PVC	2011	0.012	52.10	281.73	281.63	284.72	284.60	302-964	0.19%	0.74	78.2	516	1275	1790	0	4.8	3.6	17.3	15.0	17,520	1,200	3.3	11.0	14.2	31.5	6.1%	22.1%	40.3%	OK	OK	OK	46.7	1,538																					
B4	B4	B4a	366	PVC	2011	0.012	34.95	281.60	281.15	284.60	284.64	302-964	1.29%	1.92	202.6	629	1790	2419	0	6.4	3.5	22.7	6.1	17,520	1,200	1.3	14.2	15.6	38.2	3.2%	11.2%	18.9%	OK	OK	OK	164.4	5,408																					
B4a	B4a	B4b	366	PVC	2011	0.012	35.29	281.12	280.55	284.64	284.19	302-964	1.6%	2.15	226.9		2419	2419	0	6.4	3.5	22.7		17,520	1,200	0.0	15.6	15.6	38.2	2.8%	10.0%	16.6%	OK	OK	OK	188.7	6,208																					
B4b	B4b	B3	366	PVC	2011	0.012	39.24	280.52	280.20	284.19	282.77	302-964	0.8%	1.53	161.3		2419	2419	0	6.4	3.5	22.7		17,520	1,200	0.0	15.6	15.6	38.2	4.0%	14.1%	23.7%	OK	OK	OK	123.0	4,047																					
B3	B3	B2	366	PVC	2011	0.012	98.96	280.17	278.22	282.77	281.32	302-964	2.0%	2.38	250.7	115	2419	2534	0	6.7	3.5	23.6	2.2	17,520	1,200	0.5	15.6	16.0	39.7	2.7%	9.4%	15.8%	OK	OK	OK	211.0	6,941																					
B2	B2	B2a	366	PVC	2011	0.012	28.25	278.19	278.04	281.32	281.49	302-964	0.5%	1.23	130.1	64	2534	2598	0	6.9	3.5	24.2	1.5	17,520	1,200	0.3	16.0	16.4	40.5	5.3%	18.6%	31.2%	OK	OK	OK	89.6	2,947																					
B2a	B2a	Smith&Loveless LS	366	PVC	2011	0.012	8.70	278.01	277.85	281.49	281.50	302-964	1.8%	2.30	242.2	162	2598	2760	0	7.3	3.5	25.5	2.6	17,520	1,200	0.6	16.4	16.9	42.5	3.0%	10.5%	17.5%	OK	OK	OK	199.7	6,570																					

North West Trunk Sewer System Profile
Future PWWF



Sanitary Sewer Capacity Assessment

Collection Pipe Spreadsheet Model

KWL No. 3233.001

filespec:

ft to m

in to mm

n =

n =

0.3048

25.4

0.012 PVC

0.013 AC

RDII

GWl

17,520 L/ha/day

1,200 L/ha/day

Per unit lot size

Per household size

Peak Factor

Per unit PWWF

0.05 ha

2.1 pop/unit

3.5

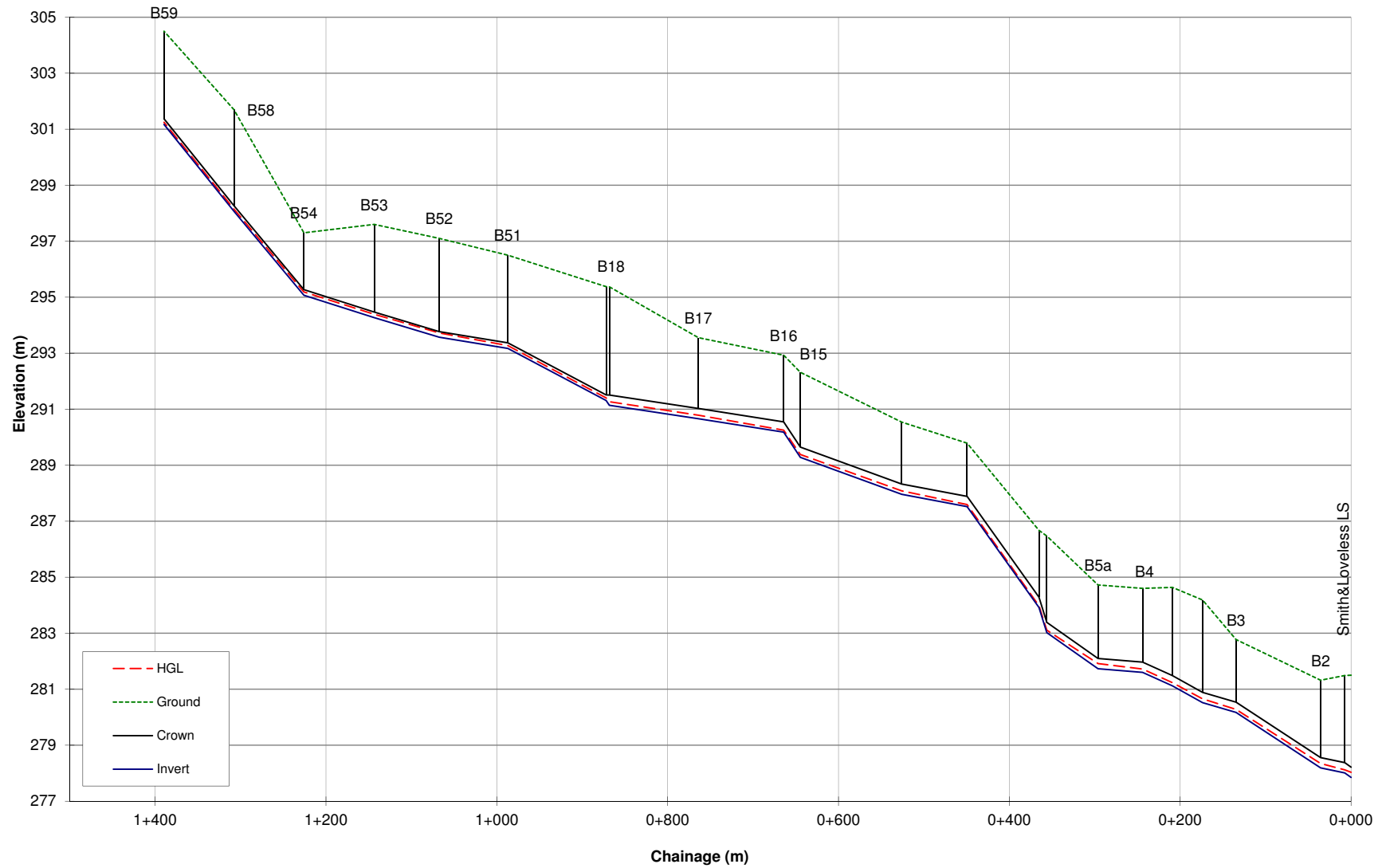
0.0304 L/s/unit

North West Sector - Long Range Future + Electoral Area 'A' Growth Scenario

per/capita flow 230 L/cap/d

Pipe #	u/s MH	d/s MH	Dia mm	Material	Year Const	Mannings 'n'	Length (m)	u/s Invert (m)	d/s Invert (m)	u/s Rim (m)	d/s Rim (m)	Ref As Built #	Slope (%)	Velocity (m/s)	Capacity (Qf) L/s	Trib PE Pop	u/s Pop	TOTAL Pop	Pump Flow (L/s)	ADWF (L/s)	Harmon (P.F.)	PDWF (L/s)	Trib Area	RDII Rate (L/ha/d)	GWl Rate (L/ha/d)	Trib I&I (L/s)	u/s I&I (L/s)	Total I&I (L/s)	PWWF (L/s)	ADWF/Qf (I) (%)	PDWF/Qf (II) (%)	PWWF/Qf (III) (%)	Comments (I)	Comments (II)	Comments (III)	Remaining Capacity (L/s)	Remaining Capacity (units)
<descr>	<descr>	<descr>	<input>	<descr>	<desc>	<ref>	<input>	<input>	<input>			<desc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<input>	<calc>	<calc>	<calc>	<calc>	<ref>	<ref>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>
B59	B59	B58	200	AC		0.013	81.80	301.17	298.10	304.50	301.70	302-1132-01	3.75%	2.02	63.5	956		956	0	2.5	3.8	9.7	53.2	17,520	1,200	11.5		11.5	21.2	4.0%	15.3%	33.4%	OK	OK	OK	42.3	1,391
B58	B58	B54	200	AC		0.013	81.90	298.07	295.10	301.70	297.30	302-1132-01	3.63%	1.99	62.5		956	956	0	2.5	3.8	9.7		17,520	1,200	0.0	11.5	11.5	21.2	4.1%	15.5%	34.0%	OK	OK	OK	41.2	1,356
B54	B54	B53	200	AC		0.013	82.30	295.07	294.30	297.30	297.60	302-1132-01	0.94%	1.01	31.7	68	956	1024	0	2.7	3.8	10.3	2.1	17,520	1,200	0.5	11.5	12.0	22.3	8.6%	32.6%	70.4%	OK	OK	OK	9.4	309
B53	B53	B52	200	AC		0.013	76.20	294.27	293.60	297.60	297.10	302-1132-01 & 02	0.88%	0.98	30.8	7	1024	1031	0	2.7	3.8	10.4	0.2	17,520	1,200	0.0	12.0	12.0	22.4	8.9%	33.8%	73.0%	OK	OK	OK	8.3	273
B52	B52	B51	200	AC		0.013	80.30	293.57	293.20	297.10	296.50	302-1132-02	0.46%	0.71	22.3	7	1031	1039	0	2.8	3.8	10.5	0.2	17,520	1,200	0.0	12.0	12.1	22.6	12.4%	47.1%	101.3%	OK	OK	EXCEEDS CAP.	-0.3	-10
B51	B51	B18	200	AC		0.013	107.60	293.17	291.34	296.50	295.38	GIS (length)	1.70%	1.36	42.8	12	1039	1051	0	2.8	3.8	10.6	0.4	17,520	1,200	0.1	12.1	12.2	22.8	6.5%	24.8%	53.2%	OK	OK	OK	20.0	659
B18	B18	B18a	366	PVC	2011	0.012	3.77	291.31	291.17	295.38	295.37	302-964	3.71%	3.26	344.1	490	1051	1541	0	4.1	3.7	15.1	14.6	17,520	1,200	3.2	12.2	15.3	30.4	1.2%	4.4%	8.8%	OK	OK	OK	313.7	10,320
B18a	B18a	B17	366	PVC	2011	0.012	104.18	291.14	290.69	295.37	293.56	302-964	0.43%	1.11	117.4		1541	1541	0	4.1	3.7	15.1		17,520	1,200	0.0	15.3	15.3	30.4	3.5%	12.8%	25.9%	OK	OK	OK	87.0	2,861
B17	B17	B16	366	PVC	2011	0.012	99.91	290.66	290.21	293.56	292.93	302-964	0.45%	1.14	119.8		1541	1541	0	4.1	3.7	15.1		17,520	1,200	0.0	15.3	15.3	30.4	3.4%	12.6%	25.4%	OK	OK	OK	89.5	2,943
B16	B16	B15	366	PVC	2011	0.012	19.51	290.18	289.31	292.93	292.32	302-964	4.46%	3.58	377.1	29	1541	1570	0	4.2	3.7	15.3	0.9	17,520	1,200	0.2	15.3	15.5	30.8	1.1%	4.1%	8.2%	OK	OK	OK	346.2	11,390
B15	B15	B13a	366	PVC	2011	0.012	118.69	289.28	287.99	292.32	290.54	302-964	1.09%	1.77	186.2	72	1570	1642	0	4.4	3.7	16.0	2.6	17,520	1,200	0.6	15.5	16.1	32.1	2.3%	8.6%	17.2%	OK	OK	OK	154.1	5,069
B13a	B13a	B13b	366	PVC	2011	0.012	76.50	287.96	287.55	290.54	289.79	302-964	0.54%	1.24	130.7		1642	1642	0	4.4	3.7	16.0		17,520	1,200	0.0	16.1	16.1	32.1	3.3%	12.2%	24.5%	OK	OK	OK	98.7	3,246
B13b	B13b	B13c	366	PVC	2011	0.012	84.14	287.52	283.95	289.79	286.67	302-964	4.24%	3.49	367.8		1642	1642	0	4.4	3.7	16.0		17,520	1,200	0.0	16.1	16.1	32.1	1.2%	4.3%	8.7%	OK	OK	OK	335.8	11,045
B13c	B13c	B13d	366	PVC	2011	0.012	9.16	283.92	283.05	286.67	286.47	302-964	9.50%	5.22	550.3		1642	1642	0	4.4	3.7	16.0		17,520	1,200	0.0	16.1	16.1	32.1	0.8%	2.9%	5.8%	OK	OK	OK	518.3	17,049
B13d	B13d	B5a	366	PVC	2011	0.012	59.98	283.02	281.76	286.47	284.72	302-964	2.10%	2.45	258.8		1642	1642	0	4.4	3.7	16.0		17,520	1,200	0.0	16.1	16.1	32.1	1.7%	6.2%	12.4%	OK	OK	OK	226.8	7,459
B5a	B5a	B4	366	PVC	2011	0.012	52.10	281.73	281.63	284.72	284.60	302-964	0.19%	0.74	78.2	516	1642	2158	0	5.7	3.6	20.4	15.0	17,520	1,200	3.3	16.1	19.4	39.8	7.3%	26.1%	50.9%	OK	OK	OK	38.4	1,264
B4	B4	B4a	366	PVC	2011	0.012	34.95	281.60	281.15	284.60	284.64	302-964	1.29%	1.92	202.6	629	2158	2786	0	7.4	3.5	25.7	6.1	17,520	1,200	1.3	19.4	20.7	46.4	3.7%	12.7%	22.9%	OK	OK	OK	156.2	5,138
B4a	B4a	B4b	366	PVC	2011	0.012	35.29	281.12	280.55	284.64	284.19	302-964	1.6%	2.15	226.9		2786	2786	0	7.4	3.5	25.7		17,520	1,200	0.0	20.7	20.7	46.4	3.3%	11.3%	20.5%	OK	OK	OK	180.5	5,938
B4b	B4b	B3	366	PVC	2011	0.012	39.24	280.52	280.20	284.19	282.77	302-964	0.8%	1.53	161.3		2786	2786	0	7.4	3.5	25.7		17,520	1,200	0.0	20.7	20.7	46.4	4.6%	16.0%	28.8%	OK	OK	OK	114.8	3,777
B3	B3	B2	366	PVC	2011	0.012	98.96	280.17	278.22	282.77	281.32	302-964	2.0%	2.38	250.7	115	2786	2902	0	7.7	3.5	26.7	2.2	17,520	1,200	0.5	20.7	21.2	47.9	3.1%	10.6%	19.1%	OK	OK	OK	202.8	6,672
B2	B2	B2a	366	PVC	2011	0.012	28.25	278.19	278.04	281.32	281.49	302-964	0.5%	1.23	130.1	64	2902	2966	0	7.9	3.4	27.2	1.5	17,520	1,200	0.3	21.2	21.5	48.7	6.1%	20.9%	37.4%	OK	OK	OK	81.4	2,678
B2a	B2a	Smith&Loveless LS	366	PVC	2011	0.012	8.70	278.01	277.85	281.49	281.50	302-964	1.8%	2.30	242.2	162	2966	3128	0	8.3	3.4	28.5	2.6	17,520	1,200	0.6	21.5	22.1	50.6	3.4%	11.8%	20.9%	OK	OK	OK	191.6	6,302

North West Trunk Sewer System Profile
Future plus Electoral Area 'A' Growth PWWF



Sanitary Sewer Capacity Assessment
Collection Pipe Spreadsheet Model
KWL No. 3233.001
filespec:

ft to m 0.3048
in to mm 25.4

n = 0.012 PVC
n = 0.013 AC

RDII 17,520 L/ha/day
GWI 1,200 L/ha/day

Per unit lot size 0.05 ha
Per household siz 2.1 pop/unit
Peak Factor 3.5
Per unit PWWF 0.0304 L/s/unit

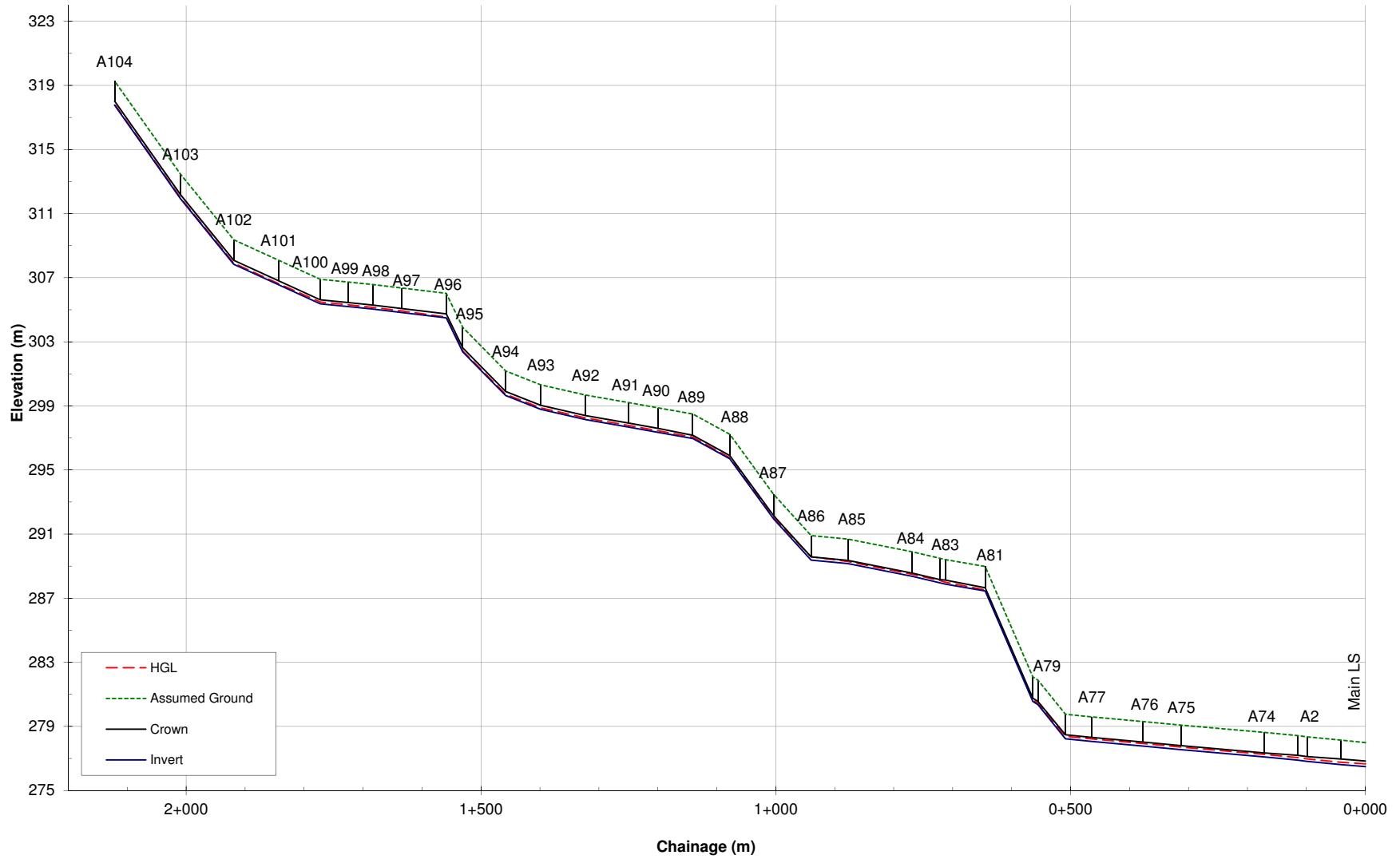
Industrial Park & Airport - Existing Scenario

per/capita flow 230 L/cap/d

Pipe #	u/s MH	d/s MH	Dia mm	Material	Year Const	Mannings 'n'	Length (m)	u/s Invert (m)	d/s Invert (m)	u/s Rim (m)	d/s Rim (m)	Ref As Built #	Slope (%)	Velocity (m/s)	Capacity (Qt) L/s	Trib PE Pop	u/s Pop	TOTAL Pop	Pump Flow (L/s)	ADWF (L/s)	Harmon (P.F.)	PDWF (L/s)	Trib Area	RDII Rate (L/ha/d)	GWII Rate (L/ha/d)	Trib I&I (L/s)	u/s I&I (L/s)	Total I&I (L/s)	PWWF (L/s)	ADWF/Qf (I) (%)	PDWF/Qf (II) (%)	PWWF/Qf (III) (%)	Comments (I)	Comments (II)	Comments (III)	Remaining Capacity (L/s)	Remaining Capacity (units)
<descr>	<descr>	<descr>	<input>	<descr>	<desc>	<ref>	<input>	<input>	<input>			<desc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<input>	<calc>	<calc>	<calc>	<calc>	<ref>	<ref>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>				<calc>	<calc>
Ground elev 323 m @	A104	A103	250 PVC	1980	0.012	112	317.77	311.95	319.27	313.45			5.20%	2.993	146.9	854		854	0	2.3	3.8	8.7	24.4	17,520	1,200	5.3	5.3	14.0	1.5%	5.9%	9.5%	OK	OK	OK	132.9	4,372	
	A103	A102	250 PVC	1980	0.012	90	311.92	307.87	313.45	309.37			4.50%	2.784	136.7		854	854	0	2.3	3.8	8.7		17,520	1,200	0.0	5.3	5.3	14.0	1.7%	6.4%	10.3%	OK	OK	OK	122.6	4,035
	A102	A101	250 PVC	1980	0.012	76	307.84	306.58	309.37	308.08			1.64%	1.681	82.5		854	854	0	2.3	3.8	8.7		17,520	1,200	0.0	5.3	5.3	14.0	2.8%	10.6%	17.0%	OK	OK	OK	68.5	2,253
	A101	A100	250 PVC	1980	0.012	70	306.55	305.40	308.08	306.90			1.64%	1.681	82.5		854	854	0	2.3	3.8	8.7		17,520	1,200	0.0	5.3	5.3	14.0	2.8%	10.6%	17.0%	OK	OK	OK	68.5	2,253
	A100	A99	250 PVC	1993	0.012	48	305.37	305.23	306.90	306.73			0.30%	0.719	35.3	0	854	854	0	2.3	3.8	8.7	0.8	17,520	1,200	0.2	5.3	5.5	14.2	6.4%	24.8%	40.2%	OK	OK	OK	21.1	694
	A99	A98	250 PVC	1993	0.012	41	305.20	305.07	306.73	306.57			0.30%	0.719	35.3		854	854	0	2.3	3.8	8.7		17,520	1,200	0.0	5.5	5.5	14.2	6.4%	24.8%	40.2%	OK	OK	OK	21.1	694
	A98	A97	250 PVC	1993	0.012	49	305.04	304.86	306.57	306.36			0.38%	0.809	39.7		854	854	0	2.3	3.8	8.7		17,520	1,200	0.0	5.5	5.5	14.2	5.7%	22.0%	35.7%	OK	OK	OK	25.5	840
	A97	A96	250 PVC	1993	0.012	76	304.83	304.52	306.36	306.02			0.40%	0.830	40.7		854	854	0	2.3	3.8	8.7		17,520	1,200	0.0	5.5	5.5	14.2	5.6%	21.4%	34.8%	OK	OK	OK	26.6	874
	A96	A95	250 PVC	1993	0.012	28	304.49	302.40	306.02	303.90			7.50%	3.594	176.4		854	854	0	2.3	3.8	8.7		17,520	1,200	0.0	5.5	5.5	14.2	1.3%	5.0%	8.0%	OK	OK	OK	162.2	5,337
	A95	A94	250 PVC	1993	0.012	73	302.37	299.68	303.90	301.18			3.70%	2.524	123.9	129	854	983	0	2.6	3.8	10.0	1.2	17,520	1,200	0.3	5.5	5.7	15.7	2.1%	8.0%	12.6%	OK	OK	OK	108.2	3,561
	A94	A93	250 PVC	1993	0.012	60	299.65	298.81	301.18	300.31			1.40%	1.553	76.2		983	983	0	2.6	3.8	10.0		17,520	1,200	0.0	5.7	5.7	15.7	3.4%	13.1%	20.6%	OK	OK	OK	60.6	1,992
	A93	A92	250 PVC	1979	0.012	76	298.78	298.17	300.31	299.67			0.80%	1.174	57.6	24	983	1007	0	2.7	3.8	10.2	1.0	17,520	1,200	0.2	5.7	5.9	16.1	4.7%	17.7%	27.9%	OK	OK	OK	41.5	1,366
	A92	A91	250 PVC	1979	0.012	73	298.14	297.70	299.67	299.20			0.60%	1.017	49.9	102	1007	1109	0	3.0	3.8	11.1	1.7	17,520	1,200	0.4	5.9	6.3	17.4	5.9%	22.3%	34.9%	OK	OK	OK	32.5	1,069
	A91	A90	250 PVC	1979	0.012	49	297.67	297.38	299.20	298.88			0.60%	1.017	49.9		1109	1109	0	3.0	3.8	11.1		17,520	1,200	0.0	6.3	6.3	17.4	5.9%	22.3%	34.9%	OK	OK	OK	32.5	1,069
	A90	A89	250 PVC	1979	0.012	59	297.35	296.99	298.88	298.49			0.60%	1.017	49.9		1109	1109	0	3.0	3.8	11.1		17,520	1,200	0.0	6.3	6.3	17.4	5.9%	22.3%	34.9%	OK	OK	OK	32.5	1,069
	A89	A88	200 PVC	1979	0.012	63	296.96	295.73	298.49	297.23			1.97%	1.587	49.9	63	1109	1172	0	3.1	3.8	11.7	1.1	17,520	1,200	0.2	6.3	6.5	18.2	6.3%	23.5%	36.5%	OK	OK	OK	31.6	1,041
	A88	A87	200 PVC	1979	0.012	74	295.70	291.98	297.23	293.48			5.00%	2.529	79.5	12	1172	1183	0	3.1	3.8	11.8	0.2	17,520	1,200	0.0	6.5	6.6	18.4	4.0%	14.9%	23.1%	OK	OK	OK	61.1	2,009
	A87	A86	200 PVC	1979	0.012	64	291.95	289.41	293.48	290.91			4.00%	2.262	71.1		1183	1183	0	3.1	3.8	11.8		17,520	1,200	0.0	6.6	6.6	18.4	4.4%	16.6%	25.9%	OK	OK	OK	52.7	1,733
	A86	A85	200 AC	1965	0.013	62	289.38	289.19	290.91	290.69			0.30%	0.572	18.0	293	1183	1476	0	3.9	3.7	14.5	7.4	17,520	1,200	1.6	6.6	8.2	22.6	21.9%	80.6%	126.1%	OK	EXCEEDS 75%	EXCEEDS CAP.	-4.7	-154
	A85	A84	200 AC	1965	0.013	109	289.16	288.39	290.69	289.89			0.70%	0.873	27.4		1476	1476	0	3.9	3.7	14.5		17,520	1,200	0.0	8.2	8.2	22.6	14.3%	52.8%	82.5%	OK	OK	EXCEEDS 75%	4.8	158
	A84	A83	200 AC	1965	0.013	46	288.36	287.99	289.89	289.49			0.80%	0.934	29.3		1476	1476	0	3.9	3.7	14.5		17,520	1,200	0.0	8.2	8.2	22.6	13.4%	49.3%	77.2%	OK	OK	EXCEEDS 75%	6.7	220
	A83	A82	200 AC	1991	0.013	10	287.96	287.91	289.49	289.41			0.57%	0.788	24.8	36	1476	1512	0	4.0	3.7	14.8	0.8	17,520	1,200	0.2	8.2	8.4	23.1	16.3%	59.8%	93.5%	OK	OK	EXCEEDS 75%	1.6	53
	A82	A81	250 PVC	1991	0.012	68	287.88	287.48	289.41	288.98			0.59%	1.008	49.5		1512	1512	0	4.0	3.7	14.8		17,520	1,200	0.0	8.4	8.4	23.1	8.1%	29.9%	46.8%	OK	OK	OK	26.3	866
	A81	A80	200 PVC	1991	0.012	81	287.45	280.59	288.98	282.09			8.50%	3.297	103.6		1512	1512	0	4.0	3.7	14.8		17,520	1,200	0.0	8.4	8.4	23.1	3.9%	14.3%	22.3%	OK	OK	OK	80.4	2,646
	A80	A79	200 PVC	1990	0.012	9	280.56	280.39	282.09	281.89			2.00%	1.599	50.2		1512	1512	0	4.0	3.7	14.8		17,520	1,200	0.0	8.4	8.4	23.1	8.0%	29.4%	46.1%	OK	OK	OK	27.1	891
	A79	A78	200 AC	1965	0.013	47	280.36	278.26	281.89	279.76			4.50%	2.215	69.6	37	1512	1548	0	4.1	3.7	15.1	1.3	17,520	1,200	0.3	8.4	8.6	23.8	5.9%	21.7%	34.1%	OK	OK	OK	45.8	1,507
	A78	A77	250 AC	1965	0.013	45	278.23	278.09	279.76	279.59			0.30%	0.664	32.6	88	1548	1637	0	4.4	3.7	15.9	2.2	17,520	1,200	0.5	8.6	9.1	25.0	13.4%	48.8%	76.8%	OK	OK	EXCEEDS 75%	7.5	248
	A77	A76	250 AC	1965	0.013	86	278.06	277.80	279.59	279.30			0.30%	0.664	32.6	37	1637	1673	0	4.5	3.6	16.2	1.1	17,520	1,200	0.2	9.1	9.4	25.6	13.7%	49.8%	78.6%	OK	OK	EXCEEDS 75%	7.0	230
	A76	A75	250 AC	1965	0.013	65	277.77	277.58	279.30	279.08			0.30%	0.664	32.6	41	1673	1714	0	4.6	3.6	16.6	0.8	17,520	1,200	0.2	9.4	9.5	26.1	14.0%	51.0%	80.2%	OK	OK	EXCEEDS 75%	6.5	213
	A75	A74	250 AC	1965	0.013	141	277.55	277.12	279.08	278.62			0.30%	0.664	32.6		1714	1714	0	4.6	3.6	16.6		17,520	1,200	0.0	9.5	9.5	26.1	14.0%	51.0%	80.2%	OK	OK	EXCEEDS 75%	6.5	213
	A74	A73	250 AC	1965	0.013	57	277.09	276.92	278.62	278.42			0.30%	0.664	32.6		1714	1714	0	4.6	3.6	16.6		17,520	1,200	0.0	9.5	9.5	26.1	14.0%	51.0%	80.2%	OK	OK	EXCEEDS 75%	6.5	213
	A3	A2	300 PVC	1978	0.012	16	276.89	276.85	278.42	278.35			0.30%	0.812	57.4		1714	2038	0	5.4	3.6	19.4	8.5	17,520	1,200	1.9	9.5	11.4	30.8	9.5%	33.8%	53.7%	OK	OK	OK	26.6	875
	A2	A1	300 PVC	1978	0.012	58	276.82	276.64	278.35	278.14			0.30%	0.812	57.4		2038	2038	0	5.4	3.6	19.4		17,520	1,200	0.0	11.4	11.4	30.8	9.5%	33.8%	53.7%	OK	OK	OK	26.6	875
	A1	Main Lift Sta	350 ?		0.012	41	276.61	276.49	278.14	277.99			0.30%	0.900	86.6		2038	2038	0	5.4	3.6	19.4		17,520	1,200	0.0	11.4	11.4	30.8	6.3%	22.4%	35.6%	OK	OK	OK		

Assume 0.03m drop at each manhole
Assume 1.5 m from manhole rim to pipe invert

Industrial Park & Airport Trunk Sewer System Profile Existing PWWF



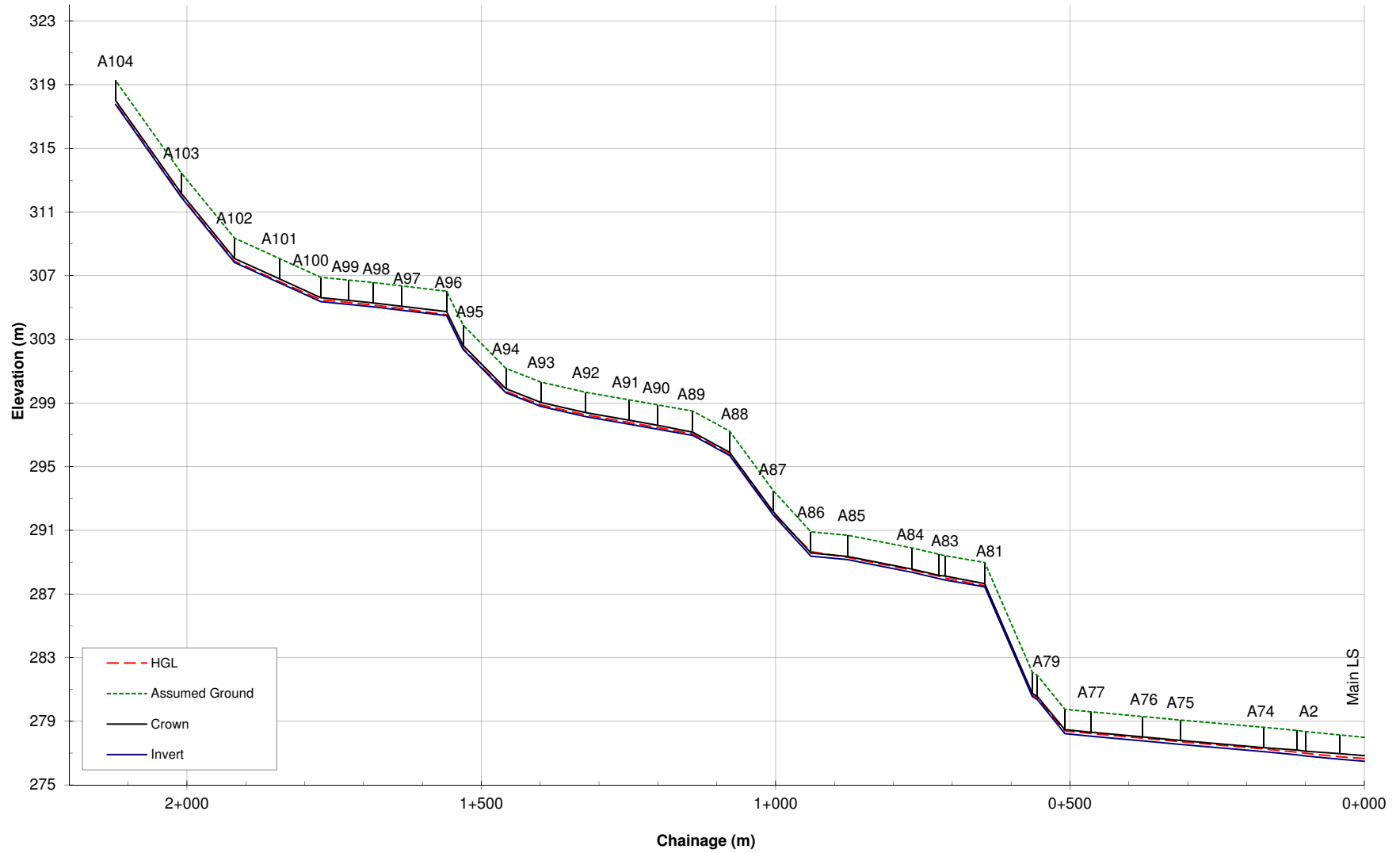
ft to m	0.3048
in to mm	25.4
n =	0.012 PVC
n =	0.013 AC

Per unit lot size	0.05 ha
Per household siz	2.1 pop/unit
Peak Factor	3.5
Per unit PWWF	0.0304 L/s/unit

Industrial Park & Airport - Long Range Future Scenario

Assume 0.03m drop at each manhole
Assume 1.5 m from manhole rim to pipe invert

Industrial Park & Airport Trunk Sewer System Profile
Future PWWF



Sanitary Sewer Capacity Assessment
Collection Pipe Spreadsheet Model
KWL No. 3233.001
filespec:

ft to m	0.3048
in to mm	25.4
n =	0.012 PVC
n =	0.013 AC

0.3048
25.4
0.012 PVC
0.013 AC

RDII	17,520	L/ha/day
GWI	1,200	L/ha/day

17,520 L/ha/day
1,200 L/ha/day

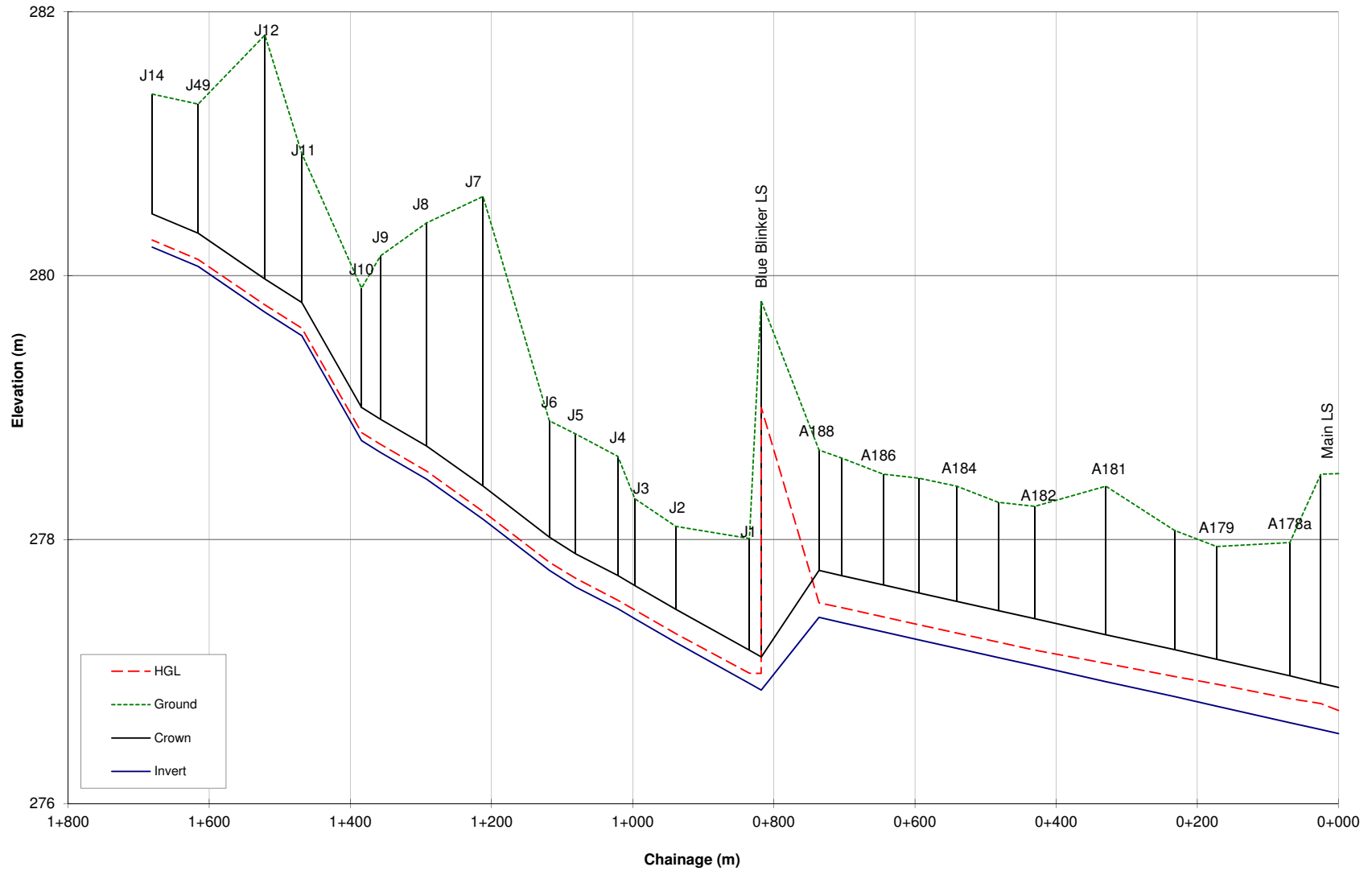
Per unit lot size	0.05 ha
Per household size	2.1 pop/unit
Peak Factor	3.5
Per unit PWWF	0.0304 L/s/unit

0.05 ha
2.1 pop/unit
3.5
0.0304 L/s/unit

per/capita flow 230 L/cap/d

South West Sector - Existing Scenario																																					
Pipe #	u/s MH	d/s MH	Dia mm	Material	Year Const	Mannings 'n'	Length (m)	u/s Invert (m)	d/s Invert (m)	u/s Rim (m)	d/s Rim (m)	Ref As Built #	Slope (%)	Velocity (m/s)	Capacity (Q) (L/s)	Trib PE Pop	u/s Pop	TOTAL Pop	Pump Flow (L/s)	ADWF (L/s)	Harmon (P.F.)	PDWF (L/s)	Trib Area	RDII Rate (L/ha/d)	GWl Rate	Trib I&I (L/s)	u/s I&I (L/s)	Total I&I (L/s)	PWWF (L/s)	ADWF/QI (I) (%)	PDWF/QI (II) (%)	PWWF/QI (III) (%)	Comments (I)	Comments (II)	Comments (III)	Remaining Capacity (L/s)	Remaining Capacity (units)
<descr>	<descr>	<descr>	<input>	<descr>	<desc>	<ref>	<input>	<input>	<input>			<desc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<input>	<calc>	<calc>	<calc>	<calc>	<ref>	<ref>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>	<calc>
J14	J14	J49	251.5	DR 35 PVC		0.012	45.22	280.22	280.07	281.38	281.30	GIS (length)	0.32%	0.75	37.2	173		173	0	0.5	4.2	1.9	5.7	17,520	1,200	1.2		1.2	3.1	1.2%	5.2%	8.5%	OK	OK	OK	34.0	1,119
J49	J49	J13	251.5	DR 35 PVC		0.012	19.19	280.07	280.00	281.30	281.20	GIS (length)	0.36%	0.80	39.5	30	173	202	0	0.5	4.1	2.2	1.1	17,520	1,200	0.2	1.2	1.5	3.7	1.4%	5.7%	9.4%	OK	OK	OK	35.8	
J13	J13	J12	251.5	DR 35 PVC		0.012	94.50	279.98	279.72	281.20	281.82	302.222-2	0.27%	0.69	34.1	15	202	217	0	0.6	4.1	2.4	0.5	17,520	1,200	0.1	1.5	1.6	4.0	1.7%	7.0%	11.6%	OK	OK	OK	30.2	992
J12	J12	J11	251.5	DR 35 PVC		0.012	52.30	279.72	279.55	281.82	280.93	302.222-3	0.34%	0.77	38.2	8	217	225	0	0.6	4.1	2.5	0.3	17,520	1,200	0.1	1.6	1.6	4.1	1.6%	6.5%	10.7%	OK	OK	OK	34.1	1,121
J11	J11	J10	251.5	DR 35 PVC		0.012	84.60	279.55	279.25	280.93	279.91	302.222-3	0.35%	0.78	38.6	15	225	240	0	0.6	4.1	2.6	0.5	17,520	1,200	0.1	1.6	1.7	4.4	1.7%	6.8%	11.3%	OK	OK	OK	34.3	1,128
J10	J10	J9	251.5	PVC		0.012	26.90	278.75	278.67	279.91	280.15	302.222-02	0.29%	0.71	35.2	7	240	247	0	0.7	4.1	2.7	0.3	17,520	1,200	0.1	1.7	1.8	4.5	1.9%	7.7%	12.8%	OK	OK	OK	30.7	1,011
J9	J9	J8	251.5	PVC		0.012	65.40	278.66	278.47	280.15	280.40	302.222-02	0.29%	0.71	35.3	10	247	257	0	0.7	4.1	2.8	0.3	17,520	1,200	0.1	1.8	1.9	4.7	1.9%	8.0%	13.2%	OK	OK	OK	30.6	1,007
J8	J8	J7	251.5	PVC		0.012	79.90	278.46	278.20	280.40	280.60	302.222-02	0.32%	0.74	36.8	9	257	266	0	0.7	4.1	2.9	0.2	17,520	1,200	0.0	1.9	1.9	4.8	1.9%	7.9%	13.1%	OK	OK	OK	32.0	1,053
J7	J7	J6	251.5	PVC		0.012	93.90	278.15	277.77	280.60	278.90	302.222-01	0.41%	0.84	41.7	7	266	273	0	0.7	4.1	3.0	0.4	17,520	1,200	0.1	1.9	2.0	5.0	1.7%	7.1%	11.9%	OK	OK	OK	36.8	1,209
J6	J6	J5	251.5	PVC		0.012	37.40	277.77	277.64	278.90	278.80	302.222-01	0.33%	0.76	37.8	273	273	0	0.7	4.1	3.0			17,520	1,200	0.0	2.0	2.0	5.0	1.9%	7.9%	13.1%	OK	OK	OK	32.9	1,081
J5	J5	J4	251.5	PVC		0.012	59.80	277.64	277.48	278.80	278.63	302.222-01	0.27%	0.69	34.3	8	273	280	0	0.7	4.1	3.1	0.7	17,520	1,200	0.1	2.0	2.1	5.2	2.2%	8.9%	15.1%	OK	OK	OK	29.1	957
J4	J4	J3	251.5	PVC		0.012	23.90	277.48	277.40	278.63	278.31	302.222-01	0.32%	0.74	36.9	280	280	0	0.7	4.1	3.1			17,520	1,200	0.0	2.1	2.1	5.2	2.0%	8.3%	14.1%	OK	OK	OK	31.7	1,043
J3	J3	J2	251.5	PVC		0.012	58.09	277.40	277.22	278.31	278.10	GIS (length)	0.31%	0.73	36.4	5	280	285	0	0.8	4.1	3.1	1.3	17,520	1,200	0.3	2.1	2.4	5.5	2.1%	8.5%	15.2%	OK	OK	OK	30.9	1,016
J2	J2	J1	251.5	PVC		0.012	104.33	277.22	276.91	278.10	278.01	GIS (length)	0.30%	0.72	35.7	285	285	0	0.8	4.1	3.1			17,520	1,200	0.0	2.4	2.4	5.5	2.1%	8.7%	15.5%	OK	OK	OK	30.1	992
J1	J1	Blue Blinker	251.5	PVC		0.012	16.81	276.91	276.86	278.01		GIS (length)	0.30%	0.72	35.7	285	285	285	0	0.8	4.1	3.1		17,520	1,200	0.0	2.4	2.4	5.5	2.1%	8.7%	15.5%	OK	OK	OK	30.2	992
Add Branch Flow	Blue Blinker	Blue Blinker	A188	150 AC		0.013	81.79	278.21	277.61	279.81	278.68	Strong Lamb and Nelson Ltd K10-4204	0.73%	0.73	13.0		285	285	0	0.8	4.1	3.1	11.2	17,520	1,200	0.0	2.4	2.4	5.5	5.9%	23.9%	42.6%	OK	OK	OK	7.5	245
	A188	A188	A187	355.6 AC CL2400		0.013	32.00	277.41	277.37	278.68	278.62	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.54	53.5	383	285	669	0	1.8	3.9	7.0	12.3	17,520	1,200	2.7	2.4	5.1	12.0	3.3%	13.0%	22.5%	OK	OK	OK	41.5	1,365
	A187	A187	A186	355.6 AC CL2400		0.013	58.89	277.37	277.30	278.62	278.50	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.53	52.5	26	669	695	0	1.8	3.9	7.2	1.1	17,520	1,200	0.2	5.1	5.3	12.5	3.5%	13.7%	23.9%	OK	OK	OK	40.0	1,315
	A186	A186	A185	355.6 AC CL2400		0.013	50.47	277.30	277.24	278.50	278.47	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.53	52.9	4	695	698	0	1.9	3.9	7.2	0.4	17,520	1,200	0.1	5.3	5.4	12.6	3.5%	13.7%	23.9%	OK	OK	OK	40.2	1,324
	A185	A185	A184	355.6 AC CL2400		0.013	53.77	277.24	277.18	278.47	278.40	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.53	52.5	4	698	702	0	1.9	3.9	7.3	0.4	17,520	1,200	0.1	5.4	5.5	12.8	3.6%	13.9%	24.3%	OK	OK	OK	39.7	1,307
	A184	A184	A183	355.6 AC CL2400		0.013	58.83	277.18	277.11	278.40	278.28	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.53	52.5	12	702	714	0	1.9	3.9	7.4	0.5	17,520	1,200	0.1	5.5	5.6	13.0	3.6%	14.1%	24.7%	OK	OK	OK	39.5	1,301
	A183	A183	A182	355.6 AC CL2400		0.013	51.82	277.11	277.04	278.28	278.25	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.53	52.2	11	714	725	0	1.9	3.9	7.5	0.4	17,520	1,200	0.1	5.6	5.7	13.2	3.7%	14.4%	25.3%	OK	OK	OK	39.0	1,283
	A182	A182	A181	355.6 AC CL2400		0.013	100.58	277.04	276.92	278.25	278.40	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.53	53.0	16	725	741	0	2.0	3.9	7.7	0.3	17,520	1,200	0.1	5.7	5.8	13.4	3.7%	14.5%	25.3%	OK	OK	OK	39.6	1,301
	A181	A181	A180	355.6 AC CL2400		0.013	97.36	276.92	276.81	278.40	278.07	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.52	51.8	67	741	809	1	3.2	3.9	12.2	0.6	17,520	1,200	0.1	5.8	5.9	18.1	6.1%	23.5%	34.9%	OK	OK	OK	33.7	1,110
	A180	A180	A179	355.6 AC CL2400		0.013	59.74	276.81	276.74	278.07	277.95	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.54	53.2		809	809	2	4.2	3.9	16.0		17,520	1,200	0.0	5.9	5.9	21.9	7.8%	30.1%	41.2%	OK	OK	OK	31.3	1,031
	A179	A179	A178a	355.6 AC CL2400		0.013	103.63	276.74	276.61	277.95	277.98	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.53	52.8		809	809	3	5.2	3.9	19.9		17,520	1,200	0.0	5.9	5.9	25.8	9.8%	37.6%	48.8%	OK	OK	OK	27.1	890
	A178a	A178a	A178	355.6 AC CL2400		0.013	43.89	276.61	276.56	277.98	278.50	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.53	52.3		809	809	4	6.2	3.9	23.7		17,520	1,200	0.0	5.9	5.9	29.6	11.8%	45.4%	56.7%	OK	OK	OK	22.7	745
	A178	A178	Main Lift Station	350		0.013	25.00	276.56	276.53	278.50	278.50	Strong Lamb and Nelson Ltd K10-4205	0.12%	0.53	50.5		809	809	5	7.2	3.9	27.6		17,520	1,200	0.0	5.9	5.9	33.5	14.2%	54.6%	66.3%	OK	OK	OK	12.0	561

South West Trunk Sewer System Profile Existing PWWF



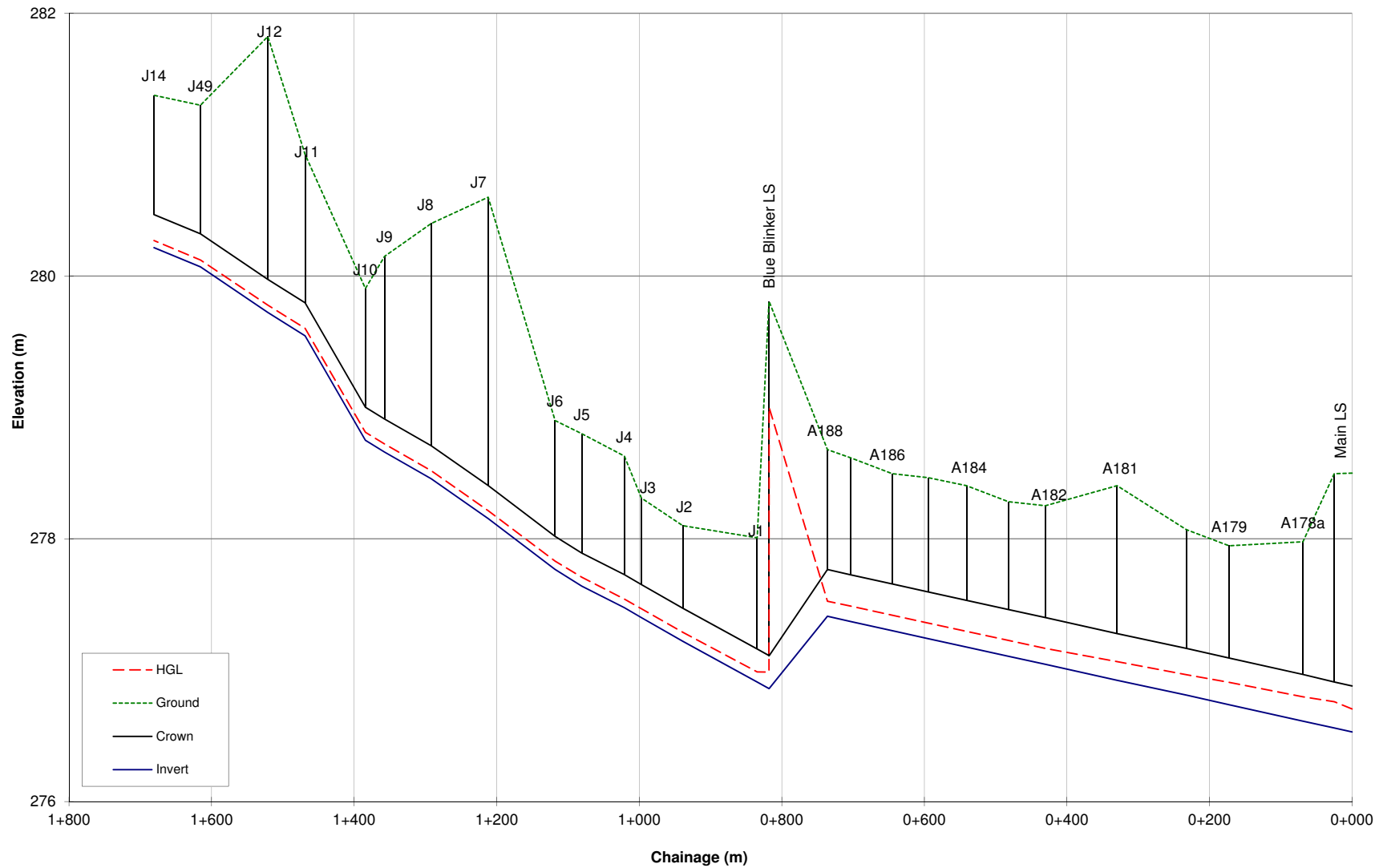
ft to m	0.3048
in to mm	25.4
n =	0.012 PVC
n =	0.013 AC

Per unit lot size	0.05 ha
Per household size	2.1 pop/unit
Peak Factor	3.5
Per unit PWWF	0.0304 L/s/unit

230

Appendix A-4c

South West Trunk Sewer System Profile
Future PWWF





KERR WOOD LEIDAL
consulting engineers

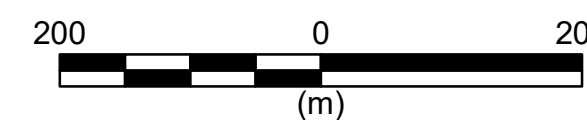
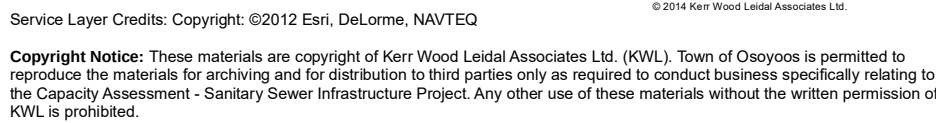
Appendix B

Figure 3-1 – Existing Development Figure 3-2 – Long Range Development (Size A1)



- Sanitary Node
- Sanitary Lift Station
- Sanitary Pipe (with Results)
- Sanitary Gravity Main
- Sanitary Forcemain

DRAFT



Project No. 3233-001	Date January 2014	Drawn By RT

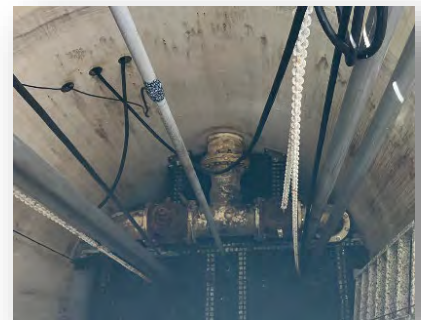
Long Range Future Development

Trunk Sewer Assessment

Figure 3-2

Starlite Lift Station

Relocation, Upgrade, and Development Feasibility



June 2023

Project No. 302-2121

ENGINEERING ■ PLANNING ■ URBAN DESIGN ■ LAND SURVEYING

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EGBC Permit to Practice
No. 1000129

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List of Acronyms

AAI	Average Annual Investment
AC	Asbestos Cement
AM	Asset Management
AWWA	American Water Works Association
DI	Ductile Iron
FRP	Fiberglass Reinforced Plastic
HDPE	High Density Polyethylene
HDD	Horizontal Directional Drilling
I&I	Inflow and Infiltration
LS	Lift Station
MWWPS	Main Wastewater Pumping Station
MMCD	Master Municipal Construction Documents
PE	Population Equivalent
PVC	Polyvinyl Chloride
PWWF	Peak Wet Weather Flow
OCP	Official Community Plan
OIB	Osoyoos Indian Band
SCADA	Supervisory Control and Data Acquisition
STEP	Septic Tank and Effluent Pump
TRUE	TRUE Consulting
VFD	Variable Frequency Drive
WWTP	Wastewater Treatment Plant

Units of Measure

ft	feet
lgpm	Imperial gallons per minute
km	kilometre
L/d	Litres per day
L/m	Litres per minute
L/s	Litres per second
lpcd	Litres per capita per day
m	metre
mg/L	milligrams per Litre
mm	millimetre
NTU	Nephelometric Turbidity Units
psi	pounds per square inch
USgpm	US gallons per minute

Referenced Reports

Capacity Assessment	Analysis of Trunk Main Sewer Capacity, “Capacity Assessment Sanitary Sewer Infrastructure” May 2014
Infrastructure Plan	Analysis of Sanitary Sewer System Capacity, “2023 Osoyoos Sanitary System Infrastructure Plan” DRAFT May 2023

Executive Summary

TRUE was engaged by the Town of Osoyoos to investigate the feasibility of upgrading, and/or relocating the existing Starlite lift station located at the north-west corner of 7710 Main Street. The original 1965 Starlite lift station has been replaced once in the same location and is approximately 29 years old. On the west side of 7710 Main Street is a narrow right-of-way that is constrained by the Holiday Inn on the west side, and the existing single-family home on the east side. During the last lift station replacement project, the existing 150mm forcemain was replaced with a newer PVC 250mm forcemain prior to the Holiday Inn being constructed.

Recently the property owner of 7710 Main Street has expressed interest in developing the property with higher density, triggering the Town's need to not only establish the impacts to the existing Starlite lift station, but to also assess the future impacts of the development. As the Holiday Inn restricts the workspace on the west side of the right-of-way, and a new structure would restrict the east side of the right-of-way, any future maintenance, repair, or replacement work within the right-of-way would be expected to be inaccessible.

To assess the future needs and the impacts of the proposed development of 7710 Main Street, TRUE has developed 5 scenarios as shown on Figure 1 in Appendix A with the following outcomes:

- Scenario 1: Maintain the existing lift station location for \$2.25-4.55M. (See Figure 2 in Appendix A)
 - Any equipment needed at the facility needs to be brought in along the beach
 - Trenchless pipe installation is proposed with a larger outer pipe casing and inner 250mm forcemain to allow the inner forcemain to be pulled out and re-installed as needed in the future at either end of the casing. Development would need to address pipe casing access needs
 - Sewage bypassing is required from the existing lift station to a proposed bypass manhole which forces sewage back into the existing forcemain downstream of the proposed works
 - Larger 67hp pumps on VFD's are proposed to meet the future PWWF
- Scenario 2: New lift station on the south-west corner of 7710 Main Street for \$4.79-\$7.09M. (see Figure 3 in Appendix A)
 - Access to the facility would be from Main Street.
 - Trenchless pipe installation is proposed with a larger outer pipe casing and inner 375mm deep sewer main to allow the inner pipe to be pulled out and re-installed as needed in the future at either end of the casing. Development would need to address pipe casing access needs.

- Sewage bypassing is required from the existing lift station to a proposed bypass manhole which forces sewage back into the existing forcemain downstream of the proposed works.
 - Additional public use land is required from 7710 Main Street.
 - A replacement lift station with 67hp pumps on VFD's is proposed to meet the future PWWF.
- Scenario 3: New lift station across Main Street from 7710 Main Street on Pioneer Walkway for \$3.43M. (see Figure 4 in Appendix A)
 - Access to the facility would be from Main Street and minimal impacts to Pioneer walkway once surface restoration is completed.
 - Trenchless pipe installation is proposed with a larger outer pipe casing and inner 375mm deep sewer main to allow the inner pipe to be pulled out and re-installed as needed in the future at either end of the casing. Development would only be potentially restricted near the existing lift station location to maintain access to the beach side of the pipe casing.
 - Sewage bypassing is required from the existing lift station to a proposed bypass manhole which forces sewage back into the existing forcemain downstream of the proposed works.
 - A replacement lift station with 44hp pumps on VFD's is proposed to meet the future PWWF.
 - Main Street is expected to be restricted to single lane alternating traffic during construction.
- Scenario 4: New lift station west of 7305 Main Street for \$3.12-3.7M. (see Figure 5 in Appendix A)
 - Access to the facility would be from Main Street and minimal impacts to Pioneer walkway once surface restoration is completed.
 - Traditional construction methods utilized.
 - Sewage bypassing is limited to short periods of time for tie-ins of existing pipes
 - A replacement lift station with 54hp pumps on VFD's is proposed to meet the future PWWF.
 - Main Street is expected to be restricted to single lane alternating traffic during construction.
- Scenario 5: New lift station on the North-west corner of 6901 Main Street for \$3.97-4.56M. (see Figure 6 in Appendix A)
 - Access to the facility would be from Harbour Key Drive
 - Traditional construction methods utilized.
 - Sewage bypassing is limited to short periods of time for tie-ins of existing pipes
 - A replacement lift station with 54hp pumps on VFD's is proposed to meet the future PWWF.
 - Main Street is expected to be restricted to single lane alternating traffic during construction but given the greater length of sewer main to be installed, the duration of traffic restriction would be greatest of all the scenarios.

Of the five scenarios, the Town would benefit longer term if Starlite is relocated beside Main Street to allow operators, maintenance crews and other parties easy access to the lift station in the event of planned or unplanned visits. Scenarios 4 and 5 would offer greater flexibility for future improvements by eliminating the narrow pipe right-of-way on 7710 Main Street while Scenario 1 and 2 could present property access challenges. To mitigate the risk of constrained property access for Scenario 1 and 2, 7710 Main Street could be purchased at a cost likely between the BC Assessment value of \$941,900 and the current MLS listing (June 2023) of \$2.3 million. The purchase of the land would address access issues and could be developed further to encourage greater public access to the north beach. As a compromise to scenario 1 and 2, Scenario 3's placement of Starlite on Pioneer walkway improves access off Main Street but requires the existing pipeline right-of-way to be maintained.

The final choice for the Starlite lift station requires a multitude of tools to assist the decision-making process. If the multivariable risk assessment tool is used, Scenario 4 offers the lowest risk and focuses primarily on traditional methods of construction. Following scenario 4 would be Scenario 1 and 5, followed by Scenario 3 and lastly Scenario 2. If capital costs are the key driving tool in the decision process, scenario 1 would offer the lowest capital cost but existing residual risks would remain. Other tools including community acceptance, local property owner biases, mayor and council opinions, Town staff opinions and other variables should also be considered in conjunction with the multivariable risk and capital cost tools outlined in this report.

The next step for the project would be selection of the chosen scenario followed by a formal geotechnical investigation to better understand the detailed geology of the area. If the soils are found to have a strong connection with Osoyoos Lake, dewatering would likely become challenging and timing the work during low lake levels would be essential.

1.0 Introduction

In response to the pre-development application for 7710 Main Street (Lot 6, KAP3531, DL 40 SDYD) for a proposed seven story residential apartment, TRUE has prepared this report. This report outlines our findings with respect to the existing Starlite lift station located near the north-west corner of the subject parcel for development, the impacted utilities located on a statutory right-of-way running along the west portion of the property and scenarios to replace the lift station in a new location. TRUE's opinion of the probable capital cost along with a decision risk-based assessment was completed to aid the Town with the decision making process.

2.0 Background

Several historical documents were reviewed during TRUE's assessment of the Starlite lift station including the following:

- The original Starlite lift station was constructed in 1965 along the north property line of 7710 Main Street and was constructed out of steel with a service life of 30 years before structural integrity was compromised.
- In 1994, the Starlite lift station was replaced to the south of the 1965 lift station with a premanufactured fiberglass structure.
- The Starlite sewage forcemain was replaced and upsized (150mm Asbestos Concrete to 250mm PVC) in 1997 from Osoyoos Lake to Main Street within the statutory right-of-way.
- Recent events captured in a TRUE Letter RE: 2007 Starlite Lift Station and Lot 6 Plan 3351 Owned by Mr. Keith Primeau:
 - In the summer of 1997, the owners of 7710 Main Street inquired about relocating the Starlite lift station to the south-west corner of the parcel, nearest to Main Street. As there was no benefit to the Town, the landowner did not pursue the relocation further.
 - From 1997 to 2007, significant growth occurred as a result of Osoyoos Indian Reserve #1 and in the east sector of town that relocation of the lift station was deemed impractical.
 - Two options were discussed including the south-west corner of 7710 Main Street or across Main Street on Pioneer Walkway with the walkway being preferred by the landowner.
 - The right-of-way along 7710 Main Street would make gravity sewer conveyance difficult due to the close proximity of building foundations, high groundwater level and reports of blasted rock in Main Street's (Highway 3) sub-grade.
- 2022 Flood Mitigation Plan prepared by TRUE in 2022: Starlite's access to the lift station should be placed at or greater than 279.5m for a flood return period of 50 years. To

address the risks of flooding, the report outlined that the facility risks could be mitigated by either relocation or flow control implementation. Flow control mitigation measures would be intended to manage inflows and infiltration into the submerged gravity conveyance system through the installation and closure of plug valves in key areas during the flood event. Relocation would place the assets at higher ground and away from the source of flood water.

Further to these previous report outcomes, the current lift station has been in service for 29 years and has considerable challenges relating to future upgrades to overcome any sanitary sewer flow increases. Currently the main access point would be through the right-of-way on 7710 Main Street which is only accessible for persons only, or along Osoyoos Lake's beach for equipment or persons. Depending on the nature and size of any new development on 7710 Main Street, repairs, rehabilitation, or replacement of pipelines or the lift station could be difficult to complete due to limited workspace, restrictive access, and shallow water levels.

2.1 Design Requirements

As part of this investigation, several requirements were identified as follows:

- Town of Osoyoos Subdivision and Development Servicing Bylaw 1100
 - Minimum grades from 0.14% to 0.40% were maintained from sizes 400mm to 200mm, respectively.
 - Minimum pipe size 200mm for gravity conveyance.
 - Manhole drop of 25mm and 50mm for horizontal alignment changes of less than 45 degrees and greater than 45 degrees but less than 90 degrees, respectively.
 - Mannings coefficient of 0.012 for PVC gravity pipe
 - Forcemain minimum velocity 0.75 m/s.
 - Hazen-Williams coefficient of 120 for worse case scenario and AWWA M32's coefficient (130-150) for best case scenario.
- MMCD Design Guidelines:
 - Gravity sewers to be designed up to 80% full (depth / pipe diameter)

Upon review of the 2023 Sanitary Sewer System Capacity and Capital Plan (Draft) prepared by TRUE, along with the previous 2014 Capacity Assessment Sanitary Sewer Infrastructure prepared by KWL, several challenges arise with lift station capacity vs PWWF as illustrated in Table 2-1 with the location of each facility shown on Figure 1 in Appendix A.

TABLE 2-1: EXISTING LIFT STATION CAPACITY AND PWWF

FACILITY	PUMP ON DUTY CYCLE	TRUE 2023 EXISTING PWWF (L/S)	TRUE 2023 FUTURE PWWF (L/S)	KWL 2014 EXISTING PWWF (L/S)	KWL 2014 FUTURE PWWF (L/S)	KWL 2014 LS STATED CAPACITY (L/S)
Starlite LS	1x Flygt 20hp NP3153 MT 433	59.2	81.7	42.3	50.3	65
44 th Ave LS	1x Flygt 25hp NP3171 HT 455	30.5	36.5	19.5	23.9	40
S&L LS	1x Flygt 20hp NP3153 MT 433	37	37.2	36.4	42.5	36.4

As shown in Table 2-1 the existing and particularly, the 2023 TRUE PWWF as modelled is expected to exceed the capacity of the lift stations as shown in red writing. The Smith & Loveless (S&L) lift station is identified as under capacity and the pumping capacity should be confirmed before any upgrades are planned. As the S&L, 44th and Starlite lift stations share a forcemain, the capacity of each lift station generally depends on the number of pumps operating along with the location of lift station along the forcemain. Capacity of each lift station would be discussed in the following section.

3.0 Conveyance Assessment

Following the historical path of the Starlite lift station and developing some new concepts to address development at 7710 Main Street, several scenarios were evaluated as follows:

- Scenario 1: Maintain the existing lift station location.
- Scenario 2: New lift station on the south-west corner of 7710 Main Street.
- Scenario 3: New lift station across Main Street from 7710 Main Street on Pioneer Walkway.
- Scenario 4: New lift station west of 7305 Main Street.
- Scenario 5: New lift station on the North-west corner of 6901 Main Street.

Each scenario was evaluated for conveyance requirements to direct flows to the lift station and pumping capacity with one to three lift stations operating (Starlite, 44th and S&L).

3.1 Scenario 1: Maintain the Existing Lift Station Location

For this scenario, the existing lift station would be maintained in its current location with all existing gravity mains remaining the same as shown on Figure 2 in Appendix A. To establish the capacity of the lift station, several drawdown-based pump tests were conducted. These tests were conducted on June 2, 2023 with a test average of 62.4 L/s with only Starlite operating and 54.6 L/s with three facilities (Starlite, S&L and 44th) operating together at once. When the field results are compared to the design work, the existing 20hp Flygt pump real life results were well within the design flow range for best and worst case as shown in Table 3-1.

Continuing with the same assumptions of best and worst case proven by real life testing, Table 3-1 shows the results with Starlite's previously stated capacity of 65 L/s. Under closer examination, Table 3-1 shows the 65 L/s stated capacity is generally within the best and worse case bounds of normal operation. Table 3-1 also illustrates that only Starlite will need upgrading as its current capacity does not meet future PWWF as highlighted in red.

TABLE 3-1: EXISTING LIFT STATION CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 20hp NP3153 MT 433	60.1-66.6	60.1-68.2	53.8-60	81.7
44 th Ave LS	1x Flygt 25hp NP3171 HT 455	47.2-53	42.3-50.9	38.3-44.1	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	53.1-60.5	49.5-55.7	37.2

To enable growth to occur, additional horsepower is needed to force higher sewage flows down the common forcemain. To achieve this, a single 67hp Flygt pump is proposed (bolded writing) with resultant flows shown in Table 3-2.

TABLE 3-2: EXISTING LIFT STATION WITH UPGRADED CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 67hp NP3202 HT 458	82*-91.3	92.7-106.9	86.9-96.8	81.7
44 th Ave LS	1x Flygt 25hp NP3171 HT 455	47.2-53	45.6-61.5	40.7-48	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	49.9-59.9	42-47.5	37.2

**Pump simulated to operate at reduced speed to illustrate lower flows and lower head requirements.*

As part of the design process, Figure 3-1 below illustrates the Starlite's Total Dynamic Head (TDH) from its current location to the Main Wastewater Pumping Station with one facility running and all three facilities (S&L, 44th Ave and Starlite) running at the same time.

The TDH chart is intended to show the pressure rise on the right of the chart, indicating the pump providing the head pressure. The colored line then slopes downward indicating internal pressure loss as sewage is pushed through the forcemain until the line crosscuts the pipe elevation where pressure in the pipe becomes atmospheric. At the point where pressure becomes atmospheric, the sewage would then flow by gravity (open channel flow) to its destination.

It should be noted that the slope of the TDH line often changes in its angle or slope due to the change in pipe diameter, or at points where flow rate changes due to 44th Avenue or S&L lift stations forcing additional flows into the common forcemain. Additionally, the pump's input head (where the line starts on the left hand side of the TDH chart) varies due to the hydraulically balanced pressure and flow balanced with the manufacturer's (Flygt) pump performance curve.

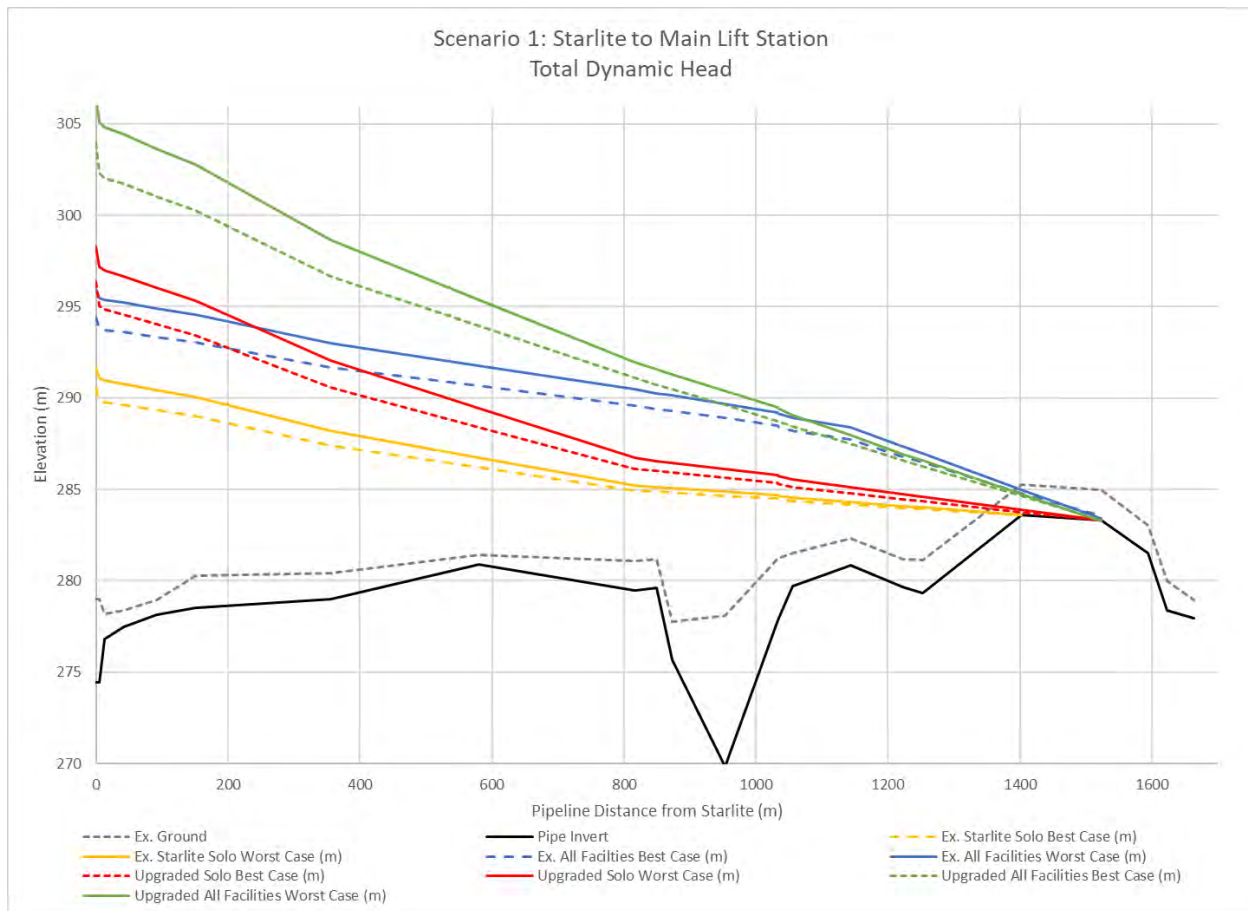


FIGURE 3-1: EXISTING STARLITE LIFT STATION LOCATION WITH EXISTING AND PROPOSED PUMPS

The discharge point of the effluent is expected to change as flows increase as shown on Figure 3-1 by noting the location where the TDH colored line connects with the pipeline elevation. Where the two lines intercept is the estimated location where pressurized flow changes to open channel (partially full pipe) flow. In this case, the switch from pressurized flow to open channel changes occurs at nearly 1.4 km from the existing Starlite lift station with one Starlite pump running, where as the rest of the scenarios increase the flow rate such that the open channel flow occurs at an estimated 1.5 km from the existing Starlite lift station.

Understanding that open channel flow is expected along Kingfisher Drive, the existing air valves should be confirmed prior to increasing any flow rate beyond the current arrangement as the increased sewage flow could cause air binding, surging, air efferece and related issues.

3.1.1 Scenario 1: Constructability

To address the existing 7710 Main Street right-of-way utility congestion, building foundations, and other utilities within the right-of-way, decluttering and removal would be beneficial to the Town if

repairs were to occur. Understanding the Town's budget is often constrained, removal costs were not included in the later cost estimates.

To facilitate the proposed larger development on 7710 Main Street, a larger pipe casing would be installed to house the new Starlite forcemain. The pipe casing would allow the Town to access the entirety of the internal forcemain by allowing removal of the internal pipe as needed, one pipe segment at a time from either side of the pipe casing without digging the length of the right-of-way. The pipe could be installed using traditional open cut trenching, but installation would be difficult with the existing home on 7710 Main Street on the east side of the pipeline and the Holiday Inn on the west side of the pipeline. To enable independent installation of the pipe on the right-of-way located within 7710 Main Street, trenchless technology has been assumed for installing the forcemain pipe casing. The first technology could be HDD, but would require the drill rig to be placed back from the start point of the pipe casing, and in this instance, would most likely put the drill rig into Osoyoos Lake. Although possible, a barge or fixed platform would need to be brought in to allow the HDD drill rig to sit back as required. Once the HDD is underway to do the drill and to install the pipe casing, additional risks could impact the aquatic environment due to the risk of leaking hydraulic fluids used by the drill rig and migrating drill mud used as part of the drilling process. As a result, guided pipe auguring or ramming is anticipated as the auger or air hammer could be dropped into an excavation near to the pipe entry and isolate the trenchless technology equipment from Osoyoos Lake. In this instance, several drillers expect that the drill would take 1-2 weeks to complete from buildup of the site to teardown.

Inside the larger pipe casing would be the 250mm diameter restrained forcemain pipe on low friction casing spacers. As this restrained pipe is segmented into pieces, the area needed to replace or repair the pipe would be limited to the beach area near Osoyoos Lake or near Main Street that would need to be incorporated into the approval process of any development on 7710 Main Street.

To manage risks of damaging existing pipes and allow for proper testing and tie-in of the new force main, a hot-tap bypass manhole structure along with isolation valve would be placed on the existing AWWA C900 PVC forcemain on the south-west corner of 7710 Main Street near Main Street. Conceptually, a pair of 75hp Godwin NC150S Dri-prime diesel pumps (one duty and one standby) would be placed near the existing lift station, air plugs placed into the pipe between the last manhole and Starlite wet well, and Godwin pump suction pipes dropped into the manhole. Once the manhole is surcharged, the duty Godwin pump would suck out the raw sewage and pump it around the trench work in an aboveground HDPE pipeline to the bypass manhole connecting to the existing downstream forcemain, allowing sewage to be returned on its path to the Main Street lift station. Once the bypass is underway, the wet well piping, valving and appurtenances could be repaired, rehabilitated, or replaced if budgets allow. At a minimum, the downtime would allow the proposed pumps to be installed along with new VFDs.

3.2 Scenario 2: Location A Replacement Starlite Lift Station

For this scenario, the existing lift station would be replaced and installed at the South-west corner of 7710 Main Street as shown on Figure 3 in Appendix A.

Like Scenario 1, a single 67hp Flygt pump is proposed (bolded writing) to achieve the target future wet weather flow as shown in Table 3-3.

TABLE 3-3: LOCATION A REPLACEMENT LIFT STATION WITH UPGRADED CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP(S) ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 67hp NP3202 HT 458	81.7*-121.5	81.7*-116.8	81.7*-94.7	81.7
44 th Ave LS	2x Flygt 25hp NP3171 HT 455	55.7-64.1	44-61.6	41.8-49.2	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	48.9-60	46.2-52	37.2

**Note: Starlite's replacement lift station is assumed to function under both wet well level and flow-based control logic*

As indicated in Table 3-3, 44th Avenue will require upgrading to include the 3rd pump as the shorter pipeline for Starlite increases the flow sufficiently to force 44th Avenue's pump to dip below the target future wet weather flow. It will be essential that the Starlite replacement lift station run under VFD control to not only save energy but slow the pump cycle along with improving the pump reliability.

Under this scenario, the 44th Avenue lift station with both pumps running does bring concerns of increased risks of clogging, increased energy consumption and reduced service life due to the new operating point with Starlite and S&L operating. If the Town would like to minimize these risks, a single Flygt 3202 may be a reasonable alternative but should be confirmed. Alternatively, evaluating the annual pump run hours throughout the year may show that this risk is limited to short periods of time in the summer combined with rainfall events. A simple upgrade to the PLC may alleviate unwanted operator call-outs due to clogging through implementing, if not already in place, advanced Flygt self-cleaning clog removal programming.

Figure 3-2 below shows the Starlite Total Dynamic Head (TDH) to the Main Street Lift Station with one facility running and the worst case of all three facilities (S&L, 44th Ave and Starlite) running at the same time.

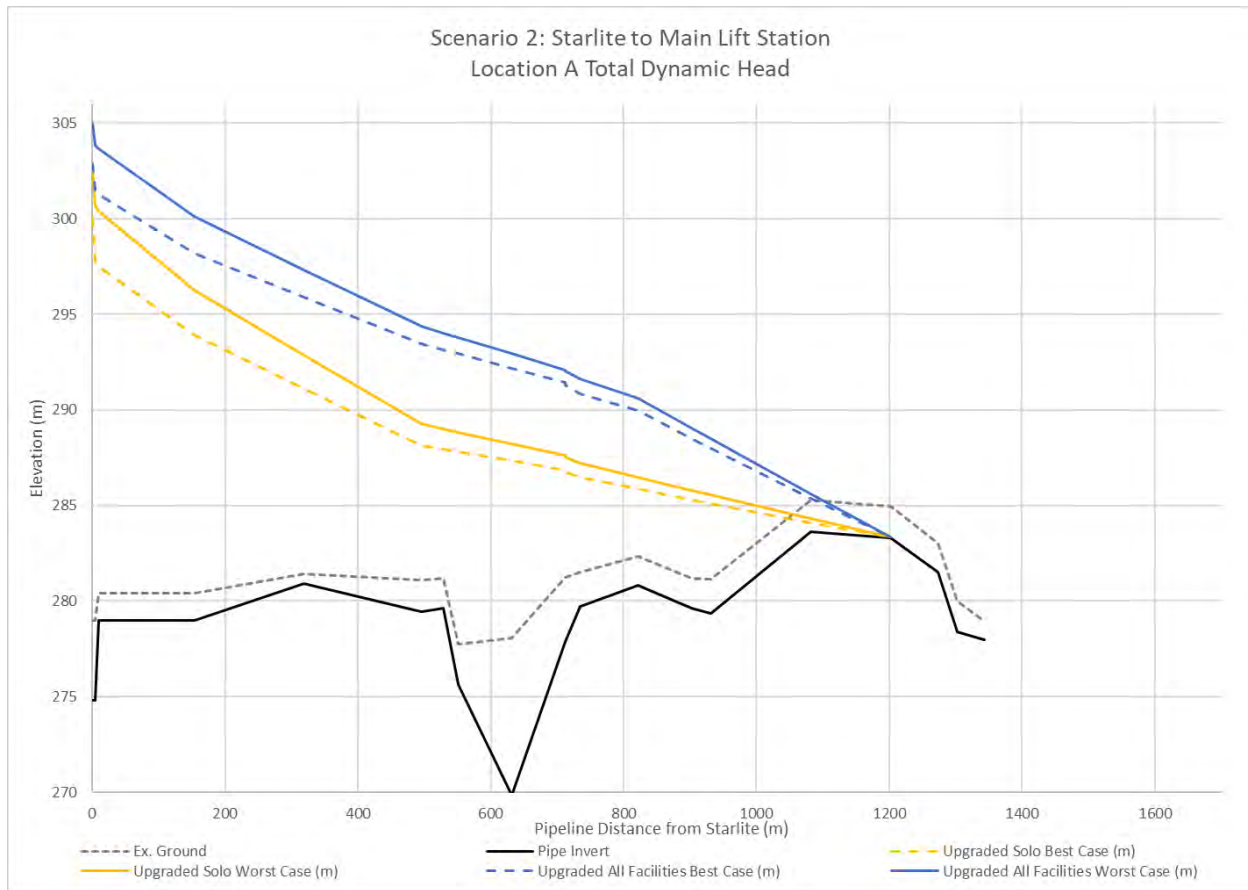


FIGURE 3-2: LOCATION A REPLACEMENT STARLITE LIFT STATION LOCATION WITH PROPOSED PUMP(S)

For this scenario, as shown in Figure 3-2 the point of switching from pressurized flow to open channel occurs at nearly 1.2 km from the proposed replacement Starlite lift station. As noted in scenario 1, at 1.2 km and onward, air management through the existing air valves along Kingfisher Drive should be confirmed for sizing when upgrading occurs.

3.2.1 Scenario 2: Constructability

For this scenario, constructability comments of Scenario 1 remain the same, but instead of a shallow forcemain, a 375mm (15") SDR35 PVC pipe would need to be installed upwards of 5.7m deep at a 2% grade to create a reasonable conservative slope for the trenchless pipe installers install the pipe casing for the host gravity pipe to be slide into the pipe casing. Unlike the forcemain, a stiff pipe casing would be required to ensure the internal gravity pipe is held at a constant grade from end to end of the casing. Additionally, the 2% slope of the inlet pipe to the lift station wet well would assist with conforming to guidance provided in the first edition AWWA E110-22 Solids Handling Pump standard along with other Hydraulic Institute standards.

Like Scenario 1, decluttering of the 7710 Main Street right-of-way of un-used assets could be beneficial to the Town by increasing accessibility of necessary utilities along with avoiding

confusion in the future for any repairs, rehabilitation or replacement projects. Understanding that Scenario 2 will take considerably longer to construct than Scenario 1, the Town could entertain the idea of connecting the bypass diesel pump manifold to the existing Startlite lift station. As HDPE is fully restrained, an aboveground HDPE pipeline would be more than capable of allowing the Startlite lift station to convey sewage in the temporary HDPE pipe during construction and only use the diesel pumps during tie-ins and periods of power loss. This approach would also reduce the carbon footprint of the project by only operating temporary diesel pumps when electrically driven pumps are not available.

The wet well is expected to have a finished floor elevation of 273.2m, or nearly 7.1m deep (assuming a ground elevation of 280.3m) plus additional depth for the concrete slab and any leveling granular material used. Understanding that blast rock has been found in the past in the highway (Main Street) road subgrade, a geotechnical investigation should be conducted. Open cut with shoring or sheet piles is probable as illustrated on Figure 3 in Appendix A or alternatively a method of a sinking caisson with hydrovac soil removal could be done. In either method, detailed survey, slope analysis, dewatering risks, and geotechnical will be essential as the depth of the wet well could prove to be impractical from either a constructability or cost prospective. Well point or deep well dewatering may be required if dewatering cannot be done conventionally. Additionally, with such deep depths and proximity to the lake, floatation of the wet well will need to be considered, and typically concrete will need to cast around the FRP wet well to resist such forces.

In any case, due to the depth of all the proposed assets, additional right-of-way lands on 7710 Main Street is needed to facilitate the installation of the lift station.

3.3 Scenario 3: Location B Replacement Startlite Lift Station

Scenario 3 builds on Scenario 2 but relocates the lift station across Main Street and onto Pioneer Walkway. Pioneer Walkway would be expected to only be impacted during construction and the original alignment can be restored when the new lift station is completed as shown on Figure 4 in Appendix A.

As noted previously in the report, a larger pump is required to meet future PWWF but given the shorter pipeline distance to the common forcemain, a single 44hp Flygt pump is proposed (bolded writing) to achieve the target future PWWF as shown in Table 3-4.

TABLE 3-4: LOCATION B REPLACEMENT LIFT STATION WITH UPGRADED CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP(S) ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 44hp NP3202 HT 462	81.7*-123.5	81.7*-108	81.7*-98	81.7
44 th Ave LS	2x Flygt 25hp NP3171 HT 455	55-7-64.1	46.2-64	41.6-49.2	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	49.5-60	46.3-52.1	37.2

**Note: Starlite's replacement lift station is assumed to function under both wet well level and flow-based control logic*

Following similar results from Scenario 2, higher flow rates coming from Starlite along with S&L lift stations causes the 44th Avenue lift station to operate in a zone where there are concerns of increased risks of clogging, increased energy consumption and reduced service life. As noted in Scenario 2, different pumps or advanced VFD control logic would aid in reducing the number of callouts to the Town's operators to remove blockages.

Figure 3-3 below shows the Starlite Total Dynamic Head (TDH) to the Main Wastewater Pumping Station with one facility running and the worst case of all three facilities (S&L, 44th Ave and Starlite) running at the same time. As noted in Scenario 1 & 2, the TDH curve is hydraulically balanced with the manufacturer's (Flygt) pump performance curve and the total system pressure losses.

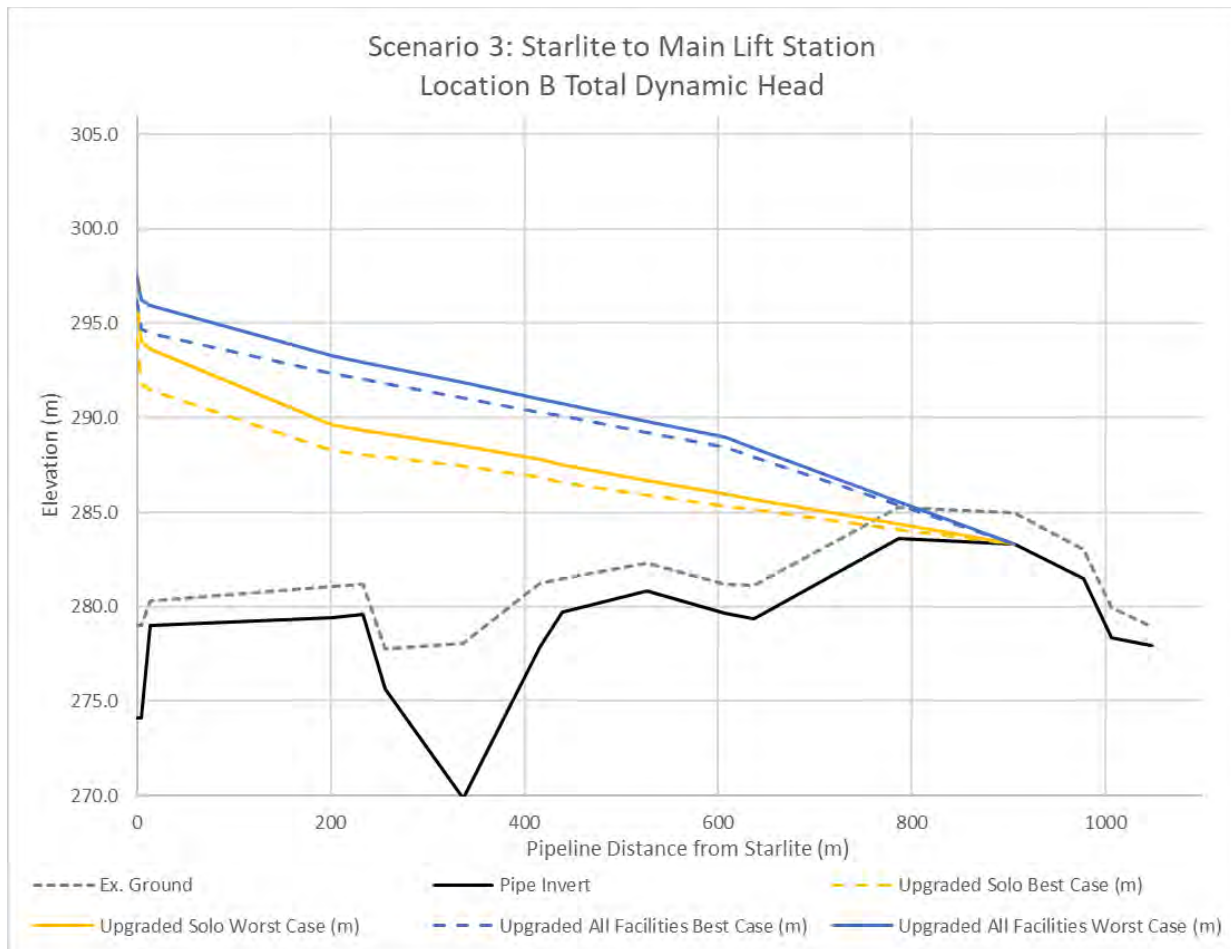


FIGURE 3-3: LOCATION B REPLACEMENT STARLITE LIFT STATION LOCATION WITH PROPOSED PUMP(S)

Figure 3-3 illustrates that the discharge location is approximately 900m from the proposed replacement Starlite lift station and does not change as a result of Starlite or all lift stations operating making future air management assessment straight forward. Like Scenario 1 and 2, the existing air valves along Kingfisher Drive should be confirmed for sizing as the increased flow rate could cause air binding, surging, air efferece and related issues, slowing the flow rate for all lift stations.

3.3.1 Scenario 3: Constructability

For this scenario, constructability risks of Scenario 2 remain the same, but the length of the gravity pipeline grows to extend to pioneer walking trail on the opposite side of Main Street, bringing the depth of burial to an estimated depth of 6.4m. As noted in Scenario 2, the 2% grade for the trenchless pipe installation remains the same for the same reasons as Scenario 2, but may require sheet piling or shoring to maintain traffic on Main Street after the risks are assessed in the future geotechnical investigation.

Like Scenario 1, decluttering of the 7710 Main Street right-of-way would be the ideal outcome pending budgets, and Scenario 2 type temporary sewage bypass pumping is needed.

The wet well is expected to have a finished floor elevation of 272.5m, or estimated at nearly 8m deep plus additional depth for the concrete slab and any leveling granular material used. Like Scenario 2, blast rock, dewatering risks, flotation, and safe working slopes would need to be addressed during detailed design and post geotechnical investigation.

3.4 Scenario 4: Location C Replacement Starlite Lift Station

For this scenario, the existing lift station would be replaced and installed on Pioneer Walkway to the west of 7305 Main Street in close proximity to the existing public use gazebo as shown on Figure 5 in Appendix A.

For this scenario, a single 54hp Flygt pump is proposed (bold writing) to achieve the target future wet weather flow as shown in Table 3-5.

TABLE 3-5: LOCATION C REPLACEMENT LIFT STATION WITH UPGRADED CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP(S) ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 54hp NP3202 HT 460	81.7*-117.1	81.7*-112.8	81.7*-99.5	81.7
44 th Ave LS	2x Flygt 25hp NP3171 HT 455	55.7-64.1	44.2-64	39.8-49.3	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	48.5-60	45.1-50.8	37.2

**Note: Starlite's replacement lift station is assumed to function under both wet well level and flow-based control logic*

Following similar results from Scenario 3, the higher combined sewage flows with Starlite and 44th Avenue lift stations operating, the 44th Avenue lift station with both pumps running does bring concerns of increased risk of clogging, increased energy consumption and reduced service life. As noted in Scenario 3, different pumps or advanced VFD control logic would aid in reducing the number of callouts to the Town's operators to remove blockages.

Figure 3-4 below shows the Starlite Total Dynamic Head (TDH) to the Main Wastewater Pumping Station with one facility running and the worst case of all three facilities (S&L, 44th Ave and Starlite) running at the same time.

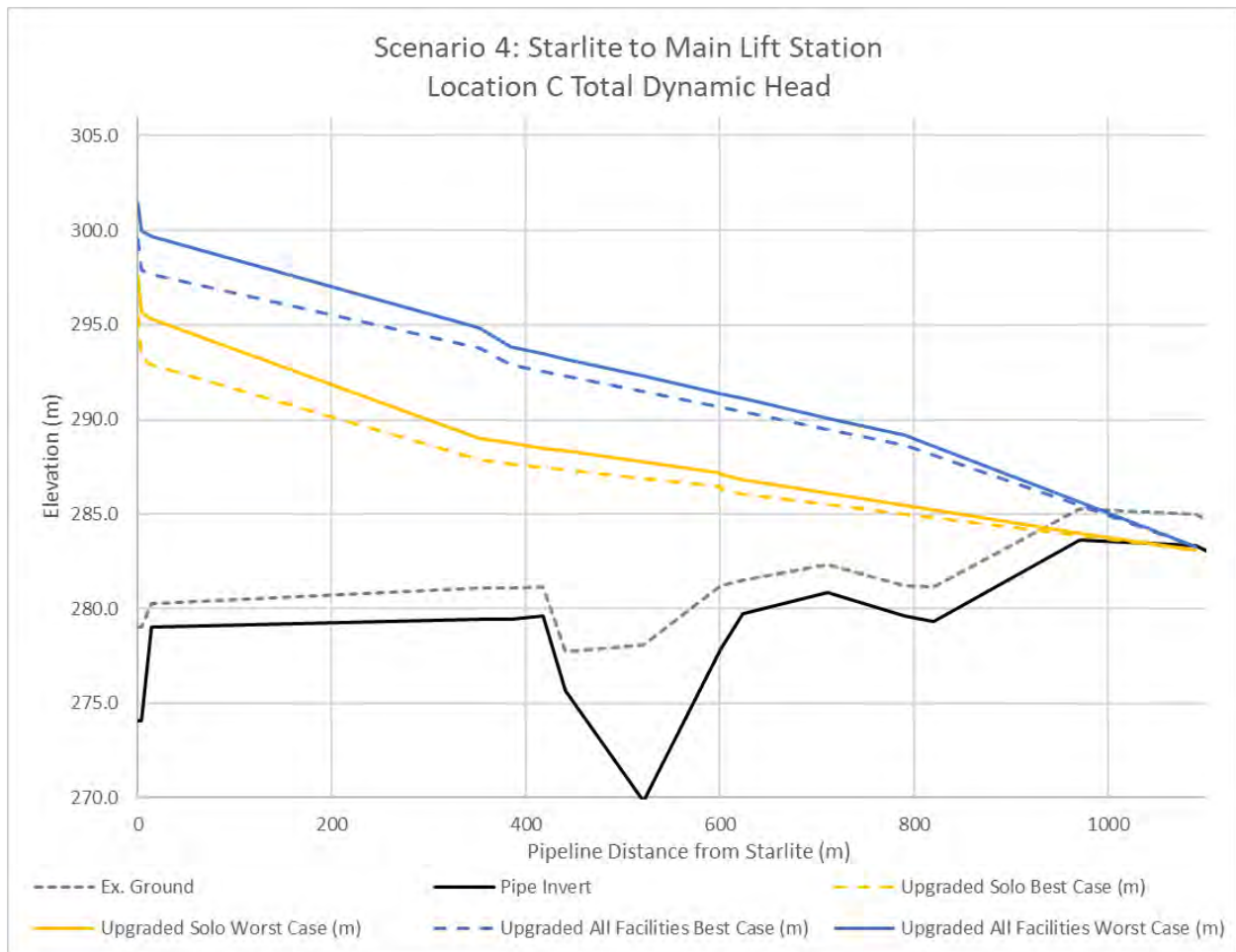


FIGURE 3-4: LOCATION C REPLACEMENT STARLITE LIFT STATION LOCATION WITH PROPOSED PUMP(S)

Following the same trend as Scenario 3, Figure 3-4 shows that one or all pumps in operation does not change for detailed design air management. Like other scenarios, as expected, air valves along Kingfisher Drive should be confirmed for sizing during detailed design.

3.4.1 Scenario 4: Constructability

For this scenario, the pipe alignment routing would differ from Scenario 1 through 3. To facilitate flow from 7906 Main Street (Holiday Inn & Suites) to 7310 Main Street (Desert Surfside) for reverse flow to the new lift station location, several options present itself.

- Option 1: Conveyance of wastewater by conventional 200mm PVC gravity main, starting at a depth of 1m burial. The pipe main and manholes would be placed along the beach to the north or south of the existing main. Challenges would include groundwater, working near water, environmental concerns and reconnecting existing sewer services which may require regarding of pipes back to the building or even pumping.
- Option 2: Gravity conveyance like Option 1, but with the exception that the pipe would be placed in the shoulder or back of curb of Main Street. This option would remove the environmental risks of working beside the lake but may still have dewatering challenged along with traffic control as Main Street would be impacted. Rerouting of building services may be challenging as services would need to be routed from the rear of the building to the front of the property.
- Option 3: Low pressure sewer could be used. This method would typically utilize septic tanks to collect solids, and clear effluent would pass onto a downstream pump tank with an effluent pump. The effluent pump would send wastewater down a smaller service pipe to the common forcemain in the street or beach. Routing of the service line would be simple as design grades are less critical than gravity lines any may be a good alternate if one or more properties cannot be connected by gravity if gravity flow is selected as the primary connection method. Challenges to low pressure sewer include locating the spot for the solids collecting septic tanks, ensuring property owners pump out their tanks on a normal frequency, and the property owners are open to installing, maintaining, and servicing their own septic tanks and effluent pump (often 2hp or less).

In any of the optional pipe routes to service the properties from 7906 Main Street to 7310 Main Street, sewage generated from the north-east side of Osoyoos including Osoyoos Indian Band IR #1, would need to be redirected from the existing Starlite lift station towards the new proposed replacement Starlite lift station location to the west of 7305 Main Street. To reverse flow, the existing pipe main running along the beach would be redirected into a new 375mm (15") pipe running between 7308 Main Street and 7310 Main Street, within the existing public corridor between the two privately owned parcels. Open cut trenching along with trench boxes is anticipated as space on the south side of Main Street is limited at best. Trenchless pipe installation was not suggested due to the cost but could be added if geotechnical or detailed design require this technology.

Once the new reverse flow main is across main street (as shown on Figure 5 in Appendix A) and in front of 7305 Main Street, the pipe would turn 90 degrees to run along Main Street's shoulder to stay away from the congested zone near the sidewalk of utilities including gas, sewage forcemain, water pipes, and other utilities. To the west of the existing public washroom parking lot and the east of the public gazebo would be the new replacement Starlite lift station.

The wet well is expected to have a finished floor elevation of 273.5m (shallowest of all options) or estimated at nearly 7m deep plus additional depth for the concrete slab and any leveling granular material used. Like Scenario 2 and 3, blast rock, dewatering risks, flotation, and safe working slopes would need to be addressed during detailed design.

The advantage of this scenario is that sewage bypassing would be limited to hours, not days or weeks at a time, reducing risks of vandalism along with around the clock monitoring. It would be anticipated that most of the assets could be installed, tested, and commissioned with fresh water prior to switching to the new system. Bypassing would likely be limited to tying in the existing and proposed sewage pipe mains.

3.5 Scenario 5: Location D Replacement Starlite Lift Station

For this scenario, the existing lift station would be replaced and installed on the northwest corner of 6901 Main Street as shown on Figure 6 in Appendix A. To allow future PWWF's to be pushed to the Main Wastewater Pumping Station, a 54hp Flygt pump is proposed (bolded writing) as shown in Table 3-6.

TABLE 3-6: LOCATION D REPLACEMENT LIFT STATION WITH UPGRADED CAPACITY UNDER VARYING CONDITIONS

FACILITY	PUMP(S) ON DUTY CYCLE	ONE FACILITY OPERATING (L/S)	TWO FACILITIES OPERATING (L/S)	THREE FACILITIES OPERATING (L/S)	TARGET FUTURE PWWF (L/S)
Starlite LS	1x Flygt 54hp NP3202 HT 460	81.7*-103.3	81.7*-110.8	81.7*-91.8	81.7
44 th Ave LS	2x Flygt 25hp NP3171 HT 455	55.7-64.1	30.9-64	30.1-36.2	36.5
S&L LS	1x Flygt 20hp NP3153 MT 433	57-62.5	49.5-60	47.6-53	37.2

**Note: Starlite's replacement lift station is assumed to function under both wet well level and flow-based control logic*

Following similar results from Scenario 4 and increased working pressures in the common forcemain, the 44th Avenue lift station with both pumps running does bring concerns of inadequate future PWWF capacity, increased risks of clogging, increased energy consumption and reduced service life. To address 44th Avenue's deficiencies as shown in Table 3-6 with red writing, a single VFD controlled 54hp pump, identical to the Starlite replacement lift station would meet the local target future PWWF.

Figure 3-5 below shows the Starlite Total Dynamic Head (TDH) to the Main Wastewater Pumping Station with one facility running and the worst case of all three facilities (S&L, 44th Ave and Starlite) running at the same time.

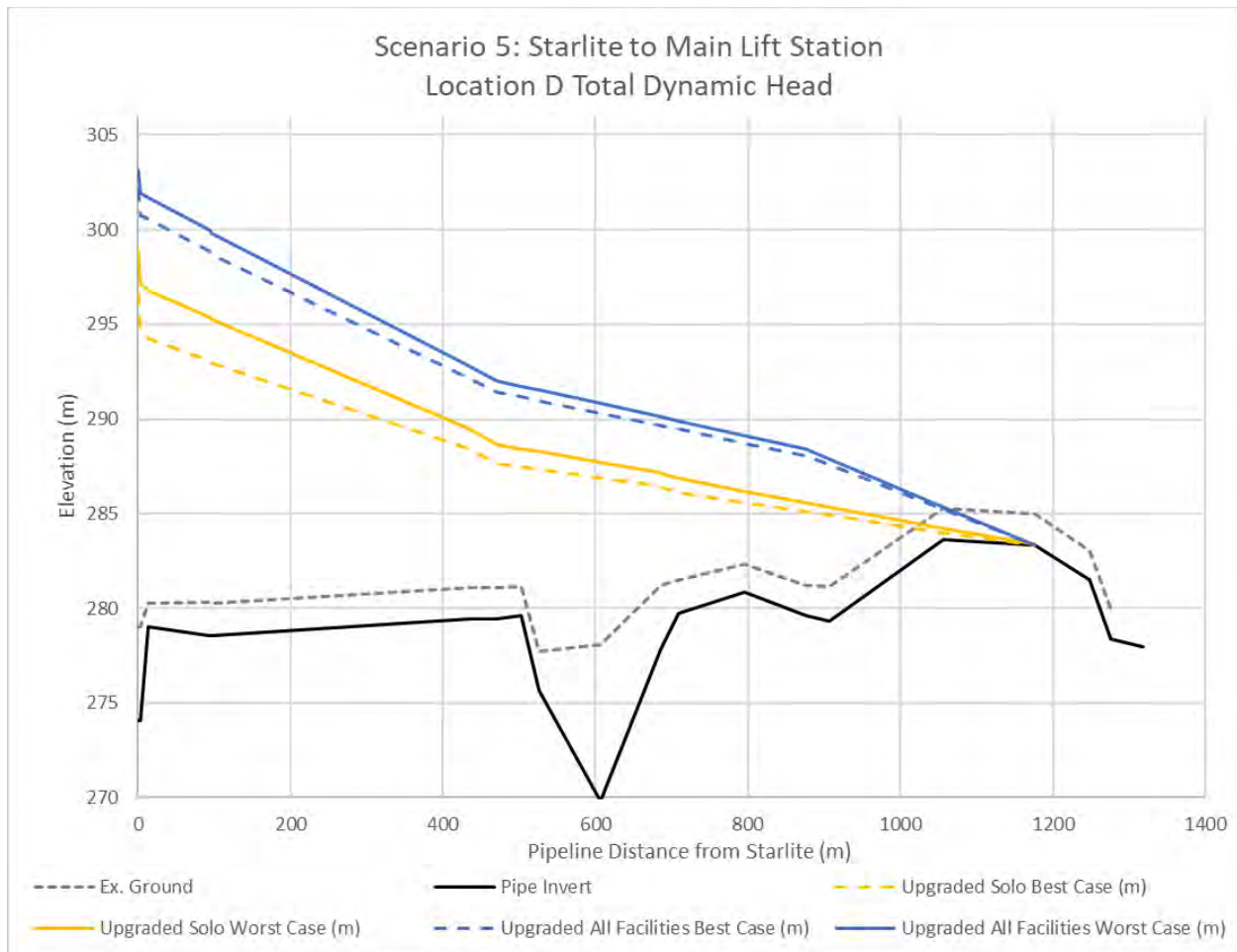


FIGURE 3-5: LOCATION D REPLACEMENT STARLITE LIFT STATION LOCATION WITH PROPOSED PUMP(S)

Also shown on Figure 3-5, the point of switching from pressurized flow to open channel changes is consistent like Scenario 2 through 4, making future air management design straight forward. The existing air valves along Kingfisher Drive should be confirmed for sizing as the increased flow rate could cause air binding, surging, air efferece and related issues, slowing the flow rate for all lift stations.

3.5.1 [Scenario 5: Constructability](#)

For this scenario, the pipe alignment routing would be identical to Scenario 4 for the beach properties from 7906 Main Street (Holiday Inn & Suites) to 7310 Main Street (Desert Surfside) to reverse flow to the new lift station location. The choice of gravity or low-pressure sewer would remain the same as Scenario 4 along with the redirected new 375mm (15") pipe running between 7308 Main Street and 7310 Main Street, within the existing public corridor between the two privately owned parcels. Open cut trenching with trench boxes is anticipated like Scenario 4.

Once the new reverse gravity flow main is across Main Street and in front of 7305 Main Street, the pipe would turn 90 degrees to run along Main Street's shoulder to stay away from the congested zone near the sidewalk of utilities including gas, sewage forcemain, water pipes, and other utilities. The proposed gravity sewer main would move east towards 6901 Main Street to a shared manhole with a new trunk main crossing Main Street south down Ponderosa Drive.

The sewage flows south of Cottonwood Drive running along the lake would be collected along Maple Drive as shown on Figure 6 in Appendix A. In collaboration with the recent sewage master plan, the gravity sewage main would be upsized to 300 (12") from

- Poplar Court to Cottonwood Drive (heading north-west)
- Along Cottonwood Drive between Maple Drive and Jackpine Lane
- New main constructed to cross Cottonwood Drive along Ponderosa Drive, with the new pipe placed to the south of the existing sewage pipes and manholes.
- New main would follow Ponderosa Drive south and terminate at the common manhole on the south side of Main Street in front of 6901 Main Street
- A new main heading south from the junction of the two new gravity pipe mains into 6901 Main Street where the new replacement Starlite lift station would be constructed

The relocation of the Starlite lift station to the east of the existing Starlite lift station presents challenges due to added lengths deepening the depths of the main, reaching nearly 6m deep. At this depth, the most practical option is to either use trench boxes, or shoring. For the Ponderosa and Main Street pipe crossing, a local detour along Harbour Key Drive would provide flexibility in timing for the Contractor.

The wet well is expected to have a finished floor elevation of 272.9m or estimated at nearly 5-7.5m deep (survey needed to confirm as ground drops off from Main Street's elevation) plus additional depth for the concrete slab and any leveling granular material used. Like Scenario 2 to 4, blast rock, dewatering risks, flotation, and safe working slopes would need to be addressed during detailed design.

Like Scenario 4, sewage bypassing would be limited to short durations rather than long periods of time as explained in Scenarios 1 to 3.

It should be noted that the work along Main Street from 7305 Main Street to 6901 Main Street may need to be delayed until the replacement Starlite lift station is operational with limited inflows as a full bypass may be needed for low sewage generation draining down Harbor Key Drive. This delay is likely due to the new pipe main installation likely undermining the existing manholes.

4.0 Transient Analysis

To meet the future PWWF, sewage flows would need to increase, and this section is intended to outline the risks and mitigation measures to offset the increased sewage flows. The existing common forcemain for the Starlite, 44th Avenue and S&L LS primarily consists of:

- 250mm, 3000mm AWWA C900 PVC Pressure PC100 (DR41)
- 350mm AWWA C906 HDPE, DR17
- 200mm Coated AWWA C200 Steel Pipe (assumed Schedule 40)
- The piping likely includes FRP and Asbestos concrete pipe, where the records do not provide sufficient information to verify their performance.

4.1.1 Transient Surge

Transient surges are typically a result of either a valve closing too quickly with sewage being pumped or a lift station losing power before backup generator can restart the facility. The type of valve, pump start/stop method and process design can impact the severity of the transient surge.

In review of all the scenarios, the transient surges are expected to be in the order of 120-130psi on PVC/ HDPE pipes and approaching 760psi on steel pipes. Although the pipes technically can withstand the transient surge pressures, new measures would be suggested particularly as the strength of the asbestos concrete and FRP pipe is not well understood.

4.1.2 Mitigation Measures

The common forcemain has few existing buried slow closing valves which should be maintained to ensure inadvertent closure of a valve by operators.

Inside the wet wells and valve chambers, the current quarter turn valves should have either labels placed near the valves warning the operators to not close them while pumps are operating or add gearboxes to mechanically force the valve closure to be slow.

The existing check valves appear to be the typical HDL/Flygt ball check valves which when combined with the current across-the-line starting (on-off pump cycling), can lead to considerable transient surges. Pressure monitoring would be suggested at the existing lift stations to see if check valves should be changed to a lower surge type or VFD's installed. These pressure gauges use an isolated diaphragm to avoid the typical solids plugging of the conventional pressure gauge. The added benefit of installing these gauges would be the ability of operators, maintenance or consultants to witness pressure characteristics during operation to assist with troubleshooting problems or monitoring pumps for wear to schedule pump repairs.

For the replacement Starlite lift station, the mechanical design is expected to include AWWA C520 slow closing knife gate valves, AWWA C517 buried service plug valves, anti-surge type AWWA

C512 solid float combination air valve with air accumulator chamber, AWWA C504 check valves and variable frequency drives on each AWWA E110 submersible pumps.

5.0 Probable Capital Costs

TRUE has prepared our opinion of probable capital costs based on projects of similar nature and of similar scope. The values suggested in these estimates were prepared with site information available at the time and based on probable conditions affecting the project. The estimates are intended to indicate an approximate magnitude of cost used for planning, assists with establishing a more specific project definition and used for preliminary project discussion for long term capital plans.

As part of the probable capital cost estimate preparation, TRUE included the construction risk, construction contingency and an allowance for engineering fees was incorporated.

5.1 Construction Risk

Construction risk is defined as the uncertainty identified in preparing the opinion of probable capital cost estimate which covers quantity/quality of pre-engineering information, tender and construction schedules, the construction market, and non-quantifiable items.

- Quality/Quantity of pre-engineering information (15%): work involving removal or extension of existing facilities or pipelines when existing conditions are not completely known. Geotechnical soil excavation revealing presence of rock, or other unsuitable soils, groundwater, and other soil conditions.
- Tender and Construction Schedule (5%): projects with a tight schedule that could result in delays beyond a critical date (such as winter or bird nesting windows or in-stream fish windows) that lead to additional labour or construction method costs. Additional costs relating to delays in permits, project award or permissions with regulators. Risks of construction bidding process it is proposed to occur in one season only or several construction seasons.
- Construction Market (15%): Availability of contractors, materials and conditions within the supply chain. Risks of changing local contractor availability or desire by the local contractors to bid along with inflation and evolving labour costs.
- Non-quantifiable Items (5%): Risks related to new materials or construction methods, availability of special materials or local materials or equipment that cannot be determined in advance and premium placed on specialized or local contractor work.

5.2 Construction Contingency

Construction Contingency (10%) is an allowance for unforeseeable elements of costs which covers small design changes, supply problems, small quantity changes, reasonable inflation rate, and other undefined risks which could not be identified prior to construction.

5.3 Engineering Costs

The engineering costs are an allowance taken from federally available guidelines based on historical statistical analysis of probable engineering effort that are a percentage of the project capital cost. The engineering fee allowance includes:

- Project definition: planning, conceptualization, and defining scope, standard and requirements of project.
- Investigation and Design: Field survey, geotechnical investigation, preliminary and detailed design including up to preparing for tendering.
- Construction Inspections: field activities required to oversee the construction phase of the project.
- Project Management: engineering activities required to oversee the delivery of the project including engineering budgets, progress payments to the contractor, schedule monitoring and cost control.

For this project, an engineering fee allowance of 14.5% to 16.1% has been calculated. Although this fee could be seen as a fixed amount, the value is a statistical amount based on historical federal statistics intended solely for budget planning and should be more or less than the actual effort. Factors that impact the engineering fee include, but are not limited to: Experience of the contractor, supply chain health, contractor labor or equipment availability, contractor/sub-contractor/vender financial health to deliver the project, number of changes by the contractor or owner during stages of the project, changing ground conditions or unforeseen additional effort required to complete the project.

5.4 Estimates

TRUE has summarized the cost estimates below and a more detailed opinion of the probable capital cost estimate is available in Appendix B. The percentages as described above for construction risk, contingency and engineering fee allowance were incorporated into the cost estimates.

The reader should understand the current market conditions are still under considerable stress and our estimate is our opinion of costs based on rapidly changing historical costs. Stresses appear to remain in the market and can include high inflationary conditions, changing

demographics as baby boomers leave the work force, supply chain problems and global market conditions impacting the entire supply chain including key considerations such as foundry availability, raw material costs, and manufacturing capacity.

5.4.1 Scenario 1: Maintain Existing Lift Station Location

For this scenario, the opinion of probable capital cost is shown in Table 5-1.

TABLE 5-1: SCENARIO 1 OPINION OF PROBABLE CAPITAL COST

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$2,000
2	Temporary Sewage Bypass	\$320,000
3	Sewage Forcemain Installed by Trenchless Pipe Installation and Tie-ins	\$501,000
4	Lift Station Upgrades	\$234,000
5	Electrical Upgrades	\$235,000
Subtotal		\$1,292,000
Construction Risk & Contingency		\$646,000
Engineering Fee Allowance		\$208,000
Taxes (GST)		\$107,000
Project Amount (SAY)		\$2,250,000
Optional Purchase of 7710 Main Street at MLS asking price		\$2,299,000
Optional Purchase of Land Project Amount (SAY)		\$4,549,000

5.4.2 Scenario 2: Location A Replacement Lift Station

For this scenario, the opinion of probable capital cost is shown in Table 5-2.

TABLE 5-2: SCENARIO 2 OPINION OF PROBABLE CAPITAL COST

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$2,000
2	Temporary Sewage Bypass	\$480,000
3	Sewage Gravity Main Installed by Trenchless Pipe Installation, Manholes and Tie-ins	\$594,000
4	Reconnect Sanitary Sewer Services	\$8,000
5	Lift Station Replacement	\$744,000
6	Sheet Piles and Dewatering Allowance	\$600,000
7	Electrical Upgrades	\$345,000
Subtotal		\$2,773,000
Construction Risk & Contingency		\$1,386,000
Engineering Fee Allowance		\$402,000
Taxes (GST)		\$228,000
Project Amount (SAY)		\$4,790,000
Optional Purchase of 7710 Main Street at MLS asking price		\$2,299,000
Optional Purchase of Land Project Amount (SAY)		\$7,089,000

5.4.3 Scenario 3: Location B Replacement Lift Station

For this scenario, the opinion of probable capital cost is shown in Table 5-3.

TABLE 5-3: SCENARIO 3 OPINION OF PROBABLE CAPITAL COST

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$10,000
2	Temporary Sewage Bypass	\$224,000
3	Sewage Gravity Main Installed by Trenchless Pipe Installation, Manholes and Tie-ins	\$604,000
4	Reconnect Sanitary Sewer Services	\$8,000
5	Lift Station Replacement	\$676,000
6	Dewatering Allowance	\$100,000
7	Electrical Upgrades	\$345,000
Subtotal		\$1,967,000
Construction Risk & Contingency		\$983,000
Engineering Fee Allowance		\$317,000
Taxes (GST)		\$163,000
Project Amount (SAY)		\$3,430,000

5.4.4 Scenario 4: Location C Replacement Lift Station

Unlike Scenario 1 to 3, this scenario takes the higher costs of trenchless pipe installation through the tight corridor and invests in linear pipe works along less constrained routes. The probable capital cost is shown in Table 5-3 and as noted before, is based on conveying sewage to the replacement lift station using gravity pipes with manholes.

TABLE 5-4: SCENARIO 4 OPINION OF PROBABLE CAPITAL COST WITH GRAVITY PIPE CONVEYANCE

<i>ITEM</i>	<i>DESCRIPTION</i>	<i>ESTIMATED AMOUNT</i>
1	Traffic Control	\$20,000
2	Temporary Sewage Bypass	\$64,000
3	Sewage Gravity Mains with Manholes	\$466,000
4	Reconnect Sanitary Sewer Services	\$91,000
5	Lift Station Replacement	\$706,000
6	Dewatering Allowance	\$100,000
7	Electrical Upgrades	\$345,000
Subtotal		\$1,792,000
Construction Risk & Contingency		\$896,000
Engineering Fee Allowance		\$288,000
Taxes (GST)		\$149,000
Project Amount (SAY)		\$3,120,000

As an alternate to traditional gravity mains with manholes, septic tanks with low pressure effluent pumps were investigated to see if cost efficiencies could be gained as shown in Table 5-5.

TABLE 5-5: SCENARIO 4 OPINION OF PROBABLE CAPITAL COST WITH STEP SYSTEMS

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$20,000
2	Temporary Sewage Bypass	\$160,000
3	Sewage Gravity Mains with Manholes	\$381,000
4	Reconnect Sanitary Sewer Services using STEP System	\$427,000
5	Lift Station Replacement	\$706,000
6	Dewatering Allowance	\$100,000
7	Electrical Upgrades	\$345,000
Subtotal		\$2,139,000
Construction Risk & Contingency		\$1,070,000
Engineering Fee Allowance		\$310,000
Taxes (GST)		\$176,000
Project Amount (SAY)		\$3,700,000

5.4.5 Scenario 5: Location D Replacement Lift Station

For this scenario, the method of using traditional means of construction is utilized like Scenario 4, but the sewage conveyance network is separated on Ponderosa Drive to address long term capacity constraints (completed by TRUE on another report) and relocate the lift station away from nearby beaches or public nature walkways. The probable capital cost is shown in Table 5-6 and as noted before, is based on conveying sewage to the replacement lift station using gravity pipes with manholes.

TABLE 5-6: SCENARIO 5 OPINION OF PROBABLE CAPITAL COST WITH GRAVITY PIPE CONVEYANCE

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$75,000
2	Temporary Sewage Bypass	\$112,000
3	Sewage Gravity Mains with Manholes	\$808,000
4	Reconnect Sanitary Sewer Services	\$91,000
5	Lift Station Replacement	\$716,000
6	Dewatering Allowance	\$150,000
7	Electrical Upgrades	\$345,000
Subtotal		\$2,297,000
Construction Risk & Contingency		\$1,148,000
Engineering Fee Allowance		\$333,000
Taxes (GST)		\$189,000
Project Amount (SAY)		\$3,970,000

As an alternate to traditional gravity mains with manholes, septic tanks with low pressure effluent pumps were investigated to see if cost efficiencies could be gained as shown in Table 5-7.

TABLE 5-7: SCENARIO 3 OPINION OF PROBABLE CAPITAL COST WITH STEP SYSTEMS

ITEM	DESCRIPTION	ESTIMATED AMOUNT
1	Traffic Control	\$75,000
2	Temporary Sewage Bypass	\$192,000
3	Sewage Gravity Mains with Manholes	\$736,000
4	Reconnect Sanitary Sewer Services using STEP System	\$427,000
5	Lift Station Replacement	\$716,000
6	Dewatering Allowance	\$150,000
7	Electrical Upgrades	\$345,000
Subtotal		\$2,642,000
Construction Risk & Contingency		\$1,321,000
Engineering Fee Allowance		\$383,000
Taxes (GST)		\$217,000
Project Amount (SAY)		\$4,560,000

6.0 Risk Assessment

For any project, it becomes necessary to look at several variables for each project and quantify the risks and weighted importance of each variable to assist with design choices. For this assessment to occur, several variables were defined and ranked into consequence categories as follows:

		Consequence				
		Negligible 1	Minor 2	Moderate 3	Major 4	Catastrophic 5
Likelihood	5 Almost certain	Moderate 5	High 10	Extreme 15	Extreme 20	Extreme 25
	4 Likely	Moderate 4	High 8	High 12	Extreme 16	Extreme 20
	3 Possible	Low 3	Moderate 6	High 9	High 12	Extreme 15
	2 Unlikely	Low 2	Moderate 4	Moderate 6	High 8	High 10
	1 Rare	Low 1	Low 2	Low 3	Moderate 4	Moderate 5

FIGURE 6-1: RISK MATRIX USED FOR ASSESSMENT

- Negligible:
 - Minor issue of little concern to community
 - Cost \$1,000
- Minor:
 - Isolated case, some small disruptions
 - Cost \$1,000 – \$50,000
- Moderate:
 - Requires attention, inconvenience.
 - \$50,000 – \$200,000
- Major:
 - Requires urgent attention, impact delays
 - \$200,000 – \$1M
- Catastrophic:
 - Wide damage, lasting weeks, impact high
 - \$>1M

6.1 Assessment Results

The subjective risk of occurrence was developed based on the author's opinion which was developed by leveraging historical experience in the industry. Figure 6-1 shows the risk matrix used to populate Table 6-1 with assumptions for each parameter described in later sections.

TABLE 6-1: WEIGHTED OPTION RESULTS

VARIABLE	SCENARIO					% WEIGHT
	1	2	3	4	5	
Health and Safety	15	12	8	3	3	5
Public Disruption	5	10	15	15	15	5
Town of Osoyoos Coordination	10	15	15	5	5	5
Temporary Bypass	15	15	15	2	2	10
Traffic Disruption	1	8	8	25	25	10
Construction Schedule	6	8	6	2	2	5
Constructability	12	16	12	6	8	10
Long Term Serviceability	9	6	4	2	2	10
Capital Costs	11	25	17	16	21	20
Operation and Maintenance	12	8	8	4	4	20
Total	10.2	13.4	11.3	8.8	9.9	100

When reviewing Table 6-1, lower risk scores along with a lower weighted average total score is the key driver for this assessment. Although only one tool in the decision process, Table 6-1 provides some aid to the Town for selecting a scenario to proceed with.

6.1.1 Health and Safety

Safety is top of mind for all workers including the Town of Osoyoos, contractors, consultants, and the public. Scenario 1 places high risk onto the Town due to the lack of dry valve chamber, limited access for emergency crews to respond and minimal space available for pipeline installation.

Scenarios 2 and 3 increase the risk to workers working near existing building foundations to install the assets but this scenario reduces the access risks and moves key maintenance valves outside the wet well confined space. Scenario 4 and 5 move construction to traditional risks and reduce risks identified in 1, 2, and 3.

6.1.2 Public Disruption

The public would generally be impacted by loss of service on walkways, beaches, parallel parking and traffic flow in all scenarios to a differing degree with Scenarios 3 to 5 having the largest impact, while Scenario 2 reducing the risk and Scenario 1 having the lowest probable impact.

6.1.3 Town of Osoyoos Coordination

Shutting down lift stations and assisting with sewage bypass systems can be time consuming for the Town and place additional efforts on the Town to assist during emergencies. Scenario 1 through 3 have similar efforts with Scenario 1 having a reduced construction time due to shallower trenching of the forcemain. Scenario 4 and 5 has minimal impacts to the Town except for sewage tie-ins for short periods of time.

6.1.4 Temporary Bypass

Bypassing sewage can be challenging as failure of one temporary pump can lead to sewage entering Osoyoos Lake. To offset this, the existing Starlite lift station could be modified to provide service for longer periods of time using aboveground temporary piping. While the modifications of the lift station are underway, temporary diesel pumps would be used. The temporary pipe would likely be HDPE to resist damage from vandalism, mechanical damage, and quick deployment onto the surface around the active construction site. A bypass manhole would be required to re-inject sewage bypassed from Starlite LS to the existing sewage main, and this work would be specialized and would need to be coordinated for timing. Scenarios 1 to 3 would require bypassing for extended periods of time, whereas scenarios 4 and 5 would only require short duration bypasses for tie-ins.

6.1.5 Traffic Disruption

Several of the scenarios including 4, and 5 will require traffic to be diverted to single lane while Scenario 3 assumes trenchless pipe installation can be done. Scenario 5 can use Harbour Key Drive as a detour for the Ponderosa Drive to Main Street pipe crossing to reduce the traffic flow impact. Scenario 2 would have limited impacts to traffic relating to the congested space available to install the lift station to deep depths. No impact for Scenario 1.

6.1.6 Construction Schedule

Timing of the work should be done during low lake levels to mitigate dewatering efforts, and water removed from the trench would need to be discharged to the storm system, sanitary sewer system or as directed by the environmental professional. Depending on the contractor's crew size, method of installation and as-found ground conditions, Scenario 1 and 2 would hold the higher

risk due to the lack of available work area, while Scenario 1 and 3 would could leverage trenchless pipe installation to offset some of the trenching risks related to the close building foundations. Scenario 4 and 5 have traditional risks with higher risks relating to the option of installing new pipework in the beach.

6.1.7 Constructability

Scenario 1 through 3 have the higher challenges to construct due to the narrow nature of the right-of-way and trenchless pipe installation could be used to help with building foundation challenges if the ground conditions allow. Whereas Scenario 4 and 5 are traditional in nature, and the main challenge is managing traffic flow on Main Street and managing dewatering due to the deep depth of the proposed improvements.

6.1.8 Long Term Serviceability

Osoyoos is likely to grow in population and change in parcel usage over time, and Scenario 1 brings the greatest challenges unless 7710 Main Street is purchased by the Town. Beach only access along with proximity to the lake makes access difficult for serving the lift station, fueling / maintaining the generator and day-to-day operational monitoring. Scenario 2 through 5 offer better access for the Town but Scenario 2 could have challenges with the existing parcel owner to obtain additional right-of-way for the lift station. Scenarios 3 to 5 would provide the most access to the Town and would allow the community to change as the parcel owners see fit with Scenario 4 and 5 having the highest flexibility as there would be no right-of-way on 7710 Main Street.

6.1.9 Capital Costs

Budgets for smaller municipalities are constrained, and as such, a proportional cost weighting is used for this aspect rather than the risk matrix. The opinion of probable capital cost was utilized to establish the ranking that was pro-rated to a value of 25 for the highest cost.

6.1.10 Operation and Maintenance

Scenario 2 through 5 have a new facility proposed, reducing the initial O&M costs along with reasonable access to the facilities. Scenario 1 would have the least access and would increase risks of maintenance costs if entirely accessed by the beach. The pipeline for Scenario 1 through 3 would present additional costs down the road if the pipeline were to leak, need replacement or repair. A bypass manhole would be needed for all three scenarios to ensure service is maintained when pipeline repairs are done within the 7710 Main Street right-of-way.

7.0 Conclusions and Recommendations

The existing Starlite lift station has considerable challenges to address future PWWF's along with primarily beach access and limited walking space access through a narrow sewer main and sewer

forcemain right-of-way along the west side of 7710 Main Street. In all the scenarios, deep depths of excavation come out as a considerable risk due to the proximity of Osoyoos Lake, proximity of nearby structures, and the probable presence of blast rock under Main Street (Highway 3).

Of the five scenarios, the Town would benefit longer term if Starlite is relocated beside Main Street to allow operators, maintenance crews and other parties easy access to the lift station in the event of planned or unplanned visits. Scenarios 4 and 5 would offer greater flexibility for future improvements by eliminating the narrow pipe right-of-way on 7710 Main Street while scenarios 1 and 2 could present property access challenges. To mitigate the risk of property access for Scenario 1 and 2, 7710 Main Street could be purchased by the Town at a cost likely between the BC Assessment value of \$941,900 and the current MLS listing (June 2023) of \$2.3 million. The purchase of the land would address lift station and pipeline access issues and could be developed to encourage greater public access to the north beach. As a compromise to Scenario 1 and 2, Scenario 3's placement of Starlite on Pioneer walkway improves access off Main Street but requires the existing pipeline right-of-way to be maintained.

7.1 Recommendations and Next Steps

The final choice for the Starlite lift station requires a multitude of tools to assist the decision-making process. If the multivariable risk assessment tool is used, Scenario 4 offers the lowest risk and focuses primarily on traditional methods of construction. Following Scenario 4 would be Scenario 1 and 5, followed by Scenario 3 and lastly Scenario 2. If capital costs are the key driving tool in the decision process, Scenario 1 would offer the lowest capital cost but existing residual risks would remain. Other tools including community acceptance, local property owner biases, mayor and council opinions, Town staff opinions and other variables should also be considered in conjunction with the multivariable risk and capital cost tools outlined in this report.

For Scenarios 1 to 3, any large scale multifamily or commercial development could proceed if the proposed pipe casing is installed within the existing right-of-way to minimize risks of needing to open a trench along the west edge of the property. To minimize risks for the Town's future repairs, rehabilitation or replacement of the sewer main within the pipe casing, any future development on 7710 Main Street should be done in consultation with the Town and the Town's engineering team.

The next step for the project would be selection of the scenario followed by a formal geotechnical investigation to better understand the detailed geology of the area. If the soils are found to have a strong connection with Osoyoos Lake, dewatering would likely become challenging and timing the work during low lake levels would be essential. In review of recent Zosel Dam reports, construction could start as soon as November 1 and be complete no later than March 31 as lake levels is permitted to drop to 909 ft from a peak permitted level of 912 ft to 912.5 ft during the period of June 1st to September 15th.

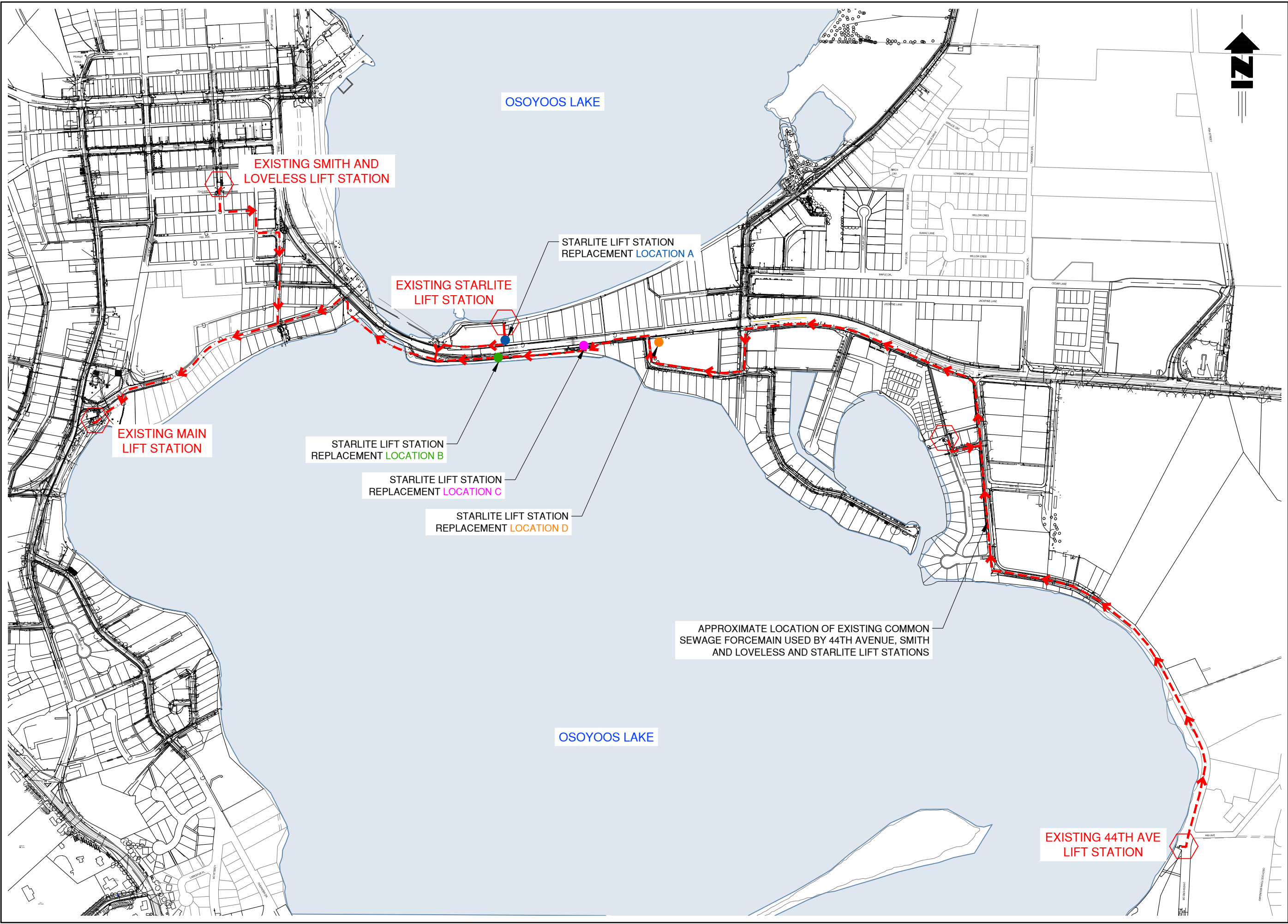
8.0 References

Nk'mip (Osoyoos Lake) The Heard of the Watershed Final Report, 2022 Osoyoos Lake Water Science Forum

2021 Annual Report to the International Joint Commission, International Osoyoos Lake Board of Control.

APPENDIX A

Figures



LEGEND		
EXISTING	PROPOSED	DESCRIPTION
		SANITARY MAIN
		SANITARY FORCEMAIN
		SANITARY MANHOLE
		CLEANOUT
		LIFT STATION
		SEPTIC TANK
		STORM MAIN
		STORM DITCH
		CULVERT
		STORM MANHOLE
		CATCHBASIN
		STORM DRYWELL
		WATER MAIN
		WATER SERVICE
		CURB STOP
		VALVE
		STANDPIPE
		HYDRANT
		AIR RELEASE VALVE
		REDUCER
		SPRINKLER
		IRRIGATION BOX
		END CAP
		POWER
		TEL
		CABLE
		STREET LIGHT
		GAS
		POWER POLE
		LAMP STANDARD
		LIGHT POST
		POLE ANCHOR
		JUNCTION BOX
		TEST PIT
		IRON PIN
		SURVEY CONTROL

CONSULTANT SEAL



**STARLITE LIFT
STATION UPGRADES
& DEVELOPMENT
CONSIDERATIONS**

**OVERALL LIFT
STATION
LOCATION
PLAN**

SCALE 1:7500 (11x17)

DESIGN BY DK

DRAWN BY MC

DATE JUNE 2023

PROJECT REFERENCE No. 302-2121

DRAWING No.

302-2121

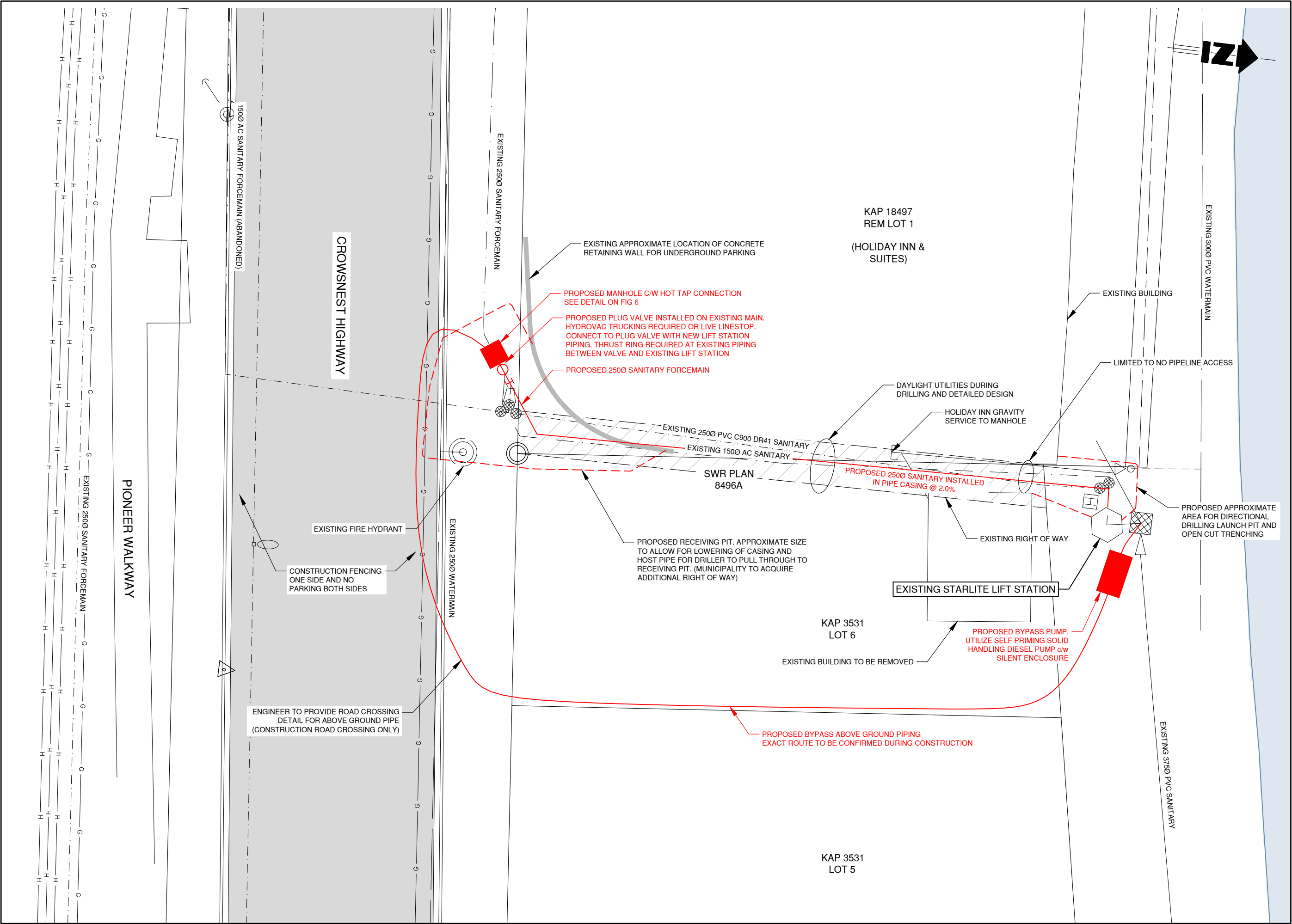
FIG.1

SHEET

OF

ISSUE/REV.

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LEGEND		
EXISTING	PROPOSED	DESCRIPTION
		SANITARY MAIN
		SANITARY FORCEMAIN
		SANITARY MANHOLE
		CLEANOUT
		LIFT STATION
		SEPTIC TANK
		STORM MAIN
		STORM DITCH
		CULVERT
		STORM MANHOLE
		CATCHBASIN
		STORM DRYWELL
		WATER MAIN
		WATER SERVICE
		CURB STOP
		VALVE
		STANDPIPE
		HYDRANT
		AIR RELEASE VALVE
		REDUCER
		SPRINKLER
		IRRIGATION BOX
		END CAP
		POWER
		TEL
		CABLE
		STREET LIGHT
		GAS
		POWER POLE
		LAMP STANDARD
		LIGHT POST
		POLE ANCHOR
		JUNCTION BOX
		TEST PIT
		IRON PIN
		SURVEY CONTROL

CONSULTANT SEAL



STARLITE LIFT STATION UPGRADES & DEVELOPMENT CONSIDERATIONS

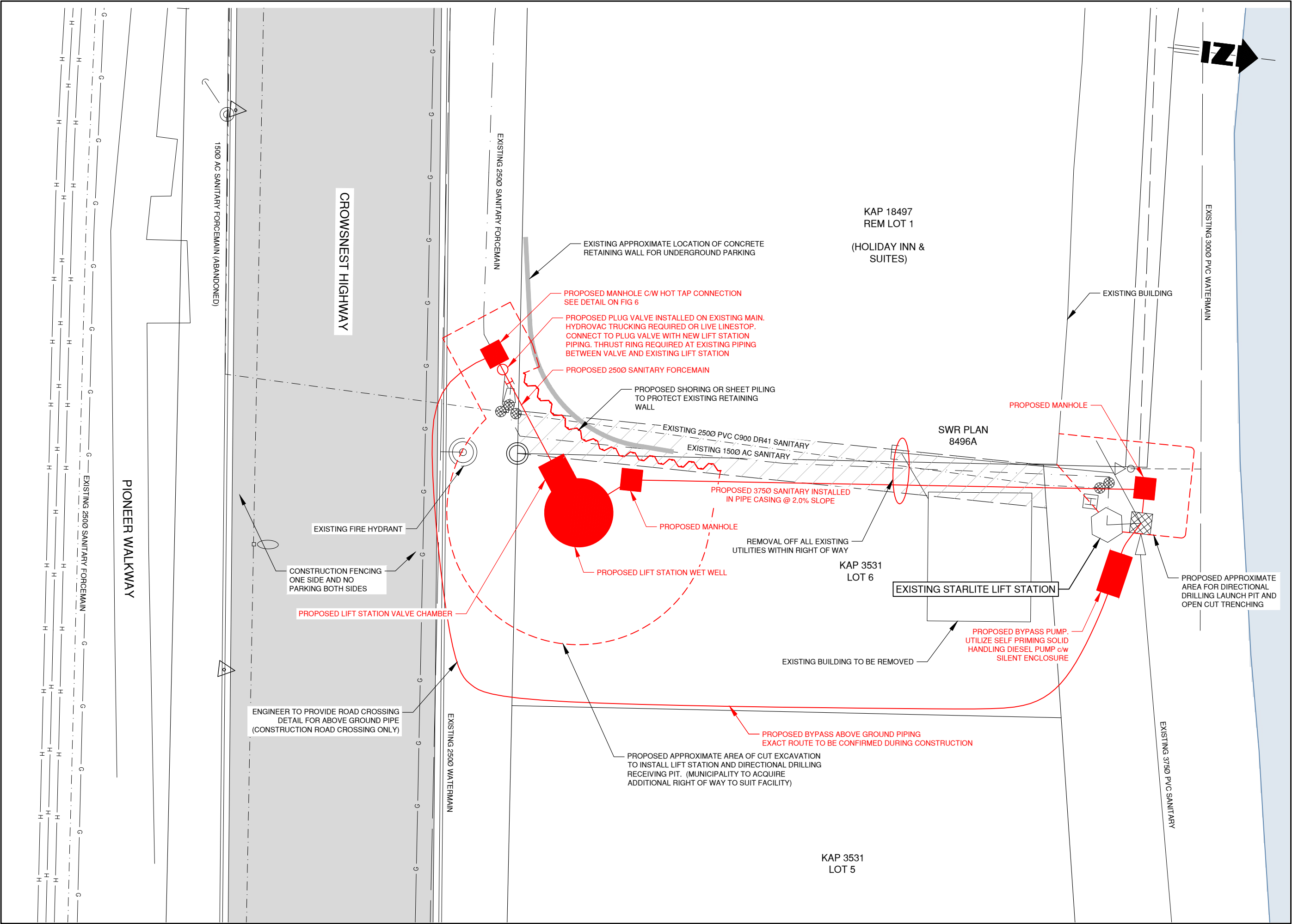
STARLITE LIFT STATION LOCATION

SCENARIO 1 EXISTING CONDITIONS

SCALE	1:250 (11x17)
DESIGN BY	DK
DRAWN BY	MC
DATE	JUNE 2023
PROJECT REFERENCE No.	302-2121

DRAWING No.	302-2121	SHEET	OF
	FIG.2	ISSUE/REV.	

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LEGEND		
EXISTING	PROPOSED	DESCRIPTION
		SANITARY MAIN
		SANITARY FORCEMAIN
		SANITARY MANHOLE
		CLEANOUT
		LIFT STATION
		SEPTIC TANK
		STORM MAIN
		STORM DITCH
		CULVERT
		STORM MANHOLE
		CATCHBASIN
		STORM DRYWELL
		WATER MAIN
		WATER SERVICE
		CURB STOP
		VALVE
		STANDPIPE
		HYDRANT
		AIR RELEASE VALVE
		REDUCER
		SPRINKLER
		IRRIGATION BOX
		END CAP
		POWER
		TEL
		CABLE
		STREET LIGHT
		GAS
		POWER POLE
		LAMP STANDARD
		LIGHT POST
		POLE ANCHOR
		JUNCTION BOX
		TEST PIT
		IRON PIN
		SURVEY CONTROL

CONSULTANT SEAL



STARLITE LIFT STATION UPGRADES & DEVELOPMENT CONSIDERATIONS

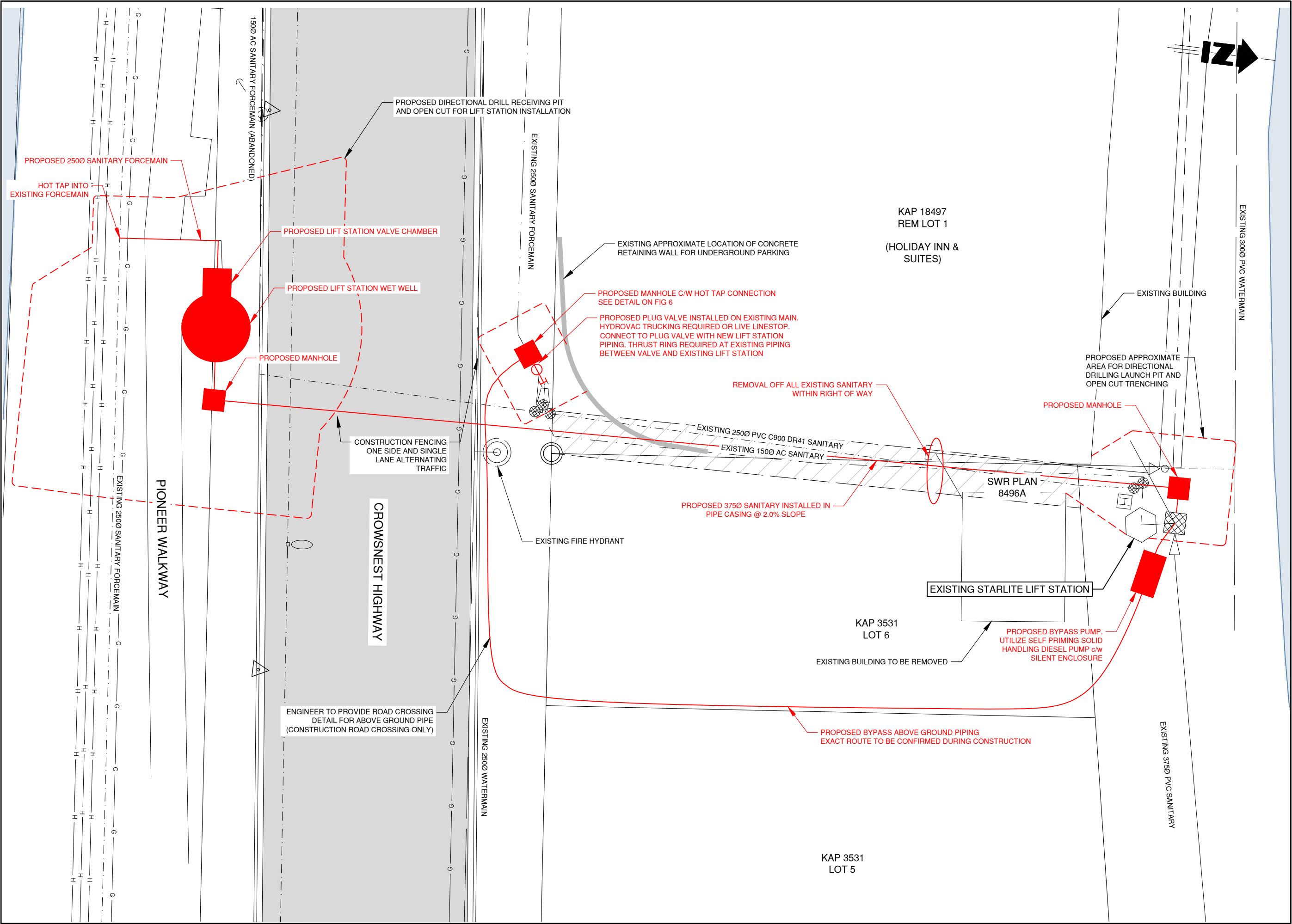
STARLITE LIFT STATION REPLACEMENT

SCENARIO 2 LOCATION A

SCALE	1:250 (11x17)
DESIGN BY	DK
DRAWN BY	MC
DATE	JUNE 2023
PROJECT REFERENCE No.	302-2121

DRAWING No.	302-2121	SHEET	OF
	FIG.3	ISSUE/REV.	

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LEGEND		
EXISTING	PROPOSED	DESCRIPTION
		SANITARY MAIN
		SANITARY FORCE MAIN
		SANITARY MANHOLE
		CLEANOUT
		LIFT STATION
		SEPTIC TANK
		STORM MAIN
		STORM DITCH
		CULVERT
		STORM MANHOLE
		CATCHBASIN
		STORM DRYWELL
		WATER MAIN
		WATER SERVICE
		CURB STOP
		VALVE
		STANDPIPE
		HYDRANT
		AIR RELEASE VALVE
		REDUCER
		SPRINKLER
		IRRIGATION BOX
		END CAP
		POWER
		TEL
		CABLE
		STREET LIGHT
		GAS
		POWER POLE
		LAMP STANDARD
		LIGHT POST
		POLE ANCHOR
		JUNCTION BOX
		TEST PIT
		IRON PIN
		SURVEY CONTROL

CONSULTANT SEAL



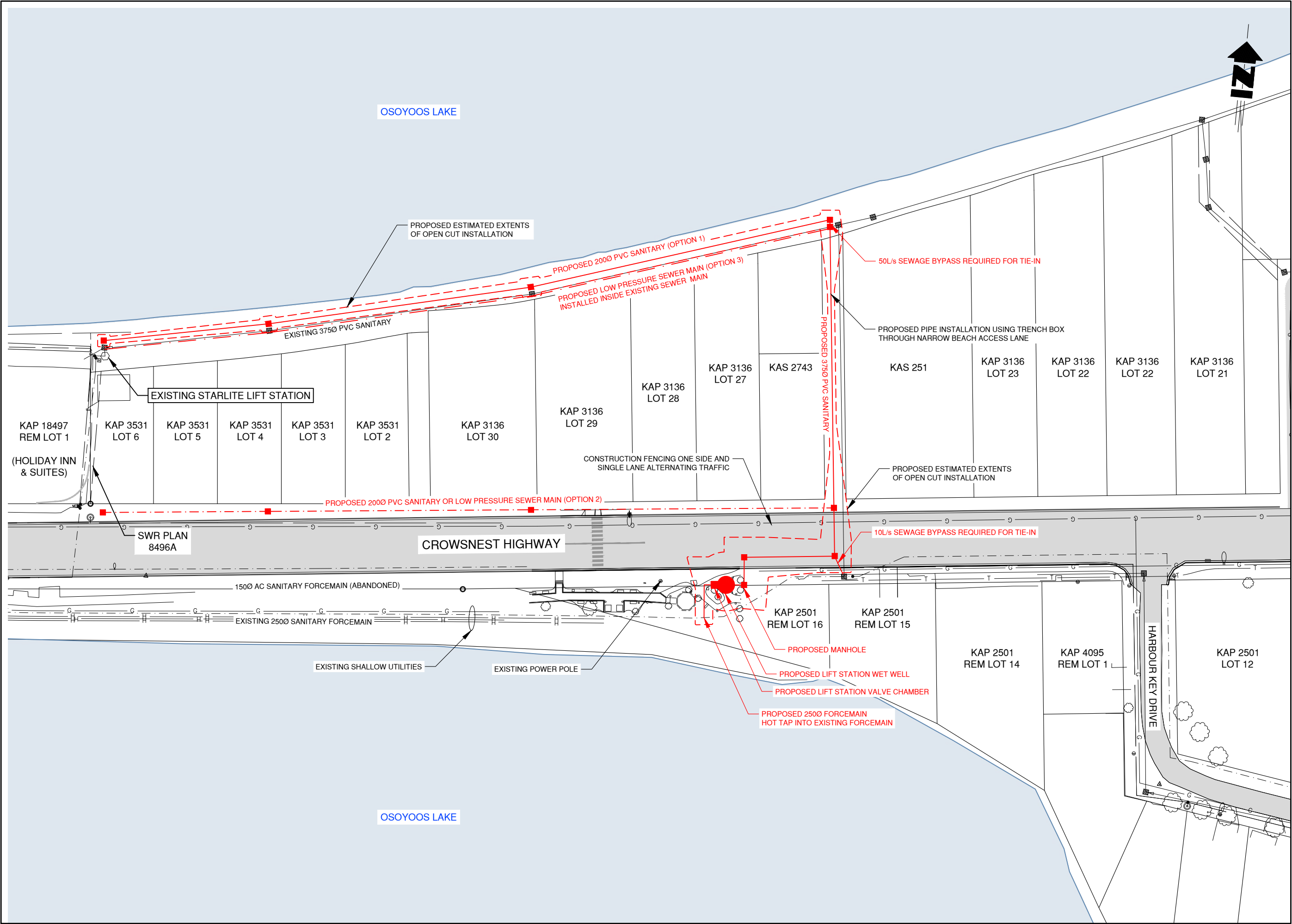
**STARLITE LIFT
STATION UPGRADES
& DEVELOPMENT
CONSIDERATIONS**

**STARLITE LIFT
STATION
REPLACEMENT**

**SCENARIO 3
LOCATION B**

SCALE	1:250 (11x17)
DESIGN BY	DK
DRAWN BY	MC
DATE	JUNE 2023
PROJECT REFERENCE No.	302-2121
DRAWING No.	302-2121
SHEET	OF
ISSUE/REV.	
FIG.4	

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LEGEND		
EXISTING	PROPOSED	DESCRIPTION
		SANITARY MAIN
		SANITARY FORCEMAIN
		SANITARY MANHOLE
		CLEANOUT
		LIFT STATION
		SEPTIC TANK
		STORM MAIN
		STORM DITCH
		CULVERT
		STORM MANHOLE
		CATCHBASIN
		STORM DRYWELL
		WATER MAIN
		WATER SERVICE
		CURB STOP
		VALVE
		STANDPIPE
		HYDRANT
		AIR RELEASE VALVE
		REDUCER
		SPRINKLER
		IRRIGATION BOX
		END CAP
		POWER
		TEL
		CABLE
		STREET LIGHT
		GAS
		POWER POLE
		LAMP STANDARD
		LIGHT POST
		POLE ANCHOR
		JUNCTION BOX
		TEST PIT
		IRON PIN
		SURVEY CONTROL

CONSULTANT SEAL



STARLITE LIFT STATION UPGRADES & DEVELOPMENT CONSIDERATIONS

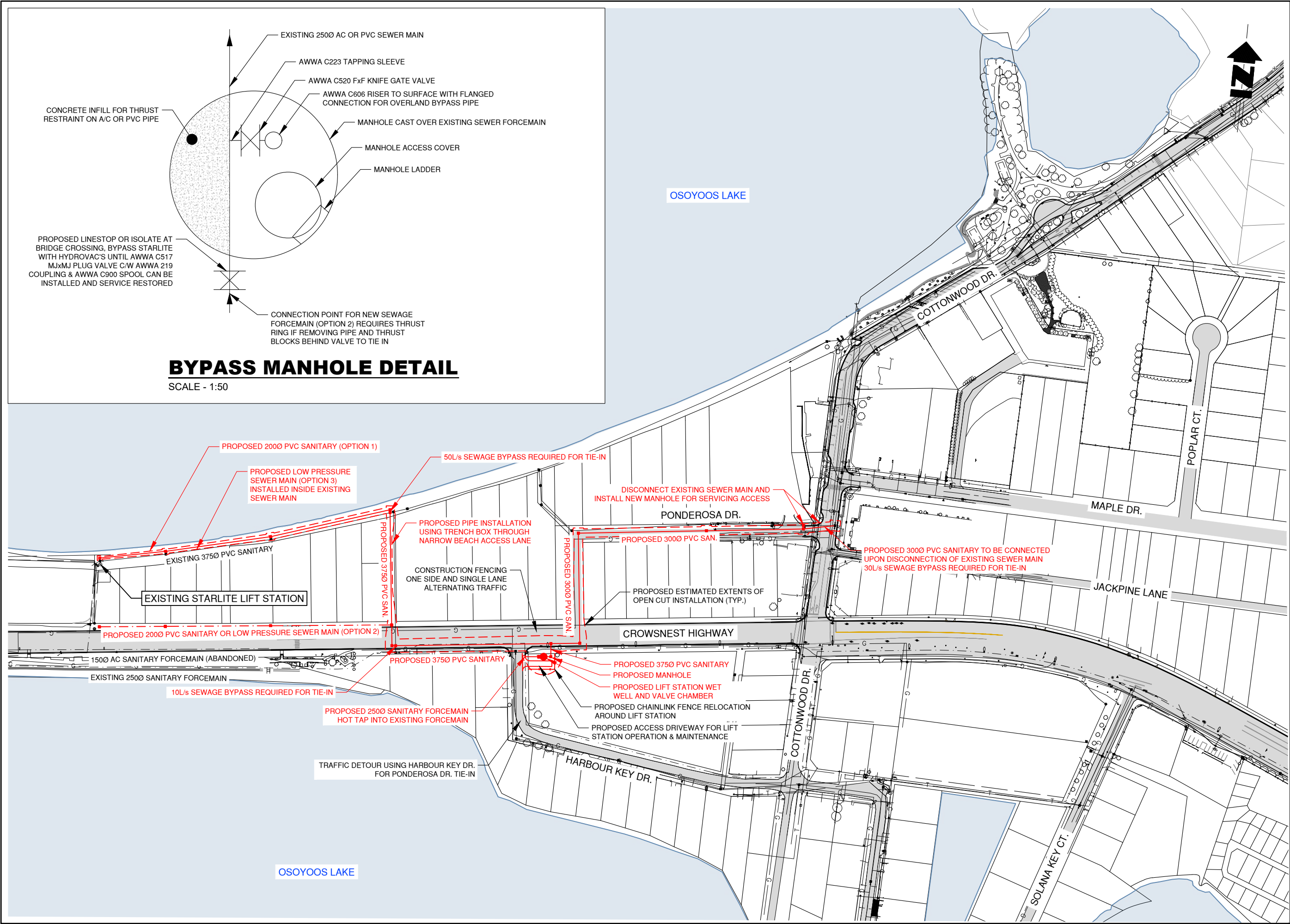
STARLITE LIFT STATION REPLACEMENT

SCENARIO 4 LOCATION C

SCALE	1:1000 (11x17)
DESIGN BY	DK
DRAWN BY	MC
DATE	JUNE 2023
PROJECT REFERENCE No.	302-2121
DRAWING No.	302-2121
SHEET	OF
ISSUE/REV.	

FIG.5

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BYPASS MANHOLE DETAIL

SCALE - 1:50

LEGEND		
EXISTING	PROPOSED	DESCRIPTION
		SANITARY MAIN
		SANITARY FORCEMAIN
		SANITARY MANHOLE
		CLEANOUT
		LIFT STATION
		SEPTIC TANK
		STORM MAIN
		STORM DITCH
		CULVERT
		STORM MANHOLE
		CATCHBASIN
		STORM DRYWELL
		WATER MAIN
		WATER SERVICE
		CURB STOP
		VALVE
		STANDPIPE
		HYDRANT
		AIR RELEASE VALVE
		REDUCER
		SPRINKLER
		IRRIGATION BOX
		END CAP
		POWER
		TEL
		CABLE
		STREET LIGHT
		GAS
		POWER POLE
		LAMP STANDARD
		LIGHT POST
		POLE ANCHOR
		JUNCTION BOX
		TEST PIT
		IRON PIN
		SURVEY CONTROL

CONSULTANT SEAL



STARLITE LIFT STATION UPGRADES & DEVELOPMENT CONSIDERATIONS

STARLITE LIFT STATION REPLACEMENT

SCENARIO 5 LOCATION D

SCALE	1:2500 (11x17)
DESIGN BY	DK
DRAWN BY	MC
DATE	JUNE 2023
PROJECT REFERENCE No.	302-2121
DRAWING No.	302-2121
SHEET	OF
ISSUE/REV.	

FIG.6

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APPENDIX B

Cost Estimates



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
 Scenario 1: Maintain Existing Lift Station Location

PAYMENT				UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA	DESCRIPTION				
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$2,000	\$2,000
Subtotal Part 1.0							\$2,000
PART 2.0 - UTILITIES							
2.01	33 30 01	1.6.8	Temporary Sewer Bypass	Day	20	\$16,000	\$320,000
2.02	33 30 01	1.6.9	Trenchless Pipe Installation of Pipe Casing c/w Casing Spacers and End Seals	LS	1	\$380,000	\$380,000
2.03	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	I.m.	5	\$500	\$2,500
2.04	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Restrained and Installed in Casing Pipe	I.m.	45	\$250	\$11,250
2.05	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ-MJ	Each	2	\$8,000	\$16,000
2.06	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	2	\$4,000	\$8,000
2.07	33 34 01	1.8.9	Reinforced Concrete Thrust Ring	Each	1	\$8,000	\$8,000
2.08	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	2	\$20,000	\$40,000
2.09	33 34 01	1.8.11	Sewage Bypass Manhole c/w AWWA C223 Tapping Sleeve, AWWA C520 Knife Gate Valve and AWWA C606 Fittings/Flanges	LS	1	\$35,000	\$35,000
2.10	33 44 01	1.8.12	67hp AWWA E110 Flygt Submersible Pump	Each	2	\$117,000	\$234,000
Subtotal Part 2.0							\$1,054,750
PART 3.0 - ELECTRICAL							
3.01	26 XX XX SS		New VFD Kiosk	LS	1	\$75,000	\$75,000
3.02	26 XX XX SS		New Generator	LS	1	\$80,000	\$80,000
3.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
3.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$70,000	\$70,000
Subtotal Part 3.0							\$235,000

PAYMENT				EST.	UNIT	TOTAL	
ITEM	SECTION	PARA	DESCRIPTION	UNIT	QUANT.	PRICE	PAYMENT
SUMMARY							
			Part 1.0 - General Requirements				\$2,000
			Part 2.0 - Utilities				\$1,054,750
			Part 3.0 - Electrical				\$235,000
			Subtotal Parts 1.0 to 3.0				\$1,291,750
			Construction Risk & Contingency Allowance (50%)				\$646,000
			Engineering Fee Allowance (16.1%)				\$208,000
			GST (5% of Subtotal and Contingency Allowance)				\$107,000
			Total Contract Sum (SAY)				\$2,250,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 2: Location A Replacement Lift Station

PAYMENT				UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA	DESCRIPTION				
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$2,000	\$2,000
Subtotal Part 1.0							\$2,000
PART 2.0 - UTILITIES							
2.01	33 01 30.1		CCTV Pipeline Inspection	l.m.	40	\$50	\$2,000
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
2.02	33 30 01		Remove and dispose of existing sewer system Mainline Piping	l.m.	80	\$150	\$12,000
2.03	33 30 01		Remove and dispose of existing sewer system Service Piping	l.m.	20	\$80	\$1,600
2.04	33 30 01	1.6.3	Sanitary Service Connections, Disconnect from existing and Connect to New Main	Each	1	\$1,000	\$1,000
2.05	33 30 01	1.6.1, 1.6.2	Service Sewer Pipe PVC DR28, Re-grade to Existing Building, Allowance	l.m.	20	\$250	\$5,000
2.06	33 30 01	1.6.7	Sanitary Tie-In, existing manhole to new manhole	Each	1	\$8,000	\$8,000
2.07	33 30 01	1.6.8	Temporary Sewer Bypass	Day	30	\$16,000	\$480,000
2.08	33 30 01	1.6.9	Trenchless Pipe Installation of Pipe Casing c/w Casing Spacers and End Seals	LS	1	\$380,000	\$380,000
2.09	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	l.m.	10	\$500	\$5,000
2.10	33 34 01	1.8.2, 1.8.3	Sewer Pipe AWWA C900 PVC DR18 375 mm diameter, Restrained and Installed in Casing Pipe	l.m.	40	\$350	\$14,000
2.11	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ-MJ	Each	1	\$8,000	\$8,000
2.12	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	2	\$4,000	\$8,000
2.13	33 34 01	1.8.9	Reinforced Concrete Thrust Ring	Each	1	\$8,000	\$8,000
2.14	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	1	\$20,000	\$20,000
2.15	33 34 01	1.8.11	Sewage Bypass Manhole c/w AWWA C223 Tapping Sleeve, AWWA C520 Knife Gate Valve and AWWA C606 Fittings/Flanges	LS	1	\$35,000	\$35,000
2.16	33 44 01	1.8.12	67hp AWWA E110 Flygt Submersible Pump	Each	2	\$117,000	\$234,000

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 2.0 - UTILITIES (Continued)							
2.17	33 44 01	1.8.13	Xylem FRP Wet Well and Valve Chamber c/w Flygt guide rails, AWWA C220 piping, AWWA C226/C228/C606 fittings, AWWA C520 knife gate valves, AWWA C504 check valve, AWWA C512 combo air valve, AWWA C751 Flowmeter, Carbon Odor Control, Dexon Blower and Pressure Indication.	LS	1	\$510,000	\$510,000
2.19	33 44 01	1.5.1.1	Manhole: base, lid, slab,cover and frame 1200mm diameter	Each	2	\$9,000	\$18,000
2.20	33 44 01	1.5.1.4	Re-bench Existing Manhole	Each	1	\$2,000	\$2,000
2.21	33 44 01	1.5.1.2	Manhole Riser and Tee Riser 1200mm diameter	v.m.	12	\$2,000	\$24,000
2.22	33 30 01		Sheet Pile Wall and Dewatering Allowance	LS	1	\$600,000	\$600,000
Subtotal Part 2.0							\$2,425,600

PART 3.0 - ELECTRICAL

3.01	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
3.02	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
3.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
3.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
Subtotal Part 3.0							\$345,000

SUMMARY

Part 1.0 - General Requirements	\$2,000
Part 2.0 - Utilities	\$2,425,600
Part 3.0 - Electrical	\$345,000
Subtotal Parts 1.0 to 3.0	\$2,772,600
Construction Risk & Contingency Allowance (50%)	\$1,386,000
Engineering Fee Allowance (14.5%)	\$402,000
GST (5% of Subtotal and Contingency Allowance)	\$228,000
Total Contract Sum (SAY)	\$4,790,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 3: Location B Replacement Lift Station

PAYMENT				EST.	UNIT	TOTAL	
ITEM	SECTION	PARA	DESCRIPTION	UNIT	QUANT.	PRICE	PAYMENT
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$10,000	\$10,000
Subtotal Part 1.0							\$10,000
PART 2.0 - UTILITIES							
2.01	33 01 30.1		CCTV Pipeline Inspection	l.m.	75	\$50	\$3,750
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
2.02	33 30 01		Remove and dispose of existing sewer system Mainline Piping	l.m.	80	\$150	\$12,000
2.03	33 30 01		Remove and dispose of existing sewer system Service Piping	l.m.	20	\$80	\$1,600
2.04	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 375mm diameter, Native Backfill	l.m.	5	\$1,100	\$5,500
2.05	33 30 01	1.6.3	Sanitary Service Connections, Disconnect from existing and Connect to New Main	Each	1	\$1,000	\$1,000
2.06	33 30 01	1.6.1, 1.6.2	Service Sewer Pipe PVC DR28, Re-grade to Existing Building, Allowance	l.m.	20	\$250	\$5,000
2.07	33 30 01	1.6.7	Sanitary Tie-In, existing manhole to new manhole	Each	1	\$8,000	\$8,000
2.08	33 30 01	1.6.8	Temporary Sewer Bypass	Day	14	\$16,000	\$224,000
2.09	33 30 01	1.6.9	Trenchless Pipe Installation of Pipe Casing c/w Casing Spacers and End Seals	LS	1	\$380,000	\$380,000
2.10	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	l.m.	10	\$500	\$5,000
2.11	33 34 01	1.8.2, 1.8.3	Sewer Pipe AWWA C900 PVC DR18 375 mm diameter, Restrained and Installed in Casing Pipe	l.m.	70	\$350	\$24,500
2.12	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ-MJ	Each	1	\$8,000	\$8,000
2.13	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	1	\$4,000	\$4,000
2.14	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	1	\$20,000	\$20,000
2.15	33 34 01	1.8.11	Sewage Bypass Manhole c/w AWWA C223 Tapping Sleeve, AWWA C520 Knife Gate Valve and AWWA C606 Fittings/Flanges	LS	1	\$35,000	\$35,000
2.16	33 44 01	1.8.12	44hp AWWA E110 Flygt Submersible Pump	Each	2	\$83,000	\$166,000

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 2.0 - UTILITIES (Continued)							
2.17	33 44 01	1.8.13	Xylem FRP Wet Well and Valve Chamber c/w Flygt guide rails, AWWA C220 piping, AWWA C226/C228/C606 fittings, AWWA C520 knife gate valves, AWWA C504 check valve, AWWA C512 combo air valve, AWWA C751 Flowmeter, Carbon Odor Control, Dexon Blower and Pressure Indication.	LS	1	\$510,000	\$510,000
2.18	33 44 01	1.5.1.1	Manhole: base, lid, slab,cover and frame 1200mm diameter	Each	2	\$9,000	\$18,000
2.19	33 44 01	1.5.1.4	Re-bench Existing Manhole	Each	1	\$2,000	\$2,000
2.20	33 44 01	1.5.1.2	Manhole Riser and Tee Riser 1200mm diameter	v.m.	14	\$2,000	\$28,000
5.32	33 30 01		Dewatering Allowance	LS	1	\$100,000	\$100,000
Subtotal Part 2.0							\$1,611,350
PART 3.0 - ELECTRICAL							
3.01	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
3.02	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
3.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
3.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
Subtotal Part 3.0							\$345,000
SUMMARY							
Part 1.0 - General Requirements							\$10,000
Part 2.0 - Utilities							\$1,611,350
Part 3.0 - Electrical							\$345,000
Subtotal Parts 1.0 to 3.0							\$1,966,350
Construction Risk & Contingency Allowance (50%)							\$983,000
Engineering Fee Allowance (16.1%)							\$317,000
GST (5% of Subtotal and Contingency Allowance)							\$163,000
Total Contract Sum (SAY)							\$3,430,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 4: Location C Replacement Lift Station (Option 1 & 2 Gravity Conveyance)

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$20,000	\$20,000
Subtotal Part 1.0							\$20,000
PART 2.0 - UTILITIES							
2.01	33 01 30.1		CCTV Pipeline Inspection	I.m.	340	\$15	\$5,100
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
2.02	33 30 01		Remove and dispose of existing sewer system Service Piping	I.m.	240	\$80	\$19,200
2.03	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 200mm diameter, Native Backfill	I.m.	210	\$350	\$73,500
2.04	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 375mm diameter, Native Backfill	I.m.	130	\$1,100	\$143,000
2.05	33 30 01	1.6.3	Sanitary Service Connections, Disconnect from existing and Connect to New Main	Each	12	\$1,000	\$12,000
2.06	33 30 01	1.6.1, 1.6.2	Service Sewer Pipe PVC DR28, Re-grade to Existing Building, Allowance	I.m.	240	\$250	\$60,000
2.07	33 30 01	1.6.7	Sanitary Tie-In, existing manhole to new manhole	Each	2	\$8,000	\$16,000
2.08	33 30 01	1.6.8	Temporary Sewer Bypass	Day	4	\$16,000	\$64,000
2.09	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	I.m.	12	\$500	\$6,000
2.10	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ-MJ	Each	1	\$8,000	\$8,000
2.11	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	1	\$4,000	\$4,000
2.12	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	1	\$20,000	\$20,000
2.13	33 44 01	1.8.12	54hp AWWA E110 Flygt Submersible Pump	Each	2	\$98,000	\$196,000
2.14	33 44 01	1.8.13	Xylem FRP Wet Well and Valve Chamber c/w Flygt guide rails, AWWA C220 piping, AWWA C226/C228/C606 fittings, AWWA C520 knife gate valves, AWWA C504 check valve, AWWA C512 combo air valve, AWWA C751 Flowmeter, Carbon Odor Control, Dexon Blower and Pressure Indication.	LS	1	\$510,000	\$510,000
2.15	33 44 01	1.5.1.1	Manhole: base, lid, slab,cover and frame 1200mm diameter	Each	9	\$9,000	\$81,000

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 2.0 - UTILITIES (Continued)							
2.16	33 44 01	1.5.1.4	Re-bench Existing Manhole	Each	1	\$2,000	\$2,000
2.17	33 44 01	1.5.1.6	Add Manhole to Existing System 1200mm diameter	Each	1	\$15,000	\$15,000
2.18	33 44 01	1.5.1.2	Manhole Riser and Tee Riser 1200mm diameter	v.m.	21	\$2,000	\$42,000
5.32	33 30 01		Dewatering Allowance	LS	1	\$100,000	\$100,000
Subtotal Part 2.0							\$1,426,800

PART 3.0 - ELECTRICAL							
3.01	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
3.02	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
3.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
3.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
Subtotal Part 3.0							\$345,000

SUMMARY	
Part 1.0 - General Requirements	\$20,000
Part 2.0 - Utilities	\$1,426,800
Part 3.0 - Electrical	\$345,000
Subtotal Parts 1.0 to 3.0	\$1,791,800
Construction Risk & Contingency Allowance (50%)	\$896,000
Engineering Fee Allowance (16.1%)	\$288,000
GST (5% of Subtotal and Contingency Allowance)	\$149,000
Total Contract Sum (SAY)	\$3,120,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 4: Location C Replacement Lift Station (Option 3 STEP Systems)

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$20,000	\$20,000
Subtotal Part 1.0							\$20,000
PART 2.0 - UTILITIES							
2.01	33 01 30.1		CCTV Pipeline Inspection	I.m.	131	\$30	\$3,930
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
2.02	33 30 01		Remove and dispose of existing sewer system Service Piping	I.m.	240	\$80	\$19,200
2.03	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 375mm diameter, Native Backfill	I.m.	131	\$1,100	\$144,100
2.04	33 30 01	1.6.3	Sanitary Service Connections, Disconnect from existing and Connect to New Main	Each	12	\$1,000	\$12,000
2.05	33 30 01	1.6.7	Sanitary Tie-In, existing manhole to new manhole	Each	2	\$8,000	\$16,000
2.06	33 30 01	1.6.8	Temporary Sewer Bypass	Day	10	\$16,000	\$160,000
2.07	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	I.m.	12	\$500	\$6,000
2.08	33 34 01	1.8.2, 1.8.3	Sewer Pipe AWWA C906 HDPE DR11, Common Main for STEP Systems	I.m.	210	\$200	\$42,000
2.09	33 34 01	1.8.2, 1.8.3	Sewer Pipe AWWA C901 HDPE DR9 50mm diameter, Service pipe for STEP Systems	I.m.	240	\$250	\$60,000
2.10	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ-MJ	Each	1	\$8,000	\$8,000
2.11	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	1	\$4,000	\$4,000
2.12	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	1	\$20,000	\$20,000
2.13	33 44 01	1.8.12	54hp AWWA E110 Flygt Submersible Pump	Each	2	\$98,000	\$196,000
2.14	33 44 01	1.8.13	Xylem FRP Wet Well and Valve Chamber c/w Flygt guide rails, AWWA C220 piping, AWWA C226/C228/C606 fittings, AWWA C520 knife gate valves, AWWA C504 check valve, AWWA C512 combo air valve, AWWA C751 Flowmeter, Carbon Odor Control, Dexon Blower and Pressure Indication.	LS	1	\$510,000	\$510,000

PAYMENT				EST.	UNIT	TOTAL	
ITEM	SECTION	PARA	DESCRIPTION	UNIT	QUANT.	PRICE	PAYMENT
PART 2.0 - UTILITIES (Continued)							
2.15	33 44 01	1.5.1.1	Manhole: base, lid, slab,cover and frame 1200mm diameter	Each	4	\$9,000	\$36,000
2.16	33 44 01	1.5.1.4	Re-bench Existing Manhole	Each	1	\$2,000	\$2,000
2.17	33 44 01	1.5.1.6	Add Manhole to Existing System 1200mm diameter	Each	1	\$15,000	\$15,000
2.18	33 44 01	1.5.1.2	Manhole Riser and Tee Riser 1200mm diameter	v.m.	17	\$2,000	\$34,000
2.19	33 34 01		Low Pressure STEP System (one per Lot)	Each	12	\$28,000	\$336,000
5.32	33 30 01		Dewatering Allowance	LS	1	\$100,000	\$100,000
Subtotal Part 2.0							\$1,774,230

PART 3.0 - ELECTRICAL

3.01	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
3.02	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
3.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
3.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
Subtotal Part 3.0							\$345,000

SUMMARY

Part 1.0 - General Requirements	\$20,000
Part 2.0 - Utilities	\$1,774,230
Part 3.0 - Electrical	\$345,000
Subtotal Parts 1.0 to 3.0	\$2,139,230
Construction Risk & Contingency Allowance (50%)	\$1,070,000
Engineering Fee Allowance (14.5%)	\$310,000
GST (5% of Subtotal and Contingency Allowance)	\$176,000
Total Contract Sum (SAY)	\$3,700,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 5: Location D Replacement Lift Station (Option 1 & 2 Gravity Conveyance)

PAYMENT				UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA	DESCRIPTION				
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$75,000	\$75,000
Subtotal Part 1.0							\$75,000
PART 2.0 - EARTHWORKS							
2.01	31 11 07	1.4.1, 1.4.2	Clearing and Grubbing	m ²	250	\$4	\$1,000
2.02	31 22 01	1.4.1	Topsoil Stripping and Reuse	m ²	250	\$6	\$1,500
Subtotal Part 2.0							\$2,500
PART 3.0 - ROADS AND SITE IMPROVEMENTS							
3.01	32 11 16.1	1.4.2	Granular Sub-Base 300mm Thickness for Roads or Sidewalks (Access Driveway)	m ²	100	\$30	\$3,000
3.02	32 11 23	1.4.2	Granular Base 100mm Thickness for Roads or Sidewalks (Access Driveway)	m ²	100	\$20	\$2,000
3.03	32 12 16	1.5.7	Saw Cut Asphalt or Concrete Pavements (Access Driveway)	lm	15	\$20	\$300
3.04	34 31 13	1.5.3	Remove & Reinstall Chain Link Fence (Lift Station Outside Existing Fence)	l.m.	30	\$80	\$2,400
Subtotal Part 3.0							\$7,700
PART 4.0 - UTILITIES							
4.01	33 01 30.1		CCTV Pipeline Inspection	l.m.	705	\$15	\$10,575
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
4.02	33 30 01		Remove and dispose of existing sewer system Service Piping	l.m.	240	\$80	\$19,200
4.03	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 200mm diameter, Native Backfill	l.m.	210	\$350	\$73,500
4.04	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 300mm diameter, Native Backfill	l.m.	280	\$450	\$126,000
4.05	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 375mm diameter, Native Backfill	l.m.	215	\$1,100	\$236,500
4.06	33 30 01	1.6.3	Sanitary Service Connections, Disconnect from existing and Connect to New Main	Each	12	\$1,000	\$12,000
4.07	33 30 01	1.6.1, 1.6.2	Service Sewer Pipe PVC DR28, Re-grade to Existing Building, Allowance	l.m.	240	\$250	\$60,000

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 4.0 - UTILITIES (Continued)							
4.08	33 30 01	1.6.7	Sanitary Tie-In, existing manhole to new manhole	Each	3	\$8,000	\$24,000
4.09	33 30 01	1.6.8	Temporary Sewer Bypass	Day	7	\$16,000	\$112,000
4.10	33 34 01	1.8.2, 1.8.3	Forcemain Pipe AWWA C900 PVC DR18 250 mm diameter, Native Backfill	l.m.	10	\$500	\$5,000
4.11	33 34 01	1.8.3	Forcemain AWWA C517 Plug Valves 250mm, MJ- MJ	Each	1	\$8,000	\$8,000
4.12	33 34 01	1.8.3	Forcemain AWWA C110 Elbows, 250mm, H-H c/w AWWA C111 Restraints and AWWA C217 Protection	Each	2	\$4,000	\$8,000
4.13	33 34 01	1.8.10	Forcemain Tie-In, 250mm diameter into existing 250mm diameter	Each	1	\$20,000	\$20,000
4.14	33 44 01	1.8.12	54hp AWWA E110 Flygt Submersible Pump	Each	2	\$98,000	\$196,000
4.15	33 44 01	1.8.13	Xylem FRP Wet Well and Valve Chamber c/w Flygt guide rails, AWWA C220 piping, AWWA C226/C228/C606 fittings, AWWA C520 knife gate valves, AWWA C504 check valve, AWWA C512 combo air valve, AWWA C751 Flowmeter, Carbon Odor Control, Dexon Blower and Pressure Indication.	LS	1	\$510,000	\$510,000
4.16	33 44 01	1.5.1.1	Manhole: base, lid, slab,cover and frame 1200mm diameter	Each	12	\$9,000	\$108,000
4.17	33 44 01	1.5.1.4	Re-bench Existing Manhole	Each	2	\$2,000	\$4,000
4.18	33 44 01	1.5.1.6	Add Manhole to Existing System 1200mm diameter	Each	2	\$15,000	\$30,000
4.19	33 44 01	1.5.1.2	Manhole Riser and Tee Riser 1200mm diameter	v.m.	52	\$2,000	\$104,000
4.20	33 30 01		Dewatering Allowance	LS	1	\$150,000	\$150,000
						Subtotal Part 4.0	\$1,866,775
PART 5.0 - ELECTRICAL							
5.01	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
5.02	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
5.03	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
5.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
						Subtotal Part 5.0	\$345,000

SUMMARY

Part 1.0 - General Requirements	\$75,000
Part 2.0 - Earthworks	\$2,500
Part 3.0 - Roads and Site Improvements	\$7,700
Part 4.0 - Utilities	\$1,866,775
Part 5.0 - Electrical	\$345,000
Subtotal Parts 1.0 to 5.0	\$2,296,975
Construction Risk & Contingency Allowance (50%)	\$1,148,000
Engineering Fee Allowance (14.5%)	\$333,000
GST (5% of Subtotal and Contingency Allowance)	\$189,000
Total Contract Sum (SAY)	\$3,970,000

Prepared by: D. Kronebusch, P. Eng.



Town of Osoyoos
Starlite Lift Station Relocation, Upgrade and Development Feasibility
Opinion of Probable Capital Cost Estimate
Scenario 5: Location D Replacement Lift Station (Option 3 STEP Systems)

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 1.0 - GENERAL REQUIREMENTS							
1.01	01 55 00	1.5.1	Traffic control, vehicle access, and site fencing	LS	1	\$75,000	\$75,000
Subtotal Part 1.0							\$75,000
PART 2.0 - EARTHWORKS							
2.02	31 11 07	1.4.1, 1.4.2	Clearing and Grubbing	m²	250	\$6	\$1,500
3.03	31 22 01	1.4.1	Topsoil Stripping and Reuse	m²	250	\$4	\$1,000
Subtotal Part 2.0							\$2,500
PART 3.0 - ROADS AND SITE IMPROVEMENTS							
3.01	32 11 16.1	1.4.2	Granular Sub-Base 300mm Thickness for Roads or Sidewalks (Incidental)	m²	100	\$30	\$3,000
3.02	32 11 23	1.4.2	Granular Base 100mm Thickness for Roads or Sidewalks (Incidental)	m²	100	\$20	\$2,000
3.03	32 12 13.1	1.5.1	Asphalt Tack Coat (Incidental)	Note	-	-	-
3.04	32 12 13.2	1.5.1	Asphalt Prime (Incidental)	Note	-	-	-
3.05	32 12 16	1.5.7	Saw Cut Asphalt or Concrete Pavements (Incidental)	lm	15	\$20	\$300
3.06	32 12 16	1.5.8	Permanent Pavement Restoration (Incidental)	Note	-	-	-
3.07	32 17 23	1.5.2	Permanent Painted Pavement Markings (Incidental)	Note	-	-	-
3.08	34 31 13	1.5.3	Remove & Reinstall Chain Link Fence (Lift Station Outside Existing Fence)	l.m.	30	\$80	\$2,400
3.09	32 92 23	1.8.2	Nursery Sod	Note	-	-	-
3.10	32 93 01	1.9.1	Remove, and Reinstall Existing Trees, Shrubs and Ground Cover	Note	-	-	-
Subtotal Part 3.0							\$7,700
PART 4.0 - UTILITIES							
4.01	33 01 30.1		CCTV Pipeline Inspection	l.m.	495	\$30	\$14,850
2.02	33 30 01		Abandon / Remove existing lift station and electrical	LS	1	\$50,000	\$50,000
4.02	33 30 01		Remove and dispose of existing sewer system Service Piping	l.m.	240	\$80	\$19,200
4.03	33 30 01	1.6.1, 1.6.2	Sewer Pipe PVC DR35 300mm diameter, Native Backfill	l.m.	280	\$450	\$126,000

Subtotal Part 4.0	\$2,211,550

PAYMENT			DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PAYMENT
ITEM	SECTION	PARA					
PART 5.0 - ELECTRICAL							
5.04	26 XX XX SS		New VFD Kiosk	LS	1	\$175,000	\$175,000
5.04	26 XX XX SS		New Generator	LS	1	\$100,000	\$100,000
5.04	26 XX XX SS		BC Hydro Costs	LS	1	\$10,000	\$10,000
5.04	26 XX XX SS		Conduits, Cables, Wiring, and Misc	LS	1	\$60,000	\$60,000
Subtotal Part 5.0							\$345,000

SUMMARY							
			Part 1.0 - General Requirements				\$75,000
			Part 2.0 - Earthworks				\$2,500
			Part 3.0 - Roads and Site Improvements				\$7,700
			Part 4.0 - Utilities				\$2,211,550
			Part 5.0 - Electrical				\$345,000
			Subtotal Parts 1.0 to 5.0				\$2,641,750
			Construction Risk & Contingency Allowance (50%)				\$1,321,000
			Engineering Fee Allowance (14.5%)				\$383,000
			GST (5% of Subtotal and Contingency Allowance)				\$217,000
			Total Contract Sum (SAY)				\$4,560,000

Prepared by: D. Kronebusch, P. Eng.



September 12, 2014

302-1474

Town of Osoyoos

Long Range Sanitary Sewer System Expansion Plan

DISCUSSION PAPER

Background

Since 1978, the Town of Osoyoos has practiced beneficial re-use (irrigation) for the disposal of treated wastewater. The Town was the first system in British Columbia to use treated wastewater for irrigation on a golf course. In the period 1978 to date, the reclaimed water irrigation system has been expanded onto an additional 27 holes on the Golf Course, a playing field complex in the infield of the Desert Park horseracing track, and ball diamonds. In 2015, reclaimed water will be used to irrigate the playing fields at the Osoyoos Secondary School.

In conjunction with a Liquid Waste Management Plan undertaken in the mid-1980's, it was recognized that the long term viability of the Town's land based reclaimed water re-use strategy for wastewater effluent disposal was dependent on a land base for irrigation purposes. At this time, the Town was encouraged by Provincial Agencies to register reserves on vacant, natural state Crown land in area of the landfill for future "spray irrigation" with treated wastewater effluent. The Town proceeded with registration of reserves over Crown land north and south of the landfill access road for spray irrigation purposes.

Through the 1980's, Provincial Agencies became increasingly aware of the importance of vacant, undeveloped Crown land on the Osoyoos West Bench as natural state habitat. In 1998, the Town of Osoyoos relinquished its reserve on Crown lands located on the south side of the landfill access road to enable the Desert Interpretive Centre to proceed.

The Town currently retains a reserve over a 16.6 ha area of Crown land referred to as LAR 75 (refer to TRUE Drawing No. 302-1474-01) located east of the landfill. The Province has indicated to the Town that more information is required on the expansion of the Town's wastewater disposal systems to justify "continuance" of the reserve on LAR 75. This "Discussion Paper" has the objective of presenting a long range expansion plan for the Town's wastewater treatment and disposal infrastructure to support continuance of the Town's Reserve on LAR 75.

Population and Flow Projections

The Town of Osoyoos wastewater treatment and disposal infrastructure comprises 3 components.

- aerated lagoon treatment system
- winter effluent storage basins
- areas for irrigation with reclaimed water

For a 40 year population projection, i.e. to year 2053, it is anticipated that:

- wastewater treatment capacity expansion will be located at the existing site. No additional land will be required for new or expanded treatment works.
- the existing winter storage basins have adequate capacity for a 20 year projected population. A third winter storage basin will be necessary for the 20 to 40 year horizon.
- the reclaimed water irrigation system will have to be expanded over the next 40 years as the Town grows and/or the service area of the sewer system is expanded into Electoral Area A and Osoyoos IR No. 1.

In 2013, the Town retained Kerr Wood Leidal and Associates (KWL) to undertake a capacity assessment of the Town's sanitary sewer infrastructure. A component of this review was the preparation of short and long term (20 year) projections of the service population of the Town's sanitary sewer system. Projections from the KWL report have been used to determine requirements for winter period reclaimed water storage and irrigation areas.

Winter period storage requirements are determined by the permanent (census) population serviced by the sanitary sewer system. Based on projections in the KWL report to 2033, Table 1 presents permanent (census) population projections to year 2053.

Table 1: Permanent (Census) Population Projections

Year	In Town KWL	In Town Adjusted to Census	Electoral Area A	Total
2013 ⁽¹⁾	4776	4921	288	5209
2018 ⁽¹⁾	4892	5045	623	5668
2023 ⁽¹⁾	5092	5246	948	6194
2033 ⁽¹⁾	5702	5875	1197	7072
2043 ⁽²⁾	6317	6463	1316	7779
2053 ⁽¹⁾	6956	7168	1460	8628

(1) data to 2033 from KWL. In Town population adjusted to Census

(2) data to 2053 extrapolated from KWL average growth rate for period 2013 to 2033

Table 4.2 suggests that a permanent (census) population of 8628 may be serviced by the Town sewer system by 2053 of which 17% would be within Electoral Area A.

Irrigation area requirements are determined by the total equivalent population serviced by the Town's sanitary sewer system. In this case, wastewater generated by seasonal populations will, unlike winter storage, be reflected in irrigation area requirements. Table 2 is a projection of seasonal maximum population equivalents serviced by the Town's sanitary sewer system.

Table 2: Seasonal Population Equivalents Served by Town Sanitary Sewer System

Year	In Town	Electoral Area A and OIB	Total	Project Annual Volume
2013 ⁽¹⁾	10526	1308	11824 ⁽¹⁾	803,000m ³ ⁽³⁾
2018	10718	1703	12421 ⁽¹⁾	844,600m ³
2023	11030	2099	13129 ⁽¹⁾	892,700m ³
2033	11997	2490	14487 ⁽²⁾	985,100m ³
2043			16073 ⁽²⁾	1,093,000m ³
2053			17734 ⁽²⁾	1,206,000m ³

(1) from KWL Report

(2) projected based on KWL growth rate for 2013 to 2033

(3) Actual Annual Volume for 2013

Irrigation Area Requirements

Irrigation area requirements have been derived on the basis of historical data for the golf course. From annual reports submitted to the Ministry of Environment, historical application rates are summarized following.

Reclaimed Water Irrigation Rates on Golf Course	
2009	1.02 m
2010	0.91m
2011	0.99 m
2012	0.93 m
2013	1.00 m

Irrigation area requirements are based on an application rate of 1.0m. This design application rate modestly underestimates the land requirements. In a “wetter than normal” season, it is assumed that surplus reclaimed water quantities would be disposed of by infiltration.

Table 2 provides a projection of annual wastewater volumes. From these annual volume projections, irrigation area requirements are summarized in Table 3.

Table 3: Reclaimed Water Irrigation Area Projections

Year	Projected Annual Volume	Irrigation Area Required
2013	803,000 m ³	75.3 ha
2018	844,600 m ³	79.4 ha
2023	892,700 m ³	84.3 ha
2033	985,100 m ³	93.5 ha
2043	1,093,000 m ³	104.3 ha
2054	1,206,000 m ³	115.6 ha

Drawing 302-1474-01 illustrates existing areas irrigated with reclaimed water and opportunities for expansion in the future. The areas illustrated on drawing 302-1474-01 are briefly summarized as follows.

Existing Reclaimed Water Irrigation Area

(1) Golf Course	65.5 ha
(2) Ball Diamonds	2.4 ha
(3) Desert Park Infield	5.0 ha
(4) Secondary School (start 2015)	<u>3.8 ha</u>
Total Existing	76.8 ha

Expansion Area Opportunities

(1) Parcel South of Secondary School (Rem 1003 Plan 31183). Previously used as a site for sludge containment and drying tubes. Community Plan designation is institutional.	3.5 ha
(2) South End of Desert Park. Supply infrastructure is in place and irrigation system design has been prepared. Project on hold awaiting better understanding of Desert Park expansion plans. Previously cleared and graded. OCP designation is parks and recreation.	1.5 ha
(3) East Side of Desert Park Site. Generally natural state habitat. High environmental sensitivity designation in OCP.	4.8 ha
(4) Crown Land west of Desert Park Site. Identified as not sensitive in OCP with a parks and recreation use designation. Not likely to be well received by some residents in Dividend Ridge.	3 ha
(5) Airport Site. Available land area depends on future land use. Minimum land area assumed to be 10% of parcel area as a DP landscaping requirement.	1.5 ha
(6) LAR 75 East of Landfill. This parcel is currently utilized for cattle grazing in the spring. Town has Reserve on parcel for “spray irrigation”.	16.6

Expansion Opportunities Total	30.9 ha

Total Existing and Opportunities	107.7 ha

The above total of 107.7 ha is only 8 ha less than the 40 year projected requirement of 115.6 ha. LAR 75 at the landfill represents more than 50% of the expansion opportunity and is therefore of critical importance to the long term viability of the Town’s reclaimed water irrigation system.

Other potential opportunities for future use reclaimed water are:

- industrial uses, i.e. concrete manufacture, cooling, etc.
- expanded irrigation on the Desert 18 of the golf course.
- irrigation of vineyards north of the airport site.
- irrigation of vineyards or other grassland type cover on completed areas of the landfill site.

Winter Effluent Storage

Future requirements for winter storage are based on actual utilization of the existing winter storage reservoirs and permanent population projections presented in Table 1. Historical winter storage utilization is summarized following.

Storage Period	Storage Volume Utilized
2009 – 2010	253,000 m ³
2010 – 2011	273,000 m ³
2011 – 2012	262,000 m ³
2012 – 2013	259,000 m ³

Utilization in the 2010 to 2011 winter period of 273,000 m³ equates to 56m³/capita. On this basis, winter storage projections are based on a requirement of 60m³/capita.

In addition to storage for the winter period, typically mid-October to the end of March, the Town's Operating Certificate as issued by the Ministry of Environment, specifies that wastewater has to be retained for a minimum of 60 days. It is assumed that the 60 day storage period would be calculated based on the average daily inflow to the system.

Storage requirement projections for 2033 and 2053 permanent population horizons are:

	2033	2053
Permanent (Census) Population	7072	8628
60 Days Storage	162,000 m ³	198,000 m ³
Winter Storage Required @ 60m ³ /capita	424,000 m ³	517,000 m ³
Total Required	586,000 m ³	715,000 m ³
(less) Existing	570,000 m ³	570,000 m ³
Additional Storage Required	Existing Adequate	145,000 m ³

A storage pond having a capacity of 150,000m³ will have a footprint of 3 to 3.5 ha depending on the design water depth. The landfill area is considered as the only feasible area for siting of a third winter storage pond. There are no sites 3 to 3.5 ha in size on the West Bench area within Town boundaries which are considered potentially suitable for a winter effluent storage reservoir.

As shown on drawing 302-1474-01, two site options are considered as potentially suitable for a winter effluent storage reservoir in the area of the landfill. The site options are described as follows.

- gravel pit accessed from Highway 3 located at the south boundary of the Desert Interpretive Centre site. The operating status of the gravel pit is unknown however pits represent good sites for storage reservoir. Existing excavations and side slopes can be incorporated into the storage reservoir design. In the review process by the province of LAR parcels in the landfill area in 2007, inquiries were made as to the Town's potential interest in the gravel pit.
- western portion of LAR 75 adjoining the landfill site. Previously prepared long range system expansion plans illustrated a third winter storage reservoir at this location. A major disadvantage of this site is that the storage reservoir footprint would occupy about 3.5 ha of irrigable land. An advantage of the site is that excavation from the winter storage reservoir could be used for cover at the landfill.

Prepared by:

A handwritten signature in black ink, appearing to read 'T. Underwood', with a stylized, flowing script.

T.R. Underwood, P. Eng.



September 12, 2014

302-1474

Town of Osoyoos

Long Range Sanitary Sewer System Expansion Plan

DISCUSSION PAPER

Background

Since 1978, the Town of Osoyoos has practiced beneficial re-use (irrigation) for the disposal of treated wastewater. The Town was the first system in British Columbia to use treated wastewater for irrigation on a golf course. In the period 1978 to date, the reclaimed water irrigation system has been expanded onto an additional 27 holes on the Golf Course, a playing field complex in the infield of the Desert Park horseracing track, and ball diamonds. In 2015, reclaimed water will be used to irrigate the playing fields at the Osoyoos Secondary School.

In conjunction with a Liquid Waste Management Plan undertaken in the mid-1980's, it was recognized that the long term viability of the Town's land based reclaimed water re-use strategy for wastewater effluent disposal was dependent on a land base for irrigation purposes. At this time, the Town was encouraged by Provincial Agencies to register reserves on vacant, natural state Crown land in area of the landfill for future "spray irrigation" with treated wastewater effluent. The Town proceeded with registration of reserves over Crown land north and south of the landfill access road for spray irrigation purposes.

Through the 1980's, Provincial Agencies became increasingly aware of the importance of vacant, undeveloped Crown land on the Osoyoos West Bench as natural state habitat. In 1998, the Town of Osoyoos relinquished its reserve on Crown lands located on the south side of the landfill access road to enable the Desert Interpretive Centre to proceed.

The Town currently retains a reserve over a 16.6 ha area of Crown land referred to as LAR 75 (refer to TRUE Drawing No. 302-1474-01) located east of the landfill. The Province has indicated to the Town that more information is required on the expansion of the Town's wastewater disposal systems to justify "continuance" of the reserve on LAR 75. This "Discussion Paper" has the objective of presenting a long range expansion plan for the Town's wastewater treatment and disposal infrastructure to support continuance of the Town's Reserve on LAR 75.

Population and Flow Projections

The Town of Osoyoos wastewater treatment and disposal infrastructure comprises 3 components.

- aerated lagoon treatment system
- winter effluent storage basins
- areas for irrigation with reclaimed water

For a 40 year population projection, i.e. to year 2053, it is anticipated that:

- wastewater treatment capacity expansion will be located at the existing site. No additional land will be required for new or expanded treatment works.
- the existing winter storage basins have adequate capacity for a 20 year projected population. A third winter storage basin will be necessary for the 20 to 40 year horizon.
- the reclaimed water irrigation system will have to be expanded over the next 40 years as the Town grows and/or the service area of the sewer system is expanded into Electoral Area A and Osoyoos IR No. 1.

In 2013, the Town retained Kerr Wood Leidal and Associates (KWL) to undertake a capacity assessment of the Town's sanitary sewer infrastructure. A component of this review was the preparation of short and long term (20 year) projections of the service population of the Town's sanitary sewer system. Projections from the KWL report have been used to determine requirements for winter period reclaimed water storage and irrigation areas.

Winter period storage requirements are determined by the permanent (census) population serviced by the sanitary sewer system. Based on projections in the KWL report to 2033, Table 1 presents permanent (census) population projections to year 2053.

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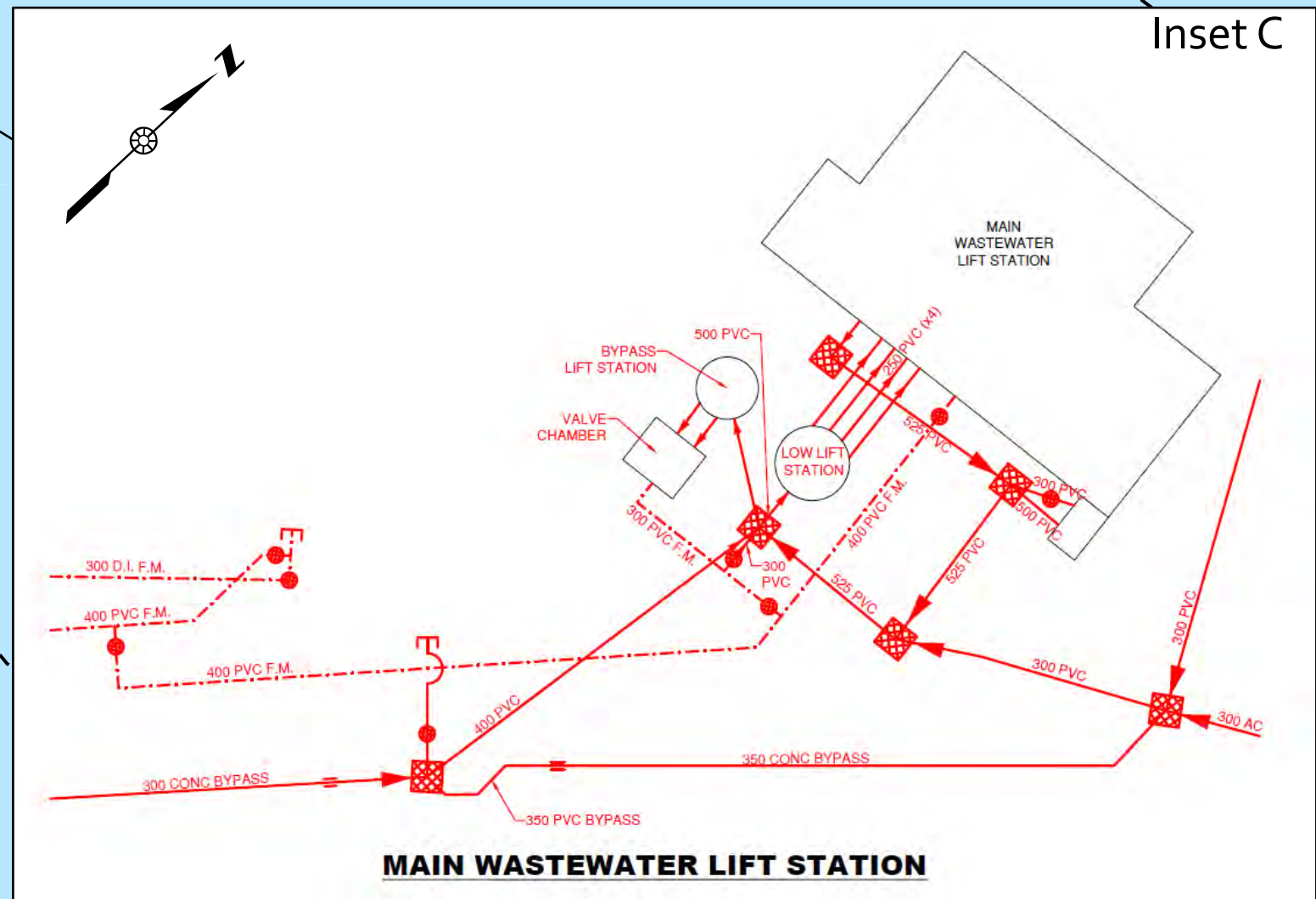
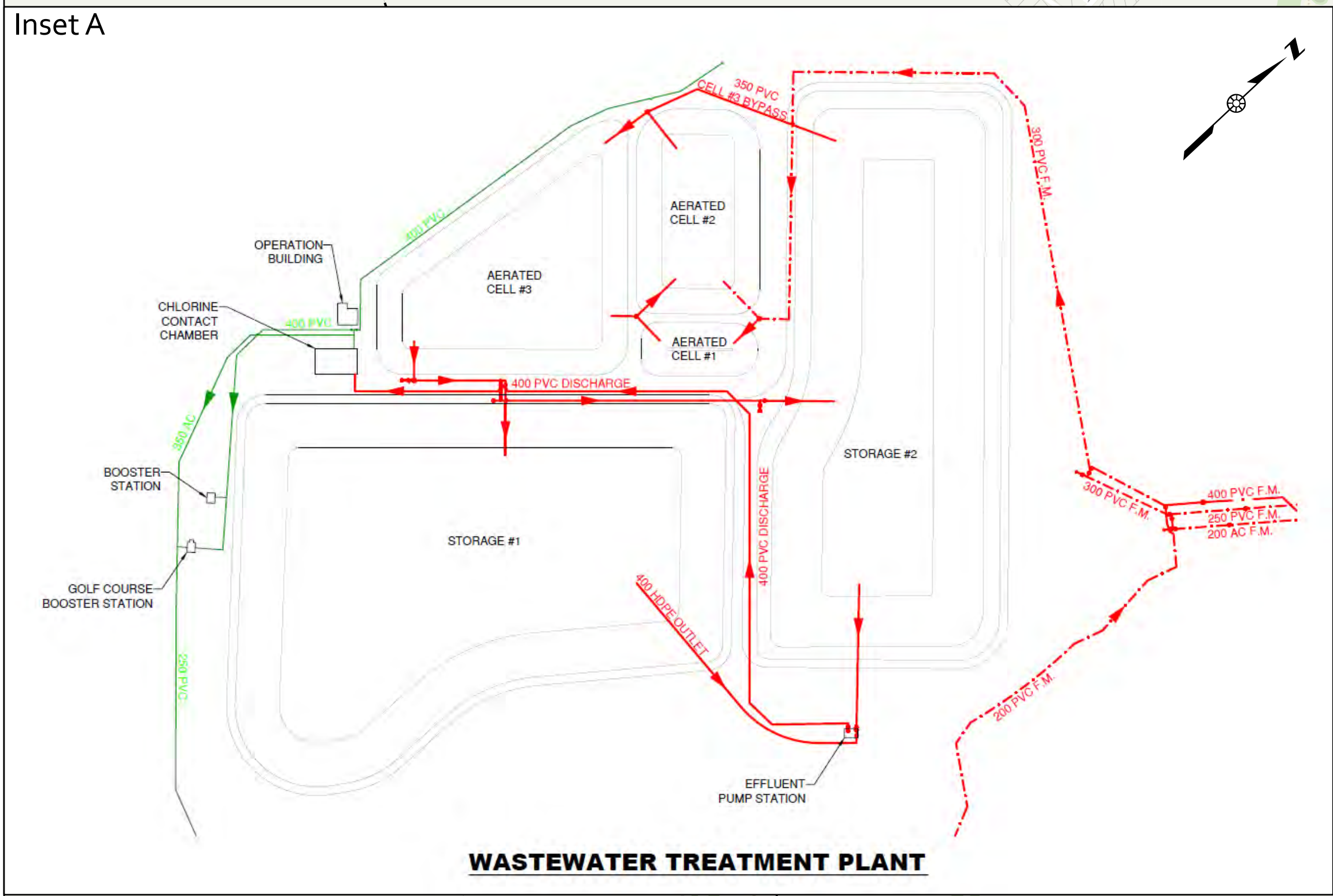
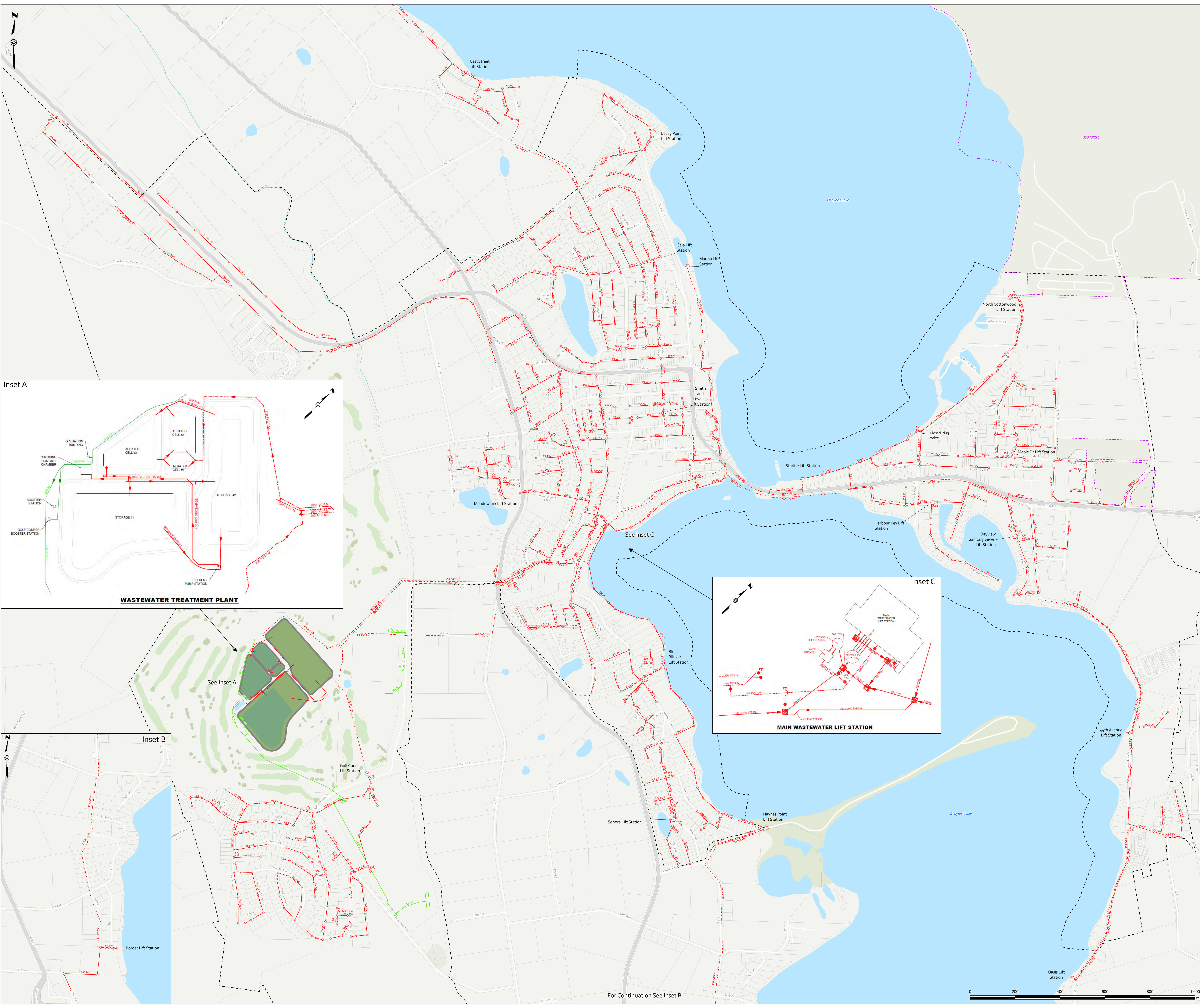
Prepared by:

A handwritten signature in black ink, appearing to read 'T. Underwood', with a stylized, cursive script.

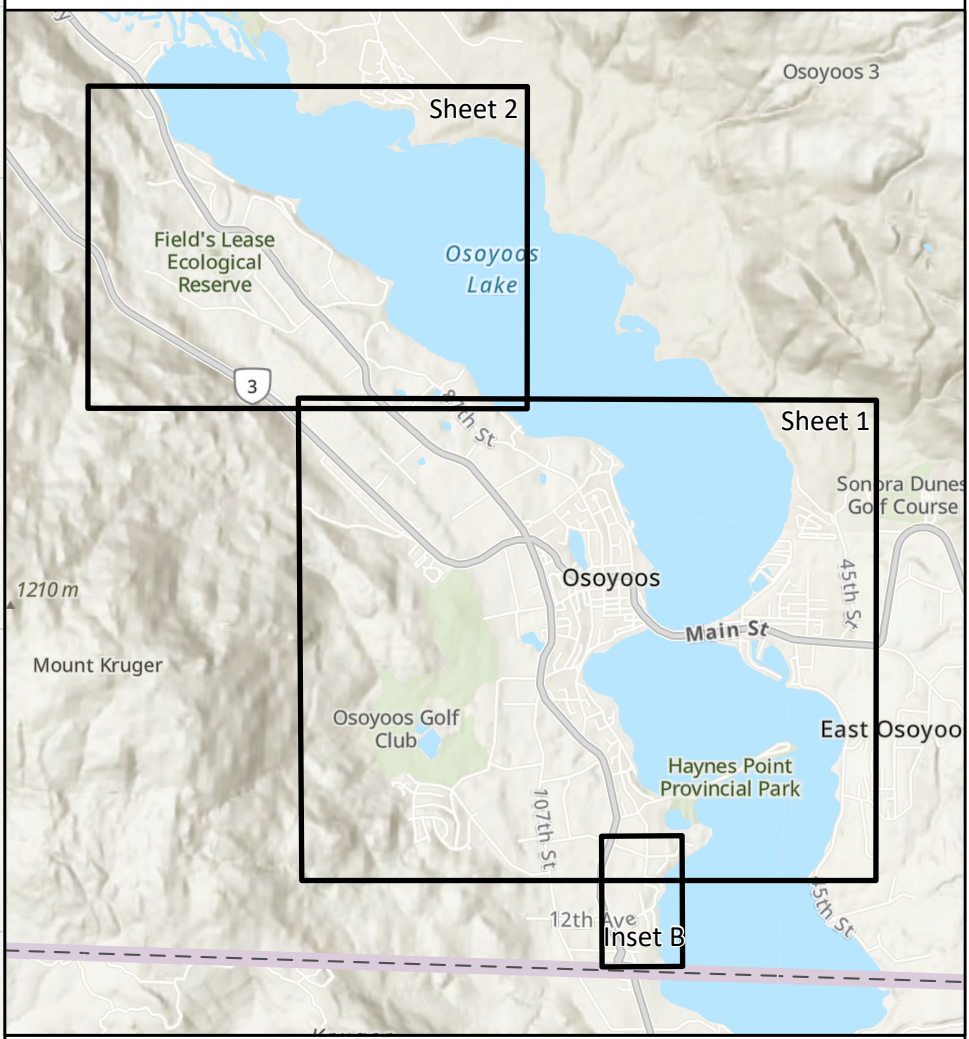
T.R. Underwood, P. Eng.

APPENDIX B

Composite Drawing



- Legend**
- Sanitary Fitting
 - Sanitary Lift Station
 - Sanitary Valve
 - Sanitary Manhole
 - Sanitary Forcemain
 - Sanitary Gravity
 - Reclaimed Water Valve
 - Reclaimed Pressurized Water
 - Wastewater Treatment Plant
 - First Nation Reserve Boundary
 - Municipal Boundary
 - Sanitary Gravity



**SANITARY SYSTEM
COMPOSITE**

**MUNICIPAL SYSTEM
OVERALL PLAN**

SCALE	1:5,000
DESIGN BY	
DRAWN BY	ET
DATE	5/5/2023
PROJECT REF No.	302-2051

DRAWING No.
305-2051-01

SHEET
1 of 2
REVISION
01

APPENDIX C

Replacement Costs

Pipe Replacement Costs

Size (mm)	Replacement cost per m
75	\$ 437.50
100	\$ 500.00
150	\$ 687.50
200	\$ 750.00
250	\$ 937.50
300	\$ 1,125.00
350	\$ 1,312.50
375	\$ 1,500.00
400	\$ 1,687.50
450	\$ 1,875.00
500	\$ 2,062.50
525	\$ 2,250.00
1000	\$ -

* The costs above include services, manholes, and road restoration.

* Forcemain replacement has been assumed the same price per meter as gravity pipe except for the Lake Crossing

Lift Station Replacement Costs

Lift Station	Structure Replacement Cost	Electrical Replacement Cost	Pump Replacement Cost
Main Lift Station	\$ 1,500,000.00	\$ 500,000.00	\$ 250,000.00
Starlite	\$ 1,000,000.00	\$ 250,000.00	\$ 62,500.00
Smith and Loveless	\$ 900,000.00	\$ 250,000.00	\$ 94,000.00
44th Ave	\$ 900,000.00	\$ 250,000.00	\$ 150,000.00
Maple Drive	\$ 700,000.00	\$ 200,000.00	\$ 150,000.00
Border	\$ 700,000.00	\$ 200,000.00	\$ 62,500.00
Cottonwood	\$ 700,000.00	\$ 200,000.00	\$ 150,000.00
Bayview	\$ 700,000.00	\$ 200,000.00	\$ 50,000.00
Oasis	\$ 700,000.00	\$ 200,000.00	\$ 150,000.00
Inkaneep Point	\$ 700,000.00	\$ 200,000.00	\$ 94,000.00
Roberts Point	\$ 700,000.00	\$ 200,000.00	\$ 62,500.00
81 st	\$ 700,000.00	\$ 200,000.00	\$ 94,000.00
Golf Course	\$ 700,000.00	\$ 200,000.00	\$ 50,000.00
95st	\$ 700,000.00	\$ 200,000.00	\$ 62,500.00
87st	\$ 700,000.00	\$ 200,000.00	\$ 62,500.00
Haynes Point	\$ 700,000.00	\$ 200,000.00	\$ 50,000.00
Blue Blinker	\$ 700,000.00	\$ 200,000.00	\$ 150,000.00
Lacey Point	\$ 700,000.00	\$ 200,000.00	\$ 50,000.00
Meadowlark	\$ 700,000.00	\$ 200,000.00	\$ 50,000.00
Harbour Key	\$ 700,000.00	\$ 200,000.00	\$ 50,000.00
Gala	\$ 700,000.00	\$ 200,000.00	\$ 50,000.00

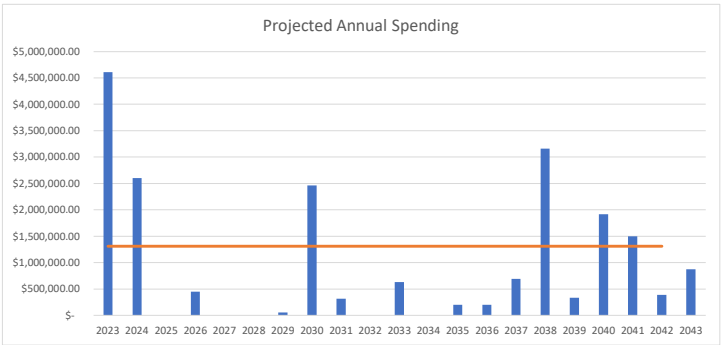
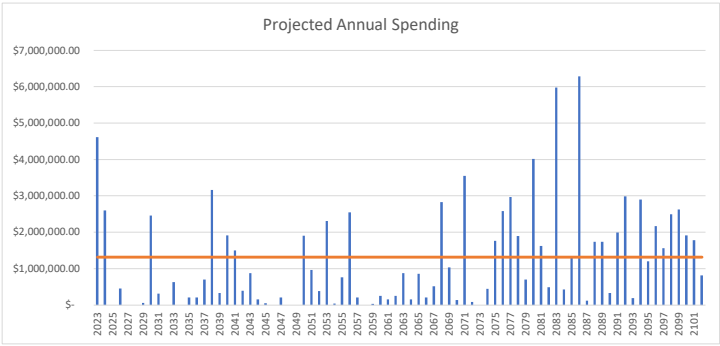
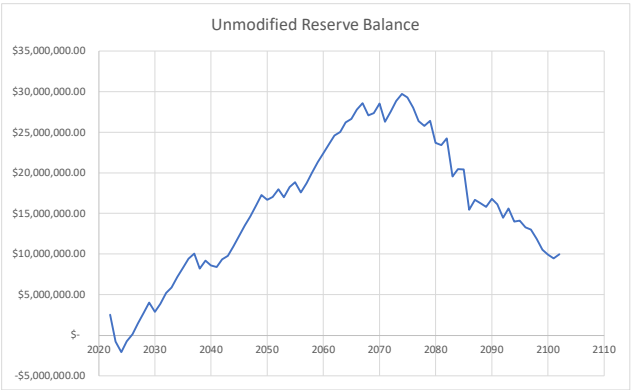
APPENDIX D

Unmodified Reserve Balance Model

Author ED
Date 5/12/2023
Assumptions No interest on positive value, none on negative either

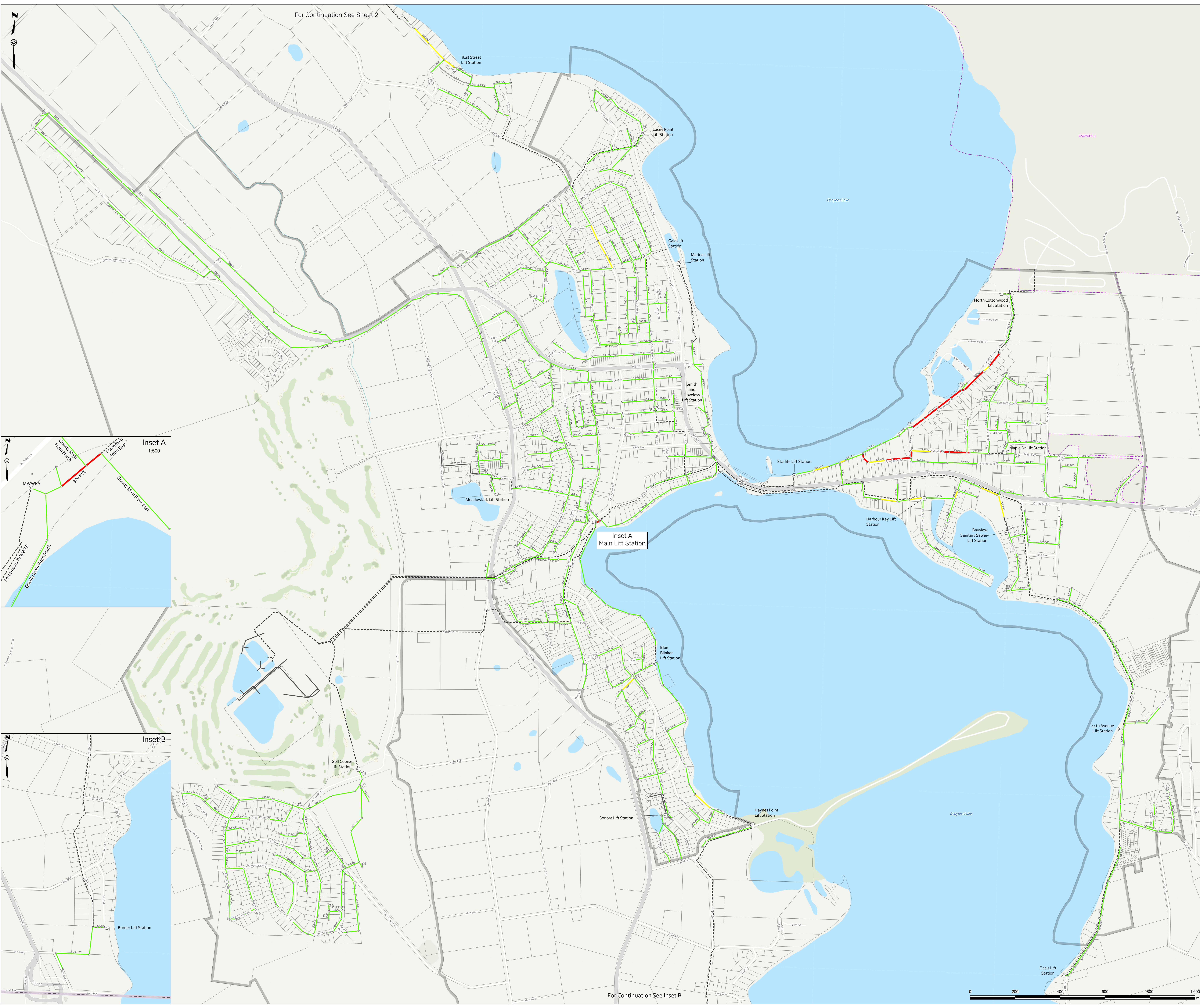
Reserve Balance: 8-May-23	\$	2,527,498.20	Gravity Pipe AAI	\$	448,392.31	Total AAI	\$	1,315,415.69	2023	2023
			Forcemain AAI	\$	182,965.04		\$	1,315,415.69	2102	2043
			LS Structure AAI	\$	216,000.00					
			LS Electrical AAI	\$	243,333.33					
			LS Pump AAI	\$	74,725.00					
			WWTP AAI	\$	150,000.00					

Year	Gravity Pipe Replacement	Forcemain Replacement	LS Structure Replacement	LS Electrical Replacement	LS Pump Replacement	Total Spending	Total AAI	Change in Reserve	Actual Reserve Balance
2022									\$ 2,527,498.20
2023	\$ 3,780,138.75	\$ -	\$ -	\$ -	\$ 831,500.00	\$ 4,611,638.75	\$ 1,315,415.69	-\$3,296,223.06	768,724.86
2024	\$ -	\$ -	\$ -	\$ 2,550,000.00	\$ 50,000.00	\$ 2,600,000.00	\$ 1,315,415.69	-\$ 1,284,584.31	2,053,309.18
2025	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ 1,315,415.69	\$ 1,315,415.69	737,893.49
2026	\$ -	\$ -	\$ -	\$ 450,000.00	\$ -	\$ 450,000.00	\$ 1,315,415.69	\$ 865,415.69	127,522.19
2027	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ 1,315,415.69	\$ 1,315,415.69	1,442,937.88
2028	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ 1,315,415.69	\$ 1,315,415.69	2,758,353.56
2029	\$ 56,431.38	\$ -	\$ -	\$ -	\$ -	\$ 56,431.38	\$ 1,315,415.69	\$ 1,258,984.31	4,017,337.88
2030	\$ 2,459,828.25	\$ -	\$ -	\$ -	\$ -	\$ 2,459,828.25	\$ 1,315,415.69	-\$ 1,144,412.56	2,872,925.31
2031	\$ -	\$ -	\$ -	\$ -	\$ 312,500.00	\$ 312,500.00	\$ 1,315,415.69	\$ 1,002,915.69	3,875,841.00
2032	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ 1,315,415.69	\$ 1,315,415.69	5,191,256.68
2033	\$ 629,893.50	\$ -	\$ -	\$ -	\$ -	\$ 629,893.50	\$ 1,315,415.69	\$ 685,522.19	5,876,778.87
2034	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ 1,315,415.69	\$ 1,315,415.69	7,192,194.55
2035	\$ -	\$ -	\$ -	\$ 200,000.00	\$ -	\$ 200,000.00	\$ 1,315,415.69	\$ 1,115,415.69	8,307,610.24
2036	\$ -	\$ -	\$ -	\$ 200,000.00	\$ -	\$ 200,000.00	\$ 1,315,415.69	\$ 1,115,415.69	9,423,025.93
2037	\$ 380,514.00	\$ -	\$ -	\$ 250,000.00	\$ 62,500.00	\$ 693,014.00	\$ 1,315,415.69	\$ 622,401.69	10,045,427.61
2038	\$ 857,612.25	\$ -	\$ -	\$ 2,300,000.00	\$ -	\$ 3,157,612.25	\$ 1,315,415.69	-\$ 1,842,196.56	8,203,231.05
2039	\$ 330,734.25	\$ -	\$ -	\$ -	\$ -	\$ 330,734.25	\$ 1,315,415.69	\$ 984,681.44	9,187,912.48
2040	\$ 1,753,272.38	\$ 110,543.22	\$ -	\$ -	\$ 50,000.00	\$ 1,913,815.60	\$ 1,315,415.69	-\$ 598,399.91	8,589,512.57
2041	\$ 152,975.25	\$ -	\$ -	\$ 1,250,000.00	\$ 94,000.00	\$ 1,496,975.25	\$ 1,315,415.69	-\$ 181,559.56	8,407,953.01
2042	\$ 296,647.50	\$ -	\$ -	\$ -	\$ 94,000.00	\$ 390,647.50	\$ 1,315,415.69	\$ 924,768.19	9,332,721.19
2043	\$ -	\$ -	\$ -	\$ -	\$ 875,000.00	\$ 875,000.00	\$ 1,315,415.69	\$ 440,415.69	9,773,136.88
2044	\$ -	\$ -	\$ -	\$ -	\$ 150,000.00	\$ 150,000.00	\$ 1,315,415.69	\$ 1,165,415.69	10,938,552.57
2045	\$ -	\$ -	\$ -	\$ -	\$ 50,000.00	\$ 50,000.00	\$ 1,315,415.69	\$ 1,265,415.69	12,203,968.25
2046	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ 1,315,415.69	\$ 1,315,415.69	13,519,383.94
2047	\$ -	\$ -	\$ -	\$ 200,000.00	\$ -	\$ 200,000.00	\$ 1,315,415.69	\$ 1,115,415.69	14,634,799.62
2048	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ 1,315,415.69	\$ 1,315,415.69	15,950,215.31
2049	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ 1,315,415.69	\$ 1,315,415.69	17,265,631.00
2050	\$ -	\$ -	\$ 1,400,000.00	\$ 500,000.00	\$ -	\$ 1,900,000.00	\$ 1,315,415.69	-\$ 584,584.31	16,681,046.68
2051	\$ 291,834.81	\$ -	\$ -	\$ 200,000.00	\$ 469,500.00	\$ 961,334.81	\$ 1,315,415.69	\$ 354,080.87	17,035,127.55
2052	\$ 176,209.69	\$ -	\$ -	\$ 200,000.00	\$ -	\$ 376,209.69	\$ 1,315,415.69	\$ 939,206.00	17,974,333.55
2053	\$ 5,218.81	\$ -	\$ -	\$ 2,300,000.00	\$ -	\$ 2,305,218.81	\$ 1,315,415.69	-\$ 989,803.13	16,984,530.43
2054	\$ 38,669.13	\$ -	\$ -	\$ -	\$ -	\$ 38,669.13	\$ 1,315,415.69	\$ 1,276,746.56	18,261,276.99
2055	\$ 56,329.63	\$ -	\$ 700,000.00	\$ -	\$ -	\$ 756,329.63	\$ 1,315,415.69	\$ 559,086.06	18,820,363.05
2056	\$ 1,294,293.50	\$ -	\$ -	\$ 1,250,000.00	\$ -	\$ 2,544,293.50	\$ 1,315,415.69	-\$ 1,228,877.81	17,591,485.23
2057	\$ 153,263.25	\$ -	\$ -	\$ -	\$ 50,000.00	\$ 203,263.25	\$ 1,315,415.69	\$ 1,112,152.44	18,703,637.67
2058	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ 1,315,415.69	\$ 1,315,415.69	20,019,053.36
2059	\$ 31,821.00	\$ -	\$ -	\$ -	\$ -	\$ 31,821.00	\$ 1,315,415.69	\$ 1,283,594.69	21,302,648.04
2060	\$ -	\$ -	\$ -	\$ -	\$ 250,000.00	\$ 250,000.00	\$ 1,315,415.69	\$ 1,065,415.69	22,368,063.73
2061	\$ 103,212.00	\$ -	\$ -	\$ -	\$ 50,000.00	\$ 153,212.00	\$ 1,315,415.69	\$ 1,162,203.69	23,530,267.41
2062	\$ -	\$ -	\$ -	\$ 200,000.00	\$ 50,000.00	\$ 250,000.00	\$ 1,315,415.69	\$ 1,065,415.69	24,595,683.10
2063	\$ -	\$ -	\$ -	\$ -	\$ 875,000.00	\$ 875,000.00	\$ 1,315,415.69	\$ 440,415.69	25,036,098.78
2064	\$ -	\$ -	\$ -	\$ -	\$ 150,000.00	\$ 150,000.00	\$ 1,315,415.69	\$ 1,165,415.69	26,201,514.47
2065	\$ -	\$ 309,597.24	\$ -	\$ 500,000.00	\$ 50,000.00	\$ 859,597.24	\$ 1,315,415.69	\$ 455,818.45	26,657,332.92
2066	\$ -	\$ -	\$ -	\$ 200,000.00	\$ -	\$ 200,000.00	\$ 1,315,415.69	\$ 1,115,415.69	27,772,748.60
2067	\$ -	\$ 309,050.05	\$ -	\$ 200,000.00	\$ -	\$ 509,050.05	\$ 1,315,415.69	\$ 806,365.64	28,579,114.24
2068	\$ 1,364.69	\$ 525,010.85	\$ -	\$ 2,300,000.00	\$ -	\$ 2,826,375.54	\$ 1,315,415.69	-\$ 1,510,959.85	27,068,154.39
2069	\$ 334,059.75	\$ -	\$ 700,000.00	\$ -	\$ -	\$ 1,034,059.75	\$ 1,315,415.69	\$ 281,355.94	27,349,510.33
2070	\$ 135,273.00	\$ -	\$ -	\$ -	\$ -	\$ 135,273.00	\$ 1,315,415.69	\$ 1,180,142.69	28,529,653.01
2071	\$ 1,825,079.44	\$ -	\$ -	\$ 1,250,000.00	\$ 469,500.00	\$ 3,544,579.44	\$ 1,315,415.69	-\$ 2,229,163.75	26,300,489.26
2072	\$ 78,678.75	\$ -	\$ -	\$ -	\$ -	\$ 78,678.75	\$ 1,315,415.69	\$ 1,236,736.94	27,537,226.20
2073	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ 1,315,415.69	\$ 1,315,415.69	28,852,641.88
2074	\$ 444,862.44	\$ -	\$ -	\$ -	\$ -	\$ 444,862.44	\$ 1,315,415.69	\$ 870,553.25	29,723,195.13
2075	\$ 1,058,510.81	\$ -	\$ 700,000.00	\$ -	\$ -	\$ 1,758,510.81	\$ 1,315,415.69	-\$ 443,095.13	29,280,100.00
2076	\$ 1,880,050.13	\$ -	\$ 700,000.00	\$ -	\$ -	\$ 2,580,050.13	\$ 1,315,415.69	-\$ 1,264,634.44	28,015,465.56
2077	\$ 539,316.75	\$ 680,553.09	\$ 1,500,000.00	\$ 200,000.00	\$ 50,000.00	\$ 2,969,869.84	\$ 1,315,415.69	-\$ 1,654,454.15	26,361,011.41
2078	\$ 214,329.75	\$ 281,197.72	\$ 1,400,000.00	\$ -	\$ -	\$ 1,895,527.47	\$ 1,315,415.69	-\$ 580,111.78	25,780,899.63
2079	\$ -	\$ -	\$ 700,000.00	\$ -	\$ -	\$ 700,000.00	\$ 1,315,415.69	\$ 615,415.69	26,396,315.32
2080	\$ -	\$ 2,561,321.34	\$ 700,000.00	\$ 500,000.00	\$ 250,000.00	\$ 4,011,321.34	\$ 1,315,415.69	-\$ 2,695,905.66	23,700,409.66
2081	\$ 294,438.75	\$ 1,073,505.59	\$ -	\$ 200,000.00	\$ 50,000.00	\$ 1,617,944.34	\$ 1,315,415.69	-\$ 302,528.66	23,397,881.00
2082	\$ -	\$ 233,089.67	\$ -	\$ 200,000.00	\$ 50,000.00	\$ 483,089.67	\$ 1,315,415.69	\$ 832,326.01	24,230,207.02
2083	\$ -	\$ 2,806,173.42	\$ -	\$ 2,300,000.00	\$ 875,000.00	\$ 5,981,173.42	\$ 1,315,415.69	-\$ 4,665,757.73	19,564,449.29
2084	\$ 274,993.56	\$ -	\$ -	\$ -	\$ 150,000.00	\$ 424,993.56	\$ 1,315,415.69	\$ 890,422.12	20,454,871.41
2085	\$ 1,290,036.94	\$ -	\$ -	\$ -	\$ 50,000.00	\$ 1,340,036.94	\$ 1,315,415.69	-\$ 24,621.25	20,430,250.16
2086	\$ 332,075.25	\$ 5,024.82	\$ 4,700,000.00	\$ 1,250,000.00	\$ -	\$ 6,287,100.07	\$ 1,315,415.69	-\$ 4,971,684.39	15,458,565.77
2087	\$ 111,714.00	\$ -	\$ -	\$ -	\$ -	\$ 111,714.00	\$ 1,315,415.69	\$ 1,203,701.69	16,662,267.46
2088	\$ 1,738,999.31	\$ -	\$ -	\$ -	\$ -	\$ 1,738,999.31	\$ 1,315,415.69	-\$ 423,583.63	16,238,683.83
2089	\$ 1,691,954.81	\$ 45,792.81	\$ -	\$ -	\$ -	\$ 1,737,747.62	\$ 1,315,415.69	-\$ 422,331.94	15,816,351.89
2090	\$ 323,118.56	\$ -	\$ -	\$ -	\$ -	\$ 323,118.56	\$ 1,315,415.69	\$ 992,297.12	16,808,649.02
2091	\$ 948,979.81	\$ 575,964.66	\$ -	\$ -	\$ 469,500.00	\$ 1,994,444.47	\$ 1,315,415.69	-\$ 679,028.78	16,129,620.23
2092	\$ -	\$ 2,081,030.85	\$ 700,000.00	\$ 200,000.00	\$ -	\$ 2,981,030.85	\$ 1,315,415.69	-\$ 1,665,615.16	14,464,005.07
2093	\$ 190,717.31	\$ -	\$ -	\$ -	\$ -	\$ 190,717.31	\$ 1,315,415.69	\$ 1,124,698.37	15,588,703.45
2094	\$ -	\$ 2,896,746.09	\$ -	\$ -	\$ -	\$ 2,896,746.09	\$ 1,315,415.69	-\$ 1,581,330.41	14,007,373.04
2095	\$ -	\$ -	\$ 700,000.00	\$ 500,000.00	\$ -	\$ 1,200,000.00	\$ 1,315,415.69	\$ 115,415.69	14,122,788.73
2096	\$ 1,269,910.50	\$ -	\$ 700,000.00	\$ 200,000.00	\$ -	\$ 2,169,910.50	\$ 1,315,415.69	-\$ 854,494.81	13,268,293.91
2097	\$ 309,429.00	\$ 101,274.72	\$ 900,000.00	\$ 200,000.00	\$ 50,000.00	\$ 1,560,703.72	\$ 1,315,415.69	-\$ 245,288.03	13,023,005.88
2098	\$ 193,240.50	\$ -	\$ -	\$ 2,300,000.00	\$ -	\$ 2,493,240.50	\$ 1,315,415.69	-\$ 1,177,824.81	11,845,181.07
2099	\$ 2,621,800.50	\$ -	\$ -	\$ -	\$ -	\$ 2,621,800.50	\$ 1,315,415.69	-\$ 1,306,384.81	10,538,796.25
2100	\$ 1,662,898.69	\$ -	\$ -	\$ -	\$ 250,000.00	\$ 1,912,898.69	\$ 1,315,415.69	-\$ 597,483.00	9,941,313.25
2101	\$ 436,511.44	\$ 41,327.17	\$ -	\$ 1,250,000.00	\$ 50,000.00	\$ 1,777,838.61	\$ 1,315,415.69	-\$ 462,422.92	9,478,890.33
2102	\$ 759,422.25	\$ -	\$ -	\$ -	\$ 50,000.00	\$ 809,422.25	\$ 1,315,415.69	\$ 505,993.44	9,984,883.76

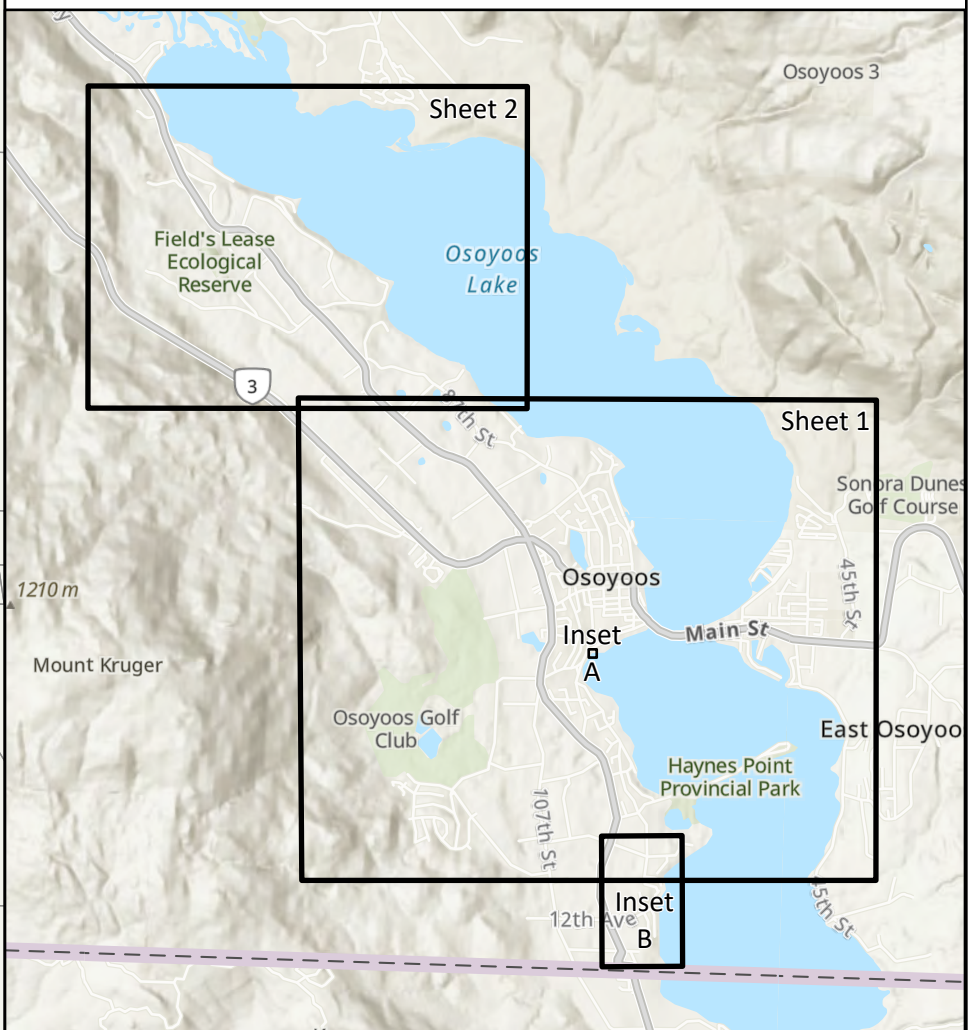


APPENDIX E

System Capacity – Existing Conditions



- Legend**
- Sanitary Gravity Main - Capacity
- Not Deficient (<50%)
 - Minor Deficiency (50-90%)
 - Major Deficiency (>90%)
 - Not Included in Model
- Sanitary System
- Sanitary Manhole
 - Sanitary Lift Station
 - Sanitary Forcemain
- First Nation Reserve Boundary
- Municipal Boundary



**SANITARY SYSTEM
COMPOSITE**

**MUNICIPAL SYSTEM
EXISTING CAPACITY**

SCALE	1:5,000
DESIGN BY	ED
DRAWN BY	ET
DATE	5/4/2023
PROJECT REF No.	302-2051

DRAWING No.
305-2051-02

SHEET
1 of 2
REVISION
01



Legend

Sanitary Gravity Main - Capacity

Not Deficient (<50%)

Minor Deficiency (50-90%)

Major Deficiency (>90%)

Not Included in Model

Sanitary System

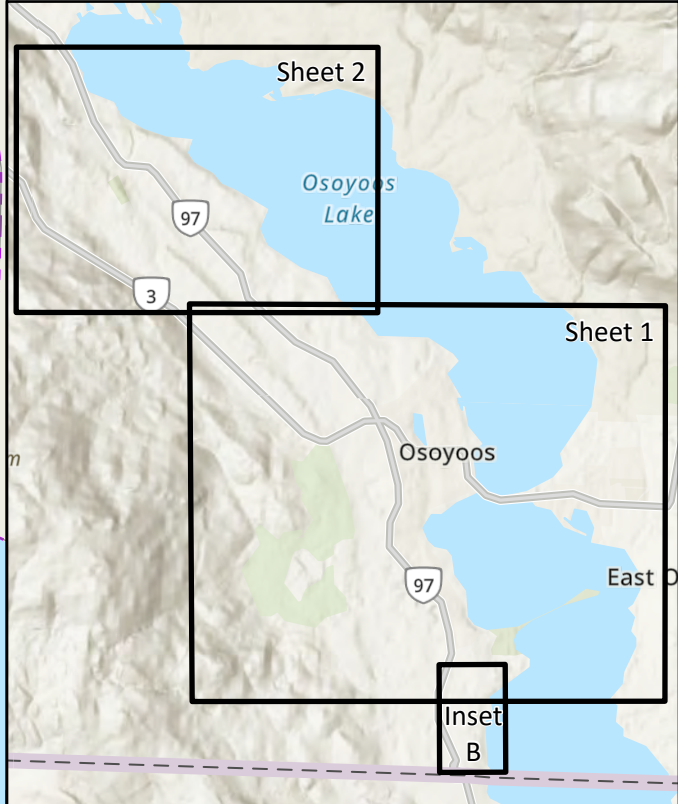
Sanitary Manhole

Sanitary Lift Station

Sanitary Forcemain

First Nation Reserve Boundary

Municipal Boundary



**SANITARY SYSTEM
COMPOSITE**

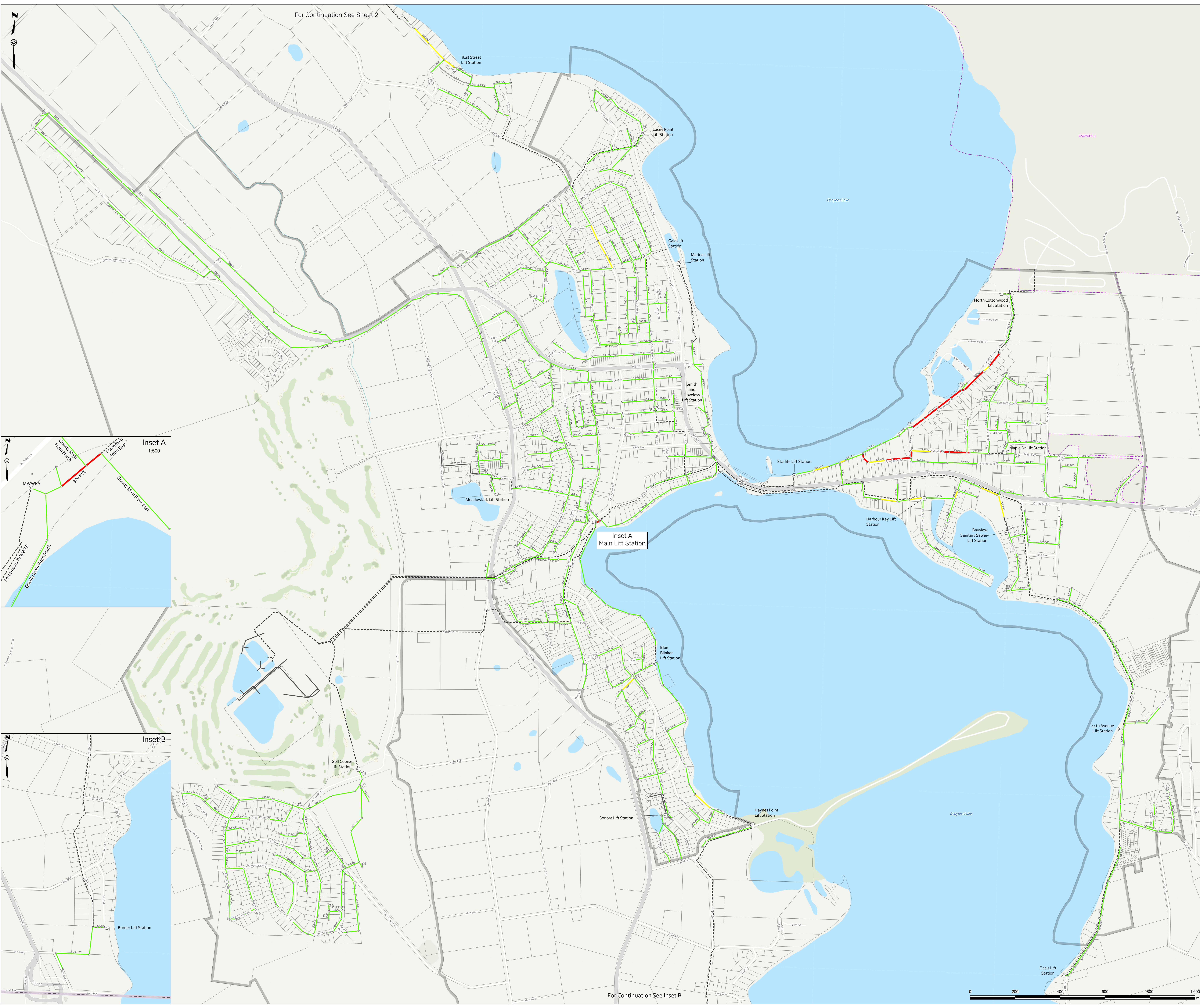
**NORTHWEST SECTOR
EXISTING CAPACITY**

SCALE	1:5,000
DESIGN BY	
DRAWN BY	ET
DATE	5/1/2023
PROJECT REF No.	302-2051

DRAWING No. 305-2051-02	SHEET 2 OF 2
	REVISION --

APPENDIX F

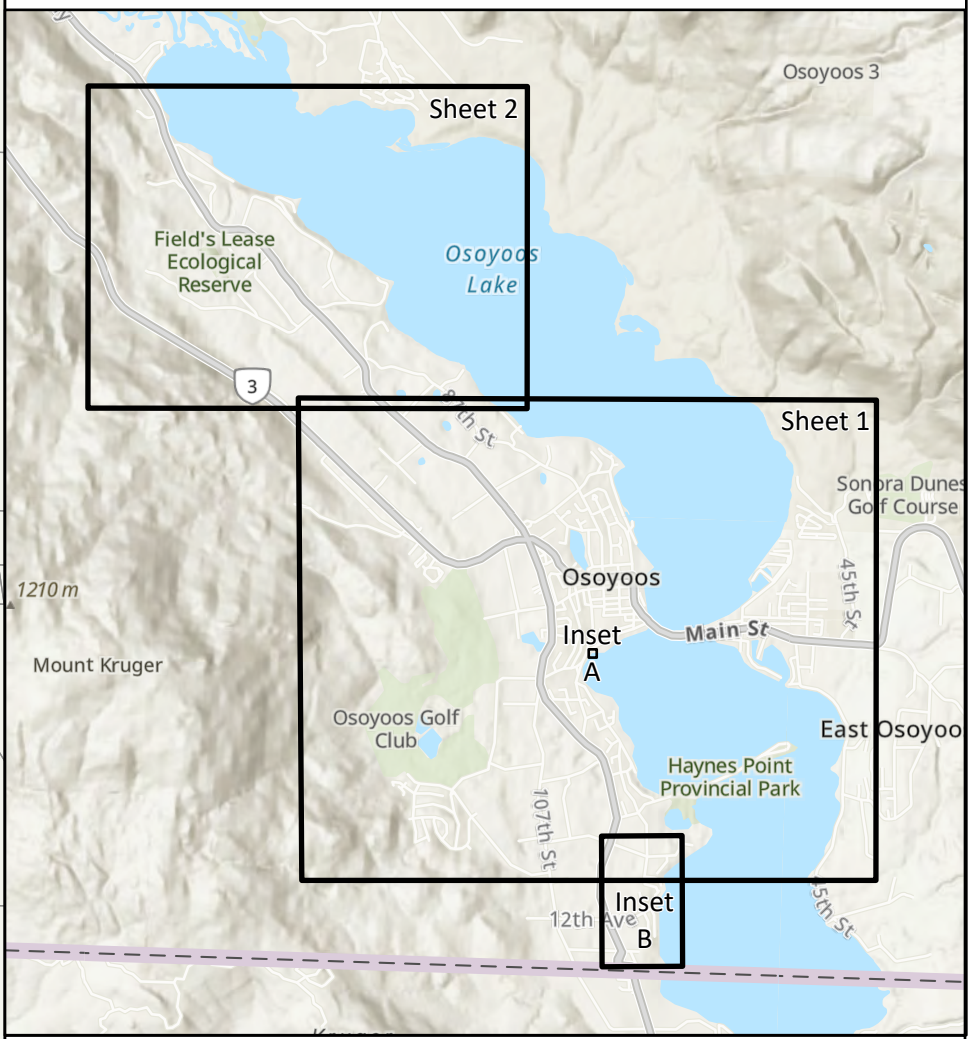
System Capacity - Future



For Continuation See Sheet 2

For Continuation See Inset B

- Legend**
- Sanitary Gravity Main - Capacity
- Not Deficient (<50%)
 - Minor Deficiency (50-90%)
 - Major Deficiency (>90%)
 - Not Included in Model
- Sanitary System
- Sanitary Manhole
 - Sanitary Lift Station
 - Sanitary Forcemain
- First Nation Reserve Boundary
- Municipal Boundary



**SANITARY SYSTEM
COMPOSITE**

**MUNICIPAL SYSTEM
EXISTING CAPACITY**

SCALE	1:5,000
DESIGN BY	ED
DRAWN BY	ET
DATE	5/4/2023
PROJECT REF No.	302-2051

DRAWING No. 305-2051-02	SHEET 1 of 2
	REVISION 01





Legend

Sanitary Gravity Main - Capacity

Not Deficient (<50%)

Minor Deficiency (50-90%)

Major Deficiency (>90%)

Not Included in Model

Sanitary System

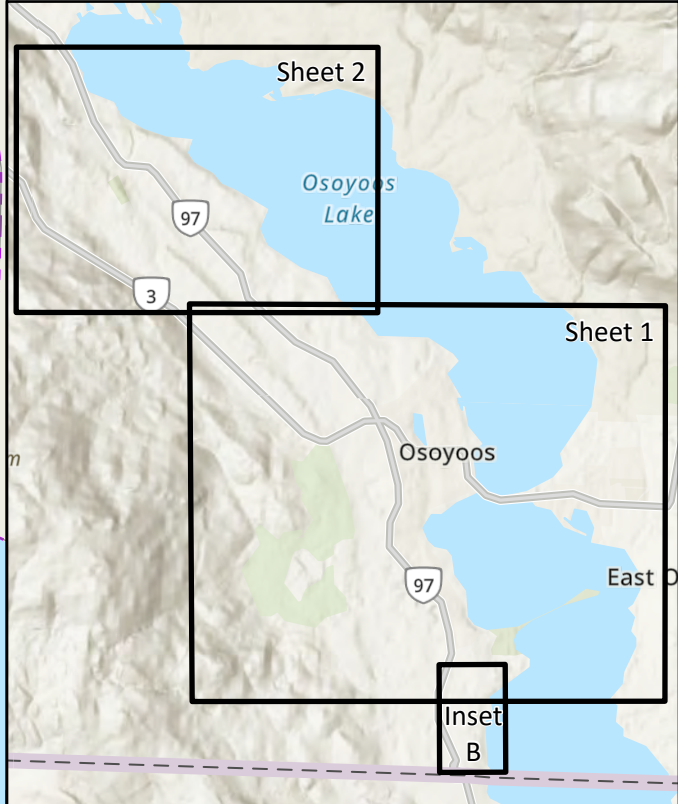
Sanitary Manhole

Sanitary Lift Station

Sanitary Forcemain

First Nation Reserve Boundary

Municipal Boundary



**SANITARY SYSTEM
COMPOSITE**

**NORTHWEST SECTOR
EXISTING CAPACITY**

SCALE	1:5,000
DESIGN BY	
DRAWN BY	ET
DATE	5/1/2023
PROJECT REF No.	302-2051

DRAWING No. 305-2051-02	SHEET 2 OF 2
	REVISION --

APPENDIX G

Project Sheets

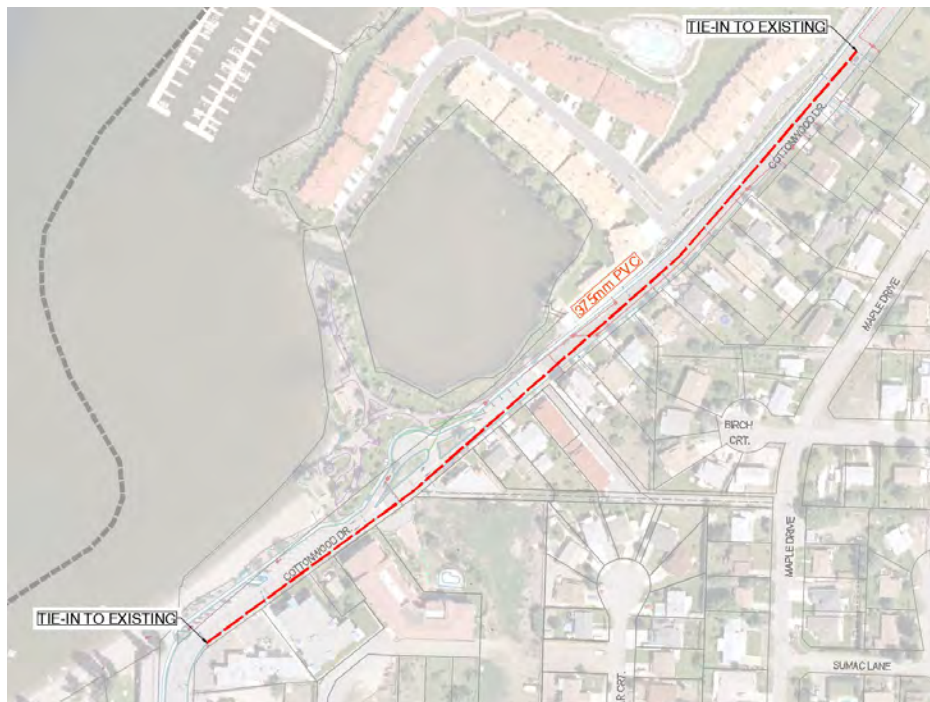


Project: C1 - Cottonwood Drive Trunk Main Upsize

Priority: Medium
Trigger: Development

Type: Upgrade

Location



Issue

Under current PWWF conditions, the Cottonwood trunk main is undersized. Future proposed development both within the Town boundary as well as on OIB lands further overload the existing pipe.

Scope

Approximately 515 meters of existing 250mm PVC pipe will be required to be upsized to 375mm to accommodate future flows from both the Town and additional OIB development.

Time Frame

+/- 3 years



Town of Osoyoos
C1 - Cottonwood Drive Trunk Main Upsize
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C1 - Cottonwood Drive Trunk Main Upsize</u>					
1.1	Wastewater System Components				
	375mm PVC Sanitary Forcemain - Supply and Insta	lm	515	\$450	\$231,750
	1200mm Manhole	ea	13	\$12,000	\$156,000
	Sanitary Sewer Services	ea	29	\$7,500	\$217,500
	System bypass throughout construction	L.S.	1	\$50,000	\$50,000
	Forcemain tie in	L.S.	1	\$20,000	\$20,000
1.2	Other Construction				
	Removals	m ²	2060	\$12	\$24,720
	Asphalt Restoration	m ²	2060	\$80	\$164,800
	Concrete Sidewalk	lm	515	\$180	\$92,700
	Curb and Gutter	lm	515	\$180	\$92,700
	Traffic Control	L.S.	1	\$50,000	\$50,000
	Dewatering	L.S.	1	\$50,000	\$50,000
	Beach Walkway Reinstatement	L.S.	1	\$50,000	\$50,000
Subtotal					\$1,200,000
Archaeology Allowance					\$40,000
Engineering (15%)					\$180,000
Contingency (30%)					\$372,000
TOTAL					\$1,792,000

Prepared by: _____

Checked by: _____



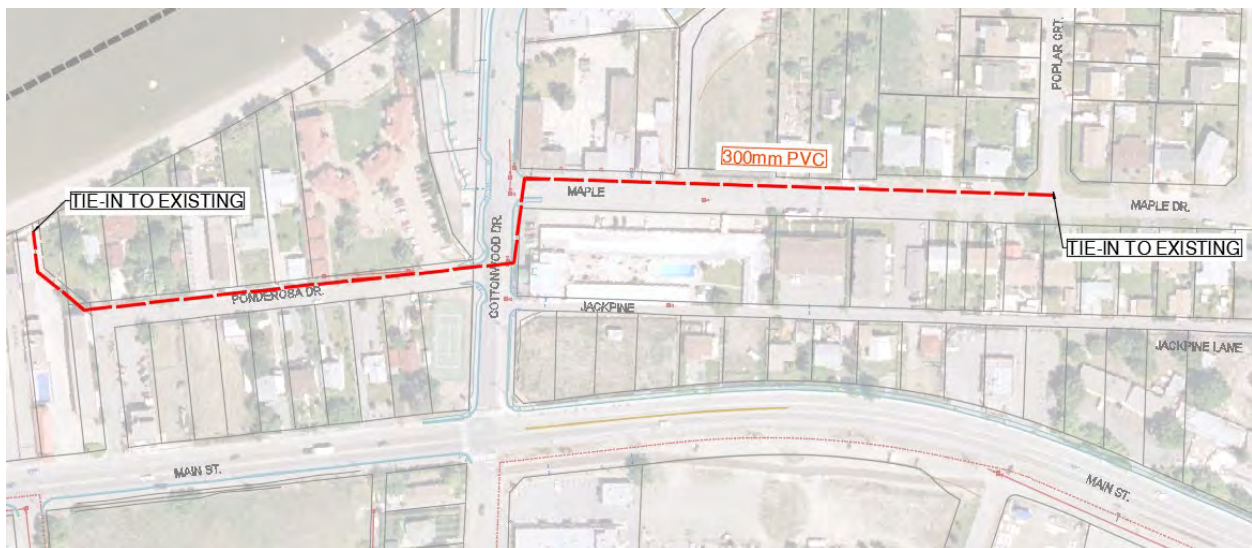
Project: C2 - Ponderosa/Maple Trunk Main Upsize

Priority: Medium

Type: Upgrade

Trigger: Development

Location



Issue

Similar to the Cottonwood trunk main, the existing Maple Drive and Ponderosa Drive trunk mains are undersized for future conditions. Additional development on OIB lands as well as Town densification overload the pipe at PWWF.

Scope

Approximately 535 meters of existing 200mm PVC pipe will be required to be upsized to 300mm to accommodate future flows from both the Town and OIB development.

Time Frame

+/- 3 years



Town of Osoyoos
C2 - Ponderosa/Maple Trunk Main Upsize
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C2 - Ponderosa/Maple Trunk Main Upsize</u>					
1.1	Wastewater System Components				
	300mm PVC Sanitary Force Main - Supply and Install	lm	535	\$350	\$187,250
	1200mm Manhole	ea	9	\$12,000	\$108,000
	Sanitary Sewer Services	ea	15	\$7,500	\$112,500
	System bypass throughout construction	L.S.	1	\$30,000	\$30,000
	Forcemain tie in	L.S.	1	\$20,000	\$20,000
1.2	Other Construction				
	Removals	m ²	2140	\$12	\$25,680
	Asphalt Restoration	m ²	2140	\$80	\$171,200
	Concrete Sidewalk	lm	535	\$180	\$96,300
	Curb and Gutter	lm	535	\$180	\$96,300
	Traffic Control	L.S.	1	\$15,000	\$15,000
	Dewatering	L.S.	1	\$20,000	\$20,000
1.3	Lift Station Upgrades				
	PLC and SCADA system upgrades for monitoring of flows	L.S.	1	\$75,000	\$75,000
	VFD upgrades to allow for flow control	L.S.	1	\$75,000	\$75,000
				Subtotal	\$1,032,000
				Archaeology Allowance	\$40,000
				Engineering (15%)	\$155,000
				Contingency (30%)	\$322,000
				TOTAL	\$1,549,000

Prepared by: _____

Checked by: _____



Project: C3 - Starlite Lift Station Capacity Increase

Priority: Medium

Type: Upgrade

Trigger: Development

Location Map



Issue

The existing Starlite Lift station is undersized for future conditions. Additionally, the current location is proving to be an ongoing operational issue. TRUE has recently completed an options analysis for the relocation of the lift station, and it is anticipated that one of the options discussed will be chosen. For additional information as well as a more detailed cost estimate please refer to the *2023 Starlite Relocation Assessment*.

Scope

The exact scope of this project will depend on the option chosen as per the *2023 Starlite Relocation Assessment*; however it is anticipated that it will include pump upgrades, and wetwell relocation.

Time Frame

+/- 2 years



Town of Osoyoos
C3 - Starlite Lift Station Pumping Upgrades
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C3 - Starlite Lift Station Pumping Upgrades</u>					
1.1	Lift Station Components				
	<i>Middle of the road cost estimate compared to 2023 Starlite LS Relocarion Option Report</i>	L.S.	1	<u>\$2,500,000</u>	<u>\$2,500,000</u>
				Subtotal	<u>\$2,500,000</u>
				Engineering (15%)	\$375,000
				Contingency (30%)	\$750,000
				TOTAL	<u><u>\$3,625,000</u></u>

Prepared by:

Checked by:



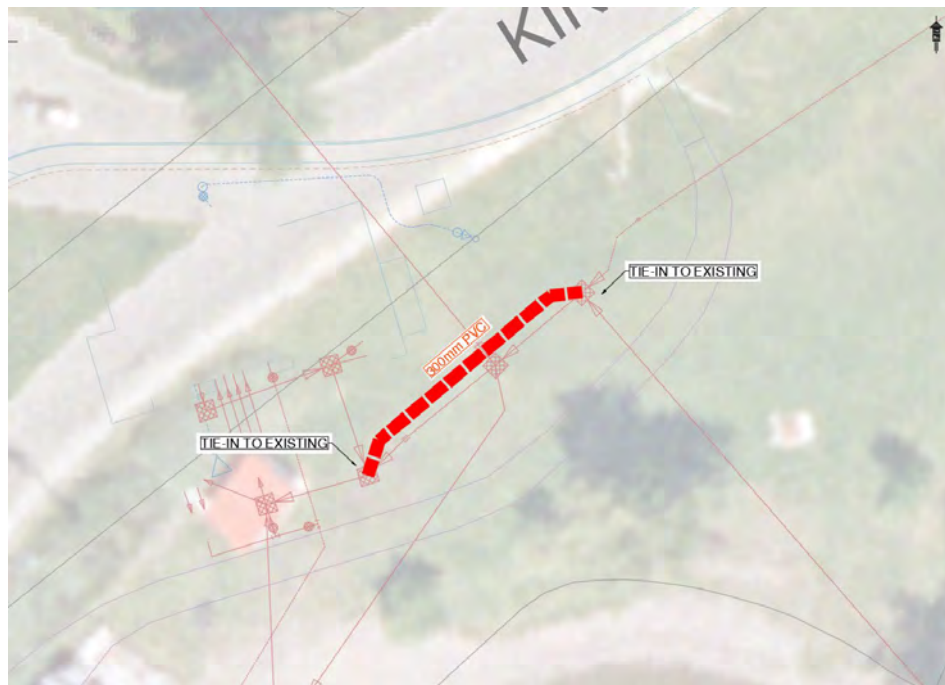
Project: C4 - Main Wastewater Pumping station Forcemain Upgrades

Priority: Medium

Type: Upgrade

Trigger: Development

Location Map



Issue

There is the possibility of surcharging above the pipe invert in the outlet manhole while Smith and Loveless, Starlite, and 44th Ave Lift Stations are pumping due to a constraining section of 300mm gravity pipe.

Scope

The proposed solution to this problem is to extend the 300mm forcemain approximately 25 meters southwest and tie it back into the gravity system beyond the 300mm gravity section.

Time Frame

+/- 3 years



Town of Osoyoos
C4 - Main Wastewater Pumping Station Forcemain Upgrades
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C4 - Main Wastewater Pumping Station Forcemain Upgrades</u>					
1.1	Wastewater System Components				
	300mm PVC Sanitary Force Main - Supply and Install	lm	25	\$1,000	\$25,000
	Forcemain bends, thrust blocks, and appurtenances	L.S.	1	\$10,000	\$10,000
	Tie-in to manhole	L.S.	1	\$15,000	\$15,000
	Tie-in to forcemain	L.S.	1	\$25,000	\$25,000
1.2	Other Construction				
	Removals	L.S.	1	\$5,000	\$5,000
	Site preparation	m ²	100	\$15	\$1,500
	Site restoration	m ²	100	\$45	\$4,500
	Dewatering	L.S.	1	\$30,000	\$30,000
	Permitting	L.S.	1	\$18,000	\$18,000
	Vac-truck allowance for tie-ins (3 trucks)	hrs	10	\$1,000	\$30,000
Subtotal					\$164,000
Archaeology Allowance					\$40,000
Engineering (15%)					\$25,000
Contingency (30%)					\$61,000
TOTAL					\$290,000

Prepared by: _____

Checked by: _____



Project: C5 – SCADA System Upgrades

Priority: Medium

Type: Information Collection

Trigger: Operations and Maintenance

Location

All Lift Stations

Issue

Various lift stations throughout the sanitary collection network are not currently connected to the SCADA system and logging relevant data. It is recommended to review all lift stations and connect any instrumentation that should be connected. These include: pump on/off, pump run times, flow meters, and level sensors.

Scope

It is recommended to review all lift stations and upgrade/connect any instrumentation that should be connected and providing relevant information. Such information includes pump on/off status, pump run times, flow meters, and level sensors. A full scope can be provided if the Town wishes to reach out to its electrical consultant.

Time Frame

+/- 1 year



Town of Osoyoos
C5 - SCADA System Upgrades
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C5 - SCADA System Upgrades</u>					
1.1	SCADA				
	<i>Review of the current status and recommended upgrades at each lift station</i>	L.S.	1	<u>\$363,000</u>	<u>\$363,000</u>
TOTAL					<u><u>\$363,000</u></u>

Prepared by:

Checked by:



Project: C6 – WWTP Cell 2 Desludging

Priority: High

Type: O&M

Trigger: Operations and Maintenance

Location



Issue

In spring 2023, an overturn in Cell 2 caused a large release of odour to the surrounding area. This was likely due to an accumulation of sludge on the bottom of Cell 2. Additionally, the regular O&M process of desludging has not happened in approximately 10 years..

Scope

Survey, design, and construction services to address the sludge buildup and odour concerns in Cell 2. This project will likely include a sludge survey, lab testing, detailed design package, contract administration, and construction services.

Time Frame

+/- 1 year



Town of Osoyoos
C6 - WWTP Cell 2 Desludging
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C6 - WWTP Cell 2 Desludging</u>					
1.1	WWTP Cell 2 work				
	<i>Initial sludge survey and quantification</i>	L.S.	1	\$120,000	\$120,000
	<i>Preliminary design phase and client review</i>	L.S.	1	\$80,000	\$80,000
	<i>Detailed design</i>	L.S.	1	\$160,000	\$160,000
	<i>Contract administration</i>	L.S.	1	\$40,000	\$40,000
	<i>Construction services c/w baffle lining</i>	L.S.	1	\$1,600,000	\$1,600,000

Subtotal	\$2,000,000
Engineering (15%)	\$300,000
Contingency (30%)	\$600,000
TOTAL	\$2,900,000

Prepared by:

Checked by:



Project: C7 – Air Piping Upgrades at WWTP

Priority: Medium

Type: O&M

Trigger: Pipe Failure

Location



Issue

There are various areas where some of the air supply piping is leaking or cracked and should be addressed. Specifically, approximately at the yellow circle the cast iron supply pipe is leaking at a joint, and at the orange circle one of the HDPE lines is cracked off the header. Other repairs may be noted and require to be addressed as well.

Scope

Install new individual air supply piping for Cells 1, 2, and 3 as well as blower and header upgrades.

Time Frame

+/- 2 years



Town of Osoyoos
C7 - Air Piping Upgrades at WWTP
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C7 - Air Piping Upgrades at WWTP</u>					
1.1	WWTP Air Supply Piping				
	<i>New air headers for each Cell</i>	L.S.	1	<u>\$500,000</u>	<u>\$500,000</u>
	<i>Cell 1 blower</i>	L.S.	1	<u>\$300,000</u>	<u>\$300,000</u>
	<i>Modifications to air headers and blower building for individual cell air supply</i>	L.S.	1	<u>\$200,000</u>	<u>\$200,000</u>
				Subtotal	<u>\$1,000,000</u>
				Engineering (15%)	\$150,000
				Contingency (30%)	\$300,000
				TOTAL	<u><u>\$1,450,000</u></u>

Prepared by:

Checked by:



Project: C8 – Manhole Smart Covers

Priority: Medium

Type: Information Collection

Trigger: Aging Infrastructure

Location

Various, throughout Town

Issue

The PCSWMM desktop model used to develop the Sanitary Capital plan identified various locations throughout Town where surcharging in certain manholes is possible.

Scope

It is recommended that the Town purchase approximately 3 Smart Covers, as well as install a weather station to gain information on the possibility of surcharging, as well as identifying areas where I&I greatly affects the sanitary flows.

Time Frame

+/- 1 year



Town of Osoyoos
C8 - Manhole Smart Covers and Weather Station
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C8 - Manhole Smart Covers and Weather Station</u>					
1.1	Manhole Smart Cover <i>Smart Manhole Covers</i>	ea	3	<u>\$25,000</u>	<u>\$75,000</u>
1.2	Localized Weather Station <i>Weather Station Package</i>	L.S.	1	<u>\$50,000</u>	<u>\$50,000</u>

Subtotal	<u>\$125,000</u>
Engineering (15%)	\$19,000
Contingency (30%)	\$38,000
TOTAL	<u><u>\$182,000</u></u>

Prepared by:

Checked by:



Project: C9 – CCTV Program

Priority: Medium

Type: Information Collection

Trigger: Aging Infrastructure

Location

Various, throughout Town

Issue

Previous CCTV programs have outlined that there are certain areas throughout the Town where the condition of the sanitary collection gravity pipe is not as expected.

Scope

It is recommended to develop a CCTV program that focuses on certain areas of the Town so that there can be a better understanding of the condition of the pipes installed. Specifically, there should be a focus on PVC installed between 1980 and 1995 as previous inspections have shown that the condition of pipes installed between these dates has generally been worse than anticipated.

Time Frame

+/- 2 years, with additional CCTV in the proceeding years



Town of Osoyoos
C9 - CCTV Program
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C9 - CCTV Program</u>					
1.1	CCTV Camera Inspections <i>Price per lineal meter</i>	lm	9,600	<u>\$10</u>	<u>\$96,000</u>

Subtotal	<u>\$96,000</u>
Engineering (15%)	\$14,000
Contingency (30%)	\$29,000
TOTAL	<u><u>\$139,000</u></u>

Prepared by:

Checked by:



Project: C10 – Forcemain Odour Reduction

Priority: Medium

Type: O&M

Trigger: Operations and Maintenance

Location

Various forcemains throughout Town

Issue

There are certain long-run forcemains that receive a chemical that inhibits the production of H₂S, as well as limits odours coming from them. Some of the injection systems are rudimentary and require formalization of the storage and injection quills. Additionally, dosing amount and frequency should be uploaded to SCADA.

Scope

Review the BioMaxx injection system in the forcemains where it is injected and upgrade them as required.

Time Frame

+/- 5 years



Town of Osoyoos
C10 - Forcemain Odour Reduction
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C10 - Forcemain Odour Reduction</u>					
1.1	Existing Systems				
	<i>Current system review</i>	L.S.	1	<u>\$50,000</u>	<u>\$50,000</u>
	<i>Current system upgrades</i>	L.S.	1	<u>\$125,000</u>	<u>\$125,000</u>
1.2	Future Systems				
	<i>Allowance for future system installations</i>	L.S.	1	<u>\$75,000</u>	<u>\$75,000</u>

Subtotal	<u>\$250,000</u>
Engineering (15%)	\$38,000
Contingency (30%)	\$75,000
TOTAL	<u><u>\$363,000</u></u>

Prepared by:

Checked by:



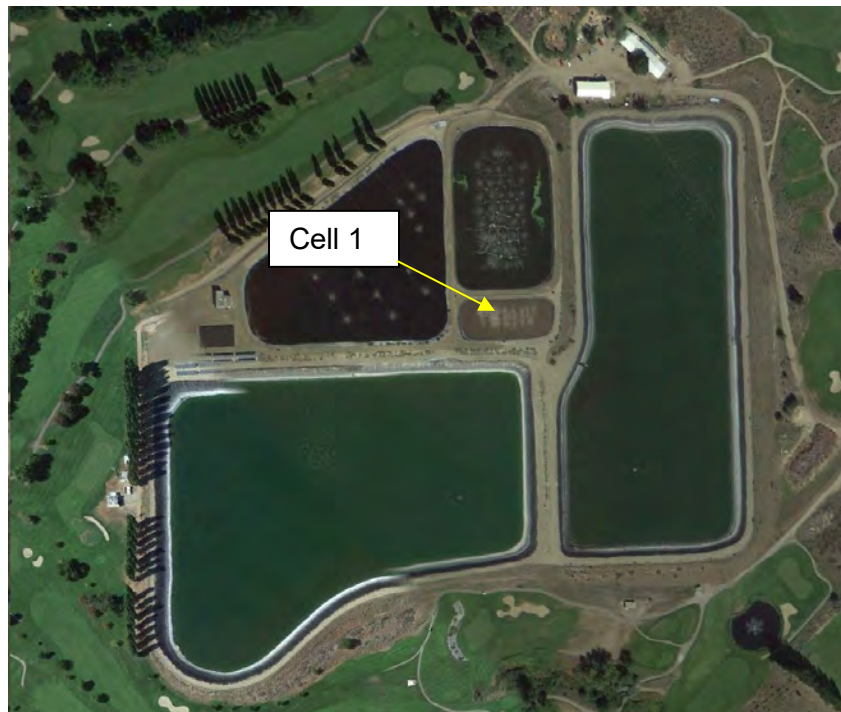
Project: C11 – Cell 1 MBBR Conversion

Priority: Medium

Type: Upgrade

Trigger: Development

Location



Issue

Future flows and regulatory requirements will require treatment upgrades. Currently, it is anticipated that Cell 1 will be converted into a Moving Bed Biofilm Reactor to limit the need for sludge recirculation, however other processes may be pursued.

Scope

Convert Cell 1 to a MBBR including air piping and supply upgrades.

Time Frame

+/- 3 years



Town of Osoyoos
C11 - Cell 1 MBBR Conversion
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C11 - Cell 1 MBBR Conversion</u>					
1.1	Wastewater System Components				
	400 m ³ concrete tanks	L.S.	1	\$650,000	\$650,000
	Coarse bubble aeration system	L.S.	1	\$1,150,000	\$1,150,000
	Additional blower capacity	L.S.	1	\$250,000	\$250,000
	Additional air piping	L.S.	1	\$90,000	\$90,000
	Additional WWTP pipe network	L.S.	1	\$75,000	\$75,000
	Allowance for Archaeology	L.S.	1	\$35,000	\$35,000
1.2	Cell 1 Building				
	Building Construction	sq ft	5000	\$500	\$2,500,000
	Odour Control	L.S.	1	\$500,000	\$500,000
				Subtotal	\$5,250,000
				Engineering (15%)	\$788,000
				Contingency (30%)	\$1,575,000
				TOTAL	\$7,613,000

Prepared by:

Checked by:



Project: C12 – Golf Course LS Screening

Priority: Medium

Type: Upgrade

Trigger: Development

Location



Issue

Existing Golf Course lift station does not have screening, whereas all other wastewater is screened at the MWWPS. Screened effluent is required for the MBBR process therefore this must happen prior to any upgrades at the WWTP.

Scope

Install a primary screening facility at the Golf Course LS to improve effluent quality for the MBBR process.

Time Frame

+/- 3 years



Town of Osoyoos
C12 - Golf Course Primary Screening Facility
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C12 - Golf Course Primary Screening Facility</u>					
1.1	Lift Station Upgrades				
	Removals	L.S.	1	\$25,000	\$25,000
	Site Works	L.S.	1	\$50,000	\$50,000
	Primary Screening Facility	L.S.	1	\$350,000	\$350,000
	SCADA Upgrades	L.S.	1	\$120,000	\$75,000

Subtotal	\$500,000
Engineering (15%)	\$75,000
Contingency (30%)	\$150,000
TOTAL	\$725,000

Prepared by:

Checked by:



Project: C13 – Reclaimed WW Storage and Extension Study

Priority: Medium

Type: Study

Trigger: Expansion

Location

Reclaimed WW Application Area

Issue

A study was completed in 2014 looking at the long-range forecast for the reclaimed WW generation, storage, and application. This plan is now outdated and should be updated to reflect current conditions.

Scope

It is recommended that this study be reviewed and updated as the loading and application forecasts may have changed.

Time Frame

+/- 2 years



Town of Osoyoos
C13 - Reclaimed WW Expansion Study
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C13 - Reclaimed WW Expansion Study</u>					
1.1	Reclaimed WW Expansion Study Consulting Fees	L.S.	1	<u>\$109,000</u>	<u>\$109,000</u>
				TOTAL	<u><u>\$109,000</u></u>

Prepared by:

Checked by:



Project: C14 – Onsite Chlorine Generation

Priority: Medium

Type: Upgrade

Trigger: Aging Infrastructure

Location



Issue

Currently, the Town relies on Chlorine gas to dose the reclaimed wastewater prior to retention in the storage basins.

Scope

The Town would like to replace the existing system with a safer alternative to the chlorine gas, such as onsite chlorine generation. This will require bulk solids handling, a storage facility, as well as a generation system.

Time Frame

+/- 3 years



Town of Osoyoos
C14 - WWTP Onsite Chlorine Generation
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C14 - WWTP Onsite Chlorine Generation</u>					
1.1	Existing System				
	Removal of existing gas system	L.S.	1	\$25,000	\$25,000
	Disposal and Environmental Fees	L.S.	1	\$50,000	\$50,000
1.2	New System				
	WWTP Site Improvements	L.S.	1	\$225,000	\$225,000
	Onsite salt storage facility and handling	L.S.	1	\$550,000	\$550,000
	Onsite conveyance and generation system	L.S.	1	\$650,000	\$650,000
				Subtotal	\$1,500,000
				Engineering (15%)	\$225,000
				Contingency (30%)	\$450,000
				TOTAL	\$2,175,000

Prepared by: _____

Checked by: _____



Project: C15 – Chlorine Contact Tank Recirculation

Priority: Medium

Type: Upgrade

Trigger: Growth

Location



Issue

As reclaimed WW flows increase, there is insufficient CT in the current contact chamber prior to flowing to the storage basins.

Scope

Expansion and/or recirculation will be needed on the reclaimed WW contact chamber to ensure that the proper CT is being achieved.

Time Frame

+/- 3 years



Town of Osoyoos
C15 - Chlorine Contact Tank Recirculation Upgrades
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C15 - Chlorine Contact Tank Recirculation Upgrades</u>					
1.1	Existing System				
	Existing system removals	L.S.	1	\$25,000	\$25,000
	Existing system retrofit and upgrades	L.S.	1	\$225,000	\$225,000
				Subtotal	\$250,000
				Engineering (15%)	\$38,000
				Contingency (30%)	\$75,000
				TOTAL	\$363,000

Prepared by:

Checked by:



Project: C16 – WWTP Auxiliary Power

Priority: Medium

Type: Install

Trigger: Emergency Preparedness

Location



Issue

There is currently no backup power at the WWTP. In the event of a power outage, the treatment is inoperable and may result in treatment complications.

Scope

Install a sufficiently large generator to provide adequate power to the WWTP in the event of a power outage.

Time Frame

+/- 1 year



Town of Osoyoos
C16 - WWTP Auxiliary Power
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C16 - WWTP Auxiliary Power</u>					
1.1	WWTP Site Work				
	<i>WWTP Sitegrading improvements</i>	L.S.	1	\$25,000	\$25,000
	<i>Generator Pad</i>	L.S.	1	\$10,000	\$10,000
	<i>SCADA Integration</i>	L.S.	1	\$35,000	\$35,000
1.2	Generator Supply				
	<i>Supply</i>	L.S.	1	\$80,000	\$80,000
				Subtotal	\$150,000
				Engineering (15%)	\$23,000
				Contingency (30%)	\$45,000
				TOTAL	\$218,000

Prepared by:

Checked by:



Project: C17 – Wastewater Master Plan

Priority: Medium

Type: Study

Trigger: Expansion

Location

Wastewater collection, treatment, and disposal system.

Issue

The WW Master Plan has not been updated.

Scope

It is recommended to create a Master Plan for the collection, treatment, and disposal of wastewater throughout the entire treatment network.

Time Frame

+/- 5 years



Town of Osoyoos
C17 - Wastewater Collection and Treatment Master Plan Update
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C17 - Wastewater Collection and Treatment Master Plan Updat</u>					
1.1	Wastewater Collection and Treatment Master Plan Update Consulting Fees	L.S.	1	<u>\$45,000</u>	<u>\$45,000</u>
				TOTAL	<u><u>\$45,000</u></u>

Prepared by:

Checked by:



Project: C18 – Magnolia Place Reconstruction

Priority: Medium

Type: Upgrade

Trigger: Asset Renewal

Location



Issue

The sanitary pipe in Magnolia Drive is failing and requires upgrades.

Scope

The existing 200mm PVC pipe is being replaced with new 200mm PVC and will be graded to support proper flow.

Time Frame

+/- 2 years



Town of Osoyoos
C18 - Magnolia Drive Reconstruction
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C18 - Magnolia Drive Reconstruction</u>					
1.1	Wastewater System Components				
	200 PVC Sanitary Force Main - Supply and Install	lm	150	\$350	\$52,500
	1200mm Manhole	ea	7	\$12,000	\$84,000
	Sanitary Sewer Services	ea	5	\$7,500	\$37,500
	System bypass throughout construction	L.S.	1	\$30,000	\$30,000
1.2	Other Construction				
	Removals	m ²	500	\$20	\$10,000
	Asphalt Restoration	m ²	500	\$100	\$50,000
	Traffic Control	L.S.	1	\$17,000	\$17,000
	Dewatering	L.S.	1	\$50,000	\$50,000
	Archaeology	L.S.	1	\$45,000	\$45,000
				Subtotal	\$240,000
				Engineering (15%)	\$36,000
				Contingency (30%)	\$72,000
				TOTAL	\$348,000

Prepared by:

Checked by:



Project: C19 – WWTP Preliminary Design and Planning

Priority: Medium

Type: Study

Trigger: Expansion

Location



Issue

As discussed in Project C11, treatment upgrades will be required at the WWTP as growth proceeds.

Scope

Prior to detailed design and process selection, it is recommended that

Time Frame

+/- 2 years



Town of Osoyoos
C19 - Wastewater Treatment Plant Design and Planning
Class 'D' Cost Estimate

ITEM NO.	DESCRIPTION	UNIT	EST. QUANT.	UNIT PRICE	TOTAL PRICE
<u>C19 - Wastewater Treatment Plant Design and Planning</u>					
1.1	C19 - Wastewater Treatment Plant Design and Planning				
	<i>Consulting Fees</i>	L.S.	1	<u>\$700,000</u>	<u>\$700,000</u>
	<i>Permitting</i>	L.S.	1	<u>\$300,000</u>	<u>\$300,000</u>
TOTAL					<u><u>\$1,000,000</u></u>

Prepared by:

Checked by:

APPENDIX H

Assets Removed from AM Analysis

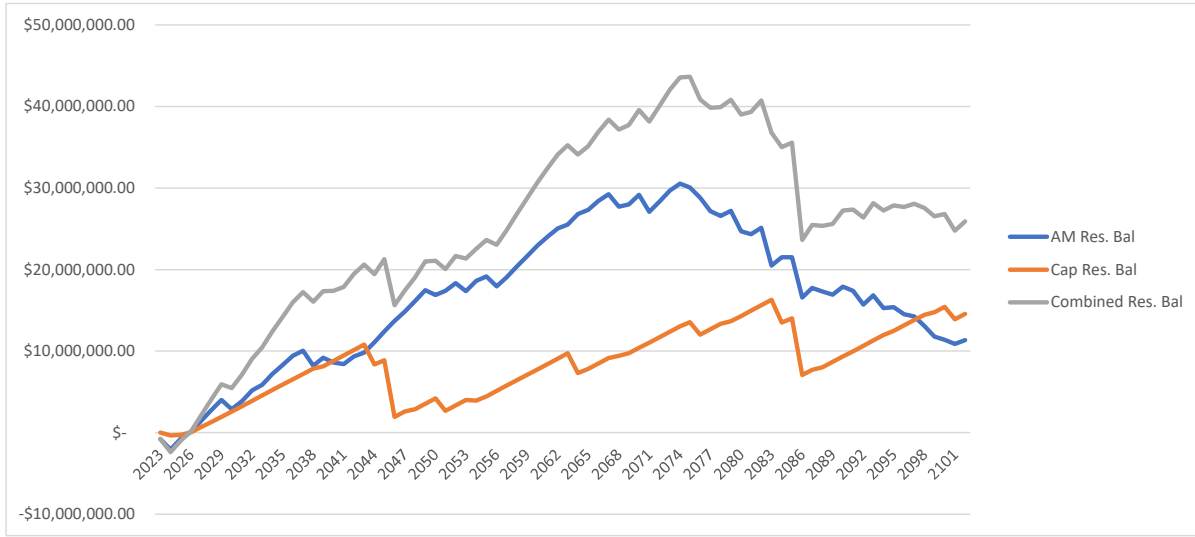
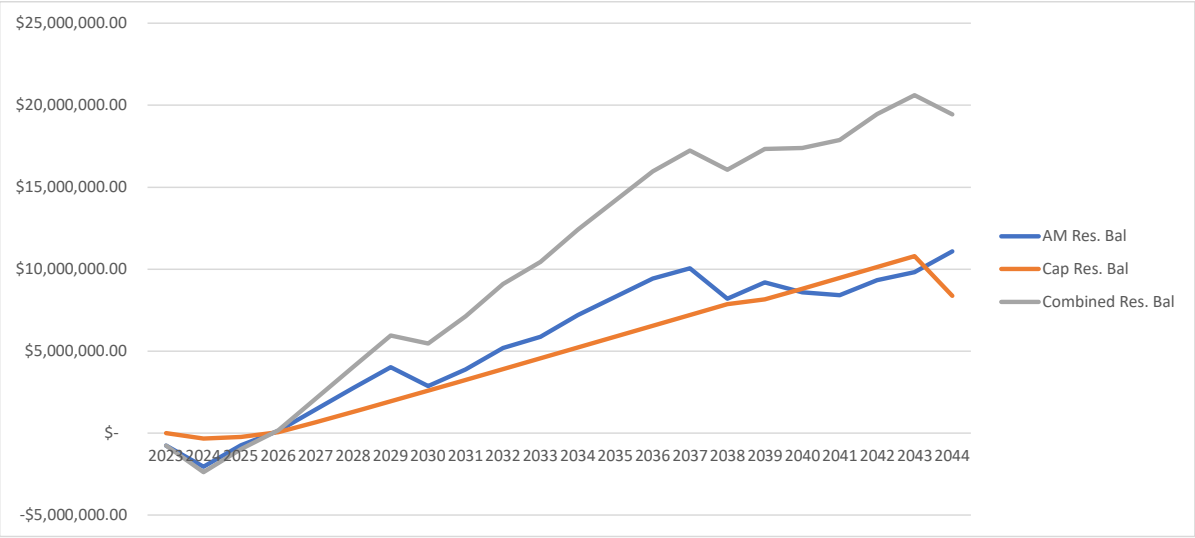
Item	QTY	Unit	Year Installed	Expected Service Life (Years)	Useful Life Remaining (Years)	Estimated Replacement Date	Current Total Replacement Cost	Average Annual Investment
Cottonwood Upsizing C1								
375Ø PVC Sanitary Force Main - Supply and Install	515	l.m.	2026	80	2106	2106	\$231,750	\$2,897
1200mm Manhole	13	ea	2026	80	2106	2106	\$156,000	\$1,950
Sanitary Sewer Services	29	ea	2026	80	2106	2106	\$217,500	\$2,719
System bypass throughout construction	1	L.S.	2026	80	2106	2106	\$50,000	\$625
Force main tie in	1	L.S.	2026	80	2106	2106	\$20,000	\$250
Removals	2060	m²	2026	80	2106	2106	\$24,720	\$309
Asphalt Restoration	2060	m²	2026	80	2106	2106	\$164,800	\$2,060
Concrete Sidewalk	515	l.m.	2026	80	2106	2106	\$92,700	\$1,159
Curb and Gutter	515	l.m.	2026	80	2106	2106	\$92,700	\$1,159
Traffic Control	1	L.S.	2026	80	2106	2106	\$50,000	\$625
Dewatering	1	L.S.	2026	80	2106	2106	\$50,000	\$625
Beach Walkway Reinstatement	1	L.S.	2026	80	2106	2106	\$50,000	\$625
						Subtotal	\$1,200,170	\$15,002
						Engineering (15%) and Contingency (30%)	\$540,000	\$6,751
C1						Total	\$1,740,170	\$21,753
Maple Drive Upsizing C2								
300Ø PVC Sanitary Force Main - Supply and Install	535	l.m.	2026	80	2106	2106	\$187,250	\$2,341
1200mm Manhole	9	ea	2026	80	2106	2106	\$108,000	\$1,350
Sanitary Sewer Services	15	ea	2026	80	2106	2106	\$112,500	\$1,406
System bypass throughout construction	1	L.S.	2026	80	2106	2106	\$30,000	\$375
Force main tie in	1	L.S.	2026	80	2106	2106	\$20,000	\$250
Removals	2140	m²	2026	80	2106	2106	\$25,680	\$321
Asphalt Restoration	2140	m²	2026	80	2106	2106	\$171,200	\$2,140
Concrete Sidewalk	535	l.m.	2026	80	2106	2106	\$96,300	\$1,204
Curb and Gutter	535	l.m.	2026	80	2106	2106	\$96,300	\$1,204
Traffic Control	1	L.S.	2026	80	2106	2106	\$15,000	\$188
Dewatering	1	L.S.	2026	80	2106	2106	\$20,000	\$250
PLC and SCADA system upgrades for monitoring of flows	1	L.S.	2026	80	2106	2106	\$75,000	\$938
VFD upgrades to allow for flow control	1	L.S.	2026	80	2106	2106	\$75,000	\$938
						Subtotal	\$1,032,230	\$12,903
						Engineering (15%) and Contingency (30%)	\$465,000	\$5,806
C2						Total	\$1,497,230	\$18,709
Starlite Lift Station Relocation C3								
Relocate Starlite L.S. to location as determined by the Town	1	L.S.	2025	80	2105	2105	\$2,500,000	\$31,250
						Subtotal	\$2,500,000	\$31,250
						Engineering (15%) and Contingency (30%)	\$1,125,000	\$14,063
C3						Total	\$3,625,000	\$45,313
Force main Extension near WWTP C4								
The S&L, Starlite, and 44th Ave force main need to be extended approximately 30m to reduce the surcharging effects near the MWWPS	1	L.S.	2027	80	2107	2107	\$200,000	\$2,500
						Subtotal	\$200,000	\$2,500
						Engineering (15%) and Contingency (30%)	\$90,000	\$1,125
C4						Total	\$290,000	\$3,625
SCADA System Upgrades C5								
Confirm and update that SCADA is working and logging at each lift station	1	L.S.	2024	15	2039	2039	\$250,000	\$16,667
						Subtotal	\$250,000	\$16,667
						Engineering (15%) and Contingency (30%)	\$113,000	\$7,500
C5						Total	\$363,000	\$24,167
WWTP Cell 2 Desludging C6								
Desludge Cell 2 at the WWTP to reduce odour complaints	1	L.S.	2024	20	2044	2044	\$2,000,000	\$100,000
						Subtotal	\$2,000,000	\$100,000
						Engineering (15%) and Contingency (30%)	\$900,000	\$45,000
C6						Total	\$2,900,000	\$145,000
Air Piping Updates at WWTP C7								
Address the leaking air piping in various locations at the WWTP	1	L.S.	2025	80	2105	2105	\$1,000,000	\$12,500
						Subtotal	\$1,000,000	\$12,500
						Engineering (15%) and Contingency (30%)	\$450,000	\$5,625
C7						Total	\$1,450,000	\$18,125
Manhole Smart Covers C8								
Install Manhole Smart Covers in areas of concern to gain an understanding of the current surcharging in key manholes	1	L.S.	2024	20	2044	2044	\$125,000	\$6,250
						Subtotal	\$125,000	\$6,250
						Engineering (15%) and Contingency (30%)	\$56,000	\$2,813
C8						Total	\$181,000	\$9,063
CCTV Program C9								
Develop a program to inspect all PVC pipe installed between 1980 and 1995 that has not been previously inspected through CCTV.	1							

Item	QTY	Unit	Year Installed	Expected Service Life (Years)	Useful Life Remaining (Years)	Estimated Replacement Date	Current Total Replacement Cost	Average Annual Investment	
Cottonwood Upsizing									
C1	515	l.m.	2026	80	2106	2106	\$231,750	\$2,897	
375Ø PVC Sanitary Forcemain - Supply and Install	13	ea	2026	80	2106	2106	\$156,000	\$1,950	
1200mm Manhole	29	ea	2026	80	2106	2106	\$217,500	\$2,719	
Sanitary Sewer Services	1	L.S.	2026	80	2106	2106	\$50,000	\$625	
System bypass throughout construction	1	L.S.	2026	80	2106	2106	\$20,000	\$250	
Forcemain tie in	2060	m²	2026	80	2106	2106	\$24,720	\$309	
Removals	2060	m²	2026	80	2106	2106	\$164,800	\$2,060	
Asphalt Restoration	515	l.m.	2026	80	2106	2106	\$92,700	\$1,159	
Concrete Sidewalk	515	l.m.	2026	80	2106	2106	\$92,700	\$1,159	
Curb and Gutter	1	L.S.	2026	80	2106	2106	\$50,000	\$625	
Traffic Control	1	L.S.	2026	80	2106	2106	\$50,000	\$625	
Dewatering	1	L.S.	2026	80	2106	2106	\$50,000	\$625	
Beach Walkway Reinstatement	1	L.S.	2026	80	2106	2106	\$50,000	\$625	
						Subtotal	\$1,200,170	\$15,002	
						Engineering (15%) and Contingency (30%)	\$540,000	\$6,751	
C1						Total	\$1,740,170	\$21,753	
Maple Drive Upsizing									
C2	535	l.m.	2026	80	2106	2106	\$187,250	\$2,341	
300Ø PVC Sanitary Force Main - Supply and Install	9	ea	2026	80	2106	2106	\$108,000	\$1,350	
1200mm Manhole	15	ea	2026	80	2106	2106	\$112,500	\$1,406	
Sanitary Sewer Services	1	L.S.	2026	80	2106	2106	\$30,000	\$375	
System bypass throughout construction	1	L.S.	2026	80	2106	2106	\$20,000	\$250	
Forcemain tie in	2140	m²	2026	80	2106	2106	\$25,680	\$321	
Removals	2140	m²	2026	80	2106	2106	\$171,200	\$2,140	
Asphalt Restoration	535	l.m.	2026	80	2106	2106	\$96,300	\$1,204	
Concrete Sidewalk	535	l.m.	2026	80	2106	2106	\$96,300	\$1,204	
Curb and Gutter	1	L.S.	2026	80	2106	2106	\$15,000	\$188	
Traffic Control	1	L.S.	2026	80	2106	2106	\$20,000	\$250	
Dewatering	1	L.S.	2026	80	2106	2106	\$75,000	\$938	
PLC and SCADA system upgrades for monitoring of flows	1	L.S.	2026	80	2106	2106	\$75,000	\$938	
VFD upgrades to allow for flow control	1	L.S.	2026	80	2106	2106	\$75,000	\$938	
						Subtotal	\$1,032,230	\$12,903	
						Engineering (15%) and Contingency (30%)	\$465,000	\$5,806	
C2						Total	\$1,497,230	\$18,709	
Starlite Lift Station Relocation									
C3	1	L.S.	2025	80	2105	2105	\$2,500,000	\$31,250	
Relocate Starlite LS to location as determined by the Town							Subtotal	\$2,500,000	\$31,250
						Engineering (15%) and Contingency (30%)	\$1,125,000	\$14,063	
C3						Total	\$3,625,000	\$45,313	
Forcemain Extension near WWTP									
C4	1	L.S.	2027	80	2107	2107	\$200,000	\$2,500	
The S&L, Starlite, and 44th Ave forcemain need to be extended approximately 30m to reduce the surcharging effects near the MWWPS							Subtotal	\$200,000	\$2,500
						Engineering (15%) and Contingency (30%)	\$90,000	\$1,125	
C4						Total	\$290,000	\$3,625	
SCADA System Upgrades									
C5	1	L.S.	2024	15	2039	2039	\$250,000	\$16,667	
Confirm and update that SCADA is working and logging at each lift station							Subtotal	\$250,000	\$16,667
						Engineering (15%) and Contingency (30%)	\$113,000	\$7,500	
C5						Total	\$363,000	\$24,167	
WWTP Cell 2 Desludging									
C6	1	L.S.	2024	20	2044	2044	\$2,000,000	\$100,000	
Desludge Cell 2 at the WWTP to reduce odour complaints							Subtotal	\$2,000,000	\$100,000
						Engineering (15%) and Contingency (30%)	\$900,000	\$45,000	
C6						Total	\$2,900,000	\$145,000	
Air Piping Updates at WWTP									
C7	1	L.S.	2025	80	2105	2105	\$1,000,000	\$12,500	
Address the leaking air piping in various locations at the WWTP							Subtotal	\$1,000,000	\$12,500
						Engineering (15%) and Contingency (30%)	\$450,000	\$5,625	
C7						Total	\$1,450,000	\$18,125	
Manhole Smart Covers									
C8	1	L.S.	2024	20	2044	2044	\$125,000	\$6,250	
Install Manhole Smart Covers in areas of concern to gain an understanding of the current surcharging in key manholes							Subtotal	\$125,000	\$6,250
						Engineering (15%) and Contingency (30%)	\$56,000	\$2,813	
C8						Total	\$181,000	\$9,063	
CCTV Program									
C9	1	L.S.	2025	10	2035	2035	\$96,000	\$9,600	
Develop a program to inspect all PVC pipe installed between 1980 and 1995 that has not been previously inspected through CCTV.							Subtotal	\$96,000	\$9,600
						Engineering (15%) and Contingency (30%)	\$43,000	\$4,320	
C9						Total	\$139,000	\$13,920	
Forcemain Odour Reduction									
C10	1	L.S.	2028	20	2048	2048	\$250,000	\$12,500	
Inspect and update the BioMaxx injection systems in certain lift stations where the injection systems are present.							Subtotal	\$250,000	\$12,500
						Engineering (15%) and Contingency (30%)	\$113,000	\$5,625	
C10						Total	\$363,000	\$18,125	
Cell 1 MBBR Conversion									
C11	1	L.S.	2026	40	2066	2066	\$5,250,000	\$131,250	
Convert the existing aeration cell at the WWTP to an MBBR							Subtotal	\$5,250,000	\$131,250
						Engineering (15%) and Contingency (30%)	\$2,363,000	\$59,063	
C11						Total	\$7,613,000	\$190,313	

Golf Course Lift Station Screening C12								
Install pree screening at the Golf Course LS	1	L.S.	2025	80	2105	2105	\$500,000	\$6,250
Subtotal							\$500,000	\$6,250
Engineering (15%) and Contingency (30%)							\$225,000	\$2,813
C12 Total							\$725,000	\$9,063
Reclaimed WW Storage and Extension Study C13								
Study to assess the reclaimed WW system and determine when extensions will be required.	1	L.S.	2025	20	2045	2045	\$75,000	\$3,750
Subtotal							\$75,000	\$3,750
Engineering (15%) and Contingency (30%)							\$34,000	\$1,688
C13 Total							\$109,000	\$5,438
Onsite Chlorine Generation C14								
Convert the existing gas system to onsite generation. This project will partially be funded by the replacement of the existing system	1	L.S.	2026	25	2051	2051	\$1,500,000	\$60,000
Subtotal							\$1,500,000	\$60,000
Engineering (15%) and Contingency (30%)							\$675,000	\$27,000
C14 Total							\$2,175,000	\$87,000
Chlorine Contact Chamber Recirculation C15								
Upgrades to the CT tank for the reclaimed water at the WWTP.	1	L.S.	2029	25	2054	2054	\$250,000	\$10,000
Subtotal							\$250,000	\$10,000
Engineering (15%) and Contingency (30%)							\$113,000	\$4,500
C15 Total							\$363,000	\$14,500
WWTP Auxiliary Power C16								
Install backup power generator at the WWTP	1	L.S.	2024	25	2049	2049	\$150,000	\$6,000
Subtotal							\$150,000	\$6,000
Engineering (15%) and Contingency (30%)							\$68,000	\$2,700
C16 Total							\$218,000	\$8,700
Wastewater Master Plan Review C17								
Complete a detailed review/update of the master plan	1	L.S.	2028	5	2033	2033	\$31,000	\$6,200
Subtotal							\$31,000	\$6,200
Engineering (15%) and Contingency (30%)							\$14,000	\$2,790
C17 Total							\$45,000	\$8,990
Magnolia Place Reconstruction C18								
Construction for Magnolia as per design	1	L.S.	2024	80	2104	2104	\$240,000	\$3,000
Subtotal							\$240,000	\$3,000
Engineering (15%) and Contingency (30%)							\$108,000	\$1,350
C18 Total							\$348,000	\$4,350
WWTP Design and Planning C19								
Preliminary upgrades and planning for WWTP	1	L.S.	2024	80	2104	2104	\$700,000	\$8,750
Subtotal							\$700,000	\$8,750
Engineering (15%) and Contingency (30%)							\$315,000	\$3,938
C19 Total							\$1,015,000	\$12,688

APPENDIX I

Capital Reserve Balance Model



APPENDIX J

Modified Work Forecast Reserve Model

Date 6/22/2023
Assumptions No interest on positive value, none on negative either

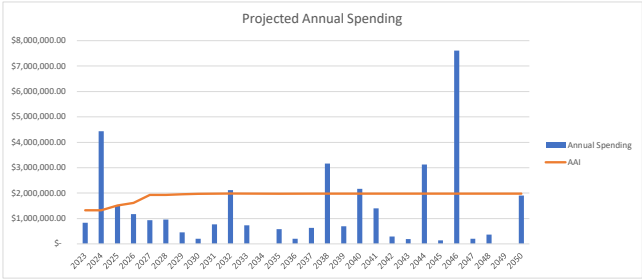
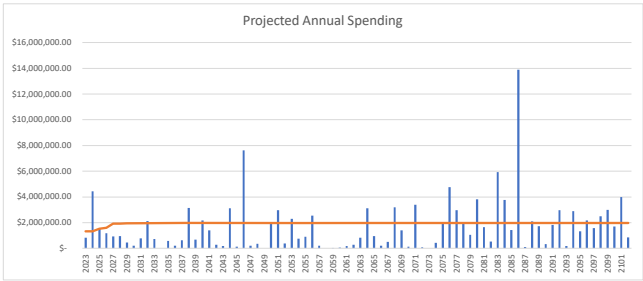
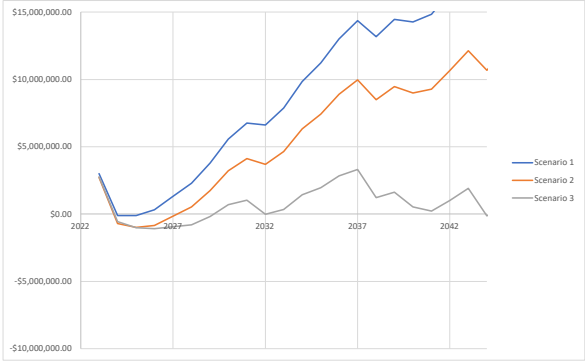
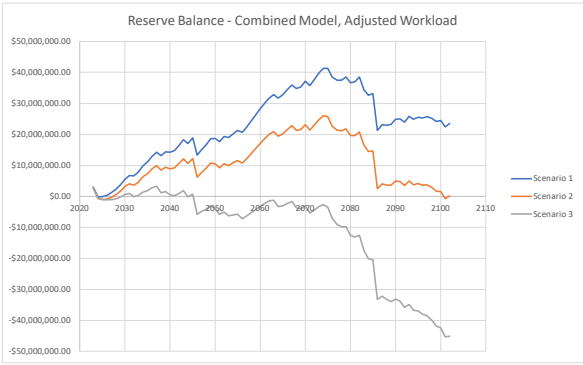
AAI to be 0 in 2033 for AM \$ 1,022,450.18
AAI to be 0 in 2033 for all \$ 1,087,750.18

Reserve Balance: \$ 2,527,498.20
8-May-23 Assumed No of Connections 4500

Scenario 1 is normal AAI plus upgrades, consideres starting res
Scenario 2 is the AAI to be 0 in 2033 + Upgrades and O&M AAI. Includes starting reserve
Scenario 3 is adjusted AAI but still hit 0 by 2033

repeated value

	Scenario 1					Scenario 2					Scenario 3				
Year	AM Spending	Capital Spending	Total Spending	Total AAI	Change in Reserve	Actual Reserve	Balance	AAI	Res Bal	AAI	Res Bal	AAI	Res Bal		
2023	\$ 831,500.00	\$ -	\$ 831,500.00	\$ 1,315,415.69	\$483,915.69	\$3,011,413.89	\$ 1,022,450.18	\$2,718,448.38	\$ 1,087,750.18	\$2,783,748.38	\$ 1,087,750.18	\$2,783,748.38	\$ 1,087,750.18		
2024	\$ 3,890,000.00	\$ 544,000.00	\$ 4,434,000.00	\$ 1,315,415.69	-\$3,118,584.31	-\$107,170.43	\$ 1,022,450.18	-\$693,101.44	\$ 1,087,750.18	-\$562,501.44	\$ 1,087,750.18	-\$562,501.44	\$ 1,087,750.18		
2025	\$ 1,420,000.00	\$ 109,000.00	\$ 1,529,000.00	\$ 1,519,382.35	-\$9,617.65	-\$116,788.08	\$ 1,226,416.85	-\$995,684.59	\$ 1,087,750.18	-\$1,003,751.26	\$ 1,087,750.18	-\$1,003,751.26	\$ 1,087,750.18		
2026	\$ 1,170,500.00	\$ -	\$ 1,170,500.00	\$ 1,611,239.85	\$440,739.85	\$323,951.78	\$ 1,318,274.35	-\$847,910.25	\$ 1,087,750.18	-\$1,086,501.08	\$ 1,087,750.18	-\$1,086,501.08	\$ 1,087,750.18		
2027	\$ 930,000.00	\$ -	\$ 930,000.00	\$ 1,929,014.60	\$999,014.60	\$1,322,966.38	\$ 1,636,049.10	-\$141,861.15	\$ 1,087,750.18	-\$928,750.90	\$ 1,087,750.18	-\$928,750.90	\$ 1,087,750.18		
2028	\$ 960,000.00	\$ -	\$ 960,000.00	\$ 1,932,639.60	\$972,639.60	\$2,295,605.98	\$ 1,639,674.10	\$537,812.95	\$ 1,087,750.18	-\$801,000.72	\$ 1,087,750.18	-\$801,000.72	\$ 1,087,750.18		
2029	\$ 460,000.00	\$ -	\$ 460,000.00	\$ 1,959,754.60	\$1,499,754.60	\$3,795,360.58	\$ 1,666,789.10	\$1,744,602.04	\$ 1,087,750.18	-\$173,250.54	\$ 1,087,750.18	-\$173,250.54	\$ 1,087,750.18		
2030	\$ 200,000.00	\$ -	\$ 200,000.00	\$ 1,974,254.60	\$1,774,254.60	\$5,569,615.19	\$ 1,681,289.10	\$3,225,891.14	\$ 1,087,750.18	\$714,499.64	\$ 1,087,750.18	\$714,499.64	\$ 1,087,750.18		
2031	\$ 770,000.00	\$ -	\$ 770,000.00	\$ 1,974,254.60	\$1,204,254.60	\$6,773,869.79	\$ 1,681,289.10	\$4,137,180.24	\$ 1,087,750.18	\$1,032,249.82	\$ 1,087,750.18	\$1,032,249.82	\$ 1,087,750.18		
2032	\$ 2,120,000.00	\$ -	\$ 2,120,000.00	\$ 1,974,254.60	-\$145,745.40	\$6,628,124.39	\$ 1,681,289.10	\$3,698,469.33	\$ 1,087,750.18	\$0.00	\$ 1,087,750.18	\$0.00	\$ 1,087,750.18		
2033	\$ 730,000.00	\$ -	\$ 730,000.00	\$ 1,974,254.60	\$1,244,254.60	\$7,872,378.99	\$ 1,681,289.10	\$4,649,758.43	\$ 1,087,750.18	\$357,750.18	\$ 1,087,750.18	\$357,750.18	\$ 1,087,750.18		
2034	\$ -	\$ -	\$ -	\$ 1,974,254.60	\$1,974,254.60	\$9,846,633.60	\$ 1,681,289.10	\$6,331,047.53	\$ 1,087,750.18	\$1,445,500.38	\$ 1,087,750.18	\$1,445,500.38	\$ 1,087,750.18		
2035	\$ 575,500.00	\$ -	\$ 575,500.00	\$ 1,974,254.60	\$1,398,754.60	\$11,245,388.20	\$ 1,681,289.10	\$7,436,636.62	\$ 1,087,750.18	\$1,957,750.54	\$ 1,087,750.18	\$1,957,750.54	\$ 1,087,750.18		
2036	\$ 200,000.00	\$ -	\$ 200,000.00	\$ 1,974,254.60	\$1,774,254.60	\$13,019,642.80	\$ 1,681,289.10	\$8,918,125.72	\$ 1,087,750.18	\$2,845,500.72	\$ 1,087,750.18	\$2,845,500.72	\$ 1,087,750.18		
2037	\$ 630,514.00	\$ -	\$ 630,514.00	\$ 1,974,254.60	\$1,343,740.60	\$14,363,383.40	\$ 1,681,289.10	\$9,968,900.82	\$ 1,087,750.18	\$3,302,736.90	\$ 1,087,750.18	\$3,302,736.90	\$ 1,087,750.18		
2038	\$ 3,157,612.25	\$ -	\$ 3,157,612.25	\$ 1,974,254.60	-\$1,183,357.65	\$13,180,025.76	\$ 1,681,289.10	\$8,492,577.66	\$ 1,087,750.18	\$1,232,873.83	\$ 1,087,750.18	\$1,232,873.83	\$ 1,087,750.18		
2039	\$ 330,734.25	\$ 363,000.00	\$ 693,734.25	\$ 1,974,254.60	\$1,280,520.35	\$14,460,546.11	\$ 1,681,289.10	\$9,480,132.51	\$ 1,087,750.18	\$1,620,900.72	\$ 1,087,750.18	\$1,620,900.72	\$ 1,087,750.18		
2040	\$ 2,163,815.60	\$ -	\$ 2,163,815.60	\$ 1,974,254.60	-\$189,560.99	\$14,270,985.11	\$ 1,681,289.10	\$8,997,606.01	\$ 1,087,750.18	\$550,825.34	\$ 1,087,750.18	\$550,825.34	\$ 1,087,750.18		
2041	\$ 1,402,975.25	\$ -	\$ 1,402,975.25	\$ 1,974,254.60	\$571,279.35	\$14,842,264.47	\$ 1,681,289.10	\$9,275,919.86	\$ 1,087,750.18	\$235,600.27	\$ 1,087,750.18	\$235,600.27	\$ 1,087,750.18		
2042	\$ 296,647.50	\$ -	\$ 296,647.50	\$ 1,974,254.60	\$1,677,607.10	\$16,519,871.57	\$ 1,681,289.10	\$10,660,561.45	\$ 1,087,750.18	\$1,026,702.95	\$ 1,087,750.18	\$1,026,702.95	\$ 1,087,750.18		
2043	\$ 194,000.00	\$ -	\$ 194,000.00	\$ 1,974,254.60	\$1,780,254.60	\$18,300,126.17	\$ 1,681,289.10	\$12,147,850.55	\$ 1,087,750.18	\$1,920,453.13	\$ 1,087,750.18	\$1,920,453.13	\$ 1,087,750.18		
2044	\$ 50,000.00	\$ 3,081,000.00	\$ 3,131,000.00	\$ 1,974,254.60	-\$1,156,745.40	\$17,143,380.77	\$ 1,681,289.10	\$10,698,139.65	\$ 1,087,750.18	-\$122,796.69	\$ 1,087,750.18	-\$122,796.69	\$ 1,087,750.18		
2045	\$ -	\$ 139,000.00	\$ 139,000.00	\$ 1,974,254.60	\$1,835,254.60	\$18,978,635.38	\$ 1,681,289.10	\$12,240,428.74	\$ 1,087,750.18	\$825,953.49	\$ 1,087,750.18	\$825,953.49	\$ 1,087,750.18		
2046	\$ -	\$ 7,613,000.00	\$ 7,613,000.00	\$ 1,974,254.60	-\$5,638,745.40	\$13,339,889.98	\$ 1,681,289.10	\$6,308,717.84	\$ 1,087,750.18	-\$5,699,296.33	\$ 1,087,750.18	-\$5,699,296.33	\$ 1,087,750.18		
2047	\$ 200,000.00	\$ -	\$ 200,000.00	\$ 1,974,254.60	\$1,774,254.60	\$15,114,144.58	\$ 1,681,289.10	\$7,790,096.94	\$ 1,087,750.18	-\$4,811,546.15	\$ 1,087,750.18	-\$4,811,546.15	\$ 1,087,750.18		
2048	\$ -	\$ 363,000.00	\$ 363,000.00	\$ 1,974,254.60	\$1,611,254.60	\$16,725,399.18	\$ 1,681,289.10	\$9,108,296.03	\$ 1,087,750.18	-\$4,086,795.97	\$ 1,087,750.18	-\$4,086,795.97	\$ 1,087,750.18		
2049	\$ -	\$ -	\$ -	\$ 1,974,254.60	\$1,974,254.60	\$18,699,653.79	\$ 1,681,289.10	\$10,789,585.13	\$ 1,087,750.18	-\$2,999,045.79	\$ 1,087,750.18	-\$2,999,045.79	\$ 1,087,750.18		
2050	\$ 1,900,000.00	\$ -	\$ 1,900,000.00	\$ 1,974,254.60	\$74,254.60	\$18,773,908.39	\$ 1,681,289.10	\$10,570,874.23	\$ 1,087,750.18	-\$3,811,295.61	\$ 1,087,750.18	-\$3,811,295.61	\$ 1,087,750.18		
2051	\$ 804,334.81	\$ 2,175,000.00	\$ 2,979,334.81	\$ 1,974,254.60	-\$1,805,080.21	\$17,768,828.18	\$ 1,681,289.10	\$9,272,828.51	\$ 1,087,750.18	-\$5,699,296.33	\$ 1,087,750.18	-\$5,699,296.33	\$ 1,087,750.18		
2052	\$ 376,209.69	\$ -	\$ 376,209.69	\$ 1,974,254.60	\$1,598,044.91	\$19,366,873.09	\$ 1,681,289.10	\$10,577,907.92	\$ 1,087,750.18	-\$4,991,339.75	\$ 1,087,750.18	-\$4,991,339.75	\$ 1,087,750.18		
2053	\$ 2,305,218.81	\$ -	\$ 2,305,218.81	\$ 1,974,254.60	-\$330,964.21	\$19,035,908.88	\$ 1,681,289.10	\$9,953,978.20	\$ 1,087,750.18	-\$6,208,808.38	\$ 1,087,750.18	-\$6,208,808.38	\$ 1,087,750.18		
2054	\$ 38,669.13	\$ 726,000.00	\$ 764,669.13	\$ 1,974,254.60	\$1,209,585.48	\$20,245,494.36	\$ 1,681,289.10	\$10,870,598.18	\$ 1,087,750.18	\$5,885,727.32	\$ 1,087,750.18	\$5,885,727.32	\$ 1,087,750.18		
2055	\$ 756,329.63	\$ 139,000.00	\$ 895,329.63	\$ 1,974,254.60	\$1,078,924.98	\$21,324,419.34	\$ 1,681,289.10	\$11,656,557.65	\$ 1,087,750.18	-\$5,693,306.77	\$ 1,087,750.18	-\$5,693,306.77	\$ 1,087,750.18		
2056	\$ 2,544,293.50	\$ -	\$ 2,544,293.50	\$ 1,974,254.60	-\$570,038.90	\$20,754,380.44	\$ 1,681,289.10	\$10,793,553.24	\$ 1,087,750.18	-\$7,149,850.09	\$ 1,087,750.18	-\$7,149,850.09	\$ 1,087,750.18		
2057	\$ 215,763.25	\$ -	\$ 215,763.25	\$ 1,974,254.60	\$1,758,491.35	\$22,512,871.79	\$ 1,681,289.10	\$12,259,079.09	\$ 1,087,750.18	-\$6,277,863.16	\$ 1,087,750.18	-\$6,277,863.16	\$ 1,087,750.18		
2058	\$ -	\$ -	\$ -	\$ 1,974,254.60	\$1,974,254.60	\$24,487,126.40	\$ 1,681,289.10	\$13,940,368.19	\$ 1,087,750.18	-\$5,190,112.98	\$ 1,087,750.18	-\$5,190,112.98	\$ 1,087,750.18		
2059	\$ 31,821.00	\$ -	\$ 31,821.00	\$ 1,974,254.60	\$1,942,433.60	\$26,429,560.00	\$ 1,681,289.10	\$15,589,836.28	\$ 1,087,750.18	-\$4,134,183.80	\$ 1,087,750.18	-\$4,134,183.80	\$ 1,087,750.18		
2060	\$ 50,000.00	\$ -	\$ 50,000.00	\$ 1,974,254.60	\$1,924,254.60	\$28,353,814.60	\$ 1,681,289.10	\$17,221,125.38	\$ 1,087,750.18	-\$3,096,433.62	\$ 1,087,750.18	-\$3,096,433.62	\$ 1,087,750.18		
2061	\$ 197,212.00	\$ -	\$ 197,212.00	\$ 1,974,254.60	\$1,777,042.60	\$30,130,857.20	\$ 1,681,289.10	\$18,705,202.48	\$ 1,087,750.18	-\$2,205,895.44	\$ 1,087,750.18	-\$2,205,895.44	\$ 1,087,750.18		
2062	\$ 294,000.00	\$ -	\$ 294,000.00	\$ 1,974,254.60	\$1,680,254.60	\$31,811,111.81	\$ 1,681,289.10	\$20,092,491.57	\$ 1,087,750.18	-\$1,412,145.26	\$ 1,087,750.18	-\$1,412,145.26	\$ 1,087,750.18		
2063	\$ 831,500.00	\$ -	\$ 831,500.00	\$ 1,974,254.60	\$1,142,754.60	\$32,953,866.41	\$ 1,681,289.10	\$20,942,280.67	\$ 1,087,750.18	-\$1,155,895.98	\$ 1,087,750.18	-\$1,155,895.98	\$ 1,087,750.18		
2064	\$ 50,000.00	\$ 3,081,000.00	\$ 3,131,000.00	\$ 1,974,254.60	-\$1,156,745.40	\$31,797,121.01	\$ 1,681,289.10	\$19,492,569.77	\$ 1,087,750.18	-\$3,199,144.90	\$ 1,087,750.18	-\$3,199,144.90	\$ 1,087,750.18		
2065	\$ 809,597.24	\$ 139,000.00	\$ 948,597.24	\$ 1,974,254.60	\$1,025,657.37	\$32,822,778.38	\$ 1,681,289.10	\$20,225,261.63	\$ 1,087,750.18	-\$1,059,991.96	\$ 1,087,750.18	-\$1,059,991.96	\$ 1,087,750.18		
2066	\$ 200,000.00	\$ -	\$ 200,000.00	\$ 1,974,254.60	\$1,774,254.60	\$34,597,032.98	\$ 1,681,289.10	\$21,706,550.72	\$ 1,087,750.18	-\$2,172,241.78	\$ 1,087,750.18	-\$2,172,241.78	\$ 1,087,750.18		
2067	\$ 509,050.05	\$ -	\$ 509,050.05	\$ 1,974,254.60	\$1,465,204.55	\$36,062,237.53	\$ 1,681,289.10	\$22,878,789.77	\$ 1,087,750.18	-\$1,593,541.65	\$ 1,087,750.18	-\$1,593,541.65	\$ 1,087,750.18		
2068	\$ 2,826,375.54	\$ 363,000.00	\$ 3,189,375.54	\$ 1,974,254.60	-\$1,215,120.93	\$34,847,116.60	\$ 1,681,289.10	\$21,370,703.33	\$ 1,087,750.18	-\$3,695,167.00	\$ 1,087,750.18	-\$3,695,167.00	\$ 1,087,750.18		
2069	\$ 1,034,059.75	\$ 363,000.00	\$ 1,397,059.75	\$ 1,974,254.60	\$577,194.85	\$35,424,311.45	\$ 1,681,289.10	\$21,654,932.68	\$ 1,087,750.18	-\$4,004,476.57	\$ 1,087,750.18	-\$4,004,476.57	\$ 1,087,750.18		
2070	\$ 135,273.00	\$ -	\$ 135,273.00	\$ 1,974,254.60	\$1,838,981.60	\$37,263,293.05	\$ 1,681,289.10	\$23,200,948.77	\$ 1,087,750.18	-\$1,051,999.39	\$ 1,087,750.18	-\$1,051,999.39	\$ 1,087,750.18		
2071	\$ 3,387,579.44	\$ -	\$ 3,387,579.44	\$ 1,974,254.60											

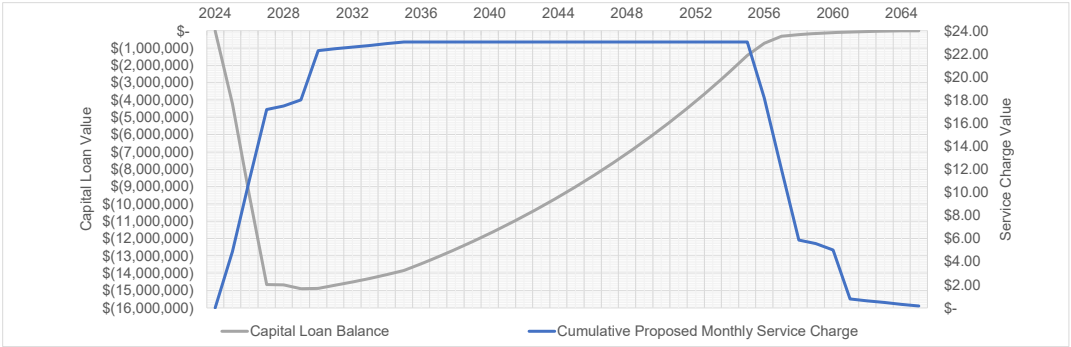


APPENDIX K

Loan Financial Model

	Cottonwood Upsizing (UL80)	Maple Drive Upsizing (UL80)	Starlite Lift Station Relocation (UL80)	Forcemain Extension Near WWTP (UL80)	WWTP Odour Control (UL20)	Cell 1 MBBR Conversion (UL40)	Air Piping Upgrades @ WWTP (UL80)	CCTV Program (UL10)	Forcemain Odour Reduction (UL20)	Cl2 Onsite Generation (UL25)	CT Tank Recirc (UL25)	Golf Course LS Screening (UL80)	WWTP Aux Power (UL 25)	Magnolia Place Reconstruction (UL80)	WWTP Prelim Design					
2023																	\$ -	\$ -	\$ -	\$ -
2024					\$ 2,900,000.00								\$ 218,000.00	\$ 348,000.00	\$ 1,015,000.00		\$ 4,481,000.00	\$ 262,639.76	\$ (4,218,360.24)	\$ 4.86
2025			\$ 3,625,000.00				\$ 1,450,000.00	\$ 139,000.00				\$ 500,000.00					\$ 5,714,000.00	\$ 597,547.94	\$ (9,529,700.55)	\$ 11.07
2026	\$ 750,000.00	\$ 550,000.00				\$ 2,000,000.00		\$ 139,000.00		\$ 2,175,000.00							\$ 5,614,000.00	\$ 926,594.93	\$ (14,657,377.78)	\$ 17.16
2027				\$ 150,000.00				\$ 139,000.00									\$ 289,000.00	\$ 943,533.76	\$ (14,680,014.87)	\$ 17.47
2028								\$ 139,000.00	\$ 363,000.00								\$ 502,000.00	\$ 972,956.92	\$ (14,887,274.64)	\$ 18.02
2029								\$ 139,000.00			\$ 363,000.00						\$ 502,000.00	\$ 1,202,895.75	\$ (14,874,170.98)	\$ 22.28
2030								\$ 139,000.00									\$ 139,000.00	\$ 998,042.79	\$ (14,702,314.89)	\$ 22.43
2031								\$ 139,000.00									\$ 139,000.00	\$ 1,006,189.84	\$ (14,514,371.99)	\$ 22.58
2032								\$ 139,000.00									\$ 139,000.00	\$ 1,014,336.89	\$ (14,309,599.09)	\$ 22.73
2033								\$ 139,000.00									\$ 139,000.00	\$ 1,022,483.94	\$ (14,087,218.62)	\$ 22.88
2034								\$ 139,000.00									\$ 139,000.00	\$ 1,030,630.99	\$ (13,846,417.13)	\$ 23.03
2035																	\$ -	\$ 1,030,630.99	\$ (13,455,490.62)	\$ 23.03
2036																	\$ -	\$ 1,030,630.99	\$ (13,046,503.29)	\$ 23.03
2037																	\$ -	\$ 1,030,630.99	\$ (12,618,620.75)	\$ 23.03
2038																	\$ -	\$ 1,030,630.99	\$ (12,170,970.04)	\$ 23.03
2039																	\$ -	\$ 1,030,630.99	\$ (11,702,637.87)	\$ 23.03
2040																	\$ -	\$ 1,030,630.99	\$ (11,212,668.75)	\$ 23.03
2041																	\$ -	\$ 1,030,630.99	\$ (10,700,063.06)	\$ 23.03
2042																	\$ -	\$ 1,030,630.99	\$ (10,163,774.98)	\$ 23.03
2043																	\$ -	\$ 1,030,630.99	\$ (9,602,710.39)	\$ 23.03
2044																	\$ -	\$ 1,030,630.99	\$ (9,015,724.62)	\$ 23.03
2045																	\$ -	\$ 1,030,630.99	\$ (8,401,620.11)	\$ 23.03
2046																	\$ -	\$ 1,030,630.99	\$ (7,759,143.97)	\$ 23.03
2047																	\$ -	\$ 1,030,630.99	\$ (7,086,985.43)	\$ 23.03
2048																	\$ -	\$ 1,030,630.99	\$ (6,383,773.17)	\$ 23.03
2049																	\$ -	\$ 1,030,630.99	\$ (5,648,072.50)	\$ 23.03
2050																	\$ -	\$ 1,030,630.99	\$ (4,878,382.46)	\$ 23.03
2051																	\$ -	\$ 1,030,630.99	\$ (4,073,132.74)	\$ 23.03
2052																	\$ -	\$ 1,030,630.99	\$ (3,230,680.48)	\$ 23.03
2053																	\$ -	\$ 1,030,630.99	\$ (2,349,306.93)	\$ 23.03
2054																	\$ -	\$ 1,030,630.99	\$ (1,427,213.92)	\$ 23.03
2055																	\$ -	\$ 767,991.23	\$ (725,159.96)	\$ 18.17
2056																	\$ -	\$ 433,083.05	\$ (325,579.30)	\$ 11.96
2057																	\$ -	\$ 104,036.06	\$ (236,585.01)	\$ 5.87
2058																	\$ -	\$ 87,097.23	\$ (160,418.01)	\$ 5.56
2059																	\$ -	\$ 57,674.07	\$ (110,155.25)	\$ 5.01
2060																	\$ -	\$ 40,735.24	\$ (74,509.18)	\$ 0.75
2061																	\$ -	\$ 32,588.20	\$ (45,363.31)	\$ 0.60
2062																	\$ -	\$ 24,441.15	\$ (23,017.95)	\$ 0.45
2063																	\$ -	\$ 16,294.10	\$ (7,787.28)	\$ 0.30
2064																	\$ -	\$ 8,147.05	\$ (0.00)	\$ 0.15
2065																	\$ -	\$ -	\$ (0.00)	\$ -
2066																	\$ -	\$ -	\$ (0.00)	\$ -
2067																	\$ -	\$ -	\$ (0.00)	\$ -
2068																	\$ -	\$ -	\$ (0.00)	\$ -
2069																	\$ -	\$ -	\$ (0.00)	\$ -
2070																	\$ -	\$ -	\$ (0.00)	\$ -
2071																	\$ -	\$ -	\$ (0.00)	\$ -
2072																	\$ -	\$ -	\$ (0.00)	\$ -

<



Interest on positive balance	2.0%
Interest on negative balance	4.6%

Reserve Balance (Current)	\$ -
Approximate Number of Users	4500

Start	End	MSC	
2023	2053	\$ -	
2024	2054	\$ 4.86	4.863699182
2025	2055	\$ 6.20	6.202003375
2026	2056	\$ 6.09	6.093462889
2027	2057	\$ 0.31	0.313682005
2028	2058	\$ 0.54	0.54487324
2029	2059	\$ 4.26	4.258126449
2030	2060	\$ 0.15	0.150871276
2031	2061	\$ 0.15	0.150871276
2032	2062	\$ 0.15	0.150871276
2033	2063	\$ 0.15	0.150871276
2034	2064	\$ 0.15	0.150871276

End of Analysis